The Necessity of Transverse Steel Reinforcement for Confinement in Structural Reinforced Concrete Walls using Nonlinear Static and Dynamic Analysis Method

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Abstract - *Reinforced concrete walls are one of the most important and useful earthquake-resistant systems. Regarding provide adequate performance against seismic forces, its ductility should be provided by considering some design principles. Since confining the concrete will increase the ductility of the reinforced concrete member, designing instructions try to increase the ductility of the wall by specifying a certain length of wall edges to implement specific transverse steel reinforcement in this area. In this study, the need for the transverse steel bars to apply confinement in concrete is compared with the equations suggested by previous studies for the displacement-based design of structural bearing walls. For this purpose, nonlinear static analysis and nonlinear time history analysis was utilized. The results of the study indicate that the lateral deformation of the structures is less than the final limit specified by the regulations, even without considering the transverse steel bars for concrete confinement*

Key Words: Reinforced concrete structural wall, Nonlinear dynamic analysis, Pushover, OpenSees

1. INTRODUCTION

On March 3, 1985, an earthquake measuring 7.8 on Richter occurred in an area near the central coast of Chile. The city, which influenced by the earthquake, was the city of Vienna Delmar, with about 400 modern concrete reinforced buildings designed and built on engineering principles [1]. Most of the buildings had numerous reinforced concrete structures designed to deal with lateral forces. Initial reports indicated that the buildings had shown a very favourable performance so that the damage was very low, and in many cases, they were unhurt. Further research [2] showed that although the seismic requirements of the Chilean statue were similar to the U.S. requirements for high-risk areas, the required details by the Chilean regulations were not strict,[1]; also, the Chilean monitoring was less strict comparing to the U.S. Standards. Those buildings in Chile, according to the U.S. classification regulations (such as the Uniform Building Code UBC-88), were in the group structure with a load-bearing wall. Regarding these regulations, the design forces for this group were greater than the flexural frame system and the dual system. Also, the details needed to form doing more plasticity and more accurate monitoring for the load-bearing wall system, compared to the other two systems was mentioned. Consequently, due to the high cost of implementation of great details and requirements for increased structural resistance, the performance of the bearing wall system was rarely economically justified in the United States, while the bearing wall system operating in Chile was completely economical.

There are various parameters that can affect the structural behavior of systems. For example, the geometrical variables and materials used in members playing a significant role in structures,[3]–[6]. On the other hand, many researchers showed that ductility of the lateral load-carrying structural systems has a significant influence on the performance of the building in earthquakes. For instance, Alimohammadi and Lotfollahi Yaghin (2019) [7] studied the seismic behavior of light steel frame (LSF) structures using two lateral load bracing systems and LSF shear walls. They studied and compared different seismic behavior factors such as ductility, added resistance coefficient, coefficient of hardness, etc., of these two different lateral resistance systems using nonlinear pushover analysis. They compared their presented coefficient of ductility for various variables in their study, and they showed the accordance of them with the Iranian design regulations for all seismic zones. Their study results revealed that the more deformable lateral resistance building systems could be more building performance efficient in all seismicity zone as well. In another study, Alimohammadi et al. (2019) [8] studied the effects of different shapes of openings on seismic behavior of the concrete shear walls using nonlinear static analysis (Pushover). They compared the changing of many seismic factors with the opening shapes in the lateral systems, such as the results of the ductility coefficient, energy absorption, hardness, added resistance, and so on. From their simulation results, they concluded that the various shapes of openings affect the resistance and the hardening of shear walls. The effect of lack of the shear walls on the seismic performance and properties of short, medium, and tall buildings is harder than the shear wall with the openings. Also, the closer the openings to the edge of the wall showed the more decrease in strength and ductility [8]. In lateral resistance systems, it is obvious that the most critical columns are those located at the nearest external frame of the structure, should be considered as highly important members in progressive collapse potential reduction as key members [9] and Increasing the stiffness of the column head results in the decreasing of the displacement of the column header as well [10].

According to the description given, as well as the proper functioning load-bearing wall systems, the researchers concluded that load-bearing walls with limited details could be considered as an effective and economical system against earthquakes [11].

2. Displacement -based design method of structural wall

According to the previous studies, [11] presented the method of displacement-based design of structural walls. In this design method, the need for concrete confinement in the boundary components of the wall is related to the expected response from the structure. They indicated that regulations of the U.S. are very conservative for a wide range of systems used in which structural walls are utilized to deal with the lateral loads. Wallace and Mull also found that the primary variables are influencing wall details such as the ratio of the total area of the walls in one direction to the floor area, the wall aspect ratio (height to length ratio), wall configuration (cross-section), axial force and the ratio of wall longitudinal reinforcement [11]. Displacement - based design method requires a simplified relationship for the displacement range. To achieve this spectrum, the acceleration spectrum is needed, which can be achieved by available codes such as ASCE standards [12]. In this study Iran's standard for seismic design is utilized [13]. Assuming the region with high seismicity risk (A = 0.3) and type III of the land ($T_0 = 0.15$ and S = 1.75), and with Eq. (1) it is possible to obtain the elastic displacement range at different times. In figure (1), in addition to the elastic displacement spectrum, the simple displacement range is also illustrated in Eq. (2) As noted, this simplified range, which is linear, is the curve of the elastic displacement spectrum. Consequently, it can be used to compute the maximum displacement at different periods.

$$S_d = \frac{T^2}{4\pi^2} S_a \tag{1}$$

$$S_d = 0.25T \tag{2}$$

The fundamental time period of the structures with R.C. walls can be estimated by analyzing a cantilever beam [14]. On the reinforced concrete structural walls, by considering the effect of stiffness reduction caused by concrete cracks, the variation of fundamental time in the structure can be estimated using in Eq. (3) where n is the number of stories, w is the weight of the story level, h_s is the average height of stories, E_c is the modulus of elasticity of the concrete, h_w is the wall height, l_w is the length of the wall, p is the ratio of the total area of walls to floor area, and g is the gravitational acceleration of the earth [15].

$$T = 8.8 \frac{h_w}{l_w} n \sqrt{\frac{w h_s}{g E_c p}} \tag{3}$$

The validity of Eq. (3) has been approved by Wallace and Mull by measuring the time of the main period of reinforced concrete structures. Figure (2) shows the variation of the fundamental time period, according to Eq. (3) assuming w=640 kg/m and E_c=27E+08 kg/m^2.
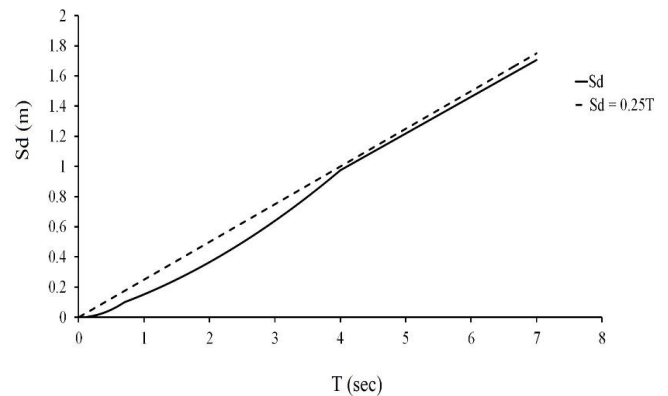

**Fig- 1** Diagram of elastic displacement spectra

Considering the difference between a free degree systems with a real building that is a few degrees free, the roof displacement can be 1.5 times the spectral displacement. Accordingly, with Eq. (2) and (3) the relative displacement (drift) is achieved at the roof level from Eq. (4), with the help of this relationship, the ultimate curvature value for the wall is calculated from Eq. (6) and (7) [15]. Figure (3) represents the way of changing the relative roof displacement to different proportions of the total area in the wall of one direction (p).

Figure (4) shows a typical cross-section of the reinforced concrete structural wall. For this section, the maximum strain of concrete can be obtained from Eq. (7) [15].
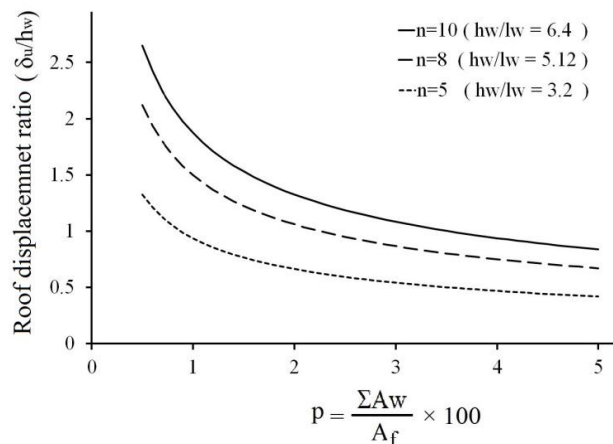
$$\frac{\delta_u}{h_w} = 0.00029 \frac{h_w}{l_w} \sqrt{\frac{1}{p}} \tag{4}$$

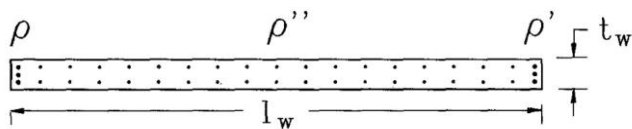$$\varphi_u l_w = 0.0025 \left(1 - \frac{1}{2} \frac{h_w}{l_w}\right) + 2 \frac{\delta_u}{h_w} \tag{5}$$

$$\varphi_u l_w = 0.0025 + \frac{h_w}{l_w} \left(\frac{1}{1707\sqrt{p}} - 0.00125\right) \tag{6}$$
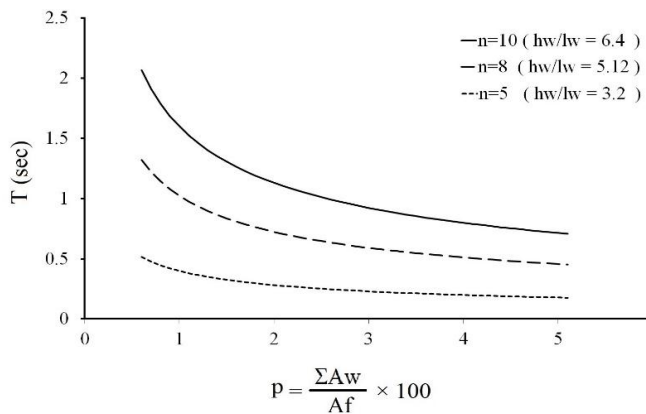
Fig-2- Structural natural period for different ratios of wall area to the floor area

Fig-3 Relative roof displacement for different ratios of wall area to the floor area

Fig-4 Typical section of reinforced concrete wall

$$\varepsilon_{cmax} = \left[\frac{\left(\rho + \rho'' - \frac{\gamma}{\alpha}\rho'\right)\frac{\alpha f_y}{f_c'} + \frac{P}{l_w t_w f_c'}}{(0.85\beta_1 + 2\rho'' \frac{\alpha f_y}{f_c'})}\right]\varphi_u l_w \qquad (7)$$

In the above equation ε_{cmax}, the maximum stress of strain in concrete, ρ tensile reinforcement ratio equals to $A_s/t_w l_w$, ρ' the ratio of the compressive reinforcement is $A'_s/t_w l_w$, ρ'' the ratio of distributed reinforcement in the wall web is $A''_s / t_w l_w$, β_1 is a reduction factor for utilizing uniform equivalent stress, f_y is the yield stress of the steel rebars, f_c is the ultimate compressive strength of the concrete, P is the axial force, L_w is the length of the wall, t_w is the wall thickness, α=1.5 and γ=1.25 the coefficient of over strength and strain hardening effect of steel for tensile and pressure,

respectively and φ_u is the ultimate curvature. By accessing to the ultimate curvature amount (φ_u) the maximum strain of concrete can be predicted in the wall cross-section.

3. Numerical models and design of reinforced concrete walls

In this study, in order to investigate the necessity of transverse rebar in the boundary areas of a concrete structural wall, a simple reinforced concrete structure is considered, as shown in figure 5-a. The structure is designed in two height of five and eight stories in which the height of all stories is considered 3.2 meters. The lateral load resisting system in both directions is R.C. structural walls wit rectangular cross-section. The ratio of the total area of the wall to the area of the floor in the y-direction is constant and equal to 1.9 %. In the first model, eight walls in the y-direction are assumed shown in figure 5-b. In this case, the walls' behavior will be as a shear wall. Due to the low number of walls in the plan, the lateral contribution forces will be higher than the gravity force. In the other two models, shown in figures 5-c and 5-d, by keeping the total area of the existing walls on the floor area resulting in thinner walls, the number of walls has increased. As a result, their portion from lateral forces is reduced, and their performance will be like a bearing wall. In all models, two-way reinforced concrete slabs are considered a floor system. Dead load is 600 kg/m2 and live load is 200 kg/m2. The properties of the material used in the design of the models are presented in Table 1. In modeling of structures, the stiffness of the walls has been neglected in the weaker direction as well as the frame and wall interaction, and it is assumed that lateral forces are resisted only by structural walls. The models are designed utilizing ETABS 9.7.4 software [16]. The longitudinal reinforcement is distributed uniformly along the wall cross-section. In this study, the ninth edition of Iran's national building design standard is utilized.

Table 1- properties of materials used in model design

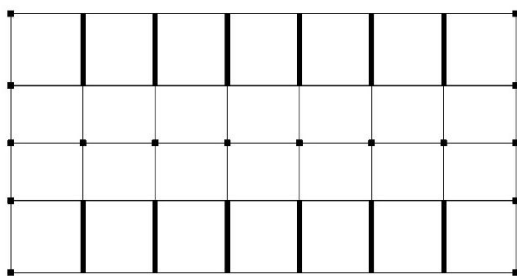| Concrete		Steel Reinforcement	
f_c (Mpa)	E_c (Gpa)	f_y (Mpa)	E_s (Gpa)
25	26.52	400	210

In the design of models that have the same number of stories, in order to have the correct comparison of results in the nonlinear analysis, the ratio of demand to the capacity of the wall (D/C) is maintained, unless it is necessary to put the minimum ratio of longitudinal bars (ρ = 0.0025).
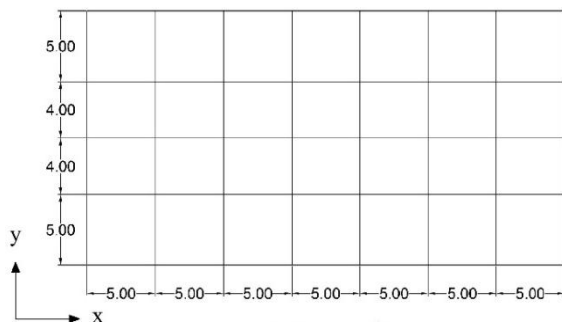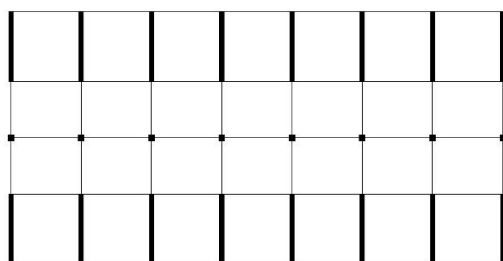
(a)
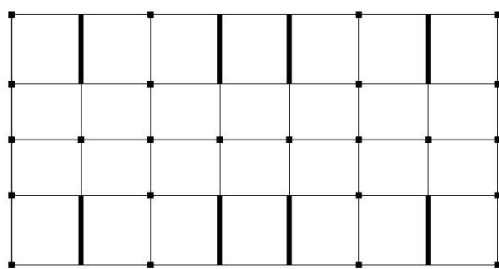
(b)

(c)

(d)

Fig-5 Hypothetical model for reinforced concrete structures: (a) dimensions of the building plan, (b) structures with 8 shear walls, Figure (d) structures with 12 bearing walls, Figure (d) structures with 16 bearing walls.

The results obtained for the design of the walls with their geometrical specifications are expressed in Table 2. In this table, the models are named SW-m-n, where m represents the number of walls in the y-direction, and n represents the number of stories of structures. Also, in this table r is equal to the apparent wall ratio, t_wwall thickness, ρ, ρ' and ρ,'' as shown in Fig. (4), respectively, as the ratio of the longitudinal bars in the tensile, compressive, and wall areas. The l_c and h_c is the length and height required for trapping of the reinforcing according to the criteria mentioned by the design code regulations. Figure 6 shows, for example, the maximum amount of concrete compressive stress, as well as the length and height needed to create transverse reinforcement (l_cand h_c) for the SW-8-8 model. In Table (2) the periodicity obtained from linear analysis (T_a) with the results for the periodicity of relation (3), (T_c), as well as with the periodicity obtained from the proposed standard relation 2800 [13], (T_c), is compared. As can be seen, the results obtained from relation (3) provide a relatively closer evaluation of the main structural natural period than the proposed standard 2800 relation [13].

With the help of Eq. (6) and (7) referred to in the previous section as well as using the p, p' and p'' in Table (2), the ultimate elongation plot of the concrete is obtained for different proportions of the total area of the wall area (p), in Fig. (7) and (8), respectively, for structure 5 and 8.

As can be seen, none of the designed models will require transverse reinforcement for concrete confinement since the maximum strain created in concrete will be less than the final strain of non-confined concrete (ε_c = 0.0035).
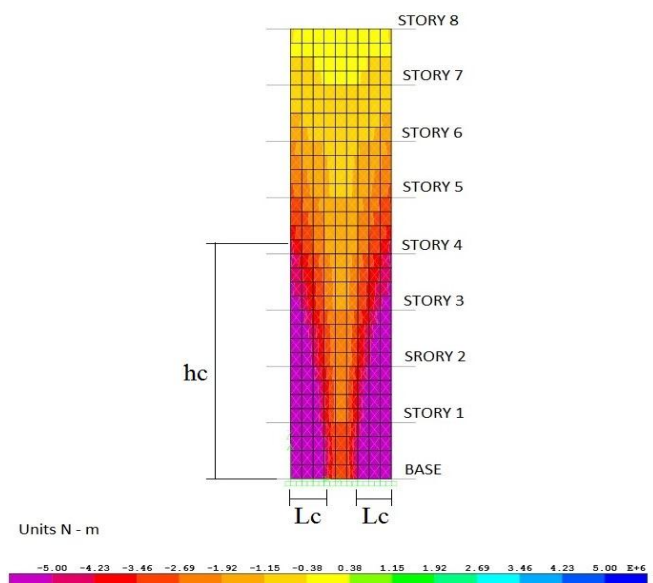
Fig-6 Length and height required for transverse reinforcement enclosures for SW-8-8

Table 2- Specifications of designed models

Model	$r = \dfrac{h_w}{L_w}$	t_w (m)	Story	Longitudinal Reinforcement	$\rho = \rho'$ (%)	ρ'' (%)	L_c (m)	h_c (m)	T_3 (sec)	T_c (sec)	
SW-8 -5	3.2	0.3	all	TO 12 @ 30	0.015	0.28	0.3	3.2	0.290	0.400	0.348
SW-12-5	3.2	0.2	all	TO 12 @ 30	0.022	0.43	0.5	4.0	0.290	0.400	0.348
SW-16-5	3.2	0.15	all	TO 12 @ 30	0.030	0.57	0.8	5.6	0.290	0.400	0.348
SW-8 -8	5.12	0.3	1-2	TO 18 @ 25	0.033	0.64	1.6	12.8	0.742	0.569	0.840
			3-4	TO 14 @ 25	-	-					
			5-8	TO 12 @ 25	-	-					
SW-12 -8	5.12	0.2	all	TO 12 @ 25	0.022	0.43	1.96	13.2	0.742	0.569	0.841
SW-16-8	5.12	0.15	all	TO 12 @ 25	0.03	0.57	2.0	16	0.742	0.569	0.841

However, according to the criteria of the design code, each model following table (2) requires trapping during different heights. To investigate this difference, in the following sections, static and dynamic nonlinear analysis, the maximum concrete strain, as well as the relative displacement [8], [9], [16] of the stories for each model without any confinements are evaluated.

4. Nonlinear Static Analysis

Nonlinear analysis requires proper analytical models for structural members, including reinforced concrete walls. Various methods have been proposed by researchers to simulate the nonlinear behavior of concrete structural walls [6], [8], [17]. These methods are mainly divided into three parts, including small scale methods, medium-scale methods, and large-scale methods. In small scale models such as the finite element model, the wall is divided into the limited number of small elements connected at a finite node.
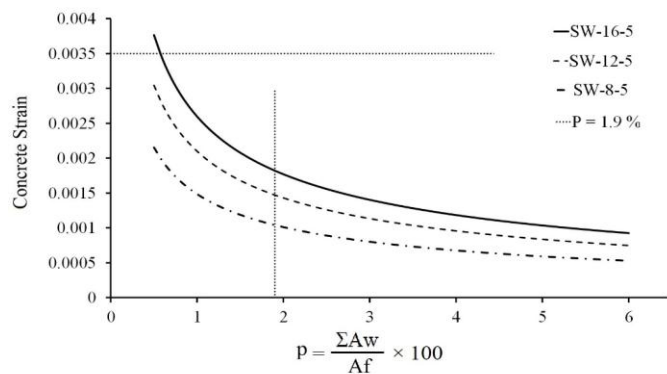
Fig-7 Diagram of changes in concrete strain versus wall-to-plan ratio changes for 5 story structure
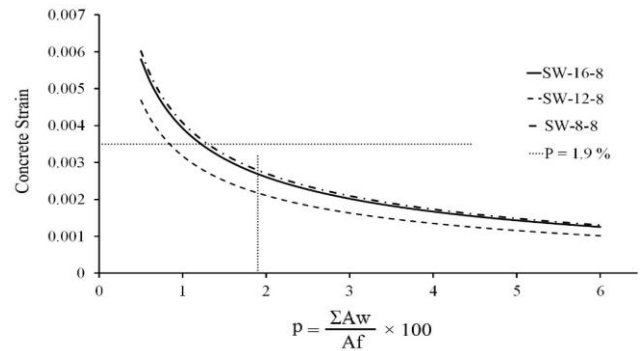
Fig 8- Diagram of variations of concrete strain against wall-to-area ratio changes for 8 story structure

Although the Finite element method has very high precision and can consider the simultaneous effects of axial force, shear force, and bending anchor well [18], this model requires solving a large volume of complex equations and consequently very long duration. Therefore, it is not possible to apply this method for large structures in practice.

Medium-scale models were based on some structural theories (fiber model of beams). In this method, motion conditions and equilibrium are calculated on the original scale, while stresses and internal variables are calculated on the local scale. The average-scale models, on the one hand, allow the use of a simple hypothesis of the beam's theory (Navier-Bernoulli or Timoshenko) that would significantly reduce the bulk of the equations. On the other hand, integration allows the rapid integration of compliance laws in the context of uniaxial strain stress theory [19].

Large-scale methods generally have different types, such as single-component model, two-component model, multi-axis spring model, truss model, multi-vertical linear model (MVLE), and so on. Large scale methods are based on displaying the overall action of shear walls such as wall

deformation, resistance, and energy absorption capacity. In this study, Vertical Multi Linear Method (MVLE) was used to model the walls. The model was first introduced by Volkano and his colleagues in 1988 [20]. In this model, several vertical elements with only axial stiffness are connected in parallel to each other at the top and bottom, as shown in Figure (9a). Two side elements (With an axial hardness of K1 and Kn) represent cross - wall parts in the flexural and axial behavior of the wall. A nonlinear horizontal spring is used to simulate the shear resistance of the wall. The relative rotation of the wall occurs around a point in the central axis of the wall and at the height of ch from the bottom of the element. The parameter selection c is based on the curvature distribution at the height between stories (h), which is a number between 0 and 1. Based on the comparison between the laboratory results and the modeling results, the value of 0.4 for c is suggested by Volkano and his colleagues [20]. As shown in Figure (9b), the MVLE elements overlap the analytical model of the wall.

In this study, OpenSees software [21] is used to perform nonlinear analysis and modeling method MVLE used to simulate the wall. Concrete01 is used for modeling nonlinear concrete behavior, and Reinforcing steel for vertical steel bars is defined based on the proposed Kent, Scott, Park, Change, and Mender relationships, respectively. Figure 10 shows the stress-strain diagram of the defined material. For nonlinear static analysis (Pushover), first, the load is applied as a load-control procedure on the structure. Then, by holding these loads constant, lateral forces are applied as displacement-control over the structure. Since the walls in the y-direction were examined in this study, lateral forces and consequently nonlinear static analysis were performed in this direction. After the capacity curve is obtained, the change of the target site has been calculated with the help of seismic improvement instructions, Journal 360 [22]. In Figure 11, the structural capacity curve of up to 1.5 times the target displacement and its ideal two-line curve are shown along with the target displacement values. Figure (12) shows the drift of the stories in the target displacement. According to Standard 2800, this ratio should not exceed 0.025 for structures less than 5 floors and 0.020 for other structures. Fig. 13 shows the maximum deflection of the concrete along the wall length and in changing the target location for different models.
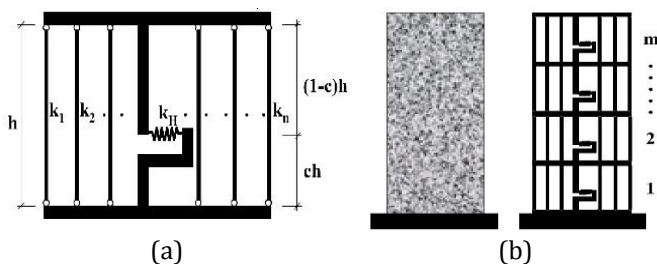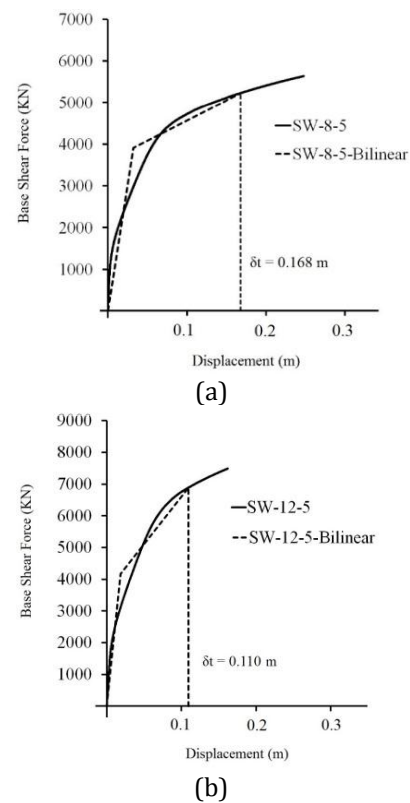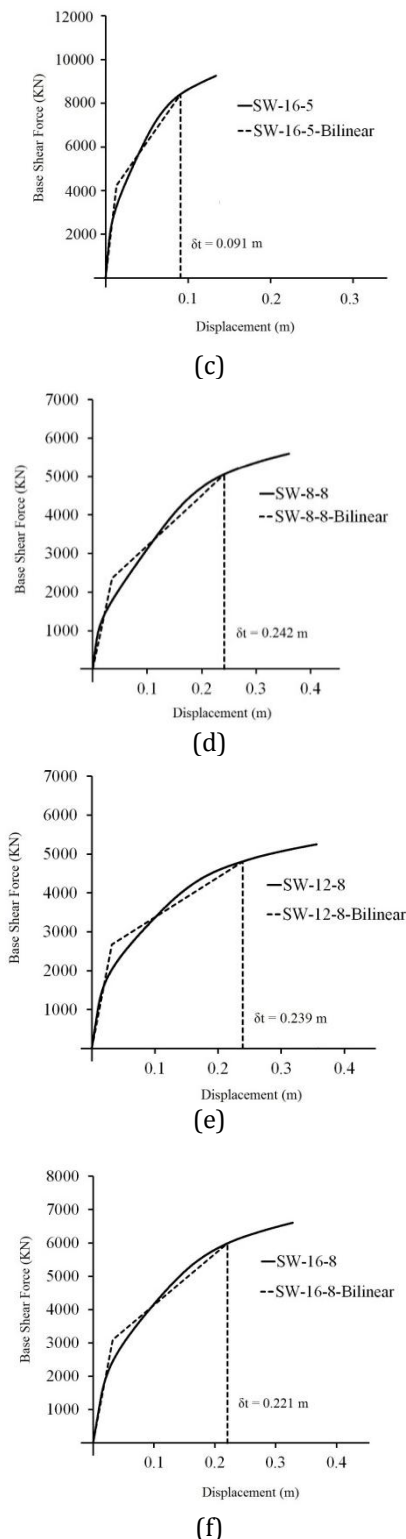

Fig- 9 Modeling of reinforced concrete wall structure using MVLE method (a) MVLE element, (b) Positioning of MVLE elements for numerical RC wall model

As observed, the maximum strain generated at the foot of the wall is less than the confining stress of the concrete (ε_c = 0.0035) in the design code.
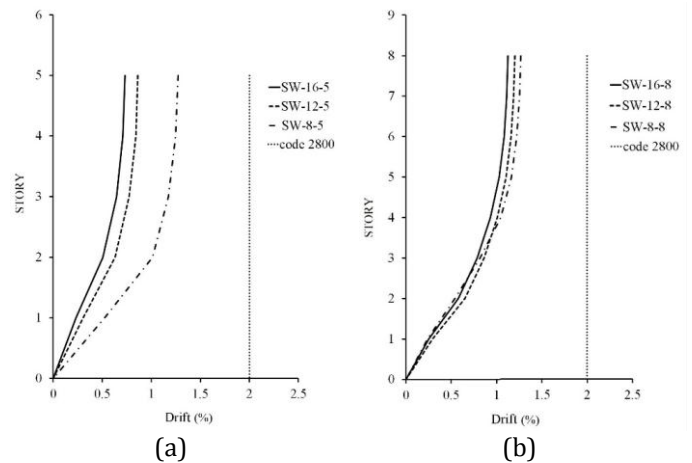

(a)

(b)
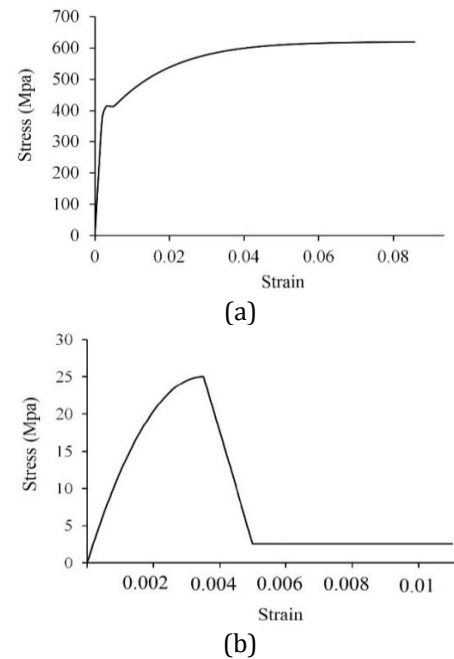
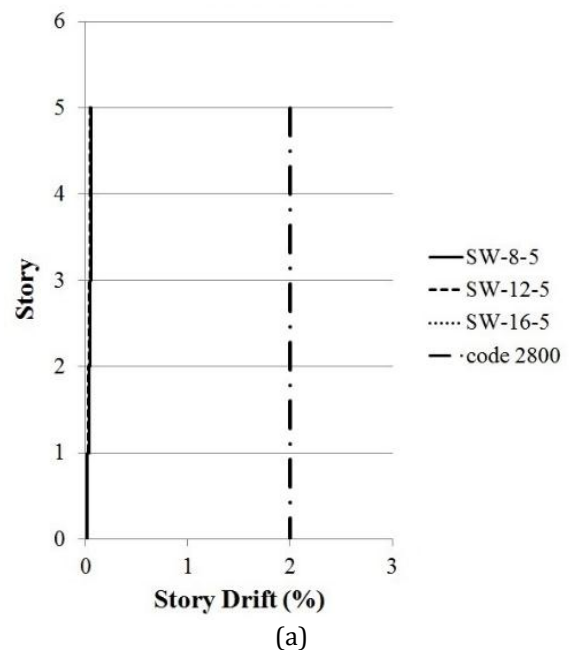Fig-10 Nonlinear stress- strain diagram of materials: (a) concrete, (b) steel rebar.

(a)

(b)

(c)

(d)

(e)

(a)

(a)

(b)

Fig-12 Story drift ratio in target displacement: (a) 5-story buildings (b) 8-story building

5. Nonlinear dynamic time history analysis

In this study, in addition to nonlinear static analysis, a nonlinear time history analysis method has been used to evaluate the designed models. For this purpose, three accelerations related to earthquakes in Loma Prieta, North Ridge, and Taiwan Smart were selected according to the conditions of the supposed region of the models. After scaling up with a standard range of regulation of 2800, they were used to evaluate the models. To the drift chart, the figures (13) and (14) show the maximum relative lateral displacement of stories for 5-story and 8-story models. It is observed that the relative displacement of the stories is lower than specified in the code.

(a)

Fig-11 Capacity curves of designed structures and ideal diagrams at target displacement (a) 5-story building with 8 structural walls, (b) 5-story building with 12 structural walls, (c) 5-story building with 16 structural walls, (d) 8-story building with 8 structural walls, (e) 8-story building with 12 structural walls, (a) 8-story building with 16 structural walls
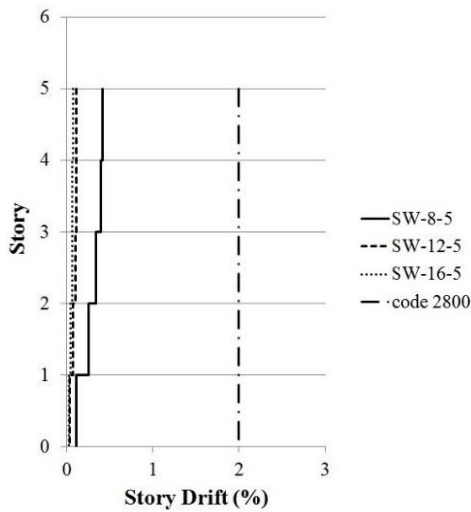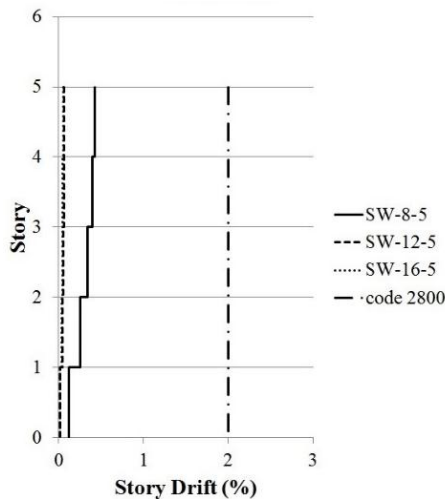
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cause trapping and thus increasing the ductility of the wall that the strain of the concrete is not confined to the maximum in a strain of the trapped concrete.
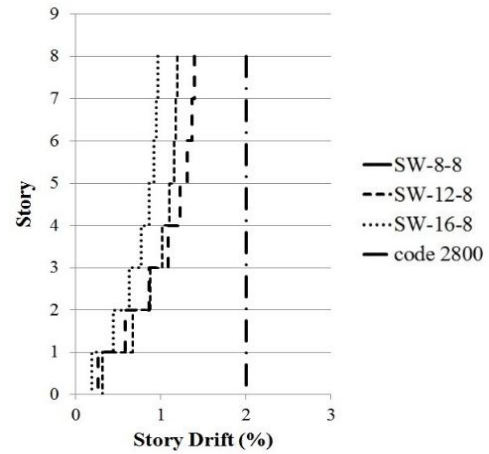

(b)

(a)

(c)

Fig-13 Relative lateral displacement of floors in 5-story models: (a) LomaPrieta, (b) Northridge, (c) Tiwan Smart, earthquakes
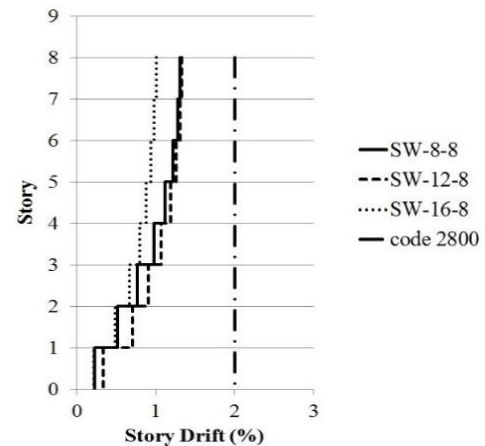
Conclusion

The necessity of using transverse rebars to secure trapping at the edges of the framed wall was investigated in this study. For this reason, theoretical models had framed walls (shearing and bearing) with a simple plan, in which the total space of the walls was fixed on the area of the floor.

In order to each of these models, the lengths required to create confinement were estimated according to the criteria of the design code regulations and based on the displacement-based design method. The results of nonlinear static and dynamic analysis represent that the rules in the bylaw are very conservative in applying transverse rebar to make confinement in concrete when the stress is more than $0.31\ f_{cd}$; Because once a time, the transverse rebars can
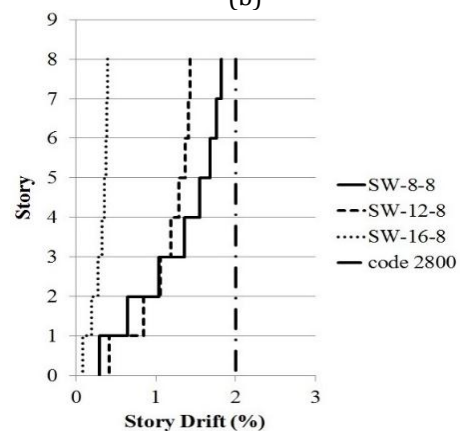

(b)

(c)

Fig 14- Relative lateral displacement of floors in 8-story models: (a) Loma Prieta, (b) Northridge, (c) Tiwan Smart, earthquakes

. It is also based on the proper functioning of structures that the simplified correlation of the displacement spectrum (Eq. (2) has sufficient validity to investigate the need for confinement informed walls. So, the other result from this study is that for a ratio of wall area to a fixed floor area, considering architectural aspects, the use of more walls with less thickness is better than the smaller number of walls but ticker because of the distribution of formed walls in the plan, the purpose of the replacing the structure, as well as the relative displacement of the lower stories, and consequently, the seismic need of the structure will be less.

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