Journal of Civil Engineering and Urbanism
Volume 4, Issue 5: 534-539 (2014) ISSN-2252-0430

Comparison of Average Strength Steel Moment Frame with a Thin Plate Steel Shear Wall and Diverging Braced Design Method Based on Performance Levels

Vahid Chegeni [1], Mohammad Reza Baradaran[2]*

[1]*Department of Civil Engineering, Arak Branch, Islamic Azad University, Arak, Iran*
[2]*Young Researchers and Elite Club, Zarghan Branch, Islamic Azad University, Zarghan, Iran*

*Corresponding author's E-mail: info@mrbr.ir; mohamadrezabaradaran@gmail.com

ABSTRACT: In this paper a comparison was done between the coefficient of behaviour of steel moment frame systems with thin steel shear wall and diverging braced design method based on performance levels. 20 different frames was used for modelling and numerical analysis by SAP2000 software to calculating the capacity curve, coefficient of behaviour, energy dissipation and point of performance using the spectral capacity. For analysis, loading, determination of joints, levels of performance and etc. of frames, UBC, ATC-4, FEMA 356, Iranian 2800, Iran's 519 code of practice was used. In general, it can be concluded from this modelling that shear wall systems have much higher energy absorbance capacity but lower ductility than the divergent braced systems in all short, medium, and long buildings. The studies conducted on spectral capacity diagrams shown that the steel shear walls had better performance than divergent braced walls.

ORIGINAL ARTICLE
Received 17 Jul. 2014
Accepted 15 Sep. 2014

Keywords: Coefficient of behaviour, Thin plate steel shear walls, Divergent brace.

INTRODUCTION

Iran is among the countries that suffered great financial losses and casualties due to frequent earthquakes. Therefore, earthquake resistant systems seem to be absolutely necessary. The steel moment frame systems are seemed optimal due to their suitable ductility and the ability to dissipate the earthquake energy. The main problem in this system is lateral displacement and lack of enough toughness. To solve this problem, the use of twofold systems consisting of steel moment frames and more robust system that actually complements the system and fixing the problem of displacement of frame has emerged. Complementary system of moment frames in double systems of Iran is composed of convergent and divergent braced.

In recent years, many countries use a new system called thin steel shear walls which is a complementary moment frame systems. The new system is well-received due to its fast implementation and economic effectiveness. But in our country, due to lack of knowledge and lack of attention in the regulations of the country in comparison to other countries, it is less used. In this study, we will compare the performance of the new method which is a novel and effective design method based on the nonlinear behaviour of structures and complementary systems for medium moment frames, that is thin plate steel shear wall and divergent braced. For comparison purposes, some frames with different height and number of openings is considered, and the point of performance of this frames is examined using spectral capacity. The average rate of energy dissipation and dual system of steel moment frames and thin plate steel shear walls is calculated (Ochoa, 1986), (International Institute of Seismology and Earthquake Engineering, 2002).

System description

This kind of systems are resistant against lateral loads, especially loads increased from 30 years ago; and have been used in construction of structures, strengthening and seismic activities in the old buildings. The positive trait of these systems for old building is the ability to servicing due to easy and comfortable implementation, saving the steel usage even 50%, lower cost. This system can be used in the steel structures and also concrete one. The steel shear walls with a thin steel layer are located between beams and columns. They are formed in different shape and dimension with or without opening (Figure 1 right). The whole system can be considered as a giant cantilevered beam, which column play the beam flange, steel walls are the soul of beam, or the role of beams is done by tough parts of its floor. To increase the resistance and preventing buckling of the steel plate, we can use horizontal and vertical stiffener plate to help its strength.

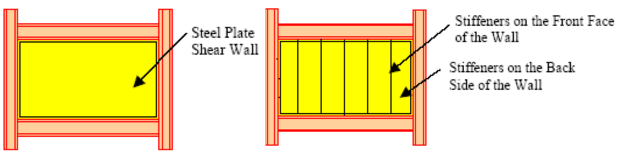

Figure 1. Steel Plate Shear Wall

Background of Discussion

Rahgozar et al. (2009), in their study entitled "Assessment of seismic vulnerability and retrofit of existing steel building with the dual construction system through nonlinear static analysis and spectral capacity", pointed out that in the third edition of 2800 code of practice, the use of steel moment frames with regular ductility was eliminated from twofold systems, and due to regulation change and construction of some important

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buildings in country based on these systems, of course with regular ductility and based on previous edition of standard 2800 (2nd edition), we can make some useful change by assessing their susceptibility and also their strengthening procedures based on new seismic rules and code of practice. (Ghasemi et al., 2011) in a study entitled "designing based on performance levels for seismic resistance using steel structures with steel shear walls" One of the top choices in the design and retrofit of structures which has attract structural experts attention in recent years is the use of thin-layer steel walls as bracing system with considerable energy loss and proper toughness. In the study of Heinz et al. (2010) entitled “Performance of steel frames and convergent braces” we examined the performance of this dual system of structural and discussed its economic advantages especially in areas of moderate seismic hazard.

Performance-based seismic design requirement

There is significant destruction in several earthquakes due to their inelastic behaviours. Since according to force-displacement curve, the structure passes from elastic domain to inelastic state due to seismic trembles and there are weak resistance here, so, the softening changes govern in this situation, which correspond with greater damage. In the performance - based design approach, a nonlinear function of structural elements is assessed, so, we can obtain a more realistic behaviour of structures, compared to the prior occurrence of a seismic event of certain strength. Perhaps the most important reason for seismic design based on performance is to encourage the use of the initiative in developing methods to improve performance. In recent rules and regulations, there are not such procedures or encouragement. The reason is that new concepts are not applicable under such drained and closed statutes.

Nonlinear static analysis

The structural behaviour is reviewed by nonlinear dynamical analysis methods aside from elastic range. In this way, the past few accelerograms of earthquakes are used. Non-linear dynamic analysis is very complex and time consuming, and as a practical computational procedure cannot be used in engineering offices. The classic design of structures, structural safety by limiting stress at the material flow is achieved, but even moderate earthquakes may now be getting some of the structural elements. Therefore, to predict the performance of buildings against earthquakes, the need for nonlinear analysis methods are felt. In compliance with the provisions of the Building Regulations, it is expected that in mild and moderate earthquakes there are not any significant structural damage and resist severe earthquakes without collapse. To achieve this, engineers need information about the distribution of forces and deformations in structural members during an earthquake, which requires a nonlinear analysis and forecasts of plastic joints and the recognition of properties of collapse mode. Non-linear static analysis methods to assess the cumulative seismic structures, particularly in the area of nonlinear deformation are useful. Recently, ATC and FEMA regulations suggests analysis of nonlinear static (push over) for studying structural behaviour in the field of nonlinear behaviours, this simple method has high accuracy. Initial assumptions can be easily applied in the calculations.

Modelling the studied frames in SAP2000 software package

20 frames with dual intermediate moment systems and shear walls and thin diverging braces have been investigated. Frames 9 and 10 are shown in Figure 2. Frames belonging to the residential building with spans of 4 meters and a height of 3 meters. Buildings located at relatively high hazard area and soil type 2. Roofs are considered as ribbed slab, and live and dead loads were calculated as 600 and 200 kg per square meter, respectively; and double studs used for braced frames sections. All columns and beams and the wind breakers are IPE and IPB and each floor level for columns of type 12 and 12 storey frames are allocated for each of the three floors in height, and the frames are frames 8 and 4 for each floor type. Table 1 listed the details of the area of 12 sq.m and Shell thickness used for studied frame systems. Different elements with weights equal with braced frame weight are used in the walls.

Table 1. Characteristics of the sections used in the studied frame system

Number of Story	Story	Beam	Column	Brace
4	1,2	IPE200	IPB300	2UNP140
4	3,4	IPE180	IPB280	2UNP120
8	1,2	IPE240	IPB340	2UNP180
8	3,4	IPE220	IPB320	2UNP160
8	5,6	IPE200	IPB300	2UNP140
8	7,8	IPE180	IPB280	2UNP120
12	1,2,3	IPE240	IPB340	2UNP180
12	4,5,6	IPE220	IPB320	2UNP160
12	7,8,9	IPE200	IPB300	2UNP140
12	10,11,12	IPE180	IPB280	2UNP120
15	1,2,3	IPE270	IPB360	2UNP200
15	4,5,6	IPE240	IPB340	2UNP180
15	7,8,9	IPE220	IPB320	2UNP160
15	10,11,12	IPE200	IPB300	2UNP140
15	13,14,15	IPE180	IPB280	2UNP120

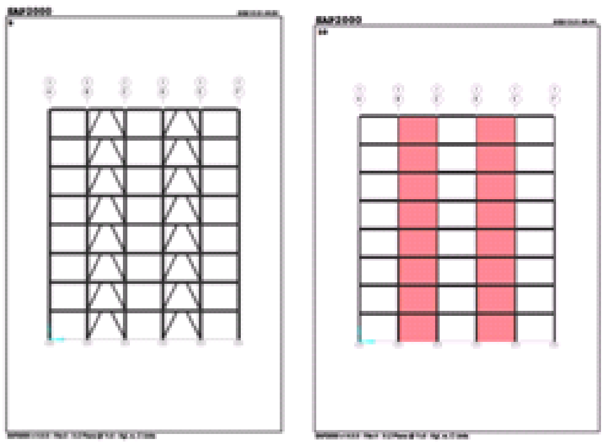

Figure 2. Investigated frames 9 and 10

Lateral reload

The incrementally increasing lateral loading is considered for the analysis of increasing over load, which have been loaded the same as the loads in Figure 3. It is an assumed kind of load that derived from loading code of

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practice, so that, the lateral loads is composed of wind and seismic load.

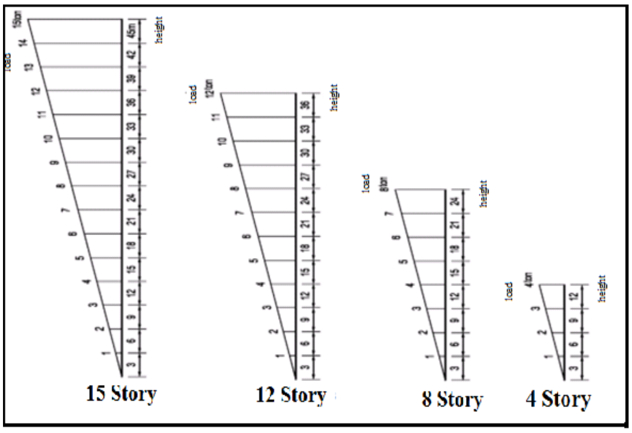

Figure 3. The method of loading the studied frames.

Examination of the results

The output of nonlinear analysis are includes stress loads and axial forces in shear walls, envelope curve which include displacement of the roof covering against applied lateral force and the capacity spectrum curve.

Tension and axial forces under loads on shear walls

Steel shear walls absorb lateral force and transmitted them to the ground via lateral elements (columns). Examples are given in the Figure 4 which uses frame 10 to show the stress and axial force applied and the rest of the shear walls also act as shear walls of this frame.

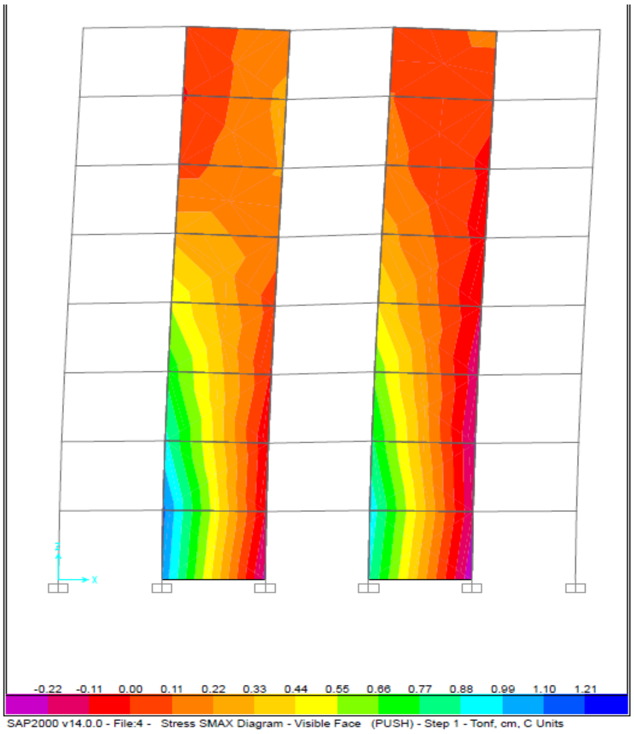

Figure 4. Diagram of maximum stress on the frame 10

Curve of base shear - roof displacement of

Shapes

The base shear based on displacement the control node curve is displayed in Figure 5 (the controller node of displacement which is defined at the time of introducing a nonlinear analysis). This curve could be used in many ways, such as calculation of received energy of structure through obtaining the chart surface and also calculating the ductility of structure, by dividing the maximum displacement load carried by the structure on maximum displacement in the elastic state of the structure. In the following curves, the hinge is in the elastic mode until the plastic mode is formed, and then entering a inelastic mode (plastic). It is observable that systems with higher resistance displace lesser under greater force and load.

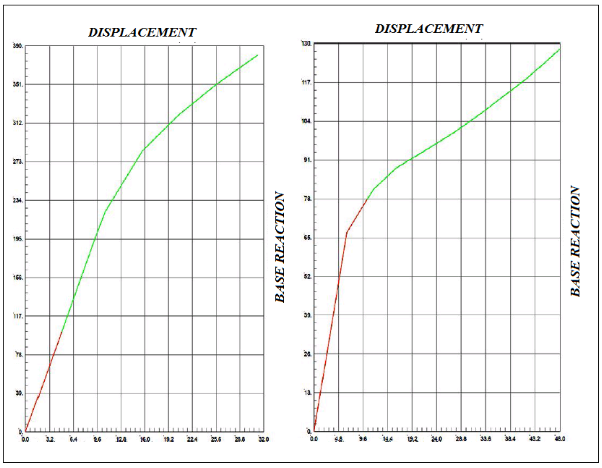

Figure 5. Power-displacement curve for frames No. 9 of 10

The spectral capacity curve

The capacity spectrum and system need curve for frames no 9 and 10 are displayed in figure 6. It is presented in a format called ADRS format. In this format, the

In this format, the spectral displacement (sd) and spectral acceleration (sa) are calculated from the standard curve and capacity needs with a series of transformation relations. Sap program calculates and reports the point of performance of the structure. These calculations are done based spectral requirements of UBC regulations.

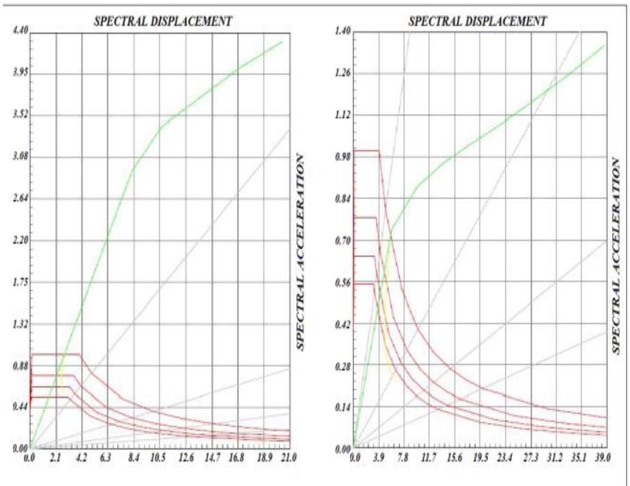

Figure 6. Spectral capacity curve of frames 9 & 10

Determination of performance point of frames

We can obtain the performance point of frames through using the intersection of capacity spectral curve and spectral requirement of code of practice. For example, from capacity spectrum curve of Frame 1 can be seen that the yield base shear and the displacement points of performance (V, D) are equal to 14.31 cm and 50.20 ton. It can be predicted that under the effect of earthquake generated standard spectrum of Regulations ATC-40, the maximum base shear and the expected shift equal to the above amounts.

Table 2. The same displacement and shear base as point of performance for studied frames (displacement based on Cm and base shear is based on Ton)

Base Shear (*ton*)	Displacement (*cm*)	Number of Frame
50.20	14.31	1
58.10	12.93	2
59.60	12.46	3
81.53	8.63	4
53.85	10.77	5
66.00	5.17	6
81.76	5.78	7
88.24	2.51	8
67.19	6.60	9
87.29	4.15	10
60.09	6.24	11
70.40	3.56	12
62.77	5.90	13
73.84	1.86	14
35.69	7.79	15
52.82	4.96	16
39.74	1.95	17
39.74	0.53	18
39.95	2.50	19
41.51	0.41	20

Calculating the frame's Coefficient of behaviour

We can calculate the structural coefficient of behaviour using structural capacity curve (base shear-displacement curve) and the spectrum of capacity. For example, we calculate the coefficient of behaviour of one frame, and then the coefficient of behaviour of the all other frames will obtain as such. The coefficients of behaviour for all frames are calculated in Table 3.

$$\Delta_{\max} = 90cm \qquad \textit{Ultimate Displacement}$$

$$\Delta_y = 28.84 \qquad \textit{Yielding Displacement}$$

$$V_P = 109.420\,ton \qquad \textit{Ultimate Shear}$$

$$V_y = 71.241\,ton \qquad \textit{Yielding Shear}$$

$$T = 1.0474\sec \qquad \textit{Time of Structure Period}$$

$$\Omega = \Omega_0 \times F_1 \times F_2 = \frac{V_y}{V_P} \times 1.05 \times 1.1$$

$$\Omega = 1.77 \qquad \textit{Coefficients of Increase Strenght}$$

$$\mu = \frac{\Delta_{\max}}{\Delta_y} = \frac{90}{28.84} = 3.12 \qquad \textit{Coefficients of Ductility}$$

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \qquad \textit{Coefficients of Decrease Ductility}$$

$$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2(\ln T - \frac{1}{5})^2\right] = 0.743 \;\textit{Sedimentary Earth}$$

$$R_\mu = \frac{2.85 - 1}{0.925} + 1 = 3.85$$

$$R_u = R_\mu \times \phi = 3.85 \times 1.77 = 6.81 \qquad \textit{Ultimate Stress}$$

$$R_w = R_u \times Y = 6.81 \times 1.4 = 9.53 \qquad \textit{Alloweable Stress}$$

In the above equation, Y is allowable stress factor which is usually about 1.4 to 1.7. In most of the regulations such as UBC code of practice, the coefficient of 1.4 is proposed.

Table 3. Coefficient of behaviour for all frames

Shear base	Displacement	Number of Frame
9.53	6.81	1
7.30	5.21	2
12.12	8.66	3
7.14	5.10	4
9.15	6.54	5
7.73	5.52	6
10.00	7.20	7
8.00	5.71	8
11.06	7.90	9
9.64	6.88	10
10.58	7.56	11
8.10	5.78	12
8.95	6.39	13
9.10	6.5	14
9.52	6.80	15
8.70	6.21	16
9.14	6.53	17
8.27	5.91	18
9.43	6.74	19
9.53	6.81	20

The rate of energy dissipation

As we have already known, work or energy is equal to the area under the load – displacement curve, in the seismic design of structures, the building that depreciate more energy before demolition is more ductile and structurally more popular and suitable. Steel moment frame systems are among systems that dissipate more energy of the earthquake. In this study, we investigated the capacity curve of the frame under study and calculate the rate of energy dissipation of these frames, and determine complement systems that will depreciate more energy. For better comparison between the twofold systems which can be seen in Figure 7, the column charts have been used.

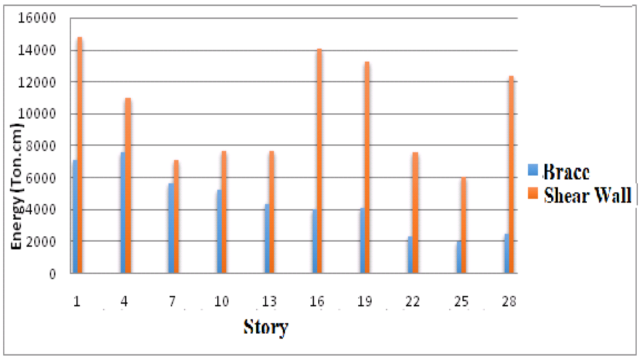

Figure 7. Energy dissipation graph

Curve of base shear - roof displacement as a group

For example, Figure 8 represent the curve of base shear based on displacement of control node (the node of displacement controller which defined in the time of introducing a nonlinear analysis state) for frames 9 and 10 (the frames that are equal in the number of openings and height).

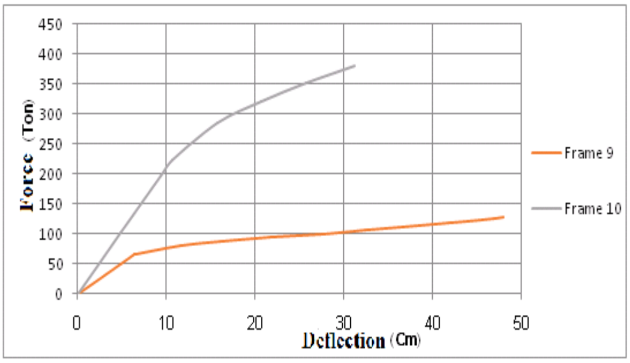

Figure 8. Force - displacement of frames 9 and 10

Comparison of the studied systems

1. With respect to the point of performance, using thin steel moment frames and shear walls for all buildings in the short, medium and long double system is better than the twofold moment frames and braces. For example, the frame number 7 (roof displacement = 5.78 cm and base shear=81.76 tons) and frame number 8 (roof displacement = 2.51 cm and base shear = 88.241 tons) shows that a system with shear walls can bear more force with lower displacement, which indicates much more resistant of this system in comparison with steel moment frame and divergent brace system at the time of the earthquake.

2. As it can be seen from Table 3, the coefficient of the behaviour and ductility of twofold moment frames and divergent braces system compared with dual systems of moment frames and shear walls are rather more in tall buildings, the coefficient of behaviour and ductility in frames with medium and low altitude is approximately the same. . In general, in all frames the coefficient of behaviour of both systems of twofold moment frames and divergent brace is higher than coefficients of behaviour specified in the 2800 code of practice for intermediate moment frame system and the braces. We can accurately studied the coefficients of behaviour of all systems by using results listed in the table 3, in general, the average coefficient for the behaviour of twofold systems with braces have been obtained as Rw=9.90 and the coefficient of the behaviour divergent of binary systems with shear walls was Rw=8.35.

3. Comparing the force-displacement curves which one sample is presented in Figure 8, shown that the dual systems of steel moment frames with shear walls attract more energy and less displacement compared to steel moment frames with divergent braces systems, it shows that this systems has a higher resistance for all frames.

4. The rate of energy dissipation in the twofold system of steel moment frames with shear walls and steel moment frame brace is mainly from the twofold steel moment frame and divergent braces. This lead to rupture of the plastic hinge region of the steel shear walls and in the other word, large uncertainty related to divergent braces.

5. with comparing the performance point (encounter of capacity spectrum with the need spectrum) it can be noted that dual systems of steel moment with maximum base shear and displacement ATC-40 of steel moment frame with shear walls can be generated by an earthquake in the range of standard regulations applicable to the twofold steel moment frame and braces systems.

6. With summation of achieved results, it seems that Iranian 2800 code of practice are more suitable for structures with medium and short stories and have higher coefficient of safety than multi-floor tall buildings. It must be noted that results of current study has been documented.

7. To change places with braces floor systems converge towards a system of shear walls (e.g., frames 3 and 4) is about 45 % higher.

8. The last floor displacement in systems with divergence bracing compared to shear wall systems (for example, frames 3 and 4) is about 45% higher.

CONCLUSIONS

1. Using systems with thin wall shear bearing capacity (energy absorption) of samples and requirement of smaller sections for beams and columns will be diverging compared to braces system, which makes steel structures lighter and more economical.

2. Using steel shear wall system has very effective role in reducing the relative displacement of the stories.

3. Generally considering a lateral load pattern based on code and regulations and in various structural systems and application of a coefficient as a behaviour coefficient for each type of structure system, regardless of the strength and ductility values cannot guarantee stability of this structure under possible earthquakes.

4. In many codes of practices, only linear elastic analysis would be used for estimating the maximum non-linear response of a structure, but the use of simplified analytical methods to estimate maximum inelastic response of structures during severe earthquakes is necessary and inevitable. Accordingly, in the conducted study, we tried to estimate the maximum inelastic response of structures, particularly the maximum need to change the location of lateral inelastic structures using the results of the linear elastic analysis.

5. By comparing these systems, we can conclude that the systems with shear wall will show a good toughness and ductility than divergent systems with braces.

6. By comparing the displacement and base shear, such as the performance point of the frames, it can be concluded that the dual system of steel moment frames with thin steel shear walls with a less displacement can bear the higher force of the earthquake.

7. First plastic hinge that made in most systems are located in the beams, so, it can be concluded that in the time of sever earthquake, the structure suffer less problem due to loss of one beam compared with breakage of a column.

8. According to Table 3, it can be concluded that the behaviour coefficients of all the frame contain divergent braces is higher compared with the behaviour coefficients cited in the code 2800 of regulations.

REFERENCES

Gasemi M R, Nejatiyadeh Y. (2011). Designing based on performance levels for seismic resistance using steel structures with steel shear walls. 6th International

Congress on Civil Engineering, Semnan University, Semnan, Iran.
Hines E, Jacob C. (2010). Eccentric braced frame system performance. Structures Congress 2010: pp 1332-1341.
Instructions seismic rehabilitation of existing buildings. (2006). Publication No. 360, Department of Technical Affairs, Technical matters and criteria developments office, and earthquake risk reduction measures.
International Institute of Seismology and Earthquake Engineering. (2002). instructions of seismic rehabilitation of existing buildings, technical standards drafting office, national Management and Planning Organization.
Ochoa A. (1986). Disposable Knee Bracing: Improvement in seismic design of steel frames. J structure eng, ASCE, 112(7), 1544 –1552.
Rahgozar R, Ghanbari H. (2009). Assessment of seismic vulnerability and retrofit of existing steel building with the dual construction system through nonlinear static analysis and spectral capacity. 8th International Congress on Civil Engineering, Shiraz University, Shiraz, Iran.
Planning and implementation of steel building. (2008). (10th topic), sixth edition, housing and urban development organization.
Sarvghadmoghdam A, Roshan R. (2000). The use of increasing non-linear static analysis in seismic assessment of buildings. The first academic-professional conference of Iranian Society of Civil Engineers.