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Evaluation of Seismic Behavior of Steel Shear Wall by Time History Analysis

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ABSTRACT

Reducing the vulnerability of buildings against earthquakes is one of the most important issues for engineers and public concern in the last decade. For designers, choosing the best option among the different lateral bracing systems that exist in terms of functional and economic is one of the most important issues in the development and or retrofitting of structures. In recent years, the steel shear wall has gradually acquired its place in the construction industry. However, much research has been done by researchers on this system; but so far, less has been done to analyze seismic behavior of this lateral bracing system At different heights. In this study, seismic behavior of steel shear wall in 3, 6 and 12-story steel structures was investigated by time history analysis method under 3 earthquake records including Northridge 1994, loma prieta 1989 and Imperial valley 1979. The results of this study indicate that the structures with steel shear wall At different heights has shown the desired performance in terms of seismic behavior including input energy, story drift, base shear and Roof center of mass displacement. The results in the 12-story structure were much better than 3 and 6-story structures.

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1. Introduction

In order to embrace the lateral forces, various types of lateral bracing systems are used while each of which has its own characteristics. Steel shear wall is one of the structural systems which has been significantly studied in recent years. The initial experiments of steel shear walls were conducted by Takahashi et al. in 1971. They found that this system is highly ductile and stiffeners affect the increase of energy absorption [1]. Timler and Kulak performed experiments on thin shear walls in the University of Alberta in Canada which revealed the ductile behavior of this system [2]. Roberts and Sabouri-Ghomi tested 16 steel shear panels under diagonal loading in the Wales University of England and showed that all the panels had adequate ductility for standing the large hysteresis loops. Yamanda tested two specimens of the one-story shear wall under cyclic loading in Kansai University of Japan and observed large deformations in buckled steel panels (with negligible reduction in the strength) [3]. Roberts and Sabouri-Ghomi [4] and Sabouri-Ghomi et al. [5] proposed the method of plate frame interaction (PFI) for analyzing the steel shear walls for different cases including with or without stiffeners, with or without openings and with the thin or thick plate. The behaviors of the plate and the frame are separately examined in this method and the interaction between them is considered. Caccesse et al. studied the effects of the plate thickness and the types of the beam to column connection on the steel shear wall using 6 laboratory specimens and observed that the failure mode of the specimens changes with the variation of the plate thickness. Moreover, in case of using a thin unstiffened plate as the steel wall the inelastic behavior begins with the yielding of the plate [6]. In 2007, Gholhaki tested two specimens of steel shear walls with clamped and simple supports for the beams and found that the effect of the beam to column connection type on the initial stiffness of walls can be ignored. The strength of the clamped specimen was 26 percent more than that of the simply supported specimen and the energy absorption of the clamped specimen was more than that of the simply supported specimen. The effect of the beam to column connection on the angle of the diagonal tension field was negligible [7]. In 1390, Darvishi et al. tested three specimens with panel width to height ratios lower than 1, equal to 1 and higher than 1 and found that increasing the column stiffness in the first case increases the structure total ductility factor and the overstrength factor. Increasing the column stiffness in the second case has no considerable effects on the ductility factor and the overstrength factor. In the third case, increasing the column stiffness reduces the ductility factor and the overstrength factor [8]. By the investigations performed by Alinia and Dastfan in 2005, 2006 and 2007 on the steel shear walls they found that the steel shear wall energy absorption depends upon the surrounding member stiffness [9–11]. Various experimental studies have been carried out in 2008 in the laboratory of the Korea Institute of Construction and Transportation Technology Evaluation and Planning with the aim of changing the bearing capacity of the shear walls constructed from steel plate and with different construction details [12]. In the same year, 4 specimens of shear walls retrofitted with central rectangular openings – soft steel was used in designing the plates – were tested in the structure laboratory of the Construction and Building Research Institute [13] and the experimental results were matched with the results of the numerical examinations. Both results showed that increasing the opening width reduces the stiffness and strength of the specimens. Besides, retrofitting the plate can slightly compensate the effect of stiffness and strength reduction [14]. In 2014, Kharazi et al.

proposed the modified plate frame interaction method. In this method, the effect of the overturning moment on the steel shear wall system response is considered in the load-displacement diagram [15]. Sabouri-Ghomi et al. investigated the effect of symmetric openings on the steel shear wall in 2011. There were two rectangular openings in the steel plate. The experiment results revealed the dependency of the panel stiffness and strength on the panel effective width. In a way that the panel stiffness and shear strength reduced by the increase of its width. The variation of other parameters such as opening height and also the distance between the openings had no considerable effects on the stiffness and the shear strength [14]. In 2011, Chen and Jhang assessed the effect of using the low yield point steel (LYP) on the designing of the steel shear wall and proved that limiting the plate width to thickness ratio to lower than 80 leads to better performance of the wall when using the LYP steel in designing a steel wall. Moreover, using the moment beam to column connection increases the system strength and the energy dissipation capacity by 28 and 18 percent, respectively, compared to the case of using shear connection [16]. Hosseinzadeh and Tehranizadeh carried out research about the steel shear wall with different numbers of the story and different width to height ratios in 2014. They realized that the plate failure occurs sooner than that of the peripheral frame in panels with the lower number of story. While the plate complete failure is postponed in the high number of story. Moreover, since the wall plates can only bear the tension forces and cannot bear the tension stresses the axial tension forces in the columns are more than the axial compression forces [17]. In 2015, Moradinejad et al. studied the effect of the steel shear wall position on the progressive collapse. The results obtained from the nonlinear static analysis revealed that positioning the steel shear wall at the corner of the plan leads to the better behavior of the structure against the progressive collapse. In addition, if the steel shear walls are installed inside the structure plan, the structure will be more unstable compared to cases of other wall positions and a more critical condition is created for the structure [18]. In 2015, Abadi et al. investigated the effect of the wall plate contribution to bearing the lateral loads and realized that if the plate contributes more to the bearing of the lateral loads, the thicknesses of the plate and the columns surrounding the wall increase and the structure will be less economic [19,20]. In the same year, the investigations performed by Abadi et al. on the steel shear wall with stiffeners revealed that the dimensions of the peripheral frame will be considerably decreased by considering the shear buckling stress to shear yield stress ratio (Cv) in the design. While the system ductility through the stories is reduced by increasing the ratio of the shear buckling stress to the steel plate yield stress [21,22].

Designing the beams and columns by the Seismic Provisions for Structural Steel Buildings of the USA (AISC 341) [23] and by the Design Guide of Steel Plate Shear Walls [24] depends on the forces imposed by the plate tension field. Increasing the steel plate thickness, in other words, increases the transferring force due to the plate tension field imposed on the boundary elements of the wall which leads to an increase of the column dimensions.

2. Problem statement and research goals

Selecting the best choice among different lateral bracing systems in terms of performance and economy is one of the most important issues for structure designers in developing and retrofitting structures. In this research, we have tried to better introduce the steel plate shear wall system which is strong against the lateral loads by assessing the seismic behavior of these systems. Therefore, the results of this research could be a suitable guide for structure designer engineers in selecting the suitable lateral bracing system.

3. Optimal design of the steel shear wall

In order to model the steel shear wall in ETABS and SAP software the strip model is used which is a suitable method for modeling the thin steel shear wall and is proposed by Thorburn [25]. A schematic view of this model is illustrated in figure 1.

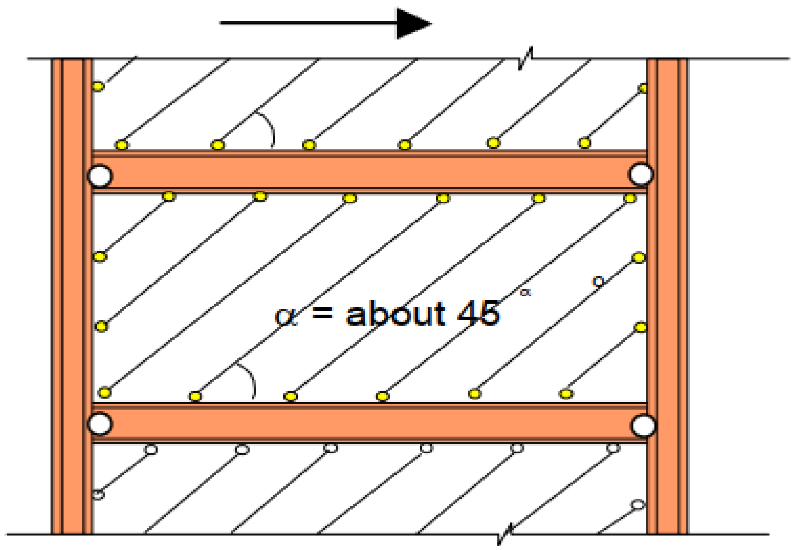

Fig. 1. Strip model.

In designing the steel shear wall strip model after making an initial assumption for the tension field angle some steps should be taken which are described as follow:

First step: specifying the initial plate thickness

In this step, first, the whole story section is assigned to the plate and the plate thickness is specified with the assumption of the tension field angle.

Second step: specifying the beam initial sections

Third step: specifying the column initial sections

Fourth step: specifying the tension field angle

The tension field angle which was firstly assumed can now be obtained based upon the obtained elements and one can modify the plate thickness using it.

Fifth step: modifying the beam and column initial sections

After modifying the steel plate thicknesses, the beam and column sections should also be modified. This helps us in reaching the answer sooner in the analysis and design step.

Sixth step: analyzing the steel shear wall system

In the first step, all the story section was assigned to the steel plate. The aim of this section is to specify the frame contribution to the story shear and finally, complete the beam and column design. Considering the frame contribution to the lateral load reduces the steel plate contribution and consequently reduces the plate thickness. The orthotropic model is used for the analysis. The analysis includes the following steps:

1. Controlling the plate strength for the plate contribution to the lateral load
2. Controlling the beam strength for bending forces caused by the gravity load, plate yielding and the column axial force
3. Controlling the column strength for forces caused by the gravity load, plate yielding and forces imposed by beam
4. Recalculating the tension field angle due to the change in the beam, column and plate characteristics

The design procedure based upon the USA code is presented in the following [26]:
For the initial design it is assumed that the plate bears the whole story shear. The tensile stress in the plate should be assumed.

$$V_n = 0.42 F_y t_w L_{cf} \sin(2\alpha) \tag{1}$$

In the LRFD method:

$$t_w \geq \frac{V_u}{\phi 0.42 F_y L_{cf} \sin(2\alpha)} \tag{2}$$

V_u ($V_n.\Phi$) is the required shear strength and Φ is the strength factor (0.9).

The initial stiffness needed for beams

$$I_c \geq 0.00307 \frac{t_w h^4}{L} \tag{3}$$

In some cases the moment of inertia required for the column is large and consequently it is difficult to choose a section.

The initial stiffness needed for the beams

It is recommended that the beam should have the minimum stiffness (to ensure the possibility of the plate complete failure):

$$I_{HBE} \geq 0.003 \frac{(\Delta t_w) L^4}{h} \tag{4}$$

l/h should be considered to be between 0.8 and 2.5 in steel shear walls.

$$0.8 \leq \frac{L}{h} \leq 2.5 \tag{5}$$

4. Validating the results obtained by modeling

The experimental results obtained by Sabouri-Ghomi and Driver et al. for the thin steel shear wall are used to validate the strip model in the SAP software.

For this purpose, first, the theoretical model proposed by Sabouri-Ghomi[27] which is illustrated in figure 2, is modeled in SAP software and then the obtained force-displacement diagram is compared with the diagram in the same reference and is illustrated in figure 3.

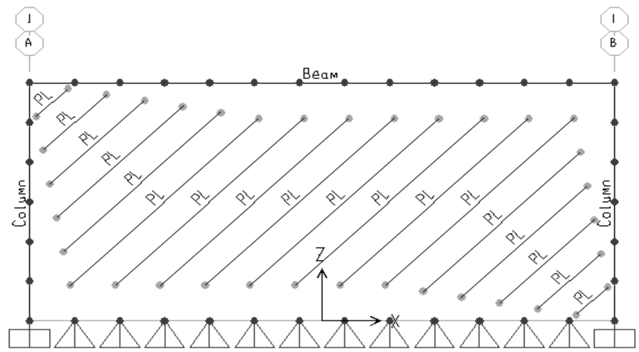

Fig. 2. Analyzed model of laboratory sample of Sabouri-Ghomi in software.

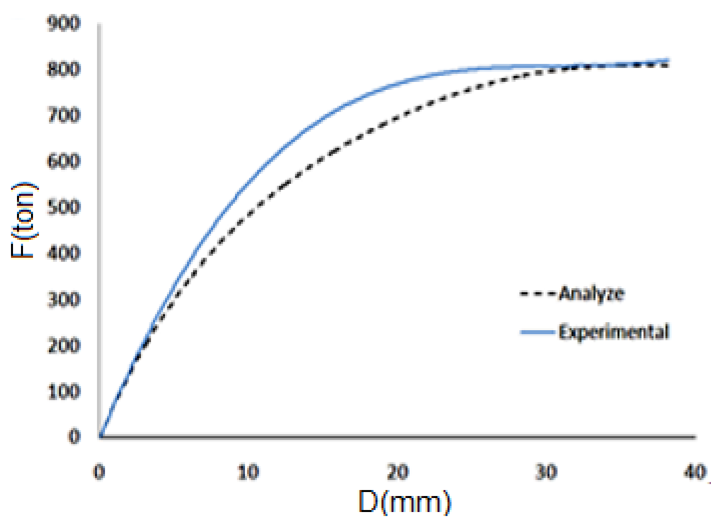

Fig. 3. Comparison of the force-displacement diagram of sample of Sabouri-Ghomi with Analyzed model in software.

According to figure 3, one can realize that the force-displacement diagram obtained by the designed model in the SAP software has good agreement with the force-displacement diagram obtained by the experimental model of Sabouri-Ghomi.

Driver et al. performed cyclic tests on the two three-story and four-story steel shear wall specimens in another experimental model [28]. Similar to figure 4, the first story of the four-story model is modeled by the software and the obtained force-displacement diagram is compared in figure 5 with the diagram they obtained during their tests. As it can be seen there is a good agreement between them. The specimen span is 3050 millimeters and the heights of the first story and the other stories are 1930 and 1830 millimeters, respectively. The beam with the section w21*55 is used in the last story in order to bear the tension forces created in that region and beams with smaller section of w12*40 are used for other stories, moreover, section w12*79 is used for columns and 4.8 millimeters thick plate is used as the infill for the first two stories and 3.4 millimeters thick plate is used as the infill for the last two stories.

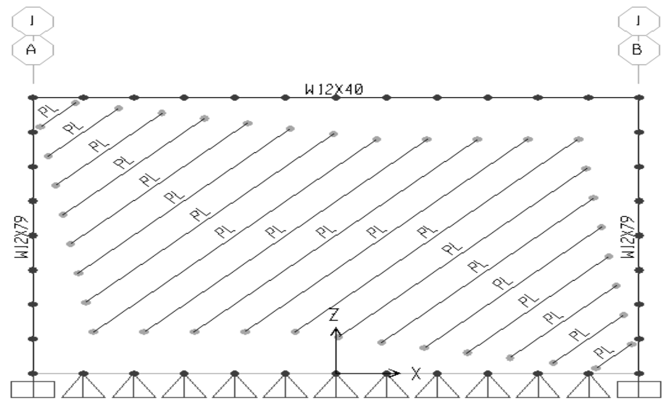

Fig. 4. Analyzed model of laboratory sample of Driver in software.

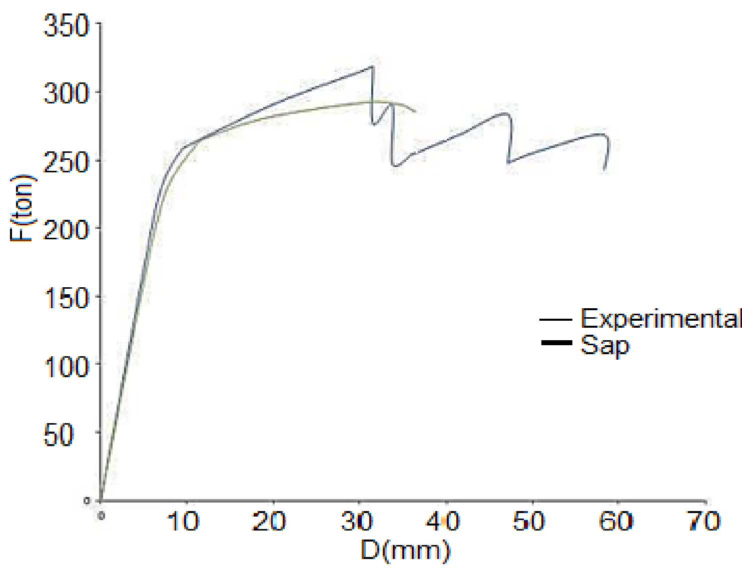

Fig. 5. Comparison of the force-displacement diagram of sample of Driver with Analyzed model in software.

5. The model introduction and the structure design

The models used in this research are 3, 6 and 12-story structures the story height in which is 3.5 and its occupancy is residential and the land soil is of type 3. The plan of these three structures is the same which is shown in figure 6. The sixth topic of the national code was used for loading this structure to specify the service loads [29] and the 2800 standard was used to calculate and distribute lateral loads [30] and ETABS 2015 and SAP2000 20 software was used for analyzing this structure and the USA AISCE code along with the tenth topic of the national building code [31] were used for designing with the method of LRFD.

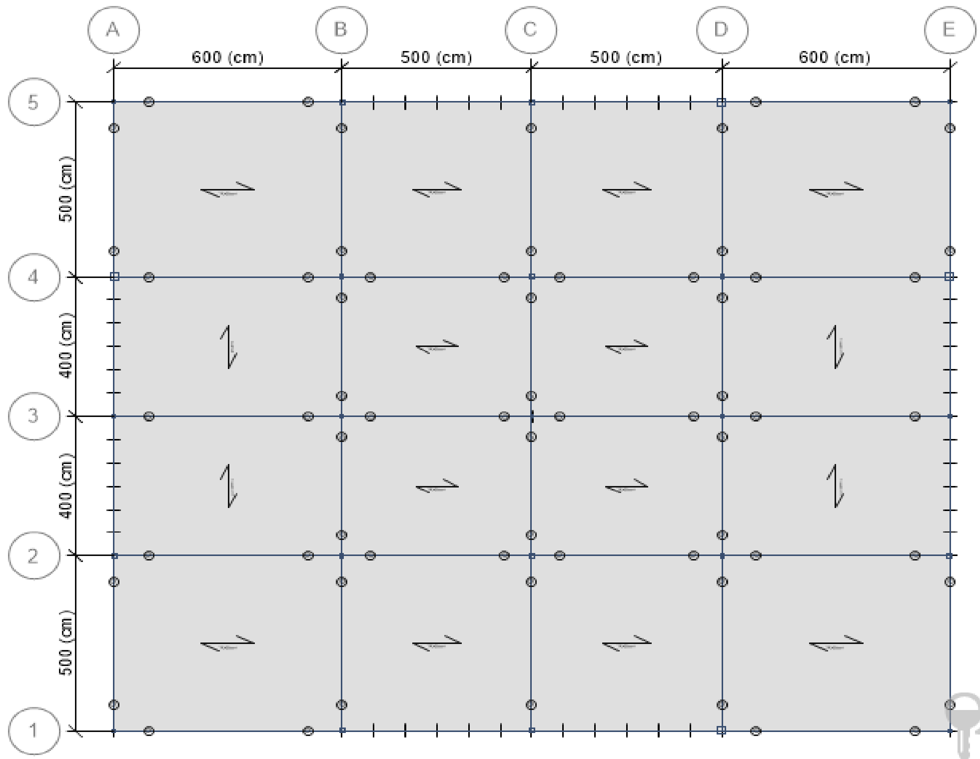

Fig. 6. Plan of structures.

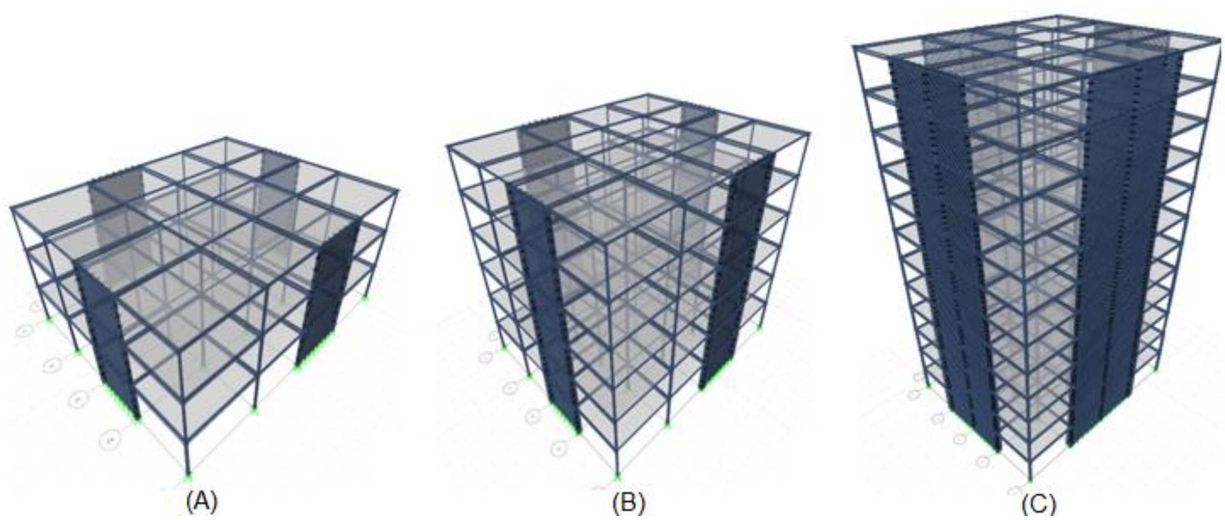

Fig. 7. 3D model of 3 (A), 6 (B) and 12-story (D) structures with Steel shear wall.

The linear static analysis was used in ETABS software to obtain the sections of the structure skeleton (figure 7). The ST37 steel with the ultimate strength of 370 megapascals was utilized for steel sections. The BOX section was used for the metallic columns, IPE section was used for beams and 3 to 7 millimeters plates were used for the steel walls.

The obtained sections are transferred to the software and after performing a nonlinear dynamic analysis the models seismic behavior was assessed using three earthquake records. Table 1 shows the informations of the earthquake records.

Table 1
The informations of the earthquake records.

Earthquake name	Station Name	Year	Magnitude	Mechanism	Distance from fault (Rjb) (km)	Distance from fault (Rrup) (km)
"Northridge"	"Ventura – Harbor & California"	1994	6.69	Reverse	54.28	58
"Loma Prieta"	"Dublin - Fire Station"	1989	6.93	Reverse Oblique	58.68	58.8
"Imperial Valley"	"Coachella Canal"	1979	6.53	strike slip	49.1	50.1

All of these accelerograms were obtained from the peer site and were scaled by fourth edition of the 2800 standard (buildings designing code againts earthquake). Figures 8 to 10 show the chart of the acceleration of these accelerograms.

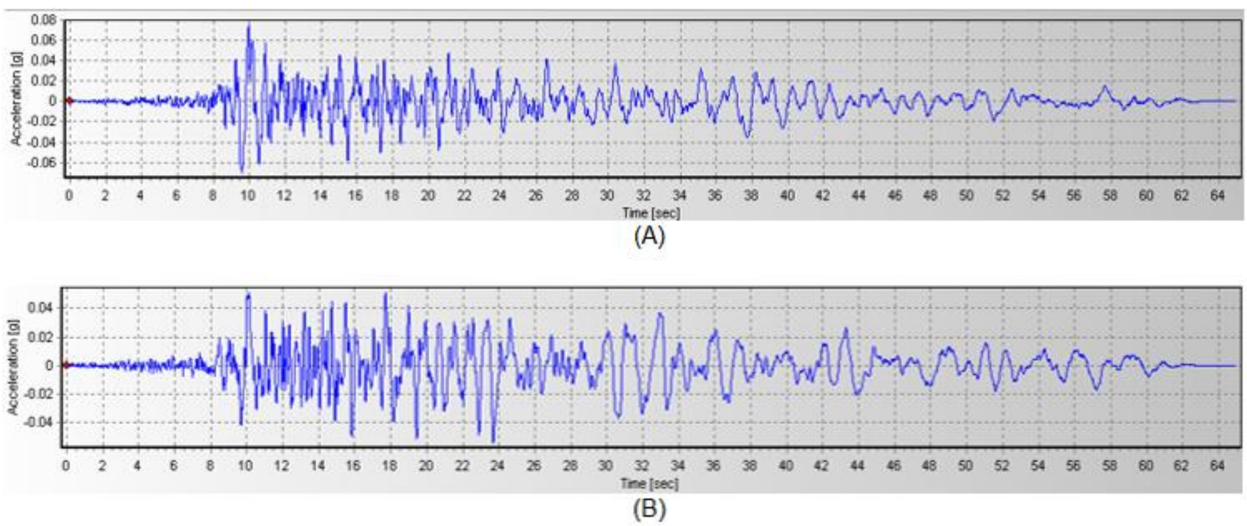

Fig. 8. Couple accelerograms of Northridge earthquake (A): X and (B): Y direction.

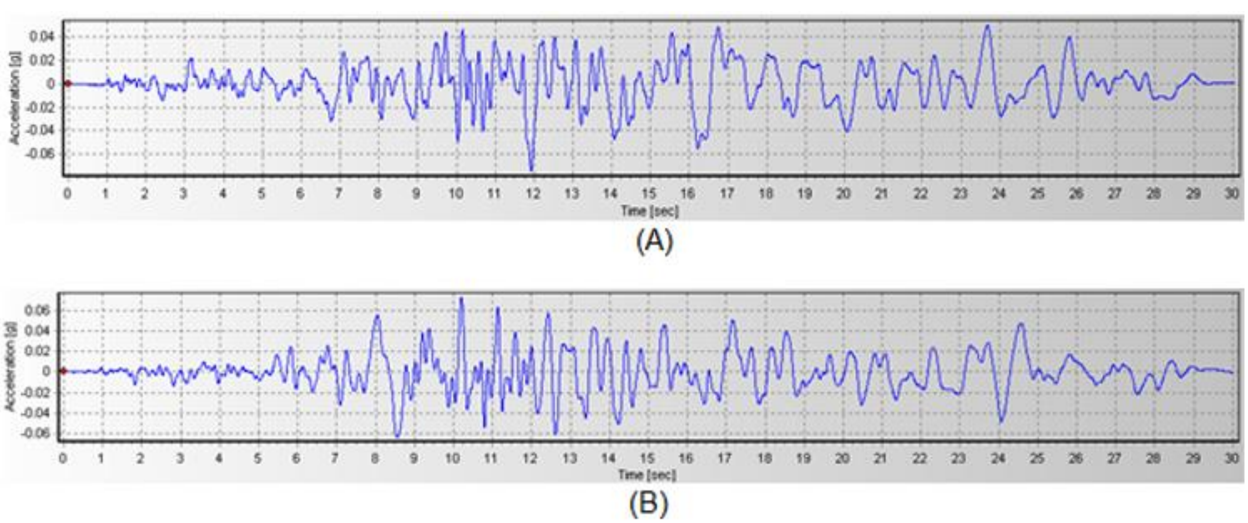

Fig. 9. Couple accelerograms of loma prieta earthquake (A): X and (B): Y direction.

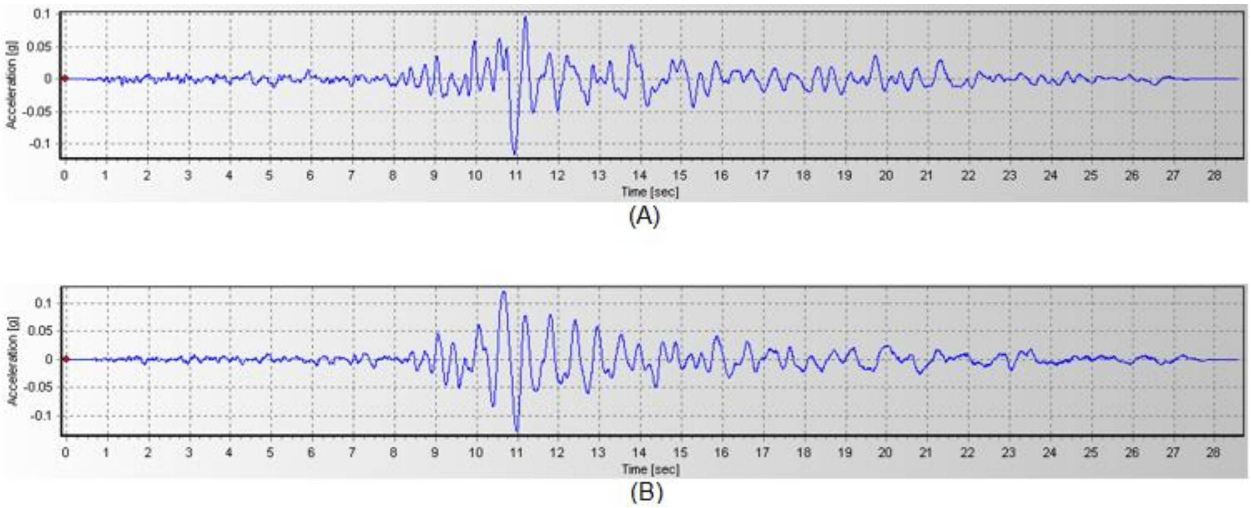

Fig. 10. Couple accelerograms of Imperial valley earthquake (A): X and (B): Y direction.

6. Evaluating the plastic hinges created in the model

The created plastic hinges are controlled and assessed after analyzing the model. The pink and blue hinges show the existence of a lateral safety level for residential buildings. When assessing the hinges created in the model having the steel shear wall it should be noted that the aim of this system is to sacrifice the shear wall for other structure members. So if the color of the plastic hinges in this system does not exceed the allowable limit; the system performance is still acceptable if the structure lateral displacement remains in the range bearable for the sacrifice members in order to prevent failure. Three samples of the plastic hinges created in the frames braced with the steel wall are presented in figure 11 in the following.

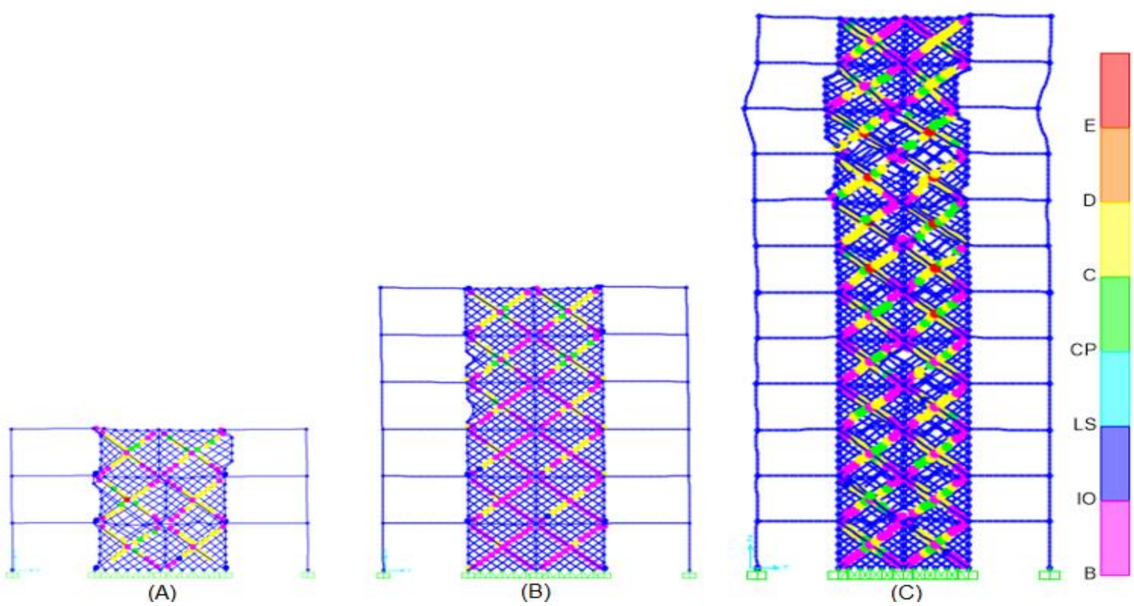

Fig. 11. Three samples of the plastic hinges created in the frames braced with the steel wall of 3 (A), 6 (B) and 12-story (D) structures.

7. Structure drift under earthquake records

The relative lateral displacement of the story which is known as drift among the engineers is the difference between the lateral displacements of the centers of mass of the upper floor and lower floor of that story. Controlling the structure drift is one of the determinative and important controls in the structure design which should not exceed the values specified in the code 2800. Calculating the real values of the story relative lateral displacement can only be done by the nonlinear analysis of the structure. The structure drift under the mentioned earthquake records is presented in the table 2.

Table 2
The maximum results of drift in 3, 6 and 12-story structures.

Earthquake name	Station Name	Direction	The maximum of drift in 3-story structure (%)	The maximum of drift in 3-story structure (%)	The maximum of drift in 3-story structure (%)
"Northridge"	"Ventura – Harbor & California"	U1	1.35	1.04	1.48
		U2	0.74	1.19	1.26
"Loma Prieta"	"Dublin - Fire Station"	U1	0.74	0.83	1.41
		U2	0.55	0.85	1.48
"Imperial Valley"	"Coachella Canal"	U1	1.03	0.96	0.84
		U2	0.87	1	0.79

As it was previously mentioned, each earthquake record has 2 accelerograms, one in X direction and other in Y direction. In the above table U1 is the accelerogram of X and U2 is the accelerogram of Y direction. According to the above table the maximum results in each 3 structures are under Northridge earthquake record presented in tables 3 to 5 in the following.

Table 3
The results of 3-story structure under Northridge earthquake record in U1 direction.

X Direction				
Story	Displacement (cm)	|Displacement|	(Up story - down story)/height	Percentage
3	-10.4568	10.4568	0.004006	0.4006
2	-9.0547	9.0547	0.012345	1.234457
1	-4.7341	4.7341	0.013526	1.3526
Y Direction				
3	4.0266	4.0266	0.00374	0.373971
2	-2.7177	2.7177	0.003191	0.319114
1	-1.6008	1.6008	0.004574	0.457371

As it was previously mentioned, the structure is analyzed and designed under 3 earthquake records (3 couple accelerograms), thus the maximum results should be examined. As it was mentioned earlier, the aim of the steel shear wall, on the other hand, is to sacrifice the shear wall for other structure members and if the colors of the plastic hinges exceeds the allowable limit, the performance of the system would be still acceptable if the structure lateral displacement remains in the range bearable for the sacrifice members in order to prevent failure. The maximum amounts of the structures drift in 3, 6 and 12-story structures are respectively 1.35, 1.19 and 1.48 percent based on table 2. This values is lower than the code allowable value which is 1.5 percent. Therefore, the structures with the steel shear wall have presented a very good performance under these three strong earthquake records.

Table 4
The results of 6-story structure under Northridge earthquake record in U2 direction.

X Direction				
Story	Displacement (cm)	|Displacement|	(Up story - down story)/height	Percentage
6	12.2393	12.2393	0.006012	0.601229
5	10.135	10.135	0.007454	0.745429
4	7.526	7.526	0.006586	0.658629
3	5.2208	5.2208	0.005969	0.596943
2	3.1315	3.1315	0.004875	0.487514
1	1.4252	1.4252	0.004072	0.4072
Y Direction				
6	19.6305	19.6305	0.009708	0.9708
5	16.2327	16.2327	0.010711	1.071057
4	12.484	12.484	0.011885	1.188514
3	8.3242	8.3242	0.010219	1.021943
2	-4.7474	4.7474	0.007109	0.710857
1	2.2594	2.2594	0.006455	0.645543

Table 5
The results of 12-story structure under Northridge earthquake record in U1 direction.

X Direction				
Story	Displacement (cm)	|Displacement|	(Up story - down story)/height	Percentage
12	34.75313	34.75313	0.010846	1.084646
11	30.95687	30.95687	0.011914	1.191411
10	26.78693	26.78693	0.00953	0.952969
9	23.45154	23.45154	0.00927	0.926965
8	20.20716	20.20716	0.01039	1.039014
7	-16.5706	16.57061	0.01042	1.042001
6	-12.9236	12.92361	0.008381	0.838082
5	-9.99032	9.990319	0.006778	0.67775
4	-7.61819	7.618193	0.006639	0.663917
3	-5.29449	5.294485	0.006907	0.69072
2	2.876965	2.876965	0.005354	0.535364
1	1.003192	1.003192	0.002866	0.286626

Y Direction				
12	43.82017	43.82017	0.009097	0.909713
11	40.63618	40.63618	0.009593	0.959257
10	37.27878	37.27878	0.009376	0.937632
9	33.99706	33.99706	0.008663	0.866255
8	30.96517	30.96517	0.008523	0.852312
7	27.98208	27.98208	0.01002	1.002
6	24.47508	24.47508	0.013094	1.309434
5	19.89206	19.89206	0.014888	1.488816
4	14.6812	14.6812	0.014776	1.477582
3	9.509663	9.509663	0.014851	1.485083
2	4.311871	4.311871	0.008945	0.894522
1	1.181044	1.181044	0.003374	0.337441

8. The input energy to structure

The steel shear walls have been used as an energy absorption system in buildings in the last three decades. One of the parameters affecting the behavior of these systems is the type of the beam to column connection. The clamped or simple beam to column connection affects the maximum base shear and the energy absorption by the steel shear walls. In this paper, the clamped connection is used in this system for the beam to column joint based on the research by Gholhaki [32] in order to increase the strength and to absorb the steel shear wall energy. When designing a building against earthquake, the more energy the building dissipates the more ductile and desirable in terms of structure it would be. The amounts of the input energy to the structure under each earthquake record are presented in the table 6. As it can be seen, the highest input energies to the 3, 6 and 12-story structures are due to the Northridge earthquake (figure 12) which is considerable. This values indicates the high energy absorption and ductility of the steel shear wall against the earthquake in tall buildings. The values of input energy are in kg/cm^2.

Table 6
The input energy to 3, 6 and 12-story structures.

Earthquake name	Station Name	Direction	The input energy to 3-story structure	The input energy to 6-story structure	The input energy to 12-story structure
"Northridge"	"Ventura – Harbor & California"	U1	$5.773*10^{7}$	$19.3*10^{7}$	$35.73*10^{7}$
		U2	$4.963*10^{7}$	$18.46*10^{7}$	$35.49*10^{7}$
"Loma Prieta"	"Dublin - Fire Station"	U1	$4.407*10^{7}$	$12.5*10^{7}$	$25.65*10^{7}$
		U2	$4.505*10^{7}$	$12.38*10^{7}$	25.97*10^7
"Imperial Valley"	"Coachella Canal"	U1	$2.688*10^{7}$	$5.444*10^{7}$	$3.982*10^{7}$
		U2	$2.784*10^{7}$	$5.614*10^{7}$	$4.319*10^{7}$

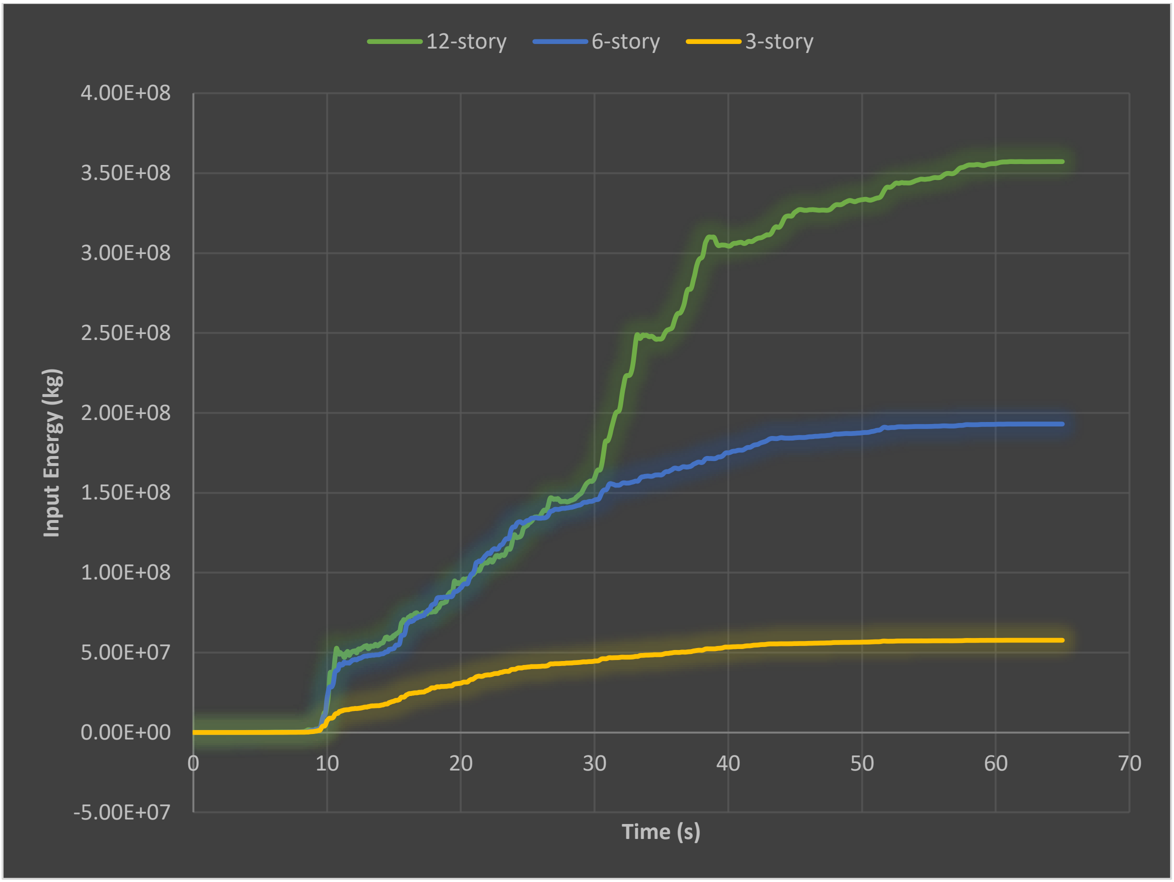

Fig. 12. The highest input energies to the 3, 6 and 12-story structures are due to the Northridge earthquake record.

The amoumt of structure ductility under the earthquake record is acceptable and investigable when the structure has good and acceptable results in drift under that earthquake record. As it was previously mentioned, according to table 2 the values of 3, 6 and 12-story structures drift are lower than the code allowable value which is 1.5 percent. Thus the Comparison of the structures ductility can be done.

9. Base shear in the buildings

The base shear is one of the main parameters in the seismic design of the structures. An acceleration is imposed on the structure when an earthquake occurs, let *m* be the structure mass and *a* be the structure acceleration, based on Newton's second law (F=ma) it is clear that the force imposed on the structure depends on its mass. The steel shear wall is made from 3 to 10 millimeters thin plate and one of its advantages is reducing the used steel in the structure. Thus, it is expected that weaker force to be exerted on each story which reduces the base shear imposed on the structure when the story mass is reduced by using the steel shear wall as a bearing system against lateral forces. The base shear values of structures modeled in this paper under the mentioned earthquake records are presented in table 7 in the following.

Table 7
The maximum of base shear in 3, 6 and 12-story buildings.

Earthquake name	Station Name	Direction	The maximum of base shear in 3-story structure(kg)	The maximum of base shear in ۶-story structure(kg)	The maximum of base shear in 12-story structure(kg)
"Northridge"	"Ventura – Harbor & California"	U1	$6.897*10^5$	$14.85*10^5$	$14.46*10^5$
		U2	$7.149*10^5$	$16.83*10^5$	$16.4*10^5$
"Loma Prieta"	"Dublin - Fire Station"	U1	$6.465*10^5$	$13.61*10^5$	$15.68*10^5$
		U2	$6.777*10^5$	$13.58*10^5$	$16.29*10^5$
"Imperial Valley"	"Coachella Canal"	U1	$6.564*10^5$	$13.57*10^5$	$11.41*10^5$
		U2	$7.498*10^5$	$12.72*10^5$	$12.62*10^5$

Based on base shear Formula (V=C.W) and considering that C is almost the same for all three buildings, the base shear must be multiplied as much as multiplying the weight of the structures but the results indicate that the values of base shear in Short and Intermediate-rise buildings are much more critical than high-rise buildings.

10. Roof center of mass displacement in the structure

The center of mass in a building is the point on which the earthquake forces resultant is imposed. Roof center of mass displacement values of structures modeled in this paper under the mentioned earthquake records are presented in table 8 in the following

Table 8
The maximum displacement of the roof center of mass in 3, 6 and 12-story buildings

Earthquake name	Station Name	Direction	The maximum displacement of the roof center of mass in 3-story building (cm)	The maximum displacement of the roof center of mass in ۶-story building (cm)	The maximum displacement of the roof center of mass in ۱۲-story building (cm)
"Northridge"	"Ventura – Harbor & California"	U1	10.46	17.02	43.82
		U2	6.82	17.05	38.25
"Loma Prieta"	"Dublin - Fire Station"	U1	5.43	14.37	43.53
		U2	4.77	14.68	50.02
"Imperial Valley"	"Coachella Canal"	U1	8.51	18.43	18.74
		U2	6.69	19.63	21.42

In order to simplify the performance assessment of the intended structure in terms of the roof center of mass displacement under the three mentioned earthquake records, the maximum values are extracted and presented in table 8. Based on this table the maximum displacement of the roof center of mass in 3, 6 and 12-story buildings respectively occurred under Northridge, Imperial

Valley and Loma Prieta earthquake records. Based on the commentary of instruction for seismic rehabilitation (publication 361) [33] the allowable displacement of the roof center of mass in a nonlinear analysis is equal to the multiplication of the allowable drift and the structure height. Based on the same publication, the allowable drift in the braced steel structures is 1.5 percent for the lateral safety performance level. On the other hand, the structures height in this research is 10.5, 21 and 42 meters so the maximum displacement of the roof center of mass would be respectively 15.75, 31.5 and 63 centimeters. Based on table 8, the maximum displacement of the roof center of mass in 3, 6 and 12-story buildings is respectively obtained 10.46, 19.63 and 50.02 centimeters in the analysis of the structure under study which indicates the suitable performance of the steel wall as the lateral bracing system.

Conclusion

The results obtained from analyzing the earthquake show that:

- The aim of the steel shear wall system is to sacrifice the shear wall for other structure members and if the colors of the plastic hinges in this system exceed the allowable limit, the performance of the system would be still acceptable if the structure lateral displacement remains in the range bearable for the sacrifice members to prevent the failure.
- Based on the values obtained from the story drift, the maximum amounts of the structures drift in 3, 6 and 12-story structures are respectively 1.35, 1.19 and 1.48 percent based on table 2. These values are lower than the allowable limit of the code which is 1.5 percent. Thus, the structures with the steel shear wall presented good performance under these three strong earthquake records.
- The results obtained from the input energy to the structure show that the amount of energy absorption of the structure increases dramatically with increasing the height. Thus the steel shear wall is one of the best lateral bracing systems in absorbing and dissipating the lateral force imposed on the structure in high-rise buildings.
- By reducing the weight of the used steel by utilizing the steel shear wall as the bearing system against the lateral forces in each story, a weaker force is applied on that story based on Newton’s second law which reduces the base shear imposed on the structure. The results indicate that the values of base shear in Short and Intermediate-rise buildings are much more critical than high-rise buildings.
- The results obtained from analyzing the intended structures reveal that the displacements of the roof center of mass under every three earthquake records are inside the code allowable range and the steel wall system had a good performance.

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