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Seismic Response of Buckling Restrained Braced Frames under Near Fault Ground Motions

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Abstract: Buckling Restrained Braces (BRBs) yield in tension and compression, exhibit stable and predictable hysteretic behavior, provide significant energy dissipation capacity and ductility and are an attractive alternative to conventional steel braces. Present study conducts numerical simulations of buckling restrained braced frames responses using a suite of near-fault ground motion records, to evaluate the seismic responses of Buckling Restrained Braced Frames (BRBFs) in near-fault areas. In order to evaluate responses, 4 buildings with buckling restrained braced frames were modeled in nonlinear analysis program. Comparing the results of numerical model with experimental results showed that the numerical model of BRBF has been accurate to simulation. To evaluate the seismic response of buildings, a 4 story model analyzed with selected records; this process repeated for buildings with 3, 9 and 12 story. The analysis results showed that the performance of the BRBFs was acceptable but the residual story drifts were very high and this affects the performance of the non-structural systems in BRBFs in near-fault areas. Some of the provisions of AISC were not conservative and the responses of the frames under near-fault ground motion were not mode-based, it was wave-based.

Key words: Steel, yielding, compression, ductility, energy

INTRODUCTION

The disadvantages of the CBF systems can be overcome if the brace can yield during both tension and compression without buckling. The braced frame that incorporate this type of brace, are called Buckling Restrained Brace (BRB).

Therefore, BRBF is a special class of CBF that precludes brace buckling. Figure 1 shows a comparison of the behavior of a BRB and a conventional brace. BRBFs have been applied to new construction and also to seismic rehabilitation of reinforced concrete buildings. As the use of BRBFs has grown, so it is essential to assess the response of this system and the recommended provisions more accurately. In this study the response of BRBFS modeled based on the experimental results and then investigated analytically.

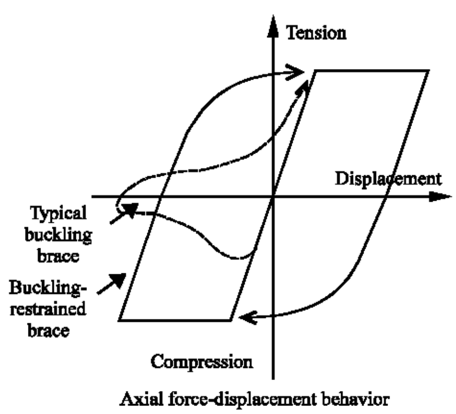

Fig. 1: Comparison of the behavior for a BRB and a conventional brace

STUDIED EXPERIMENTAL FRAME

A research program composed of numerical and experimental simulations have been conducted by Fahnestock *et al.* (2006). This experimental program have been conducted in three phase. First phase is a static loading. Second phase is an earthquake simulation, which was conducted using a hybrid pseudo-dynamic testing method and the last phase of experiment has been conducted to investigate the effects of low cycle fatigue on the responses. The prototype frame has been tested by Fahnestock *et al.* (2006) modeled and the results calibrated with the experiment results. The prototype building is a typical office building located on a stiff soil site. BRBFs are the lateral load resisting system for the building. A one bay frame of this building has been selected and experimented. The design procedure was based on the seismic provisions (AISC, 2005) and the

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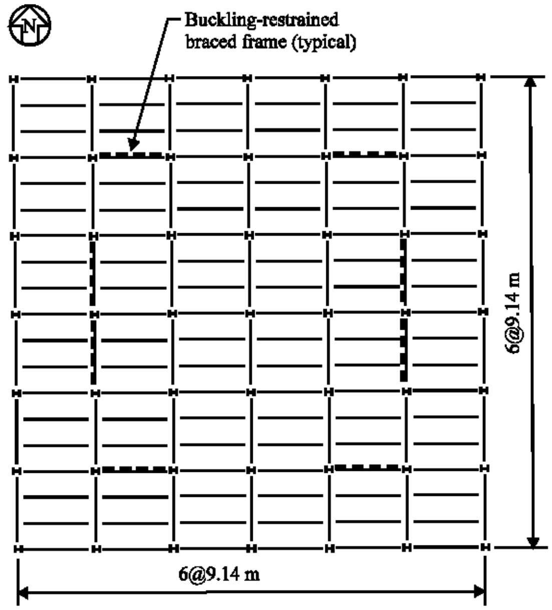

Fig. 2: Prototype building plan

equivalent-lateral-force procedure in the ICC2000. The design coefficients used in the prototype building design are the response modification coefficient, R, equal to 8, the system over-strength factor, Ω_0 equal to 2 and C_d equal to 5.5 (Sabelli, 2004).

A plan of the prototype building is shown in Fig. 2. The prototype frame is one of the BRBFs from the prototype building. The test frame is related to the prototype frame through a scale factor, λ, equal to 0.6.

Assessment of pseudo-dynamic simulation: The pseudo-dynamic test method is a relatively new experimental technique for evaluating the seismic performance of structural models in a laboratory by means of on-line computer control simulation. It is especially efficient for testing structures that are too large, heavy or strong to be tested on available shaking tables. The displacement response computed, based on a specific earthquake excitation record, is imposed on the test frame by means of electro-hydraulic actuators. Thus the quasi-statically imposed displacements of the test frame will resemble to that would actually be developed if the frame were tested dynamically.

The results of Mahin and Shing (1985) studies indicate that the pseudo-dynamic test method can be as realistic and reliable as shaking table tests. In addition it has been shown that the method can provide better controlled experimental conditions than shaking table tests for large and heavy specimens.

ANALYTICAL MODELS

Present study has been conducted using finite element analysis. The project starts in October 2008 in Structural Engineering Department, School of Civil Engineering, University of Tabriz and ended in 20th June 2009.

Modeling the experimental frame: For modeling the experimental frame the finite element analysis program ABAQUS V6.8 has been used. The prototype frame experimented by Fahnestock has been modeled with 2D frame element. Beam element has been used for beams, columns and braces. For modeling the BRBs, a boundary condition has been introduced in order to limit the buckling of brace in frame plane and out of frame plane. Frequency analysis has been used to investigate the natural periods. In this step of analysis the effect of forces has been ignored. The results compared with the results of the first phase of experiment. The records that have been used in the large-scale program applied to the analytical model and the model calibrated with the result of the experiment.

Details of BRBF analytical model: After the models calibrated, structures with 3, 9 and 12 story selected and have been studied. The design procedure was based on AISC (2005) and IBC (2000). The number of bays in all frames is 1. The length of bay is 9.14 m and the story height is 3.81. The beams and columns are A992 steel wide-flange sections, which has a yield stress of 350 Mpa and the BRBs were designed with core plates made of A36 steel, which has a nominal yield stress of 250Mpa. The loads applied to frames are dead loads equal to 540 (kg m^{-2}) for stories and 390 (kg m^{-2}) for roof and live loads equal to 340 and 100 (kg m^{-2}) for stories and roof, respectively.

Lateral loading have been done by using the equivalent-lateral load procedure. Studied models have been shown in Fig. 3.

Section properties of BRBFs: For design of 3, 9 and 12 story frames, the parameters of prototype model have been used. The prototype building has 4 braced bays in each direction. One of the frame selected and designed with several stories. Finally, there have been 4 frames, to evaluate the parameters that affected the seismic response of BRBFs. Since there is 4 frames in each direction in the building used in analysis and only one of the frames has been used during study the story shear was divided by 4. The results of design have been shown in Fig. 4. The dimensions of designed BRBs are in Table 1. These sections have been used in the process of analysis.

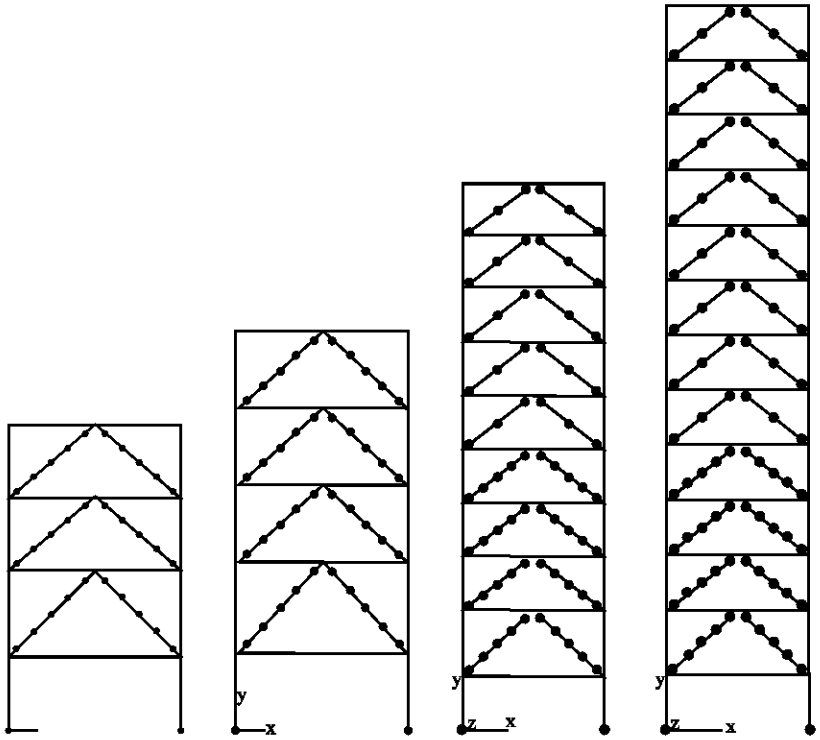

Fig. 3: Studied models

Table 1: BRBs dimensions (cm^2)

	3 story	4 story	9 story	12 story
BRB1	20	40	65	68
BRB2	15	30	58	65
BRB3	10	22.5	50	62
BRB4		12	40	60
BRB5		10	30	58
BRB6			24	50
BRB7			18	41
BRB8			12	32
BRB9			10	24
BRB10				18
BRB11				15
BRB12				12

Nonlinear dynamic analysis: To evaluate the parameters that affects on the seismic response of BRBFs and the way they effect, nonlinear dynamic analysis have been conducted. In modeling the nonlinearity, both the geometric nonlinearity and material nonlinearity with isotropic and kinematic hardening have been considered.

Selected near fault records: In this study, 7 near fault records have been used in dynamic analysis. The selections of records were based on the similarity of records with regard to source mechanism, distance to fault and top surface soil condition. The records have been selected from Pacific Earthquake Engineering Research Center strong motion database (PEER, 2000; SAC, 1997). The characteristics of records are given in Table 2. Selected records have threshold magnitude of 6.7 and all the records have been registered in distance less than 10 km. Peak Ground Acceleration (PGA) of records is given in the table in terms of g for each record.

Table 2: Selected records properties

PGA (g)	Nearest distance from rupture (km)	Magnitude	Year	Record
0.86	1.2	7.4	1978	Tabas
0.718	3.5	7.0	1989	Loma prieta
0.638	8.5	7.1	1992	Petrolia
0.432	2.0	6.7	1992	Erzinjan
0.71	1.1	7.3	1992	Landers
0.62	7.5	6.7	1994	Northridge
1.088	3.4	6.9	1995	Kobe

THE RESULTS OF DYNAMIC ANALYSIS

The results of experimental frame modeling: The frame used in Fahnestock program has been modeled with ABAQUS and the results have been compared with the experimental results. Shear-drift response for experimental simulation and analytical model has been compared in Fig. 5 and 6. It was found that good correlation was obtained between the experimental model and the finite element model.

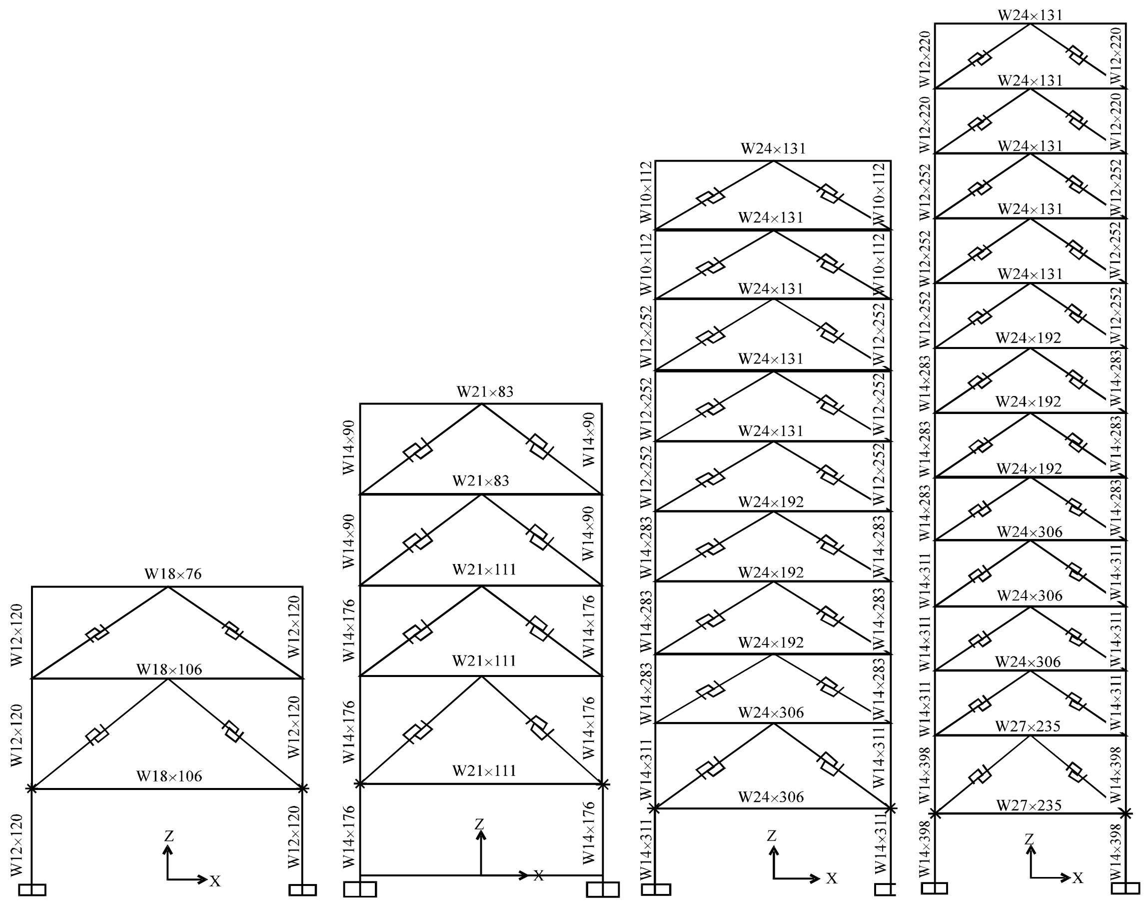

Fig. 4: Frames properties

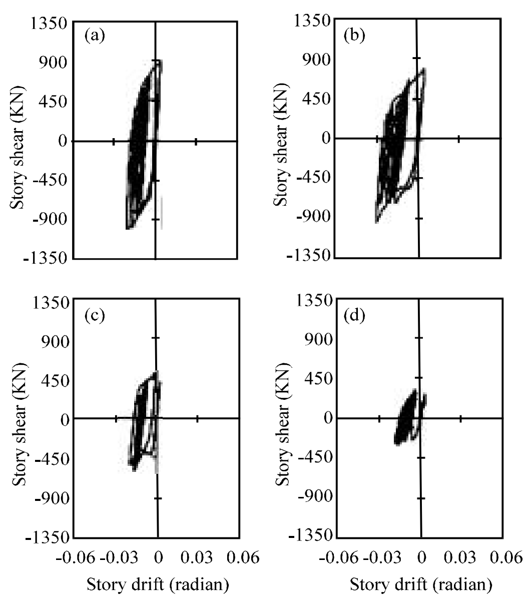

Fig. 5: Shear-drift response for experimental simulation (a) first story, (b) second story, (c) 3rd story and (d) 4th story

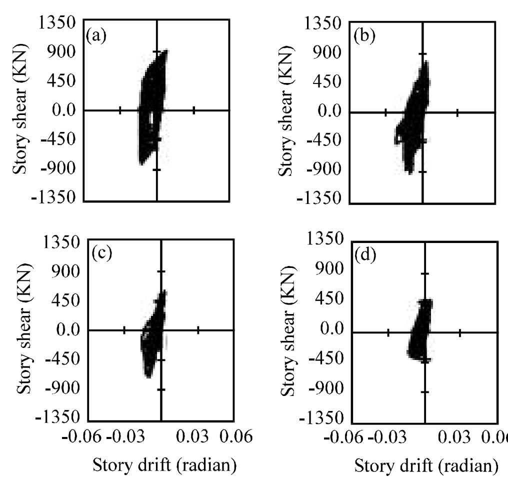

Fig. 6: Shear-drift response for numerical simulation (a) first story, (b)second story, (c) 3rd story and (d) 4th story

Analytical model results: Nonlinear dynamic analysis results of 3, 4, 9 and 12 story BRBFs under near-fault ground motions have been shown in Fig. 7 to 16 for some

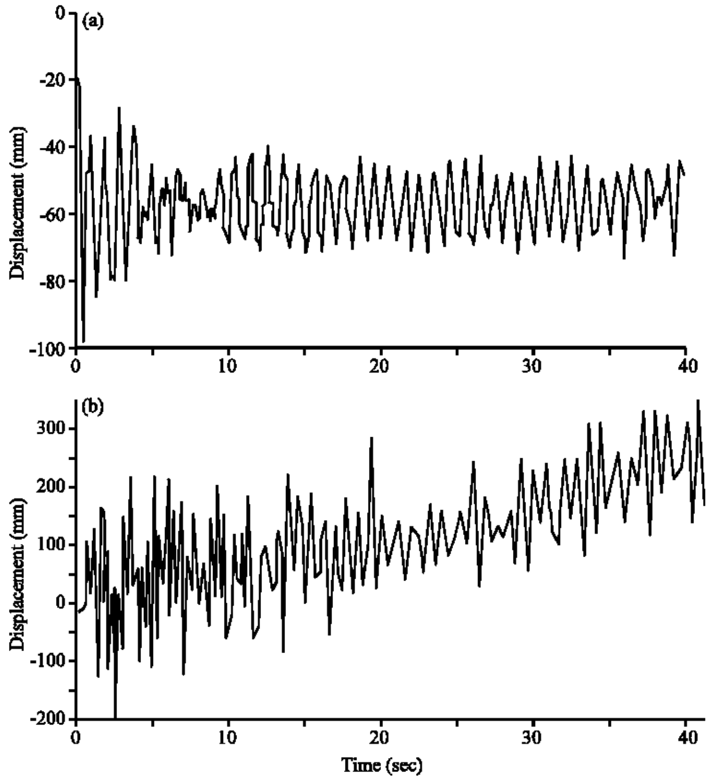

Fig. 7: Roof displacement time history for Landers record (a) 3 story frame and (b) 9 story frame

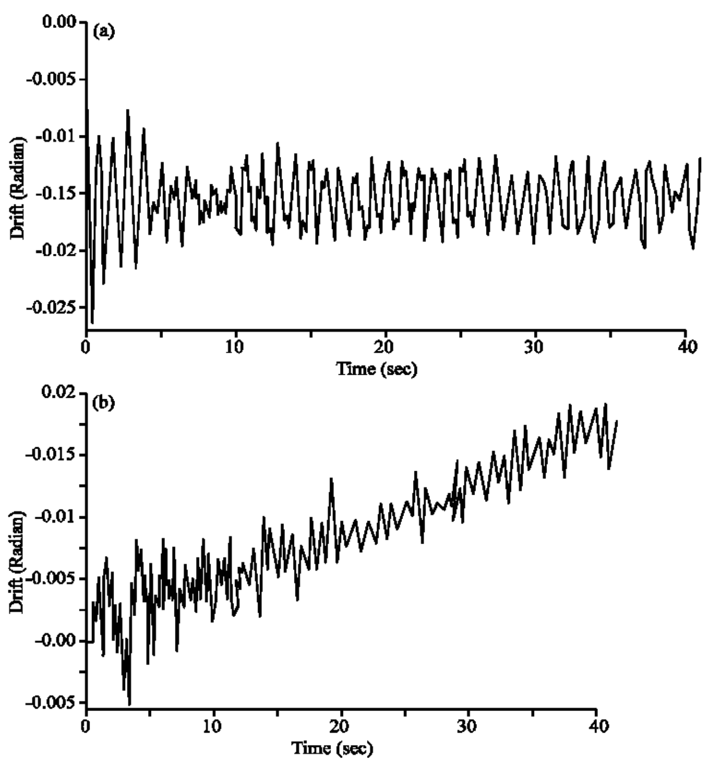

Fig. 8: Roof drift time history for Landers record (a) 3 story frame and (b) 9story frame

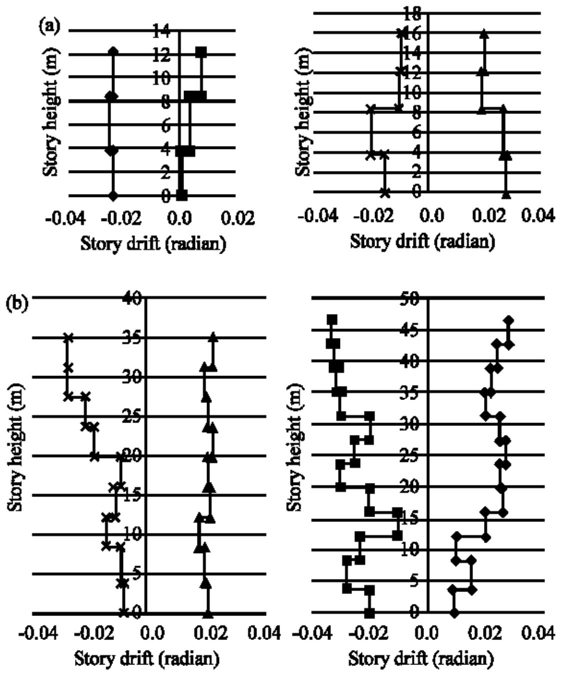

Fig. 9: The push of story drift for 3, 4, 9 and 12 story frame. (a) The push of story drift for Northridge earthquake and (b) The push of story drift for Loma Prietta earthequake

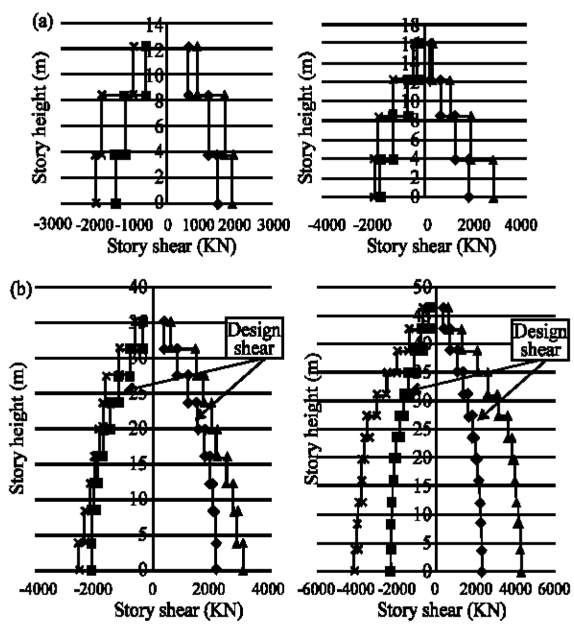

Fig. 10: The push of story shear for 3, 4, 9 and 12 story frames (a) The push of story drift for Northridge earthquake and (b) The push of story drift for Loma Prietta earthequake

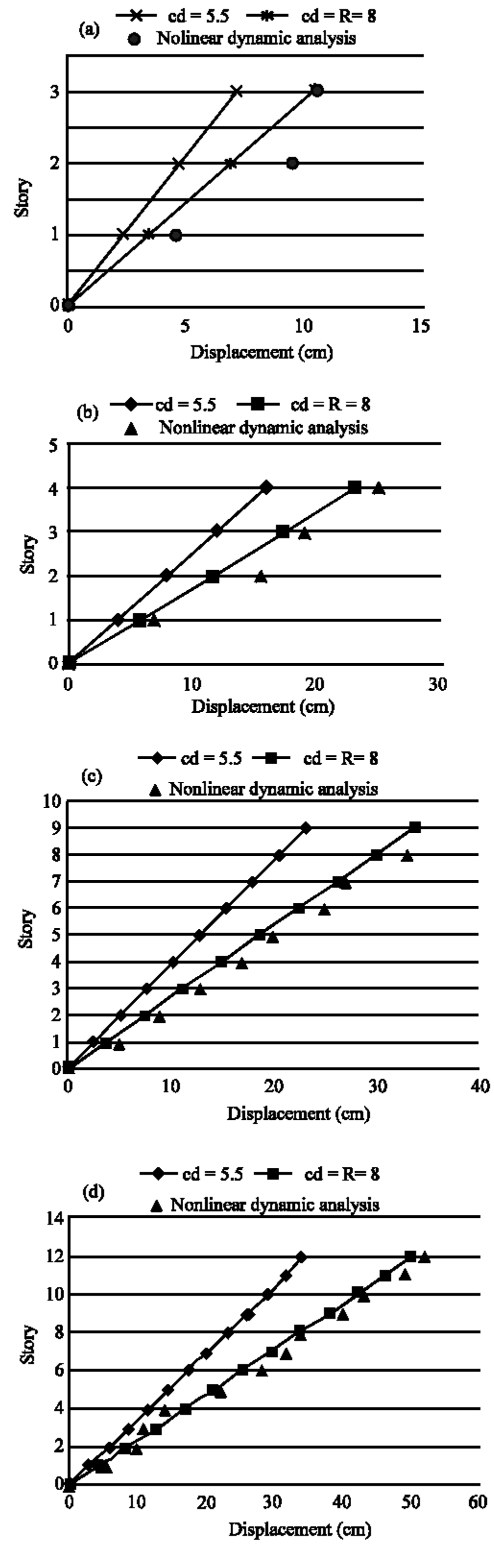

Fig. 11: Evaluation of C_d (a) 3 story frame, (b) 4 story frame, (c) 9 story frame and (d) 12 story frame

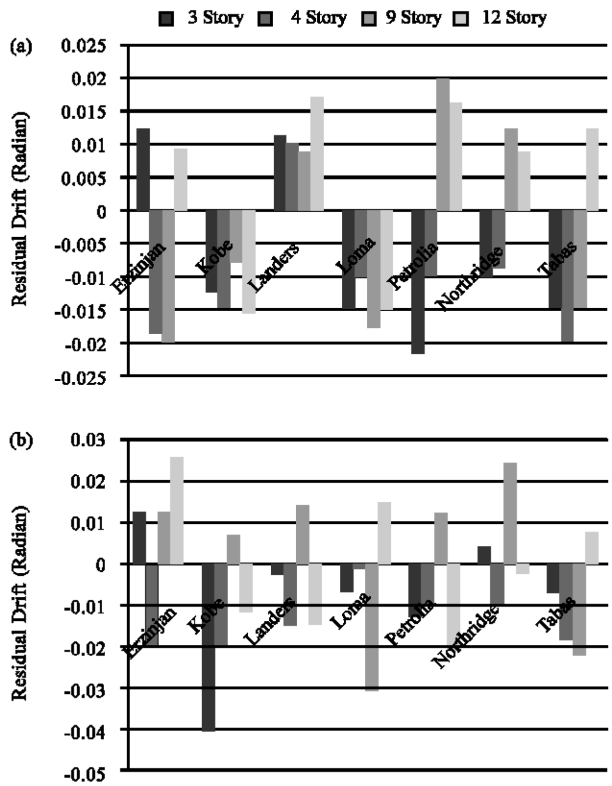

Fig. 12: (a)Maximum story residual drift and (b) Roof residual drift

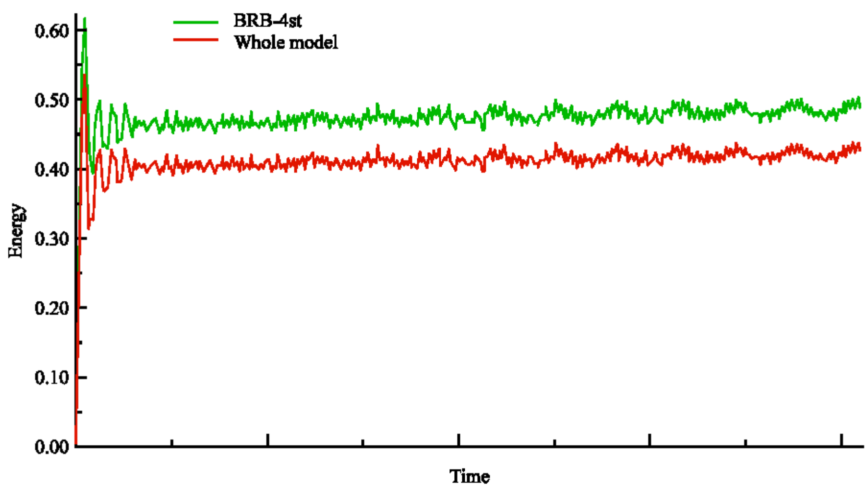

Fig. 13: The history of dissipated energy by BRB and Whole Model in 4 story frame

of the analysis. In this study, response results such as displacements, stress, strain and energy have been examined. There are extremely large amount of response result, thus only a brief examination of important results have been reported here.

Roof displacement time history and some of the records have been shown in Fig. 7. Displacement response of 3 story in Landers record is mainly in the negative direction while in 9 story frame is in the positive direction (Fig. 7). Roof drift time history for frames approximately following this pattern. Regarding to pulse-containing characteristics of near-fault records, responses are not symmetric, they had a tendency toward one direction and gradually increased. As the displacements

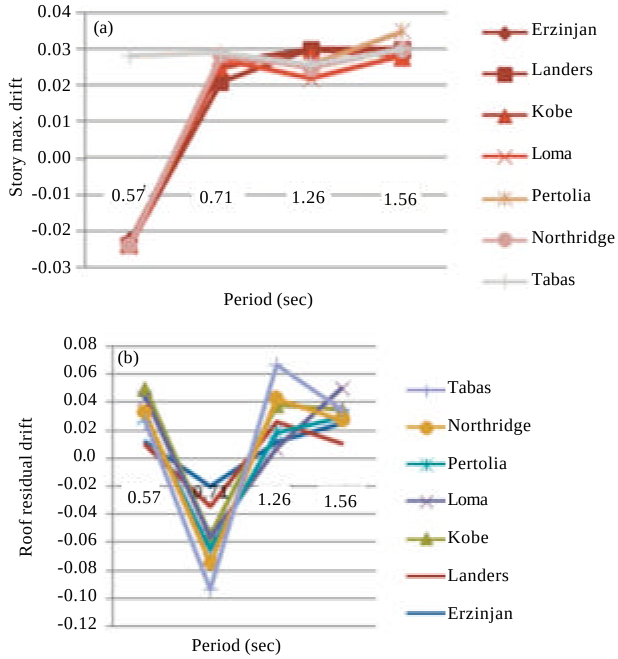

Fig. 14: Effect of vibration period on the (a) Story max drift and (b) roof residual drift

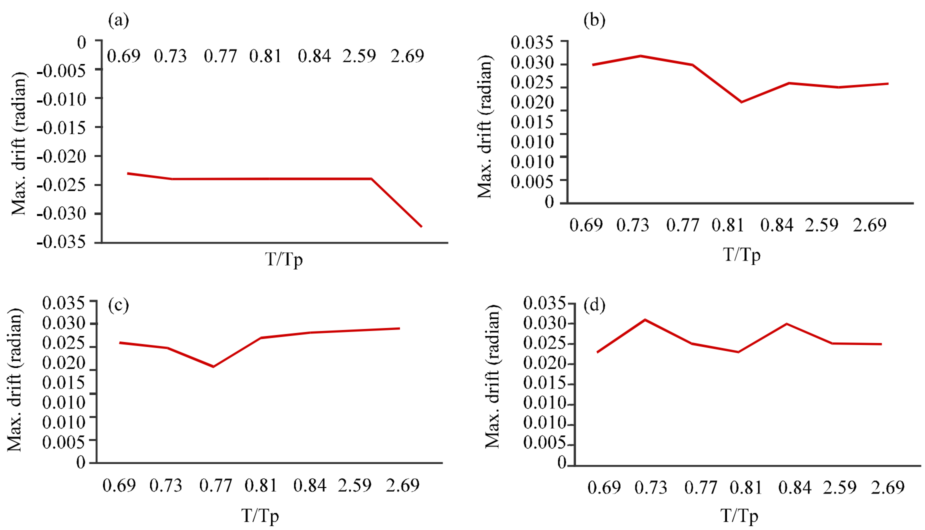

Fig. 15: The effect of T/Tp on the maximum story drift, (a) 3 story frame, (b) 4 story frame, (c) 9 story frame and (d) 12 story frame

were in one direction mainly and hadn't approach to initial position of frames, there is a relatively large residual drift at the end of the records (Fig. 8).

The push of story maximum drift and the push of story shear response have been shown in Fig. 9 and 10. As illustrted in Fig. 9 in the 3-story building, maximum story drift was mainly in lower stories and conversely in the 12-story byilding, the displacement demands were in the upper story. As the story shear has been shown in Fig. 10, the story maximum shear is higher than twice as design shear. Table 3 illustrates over strength factors for base shear, Ω_0. The V_{base} over-strength is defined as the peak V_{base} divided by the design V_{base}. The predicted Ω_0 for BRBFs by AISC was 2 while the story maximum shear was higher than twice as design shear. As illustrated in Table 3 maximum over-strength factor for

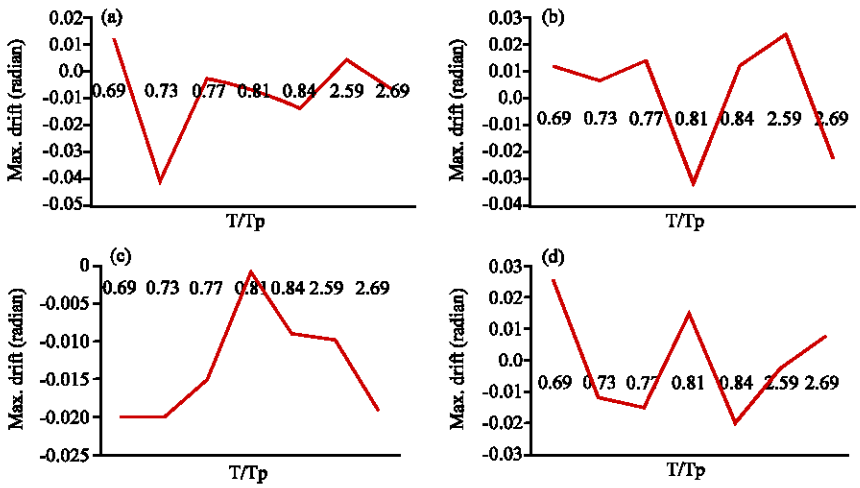

Fig. 16: The effect of T/Tp on the roof residual drift for, (a) 3 story frame, (b) 4 story frame, (c) 9 story frame and (d) 12 story frame

Table 3: Over strength factors for shear

Record	3 story	4 story	9 story	12 story
Erzinjan	1.76	2.09	2.1	2.07
Kobe	1.68	2.096	2.35	2.04
Landers	1.79	1.75	1.96	1.93
Loma Prietta	1.71	2.04	1.76	1.98
Petrolia	1.81	1.94	2.37	2.32
Northridge	1.57	2.08	1.93	2.05
Tabas	1.96	2.08	1.95	2.43
MAX	1.96	2.096	2.37	2.43
Average	1.75	2.01	2.06	2.11
Average+σ	1.87	2.13	2.28	2.3

Table 4: Maximum and cumulative ductility demands

Record	3 story	4 story	9 story	12 story
Maximum ductility demand				
Erzinjan	10.2	12	12.5	15
Kobe	11	10	11.5	17
Landers	9.8	10.6	12	16
Loma Prietta	12	12	11.6	13.8
Petrolia	11	11	12	16
Northridge	13	9.5	12.8	15.9
Tabas	12.5	13.2	13	16
MAX	13	13.2	13	17
Average	11.35	11.18	12.2	15.67
Average+σ	12.54	12.47	12.7	16.67
Cumulative ductility demand				
Erzinjan	53	76	70	65
Kobe	59	64	83	58
Landers	54	55	78	68
Loma Prietta	52	74	80	70
Petrolia	50	53	79	68
Northridge	70	48	78	66
Tabas	67	75	75	65
MAX	70	76	83	70
Average	57.85	63.57	77.57	65.71
Average+σ	65.67	75.27	81.68	69.57

BRBF system was in 12-story under Tabas record and was approximately 21% greater than the predicted value by AISC. Mean plus standard deviation of system over-strength is 2.3. The strength of BRBF connections, beams, columns and collector elements may be controlled by the system over-strength factor, for example, the columns and column splices of the prototype BRBF were designed with a load combination that contained Ω_0.

Table 5: Comparison between AISC prediction and analysis results

	μ_{max}	β	ω	βω
AISC prediction	9.4	1.18	1.50	1.778
3story	11.35	1.14	1.10	1.54
4 story	13.00	1.16	1.14	1.322
9 story	14.80	1.20	1.27	1.524
12 story	15.67	1.22	1.40	1.708

Ensuring that BRBs are detailed with adequate ductility capacity for imposed seismic demand is a critical aspect of BRBF design. The largest maximum ductility demand was in 12 story frame under Kobe earthquake and equal to 17 while the largest cumulative ductility demand was in the 9 story frame under Petrolia earthquake and equal to 83 (Table 4). Therefore it is not necessary that a large maximum ductility demand leads to a large cumulative ductility demand. Table 5 compares the ductility demand predicted by AISC with the dynamic analysis results. The predicted μ_{max} based on the seismic provisions (AISC, 2005) was 9.4. the mean μ_{max} from the nonlinear dynamic analysis ranged from 11.35 to 15.67, was greater than the prediction.

C_d is a multiplier that is applied to static elastic lateral displacements under design-level forces to estimate lateral displacements due to nonlinear dynamic response. Figure 11 shows a statistical summary of the maximum lateral displacement for structures. The proposed C_d value by Fahnestock also have been applied to the elastic displacement and compared with each other. C_d of 5.5 is too low to predict inelastic lateral displacement of BRBFs. However by setting C_d equal to R, which is equal to 8 a more accurate estimate of the mean inelastic lateral displacement is obtained.

Story maximum residual drifts and roof residual drift for 4 frames and 7 records have been shown in Fig. 12a and b, respectively. Magnitudes are in terms of radian. Since, drifts are large and the responses are one-faced naturally there would be a large amount of residual drifts. Maximum story drift was in 3 story frames and under Petrolia record and equal to 0.022 radian and maximum roof residual drift was in the 3 story and under Kobe earthquake and equal to 0.04 radian.

The history of dissipated energy by BRBs and whole model has been shown for an analysis in Fig. 13. As illustrated the most amount of dissipated energy from earthquake related to BRBs.

Table 5 illustrates the values of tension and compression adjustment factors that are equals to:

β = Ratio of the maximum compression force to the maximum tension force
ω = Ratio of the maximum tension force to the nominal yield force

The compression strength adjustment factor, β, is an important BRB parameter that is used in design. β allows for prediction of the unbalanced vertical force that will be applied to a beam attached to BRB in a chevron configuration.

The value of β in 3 story and 4 story frames was less than the values AISC predicted. But in the 9 story and 12 story frames this was a little larger than recommended value.

The effect of vibration period on the story maximum drift and maximum roof drift has been shown in Fig. 14. The amplitude of story maximum drift increased as the vibration period is increasing and the roof residual drift is the largest in all records while T = 0.76 sec. While the roof residual drift didn't follow this order. This means that roof residual drift is not coincident with maximum story drift. The 4 story frame had the largest residual drift and these drifts are mainly in the negative direction.

Story maximum drift variations with the ratio of vibration period to record pulse period have been shown in Fig. 15 and the roof residual drifts versus the ratio of vibration period to pulse period have been shown in Fig. 16. As illustrated when the structure period was near the pulse period (i.e., T/Tp tending to 1) story maximum drift increased. So when the vibration period of structure was near to record pulse period, the values of drifts and displacement were the largest. A relationship between record pulse period and residual drift couldn't be estimated.

DISCUSSION

The values of story maximum drifts have shown that BRBF are capable to reach the relatively high story drifts. As illustrated in Fig. 9 and 16 if the structure vibration period was near the pulse period, maximum story drift was mainly in lower stories and conversely when the vibration period was higher than pulse period the displacement demands was in the upper stories. In the Fahnestock prototype frame story drifts were fairly uniform over the height of frame studied but the response of BRBFs in the near-fault regions is mainly related to the ratio of vibration period to record pulse period. As the story shear have been shown in Fig. 10, the story maximum shear is higher than twice as design shear. This result has been achieved in the previous research (Fahnestock *et al*., 2006). The greater over-strength factor observed may suggest a need to reevaluate the specified value for Ω_0 in BRBFs.

The μ_{max} observed in the present study were numerically consistent with those reported by Fahnestock *et al*. (2006) and Sabelli *et al*. (2003). The μ_{max} range is around 12 which support previous findings. It was greater than the AISC prediction. As demonstrated in previous studies, BRBs with appropriate detailes (i.e., internal clearnece) are capable of sustaining μ_{max} greater than 20. As illustrated in Table 4 large μ_{max} may decrease the μ_c because of the low cycle fatigue phenomenon.

The maximum story residual drifts and roof residual drifts have been shown in Fig. 12. Since, drifts are large and the responses are one-faced in the near-fault records there were large residual drifts. The large residual drifts observed in the analysis indicate one potential drawback of the BRBF systems, as these large residual drifts may present significant challenges when seeking to return buildings with BRBFs to service after a major seismic event (Fahnestock *et al*., 2006).

The time history of absorbed energy that have been shown in Fig. 13 for a record, implies that the most amount of dissipated energy from earthquake related to BRBs. The time history of absorbed energy by Fahnestock's test frame verify this object. High capacity of dissipation of BRBs that is obvious from the hystersis loop of the BRBs, is linked to the yielding of BRBs both in tension and compression without buckling.

The value of β in 3 story , 4 story frames is less than the values AISC predicted. But in the 9story and 12 story frames this is a little larger than recommended value. These values satisfies the acceptance criterion of β in the seismic provisions (AISC). If the $\beta \le 1.3$ is near 1, less unbalanced force will be applied to the beam-brace connection in the mid span of beam.

Story maximum drift was increased with structure vibration period increasing (Fig. 14). While the roof residual drift didn't follow this order. This means that roof residual drift is not coincident with maximum story drift. The 4 story frame had the largest residual drift and these drifts are mainly in the negative direction.

The period of modeled buildings was between 0.5 to 3 sec and these buildings experienced large residual drifts. As the period of structure between 0.5 to 3 sec. is in the velocity sensitive region in the design spectrum and the effect of pulse period in velocity spectrum is obvious in the near-fault records, so if the BRBFs with natural period between 0.5 to 3 sec. were in the near-fault regions would be subjected to experience large story drifts and residual drifts.

As illustrated in Fig. 15 and 16 when the structure period was near the pulse period (i.e. T/Tp tending to 1) story maximum drift increased. This was may be because of the resonance phenomenon. So, when the vibration period of structure was near to record pulse period, the values of drifts and displacement are the largest. A relationship between record pulse period and residual drift couldn't be estimated.

The mean value of maximum displacement of frames obtained from the dynamic analysis have been shown in Fig. 11. The proposed C_d value by Fahnestock also have been applied to the elastic displacement and compared with each other. C_d of 5.5 is too low to predict inelastic lateral displacement of BRBFs. However, by setting C_d equal to R, which is equal to 8 a more accurate estimate of the mean inelastic lateral displacement is obtained. The proposed value of C_d by Fahnestock is more near the dynamic analysis results rather than AISC recommended provision. This building code procedure assumes that first-mode response is dominant and neglect higher mode effects that may produce larger drifts in certain stories. So, because of the high period pulses in near fault records, the response of the building was not mode-based, it was wave-based.

CONCLUSION

- The response of BRBFs in the near-fault regions is mainly related to the ratio of vibration period to record pulse period
- The greater over-strength factor observed may suggest a need to reevaluate the specified value for Ω_0 in BRBFs
- Large μ_{max} may decrease the μ_c because of the low cycle fatigue phenomenon
- The large residual drifts observed in the analysis indicate one potential drawback of the BRBF systems, as these large residual drifts may present significant challenges when seeking to return buildings with BRBFs to service after a major seismic event
- High capacity of dissipation of BRBs is linked to the yielding of BRBs both in tension and compression without buckling
- The β values satisfies the acceptance criterion of $\beta \leq 1.3$ in the seismic provisions (AISC)
- The roof residual drift is not coincident with maximum story drift
- If the BRBFs with natural period between 0.5 to 3 sec. were in the near-fault regions would be subjected to experience large story drifts and residual drifts
- When the structure period was near the pulse period (i.e., T/Tp tending to 1) story maximum drift increased. This was may be because of the resonance phenomenon
- C_d of 5.5 is too low to predict inelastic lateral displacement of BRBFs. The proposed value of C_d by Fahnestock is more near the dynamic analysis results rather than AISC recommended provision
- Because of the high period pulses in near fault records, the response of the building was not mode-based, it was wave-based

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