Evaluation of Seismic Behavior of Buckling Restrained Braced Frames as Dual System in Combination with Special Moment Resisting Frames

A. Mehdipanah, S.R. Mirghaderi & S.A. Razavi
Faculty of Civil Engineering, University of Tehran, Iran

SUMMARY:
In this study the performance of 4 mid-rise and 4 high-rise buildings has been assessed by conducting nonlinear static and time history analyses. These structures are designed according to stiffness-based procedure which has been developed in this article. The performances of the structures are analyzed by using PERFORM-3D. Analyses demonstrate that adding the SMRF to the BRBF system changes the natural period slightly. Also the results of this study show that the optimum stiffness proportion is when the BRBF withstand 65% of the base shear and SMRF designed for 35% of base shear by means of stiffness based design. The structure with this ratio of stiffness proportioning has more appropriate seismic behaviour in both mid-rise and high-rise buildings.

Keywords: BRBFs, MRFs, Dual System, Pushover, Time history

1. INTRODUCION

Buckling-Restrained Braced Frame (BRBF) systems have shown predictable performance and robust energy dissipation capacity when subjected to seismic loading. However, the low post-yield stiffness of Buckling-Restrained Braces (BRB) may cause BRBFs to exhibit large maximum and residual drifts. (Ariyaratana and Fahnestock, 2009) To reduce residual story drifts, it is suggested that one option is to design the BRBFs as a Dual system; the addition of special moment-resisting frames, which exhibit large deformability in the elastic range, can serve as a restoring force mechanism to partially re-center the building after a significant seismic event. (Kiggins and Uang, 2006)

BRBFs can provide significant elastic stiffness and cause small elastic drifts, while SMRFs have small lateral stiffness to the extent that limiting lateral drifts in SMRF is the governing design criteria. By combining these two systems a dual system with advantages of the two systems can be provided and the disadvantages of the two systems can be prevented as well. The flexible SMRF remains elastic after the BRBF have yielded and provide additional stiffness and prevent large drifts leading in less residual drifts for the whole structure. The force-displacement curves for individual systems are shown in Fig. 1 as well as the dual system. (Mehdipanah et al., 2012)

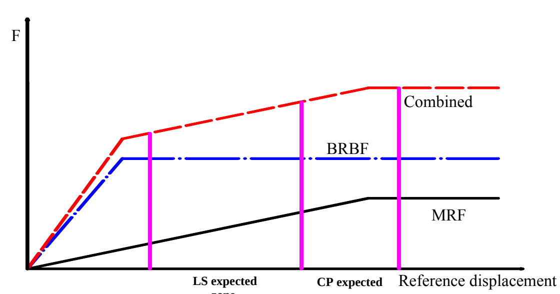


Figure 1. Force displacement response of the systems.

ASCE 7-10 (ASCE, 2010) permits a variety of structural systems to be used in combination as a dual system, yet the design requirements are limited to the following: Moment frames must be capable of resisting 25% of the seismic forces while the whole structure must be capable of resisting the entire seismic forces in proportion to their relative rigidities.

The common design practice for the dual systems consists of the following steps:
1) Design the backup system to resist 25% of the design forces without the aid of the primary system.
2) Check the combined system, for the full lateral forces.

This design procedure has been called in this paper as strength-based design procedure since the structure is designed to provide enough strength against the earthquake load. The problem with this design procedure is that the base shear distribution between two systems is not predictable. Generally the base shear is distributed according to relative stiffness of two systems, since these systems are parallel. These systems are constrained to a solid diaphragm which obliges the two systems to have the same drift in each story. By following this procedure for designing dual systems the designer is not capable of recognizing the force distribution between 2 systems. Furthermore strength-based design procedure will not help the designer to have enough information about the behavior of each system in dual configuration, because it does not lead in unique solution. So, without a nonlinear analysis, portion of forces supported by primary system and backup system is not definite.

To mitigate this problem, it is proposed the design procedure to be conducted by allocating the base shear according to stiffness of each system. This means, to design the structure based on the stiffness-based procedure rather than strength-based procedure. The stiffness-based method incorporates the following steps:
1) The two sub-systems are designed for the predefined portion of base shear.
2) The two sub-system members are tuned to reach equal story drift.

The tuning procedure is a try and error approach, however considering the decoupled factors of strength and stiffness of BRBs it is easily accomplished. The strength of a BRB member can be adjusted by cross section and its stiffness can be tuned by changing the yielding length. Reducing the yielding length is an effective way to reach higher stiffness in BRBs. The design procedure of reduced length BRBs is discussed by Razavi et al. (Razavi et al., 2011)

New building codes for seismic design are adopting a performance based design aspect. The goal of a performance-based design procedure is to initiate structures that have predictable seismic performance under multiple levels of earthquake intensity. In order to do so, it is important that the behavior of the structures is targeted in advance, both in the elastic as well as the inelastic ranges of deformation. Consequently, the determination of member strength hierarchy, failure mechanism, and structure strength become the primary elements of a performance-based design procedure (Leelataviwat et al., 1999). The stiffness based design which has been developed in this article is an effort to set force based design aside and satisfying the objectives of performance based design.

This paper presents the results of parametric studies conducted to investigate the potential benefits of using BRBFs and SMRFs in a dual system. 2 types of structures in the height (7 story and 12 story) have been considered. Each type includes 4 structures. To compare the difference between different proportion of stiffness of the BRBFs and SMRFs in dual systems, 3 dual system structures with different proportion of stiffness of the BRBFs and SMRFs systems have been assessed and compared with a bare Buckling-Restrained Braced Frame. The performance of the structures is analyzed by conducting pushover and nonlinear time history analysis.

2. DESIGN AND MODELING OF DUAL SYSTEM

All the structures have been modelled by PERFORM-3D. A benchmark has been used for 7 story structures. The benchmark structure is seven-story office building example designed by Structural steel educational council in a Steel TIPS report titled as “Seismic Design of Buckling-Restrained

Braced Frames” (Sabelli and Lopez, 2004). For the 12 story building the same plan with the same story heights has been used. The use of the same building model is intended to provide a point of reference for comparison of different systems.

To compare the different proportions of stiffness of the BRBF and SMRF systems and a bare BRBF, 3 different proportion of stiffness have been considered. The bare BRBF building has simple connections in all frames except for the braced bays of BRB frames. The three models other than bare BRBF model are: 75%-25%, 65%-35% and 55%-45% structures, which stiffness proportions of BRBF is 75%,65% and 55% respectively. In the dual systems the other frames have been changed to SMRF.

The loading parameters are described in table 2.1. The detailed loading data can be found in Steel TIPS report titled as "Seismic Design of Special Concentrically Braced Steel Frames". (Sabelli and Lopez, 2004)

Table 2.1. Loading parameters

Dead Loads:		Live Loads:
Roof Loading	59 psf	20 psf
Floor Loading	87 psf	50 psf

Buckling Restrained braces follow isotropic hardening rule due to the friction between the core segment and restraining member in high inelastic cycles. In this rule the yield force is updated after yielding in a given direction. This means that the tension (positive) yield force is updated when the brace is in compression and its incremental deformation changes from negative to positive. As a result, the expression for isotropic hardening of the positive yield force is controlled by the cumulative plastic deformation and the maximum negative deformation. (Fahnestock et al., 2003)

To calibrate PERFORM-3D parameters for the Buckling-Restrained Braces, the results of an experimental test have been used (Eryasar and Topkaya, 2009).

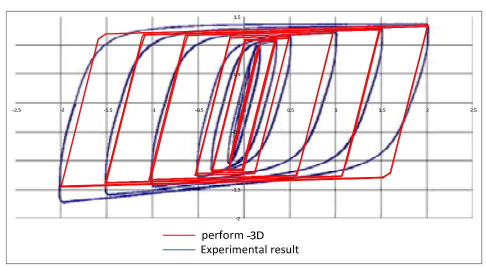


Figure 2. Result of calibrated parameters of PERFORM-3D

A single bay single story has been modeled and the parameters of BRB compound element are tuned so that the results of experimental test and analytical analysis became similar. The calibrated parameters of a single brace have been used in order to model the full-scale structure in PERFORM-3D based on the permission of AISC 341-10 (AISC, 2010).

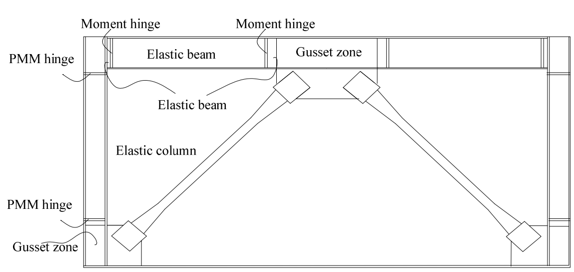


Figure 3. Modelling assumption for BRBF

For the braced bays the elastic beam and columns has been used to model the structure with the concentrated hinges, as shown in the Fig.3.

For other beam and columns FEMA beam and FEMA column has been used in PERFORM-3D. The whole structure is modeled 3 dimensionally in the PERFORM-3D.

In designing 75%-25% structure with 12 stories, braces of the 12th story did not satisfy the equal drift criterion, because the difference behaviour between 2 lateral systems is more prevailing in this type of proportioning and it was not possible to equalize 2 systems drift, so this story was excluded from this rule.

3. STRUCTURAL PERFORMANCE ASSESSMENT

The modal analysis shows that first mode period of the structure changes slightly in comparison to bare BRBF.

Table 3.1. First mode periods

Model	7 story structures first mode period (sec.)	12 story structures first mode period (sec.)
BRBF structure	0.948	1.200
75%-25% structure	0.996	1.238
65%-35% structure	1.002	1.246
55%-45% structure	1.002	1.219

3.1. Nonlinear Pushover

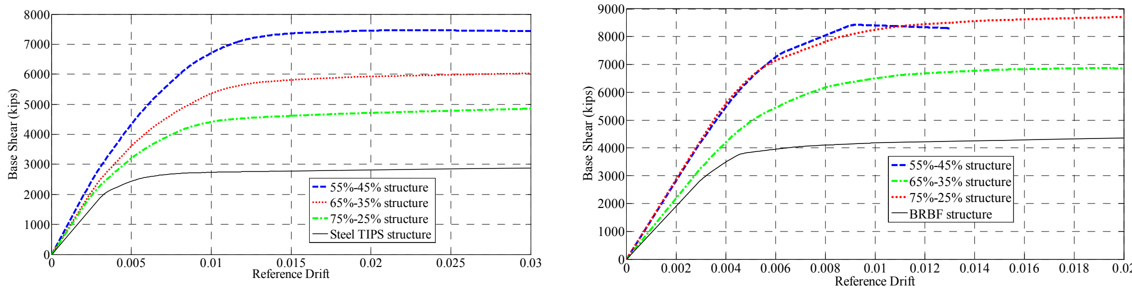
For each model, the target drift was calculated based on the pushover analyses suggested in FEMA 356 (ASCE, 2000). Recent research has shown that multiple load patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first mode shape is recommended (ASCE, 2007). For the design earthquake spectrum, Steel TIPS spectrum has been used. Based on the nonlinear static procedure the target displacements of the roofs as reference point of structures are reported in table 3.2.

Table 3.2. Target displacements according to FEMA356 for BSE-1 Hazard level

Model	7 story structures target displacements (ft)	12 story structures target displacements (ft)
BRBF structure	0.65	0.59
75%-25% structure	0.6	0.52
65%-35% structure	0.59	0.53
55%-45% structure	0.54	0.44

Results of pushover analysis have been shown in fig.9 to fig.13. The results of nonlinear static analysis show that all the components in 7 story structures satisfy FEMA 356 acceptance criteria for LS and CP performance levels at the target displacement in the BSE-1 level earthquake.

For 12 story structures, all the braces of all structures satisfy FEMA 356 acceptance criteria for LS and CP performance levels at the target displacement in the BSE-1 level earthquake. The capacity curves of structures are shown in Fig. 4. In the case of 7 story structures all the BRBs experience yielding.



(a) (b)

Figure 4. Pushover curves, (a) 7 story structures, (b) 12 story structures

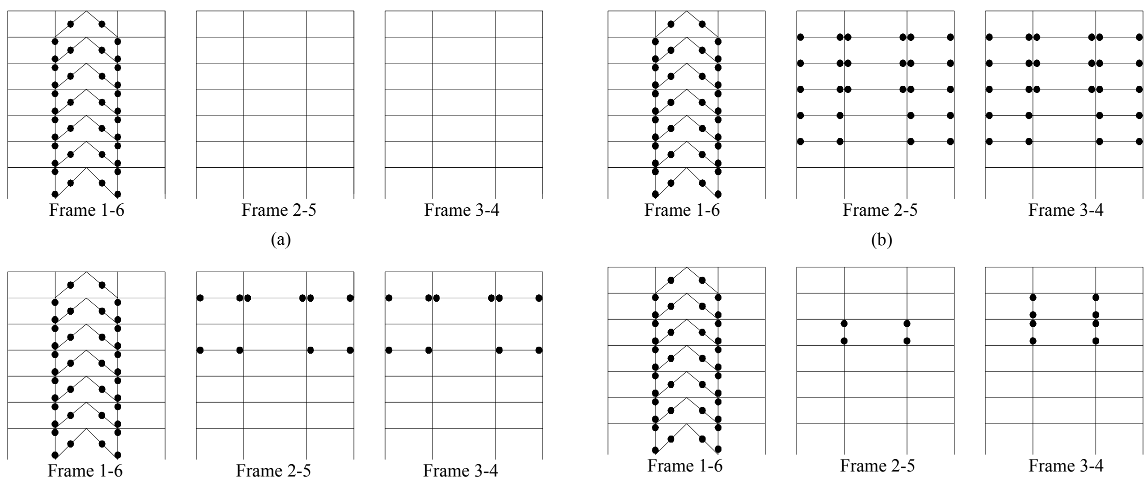


Figure 5. Plastic hinge formation at 7 story structures, (a) Steel TIPS structure, (b) 75%-25% structure, (c) 65%-35% structure, (d) 55%-45% structure

As shown in the figure 5 in addition to braces nearly all the columns of braced span have yielded in steel TIPS structure, but the deformations of the hinges are too little and the whole structure is in LS performance level.

In the case of 75%-25% structure, all of the BRBS of structure experience yielding. At higher displacements beams of the SMRF yield. Similar to the discussed structures nearly all the columns of braced spans have yielded but the deformations of these hinges are little and the whole structure is in LS performance level.

65%-35% structure is same in the mechanism with the previous structure but in comparison to 75%-25% structure, the beams of SMRF in 65%-35% structure experiences less yielding. Other beams that have not yielded are near the yielding threshold.

In the case of 75%-25% structure, all of the BRBS of structure experience yielding. In higher displacements columns of the SMRF yield. Similar to the previous structures nearly all the columns of braced span have yielded but the deformations of this hinges are too little and the whole structure is in LS performance level and none of the beams experience yielding.

As depicted in the Fig. 6 in addition to braces of 5th story up to 12th story that have experienced yielding, columns of braced span in the 6th story up to 11th story have yield in the BRBF structure, but the deformations of this hinges except 8th story columns are too little and these columns are in LS performance level, but the columns of 8^{th} story in the braced span exceed LS performance level thresholds.

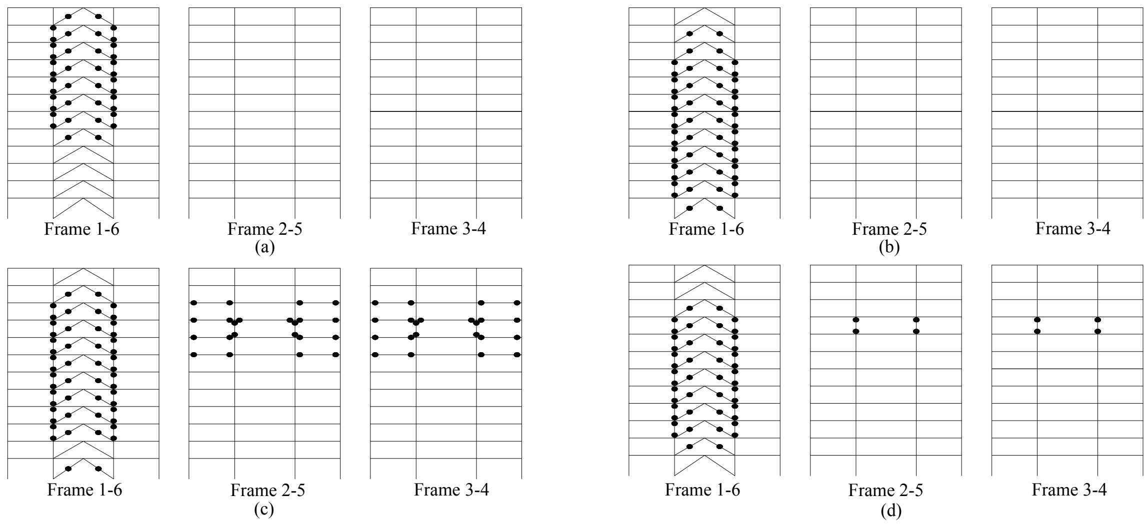


Figure 6. Plastic hinge formation at 12 story structures, (a) BRBF structure, (b) 75%-25% structure, (c) 65%-35% structure, (d) 55%-45% structure

In the case of 75%-25% structure the BRBs of the structures except the BRBs of 12^{th} story experience yielding. In further displacement columns of braced span in the second story up to 9^{th} story have yielded. But deformations of this hinges are too little and these columns are in LS performance level and none of the SMRF components experience yielding.

In the case of 65%-35% structure the BRBs of the structures except the BRBs of second and 12^{th} story experience yielding. But it should be noted that braces of second story are near the plasticization boundary. In further displacement columns of braced span in the third story up to 10^{th} story and some of the columns of 9^{th} story in the SMRF have yield. But deformations of this hinges are too little and these columns are in LS performance level and in this structure beams of 7^{th} story up to 10^{th} story have yield.

In the case of 55%-45% structure the BRBs of the second story up to 10^{th} story experience yielding. At further displacement columns of braced span in the third story up to 10^{th} story and some of the columns of 9^{th} story in the SMRF yield. But deformations of this hinges are little and these columns are in LS performance level and none of the beams of SMRF experience yielding.

3.2. Nonlinear Time History Analyses

The ground motions which have been used in this study were selected from Pacific Earthquake Engineering Center strong motion data base (PEER). 7 records were chosen and scaled to the DBE seismic level.

These records have been selected according their characteristics such as fault rupture mechanism and site classification to be similar to the structure site which has been mentioned previously. These records were assembled and scaled to match the design response spectrum. These ground motions correspond to site class D in ASCE 7-10.

For scaling, geometric-mean scaling method has been used. This method involves amplitude scaling a pair of seed motions by a single factor to minimize the sum of the squared errors between the target spectral values and the geometric mean (square root of the product, hereafter termed geomean) of the spectral ordinates for the pair at appropriate periods. This scaling procedure seeks to preserve the record-to-record dispersion of spectral ordinates and the spectral shapes of the seed ground motions(Constantinou et al., 2011).

To meet the acceptable criteria of ASCE 7-10, the average of SRSS spectra from all horizontal component pairs was multiplied by a single scale factor so that it does not fall below the corresponding ordinate of the response spectrum used in the design in the period range of *0.2T* to *1.5T*.

Table 3.3. Ground motions

ground motion number	Record ID	earthquake	Station	M
1	P0163	Imperial Valley	5053 Calexico Fire Station	6.5
2	P0006	Imperial Valley	117 El Centro Array #9	7
3	P0881	Landers	22074 Yermo Fire Station	7.3
4	P0452	Morgan Hill	57382 Gilroy Array #4	6.2
5	P0730	Superstitn Hills(B)	11369 Westmorland Fire Sta	6.7
6	P0725	Superstitn Hills(B)	01335 El Centro Imp. Co. Cent	6.7
7	P0319	Westmorland	5169 Westmorland Fire Sta	5.8

The direction of records to structure has been chosen in a way that the component with maximum PGA is directed to Y local coordinate of the structure.

By conducting nonlinear time history analysis, it is inferred that all the braces of 65%-35% structure, 55%-45% structure and steel TIPS structure remained at LS performance level in the case of 7 story structures, but in 75%-25% structure, braces of 7th story do not satisfy LS performance level when they exposed to Morgan Hill record.

In Steel Tips structure, columns of braced spans in the y direction in third story exceed LS performance thresholds when they exposed to Landers record.

In the case of 12 story structures, all braces of 65%-35% structure and 55%-45% structure remained at LS performance level, but in 75%-25% structure and bare BRBF structure, some braces do not satisfy LS and CP performance level, these deficiencies are listed in table 3.4.

Table 3.4. Some deficiencies of structures in 75%-25% structure

ground motion number	Structure	LS performance level	CP performance level
1	75%-25% structure	braces of 11 th story	-
2	75%-25% structure	braces of 11 th story	-
6	75%-25% structure	braces of 11 th story	-
5	BRBF structure	braces of 9-10-11 th story Columns of braced span in 9th story	braces of 10 th story
6	BRBF structure	braces of 9-10 th story Columns of braced span in 9th story	-

Results of nonlinear dynamic analysis are depicted in Fig. 7-10.

As shown in the Fig. 7, there are large residual drift in some stories of the Steel TIPS structure , 75%-25% structure and 55%-45% structure therefore it causes distinct local damage in this stories.

It is concluded from the Fig. 8 that in the case of 65%-35% structure equal plastic deformation exists in the height of structure in comparison to other structures which is favorable for the designer. There is extremely large amount of residual drift in the Steel TIPS structure which has only bare BRBF to withstand to lateral loads.

In mid-rise building like these structures, amount of maximum drift is not dispersed and they are approximately even.

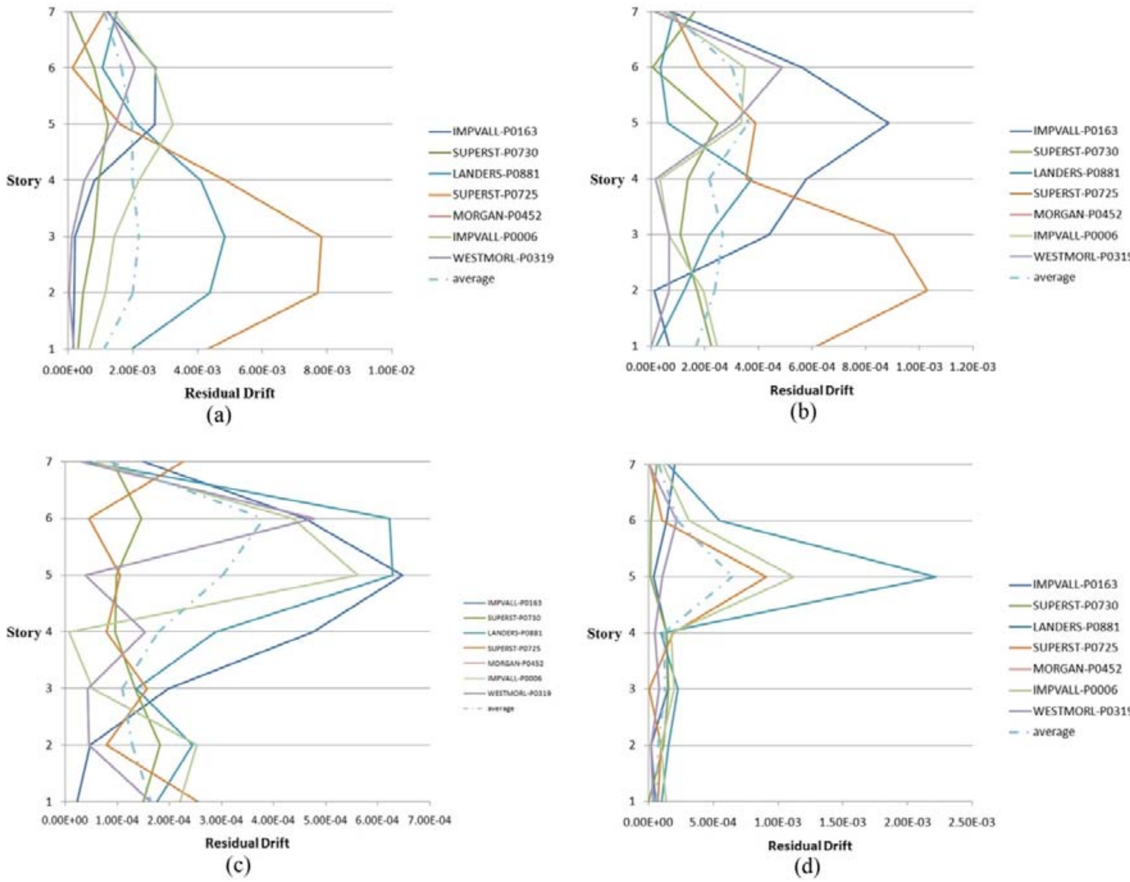


Figure 7. Residual drift vs. story; 7 story structures, (a) Steel TIPS structure, (b) 75%-25% structure, (c) 65%-35% structure, (d) 55%-45% structure

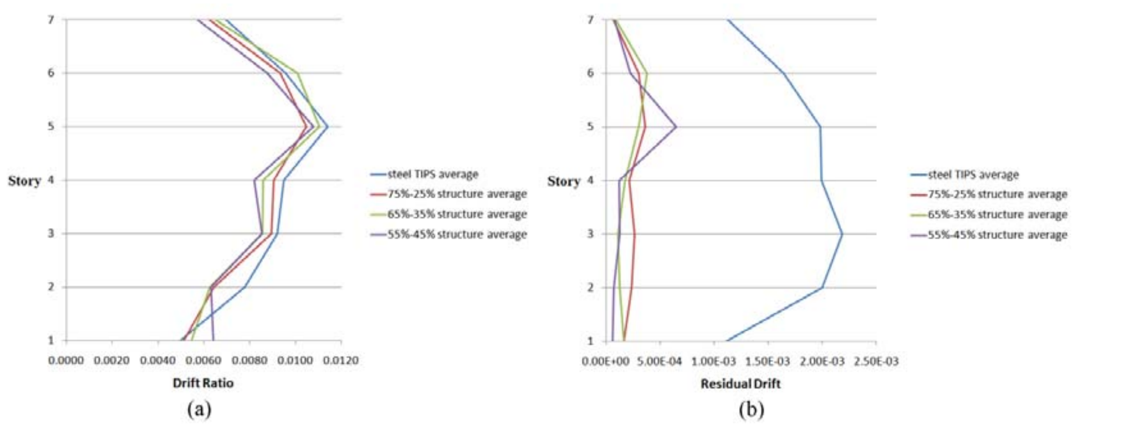


Figure 8. (a) Average max drift ratio vs. story, (b) Average residual drift vs. story; (7 story structures)

In 12 story structures, some stories of the BRBF structure, 75%-25% structure and 55%-45% structure residual drifts exceed 0.003 rad, but the maximum residual drift in 65%-35% structure reached 0.002 rad.

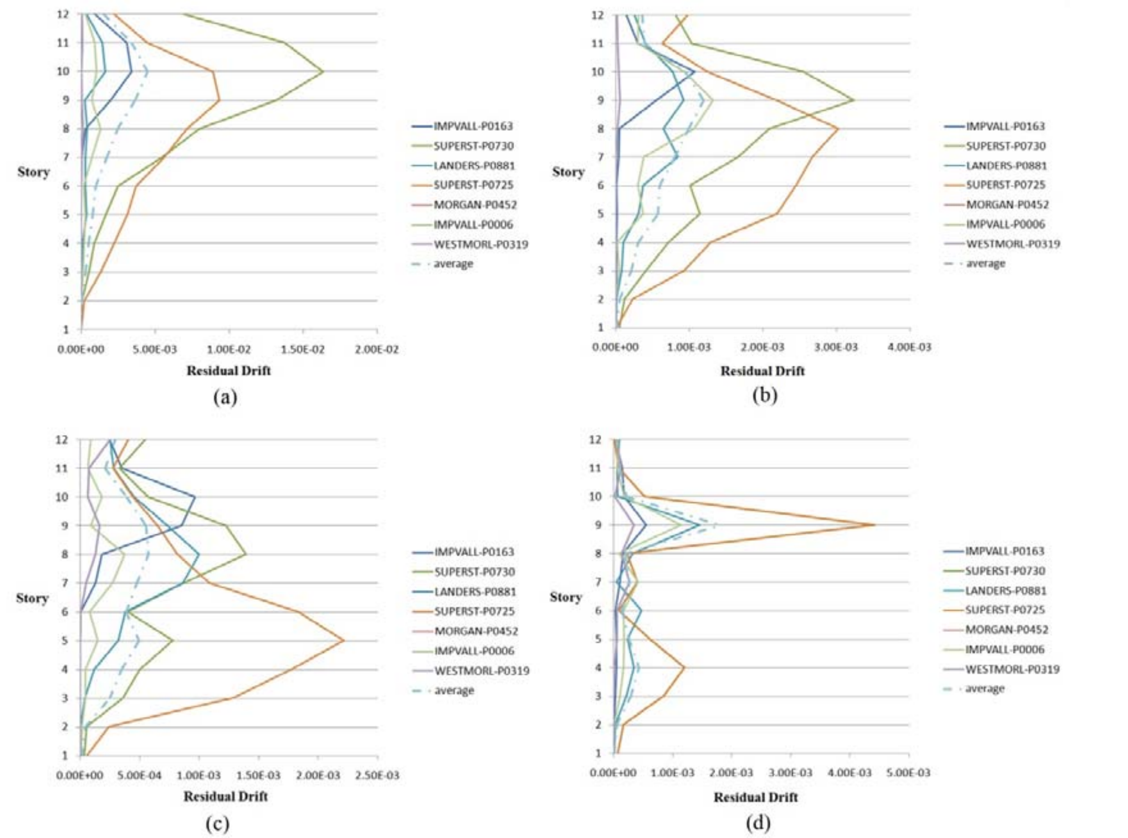


Figure 9. Residual drift vs. story; 12 story structures, (a) BRBF structure, (b) 75%-25% structure, (c) 65%-35% structure, (d) 55%-45% structure

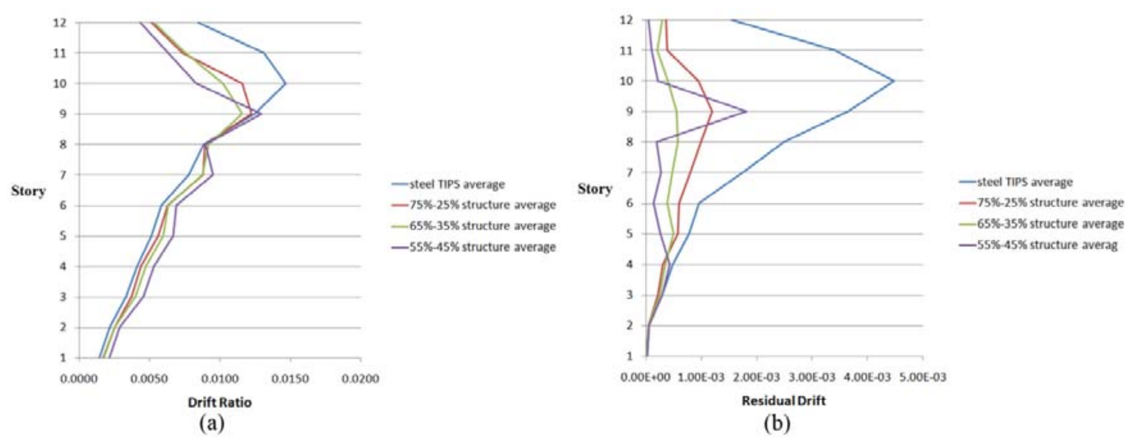


Figure 10. (a) Average max drift ratio vs. Story, (b) Average residual drift vs. story; (12 story structures)

In high-rise buildings like these structures, amount of maximum drift is less in the higher stories which may be more pleasant from serviceability point of view.

The pushover analysis showed that for 12 story-BRBF structure, local damage exists in braces and columns of the tenth story. As shown in the figures and tables, the results of nonlinear dynamic analyses are consistent with the results of nonlinear static analyses in terms of local large residual drift at this zone.

Fig. 10 depicts that amount of residual drift declined distinctly in dual systems. As mentioned previously amount of residual drift influences the performance of the structure and influences directly serviceability of structure after an earthquake. In 65%-35% structure there is same amount of plastic deformation which exists in the height of structure in comparison to other structures.

4. CONCLUSIONS

In this study the performance of 4 mid-rise and 4 high-rise has been assessed by conducting nonlinear static and time history analyses. These models are designed according to stiffness-based procedure presented in this research. The results of this study show the 65%-35% model has a better seismic behavior in both mid-rise and high-rise buildings. All of the structural elements remained in LS performance level and secondary system has been mobilized at higher levels. Moreover this method of proportioning causes less residual drifts in the height of structure. Analyses demonstrate that adding the SMRF to the BRBF system changes the natural period slightly.
Based on the findings of this research it is crucially suggested to perform nonlinear analyses for predicting deficiencies related to inherent differences of behavior of 2 systems, since these deficiencies cannot be predicted and recognized in linear static analysis.

REFERENCES

AISC 2010. Seismic Provisions for Structural Steel Buildings. *ANSI/AISC 341-10.* Chicago, Illinois: American Institue of Steel Construction.

ARIYARATANA, C. & FAHNESTOCK, L. A. 2009. Performance assessment of buckling-restrained braced frame dual systems. *In:* SAUSE, R., MAZZOLANI, F. M. & RICLES, J. M. (eds.) *Behaviour of Steel Structures in Seismic Areas.* CRC Press.

ASCE 2000. Prestandard and Commentary for the Seismic Rehabilition of Buildings. *FEMA 356* American Society of Civil Engineers.

ASCE 2007. Seismic Rehabilitation of Existing Buildings. *ASCE41-06.* American Society of Civil Engineers.

ASCE 2010. Minimum Design Loads for Buildings and Other Structures. *ASCE07-10.* American Society of Civil Engineers

CONSTANTINOU, M., KALPAKIDIS, I., FILIATRAULT, A. & LAY, R. A. E. 2011. LRFD-Based Analysis and Design Procedures for Bridge Bearings and Seismic Isolators. *Technical Report MCEER-11-0004.*: University at Buffalo, State University of New York.

ERYASAR, M. E. & TOPKAYA, C. 2009. An experimental study on steel-encased buckling-restrained brace hysteretic dampers. *Earthquake Engineering & Structural Dynamics*.

FAHNESTOCK, L. A., SAUSE, R. & RICLES, J. M. Year. Analytical and experimental studies on buckling restrained braced composite frames. *In*, 2003. 177-188.

KIGGINS, S. & UANG, C. M. 2006. Reducing residual drift of buckling-restrained braced frames as a dual system. *Engineering structures,* 28**,** 1525-1532.

LEELATAVIWAT, S., GOEL, S. C. & STOJADINOVIĆ, B. 1999. Toward performance-based seismic design of structures. *Earthquake Spectra,* 15**,** 435.

MEHDIPANAH, A., MIRGHADERI, S. R. & RAZAVI, S. A. Year. Seismic Assessment of Buckling Restrained Braced Frame Dual System with Special Moment Resisting Frame. *In:* Proceedings of the 4th International Conference on Seismic Retrofitting, 2012 Tabriz, Iran.

RAZAVI, S. A., SHEMSHADIAN, M. E., MIRGHADERI, S. R. & AHLEHAGH, S. 2011. Seismic design of buckling restrained braced frames with reduced core length. *The Structural Engineers World Congress* Italy.

SABELLI, R. & LOPEZ, W. 2004. Steel Tips: Design of Buckling-Restrained Braced Frames. *Structural Steel Educational Council*.