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SHEAR WALLS WITH DISPERSED INPUT ENERGY DISSIPATION POTENTIAL

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ABSTRACT

Shear walls are a type of structural system that provides lateral resistance to a building or structure and have relatively high stiffness and strength. However, since the energy dissipation is made within a limited part of the shear walls, they do not exhibit ductile behavior. Shear walls may be considered as an obstacle to make openings in the buildings. Coupled shear wall is one of the types of shear walls that has removed some of the problems encountered with shear walls. Yielding of link beams in the coupled shear walls may cause increase in energy dissipation and development of damage. Regular and uniform openings in the coupled shear walls reduce the architectural design limitations. As well as coupled shear walls, two other types of shear walls, namely slit shear walls and shear wall with upper connection have been investigated in this paper to develop energy dissipation along with the wall height. Such shear walls concerning the architectural design limitations are lower than the normal shear walls. In this research, the above-mentioned shear walls have been examined under the same conditions and a comparative study has been carried out concerning some parameters such as strength, stiffness, lateral displacement, cracking development and steel weight. Moreover, shear walls have been compared through considering the ratio of strength to steel weight and their ductility and the results revealed that slit and coupled shear walls showed more favorable performance.

Keywords: Shear wall, ductility, plastic hinge, cracking pattern

1. INTRODUCTION

Shear wall is a kind of resistant system against lateral forces in high and mid rise buildings.

The report obtained from the past earthquakes indicated the relatively appropriate performance of shear walls [1, 2]. The main drawback of using shear wall can be assigned to its imposed limitations to the architectural design due to the lack of openings. Although shear walls have relatively high strength and stiffness, they do not show ductile behavior.

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Ductile behavior in the shear walls occurs by yielding of flexural reinforcement at the wall base. Through forming of flexural plastic hinge, large relative displacements takeplace at the top of the shear wall having a small contribution to dissipate energy. The increase in the ductility of shear walls requires creating high ultimate capacity at the wall base. The observed damages in the shear walls during past earthquakes highlighted the damage concentration at the wall base [1, 2]. In most cases, creating small openings as door or window is necessary in the shear walls. The experimental and analytical studies in the shear walls, having uniform and regular openings that is, coupled shear walls, showed better performance compared to normal shear walls against earthquake lateral forces [3-5]. Coupled shear walls are capable to dissipate energy along the walls height and consequently the damage concentration may be reduced at the base and developed to the upper parts of the wall. In the early 1970s, another type of shear walls called slit shear walls was proposed to improve the shear walls performance against lateral forces. In these walls, the ductility and energy dissipation in the link beams have been increased through making slit. The conducted studies indicated the increase in the ductility and decrease in the stiffness and strength within the slit shear walls compared to normal shear walls [6-11].

In this study, the attempt was to reduce damage concentration at the wall base and to distribute damage along wall height. This may lead to decrease demand ductility capacity at the base. The shear walls considered in this paper include coupled, slit and shear wall with upper connection. It should be noted that energy dissipation is only due to bending and not because of ductility resulting from shear or combination of shear and bending. The applied method is based on the current bending formulas in reinforced concrete. In this research, a seven-story building has been used as a case study to investigate the performance of these three types of shear walls.

2. CHARACTERISTICS OF BUILDING

Figure 1 shows plan of a seven-story residential building in which shear walls have been employed in one direction. Iranian 2800 standard [12] has been applied to determine earthquake lateral forces. In addition, it is assumed that the design load is imposed as a concentrated load to the seventh floor.

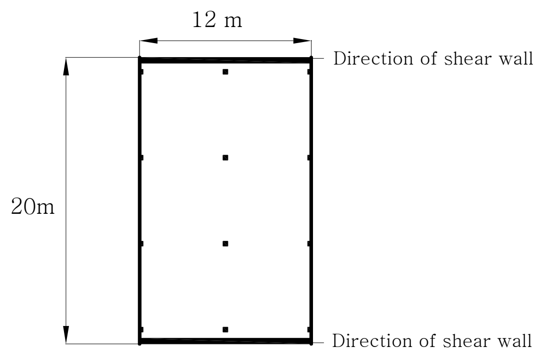
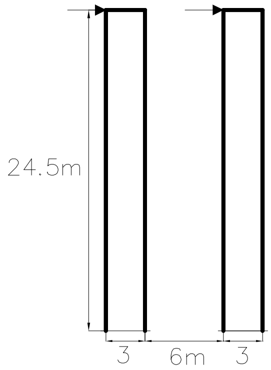


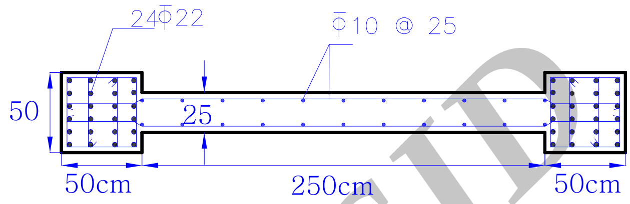
Figure 1. The plan of a seven-story building

3. USING LOW DEPTH SHEAR WALLS

In this section, four shear walls with the width of 3 m have been used in the north-south direction of the building. Figures 2(a) and 2(b) show the plan and section views of the shear wall, respectively. The imposed load to each shear wall is equal to $\frac{F}{2} = 590\,KN$.



(a)



(b)

Figure 2. (a) Plan of shear wall (b) Section of shear wall

This study is based on the examining the lateral displacements of shear wall due to the flexural deformations, so the lateral displacements because of the shear deformations have been ignored. In order to determine the moment-curvature diagram, the concrete section was divided into some layers and the associated forces to each layer were computed so as to meet the forces equilibrium and strain consistency. It should be noted that the axial strain changes is assumed to be linear. The curvature can be obtained from dividing the sum of maximum compressive and tensile strain of concrete and reinforcement, respectively to the wall depth [13].

The following displacement-curvature relationships were utilized to determine the displacement due to the flexural yielding at the wall base as well as displacement due to plastic hinge [13].

$$\Delta_p = (\phi_u - \phi_y) l_p (h_w - \frac{l_p}{2}), \text{ where } l_p = \frac{l_w}{2} \text{ and } \Delta = \Delta_y + \Delta_p \quad (1)$$

where, Δ_y is yielding displacement, Δ_p is plastic hinge displacement, Δ is total displacement, ϕ_y is yielding curvature, ϕ_u is ultimate curvature, h_w is wall height, l_w is wall length and l_p is length of flexural plastic hinge.

The amount of lateral displacement at seventh floor due to the yielding and forming flexural plastic hinge at the wall base for lateral load $\frac{F}{2}=313\,KN$ is $\Delta_y = 182\ mm$ and $\Delta_p = 270\ mm$, respectively.

In order to determine cracking and crack width in the wall, the following equations have been used [14].

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \text{in which} \quad f_r = 0.63\sqrt{f'_c} \tag{2}$$

where, f'_c is 28-day concrete strength (MPa), M_{cr} is cracking moment, f_r is modulus of flexural failure, I_g is moment inertia of the section, y_t is distance between extreme tension fiber and neutral axis. The crack width w is defined as $w=\varepsilon_s s$ in which ε_s is strain of longitudinal reinforcements and s is spacing between transverse reinforcements.

Figure 3 shows a cracking pattern in the shear wall. The cracked areas have been determined using Eq.2. The distance between cracks and crack width are proportional to distances between transverse reinforcements (s) and crack depth, respectively. As it is shown in Figure 3, contrary to lower floors, there is no crack in the upper floors and the width of cracks in the mid floors is relatively small.

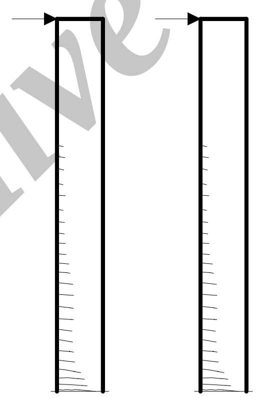
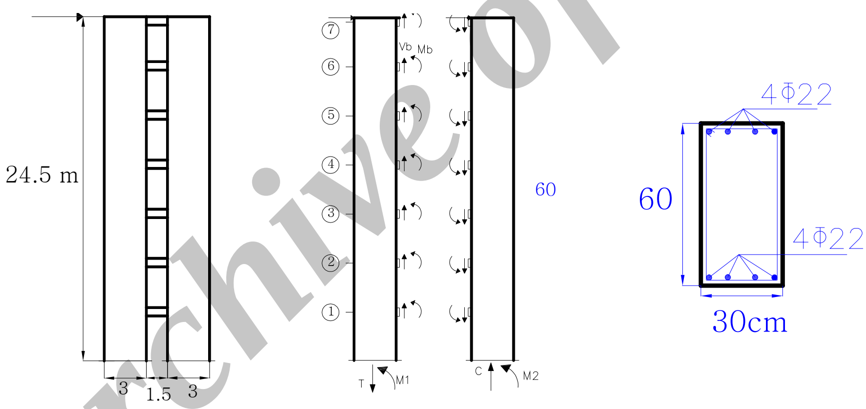


Figure 3. Cracking pattern in the shear wall

4. USING COUPLED SHEAR WALLS

The increase in the energy dissipation and development cracking in the upper floors, the shear walls were connected to each other using link beams. It should be mentioned no changes were made in the plan and walls length.

In the coupled shear walls, flexural yielding of link beams results in increasing in the energy dissipation along the wall height and it reduces the damage concentration in the lower part of the wall and it runs along the wall height. Existence of openings in the coupled shear walls satisfy the architectural design requirements and also it causes decrease in the stiffness of shear walls and accordingly, this leads to increase and decrease in the period and seismic forces, respectively. From the static viewpoint, forming flexural plastic hinge in the link beams does not have any effect on the applied forces to the wall, whereas flexural yielding of link beams can attenuate the energy and prevent accumulation of energy in the structure. Therefore, favorable performance of coupled shear walls mainly depends on the strength and ductility of shear walls and link beams. When this type of shear wall is subjected to the lateral load, the moment at the wall base is undergone by flexural moments of walls and also moment due to the axial forces (Figure 4-b, 4-c). Therefore resisting moment in coupled shear wall is $M = M_1 + M_2 + T.l$ where, M_1 and M_2 are flexural moments of shear wall, T is axial forces of the wall, l is the distance between the compressive and tensile forces. Forces equilibrium at the vertical direction of shear wall indicated that the axial forces at the wall base is equal to the sum of existing shear forces in the link beams (Figure 4(b) and 4(c)) i.e. $T = \sum_{i=1}^{7} V_i$ where, T is axial force at the wall and V_i is shear forces in the link beam of i^{th} floor.



(a) (b) (c)

Figure 4. (a) Dimensions of coupled shear walls (b) Analytical model of coupled shear walls (c) The section of link beam

If the link beams have high strength, energy dissipation due to yielding in the link beams decreases and shear walls reach to yielding state and ultimate moment happens without energy dissipation in the link beams. On the other hand, if link beams have low strength, they reach to failure state prior to yielding of shear walls and hence, energy dissipation and ductility increase in the coupled shear walls. In other words, very low flexural and shear strength of link beams may lead to failure of them before flexural yielding of walls at base. In this condition, the moment due to the axial forces is lower than that of due to the sum of flexural moments and walls behave independently. The following equation represents the ratio of moment due to the axial forces and moment of shear wall and it may be used as a criterion for correlation degree of coupled shear walls.

$$\beta = \frac{T.l}{M_1 + M_2 + T.l} \tag{3}$$

In the above equation, β is correlation degree and has a significant role effect on the energy dissipation and ductility of coupled shear walls and its range is different in different building codes [15-16].

Figure 4 shows dimensions and analytical model of coupled shear walls. Since ductile behavior of coupled shear walls depends on the plastic deformation capacity of the shear wall and link beams, therefore, the deformation is first computed along the wall height.

As it is seen in Figure 4, produced deformations along the wall height are because of three force components. Total displacement of wall is the sum of displacement due to the concentrated lateral load, moments of link beams and axial vertical force.

In order to determine the required deformation in the link beams, consistency of displacement between shear wall and link beam has been employed [17]. Figure 5(a) depicts the deformation of coupled shear walls in the plastic deformations stage. Figure 5(b) also shows consistent deformation between wall and link beam at plastic deformation state. In this case, the relation between rotations of wall and link beam is as follows:

$$\frac{\theta_b}{\theta_w} = \frac{l_w}{l_b} \tag{4}$$

where, θ_b and θ_w are rotations of link beam and wall, respectively. l_b and l_w are length of link beams and wall. The ratio of $\frac{\theta_b}{\theta_w}$ was computed equal to 2.

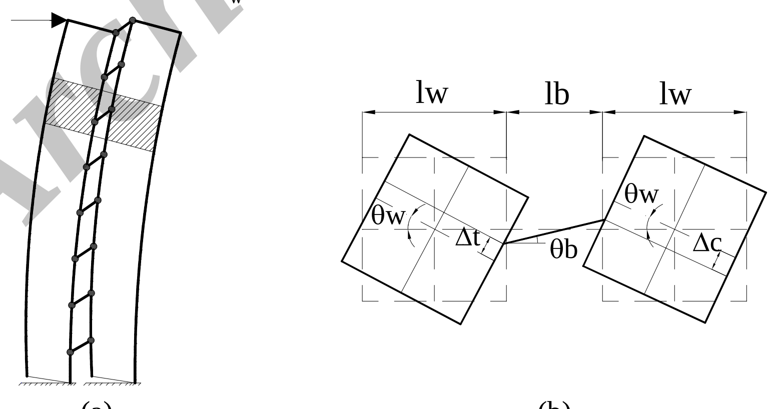


Figure 5. (a) Deformation of coupled shear walls due to the lateral load (b) Plastic deformation of link beam

The value of $\beta = 0.5$ has been used to distribute forces between walls and link beams and dimensions of link beams are assumed to be the same in all floors (Figure 4(d)).

The values of produced rotation along the wall height and also link beams are given in Table 1.

Table 1. The produced rotation (Radian) along the wall height and link beams due to the lateral forces

Story	Rotation due to the lateral load	Rotation due to the moment of link beam	Rotation due to the vertical axial force	Total rotations in the wall	Rotation in the link beam
7	4.9×10^{-3}	1.9×10^{-3}	3.0×10^{-5}	3.0×10^{-3}	6.0×10^{-3}
6	4.8×10^{-3}	1.8×10^{-3}	6.0×10^{-5}	2.9×10^{-3}	5.8×10^{-3}
5	4.5×10^{-3}	1.7×10^{-3}	9.0×10^{-5}	2.7×10^{-3}	5.4×10^{-3}
4	4.0×10^{-3}	1.5×10^{-3}	1.2×10^{-4}	2.4×10^{-3}	4.8×10^{-3}
3	3.3×10^{-3}	1.2×10^{-3}	1.5×10^{-4}	2.0×10^{-3}	4.0×10^{-3}
2	2.4×10^{-3}	8.7×10^{-4}	1.8×10^{-4}	1.4×10^{-3}	2.8×10^{-3}
1	1.3×10^{-3}	4.7×10^{-4}	2.1×10^{-4}	6.2×10^{-4}	1.2×10^{-3}

The values of rotation in the second column of Tble 1 are due to the lateral load in the wall. As it is shown in Figure 4, the third column shows the rotation due to the applied moment of link beams to the wall which is in the opposite direction of rotation due to the concentrated load. The fourth column represents the values of rotation due to the vertical axial forces which is in the opposite direction of rotation due to the concentrated load. The axial forces are produced as a result of shear forces in the link beams. The total rotations of walls are listed in the fifth column through considering the direction of rotation due to force of each wall. In the sixth column, the rotation of link beam is computed using equation 4.

Through using the curvature-moment diagram, the value of yielding and ultimate rotation of link beam are θ_{by}=0.0035 and θ_{bu}=0.379, respectively and θ_{wy}=0.0069 and θ_{wu}=0.0206 at the wall base. It should be mentioned that maximum strain of concrete and steel are 0.004 and 0.03, respectively at ultimate rotation.

Comparing between rotations in the link beams during yielding of coupled shear walls and rotation capacity of link beams revealed that link beams of third, fourth, fifth, sixth and seventh floor were yielded. The rotation link beams increases while the plastic rotation increases at the wall base. The maximum lateral displacement in the coupled shear walls can be due to the maximum rotation of beam or maximum plastic deformation capacity at the wall base. In this case, the maximum lateral displacement is due to the maximum plastic deformation capacity at the seventh floor. Therefore, lateral displacement seventh floor for the yielding state at the wall base and forming flexural plastic hinge are Δ_y=168.1 mm and

Δ_p=294.7 mm, respectively.

Figure 6 illustrates the cracking pattern in the coupled shear walls. As it is shown, there are cracks in the link beams, therefore flexural plastic hinge have been formed. In addition, flexural plastic hinge have been formed at the wall base and cracking along the wall height can only be observed in the link beams.

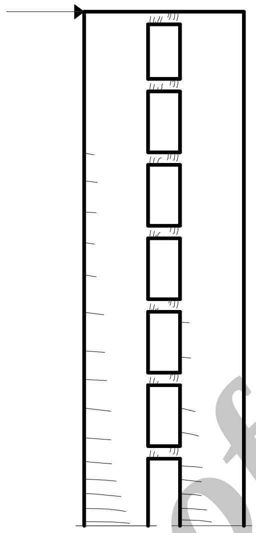


Figure 6. Cracking pattern in the coupled shear walls

5. USING SLIT SHEAR WALLS

Another way to increase of energy dissipation and ductility in the shear walls is to make slit through them. This also prevents brittle shear failures and can remove the architectural limitations of shear walls. From the static perspective, it seems that slit shear wall is softer and has more lateral displacement compared to normal shear wall. While, from the dynamic perspective, yielding along the wall height causes increase in the energy dissipation and consequently reduction in the applied load to the wall. Thus, making slit in the shear wall has positive influence on the dynamic behavior of shear wall [9-11]. Slit shear walls may be obtained through making vertical slit along the wall height. By using this method, the initial shear wall is divided into narrow shear walls and accordingly slit shear walls will be obtained (Figure 7). The produced deformation in the wall due to the lateral load is flexural [9-10]. In the slit shear walls, yielding first occurs at the connection of walls. This is mainly due to the produced shear force that exists in the connection region. The shear force is due to flexural deformation of narrow shear walls. By increasing in the lateral forces, yielding occurs at wall base and accordingly plastic deformation increases. In this condition, the wall connections fail due to the lack of sufficient plastic deformation capacity and narrow shear walls behave independently. Therefore, the role of strength and ductility of wall connections is highly critical in the energy dissipation.

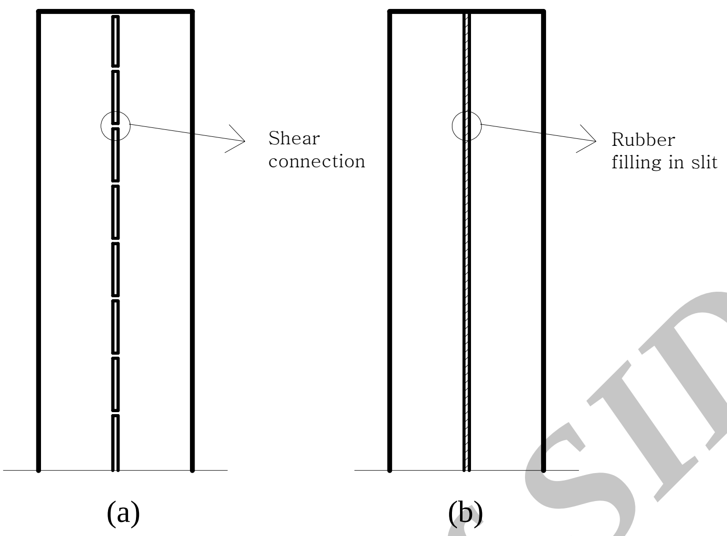


Figure 7. Slit shear wall with flexural mode deformation (a) First type (b) Second type

Another type of slit shear walls has been introduced in Figure 8(a). The applied lateral forces cause flexural behavior in the walls between slits and energy dissipation occurs through flexural yielding at the two ends of narrow walls. As the number of slits increase, the shear wall tends to behave more ductile and through forming plastic hinges, energy dissipation increases along the wall height. Figure 8(b) shows deformation due to the making slit in the wall. As it is shown, the wall deformation has been changed from flexural mode to shear mode.

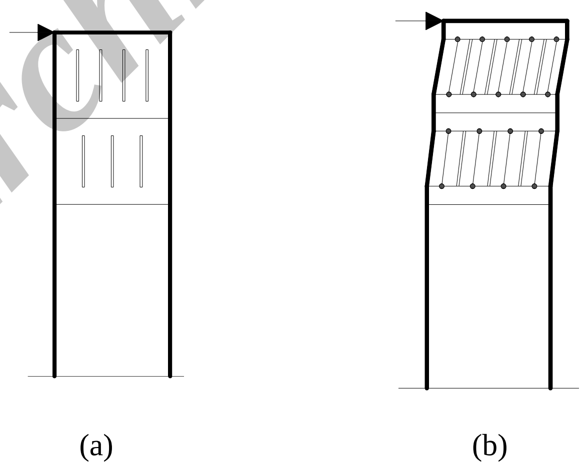


Figure 8. (a) Slit shear walls with shear mode deformation (b) Plastic mode deformation

A slit shear wall has been illustrated in Figure 9. Through making slits in the wall, the attempt was to dissipate energy along the wall height and increase in the lateral displacement and decrease in the plastic deformation at the base wall. For this purpose, nine slits were made in the sixth and seventh floor. The height and width of slits at each floor were 2.6 m and 6 cm, respectively. The applied shear force was distributed equally between narrow walls and also the effect of flexural moment has been considered as compressive and tensile forces in the side narrow walls. Seven slits were made in the fourth and fifth floor and longitudinal reinforcements had been designed using curvature-moment diagram of narrow walls. Since the effects of axial forces on the increase in the deformations due to the slits can cause decrease in the flexural moment capacity in the section, the number of slits have been decreased in the first, second and third floors. Thus, the narrow walls deformations would decrease [18].

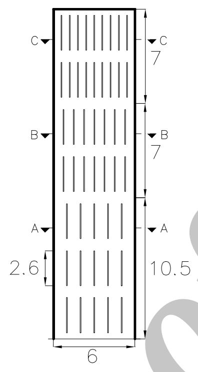


Figure 9. Geometric dimensions of slit shear walls (dimensions in meters)

Based on what mentioned, the designed reinforcements of side narrow walls in the sections A-A, B-B and C-C are 21T22, 2T28+1T25 and 3T20, respectively. Additionally, the designed reinforcements of middle narrow walls in the sections A-A, B-B and C-C are 2T10@20cm, 3T14 and 3T18, respectively.

Figure 10 shows the lateral displacement due to plastic deformations at the wall base and along the wall height. The ultimate lateral displacement of seventh floor in this example was Δ_u=590.2 *mm*.

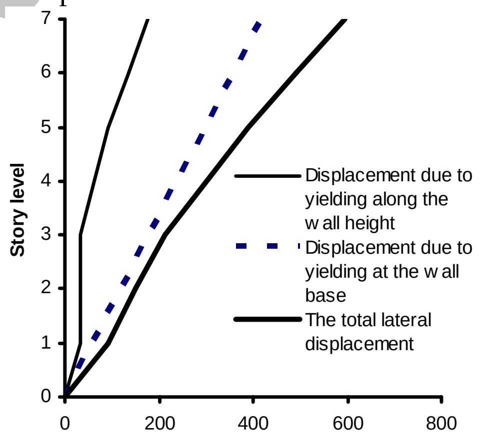


Figure 10. Lateral displacement in the slit shear walls

In Figure 10, plastic displacements in the narrow wall are plotted. This curve implies that the plastic deformations occur at the upper part of shear wall as well as the lower part of the shear wall.

The cracking pattern in the slit shear walls is depicted in Figure 11. As it can be seen, the flexural cracks have been occurred at the wall base, narrow walls and upper part of walls.

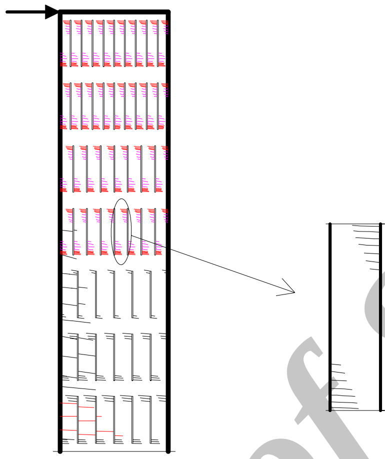


Figure 11. (a) Cracking pattern in the slit shear walls (b) Cracking pattern in the narrow walls

In order to compare the behavior of slit and normal shear walls, a normal shear wall with the length of 6 m was designed. The results revealed that ultimate lateral displacement in the seventh floor was Δ_u=419 *mm* which is smaller than that of the lateral displacement of slit shear wall. This wall was designed so as to compare with slit shear wall from the aspect of weight steel.

6. SHEAR WALL WITH UPPER CONNECTION

As stated previously, normal shear walls behave like cantilever beam against lateral loads and the energy dissipation mainly occurs at the wall base. In order to occur yielding in the upper part of the shear wall, it is assumed to prevent the rotation of upper part and change the cantilever behavior of the wall. The lateral displacement-load diagram of shear wall and constrained shear wall under the same loading and assuming the full elasto-plastic behavior is plotted in Figure 12. Dimensions and characteristics of two shear walls are considered to be the same. As it can be seen clearly from load-lateral displacement diagrams, the flexural strength of constrained shear wall is twice of the normal shear wall. In addition, the stiffness and displacement of constrained shear wall is four times and half of normal shear wall, respectively. Hence, the ductility of constrained shear wall is more than the normal shear wall. Therefore, better seismic behavior characteristics can be obtained by constraining the upper part of the wall.

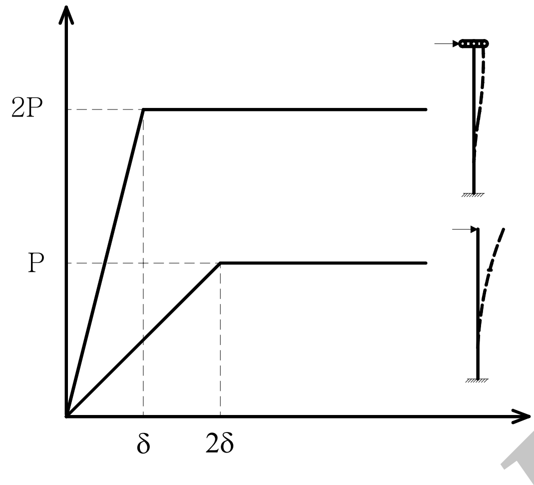


Figure 12. Load-lateral displacement diagram for normal and constraint shear walls

Through connecting the upper part of shear walls, lateral forces are transferred by bending and axial forces. Furthermore, full connection of shear walls in the upper part may cause increase in the stiffness and strength of shear walls. Also, this may provide energy dissipation in the upper part. Since it is intended that flexural yielding occurs in the upper part of the wall, the connection of shear walls have been made by using deep beam. The assumption of rigid connection can be made in the analysis and design of shear walls, because they have been connected to each other by deep beam and the height of shear walls compared to beam length is large enough.

Figures 13(a) and 13(b) show the dimensions and axial forces of shear walls. As it is shown, shear wall number 1 is subjected to tension and bending and shear wall number 2 is subjected to compression and bending.

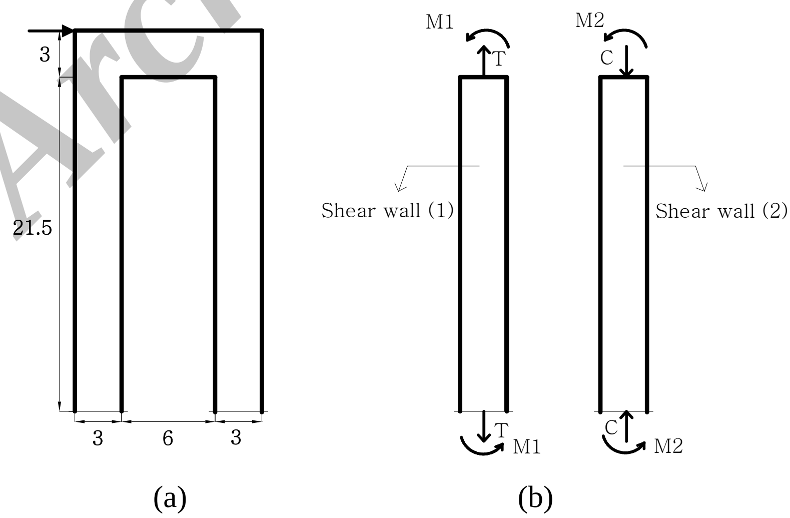


Figure 13. (a) Dimensions and geometry of shear wall with upper connection (dimensions in meter) (b) Internal forces of shear walls

The moment-curvature diagram has been employed to determine displacement during yielding and ultimate displacement of shear walls. It should be noted that the moment-curvature diagram for shear walls number 1 and 2 have been obtained for the same ratio of axial force and flexural moment. These two curves are illustrated in Figure 14. The moment-curvature diagram of shear wall has been obtained from the sum of correspondent moments of shear walls 1 and 2 at the considered curvature.

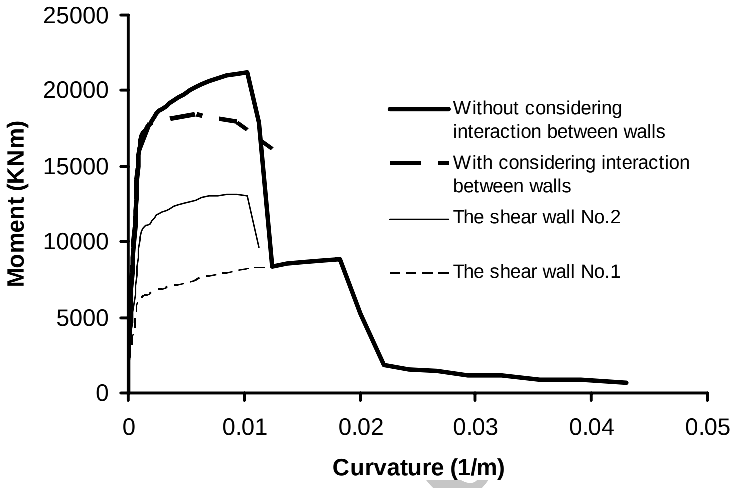


Figure 14. The moment-curvature under axial force and flexural moment

Figure 14 shows the moment-curvature diagram for shear walls number 1 and 2. The solid curve represents the system behavior with the assumption of lack of interaction between two shear walls. Since in practice, the shear wall reaches to yielding limit under tension, through forming plastic hinges at upper and lower part of the wall, the degree of indeterminacy have been reduced. The dashed curve was obtained using trial and error and represents more actual behavior of shear wall.

It should be noted that yielding first occurred in the shear wall number 1. Since the shear wall is static indeterminate, the initial values of axial force and flexural moment and consequently the moment-curvature diagram change by yielding the shear wall number 1. The moment-curvature diagram of shear wall has been plotted through considering the effect of interaction and different ratios of moment transfer between walls. Redistribution of flexural moment between walls causes increase and decrease in the ultimate curvature of shear wall number 2 and 1, respectively. The compressive forces in the shear wall number 2 result in increasing in the compressive strain of concrete and lack of using confining reinforcements in the upper strain region may lead to decrease in the ductility.

The lateral displacement in the seventh floor due to yielding and forming flexural plastic hinge with and without confining in the compressive region were Δ_p=84.0 *mm*, Δ_p=296.6 *mm* and Δ_p=444.9 *mm*.

The cracking pattern of shear wall with upper connection is illustrated in Figure 15. As it can be seen, the cracking has been developed along the wall height of shear wall number 1 and flexural plastic hinges have been formed in the upper and lower part of shear wall.

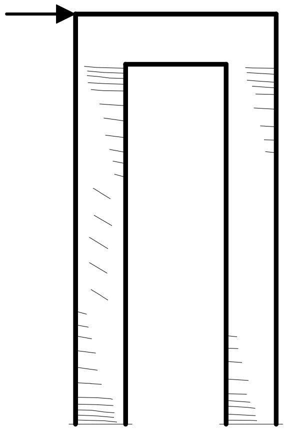


Figure 15. The cracking pattern of shear wall with upper connection

7. DISSCUSION AND RESULTS FOR DIFFERENT SHEAR WALLS

Figure 16 shows the load-lateral displacement diagram at the seventh floor so as to investigate the strength, maximum lateral displacement in the beginning of yielding, maximum plastic lateral displacement, stiffness and ductility of shear walls.

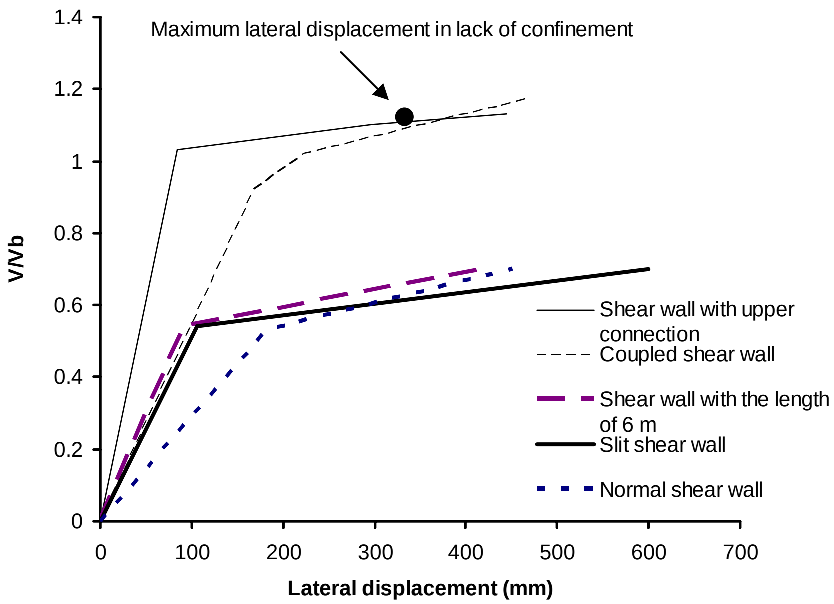


Figure 16. Load-lateral displacement of seventh floor in the considered shear walls

The main seismic parameters in the design of shear walls are given in the Table 2. Comparison of these parameters can reveal the weakness and strong points of different shear walls against lateral loads.

The shear strength and stiffness of shear walls were obtained using Figure 16 and the equation of $K = \frac{V_y}{\Delta_y}$ where, K is stiffness of shear wall in the elastic state, V_y is lateral loads at the beginning of yielding and Δ_y is lateral displacement due to the wall yielding. The drift is defined as $D = \frac{\Delta_{u7}}{H} \times 100$ where, Δ_{u7} and H are ultimate lateral displacement of seventh floor and wall height.

Due to proportional relation between flexural crack width and ultimate curvature (ϕ_u), this parameter was defined as the crack width index in the shear wall. Since the ratio of steel weight to the strength is an important parameter in the design of shear wall, the acceptance degree of design was defined considering the relation between steel weight and other parameters like ductility, crack width, etc. In the eighth column of Table 2, the acceptance degree of design is given for different shear walls and it indicates the priority of slit and coupled shear walls.

Table 2. Evaluation of different parameters in the different shear walls

Shear wall type*	**Strength (KN)**	**Stiffness (N/mm)**	**D (%)**	ϕ_u **(1/m)**	**Steel weight (ton)**	**Strength to steel weight (KN/ton)**	**Acceptance degree of design**
1	826.0	3436	1.8	0.0085	8.8	94	6
2	826.0	6926	1.7	0.0052	4.7	176	3
3	826.0	6046	2.4	0.0052	5.0	165	1
4	1380.6	6458	1.9	0.0092	9.2	150	2
5	1298.0	14469	1.2	0.0070	9.6	135	5
6	1333.4	14469	1.8	0.0110	9.8	136	4

***Shear wall type is including 1- Normal shear wall, 2- Shear wall with the length of 6 m, 3- Slit shear wall, 4- Coupled shear wall, 5- Shear wall with upper connection (without confining), 6- Shear wall with upper connection (with confining)**

As it is cited in Table 2, coupled shear wall and shear wall with upper connection have higher strength compared to slit and normal shear walls. Moreover, normal shear wall and shear wall with upper connection have the least and most stiffness, respectively. Also, slit shear wall has the highest ultimate relative displacement and this parameter is the same in other shear walls. And finally, slit shear wall has the lowest plastic curvature representing the crack width.

CONCLUSION

Conducted studies indicated that energy dissipation can be achieved in lieu of the wall height to inhibit damage concentration at the wall base. In this research, maximum lateral displacement, plastic curvature, crack width and steel weight are considered as effective factors in the performance of shear walls. The results reveal that the strength of the coupled shear wall and shear wall with upper connection are more than other shear walls. Furthermore, maximum lateral displacement of slit shear walls is greater than other shear walls. The results indicate that it is possible to design shear walls with different bearing capacity and ductility and to obtain higher capacity in the ductility and crack width control in the shear walls.

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