SEISMIC PERFORMANCE COMPARISON OF MID-RISE MOMENT RESISTING FRAME AND SHEAR WALL SYSTEM

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ABSTRACT

Nowadays, RC structural systems are being used in construction industry worldwide. The observed response of medium rise building, based on post-earthquake damage assessment reports, during the latest earthquakes in the USA, Chile, Mexico and Japan have indicated that buildings including shear walls and dual structural systems behave considerably better during strong shaking. The viewpoint that shear walls are inherently brittle is still held in many countries (correspondingly in the codes) as a consequence of failures in poorly detailed RC walls. In this investigation, several RC frames, with and without shear walls, are designed according to the seismic codes, and then their nonlinear duplicates are exposed to a set of severe quakes. The time history nonlinear analyses results of these prototypes are then used to probe the effect of the existence of shear walls through using performance based design method. It is concluded that in general the existence of shear walls results in a noteworthy decline in the seismic deformation demand. These changes happened in both the plastic rotations in main RC structural members and the inter story drifts ratio. Hence, it is concluded that the existence of shear walls mostly improves the safety of medium rise RC frames within the assessment framework of performance based design codes such as ASCE41 (2006).

INTRODUCTION

Post-earthquake damage evaluation has shown that although buildings with shear walls had an appropriate overall performance in the last twenty years sever earthquakes, in some cases, columns as well as shear walls have been damaged, due to presence of short structural elements and inadequate transverse reinforcement. These memories amongst engineers have promoted this attitude that shear walls have brittle structural manner. Indeed, investigations on the nonlinear behavior of shear walls has shown that slender shear walls with flexural dominated behavior have a good seismic performance and resilience great enough to absorb earthquake’s energy (Panagiotou et al. 2011; Zhang and Wang 2000; Ioannis and Michael 1990). These investigations also reveal that even squat shear walls could be designed in a way that their behavior would be similar to that of slender walls (Pilakoutas and Elnashai 1995; Salonikios et al. 1999). Considering these matters, designed shear walls with recent guidelines have high shear strength and also a respectable energy absorption ability which comes from yielding of longitudinal steel bars and boundary transverse reinforcement. The conviction that shear walls are inherently brittle still prevails amongst engineers, and therefore, they usually prefer to choose moment resisting RC frames without shear walls. In this study, couples of three, four and six stories 3D structures with and without shear walls have been linearly designed as a representative for low to mid rise buildings with concurrent Iranian buildings codes and for assessing the seismic performance of these designed buildings, nonlinear model of them have been reproduced in

SeismoStruct software (Seismosoft 2008). At the end, results of time history nonlinear analyses have been interpreted with the aim of performance based design method and comparison of the results has been made.

SHEAR WALLS

Currently, RC shear walls are being used in buildings as a main lateral load resisting system all over the world. Shear walls are divided into two main categories, slender shear walls, and squat shear walls. The "aspect ratio", the ratio of wall height to width, of squat shear walls is below 1.5, and their behavior is mainly controlled by the shear. This kind of wall is just used in the bottom floor of buildings for elevating the so-called seismic base of structures, or, in garages, as a retaining wall. Moreover, they are not continued through the building height and confine within the first story of the building.

The "aspect ratio" of slender shear walls is more than 2 and their behavior is controlled by the flexure (Salonikios et al. 1999; Salonikios 2007). These kinds of concrete walls are usually used in the multi-story buildings, being a part of lateral load resisting systems. All walls, utilized for designing the models in this research, are within the slender shear walls range.

In the flexural dominated shear walls, yielding of longitudinal steel bars always occurs before the failure mechanism of the wall (Ioannis and Michael 1990; Paulay and Priestley 1992). This yielding in plastic hinge zone of the shear walls, usually close to shear wall base, is the main source of energy dissipation process. In these walls, failure typically occurs when concrete strain in the compressive zone of the shear walls reaches its crushing limit state. Shear walls which are designed with recent guidelines have the aforementioned desirable flexural failure mechanism. Furthermore, undesirable modes of failure, such as web crushing or wall buckling, are usually prohibited in these code-based RC walls.

Concrete Cracking and longitudinal main steel bars yielding decrease the stiffness and strength of the shear walls. As a result, the response of the shear walls includes a complex combination of deformations (Paulay and Priestley 1992). The main deformation sources of shear walls are as follow:

$$\Delta = \Delta_{flexure} + \Delta_{shear} + \Delta_{slid} + \Delta_{B.R} \qquad (1)$$

$\Delta_{\mathbf{flexure}}$: Deformation due to flexural behavior of shear walls,
In the flexural dominated shear walls, this deformation, amongst the others has a huge contribution to total displacement of the shear wall (Panagiotou et al. 2011).

$\Delta_{\mathbf{shear}}$: Deformation due to shear behavior of shear walls,
In the dominated flexural shear walls, shear deformation, amongst the others has a less than 5% contribution to total displacement of the shear wall (Panagiotou et al. 2011).

$\Delta_{\mathbf{slid}}$: Deformation due to shear-sliding mechanism in the wall base.

$\Delta_{\mathbf{B.R}}$: Deformation due to wall base rotation.

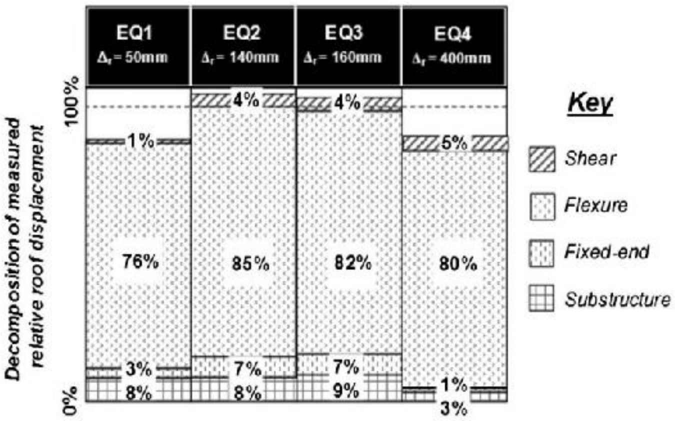

Figure 1. Contribution of four different sources of deformation to peak roof relative lateral displacement.

Figure 1 illustrates separated sources of roof top displacement in a seven-story shear wall building (Panagiotou et al. 2011). Figure data is extracted from an experimental dynamic test which was accomplished on the outdoor shake table of San Diego University. Each column in this figure displays the deformations contributing to the measured top roof displacement for different earthquake records. Each of these earthquakes has a different intensity, and they get stronger from left to right along the horizontal axes of the figure.

MODELING

For the designing process and the linear analysis of the buildings, the well-known Sap2000 software has been used. Nonlinear modelling and analysis of these buildings have been done with nonlinear finite element analysis SeismoStruct software.

For beam and column modelling in nonlinear models of the buildings, inelastic plastic hinge frame element with concentrated plasticity has been employed (Scott and Fenves 2006). Moreover, the fixed length in which plasticity of plastic hinge zone extends is opted to be 16 present of the element length in both extremes of the member. It is noteworthy to say that these elements are developed based on the fiber approach method. Additionally, the Mander (Mander et al. 1998) and Menegotto (Menegotto and Pinto 1973) nonlinear input characteristics are selected correspondingly for materials of the concrete and steel fibers, as suggested by an analyzed framed structure verified according to a full scale test of the building in the abovementioned software.

For modelling the shear wall elements, beam-column element with distributed plasticity which is also configured based on fiber approach method (Beyer et al. 2008) is picked for the nonlinear model of the building. Concrete cover, concrete core and longitudinal reinforcement bars and also boundary elements of shear walls can be modelled readily and precisely with this proposed available model. Even though fiber section elements cannot incorporate the shear effect, and inevitably they are not capable to include the shear deformations, negligible amount of shear deformation contribution of slender shear walls, as understood by the previous section, is the reason why this kind of element is employed to model the shear walls. Additionally, input parameter characteristics of materials in the shear wall elements of a verified shear wall slice building have been selected to be used in the software (Seismosoft 2008). This RC slice shear wall building was tested on an outdoor shake table under dynamic conditions (Panagiotou et al. 2006).

DETAILS OF STRUCTURAL DESIGN

In this study, there are two different groups of buildings, with and without RC shear walls. In both kinds of structure, member sizes have been proportioned for medium ductility level. Spans length, height and story numbers of the buildings have been chosen in a way that designed characteristics of the buildings could be an appropriate representative of the low to mid rise RC buildings. In this case, 5m spans length and 3.5m story height have been selected for all case study RC buildings. For having the same structural condition for both groups of buildings, in the designing procedure of the buildings with shear walls, in the first step, their beams and columns have been designed for gravity forces only. Next, shear walls have been placed in the previously designed frames, and just shear walls of the buildings have been designed for earthquake loading. With the mentioned design approach, not only are the interaction of gravity designed frames with the shear walls taken into account, but also the weight of the buildings in both groups are nearly the same.

For this purpose, Code 2800 (Iranian seismic code) has been used for earthquake loading phase, and for RC member design in both frame and shear wall systems, ACI318-99 code has been used correspondingly.

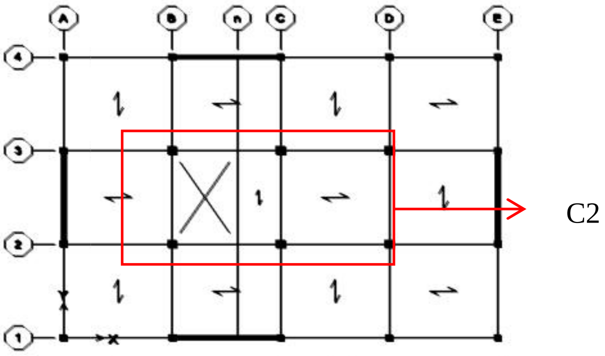

Figure 2. Geometry Plan of case study building, and position of the shear walls.

All members' size as well as rebar percentage of the RC moment resisting systems is referred in table 1 to 3.

Table 1. Member sizes and structural details for three stories moment resisting building (cm)

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	Columns-b x h (° (%))
1	40x45 (0.98-0.78)	50x50 (1.21)
2	40x35 (0.98-0.84)	50x50 (1)
3	40x35 (1.04-0.52)	50x50 (1.1)

Table 2. Member sizes and structural details for four stories moment resisting building (cm)

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	Columns-b x h (° (%))
1	45x50 (0.93-0.77)	55x55 (2.01)
2	45x50 (1.12-0.77)	55x55 (1.34)
3	40x45 (1.18-0.79)	50x50 (1.28)
4	40x45 (0.79-0..47)	50x50 (1.28)

Table 3. Member sizes and structural details for six stories moment resisting building (cm)

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	Columns-b x h (° (%))
1	45x55 (1.18-0.92)	60x60 (3)
2	45x55 (1.35-1.18)	60x60 (3)
3	45x55 (1.18-1.01)	60x60 (2.09)
4	45x55 (1.02-0.84)	60x60 (2.09)
5	45x55 (0.73-0.56)	60x60 (1.34)
6	45x55 (0.73-0.56)	60x60 (1.34)

All members' size as well as rebar percentage of the RC buildings with shear wall is referred in table 4 to 6.

Table 4. Member sizes and structural details for three stories buildings with shear walls (cm)

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	C1-b x h (° (%))	C2-b x h (° (%))	Wall-L x t (° (%))
1	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	535x30 (0.4)
2	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	530x20 (1.2)
3	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	530x20 (1.2)

Table 5. Member sizes and structural details for four stories buildings with shear walls **(cm)**

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	C1-b x h (° (%))	C2-b x h (° (%))	Wall-L x t (° (%))
1	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	545x30 (0.97)
2	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	545x30 (0.53)
3	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	530x20 (1.2)
4	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	530x20 (1.2)

Table 6. Member sizes and structural details for six stories buildings with shear walls (cm)

Story	Beams-b x h (°^{T} (%) _ °^{B} (%))	C1-b x h (° (%))	C2-b x h (° (%))	Wall-L x t (° (%))
1	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	600x50 (0.72)
2	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	600x40 (0.75)
3	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	600x40 (0.75)
4	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	550x40 (1.07)
5	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	550x40 (1.07)
6	30x30 (0.98-0.67)	30x30 (1)	40x40 (1)	550x35 (1.14)

PERFORMANCE CRITERIA

To evaluate seismic performance of a building, suitable seismic indicators are essential. In this study, seismic performance of the buildings in severe earthquakes is aimed to be assessed. In this case, appropriate seismic indicator, such as the ones corresponding to near-collapse limit state may be required. Members' Plastic hinge rotation and maximum inter-story drift ratio have been designated as a seismic performance indicator for both local and global scale respectively. In the case of plastic hinge rotation, collapse prevention (CP) performance level based on ASCE41-06 standard seems to be suitable. Two and four percent maximum inter-story drift ratio are also appropriate for the global seismic performance limit state, the first percent is based on ASCE07 design earthquake limit state, and the latter is based on ASCE41-06 standard for the CP performance level.

Plastic hinge rotation is measured differently in slender and squat shear walls. In slender shear walls, plastic rotation is the value in which the imaginary horizontal line, above the plastic hinge height, rotates around the strong axis of the wall section. In literature, there are different methods to find plastic hinge height in the base of the walls. Each guideline also gives its own distinct recommendations to read this parameter. Minimum value between half of wall length and wall height within the bottom story is recommended by ASCE41-06.

EARTHQUAKES

Time histories nonlinear analysis is the method used for structural analysis of the case study buildings. For this purpose, Sever earthquakes with return period of 2475 years have been selected to be used. To select these earthquakes, scaling methods are usually used to match earthquakes acceleration response spectrum to predefined spectra. If the severe earthquakes are supposed to be selected with this approach, maximum credible earthquake (MCE) response spectra or design response spectra may be used as the predefined spectra. In this method, intensity-based scaling of ground motion records using appropriate scale factors is required so that the mean value of the 5%-damped response spectra for the set of scaled records is not less than the predefined response spectrum (such as MCE or design spectrum) over a suitable range of period. There are different structures with different fundamental periods in this study, including buildings with and without shear walls, Therefore, a wide range of fundamental periods are encountered. In this case, a set of records whose response spectrum covers the whole range of these fundamental periods is desirable. Based on the fact that the procedure to collect the acceptable ground motion records is a sensitive and professional task, Ground motion records of SAC steel project have been selected to be applied for the analysis.

RESULT

INTERSTORY DRIFT RATIO

Multiple line graphs in figures 3 and 4 show inter-story drift ratio (IDR) variation against the story numbers (for three and six story buildings, with and without shear walls) in different applied ground motion records. The right hand side graphs are for the buildings with shear walls. In each graph, grey lines represent IDRs for each individual ground motion. On the other hand, red and blue lines summarize the set of grey lines into the maximum and the medium trend lines in turn. It is clear from the graphs that for both types of the buildings, the overall trend of IDRs remains unchanged for each individual applied ground motion. In the first story of the building with shear walls, the IDR is in its lower point. Above the first story, there is a slight rise trend until the top two or three stories. IDRs along the top stories remain almost stable, reaching their peaks at the top story. For buildings without shear walls, most of the IDRs are observed in the lower stories, and the middle stories in these buildings are where the peak points of IDRs locate.

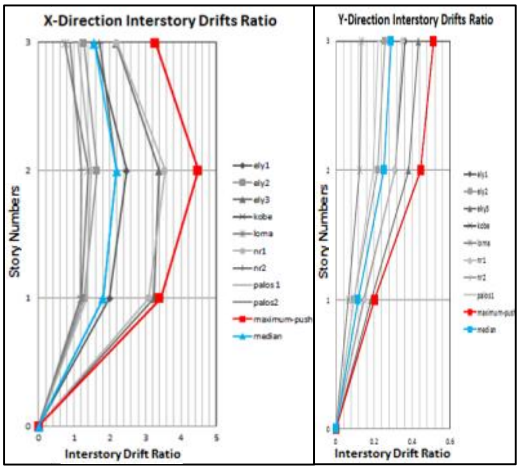

Figure 3. Inter-story drift ratio profile for the four stories buildings.

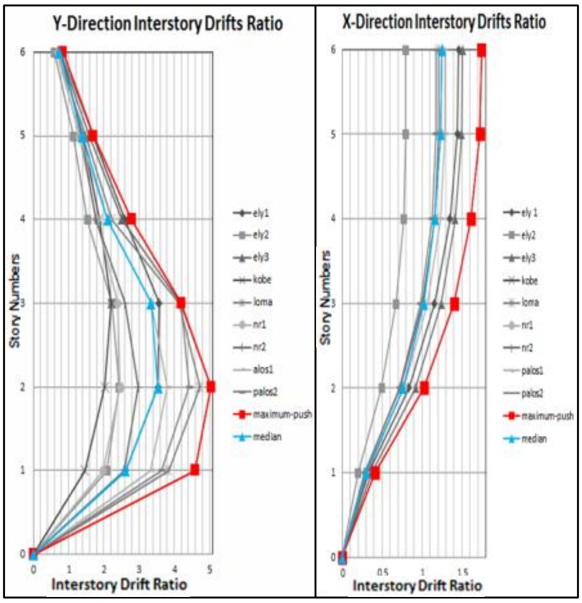

Figure 4. Inter-story drift ratio profile for the six stories buildings.

Story height is almost one of the most significant factors in seismic behavior of the buildings. As mentioned before, the peak point of IDRs occur in the top story of shear wall buildings. If there is an increase in the total height of the buildings, the value of peak points will increase as well. In these structures, a huge amount of rigid body rotation would be imposed to top stories due to yielding and plastic rotation of shear walls in the bottom stories. This rigid body rotation is cumulative in nature and causes an increase in the amount of drift ratio on the top stories of the buildings. An increase in total structural height of the moment resisting systems also causes the location of IDR peak point to go down to the lower stories. It seems the reason is the P-Delta effect existing in these buildings.

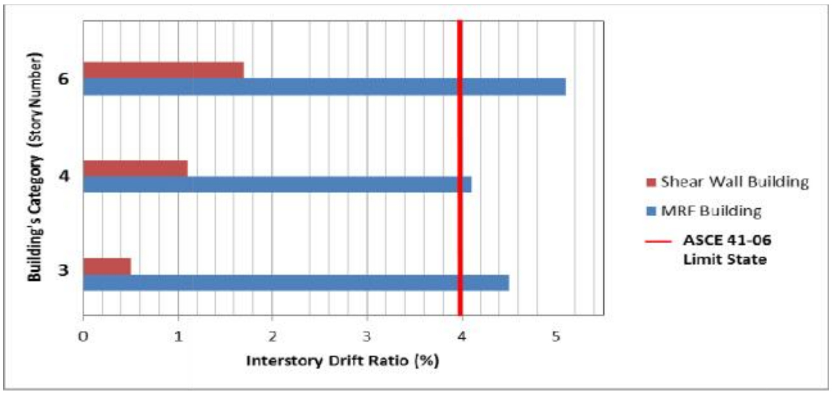

Figure 5. Comparison of the maximum Inter-story drift ratio of the buildings.

In figure 5, maximum inter-story drift ratios, which create the bar graph for both building categories along with the different involved ground motions, are extracted from x and y directions. In this figure, the horizontal and vertical axes show the IDRs and the building categories correspondingly. The IDRs are provided in percent, while the story numbers are selected for being the building category. As it can be seen in the graph, all buildings without shear walls are in critical status and have passed the IDR limit states of the ASCE41-06.

PLASTIC HINGE ROTATION

Plastic hinge location distribution in beams and columns is totally different in the buildings which have and the buildings which do not have shear walls. In the buildings without shear walls, the plastic hinges are normally developed within the lower stories of the buildings. For instance, in three story and four story buildings of Mrf systems, damaged beams are mainly concentrated in the first and the second stories. There was the same condition in six story buildings; damaged beams were mainly in first and second stories, except for a limited number in third story. In columns, damages usually occurred in the lower stories of the buildings. In general, in low to mid-rise buildings of Mrf systems, the upper stories movement is similar to a rigid body motion, and does not participate in the lateral load resisting mechanism. This rigid body motion behavior is not seen in the shear wall buildings.

In figure 6, the numbers of damaged beams and columns and the comparison of their summation are illustrated for the four story buildings. In this figure, horizontal and vertical axes represent the earthquake names and the number of damaged elements correspondingly. In buildings with shear walls, damaged elements have not been seen at all. In the six story buildings with shear walls, the condition remained the same, except for some limited numbers of damaged beams (Figure 7).

As it can be seen in figure 7, the numbers of damaged elements in shear wall systems are very limited, in comparison with the ones in the buildings without shear walls. These elements are the beams that are connected to the shear walls. It seems that when the shear walls behave nonlinearly in the sever earthquakes, considerable amount of vertical displacement would be emerged in the plastic hinge zone of the shear walls in each story separately. This vertical displacement is cumulative in nature and is transmitted directly to the beams. Therefore, upper story beams are in a more critical status than the rest of the beams. For finding that whether these few numbers of damaged beams make a significant role in the lateral carrying capacity of the buildings or not, both extreme end points of the beams that are connected to shear walls have been released and converted to simple supported ends. In this way, these beams were out of work in the lateral load resisting mechanism. According to the Figure 8, analysis has shown that due to very few growths in the IDR, these beams could be treated as secondary elements. As a result, the damaged beams have a trifle role in the seismic behavior of the building with shear walls.

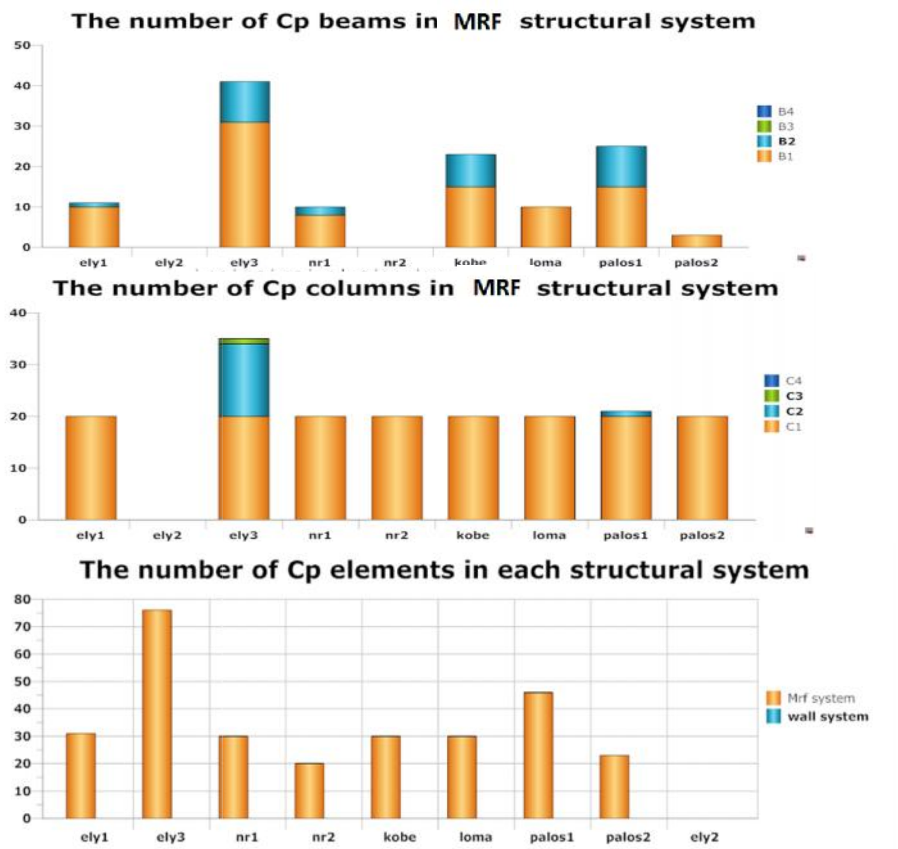

Figure 6. Damaged beams and columns of four stories buildings in the applied ground motions.

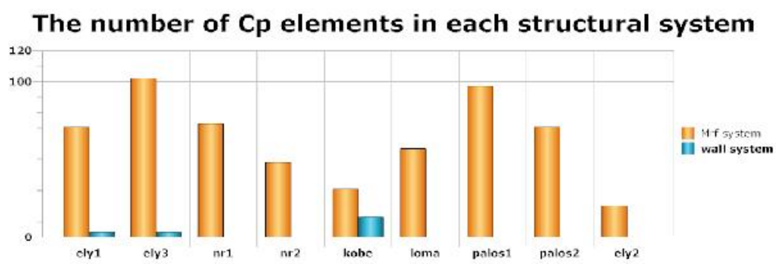

Figure 7. Damaged elements of six stories buildings in the applied ground motions.

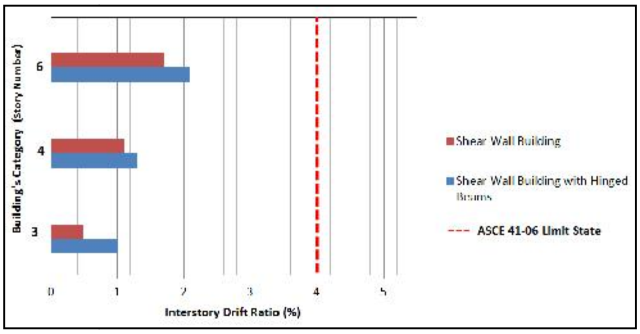

Figure 8. Inter-story drift ratio Comparison of the Shear wall buildings against the building with hinged beams.

CONCLUSION

In this study, seismic performance of the concrete mid-rise buildings, with and without shear walls have been evaluated. It seems that in the buildings with shear walls, the earthquake deformation demands are decreased dramatically. These decreases include both of the plastic hinge rotations and inter-story drift ratios at once. In some few cases, damages in beams of the aforementioned structural systems have been witnessed.

Few damaged beams besides the absence of damaged columns in these kinds of structures indicate a safer and a superior desired failure mechanism. Therefore, and according to the analysis and the evidence, it seems that the buildings with shear walls have a better seismic performance than the corresponding structural systems in which shear wall elements do not exist.

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