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Seismic behavior improvement of reinforced concrete shear wall buildings using multiple rocking systems

Mohammad Khanmohammadi *, Sajad Heydari

School of Civil Engineering, College of Engineering, University of Tehran, Tehran, Iran

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ABSTRACT

In recent years, researchers have proposed damage avoidance design philosophy, instead of traditional design concept, which is inherently damage-oriented, to mitigate suffered damage of buildings. To this end, the rocking system which is a method for limiting seismic forces to structures along with energy dissipation devices and restoring force system have been well established. A considerable number of studies have been conducted to investigate the seismic performances of base-rocking systems on precast segmental bridge piers, shear wall, and steel braced frame. In recent years, a few works have investigated the multiple rocking system behavior; but, there are still vague points about the details and response of this system. Thus, different shear wall buildings (three cases of the rocking structure and one case with a traditional design and 8, 12, 16, and 20 stories) were analyzed under two suits of ground motions levels using precise model. The results demonstrated that, if energy dissipation and post-tensioning tools are implemented properly in the multiple rocking system, (a) higher mode effects are mitigated on shear and moment actions, (b) the drift ratios do not increase approximately compared to the result of traditional wall and get closer to the result of base rocking system, (c) the values of horizontal acceleration remains almost constant with the development of rocking sections over height, (d) residual displacements of buildings are negligible, (e) the centerline elongation of shear walls are not considerable; for taller buildings, they are smaller than shorter ones, and (f) the pounding at the contact surface is not important.

1. Introduction

In recent years, researchers have recognized that using rocking systems will mitigate the suffered damage of buildings in strong earthquakes. Compared with the traditional seismic design methods in which structures are inherently exposed to damage and the financial losses caused by repair or replacement coupled with downtime can be devastating, the base rocking system is expected to mitigate damage and improve post-earthquake serviceability demand. Accordingly, damage avoidance design (DAD) philosophy has been proposed by researchers [1,2]. This concept has been improved by integrating rocking, structural flexibility, post-tensioning tools, and dissipation energy devices in order to control higher displacement demands and dynamic instability during severe earthquakes [3]. A considerable number of experimental and analytical studies have been conducted for evaluating the seismic behavior of rocking systems in terms of energy dissipation devices, self-centering tools, impact at base, and values and distribution of the considered design criteria over the height of structures [1,4–8]. The majority of reported researches have been concentrated on the behavior of precast piers of bridges [9–19], while few studies have works on the seismic behavior of precast shear wall [20–25] or steel braced frame with rocking at base [26,27].

Although the seismic performance of base rocking system for bridge piers and shear walls has been well established, no comprehensive research has been carried out on the efficiency of multiple rocking sections over the height of shear walls or steel braced buildings. Distribution of seismic demands over the height of buildings is essentially different from bents of bridges which generally behave as a-single-degree-of-freedom structures. In this regard, Wiebe and Christopoulos [28] investigated the seismic performance of multiple rocking sections over the height of five predetermined shear wall models that were designed using natural period and nearly code based assumptions. Simple and global nonlinear concentrated hinge models at rocking joints were provided to simulate the actual behavior at rocking sections. They concluded that using multiple rocking joints over the height of buildings could mitigate the effects of higher modes on shear and bending

* Corresponding author. Tel.: +98 9123543431.
E-mail address: mkhan@ut.ac.ir (M. Khanmohammadi).

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moments, while elongations, displacements, and horizontal acceleration increase. However, evaluating the efficiency of multiple rocking systems needs employing a refined and precise nonlinear model at rocking bases and over the building height; also, to develop a new system for DAD, all the considered design criteria which influence the performance of a building should be investigated. Thus in the present research, a comprehensive study was carried out by the precise model of rocking systems. The efficiency of using multiple rocking sections on shear wall buildings was also investigated on the four height levels of buildings (i.e. 8, 12, 16, and 20 stories) and three alternatives of rocking sections (i.e. base, base and mid height, and multiple rocking sections on every second story). For comparison with the conventional design, for each building, one shear wall designed by the current design code was also considered. Seven seismic performance design criteria at two levels of earthquake intensity were evaluated and compared with the traditional design building to discover the effect of multiple rocking sections over height on the structural response. These criteria include inter-story drift ratio, residual drift, elongation at centerline of wall, shear and moment forces, concrete toe crushing at the edge of contact surface in rocking sections, and distribution of dissipated energy through dissipators over height).

2. Rocking behavior and nonlinear modeling

The connection of base rocking system is detached at contact surface for providing free rotation and releasing bending moment demands. It is generally assumed that sliding movement caused by shear is prevented at rocking sections. Therefore, rocking behavior models are nearly nonlinear and elastic and have minor material nonlinearity and dissipating energy resulted from hysteretic responses. Theoretically, such a response is stable until excessive lateral displacement or toe crushing at the base degrades lateral strength and afterward instability will be probable; even although such a behavior is stable, it does not produce enough hysteretic energy to meet seismic demands. To overcome such deficiency in response, it is necessary to add a dissipation energy device. The location and type of an appropriate energy dissipation device should be also investigated. However, these issues were not the subject of the current research. Employing any dissipation energy device will cause change in the hysteretic response from nearly elastic nonlinear to flag-shaped behaviors. In addition, a tool is usually designed for getting the structure back to the original position after earthquakes. The post-tensioning cables have been widely used to improve both characteristics of self-centering and yielding lateral forces. In the current research, all the involved mechanisms needed for rocking section were taken into account over the height of shear walls. Fig. 1(a) and (b) shows physical multi-rocking section and provides a numerical nonlinear model. In order to model the rocking behavior of shear wall, an equivalent beam–column element with the properties of actual wall is defined and a horizontal rigid beam element is used at the contact surface. One of the influential parameters in modeling rocking sections is modeling the motion of neutral stress axis between contact surfaces [13]. To model contact behavior and shift in neutral axes, a number of zero length no-tension spring elements which are distributed along contact surface are defined. Normally, to prevent premature toe crushing at the contact surface, the concrete section near to edges is confined by means of adequate lateral and vertical reinforcements; hence, except cover concrete, the possibility of core concrete failure is improbable. To simplify modeling here, both core and cover concrete behaviors were defined as confined and the ultimate strain of concrete was assumed to be 0.015, which was appropriate for the designed concrete columns. The tributary area of each spring was assumed for providing force–displacement characteristic of springs. This simulation would allow for contact detachment and shift of neutral stress axes, which is expected to be significant for the results. Energy dissipation devices which were modeled by mild steel reinforcements in the current research were simulated using distinct springs in the defined position and then connected to the base or lower panel by means of a horizontal rigid beam. The locations of such mild steel rebars and the assigned stress–strain curves are shown in Fig. 1(a) and (b). The ultimate strain for mild steel rebars is limited to 0.07 to capture low cycle failure [22]. The gravity load on the wall was applied in a concentrated manner at each story.

As shown in Fig. 1(b), the 1D spring element was used for modeling the post-tensioning strands. In the current research, it was assumed that all the post-tensioning tools connect panels vertically from the top of the upper panels to the top of lower one and no horizontal movement is allowed between two panels at the contact surface. From the physical position point of view, their locations and effects on behavior were first investigated; then, the reported results were based on the best suggested location. The nonlinear stress–strain curve of the strands is shown in Fig. 1(b). In this regard, the yield strain was calculated based on yielding stress and modulus of elasticity. Also, the ultimate strain of strands was considered as 0.03 and the amount of initial stress was adopted from other recommendations as %25 of yield stress [29].

It should be noted that, in order to avoid complexity in the analysis and results, in-plane shear behavior of wall segments was assumed to be elastic and no shear displacement was taken in rocking sections. Elastic behavior was defined for out-of-plane behavior.

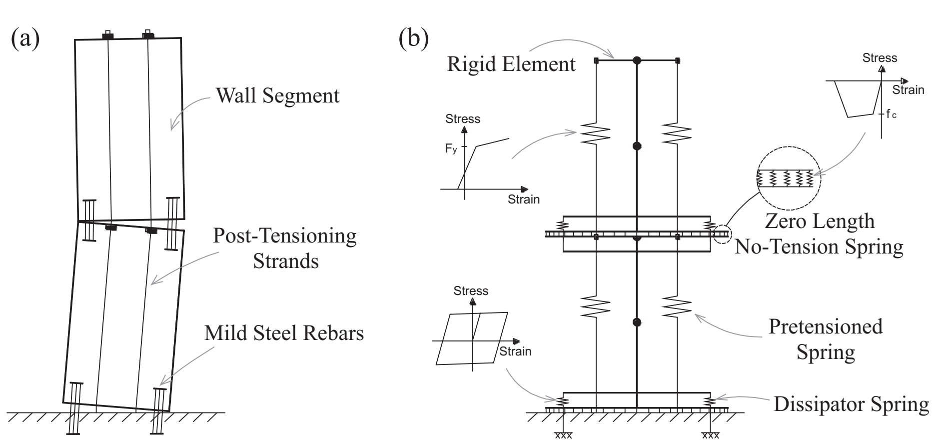

Fig. 1. Multiple rocking system (a) physical model, (b) analytical model and component behavior.

3. Verification

Numerous parameters affect the response of rocking system; therefore, it is useful to use experimental test results to provide a refined and reliable nonlinear model. To this end, the experimental program conducted by Restrepo and Rahman [21] was selected. The specimen and test setup are illustrated in Fig. 2. The specimen was a half-scale precast concrete wall which was designed to rock at the base.

The tested wall had the height of 4000 mm and effective height of 3700 mm. The wall section had the length of 1350 mm and thickness of 125 mm. Compressive strength of the unconfined concrete was $F_c = 25$ MPa and the yielding strength of reinforcement was $F_y = 400$ MPa. The defined yield stress of the strands at 0.2% offset strain was $f_{py} = 1746$ MPa and the ultimate tensile strength was $f_{pu} = 1836$ MPa. The energy dissipators had 16-mm-diameter bars with $f_y = 460$ MPa, $f_{su} = 630$ MPa, and $\varepsilon_{su} = 0.15$ (ultimate strain capacity) [21].

To prevent shear slip displacement at the base, as shown in Fig. 2, two shear keys were provided. The behavior of shear keys were modeled using a stiff spring at the base level and adequately large strength was defined to prevent nonlinearity. The nonlinear modeling of the involved elements was performed using the modeling technique discussed in the previous section, Fig. 1 and, the values suggested by Restrepo and Rahman [21].

The open source software OpenSees v. 2.2.2 [30] was employed for modeling and analysis. To compare the analytical results with the experimental ones, the cyclic reversal analysis with displacement-control approach was carried out.

The displacement amplitude and cycle numbers were the same as the ones used in the testing protocol. The force–strain of mild steel rebars as energy dissipation device and force–drift of wall specimen are illustrated in Fig. 3(a) and (b), respectively, for both analytical and experimental results. A good agreement between analytical and experimental response was found in the results pertaining to both force–strain and force–drift. As expected, the resulted response of the whole system was flag shaped which was the main characteristic of rocking systems. Therefore, the proposed analytical modeling was considered appropriate for estimating rocking systems.

4. Considered buildings

In order to investigate the behavior of multiple rocking systems in different buildings, four shear wall buildings with 8, 12, 16, and 20 stories were considered. The results of the research conducted by Pennucci et al. [31] were used to determine the specifications of geometric and stiffness of shear walls. Pennucci et al. [31] designed the components of the rocking section at the base level using displacement base design method and drift ratio of 2% was chosen as the performance criteria of the displacement based design procedure. Walls were designed using Eurocode 8 and ASCE7-10 [32,33], design of the walls included a plastic hinge at the base level such that no plastic hinge is formed at height. Accordingly, in the current study, upper parts of the wall are assumed to be elastic.

Also, the construction site of this structure was California (USA) and the soil type was considered to be C and the design reference acceleration was assumed as 0.5g [31].

The plan of the considered buildings adopted from Pennucci et al. [31] is showed in Fig. 4. The building had four shear walls (two at perimeter and two in the middle of the plan). The two walls in the middle of the plan were designed as rocking system and some effects such as torsion and off-center load of the lateral and gravity were ignored. Therefore, the 2D analysis was carried out. The total applied load to each wall due to self-weight and tributary area of floors was 2500 kN. Geometry, material of dampers, and used post-tensioning strands from all the buildings are listed in Table 1. Table 1 shows information about periods resulted from the analysis in this study and other information are adopted from Pennucci et al. [31]. Other considered material properties were as follows: compressive strength of concrete: $f_c = 40$ MPa, modulus of elasticity of concrete: $E_c = 30$ GPa, tensile strength of mild steel dampers: $F_y = 300$ MPa, modulus of elasticity of steel dampers: $E_s = 210$ GPa, yield strength of post-tensioning strands: $F_{pty} = 1560$ MPa, modulus of elasticity of strands: $E_{pt} = 195$ GPa, and ultimate tensile strength of strands: $F_{ptu} = 1860$ MPa.

Fig. 5 schematically illustrates all the considered buildings for carrying out the parametric analysis. As shown in this figure, in addition to rocking system at the base (which is shown in R-1 designation hereafter), three other models were also considered

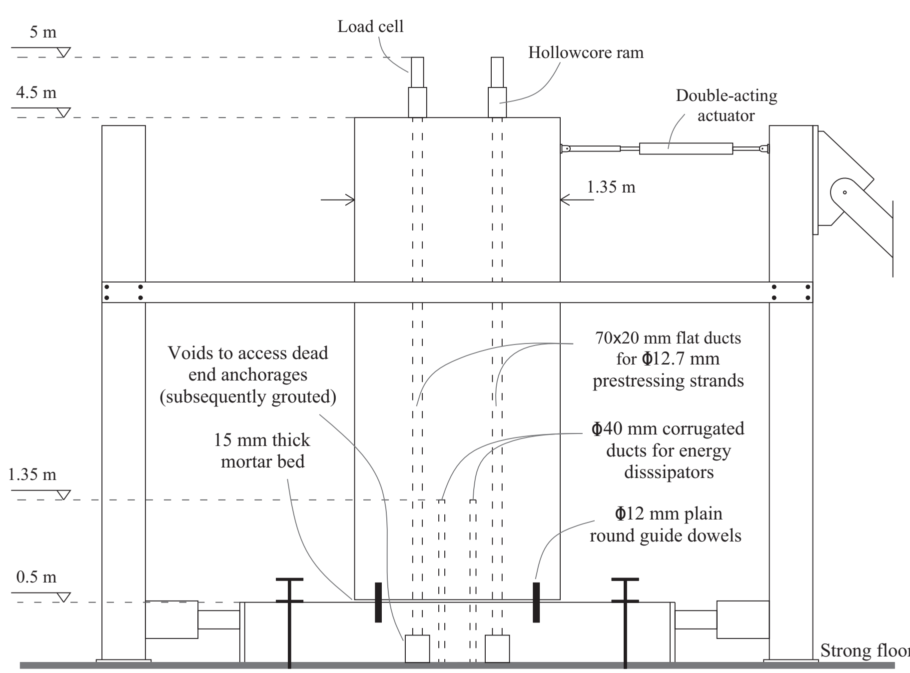

Fig. 2. The test setup and specimen adopted from [21].

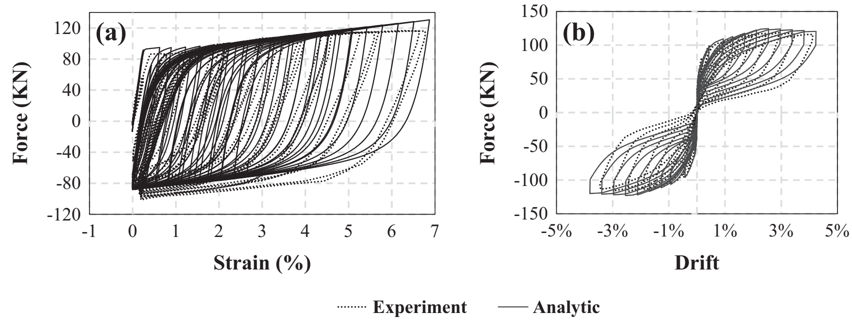

Fig. 3. Comparison of the analytic model with experimental results.

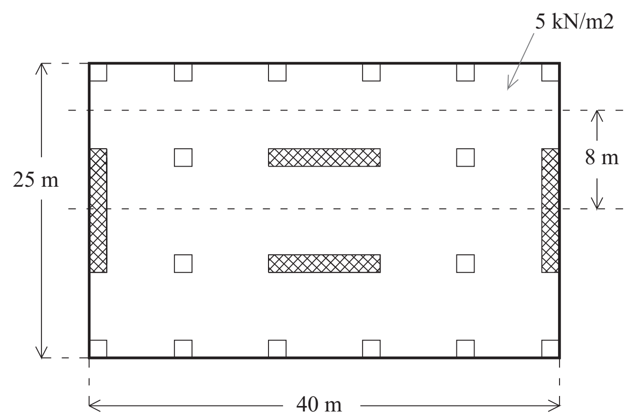

Fig. 4. Considered case study plan [31].

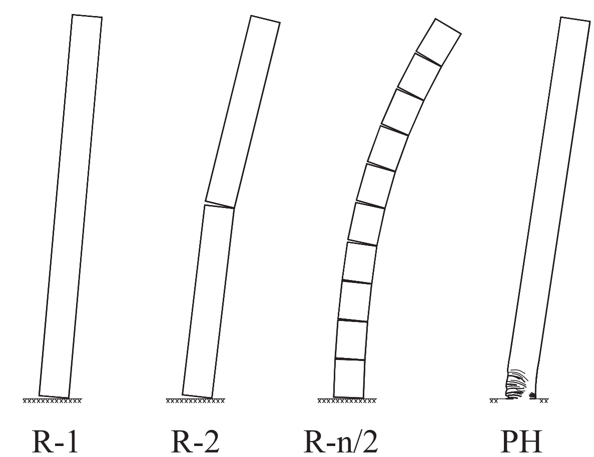

Fig. 5. Schematic representation of considered rocking sections and plastic hinge models.

as follows: two rocking sections (one at the base and the other at mid-height) with the designation of R-2, multi rocking section on every second story with the designation of R-n/2. In addition to three rocking systems, to compare the responses with the response of the code conforming shear wall as a conventional design, the fourth model was considered with the concentrated plastic hinge at the base with the designation of PH.

Behavior of all other reproduced rocking sections over the height of buildings was assumed to resemble those at the base and no changes were made in properties. Although in another research [28], the inverted triangle distribution of forces were assumed for designing the rocking sections over height, for simplicity and comparison possibility of the results between different selected models, the uniform distribution of rocking section over height was provided in the present research. Dynamic characteristics of modal analysis (period of the first mode) obtained from the above-mentioned rocking structures are listed in Table 1, representing that uniform distribution of rocking section characteristics over the height of building and presence of axial loads from post-tensioning slightly increased the first natural period of buildings, in which the number of rocking sections was increased.

Table 1
Geometry and design outcome of considered structural walls.

		8 story	12 story	16 story	20 story
Wall length (mm)		6000	7500	8000	8700
Wall thickness (mm)		400	400	500	500
Floor height (mm)		3500	3500	3500	3500
Floor seismic mass or weight per wall (kN)		2500	2500	2500	2500
Mild steel dampers (each side)		30Φ20	30Φ24	30Φ30	30Φ38
Number of strand (each side)		29	25	24	23
Period (sec)	R-1:	0.78	1.22	1.97	2.05
	R-2:	0.81	1.26	2.04	2.09
	R-n/2:	0.89	1.52	2.07	2.64

Note: each strand has an area of 99 mm^2.

5. Parametric studies

To investigate the seismic behavior of multiple rocking systems on the local and global responses of the selected buildings, a comprehensive parametric study was planned. Thus, 9 significant seismic design criteria derived from the history analysis were assessed which included maximum inter-story drift ratio, displacement distribution, residual displacement, moment distribution, shear distribution, vertical elongation, stress concentration at rocking section toes, distribution of dissipated energy over height, and total acceleration distributed on floors. Moreover, the locations of dissipated energy device and post-tensioning tools across the rocking section were the two other significant parameters for the responses, which will be discussed in the following sections. For all the building shown in Fig. 5 with different stories reported in Table 1, response history analysis was carried out and compared.

6. Earthquake records

40 records were used in this research for response history analysis: 20 records (10 with two components) were selected to check the behavior of buildings at the level of DBE (an earthquake with the exceedance probability of 10% in 50 years) as the aim of the current design codes. The other 20 records were intended to check

the behavior of the proposed system from collapse and instability points of view at maximum credible level (MCE) (an earthquake with the exceedance probability of 2% in 50 years). This set of 40 records was suggested by Somerville et al. [34] for SAC project which intended to capture both average and variability of ground motions on firm soil in Los Angeles area. The set of records at the design level (DBE) consisted of the strike-parallel and strike-normal components of 10 ground motions recorded at the distance of less than 40 km from the earthquakes with moment magnitude between 6 and 7.3. The records of MCE also consisted of the strike-parallel and strike-normal components of ten earthquakes, 5 records were near-fault and the rest 5 records were simulated based on a physical model [34]. The magnitude of MCE records varied from 6.7 to 7.2. The design spectrum and scale of the selected records were obtained based on ASCE7-10 [35]. Characteristics and scaling factors of the selected records are listed in Table 2. The amount of scaling was related to the R-n/2 mode, because it had the maximum amount among the rocking structures. The Rayleigh damping of the structures was considered 3% for the three first modes.

7. Discussing location of energy dissipation and post-tensioning tools

In the design of rocking systems, the locations of energy dissipation and post-tensioning system have not been explicitly clarified in the available design codes and instructions. Some codes and standards such as the New Zealand Concrete Standard [29] and fib Bulletin [36] suffice to offer some recommendations in this field. These regulations only recommend that energy dissipation devices should be internal or external, placed either in the base section or between coupled panels, and rely on the relative vertical movement during the rocking motion of the wall. Accordingly, in the present study, a parametric study was implemented to investigate the effect of energy dissipation and location of post-tensioning tools on responses. For the sake of brevity, just two limit cases were reported. Fig. 6 shows the two different selected locations: one near the end of wall (0.9x model) and another at middle half-length of walls (0.5x model).

Table 2
Properties of selected records.

DBE records				MCE records			
Earthquake	Dist (km)	M_W	S.F.[a]	Earthquake	Dist (km)	M_W	S.F.[a]
Imperial Valley, 1940	10	6.9	1.6	Kobe, 1995	3.4	6.9	0.75
Imperial Valley, 1979	4.1	6.5	1.4	Loma Prieta, 1989	3.5	7.0	0.95
Imperial Valley, 1979	1.2	6.5	1.5	Northridge, 1994	7.5	6.7	0.85
Landers, 1992	36	7.3	1.8	Northridge, 1994	6.4	6.7	0.85
Landers, 1992	25	7.3	1.7	Tabas, 1974	1.2	7.4	0.75
Loma Prieta, 1989	12	7.0	1.1	Elysian Park[b]	17.5	7.1	0.8
Northridge, 1994	6.7	6.7	1.5	Elysian Park[b]	10.7	7.1	0.8
Northridge, 1994	7.5	6.7	1.5	Elysian Park[b]	11.2	7.1	0.8
Northridge, 1994	6.4	6.7	1.3	Palos Verdes[b]	1.5	7.1	1
North Palm Springs, 1986	6.7	6.0	1	Palos Verdes[b]	1.5	7.1	1.1

[a] Scaled Factor.
[b] Simulated.

7.1. Location of energy dissipation tools

To investigate the effect of the location of energy dissipation device on seismic behavior, the location of energy dissipation tools was changed from 0.5x to 0.9x models and the results of the performed analysis on aforementioned seismic design criteria were assessed. The results obtained in this research demonstrated that the response of the multiple rocking structures was highly sensitive to the location of energy dissipation tools. Results of parametric studies showed that the locating of energy dissipating tools at 0.9x model presented more reasonable responses than other locations. References of the comparison were shear and moment actions over height, displacement, and drift ratios along with acceleration at floor levels between the studied buildings. As a sample of results, the distribution of acceleration on floors and drift ratios for 12 and 20 story buildings under the DBE record is illustrated in Fig. 7. The results represented that changing the location from 0.5x to 0.9x drastically decreased the acceleration and reduced the drift demands. The closer the location of energy dissipation tools to the wall center, the more severe the rocking motion would be, which can lead to increased responses of rocking wall such as acceleration and drift. It should be noted that the results were the average of maximum obtained responses from the selected DBE records. Therefore, in this paper and the following discussion, only the results of model 0.9x will be reported.

7.2. Location of post-tensioning strands

To investigate the effect of post-tensioning strands on responses and find the appropriate location, in addition to other aforementioned criteria, elongation and yielding of the strands were investigated. To determine appropriate location of post-tensioning strands, two models of 0.5x and 0.9x were assumed as the representation of limit states. Elongation of the strands was important from damage point of view, particularly in the beams which might be connected to the panel of walls over the height of building. Average of maximum elongation of the strands and dispersion of the results for different rocking structures are illustrated in Fig. 8 both for DBE and MCE records. For the sake of brevity, only two types of buildings were reported here (i.e. 12 and 20 stories). An important criterion for employing post-tensioning strands is prevention from the yielding of strands both for DBE and MCE records to assure preventing sudden and brittle failure and guaranty re-centering behavior. Results of Fig. 8 show that, when the location of strands was changed from 0.5x to 0.9x, the potential of yielding and failure of strands was increased. For the two buildings at both DBE and MCE levels and for three different types of rocking system, a safety margin was achieved, at which the strands were positioned at 0.5x, while the likelihood of yielding was growing high when the strands were arranged at 0.9x. In addition to the location of the strands, the initial value of pre-tensioned force was also another challenge. Placement of the strands at the edges of the rocking section caused yielding the strands in taller buildings under MCE records and

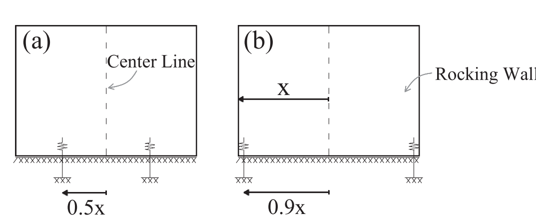

Fig. 6. Location of the energy dissipation and post-tensioning tools (a) 0.5x mode, (b) 0.9x mode.

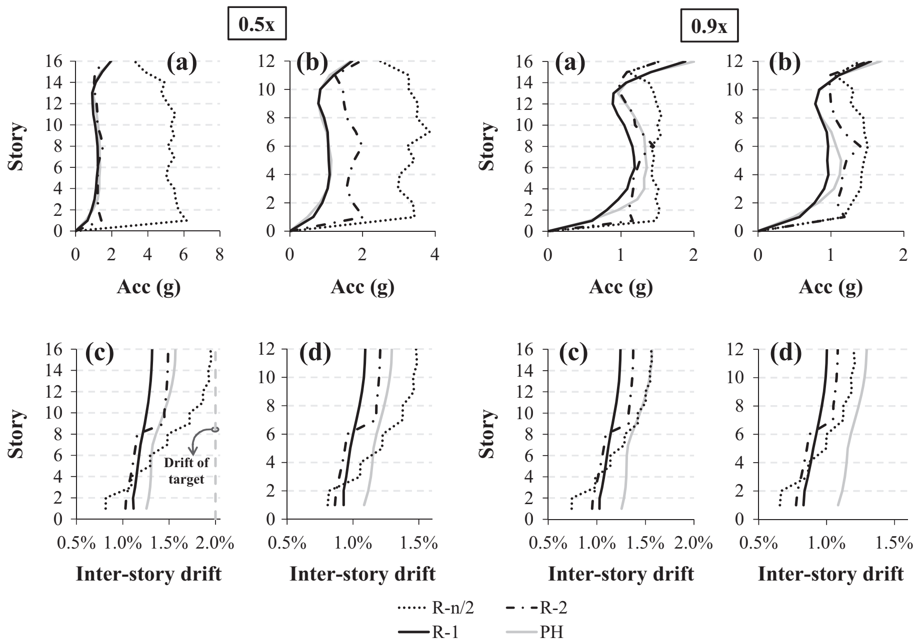

Fig. 7. The results of the average of maximum horizontal acceleration and inter-story drift for 0.5x and 0.9x models of energy dissipation tools under the DBE record.

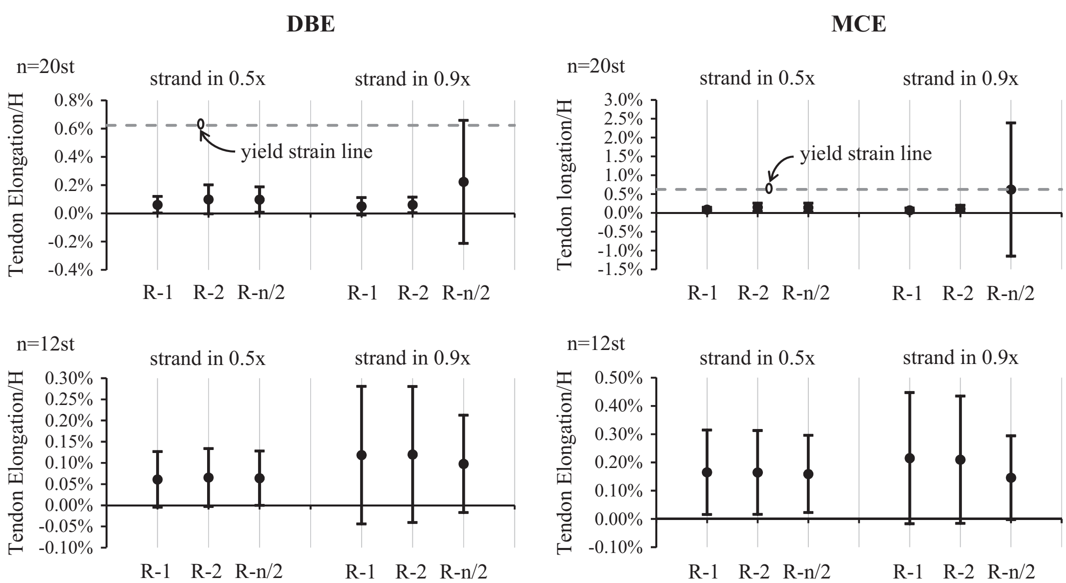

Fig. 8. The results of the average of maximum strand elongation and dispersions for 0.5x and 0.9x models under the DBE and MCE records.

substantially residual drift was probable. Thus, to avoid yielding and provide sufficient restoring force, in the current research and consistent with value recommended in the New Zealand Concrete Standard [29], the self-centering tools were designed at 0.5x position and pre-tension force was considered 25% yield stress.

8. Results of analysis

To evaluate the seismic performance of multiple rocking systems, in the following sections of the paper, seven more interesting seismic design criteria are reported. It should be noted that the results are related to the aforementioned location of post-tensioning and dissipation energy tools (i.e. 0.5x and 0.9x, respectively).

8.1. Results of inter-story drift ratio

In Fig. 9, results of the average of maximum inter-story drift ratio and standard deviation for all the buildings with 8, 12, 16, and 20 stories under the DBE and MCE records are presented. In this figure, the results of three rocking walls (i.e. base-rocking wall (R-1), the wall with two rocking sections (R-2), and the wall with a rocking section on every second floor (R-n/2)) along with the results of the plastic hinge model are reported.

Development of rocking sections over height caused the drift to increase in the upper half and decrease in the lower half parts

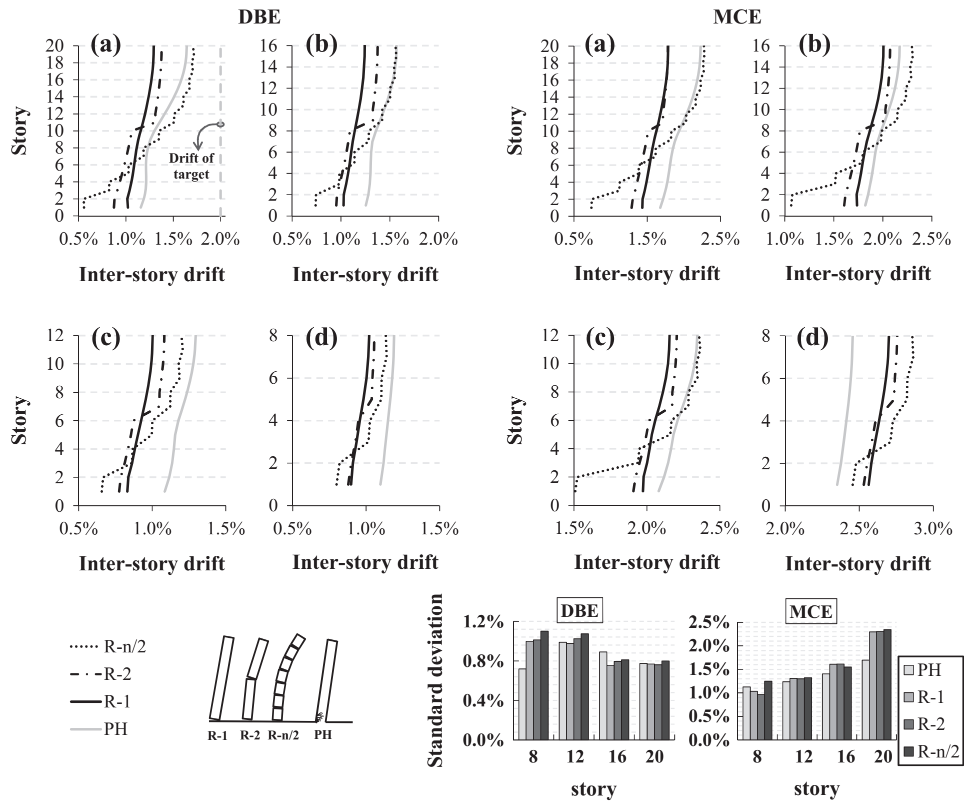

Fig. 9. The results of average of maximum inter-story drift under the DBE and MCE records.

compared with PH and R-1 models. In the 8-storey building, the results demonstrated that, under the DBE records, the drift of rocking structures was less than the PH model, while the drift of rocking buildings was more than the PH structure under MCE records. The important point was that, in most of the models under two suites of records, with the increase in structure height, the drift response of rocking structures tended to the values of PH models or even less.

The results of Fig. 9 show that, with the increase of structure height from 8 to 20 stories under DBE record, the results of inter-story drift and their standard deviations increased. Nevertheless, results of drifts in rocking buildings did not exceed 2%, which is the design drift value in the conventional seismic code designs. Under MCE records, with the increase in structure height, the results of drift and its standard deviation decreased and the average of maximum drifts in taller (20-story) and shorter (8-story) buildings was 2.4% and 2.8%, respectively. Although the values of maximum inter-story drift ratio at MCE level were more than 2%, based on the design criteria presented in tall building initiative (TBI) [37], in which the mean of the maximum drift ratios from the suite of nonlinear response history analyses under MCE level is limited to 3%, the outcome results were acceptable from design point of view.

8.2. Residual drift responses

In recent years, many researchers have recognized that the residual drift is a useful and advanced design criterion in performance-based design [38,39]. Nowadays, in both new seismic design and seismic assessment of the existing building, special attention is paid by researchers for mitigating damage of non-structural component and preventing residual displacement after earthquake (i.e. self-centering). Residual displacement is considered an important criterion and self-centering is one of the goals of DAD approaches. In Fig. 10, average of maximum residual drift, which is the displacement of stories, normalized by the building height for all structures under the DBE and MCE records are illustrated. Residual displacement is measured from response history analysis of displacement at the end of free vibration time.

Results of the analysis presented in Fig. 10 show that all buildings with different heights and rocking systems at both DBE and MCE earthquake levels had negligible residual drift compared with the PH model. Standard deviation of the residual values presented in Fig. 10 also shows non-significant values for rocking structures. The residual drift in rocking structures increased very slightly with the increase in the height of buildings and rocking sections. As given in Fig. 10, under both suites of records, the residual drift of the roof of the 20-storey structure in R-n/2 model was 0.02% of the structure height, which was negligible.

It should be also noted that the small values of resulted residual drift could be attributed to inadequate axial forces on upper floors to compensate for the restoring forces in energy dissipation tools. To overcome such behavior, changing metallic dampers to HF2V can be a useful method as was shown by past researches [40].

8.3. Elongation responses

In rocking systems, values of elongation are important and should be assessed. Rocking in sections causes additional displacement at the edge of walls; consequently, all the attached beams and floors (both in-plane and out-of-plane) are directly affected and significant damage is expected for these elements. On the other hand, rocking occurrence during an earthquake excitation causes the centerline of the wall to frequently fluctuate upward and downward; as a result, additional energy dissipation via movement of weight (or mass) is produced which is useful from

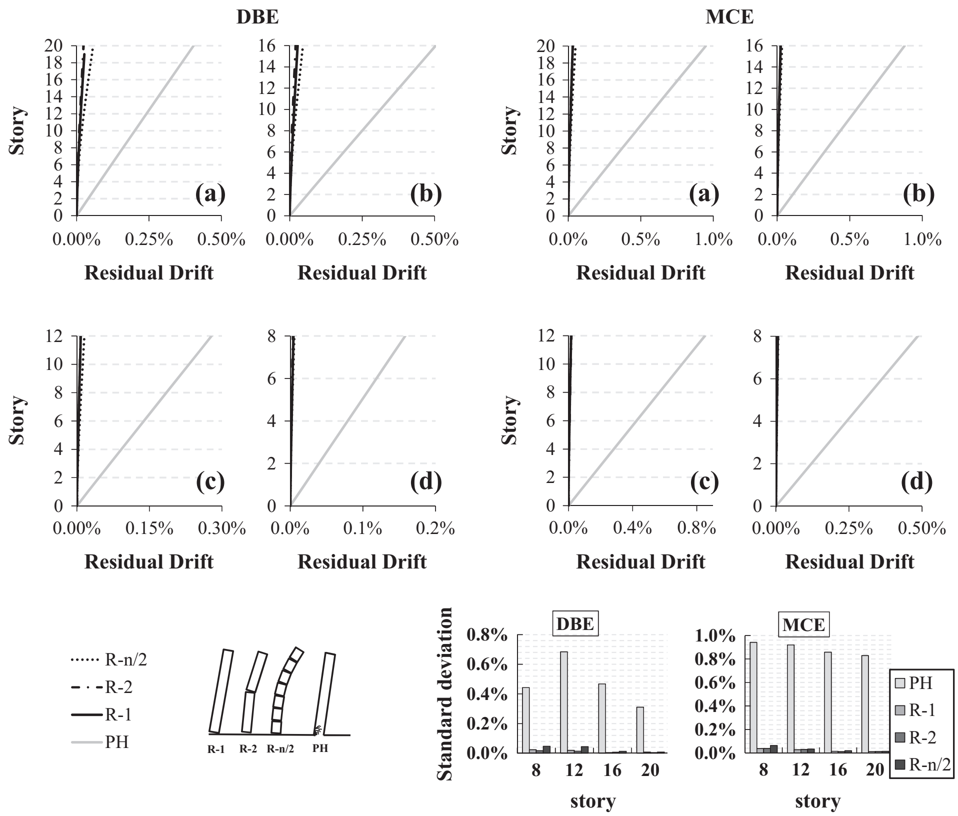

Fig. 10. The results of the average maximum residual drift under DBE and MCE records.

additional inherent damping point of view and reduction of displacement demand.

Average of maximum centerline elongation under the two groups of DBE and MCE records is shown in Fig. 11. In this figure, the horizontal axis was time and the vertical axis was the results of the vertical displacement of the roof level normalized by the total height of the structure.

The results presented in Fig. 11 show that the effect of the development of rocking sections (number of rocking sections) on elongation values was different in each structure so that the development of rocking system in shorter buildings (e.g. the 8-storey building) had no significant effect upon elongation values. However, an increase in the structure height and the development of rocking system caused elongation to decrease. Increase in the number of rocking sections in the 16-story building caused maximum centerline elongation from R-n/2 to R-1 models under DBE and MCE to reduce to 15% and 22%, respectively. As the height increased in the 20-storey structures, the maximum amount was reduced to 37% and 47%, respectively. It seems that increase in the system damping due to the fluctuation of masses could be a main reason for this reduction. Therefore, in taller buildings, using multiple rocking systems will be more efficient than low rise ones from the elongation point of view. It is important to note that these results occurred just in the case that the energy dissipation tools were used near the edges of walls (i.e. 0.9x models).

The results of Fig. 11 reveals that the rocking walls have no residual elongation at the end of earthquakes because of self-centering behavior, while significant cumulative residual elongation occurs on the plastic hinge wall. In addition, the residual elongations of plastic hinge walls gradually increase with increasing the height of wall. This behavior is one of the shortcomings of PH walls that may be irreparable.

8.4. Shear responses

Capacity design approach is prescribed for designing new shear wall buildings. Slender shear walls are designed to behave in a flexure mode and the shear strength capacity should be adequate for developing flexural displacements. From the design point of view, the amount and distribution of shear forces are important in the current seismic design codes and performance-based design approaches such as Eurocode 8 [32] and ASCE-41 [33].

Average of maximum and distribution of shear forces under DBE and MCE records are shown in Fig. 12. To better compare the results, they were normalized by the total weight of the wall. The results of Fig. 12 showed that, with the increase of rocking sections over height, with the exception of the 8 story building, values of shear forces were smaller than PH and R-1 models, which in turn represented that the current design codes and shape of shear force distribution were more regular than those without the second increase in upper half-height. With developing rocking sections, the amounts of shear forces through upper half part of the buildings tended to be constant. The difference between the values of shear force for PH and R-1 models was approximately the same and the exception was the 8-story building, in which the values of R-1 model were slightly more than those of the PH model. The behavior of the 8-story building in terms of both values and distribution shapes of shear forces over height was different from other three buildings (with 12, 16, and 20 stories). The results showed that, with reducing the building height, neither shear forces nor their distributions were significantly affected by rocking system, a case which was seen in the 8-story building.

Results of higher buildings (with 16 and 20 stories) showed that, under both sets of records, the shear force envelope of the R-1 model was almost similar to the PH model and the shear

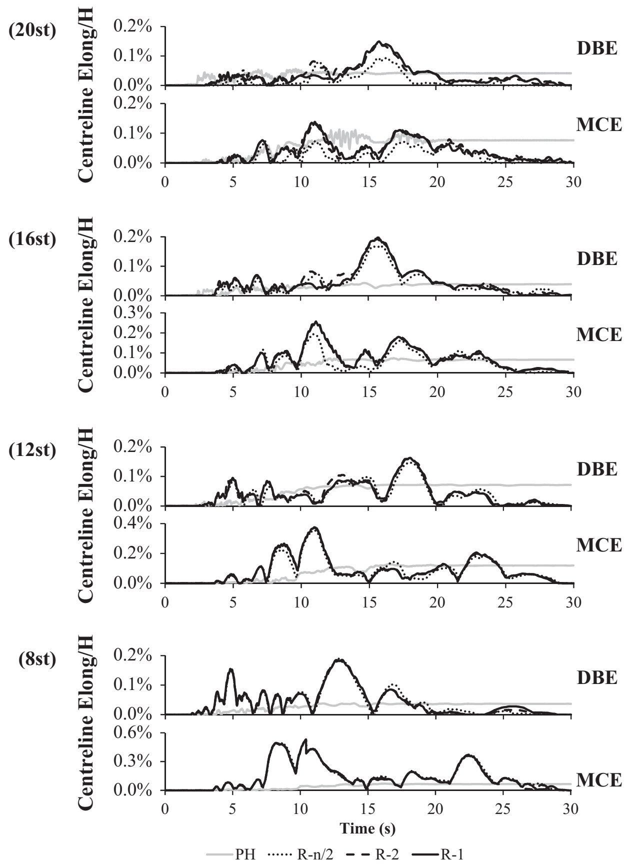

Fig. 11. The average of maximum centerline elongation of rocking walls under the DBE and MCE record.

envelope of the R-n/2 model was reduced in comparison with other models, particularly in the upper half of the structures. As an example, the shear forces in the upper half from the R-n/2 model to R-1 model under DBE and MCE records decreased by 34% and 71%, respectively. Change in shear force distribution from "S" shape in the PH and R-1 models to bi-linear (i.e. linear from the base to mid-height and constant over the upper rest of height) regarding the presence of rocking sections (R-2 and R-n/2) implied that, in 12, 16, and 20-story buildings, the prescribed distribution of shear forces by Eurocode8 [39] for the shear wall with a plastic hinge at the base was also applicable here.

8.5. Bending moment response

The current design regulations emphasize forming the plastic hinge at the base of the slender shear wall and implicitly or explicitly preventing the formation of another plastic hinge over height such as ACI-318 [41] and Eurocode8 [32], respectively. Previous studies have shown that the effect of higher dynamic modes and their contribution in response to structure are not negligible, particularly in long period structures both in linear and nonlinear behaviors [42–44]. The higher mode effects increase the action demands at mid-height of shear walls.

Fig. 13 shows the mean of maximum bending moment under the DBE and MCE records. The reported results were normalized by the product of the building weight and height ($W * H$) which was a moment that can be generated at the base by applying a point load to the roof level with magnitude equal to the total seismic weight of the structure.

The results presented in Fig. 13 show that, with developing rocking system at building height, the bending moment demands were reduced compared with PH and R-1 models. In all the buildings and under two sets of records, the PH and R-1 models showed that the maximum moment demand was created at the mid-height of buildings, instead of the base. Values of the moment demands related to R-1 models were more than those of the PH model, particularly at lower parts of buildings. In all the four models and different buildings, the natural period of R-1 models had minimum values and hence the bigger values of bending moment demands were expected. A comparison between R-2 and R-n/2 models revealed that the values of bending moments of the R-n/2 model were less and the distribution of moments was almost linear over height compared with the R-2 model. However, using two rocking sections (R-2) seems to be logical for mitigating effects of higher modes. Using rocking sections in 20-storey structures, the maximum bending moment was reduced by 66 and 67% in R-n/2 model compared with R-1 model under DBE and MCE records, respectively. it is noted that, PH and R-1 walls were designed according to Eurocode-8 in such way that no plastic hinge formed at height and demand moments over the height have to be less than plastic moment at base. However the results of Fig. 13 for these two models show that the moment over the height of walls are more than plastic moment at base. This case was reported earlier by other researchers. Studies done by Panagiotou and Restrepo [43] show that current linear design envelopes recommended by capacity design codes do not provide sufficient protection against yielding in the upper portions of the walls as intended in their design concept. However in current research, it is assumed that upper part of walls be stiff and strength to resist moment demands.

As a result, except PH and R-1 models, other rocking models successfully controlled the higher mode effects and produced approximately linear distribution of moment demands over height, which is important for developing new seismic design provisions.

8.6. Concrete crushing stress at contact edges of rocking sections

One important point in rocking section behavior is pounding on contact surfaces. Pounding of two contacted segments during reversal response can cause concrete crushing at edges and consequent reduction in rotation capacity and axial and lateral strength capacities. This phenomenon has been the subject of some recent experiments. The results of shaking table tests conducted by Whittaker et al. [45] indicated that the presence of energy dissipation systems and inherent damping can control the impact behavior and pounding at edges of concrete panels and this issue will not be worrisome. Moreover, Hamid and Mander [8] showed that by using steel plate or plastic sheets at contact surfaces of the panels, damage to edges of the rocking sections can be minimized. It is important to note that logically the concrete in the boundary region has to be confined with adequate lateral rebars to prevent any crushing of concrete and provide both higher strain and stress capacities, which was proved by the experimental test results reported by Resterpo and Rahman [21]. Although the well-confined core concrete in the boundary element of the tested walls has shown a satisfactory behavior [21] from technical point of view, in the present research, the results of contact stresses derived from the analysis were reported and discussed.

Fig. 14 illustrates that the average of maximum concrete stress at the edges of sections on the base is expected to reach the maximum in all the rocking structures during the DBE and MCE records. The results are shown for 4 groups of 8, 12, 16, and 20-story buildings and normalized by the yield stress of confined concrete.

The results presented in Fig. 14 demonstrate that, in all the rocking sections in all structures except at the base, the amount of stresses was gradually reduced from the base level up to the

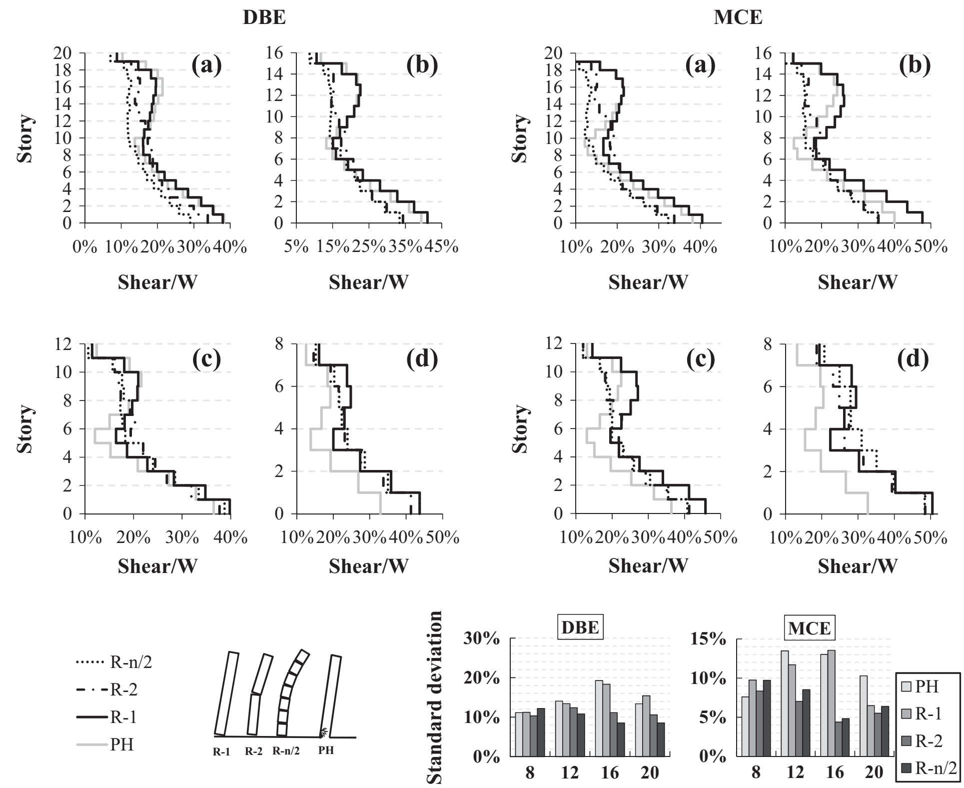

Fig. 12. The average of maximum shear forces under DBE and MCE record.

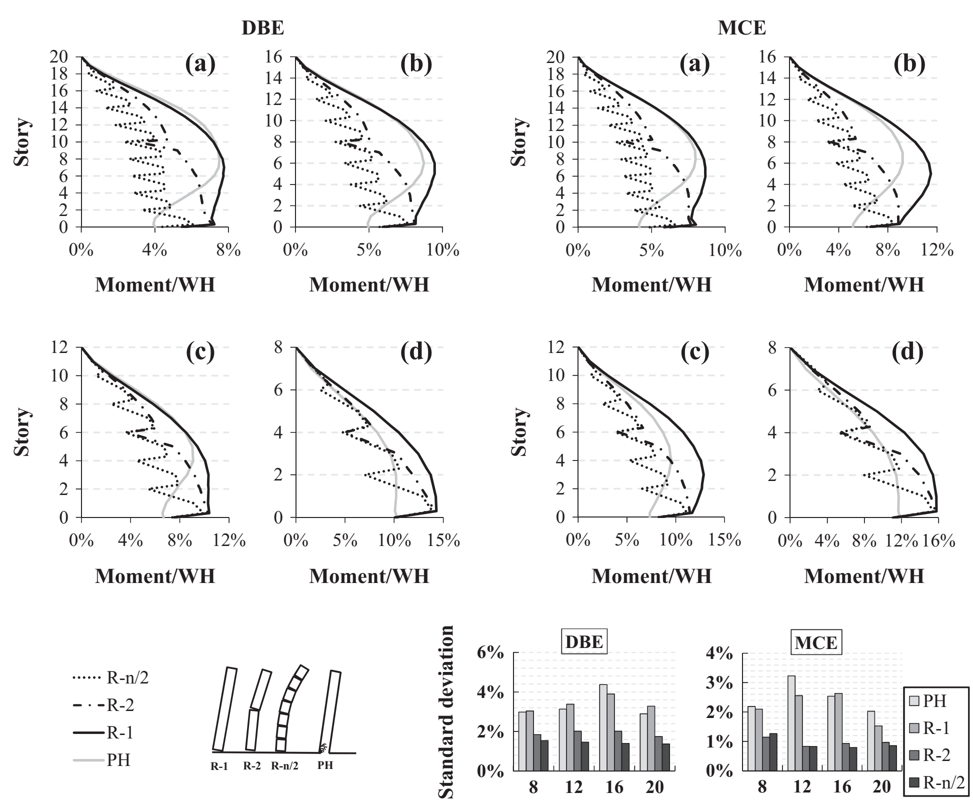

Fig. 13. The mean of maximum bending moment under the DBE and MCE record.

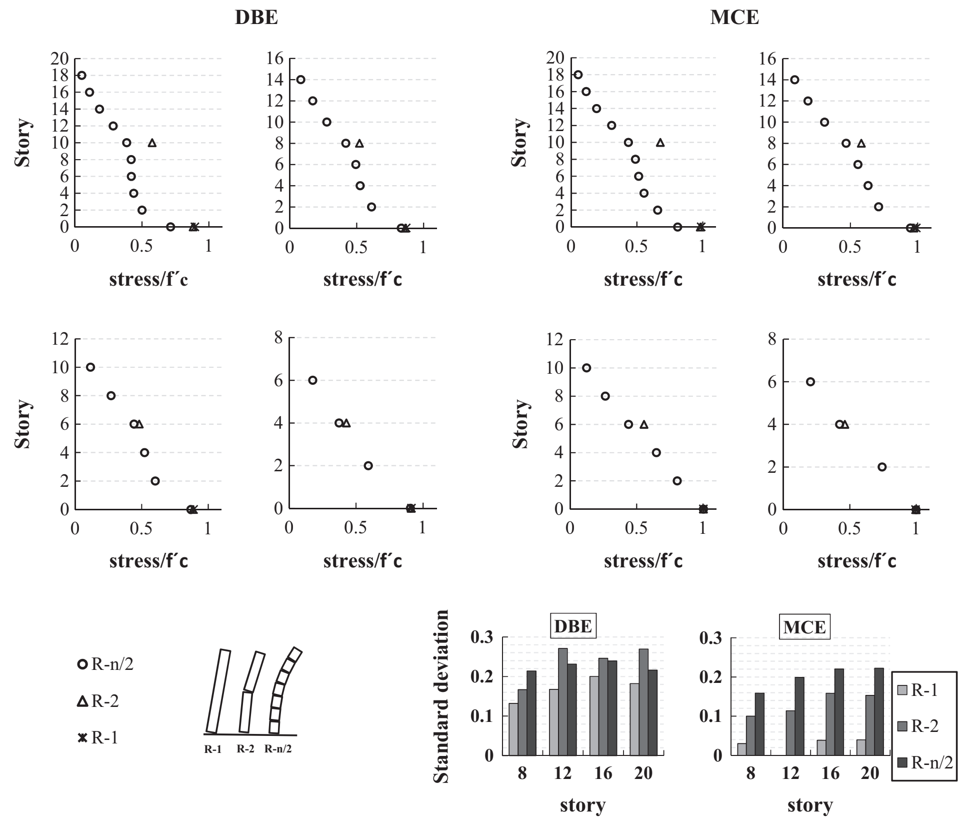

Fig. 14. The mean of maximum stresses of concrete at edges of rocking sections under DBE and MCE records.

highest rocking sections, which was consistent with gravity load distribution on the wall panels.

Hence, the maximum suffered stresses are in the rocking section at the base. The results showed that the concrete of the rocking sections did not attain the yield point in any of the models under the DBE records. Under the MCE records, stresses of the rocking sections at the base level experienced yield values in all models, except in the structures with 16 and 20 stories of R-n/2 model. In taller buildings and with the increase in the number of rocking sections, the displacement demands were developed among more sections compared with shorter buildings; consequently, the concrete stress demands at the edge of panels were decreased. Another noticeable point was that, when height increased, the amount of stresses in the lower sections was reduced under the two suites of records, while it slightly increased in the middle of height, which caused the stress distribution over the height to change from linear distribution in 8-story structures to trapezoid-shaped distribution in 20-story structures.

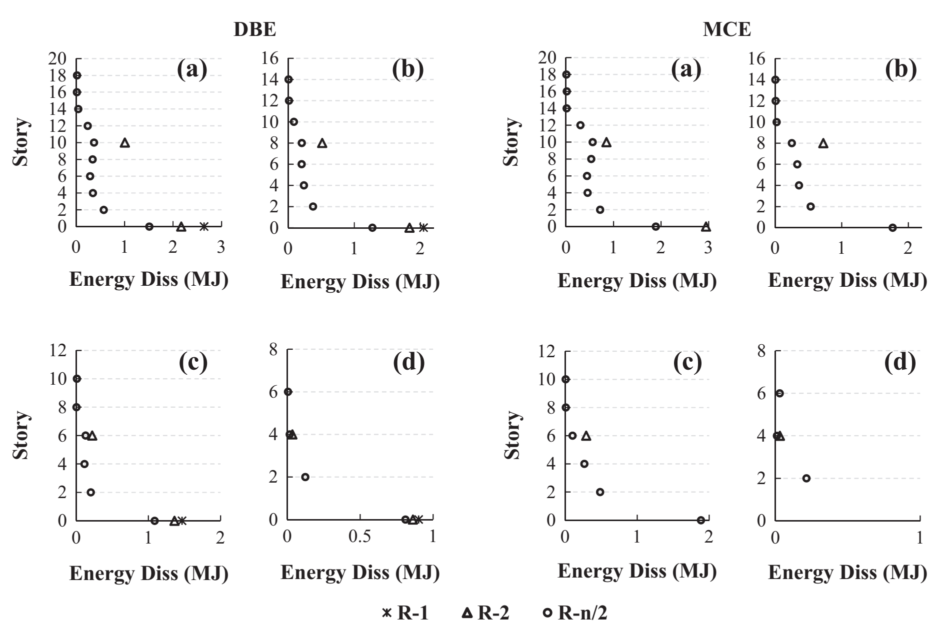

Fig. 15. The mean of cumulative energy dissipation at rocking sections under the DBE and MCE records (units are Mega Joule).

8.7. Energy dissipation in rocking sections

Distribution of energy dissipation in rocking sections over height can be useful in performing optimal design of sections. As noted earlier in the present research, the behavioral characteristics of base rocking sections have been attributed to all other sections over height; hence, the values and distribution of energy distribution over height can help to provide suitable arrangement of section locations.

Average of cumulative dissipated energy by dampers (mild steel rebars) in each rocking section under the two sets of DBE and MCE records derived from hysteretic behaviors is shown in Fig. 15. The results depicted in this figure show that the majority of energy dissipation occurred in the base rocking sections and the other sections over height dissipated lower energy. In general, sections of upper one-third of the structures had less contribution in the dissipation of energies, which was particularly applicable for R-n/2 models in both BDE and MCE records. In the R-2 models, the ratio of energy dissipated at the base to mid-height of the 8-storey structures under DBE and MCE records was 24 and 60, while for 20-story models, it was 1.1 and 2.4, respectively. Therefore, using multiple rocking section systems from the energy point of view could be more efficient in taller buildings compared with shorter ones (the studied cases included 16 and 20-story buildings). As noted in the previous sections, the effects of higher modes can be one of the main reasons for such a behavior. The energy dissipation distribution for 8, 12, and 16-story buildings was linear and varied from linear to constant for 20-story buildings from the first story up to the stories at the two-third of height. Therefore, one can ignore the implementation of the rocking sections at the upper one-third of the structures to reduce the construction costs. In this study, all the energy dissipation tools experienced a nonlinear behavior; however, they did not reach the ultimate strain capacity, which was assumed %7.

9. Conclusions

In this paper, to investigate the effects of using multiple rocking sections over the height of shear wall buildings, a series of nonlinear history analysis was performed. To this end, four buildings with different stories (i.e. 8, 12, 16, and 20 stories) were selected and developed for rocking sections over height. Using refined and precise models, the responses of the considered buildings were investigated. The main findings of the performed analysis were:

- Responses of the studied buildings whose locations of post-tensioning strands and energy dissipation devices were set at 0.5x and 0.9x, respectively, were more efficient than other locations.
- If the locations of post-tensioning strands and energy dissipation devices were 0.5x and 0.9x, respectively, the horizontal acceleration would remain almost constant with the increase of rocking sections over height.
- All the studied buildings under both DBE and MCE records passed the drift criterion of current design codes (2% and 3%, respectively) and the results showed that the residual displacements in all the studied structures were negligible.
- The higher mode effects on shear and moment action demands were mitigated using multiple rocking system. Shapes and values of shear force distribution changed from "S" to bi-linear and also moment distribution tended to be linear from the base to roof.
- Results of the analysis on the studied buildings indicated that development of rocking system in shorter buildings (e.g. 8 stories) had no significant effect upon elongations, while in taller buildings, it caused a decrease in elongations.
- No concerns were raised on pounding at contacted surface over height in rocking sections; only the concrete behavior at the base of shorter buildings (i.e. those with 8 and 12 stories) became nonlinear at MCE level, which was tolerable by the confined concrete.
- Results of hysteretic energy dissipation in the rocking sections over height showed that the presence of these sections over height in taller buildings (i.e. 16 and 20 stories) with more than one rocking section (R-2 and R-n/2) can lead to higher efficiency. The upper one-third rocking sections were not very active in the performed analysis.

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