Assessment of Various Modeling Parameters and Their Influence on Seismic Behavior of Braced Steel Frame Structures

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SUMMARY:

This paper examines the various parameters and their influence on seismic behavior of braced steel frame structures. For the performance-based analysis presented in this study, the selected buildings were modeled in OpenSees to include the effect of large displacement in the beam-column members, modelling of buckling of columns and bracings, inclusion of reduced net area section reinforcement, low-cycle fatigue, and panel zone. This study presents the results of a performance evaluation on a class of steel braced frame structures, namely, special steel concentrically braced frames (SCBFs) and buckling-restrained braced frames (BRBFs) with two-story X-brace configuration.

Keywords: Low-cycle fatigue; Buckling-restrained braced frames; Special steel concentrically braced frames

1. INTRODUCTION

In recent years the use of special steel structures in regions with high seismicity is leaning from the moment frames toward concentrically brace frames. With the increasing popularity of brace frames, and reasons such as poor performance of some special brace frames in past earthquakes, the limited experimental data in inelastic phase and also lack of proper understanding of brace profile failures in these systems, extensive theoretical and experimental investigations in this case has been started. Thus, in recent years a great number of researches conducted to understand and improve the seismic performance of concentrically braced steel structures. Extensive analytical studies have been done on systems with special and buckling-restrained bracing. Although many engineers tend to use the eccentrically braced frames, but great number of modern constructions move toward the use of concentrically brace frames.

Where braces are rigidly connected to the adjacent framing, plastic hinging and low-cycle fatigue are issues of concern at the two ends as well as the middle of braces. Therefore, for modelling the buckling and low-cycle fatigue rupture of braces, the strain history in each fibre was tracked, and a rainflow counting algorithm was used to determine the amplitude of each inelastic cycle (Uriz et al. 2008).

Laboratory and field researches show the importance of considering the effects of low-cycle fatigue to predict the overall behaviour of the braced frame structures. Uriz and Mahin (Uriz et al, 2008) proposed a low-cycle fatigue model based on the experimental and analytical studies.

Moment transfer between beams and columns create very complex stress-strain behavior in the connections. Columns in this connection suffer high normal stresses in the flanges, and create extreme shear stresses in the panel zone regions. (Gupta and Krawinkler, 1999). Hence, the effects of panel zone in the dynamic responses of the structures have been assessed.

Other tests identified the potential problems related to the decreased net area at the end of gusset plates

(Archambault, 1995). Despite, many new constructions emphasize on strengthening the net sections after gusset plates, long slot at the end of the gusset plates or lack of skill in construction lead to failure of wrap-around welded connections and joints at the decreased net area regions (Tremblay, 2002; Lee and Bruneau, 2005). Hence, the strengthening of these areas will be evaluated in this paper.
This study presents the results of the evaluation of both steel braced frame structures consisting of special steel concentrically braced frames (SCBFs) and buckling-restrained braced frames (BRBFs) with two-story X-brace configuration. Investigations suggest that inter-story drifts and fatigue durability which were designed in accordance with the current requirements of the SCBF structures may not be able to access the updated information on seismic hazards. Post-earthquake identifications and tests show that concentrically braced frames with relatively strong braces may susceptible to additional failure modes. Recent experiments in the United States and Japan proved that buckling restrained braces present a very ductile and stable behaviour as well as good resistance to fatigue failures. However, some studies indicate that BRBFs may be susceptible to premature failures. Therefore, efforts to reduce premature behavioural modes (that limit the capacity of structural system before the deadline) with the results of the various behavioural modes in the new concentrically braced frames are essential for further researches (Uriz et al. 2008). Based on this study, recommendations regarding the analysis, modelling and structural details of the concentrically braced structures are recommended. Based on this study, recommendations regarding the, analysis, modelling and structural details of the concentrically braced structures are recommended.

2. EVALUATION OF SEISMIC PERFORMANCE OF STEEL BRACED FRAME

This study has highlighted two main subjects: 1. Evaluation of modelling approaches in predicting the dynamic responses and also the expected damages of concentrically brace steel structures during earthquakes 2. Preliminary assessment of seismic performance of steel frame structures. In this paper, implementation of a powerful analytical beam-column model to simulate the effects of overall buckling of the member is expressed and the hysteretic behaviour of braces which carry considerable axial loads has been focused. This study intends to examine the dynamic behaviour and hysteretic response of both SCBFs and BRBFs. Nonlinear Time History Analysis (NTHA) is used to estimate the peak drift demands in each of the floor for each of the earthquake records.

3. STRUCTURAL MODELS

These models are similar to the ones which were developed in the NIST GCR 10-917-8 (NIST GCR 10-917-8, 2010). Both of SCBF and BRBF braced structures whether three-story or six-story, have the same height floors equal to 15 feet and width spans of 30 feet. A schematic picture of two-dimensional model created is shown in Figure 1. The models for special SCBF systems were designed in accordance with design requirements of AISC 341-05, Seismic Provisions for Structural Steel Buildings (AISC, 2005a), and ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2006). The two-story X-brace configuration of braced frames was used. These systems help to avoid large unbalanced beam loads which can occur when braces are buckled. Braces were assumed to have pinned end connections, while fully restrained connections were assumed for the beam-column connections. Round Hollow Structural Sections (HSS) were used in these special SCBF structures and buckling-restrained braces were defined based on the core area. W sections were selected for Beams and columns. Columns were assumed to be fixed at the base and orientated to resist lateral forces in strong-axis bending. More detailed information about the assumptions in the model structure and design can be seen in NIST GCR 10-917-8 (NIST GCR 10-917-8, 2010).

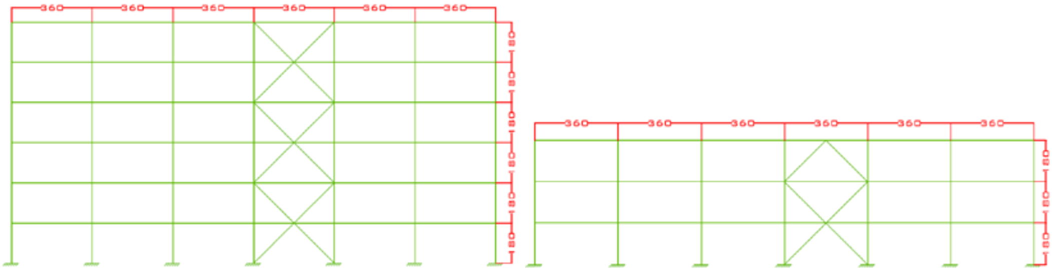


Figure 1: Schematic picture of two-dimensional model (dimensions are based on inch)

3.1. Special concentrically braced frames

Analytical model for this type of structures consist of 12 braced frames in which 6 models indicate 3-story structures and the other six models indicate the 6-story structures. More specifications of these models are as follow:

a. 3RF and 6RF: These analytical models are the simplest special concentrically braced frame structures, corresponding to three and six story models, respectively.
b. 3RZF and 6RZF: These analytical models are same as above models, but in these models the effect of low-cycle fatigue are considered.
c. 3PRZF and 6PRZF: these models are similar to 3RZF and 6RZF except for these models the effects of the panel zone in nonlinear dynamic analysis responses are considered.
d. 3BRBF and 6BRBF: these analytical models are related to BRBF systems which consider the effect of low-cycle fatigue and shear yield in panel zone.
e. 3BPRZF and 6BPRZF: these models are same as 3PRZF and 6PRZF with the difference that these models have a small initial camber in the mid-span of braces in order to capture the effect of global buckling.
f. 3NBPRZF and 6NBPRZF: these analytical models are identical to 3BPRZF and 6BPRZF, while these models do not consider any reinforcement in the net reduced cross sections.

4. MODELING APPROACH

Open System for Earthquake Engineering Simulation "Opensees" (OpenSees, 2007) was implemented to carry out the analysis. In order to account for geometric nonlinearity, a geometric transformation object that named "corotational transformation" in OpenSees was used. Among the variety of element types available in the OpenSees library, a force-based "Nonlinear Beam–Column Element" was selected for beams, columns, and buckling brace members. This element utilizes force interpolation functions for varying internal forces due to transverse displacements and explicitly satisfies equilibrium in the deformed shape. The spread of plasticity is considered along the length of the elements. The used material model was based on the Menegotto-Pinto steel model with an elastic modulus of 29,800 ksi, yield strength of 50 ksi, and a kinematic strain-hardening ratio of 0.3%. Initial camber for modelling the global buckling was chosen according to the model proposed by PEER Report 08/2008 (Uriz et al, 2008). Without this initial camber, bracing will behave as ideal hinge with no possibility of a global buckling, while a small initial camber without significant impact on member stiffness matrix can provide a beginning to the deviation and the overall buckling of the member. For simplicity in two-dimensional modelling, out-of-plane brace buckling was neglected. A simple OpenSees model for the net section has been used. To simplify this model, effects of concentration strain are neglected. Implementing the low-cycle fatigue as one of the opensees software capabilities has been performed. Boundary conditions were assumed to be fixed out of plane and rigid boundary conditions were taken at the base of the models.

5. GROUND MOTIONS

The ground motions used in this study are representative of ground motions for soil type SD which have been scaled based on the 1997 NEHRP design spectra of identical soil and hazard. More information about the ground motions which were used, are presented in Table 1.

Table 1: description of used earthquake records

EQ label	PEER-NGA Rec. Num.	Description	Earthquake Magnitude	Campbel Distance (km)	Joyner-Boore Distanc e (km)	Numbe r of Points	Time Step (sec)	PGA (g)
ATC01	953	Northridge, 1994	6.7	17.2	9.4	2999	0.01	0.42
ATC11	174	Imperial Valley, 1979	6.5	13.5	12.5	7807	0.005	0.36
ATC17	1158	Kocaeli, Turkey, 1999	7.5	15.4	13.6	5437	0.005	0.31
ATC21	900	Landers, 1999	7.3	23.8	23.6	2200	0.02	0.24
ATC23	848	Landers, 1999	7.3	20.0	19.7	11186	0.0025	0.28
ATC33	725	Superstition Hills, 1987	6.5	11.7	11.2	2230	0.01	0.45
ATC37	1244	Chi-Chi, Taiwan, 1999	7.6	15.5	10.0	18000	0.005	0.35

This ground motions are based on the Haselton and Deierlein (2007) that have been downloaded from (http://myweb.csuchico.edu/~chaselton/research/research_databases/gms_db.php).

6. COMPARISON CRITERIA AND ANALYSIS RESULTS

Figure 2 provides a summary of some results of these analyses. Inter-story drift ratios which are illustrated in these figures indicate peak values of the drift between adjacent two level floors over time for each floor that was extracted and normalized by height of the floor. Residual inter-story drifts of the floors were consists of the largest absolute difference in lateral displacement of the adjacent stories at the end of seismic excitations which were normalized by corresponding height of the floor. The results of these analyses for different models of three-story structure which contain the average and average plus one standard deviation “SD” are presented in Table. 2. The same results for six-story structures are exhibited in Table.3.

Table 2: Response summary for different models of 3-story structure

Frame		Inter-story Drift Ratio %		Residual Inter-story Drift Ratio %	
		Mean	SD	Mean	SD
3 STORY	3RF	0.517732	0.205388	0.000507	0.00048
	3RZF	0.517732	0.205388	0.000507	0.00048
	3PRZF	0.512255	0.20071	0.000452	0.000436
	3BRB	0.631742	0.243161	0.000971	0.000373
	3BRZF	0.617567	0.298125	0.000534	0.000362
	3NBPRZF	0.617546	0.298511	0.000546	0.000374

Table 3: Response summary for different models of 6-story structure

Frame		Inter-story Drift Ratio %		Residual Inter-story Drift Ratio %	
		Mean	SD	Mean	SD
6 STORY	6RF	0.582589	0.216373	0.000913	0.000708
	6RZF	0.582589	0.216373	0.000913	0.000708
	6PRZF	0.576644	0.207558	0.000798	0.000709
	6BRB	0.591675	0.140595	0.000723	0.000391
	6BRZF	0.489136	0.097286	0.002212	0.001933
	6NBPRZF	0.619986	0.328656	0.002058	0.001756

Figures 2 and 3 illustrate the results of the NTHA analysis for 6-, and 3-story structures, respectively. In these figures the peak floor drift obtained from NTHA analysis for each earthquake is plotted as a function of the intensity of earthquakes.

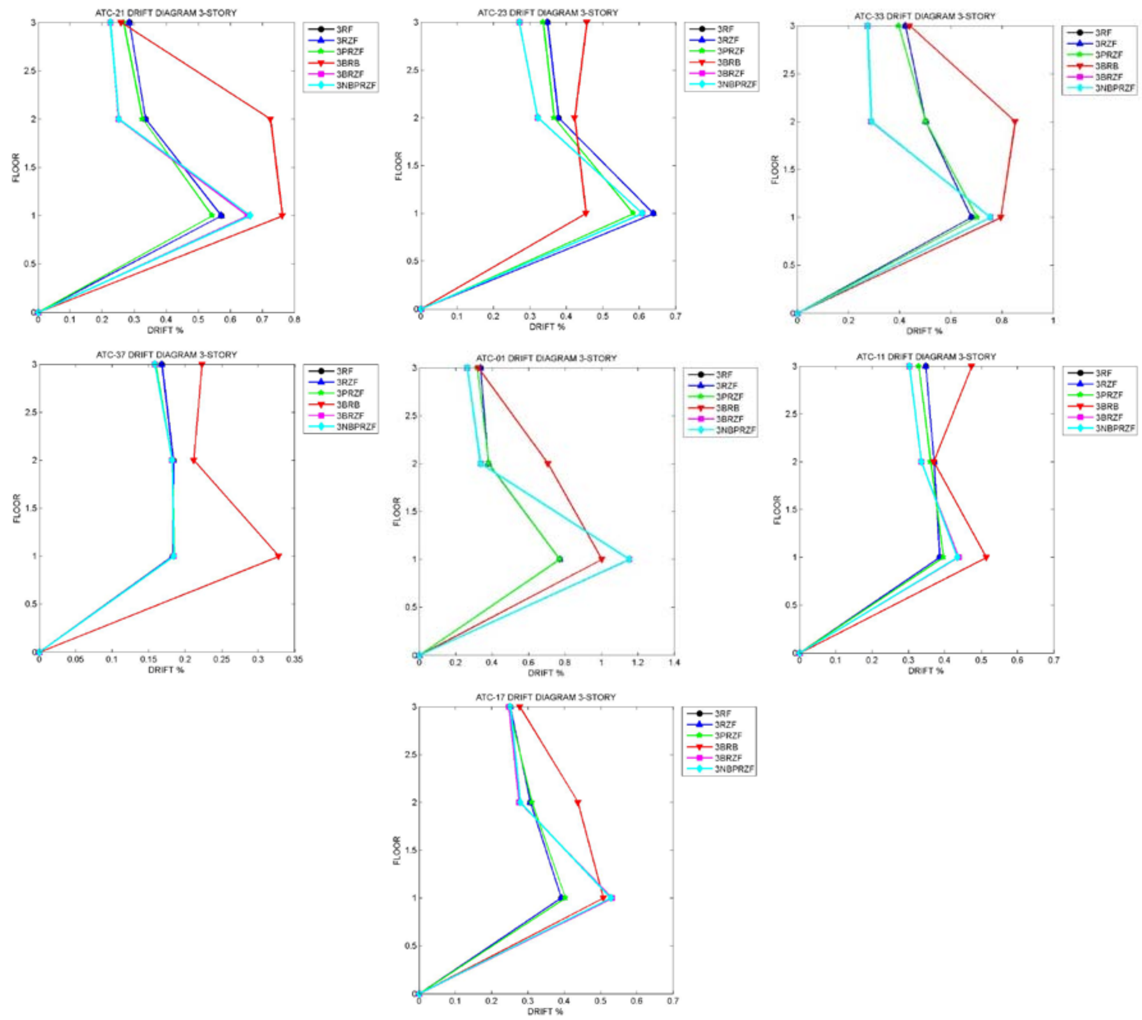
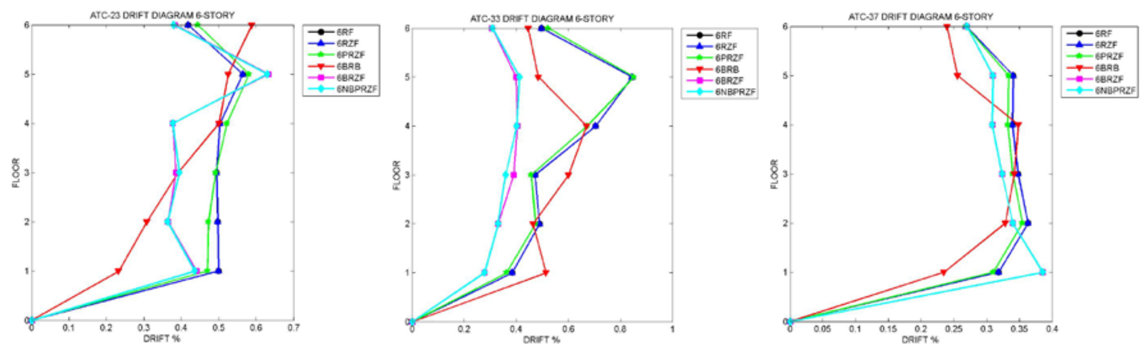


Figure 2: The peak inter-story drifts recorded from different models of the 3-story



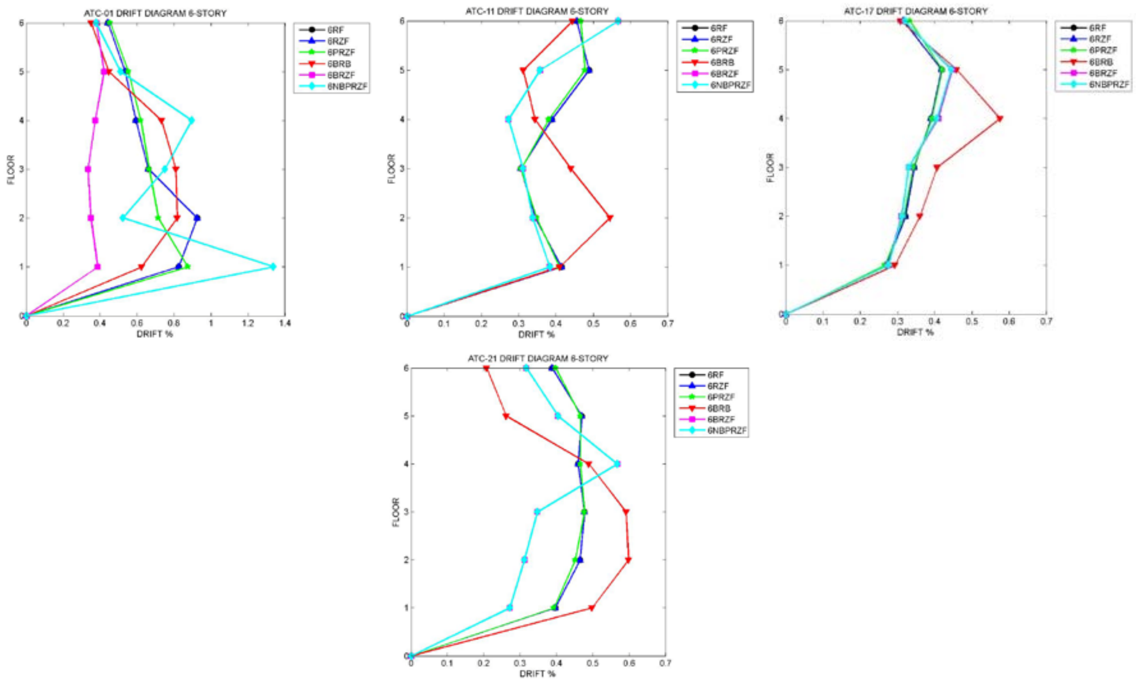


Figure 3: The peak inter-story drifts recorded from different models of the 6-story

Axial force–displacement hysteresis obtained from NTHA analysis for 3-story model and 6-story model are illustrated in Figure.4 and Figure.5, respectively.

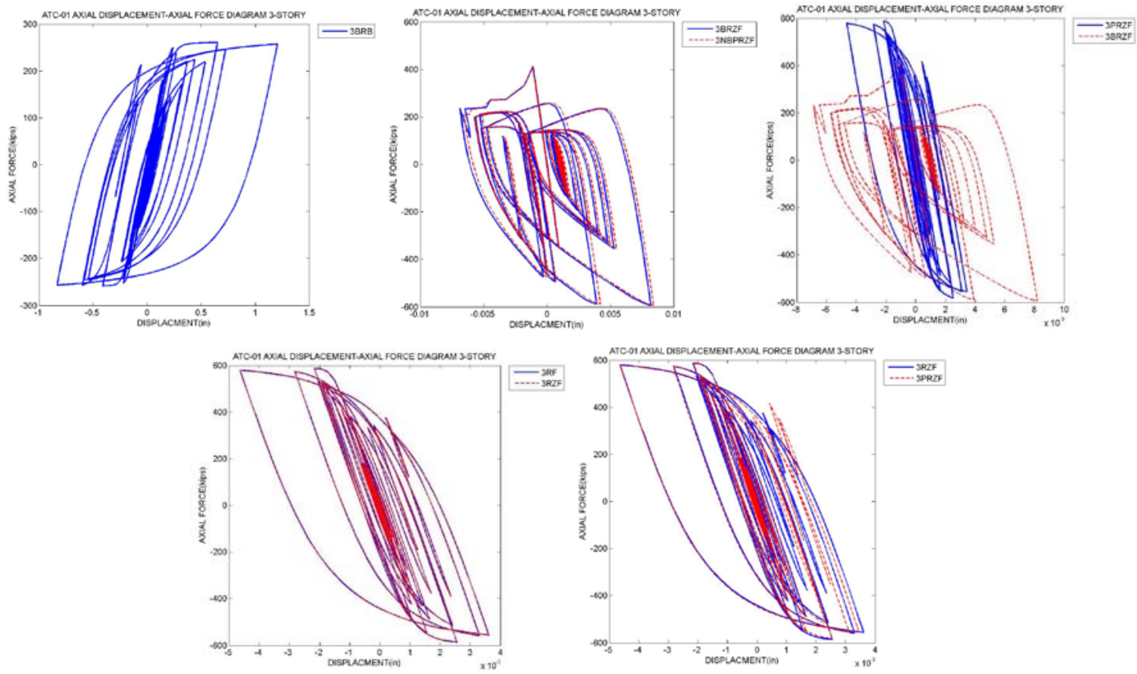


Figure 4: axial force–displacement hysteresis obtained from NTHA analysis for 3-story model

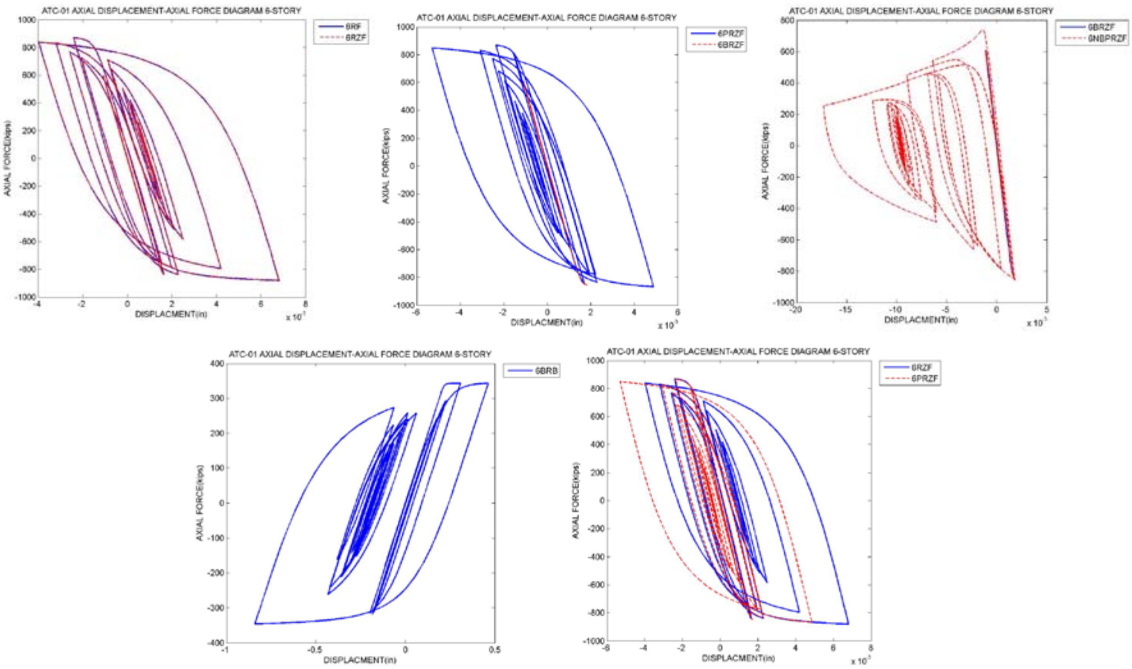


Figure 5: axial force–displacement hysteresis obtained from NTHA analysis for 6-story model

As it can be seen in the above figures, BRB model has had the best hysteresis behaviour and buckling has had the most impact on the axial force–displacement hysteresis behavior of SCBFs.

7. CONCLUSION

This study presents the results of a performance evaluation on a class of steel braced frame structures, namely, special steel concentrically braced frames (SCBFs) and buckling-restrained braced frames (BRBFs) with two-story X-brace configuration. The selected buildings were modelled to assess the effect of large displacement in the beam-column members, modelling of buckling of columns and bracings, inclusion of reduced net area section reinforcement, low-cycle fatigue, and panel zone. Results demonstrates that considering the buckling of the bracings greatly influence the behaviour of the structures. On the other hand, considering the effects of low cycle fatigue and panel zone do not significantly change the responses. Finally, buckling-restrained braced frames behave much more stable than special steel concentrically braced frames.

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