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COMPARATIVE STUDY ON SEISMIC BEHAVIOR OF SPECIAL CONCENTRIC BRACED FRAMES WITH ECCENTRIC BRACED FRAMES

Mohsen Tehranizade[1], Touraj Taghikhani[2], Mahdi Kioumarsi[3], Leila Hajnajafi[4]

1. Head, Professor, Department of civil engineering & environments, University of Amirkabir, Tehran
Email: tehz@govir.ir

2. Associate Professor, Department of civil engineering & environments, University of Amirkabir, Tehran
Email: ttaghikhany@aut.ac.ir

3. MSc student, Department of civil engineering & environments, MS Student, University of Amirkabir, Tehran
Email: mehcivil@yahoo.com

4. MSc student, Department of civil engineering & environments, MS Student, University of Amirkabir, Tehran
Email: lilanajafi@yahoo.com

ABSTRACT

Extensive damage of concentrically braced frames in recent earthquakes supported the necessity of revision in the design method of these systems. Accordingly, Special Concentrically Braced Frames (SCBF) have been introduced in recent years. It seems that these systems not only have significant inelastic deformability under seismic loads but also have less practical detailed problems than Eccentric Braced Frame (EBF).

In this manuscript, seismic behavior of these two kinds of bracing systems has been studied. The behavior of Special Concentrically Braced Frame (SCBF) systems have been compared with Eccentric Braced Frame (EBF) in 5, 10 and 15 stories buildings. Global ductility, maximum story drift and roof drift for two types of bracing frames have been investigated under nonlinear dynamic analysis. Input ground motions used in dynamic analysis is both near field and far field motions.

The buildings were designed according to NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (Fema 450) [Fema, 2003].

Results show that the frames both in modified concentric braced frames and EBF systems behave in an acceptable manner. Whereas the stiffness of SCBF frames is more than EBFs and it could suffer more base shears. The conclusions of investigating some parameters like global ductility and maximum dissipated inelastic energy are discussed in this study. These parameters also depend on level of PGA and frequency content of the seismic motion.

Key words: Special Concentrically Braced Frame, Eccentric Braced Frame, Drift, Ductility, Nonlinear analysis.

1. INTRODUCTION

Extensive damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico (Osteraas *et al.*, 1989), 1989 Loma Prieta (Kim *et al.*, 1992), 1994 Northridge (Tremblay *et al.*, 1995; Krawinkler *et al.*, 1996), and 1995 Hyogo-ken Nanbu (Hisatoku *et al.*, 1995; Tremblay *et al.*, 1996) earthquakes, raises concerns about the ultimate deformation capacity of this class of structure. Brace hysteretic behavior is asymmetric in tension and compression, and typically exhibits substantial strength deterioration when loaded monotonically or cyclically in compression. the Special Concentrically Braced Frames (SCBF) and Eccentrically Braced Frames (EBF) have had more proper ductility than CBFs. Therefore the researches about later systems have been grown these days. Researches show that both SCBF and EBF frames have sufficient ductility and stiffness (Naeim., 2001), whereas the stiffness of SCBF frames is more than EBFs and also they have less practical detailing problems than EBFs. The study of seismic behavior of these two kinds of systems and their comparison are as follow.

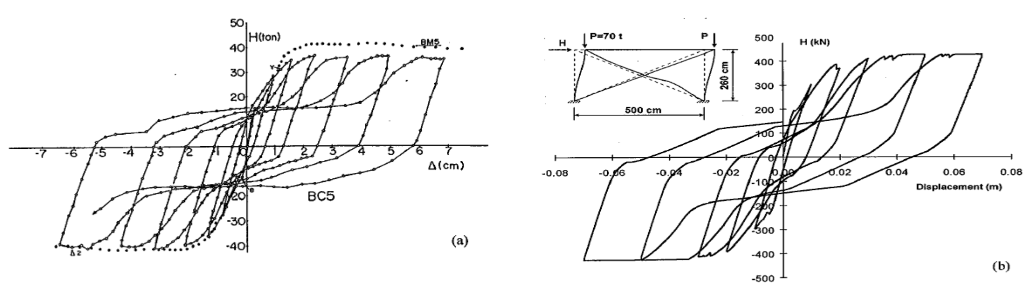
2. SPECIAL CONCENTRICALLY BRACED FRAMES

From 1997 in most codes and provisions like SEAOC-1999, BSSC-1997, Fema 450, Fema 2003 and AISC 1997 the subject of systems with special ductility have been offered. Concentrically braced systems have divided in to two groups, Special Concentrically Braced Systems (SCBF) and Ordinary Concentrically Braced Systems (OCBF).
The width-thickness ratio (λps) for sections to be used as SCBF braces should be less than this ratio for OCBF braces (AISC 2005). The reduction of braces width-thickness ratio in SCBF systems leads to the braces could reach the plastic deformation and plastic hinges have been made in braces before they get under buckling. End connections and connection plates should satisfy the strength requirements according to used code. The inelastic behavior of braces like Plastic-buckling and Tensile-yielding are guaranteed by all these requirements.
SCBF systems could sustain large inelastic deformations without significant strength reduction:

1-Tensile-brace: that the ductile portion is the whole length of bracing member

2-Compressive-brace: that the inelastic bucking causes plastic hinges in two end and middle of the bracing member.

The hysteretic diagram for one X-shaped brace in SCBF systems based on theoretical and empirical model is shown in below.



a) Based on empirical models *b) Based on numerical and theoretical models*

Figure1. The hysteretic diagram for one X-shaped brace in SCBF systems

In SCBF systems to reach the expected performance and to support the stability of inelastic deformations, the members shall be designed for special requirements. Some of these members are:

-Columns in both side of braced frame and their column splices

-Beams in both side of braced frame and their beam splices

-The connection of column to gusset plates

-The connection of beam to columns in the frames that located in the lateral force transferring.

-Diaphragms

-Connection of braces

3. ECCENTRICALLY BRACED FRAMES

EBF systems have two important specifications, ductility and stiffness (Engelhardt *et al.,* 1989). As a result of Proper ductility of eccentrically braced frames (EBF) using of this system in construction of buildings have been spreaded, cause comparative researches between this system and special concentrically braced frames (SCBF).
Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the link-beams when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links should be designed to remain essentially elastic under the maximum forces that can be generated by the fully-yielded and strain-hardened links (AISC 2005).
The design principals of EBFs can be understood more effectively by investigating the tensile strength of string of chain as illustrated in figure 2.

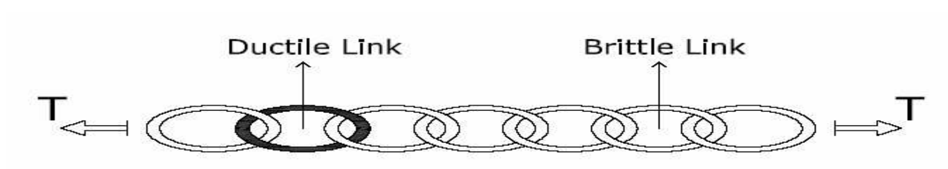


Figure 2 . Representing EBFs systems as string of chain

It can be concluded that the ductility of whole chain could be controlled by the ductility of one of its segments. The nominal tensile strength of this segment is supposed to be controlled by its ductility. Whereas other segments of the chain could be brittle and should be designed so that they have strength higher than the maximum strength of the lean segment. In EBF systems the link beam should be consider as a lean segment of the chain and other parts of system like columns and beams out of the link should be considered as brittle parts of chain (Bruneau *et al*., 1998).

4. MODELING FRAMES AND APPLIED ACCELEROGRAMS

Two EBF models both with 2 braced spans have been analyzed. One of the EBF models has link beam with length of 0.5m that represented shear-link beam and the other has the link beam with length of 2.5m that represented moment-link beam. Also two SCBF models that have 2 and 3 braced spans have been analyzed. 3 models with 5, 10, and 15 numbers of floor have been analyzed. The height of each story in all frames is 3m and the length of spans is 5m. In addition 12 frames were modeled 6 EBF frames and 6 SCBFs. All frames designed according to FEMA356.

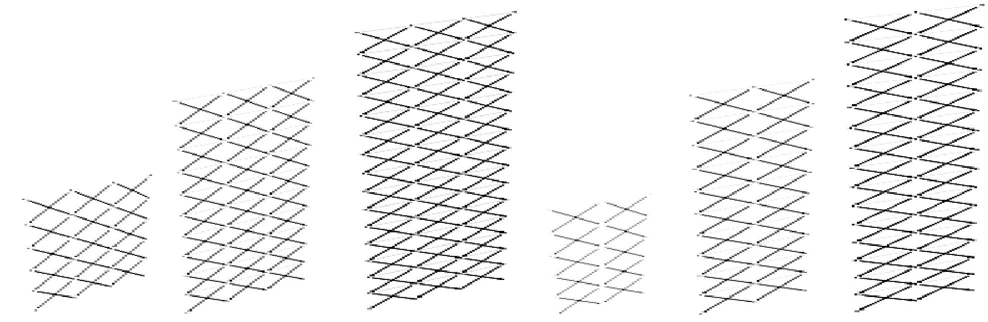


Figure3. models for SCBF systems

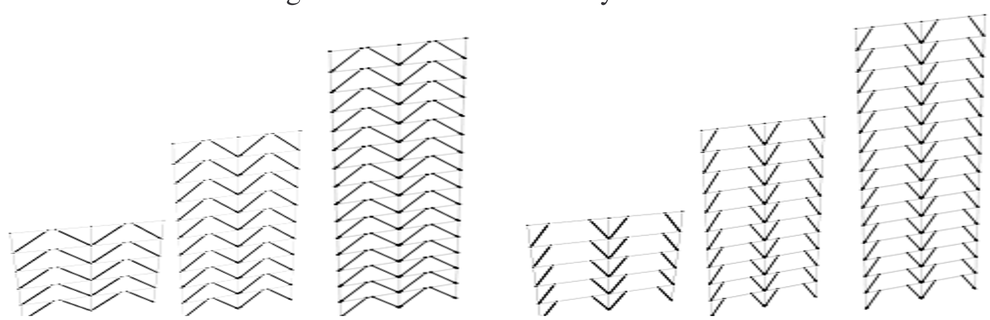


Figure4. models for EBF systems

Modeling of buildings has been done by using Programs ETABS v8.45 and RAMPerform-3D. The models hinge properties have been modeled according to FEMA356. It should be noticed that when models subjected to near-field strong motions, the hinges in EBF frames with long link-beam, deform so large that go beyond FEMA356 hinges limitations. Thus the models do not satisfy the hinge deformation limits and the results for these hinges could not be investigated.
To perform dynamic nonlinear analysis horizontal components of 6 strong motions have been applied. The properties of these accelerograms are in Table1. The accelerograms have been scaled to peak accelerations, 0.8g, 0.6g and 0.4g.

Table 1: specifications of used strong motions

earthquake	fault distance (km)	component	PGA (g)	PGV (cm/s)	PGD (cm)
IMPERIAL	54.1	VCT 75	0.122	6.4	2.09
		VCT 345	0.167	8.3	1.05
TABAS	28.8	DAY-LN	0.328	20.6	12.56
		DAY-TR	0.406	26.5	8.75
EL CENTRO	12	ELC 180	0.313	29.8	13.32
		ELC 270	0.215	30.2	23.91
NORTHRIDGE	7.1	NWH090	0.583	75.5	17.57
		NWH360	0.59	97.2	38.05
CHICHI	0.24	TCU-52-N	0.419	118.4	246.15
		TCU-52-W	0.348	159	184.42

ERZINCAN	2	ERZ-NS	0.515	83.9	27.35
		ERZ-EW	0.496	64.3	22.78

In addition by the chosen models and applied strong motions, 216 dynamic time history analysis have been done. In this research the results of these analysis and some analyze graphs have been investigated.

5. ANALYZING RESULTS OF STRUCTURES RESPONSE

5.1. INTRODUCTION OF INVESTIGATED ENERGIES

Utilizing energy relations to evaluate the operation and efficiency of designed structures is one of main factors to investigate the structures performance. Two fundamental energy equations are absolute energy equation and relative energy equation. Theses equations are as follow:

$$E_I=E_A+E_D+E_K \quad (1)$$

$$E_A=E_S+E_H \quad (2)$$

From these two equations we have:

$$E_I=(E_D+E_H)+(E_K+E_S) \quad (3)$$

In these equations E_I is the input energy, E_A stored elastic energy, E_K kinematical energy, E_D is dissipating energy by viscous linear damping equivalent to hysteric damping, E_H is dissipating energy by residual plastic deformations and E_S is the elastic strain energy. In this parameter E_I represents the demand and (E_D+E_H) represents the capacity of structure (Soong *et al*., 1997).

5.1.1. COMPARISON OF STRUCTURES INPUT ENERGY

The input energy represented the demand on the structure and is a function some characters such as type of bracing system, number of braced spans, number of stories and the amount of imported PGA to structure.

The results show that in 5 story frames, as it could be seen in figure 5, the input energy in SCBF systems with 2 and 3 braced spans is more than EBF systems.

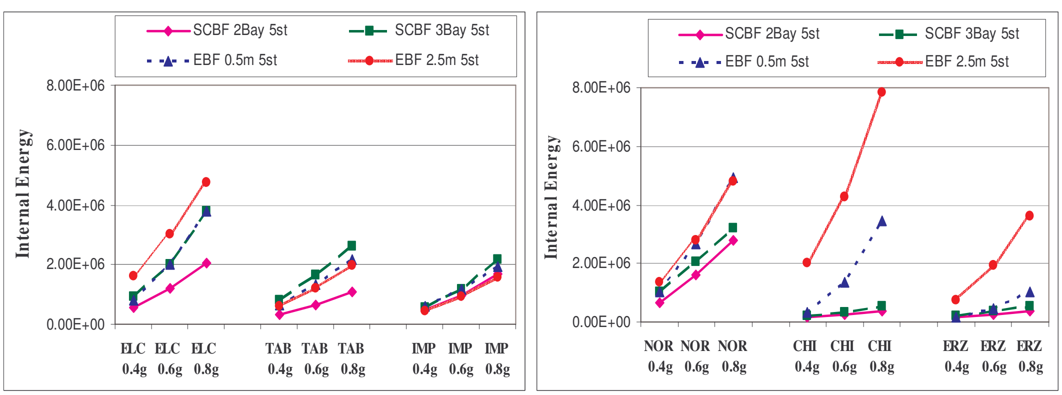


Figure5. The amount of Internal energy in 5 story frames

This fact in moment link-beam EBFs is more evident than shear link-beam EBFs. And if we apply near-field strong motions this difference would be more. In the same number of stories SCBFs with 3 braced spans have more input energy than the similar system with 2 braced spans.

In the 10 and 15 story frames the conclusions differ. In this number of story frames the SCBF systems have more input energy than the EBF systems. But like the 5 story frames the special concentrically braced frames with 3 braced spans have more input energy than the similar system with 2 braced spans.
The input energy by itself is not a good term to compare the behavior of frames; the hysteric energy to input energy ratio is the more proper parameter.

5.1.2. COMPARISON OF HYSTERIC ENERGY TO INPUT ENERGY RATIO

One of the controlling parameters of structures behavior is the absorbed energy in the stories of frames. In EBFs this energy is absyrbed by the link beams and in the SCBF systems is absorbed by the braces. the hysteric energy to input energy ratio in EBF frames are more than SCBF systems.

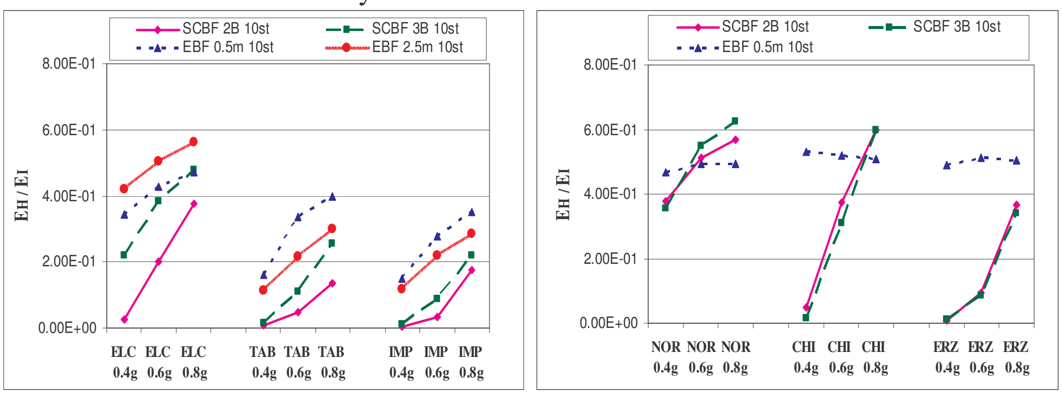


Figure6. The hysteric energy to input energy ratio in 10 story frames

The point that takes the consideration in 10 story frames is that under applying the near-field strong motions to the frame the rate of increasing hysteric energy to input energy ratio decreased and in some cases the hysteric energy to input energy ratio by increasing the imported PGA values became constant. This ratio under high PGAs for EBFs is less than SCBFs. These conclusions could be taken for 15 story frames too.

5.2. CHANGES IN BASE SHEAR

Changes in base shear of different structures could show the changes in stiffness of the frames. As we can see in figures 6 and 7, for 5 and 10 stories frames the maximum base shear is the most in SCBF systems with 3 braced spans and reduced in order in SCBF systems with 2 braced spans, shear link-beam EBF systems and moment link-beam EBFs.

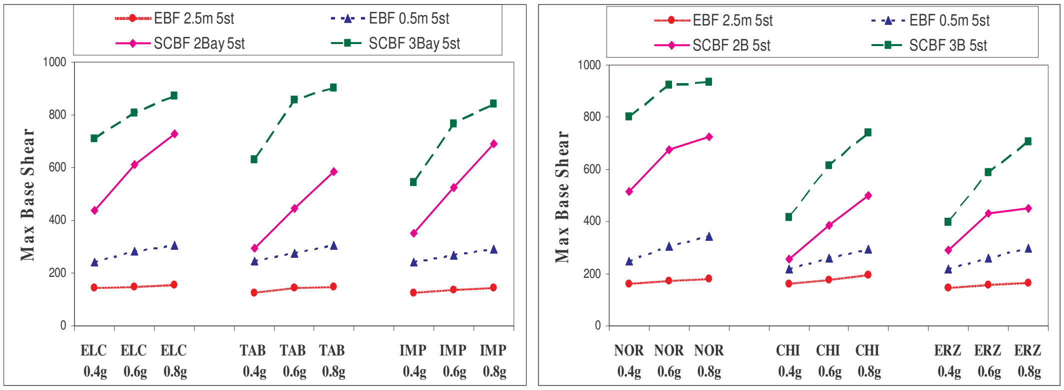


Figure7. The maximum base shear in 5 story frames

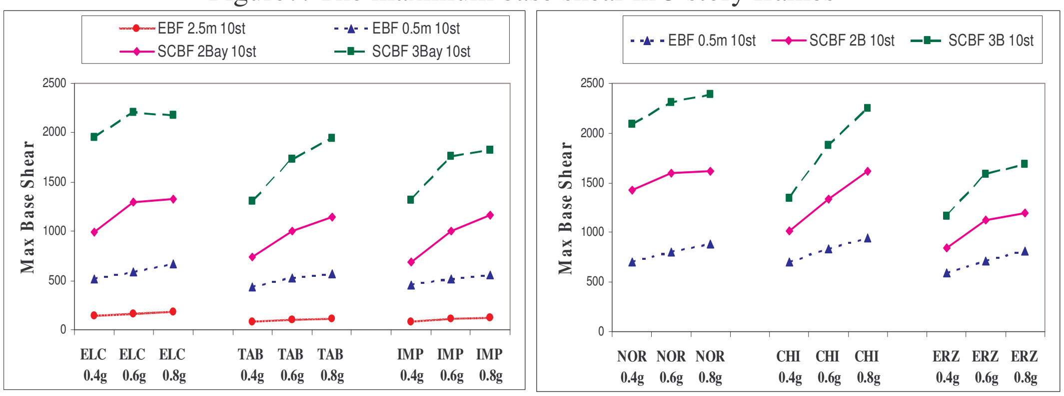


Figure8. The maximum base shear in 10 story frames

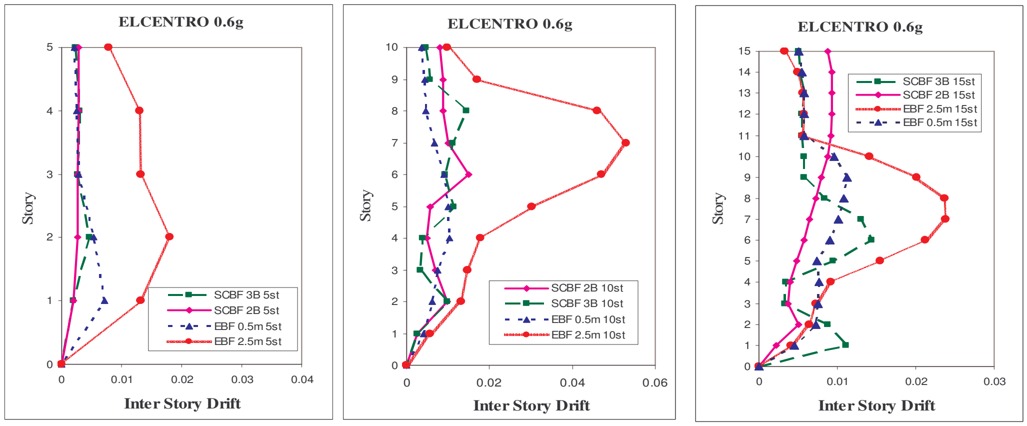
By increasing the PGAs the maximum base shear increases too. It should be said that after some increases in PGAs the rate of increase in maximum base shear reduced or even the maximum base shear become constant. The cause of this can be that by increasing the PGAs values the frame deformation increase too and Increasing in frame deformation brings about uniform shear. This fact has been shown in figure 7, for 10 story SCBF frames with 2 braced spans encountered by Northridge, Erizincan and ChiChi earthquakes.

And the last point is that the maximum base shear in SCBF systems is more sensitive to increasing the values of PGAs than the maximum base shear in EBFs.

5.3. STORIES DRIFTS

The maximum relative deformations between stories are one of the parameters to distinguish the creation of extreme flexibility in frames stories. as a global conclusion it was perceived that in all frames corresponding all strong motions both near-field and far-field motions the shear link-beam EBF systems have less drifts than moment link-beam EBFs, therefore the probability to form the flexible story in moment link-beam EBFs is more than shear link-beam EBFs.

In short frames the increase in number of spans does not have so effect on story drifts, but as the height of frames goes upper the increase in number of spans in SCBF systems causes the reduction of drifts especially in the upper stories.



(a) (b) (c)

Figure 9. comparisons of 5, 10 and 15 story drifts for different systems under Elcentro earthquake with the PGA 0.6g

Shear link-beam EBFs satisfy the story drifts requirements thoroughly. The drifts in these systems in upper stories often are less than the drifts in SCBF systems. In 10 stories building the drifts in middle stories exceeds the other stories drifts and the reason is in higher frames like 10 and 15 frames in this study, the effects of higher modes participate in the response of structure more than short frames like 5 story frames.

5.4. ROOF Displacement

One of the terms that was considered in the investigation of frames behavior is maximum roof displacement. By investigating the results of applying different earthquakes with different PGAs values, it has understood that, by increasing in number of braced spans in SCBF systems, if the structure is short the lateral deformation increase too and if structure is high the lateral deformation decrease inversely. In general we could say that increasing in number of braced spans controlled lateral displacements in upper stories more than bottom stories. The lateral displacements in shear link-beam EBFs is less than moment link-beam EBFs, the lateral displacements in shear link-beam EBF systems is small enough and can be compared with the lateral displacements of special braced systems.

6. CONCLUSIONS

-10 and 15 story EBF frames with moment link-beams under near-field strong motions represent large inter story drifts and large roof displacements.

-By considering amount of energy in the studied frames, the hysteric energy to input energy ratio is more in EBFs than the SCBF systems. The difference of this ratio for EBFs and SCBFs reduced by increasing in PGA values.

- the maximum base shear is the most in SCBF systems with 3 braced spans and reduced in order in SCBF systems with 2 braced spans, shear link-beam EBF systems and moment link-beam EBFs.

- By increasing the PGAs the maximum base shear increases too. It should be said that after some increases in PGAs the rate of the increase in maximum base shear reduced or even the maximum base shear become constant.

- It was perceived that in all frames corresponding all strong motions both near-field and far-field motions the shear link-beam EBF systems have less drifts than moment link-beam EBF systems, therefore the probability to form the flexible story in moment link-beam EBFs is more than shear link-beam EBFs.

- The relation between the increase in number of spans in SCBF systems and stories drifts is in the manner that in short frames this increase does not have so effect on story drifts, but as the height of frames goes upper the increase in number of spans in SCBF systems causes the reduction of drifts specially in the upper stories. By increasing in number of braced spans in SCBF systems, if the structure is short the lateral deformation increase too and if structure is high the lateral deformation decrease inversely. In general we could say that increasing in number of braced spans controlled lateral displacements in upper stories more than bottom stories.

7. REFERENCES

AISC 2005. American Institute of Steel Construction, *Seismic Provisions for Structural Steel Buildings*.

Bruneau, M. and Uang, C. Whittaker, A. *Ductile Design of Steel Structures*, McGraw-Hill, 1998.

Engelhardt, M. and Popov, E., *On design of eccentrically braced frames*, Earthquake Spectra, Vol. 5, No. 3, 1989, PP. 459-511.

Naeim, F., *The Seismic Design Handbook*, 2nd ed., Kluwer Publishers, 2001a.

FEMA, *FEMA 450: NEHRP Recommended provision for seismic Regulations for New buildings and Other Structures*, FEMA 450, Federal Emergency Management Agency, Washington, DC.

FEMA, *FEMA 356: NEHRP Guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, Washington, DC, 2000.

Hisatoku, T., *Reanalysis and repair of a high-rise steel building damaged by the 1995 Hyogoken-Nanbu earthquake*, Proceedings, 64th Annual Convention, Structural Engineers Association of California, Structural Engineers Assn. of California, Sacramento, 1995, pp 21-40.

Kim, H. and Goel, S., *Seismic evaluation and upgrading of braced frame structures for potential local failures*, UMCEE 92-24, Dept. of Civil Engineering and Environmental Engineering, Univ. of Michigan, Ann Arbor, Oct. 1992, 290 pages.

Krawinkler, H. et al., *Northridge earthquake of January 17, 1994: reconnaissance report, Volume 2 steel buildings*, Earthquake Spectra, 11, Suppl. C, EERI, Jan. 1996, pp 25-47.

Osteraas, J. and Krawinkler, H., *The Mexico earthquake of September 19, 1985 behavior of steel buildings*, Earthquake Spectra, 5, 1, Feb. 1989, pp 51-88.

Soong, T.T. and Dargush, G.F. 1997. *Passive Energy Dissipation Systems in Structural Engineering*, Wiley, London.

Tremblay, R. et al., *Performance of steel structures during the 1994 Northridge earthquake*, Canadian Journal of Civil Engineering, 22, 2, Apr. 1995, pp 338-360.

Tremblay, R. et al. *Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake*, Canadian Journal of Civil Engineering, 23, 3, June 1996, pp 727-756.