ACCIDENTIAL TORSIONAL IRREGULARITY IN STEEL CONCENTRICALLY BRACED FRAMES

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ABSTRACT

This study will investigate the accidental torsional response in steel structures designed by CBFs due to the variation in strength of braces under strong earthquake ground motions. For this study, it is assumed that the variation in strength of braces would come from the expected yield stress (R_yF_y) rather than minimum specified yield stress (F_y) of brace member. For this purpose, inelastic torsional response of a three and nine story building having perimeter CBFs having various brace strength configurations subjected to strong earthquake ground motions is investigated in detail. The results are presented in the form of axial stress-strain in brace members and drift ratio through nonlinear dynamic response analyses.

KEYWORDS: Inelastic Torsional Response, Seismic Design, Concentrically Braced Frame

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. 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. 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.

The nonlinear dynamic responce of the CBFs have investigated by using experimental and theoretical methods. 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.This study will investigate the accidental torsional response in steel structures designed by CBFs due to the variation in strength of braces under strong earthquake ground motions. For this study, it is assumed that the variation in strength of braces would come from the expected yield stress (R_yF_y) rather than minimum specified yield stress (F_y) of brace member. For this purpose, inelastic torsional response of a three and nine story building having perimeter CBFs having various brace strength configurations subjected to strong earthquake ground motions is investigated in detail. The results are presented in the form base shear vs. roof displacement (pushover curve, capacity curve) and drift ratio through nonlinear dynamic response analyses.

2. EVALUATION OF TORSIONAL IRREGULARITY IN ANALYTICAL STUDY

[redacted] *Description of the Buildings*

Two typical inverted- V CBFs with 3- and 9- story , representing typical low and medium -rise steel buildings were designed based on the seismic design requirements for CBFs in accordance with ASCE 7-10 , AISC 360-10 , AISC 341-10. The plan dimensions of buildings are (width) 45.75 m and (depth) 45.75m with constant span length of 9.15 m (five equal spans) for all stories.

The typical story height for the 3- and 9- story frame is 3.96 m and the height of ground for the 9- story frame is 5.49 m in Figure 1 and 2. For the 9- story building, concrete foundation walls and surrounding soil are assumed to prevent any significant horizontal displacement of the structure at the ground level, i.e. the seismic base is assumed to be at the ground level. Dead loads including selfweight of the members and live load used in the study are 3.84 kN/m^2 and 2.4 kN/m^2 respectively . European wide flange profiles are preferred with quality of S355 for columns and beams. The yield stress of braces in the building are preferred with quality of S355.The structural system for each building consists of steel perimeter moment resisting frames and interior simply-connected framing for gravity, i.e. lateral loads are carried by perimeter braced frames and interior frames are not explicitly designed to resist seismic loads in the direction of the earthquake.

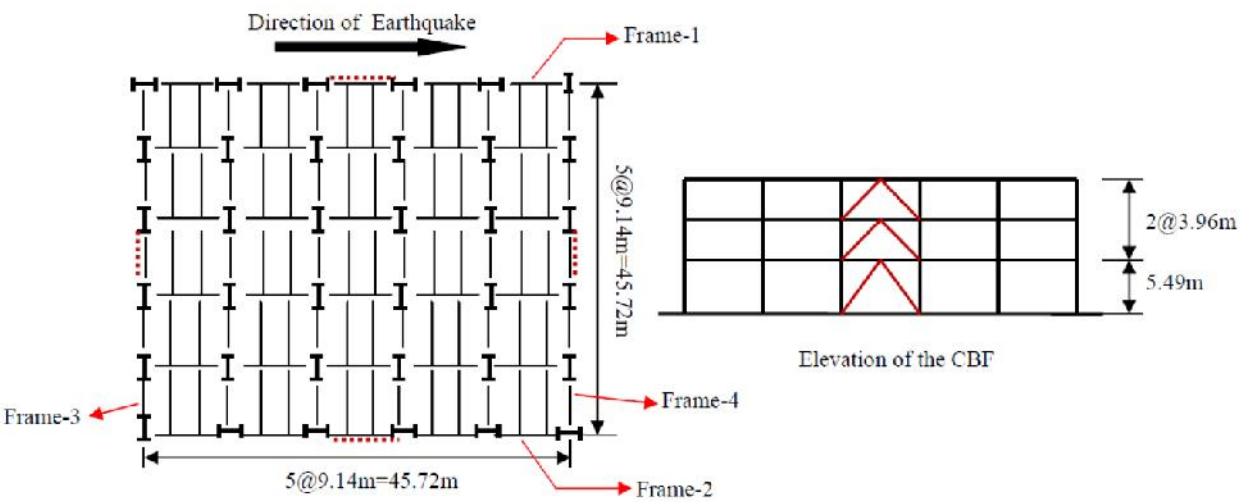


Figure 1. Plan and Elevation of the 3-story frame

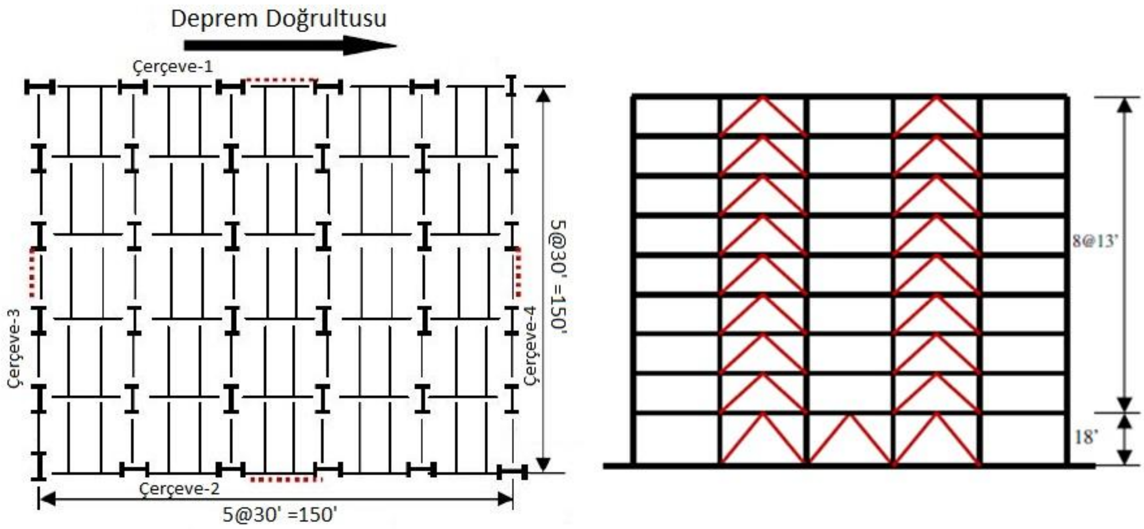


Figure 2. Plan and Elevation of the 9-story frame

Column , beams and braces are modeled by PERFORM 3D which is nonlinear time history analysis software. Beams and columns were modeled as beam-column elements, whereas inelastic steel bar element is used to model the axial behavior of braces in Figure 3.Nominal compressive strength, *Pcr*, and nominal tensile strength, *Py*, were computed based on AISC 360 (AISC 360-10, 2010) in Figure 3. Residual compressive strength, *Presidual* and the axial deformation at which it is reached were assumed to be 0.3*Pcr* based on AISC 341 (AISC 341-10, 2010) . δy is defined as the yield displacement corresponding to *Pcr*. A small yield plateau was assumed having constant length equal to δy 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, *RyFy*, was used in defining the inelastic behavior of the braces, where *Ry* is 1.4 for HSS (AISC 341-10, 2010). 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 1.254 sec for 9- story and 0.638 sec for 3- story sec. Damping ratio was taken as 5% and Rayleigh damping with the first and second natural frequencies were used in the analyses.

Table 1. Member sizes of the 3-story CBF.

Story	*Brace members*	*Unbraced bay girders*	*Braced bay girders*	*Braced bay columns*	*Exterior columns*
Roof	HSS 7 x 7 x 1/2	W 21 x 44	W 24 x 306	W 12 x 40	W 12 x 40
2nd floor	HSS 7 x 7 x 1/2	W 21 x 44	W 24 x 335	W 12 x 40	W 12 x 40
1st floor	HSS 10 x 10 x 5/8	W 21 x 44	W 36 x 395	W 12 x 65	W 12 x 40

Table 2. Member sizes of the 9-story CBF.

Story	*Brace members*	*Unbraced bay girders*	*Braced bay girders*	*Braced bay columns*	*Exterior columns*
Roof	HSS 7 x 7 x 1/2	W 21 x 44	W 33 x 221	W 14 x 48	W 14 x 48
8th floor	HSS 7 x 7 x 1/2	W 21 x 44	W 33 x 221	W 14 x 68	W 14 x 48
7th floor	HSS 8 x 8 x 5/8	W 21 x 44	W 33 x 291	W 14 x 68	W 14 x 48
6th floor	HSS 8 x 8 x 5/8	W 21 x 44	W 33 x 291	W 14x 159	W 14 x 48
5th floor	HSS 9 x 9 x 5/8	W 21 x 44	W 30 x 357	W 14x 159	W 14 x 48
4th floor	HSS 9 x 9 x 5/8	W 21 x 44	W 30 x 357	W 14 x 233	W 14 x 48
3rd floor	HSS 10 x 10 x 5/8	W 21 x 44	W 33 x 354	W 14 x 233	W 14 x 48
2nd floor	HSS 10 x 10 x 5/8	W 21 x 44	W 33 x 354	W 14 x 455	W 14 x 61
1st floor	HSS 10 x 10 x 5/8	W 21 x 44	W 36 x 395	W 14 x 455	W 14 x 61

The floor system of the buildings is assumed to provide diaphragm action and to be rigid in the horizontal plane.In design of steel inverted V-braced frames, the appropriate response modification coefficient (R= 6) , overstrength factor (Ω_o=2), and the deflection amplification factor (C_d=5), are used in determining the base shear, element design forces, and design story drift. The buildings were designed for a site where S_S is 2.0g and S_1 is 1.0g. Seismic parameters for design spectrum are S_S =1.5 (g), S_1= 0.8 (g). Two sets of ground motion records were selected from PEER Strong Ground Motion Database corresponding to 10% and 2% probability of exceedance. Each set consists of 3 records.

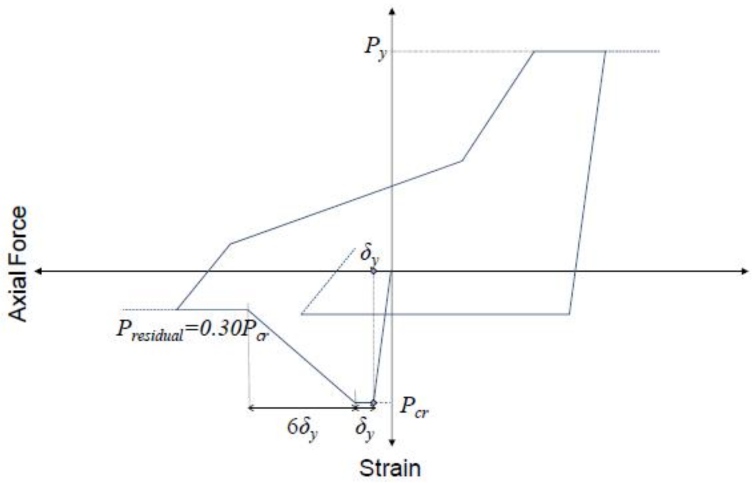


Figure 3. Hysteresis loop for axial behavior of braces

The frames were subjected to two sets of ground motions (a total of 6 ground motions) .Table 3 lists the detailed information of these ground motions. Two sets of ground motions corresponding to 10% and 2% probability of exceedance are used in nonlinear dynamic time history analyses . As specified in Figure 4 and Figure 5, ground motions corresponding to 10% probability of exceedance are presented as GM 1, GM 2 and GM 3 that cause moderate structural damage and ground motions corresponding to 2% probability of exceedance are presented as GM 4, GM 5 and GM 6 that cause heavy structural damage. Figure 8 and 9 summarizes the elastic response spectra of the ground motions. Shear wave velocity (V_s) of site class is between value of 300 m/s and 770 m/s.

Table 2. Earthquake Ground Motion Characteristics from PEER Database

Name	NGA#	Record	Scale Factor	Duration (sec)	PGA (cm/sec^2)
GM 1 (%10)	1612	Düzce	3.5588	41.0	531.89
GM 2 (%10)	4284	Basso,Tirreno	3.9035	29.00	572.00
GM 3 (%10)	451	Morgan Hill	0.8357	41.00	317.059
GM 4 (%2)	1111	Kobe	1.3103	41.00	620.973
GM 5 (%2)	4099	Park Field	2.2397	21.00	1047.708
GM 6 (%2)	4481	L'Aquila	1.7551	61.00	828.945

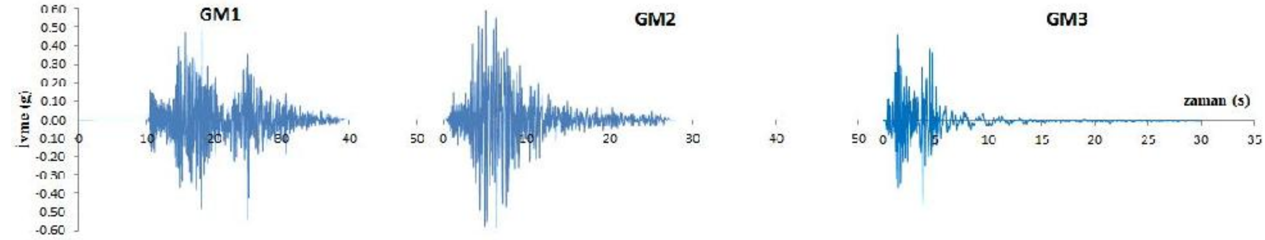


Figure 4. Time histories of the ground motions corresponding to 10% probability of exceedance

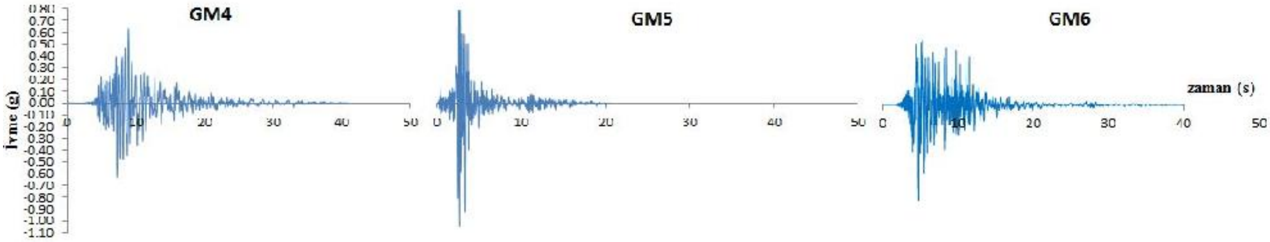
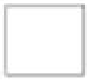
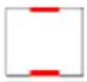
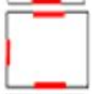
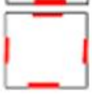
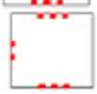
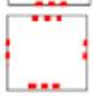


Figure 5.Time histories of the ground motions corresponding to 2% probability of exceedance

For the nonlinear dynamic response analyses, the strength of the braces was assumed to be varying for each perimeter CBF between *RyFy* and *1.2RyFy*. Eight cases were generated in order to study the inelastic torsional response of the perimeter CBFs (Table 2.2). For example, Case 1 refers to the case where the strength of the braces in frame 1 at all stories is equal to *1.2RyFy*, i.e., the strength of the braces in other frames is equal to *RyFy*. For Cases 5, 6, 7 and 8, the strength of the braces is assumed to be equal to *1.2RyFy* at only the first story braces of a given frame.

Table 3. Strength variation in the perimeter CBFs

Case No	***Çaprazın Dayanımı***	***Uygulanan Çerçeve***		***Uygulanan Kat***
Reference Case	R_yF_y	Frame – 1,2,3,4		Kat- 1,2,3
Case- 1	$1.2R_yF_y$	Çerçeve – 1		Kat- 1,2,3
Case - 2	$1.2R_yF_y$	Çerçeve - 1,2		Kat- 1,2,3
Case - 3	$1.2R_yF_y$	Çerçeve - 1,2,3		Kat- 1,2,3
Case - 4	$1.2R_yF_y$	Çerçeve – 1,2,3,4		Kat- 1,2,3
Case - 5	$1.2R_yF_y$	Çerçeve - 1		Kat- 1
Case - 6	$1.2R_yF_y$	Çerçeve - 1,2		Kat- 1
Case - 7	$1.2R_yF_y$	Çerçeve - 1,2,3		Kat- 1
Case - 8	$1.2R_yF_y$	Çerçeve – 1,2,3,4		Kat- 1

[redacted]*Analyses Results*

The results from nonlinear dynamic response analyses are presented in the form of base shear vs. roof displacement (capacity curve) and drift ratio in the braces. The results for the Cases 2, 3, and 4 and 6, 7, and 8did 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 *Fy*, *RyFy* or *1.2RyFy*, the strength variation in the braces perpendicular to the direction of earthquake does not affect the response. Torsional irregularity due to the variation in strength only exists in Cases 1 and 5.

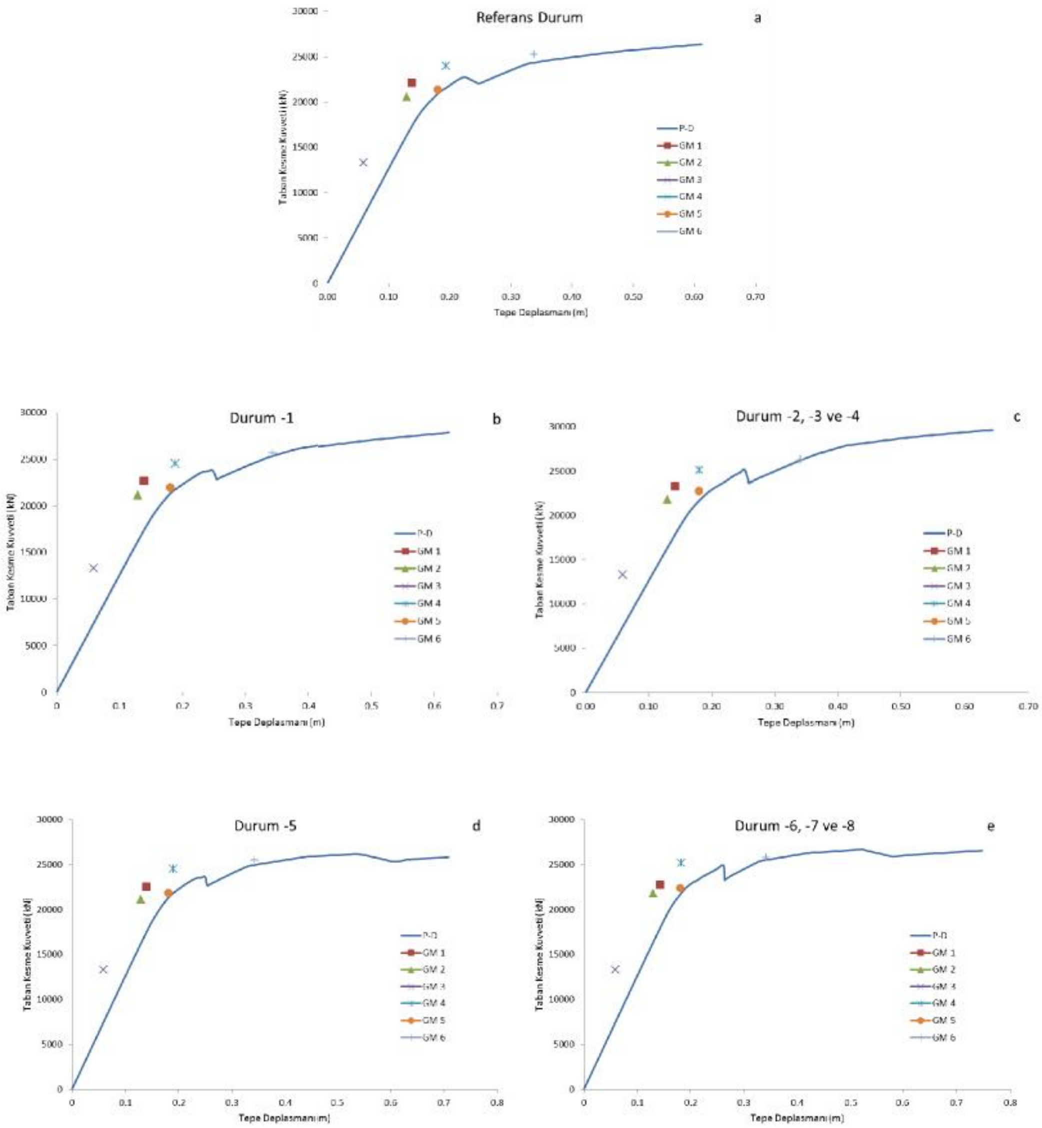


Figure 6. Base shear vs. roof displacements (pushover , capacity curves) for 9- story

Pushover curves in the form of base shear vs. roof displacement for all cases and the reference building are shown in Figure 6 for inverted V- braced 9- story building. Lateral strength of the reference building was found to be 20,000kN and lateral strength of the building slightly increased for Cases 1, 5, 6, 7 and 8. It should be noted that in Case 1, only the strength of the braces in Frame1 at all stories was taken as *1.2RyFy* in the direction of earthquake, whereas the braces in Frame 2 was assumed to have expected yield stress, *RyFy*. For Cases 5, 6, 7 and 8, where only the braces at the first story of the CBF is equal to *1.2RyFy*, lateral strength of the building was not affected. However, for Cases 2, 3 and 4, lateral strength of the building increased about 7.8% (8,050kN) compared to that of the reference building. The maximum lateral displacement of 35 cm among all cases.

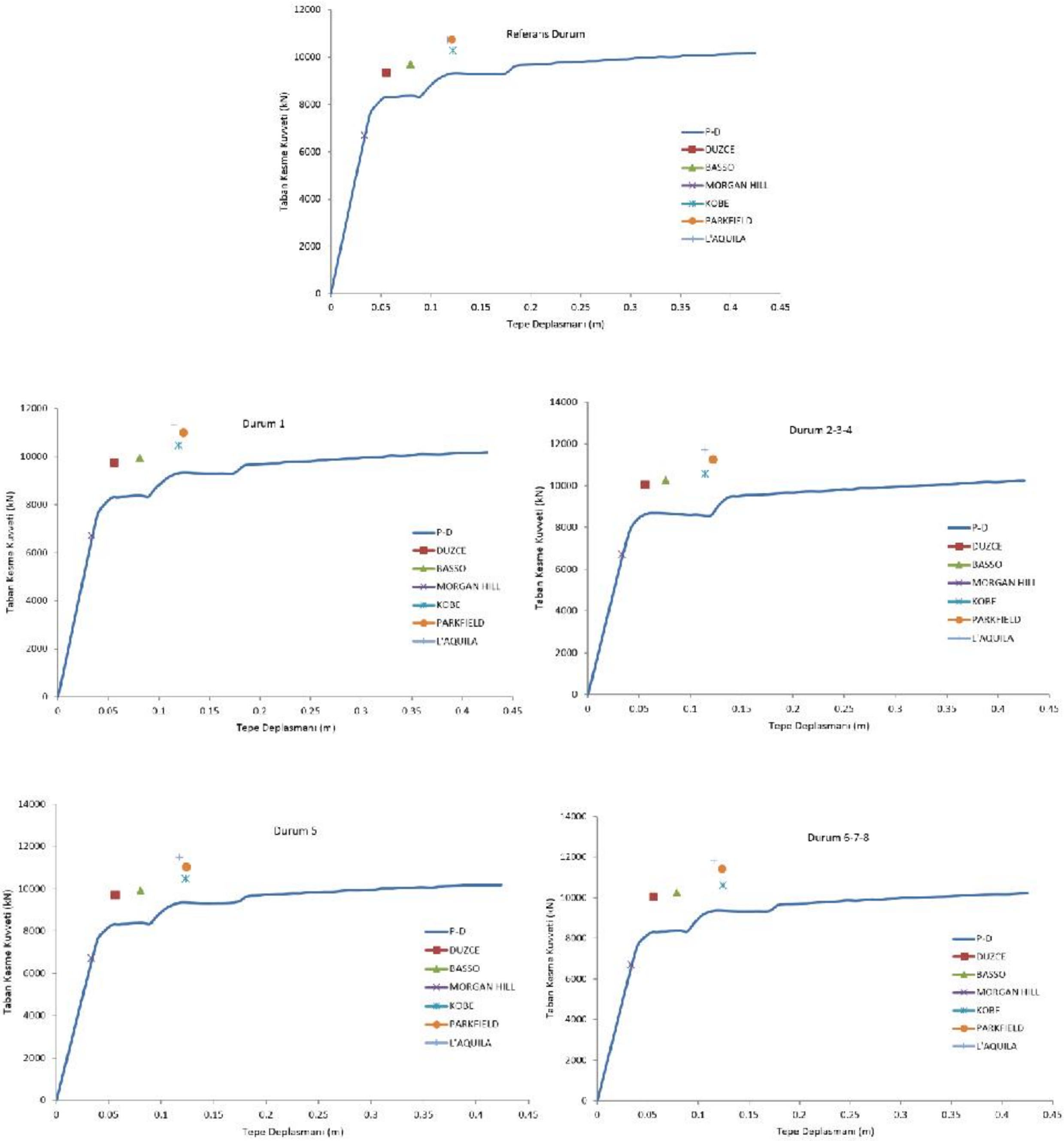


Figure 6. Base shear vs. roof displacements (pushover , capacity curves) for 3- story

Pushover curves in the form of base shear vs. roof displacement for all cases and the reference building are shown in Figure 6 for inverted V- braced 3- story building. Lateral strength of the reference building was found to be 7,000kN and lateral strength of the building slightly increased for Cases 1, 5, 6, 7 and 8. It should be noted that in Case 1, only the strength of the braces in Frame1 at all stories was taken as *1.2RyFy* in the direction of earthquake, whereas the braces in Frame 2 was assumed to have expected yield stress, *RyFy*. For Cases 5, 6, 7 and 8, where only the braces at the first story of the CBF is equal to *1.2RyFy*, lateral strength of the building was not affected. However, for Cases 2, 3 and 4, lateral strength of the building increased about 11% (770 kN) compared to that of the reference building. The maximum lateral displacement of 12 cm among all cases.

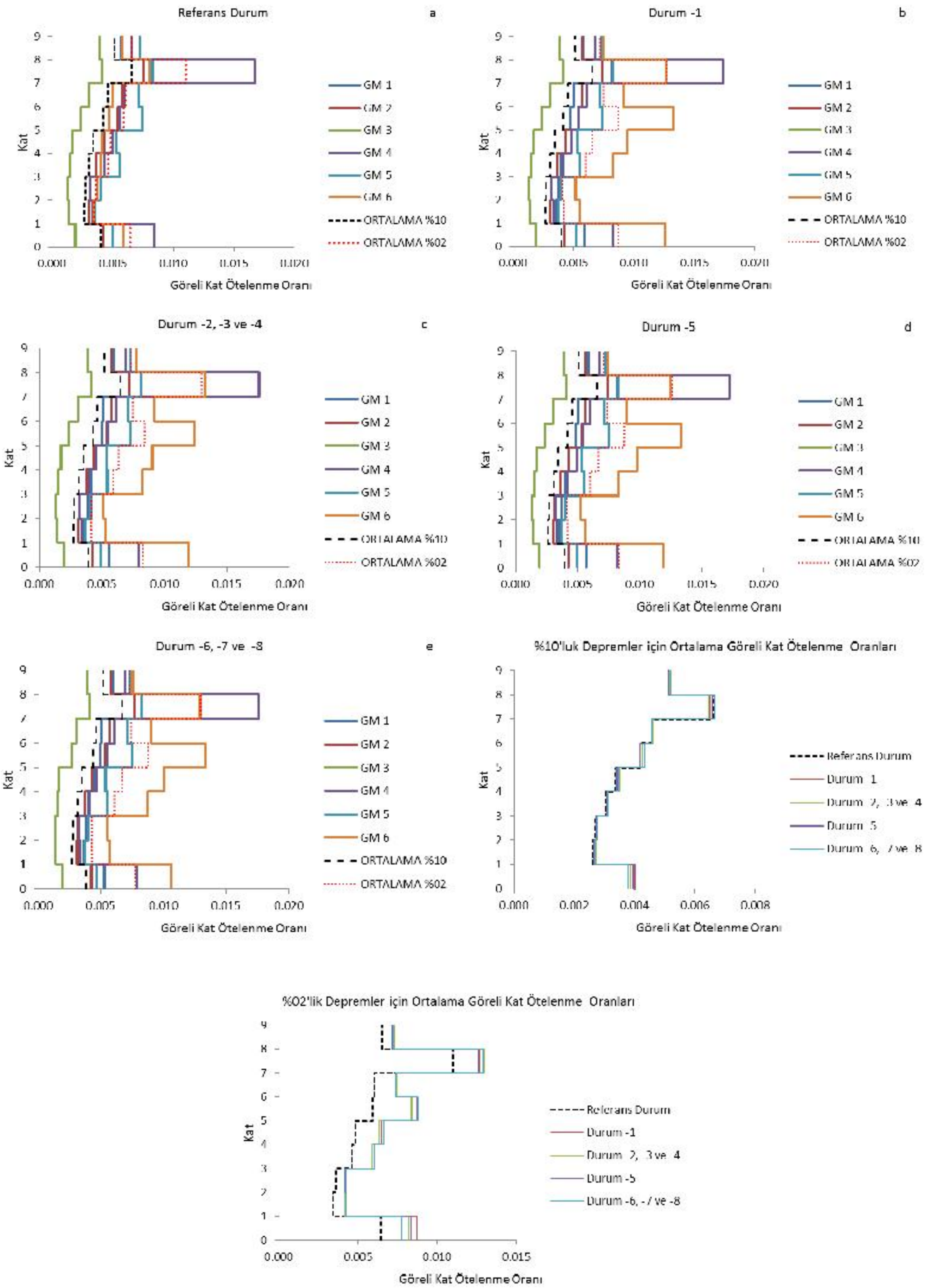


Figure 7. Story drift ratios for 9- story

Drift ratios for all cases and the reference building are shown in Figure 7 for inverted V- braced 3- story building. The maximum average drift ratio for thereference building was found to be 0.4% for ground motions corresponding to 10% probability of exceedance and 0.6% for ground motions corresponding to

10% probability of exceedance in Figure 7. For ground motions corresponding to 10% probability of exceedance, the maximum average drift ratio of first floor was found to be 0.4% for all cases and the reference building. For ground motions corresponding to 2% probability of exceedance, the maximum average drift ratio of first floor was found to be 0.6% for all cases and the reference building. GM 6 caused a maximum drift ratio 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 medium-rise CBFs.

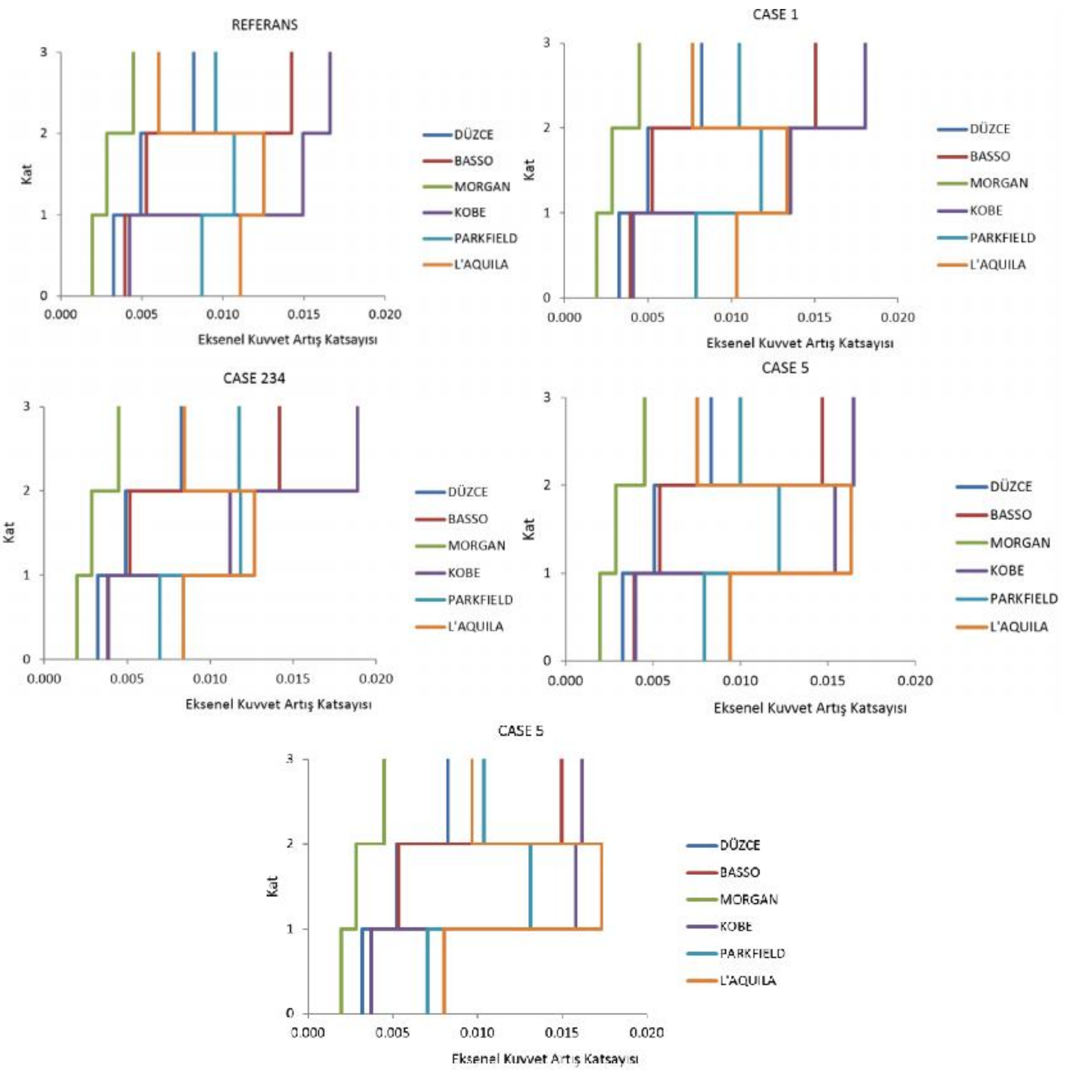


Figure 8. Story drift ratios for 3- story

Drift ratios for all cases and the reference building are shown in Figure 8 for inverted V- braced 3- story building. The maximum average drift ratio for the reference building was found to be 0.4% for ground motions corresponding to 10% probability of exceedance and 0.9% for ground motions corresponding to 10% probability of exceedance in Figure 8. For ground motions corresponding to 10% probability of exceedance, the maximum average drift ratio of first floor was found to be 0.4% for all cases and the reference building. For ground motions corresponding to 2% probability of exceedance, the maximum average drift ratio of first floor was found to be 0.75% for all cases and the reference building. GM 6 caused a maximum drift ratio 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 medium-rise CBFs.

3. CONCLUSION

This study will investigate the accidental torsional response in steel structures designed by CBFs due to the variation in strength of braces under strong earthquake ground motions corresponding to 10% and 2% probability of exceedance. For the nonlinear dynamic response analyses, the strength of the braces was assumed to be varying for each perimeter CBF between *RyFy* and *1.2RyFy*. For this purpose, inelastic torsional response of a three and nine story building having perimeter CBFs having various brace strength configurations subjected to strong earthquake ground motions is investigated .Based on the results obtained in this study, the following observations can be made.

- Lateral strength of the CBF is not affected significantly when there is there is a torsional

- 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.

- Lateral displacement of the CBFs does not change 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.
- The average maximum drift ratio for ground motions corresponding to 10% probability of exceedance is nearly same in 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.
- The average maximum drift ratio for ground motions corresponding to 2% probability of exceedance is nearly same in the torsional irregularity due to the higher strength of braces but according to reference model average drift ratios are increased.

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