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Investigation of the link beam length of a coupled steel plate shear wall

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Abstract. Steel shear wall system has been used in recent years in tall buildings due to its appropriate behavior advantages such as stiffness, high strength, economic feasibility and high energy absorption capability. Coupled steel plate shear walls consist of two steel shear walls that are connected to each other by steel link beam at each floor level. In this article the frames of 3, 10, and 15 of (C-SPSW) floor with rigid connection were considered in three different lengths of 1.25, 2.5 and 3.75 meters and link beams with plastic section modulus of 100% to the panel beam at each floor level and analyzed using three pairs of accelerograms based on nonlinear dynamic analysis through ABAQUS software and then the performance of walls and link beams at base shear, drift, the period of structure, degree of coupling (DC) and dissipated energy evaluated. The results show that the (C-SPSW) system base shear increases with a decrease in the link beam length, and the drift, main period and dissipated energy of structure decreases. Also the link beam length has different effects on parameters of coupling degrees.

Keywords: coupled steel plate shear wall; link beam; nonlinear dynamic analysis; plastic section modulus

1. Introduction

Steel Plate Shear Wall (SPSW) has been used as a resisting system against the lateral loads in the construction of new buildings and strengthening the existing buildings (especially in high-rise buildings) within the four recent decades in North America, Canada and Japan. This system has appropriate stiffness for controlling the structural deformation and because of its ductile failure mechanism, the energy dissipation is high. Due to architectural considerations, SPSW system is typically located around the core and partly because of the openings in the core two SPSW systems get together. On the other hand, given that American Regulation has limited the steel shear wall ratios to number 2.5, that's why designers use a simple span beside two SPSWs, similarly, these two walls are connected by a link beam at each floor level.

In recent years, few studies have been performed about Coupled Steel Plate Shear Wall which the first one was conducted by Zhao and Astaneh-Asl (2004) at UC Berkeley. In this research the samples of 2 and 3 floors were made half the size of the original model and side columns around the wall were made with concrete-filled cylindrical steel tubes. The section of used link beam considered the same as floor beam section and loading was cyclic. Both samples showed ductile

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behavior and in both system failure occurred at upper coupling beam. Harries *et al*. (2004) to investigate the elastic response parameters of coupled wall structures and to identify parameters that will permit an accurate initial estimate of the global behavior of a coupled system, the local behavior of the coupling beams and the interaction between the global and local behaviors. Using elastic analysis and gross section properties, the role of representative geometric parameters in the response of coupled structures was illustrated. Jadhav and Patil (2012) studied the performance of Steel Plate Shear Wall during Past Earthquakes events, the testing on steel plate and also the different case study of SPSW system. Vatansever and yardimci (2011) made direct comparisons regarding the cyclic behavior of thin steel plate shear walls (TSPSWs) with different infill-to-boundary frame connections. The behavior of TSPSW specimens are compared and discussed with emphasis on the characteristics important in seismic response, including the initial stiffness, ultimate strength and deformation modes observed during the tests. It is shown that TSPSW specimens achieve significant ductility and energy dissipation while the ultimate failure mode resulted from infill plate fracture at the net section of the infill plate-to-boundary frame connection after substantial infill plate yielding. Li *et al*. (2011) conducted a pilot under cyclic loading with a sample scale of 40% (2.5 floors) based on the original model of a 6 floors building and its results compared with numerical analysis using ABAQUS software, which suggested an appropriate response to the laboratory sample, The test results show that the CSPSW specimen behaved in a ductile manner and dissipated significant amounts of hysteresis energy during the cyclic loadings. Li *et al*. (2011) describes the recent experimental researches on the steel plate shear wall (SPSW) at National Center for Research on Earthquake Engineering (NCREE). In addition, the design implications learned from the test results are presented. The test results was suggested that the rotational demand of the coupling beam at the lowest level of a C-SPSW can be estimated as the design story drift, The strip model can predict the overall and local responses of the C-SPSW specimen very well. Further studies on the seismic design and behaviours of the C-SPSW can be conducted using strip models. Borello and Fahnestock (2011) designed several frames of 6-story Coupled Steel Plate Shear Wall and to validate design performance, the prototype buildings were modeled using OpenSees with pushover loading conditions. Also in the laboratory specimens 3 story of 6-story models were made. In this study the section of sample link beam had 100%, 200% and 400% plastic capacity of floor beams and the results showed that with the increase of link beam section than the floor beam base shear increased and drift increased at lower floors. Borello and Fahnestock (2012a, b) conducted another study and examined the behavior and mechanism of Coupled Steel Plate Shear Wall. In this study, some relations were given for DC, 32 structures, 6 and 12-story with a plan based on the model of 9-story building and two different length of link beam 1.8 and 2.8 m was modeled by the ratio of 25% to 600% plastic section of floors, These structures are studied with numerical models using monotonic nonlinear static analysis, and the results showed that with increasing the length of link beam DC does not increase always, and the optimal degree of coupling to maximize material efficiency is found to be in the range of 0.4 to 0.6. According to the detailed review of previous studies, the need for nonlinear dynamic analysis under various accelerogram along with various link beam length is felt. In this paper, some samples of 3, 10, and 15-story Coupled Steel Plate Shear Wall that symbolize short, medium and high buildings were made with ABAQUS software and three different link beam length of 1.25, 2.5 and 3.75 meters with a plastics capacity of 100 percent than the floor beam were used in the studied samples then samples analyzed by nonlinear dynamic analysis under three different couples of accelerogram.

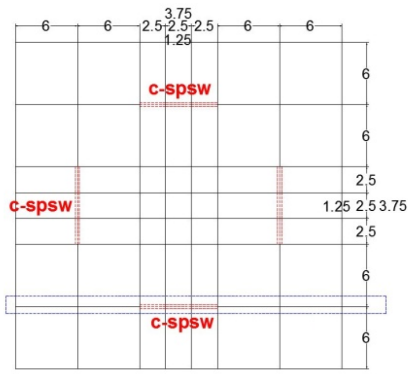


Fig. 1 Plan view

2. Analysis and modelling of structural assumptions

Nine Coupled Steel Plate Shear Wall frames with a thin plate of rigid connection that each had seven spans and different length of link beam, were modeled by using tape model that the plan geometry used in the present research was based on the plan of the square Fig. 1.

In these samples, the width of the shear panels was 2.5 meter and the length of them varied 1.25, 2.5 and 3.75 meters and the width of side spans was 6 meters on each side, frames assumed in three buildings 3, 10 and 15 floors with the height of 3 meter (Figs. 4-6).

Loading of samples was in accordance to ASCE.The intensity of live and dead load of floors and roof was considered 600, 500, 200 and 150 kg/m^2 respectively, and lateral force distribution of building was done based on Iranian Earthquake Standard 2800 assuming the terrain type 3 and based acceleration scheme of 0.35. in modeling steel ST37 with Poisson's ratio of 0.3 and elasticity modulus of 210 GPa was used.

Canadian Standards Association (CAN/CSA S16-01) and American Institute of Steel Construction (AISC 2005) have adopted SPSW as a lateral load resisting system. In these Regulations to design a SPSWs, first initial design of beam sections, columns and plate walls are done with a vertical truss with tension-only braces are made in. Based on this, instead of steel plate, a brace may be considered equivalent. Derived an equation for α that takes the following form

$$\tan\alpha = \sqrt[4]{\frac{1+\dfrac{tL}{2A_C}}{1+th\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)}} \tag{1}$$

Where t is the thickness of the infill plate A_c and A_b are the cross-sectional areas of the column and beam, respectively.

In order to simplify the iterative process of designing a steel plate shear wall, Thorburn *et al.* (1983) developed a pratt truss model, known as the equivalent brace model, that is illustrated in Fig. 2.

The infill plate at a single story is modelled as a single diagonal tension-only brace interesting the working points of the frame. The diagonal brace represents the stiffness characteristics of the tension field in the infill plate, assuming rigid boundary elements. The equation for the area of the

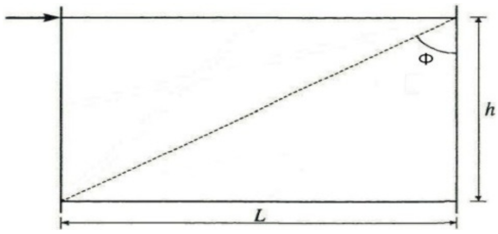


Fig. 2 Equivalent Brace Model (Thorburn *et al*. 1983)

Table 1 Elements section of panel beam, link beam and column

Section No.	Section name	Section type	*H* (Cm)	*S* (Cm)	*B* (Cm)	*t* (Cm)
B1	Pl30×0.8-15×0.8	Beam	30	0.8	15	10.8
B2	Pl30×0.8-15×0.8	Beam	30	1.5	15	1.5
B3	Pl34×2-15×2	Beam	34	2	15	2
B4	Pl42.7×1.2-28.2×2	Beam	42.7	1.2	28.2	2
B5	Pl42.8×2.3-28.9×4	Beam	42.8	2.3	28.9	4
C1	Bo×30×1.5	Column	30	30	1.5	1.5
C2	Bo×30×2	Column	30	30	2	2
C3	Bo×35×1.5	Column	35	35	1.5	1.5
C4	Bo×35×2	Column	35	35	2	2
C5	Bo×40×2	Column	40	40	2	2
C6	Bo×40×3	Column	40	40	3	3
C7	Bo×50×2	Column	50	50	2	2
C8	Bo×50×3	Column	50	50	3	3
C9	Bo×70×2	Column	70	70	2	2

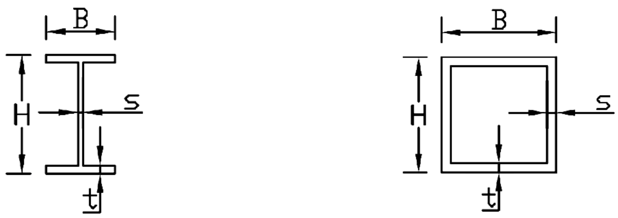


Fig. 3 Elements section type

Table 2 Sections of panel beam and around, link beam and the thickness of coupled steel plate shear wall in 3-story model

Story	Column sections	*t* (mm)	Panel beam section	Side column sections	Side beam section	Link beam sections
1	Bo×30×2	1.71	Pl30×0.8-15×0.8	Bo×30×1.5	Pl30×0.8-18×0.8	Pl30×0.8-15×0.8
2	Bo×30×2	1.71	Pl30×0.8-15×0.8	Bo×30×1.5	Pl30×0.8-15×0.8	Pl30×0.8-15×0.8
3	Bo×30×1/5	1.11	Pl30×0.8-15×0.8	Bo×30×1.5	Pl30×0.8-15×0.8	Pl30×0.8-15×0.8

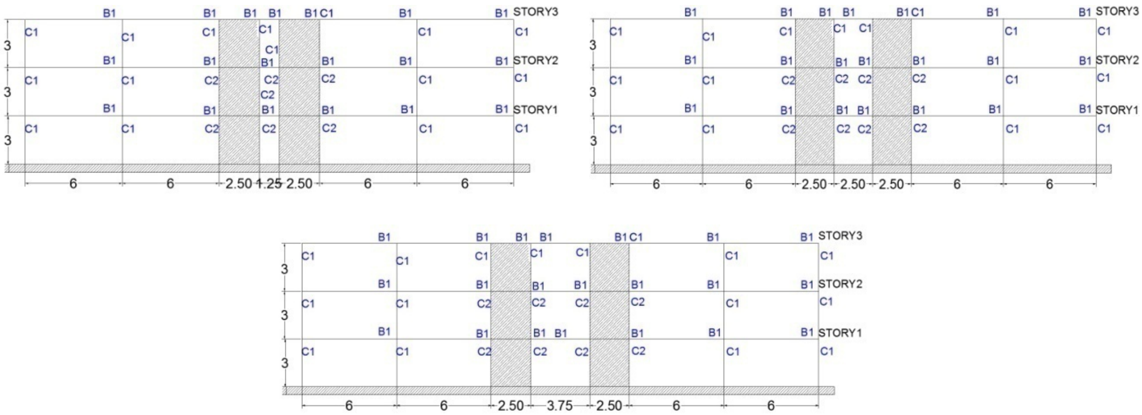


Fig. 4 3-story model with link beam length of 1.25, 2.5 and 3.75 m

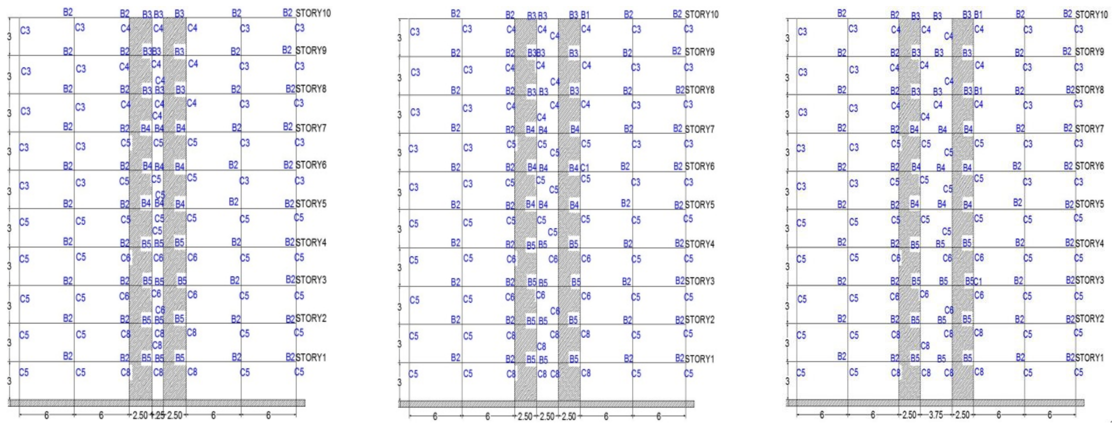


Fig. 5 10-story model with link beam length of 1.25, 2.5 and 3.75m

Table 3 Sections of panel beam and around, link beam and the thickness of Coupled steel plate shear wall in 10-story model

Story	Column sections	*t* (mm)	Panel beam section	Side column sections	Side beam section	Link beam sections
1	Bo×50×3	5.38	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
2	Bo×50×3	5.38	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
3	Bo×40×3	4.86	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
4	Bo×40×3	4.86	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
5	Bo×40×2	3.77	Pl42.7×1.2-28.2×2	Bo×40×2	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
6	Bo×40×2	3. 24	Pl42.7×1.2-28.2×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
7	Bo×40×2	3.24	Pl42.7×1.2-28.2×2	Bo×35×1. 5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
8	Bo×35×2	2.42	Pl34×2-15×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2
9	Bo×35×2	1.71	Pl34×2-15×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2
10	Bo×35×2	1.11	Pl34×2-15×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2

Table 4 Sections of panel beam and around, link beam and the thickness of Coupled steel plate shear wall in 15-story model

Story	Column sections	t (mm)	Panel beam section	Side column sections	Side beam section	Link beam sections
1	Bo×70×3	5.38	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
2	Bo×70×3	5.38	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
3	Bo×70×3	4.86	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
4	Bo×70×3	4.86	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
5	Bo×50×2	4.26	Pl42.8×2.3-28.9×4	Bo×40×2	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
6	Bo×50×2	4.26	Pl42.8×2.3-28.9×4	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
7	Bo×50×2	3.77	Pl42.8×2.3-28.9×4	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.8×2.3-28.9×4
8	Bo×40×2	3.24	Pl42.7×1.2-28.2×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
9	Bo×40×2	2.05	Pl42.7×1.2-28.2×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
10	Bo×40×2	1.71	Pl42.7×1.2-28.2×2	Bo×35×1.5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
11	Bo×40×2	1.71	Pl42.7×1.2-28.2×2	Bo×30×1.5	Pl30×1.5-15×1.5	Pl42.7×1.2-28.2×2
12	Bo×35×2	1.36	Pl34×2-15×2	Bo×30×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2
13	Bo×35×2	1.36	Pl34×2-15×2	Bo×30×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2
14	Bo×35×2	1.11	Pl34×2-15×2	Bo×30×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2
15	Bo×35×2	1.11	Pl34×2-15×2	Bo×30×1.5	Pl30×1.5-15×1.5	Pl34×2-15×2

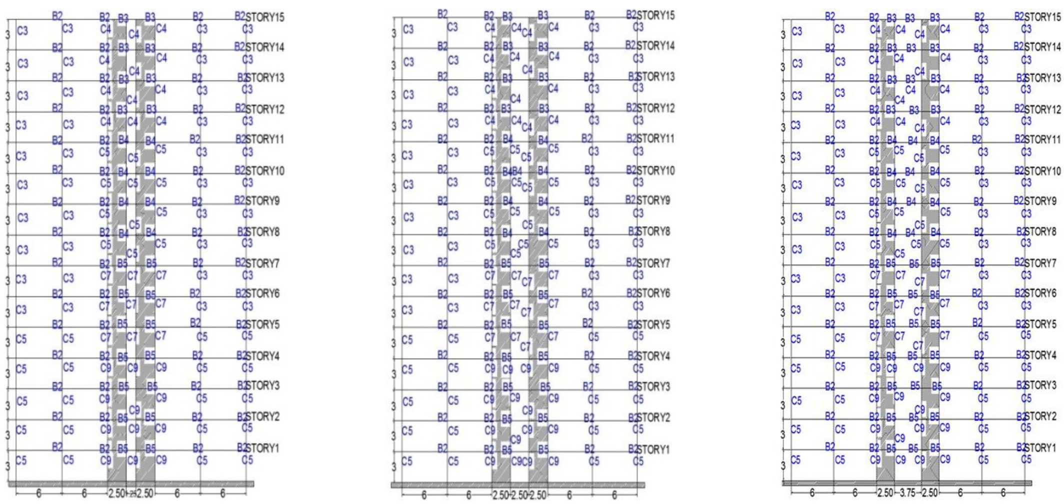


Fig. 6 15-story model with link beam length of 1.25, 2.5 and 3.75 m

brace is as follows

$$A = \frac{tL\sin^2 2\alpha}{2\sin\varphi \sin 2\varphi} \tag{2}$$

where φ is the acute angle of the brace with respect to the column and all other parameters are as defined above.

After determining the cross-sectional area of each brace (based on elastic strain energy relations) steel plate thickness was achieved. In this paper, the design has been done based on this and layout of the link beam included both flexural and shear behavior. It should be noted that the levels of columns of box, beam sections of plate beam, and sections of braces was chosen equal to double channel. Tables 1-4 show the summary of designed sections for 3, 10 and 15 story models.

3. Model verification and calibration

To calibrate finite element models of a thin plate three-story SPSW which has been tested in 2007 by Gholhaki was used. The mentioned wall had rigid connection in the plate of panels and columns thus soft and high-strength steel has been used in plates and columns. Outline and dimensions of the beam and column of SPSW are shown in Fig. 7 and mechanical properties of the components used are shown in Table 5. The σ_0 and E are yield stress and modulus of elasticity, respectively.

In this research, a finite element (FE) model for the C-SPSW specimen was constructed using the commercially available finite element software package ABAQUS. The majority of this FE model, including the infill plates and the boundary elements, was constructed using the 4-node, quadrilateral, stress/displacement shell elements with the reduced integration and a large strain

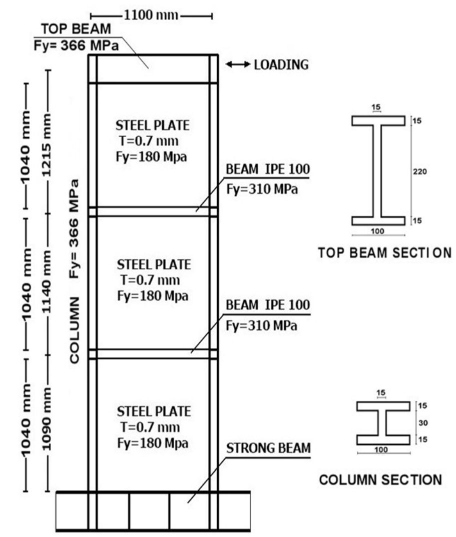


Fig. 7 Overview and details of laboratory sample

Table 5 The mechanical properties of SPSW specimens

Members	σ_0 (N/mm^2)	E (kN/mm^2)
Plate	180	206
Column	366	206
Central beams	310	206
Upper beams	366	206

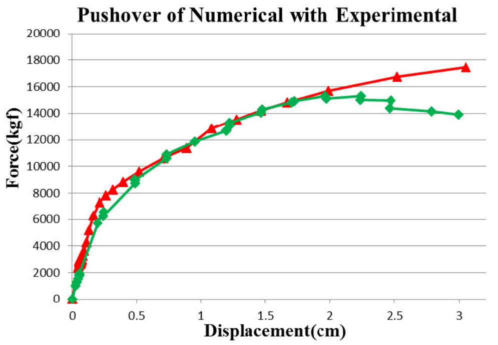


Fig. 8(a) Finite element pushover

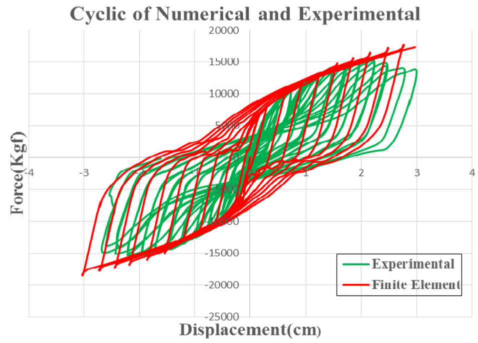


Fig. 8(b) Finite element hysteresis with test results

formulation (ABAQUS S4R Element). Shell S4R elements were also used in the modeling. For simplicity, two linear plasticity models with kinematic hardening and the Von-Mises function were used. The slope of hardening part in stress-strain curve was considered 3% of the elastic part and using sensitivity analysis and to begin buckling of diagonal tension field 3 mm initial distortion was applied to the middle of the plate. After conducting sensitivity analysis to the mesh size of 10 cm was used. (Fig. 8(a)-(b)) compares the behavioral results of laboratory specimens and finite elements in which mesh of 10 cm was used.

4. Modelling of samples in finite element software

Each system according to its components can contain different types of elements. Although while modeling the boundary members can be modeled by the line beam element, but due to the possibility of local buckling in them, plate and boundary members are modeled with shell element a four-node, two curved element reduced with integration. Each node of this element has 6 degrees of freedom, three degrees of transfer and 3 rotational degrees. Behavioral model included nonlinear geometric and material behavior, based on a two-line curves of elasto-plastic in which the used steel is ST37 and the amount of hardening is 3% and the mesh size is 10 cm.

5. The study of nonlinear dynamic response (Time history)

In nonlinear time history analysis, structural behaviour was observed partially during earthquakes and this behaviour represents the more realistic behaviour of structure during an earthquake than the other analyses. In this analysis, the effect of frequency content, the maximum acceleration and the effective duration of earthquake was well observed and it was identified that how two different earthquakes with identical maximum acceleration have different destructive effects of on a structure.

5.1 Used accelerograms

According to the Iranian 2800 standard the accelerograms that are used in determining the movement of the earth should represent the actual movement of the ground during an earthquake as much as possible. The duration of strong ground motion in accelerograms is at least 10 seconds

Table 6 Characteristics of selected earthquake

Row	Name of earthquake	Year	Station	Soil	Distance	PGA (M/S^2)	ΔT	Effective duration	Selected interval
1	Northridge	1994	CDMG 13122	3	82.3	0.104	0.02	10	21-11
2	LomaPrieta	1989	CDMG 58223	3	72.2	0.329	0.05	11	20-9
3	Tabas	1978	FERDOWS 71	3	91.1	0.107	0.02	26	36-10

Table 7 Main period of each structure based on modal analysis of 3, 10, and 15-story models

Row	1.5 T	0.2 T	Period	Model - length of link beam
	3 story-1.25 m	0.455	0.0607	0.3035
2	3 story-2.5 m	0.469	0.063	0.3127
3	3 story-3.75 m	0.489	0.065	0.3216
4	10 story-1.25 m	1.083	0.144	0.722
5	10 story-2.5 m	1.183	0.152	0.759
6	10 story-3.75 m	1.257	0.168	0.8378
7	15 story-1.25 m	1.751	0.233	1.167
8	15 story-2.5 m	1.905	0.254	1.27
9	15 story-3.75 m	2.07	0.276	1.38

or three times the original period of construction, whichever is greater, shall be elected. In this paper, according to Iranian Earthquake Regulations 2800 and assuming the soil type 3 and reviewing the accelerograms, finally three Northridge earthquake, Tabas (Iran) and Loma Prieta (California, America) were selected and all accelerograms to 0.35g scale and characteristics of these earthquakes can be seen in Table 6.

Iranian 2800 standard was used in order to scale the accelerograms. So that after scaling the accelerograms to their maximum value, the acceleration response spectra of each pair of scaled horizontal accelerograms were obtained by considering the 5% damping with SeismoSignal software after combining the response spectra, each pair of acceleration averaged with square root method, should be compared with the standard design spectrum at interval periods of 0.2 T and 1.5 T. In order to review the within period intervals, main period of structure should be obtained, thus the main period of structure obtained using modal analysis which can be seen in Table 7.

From Table 7 it can be understood that increasing the length of the link beam increases in the Coupled steel plate shear wall systems of structure period. On the other hand in order to compare the structures with different length of links beam the larger period was considered of to have the identical scale factor and loading of structures.

6. The nonlinear dynamic analysis

After creating the models in ABAQUS software, models analyzed by nonlinear dynamic, then shears, drift and DC of models were obtained. Therefore, first of all base shear was calculated then the drift of floors results, at the end DC was studied.

6.1 Base shear

Fig. 9 illustrates the diagrams of maximum shear of floors at the height of 3, 10 and 15 story models based on accelerograms. As can be seen, at 3-story model, the maximum base shear during Northridge earthquakes at 1.25 m link beam was13% and 20% respectively, greater than the length of link beam 2.5 and 3.75 meters and maximum base shear obtained from Loma Prieta earthquake at 1.25 m link beam was 4% and 13% respectively, greater than the length of link beam 2.5 and 3.75 m and maximum base shear of Tabas earthquake at 1.25 m link beam was 3% and 4% respectively, greater than the length of 2.5 and 3.75 meters link beam. At 10-story model, the maximum base shear during Northridge earthquakes at 1.25 m link beam was 10% and 33% respectively, greater than the length of link beam 2.5 and 3.75 meters and maximum base shear obtained from Loma Prieta earthquake at 1.25 m link beam was 13% and 19% respectively, greater than the length of link beam 2.5 and 3.75 m and maximum base shear of Tabas earthquake at 1.25 m link beam was approximately equal with the length of 2.5m link beam. At 15-story model, the

maximum base shear during Northridge earthquakes at 1.25 m link beam was 3% and 6% respectively, greater than the length of link beam 1.25 and 3.75 meters and maximum base shear obtained from Loma Prieta earthquake at 1.25 m link beam was 50% and 57% respectively, greater than the length of link beam 2.5 and 3.75 meters and and maximum base shear of Tabas earthquake at 1.25 m link beam was 12% and 45% respectively, greater than the length of 2.5 and 3.75 meters link beam.These results indicate that regardless to the type of accelerograms, with an increase in the length of link beam the base shear is reduced in all models.

As can be seen in Fig. 10 in Northridge, Loma Prieta and Tabas earthquake within the range of 0.3035 to 1.38 second shear/weight ratio of structure or base shear coefficient (C) at the length of

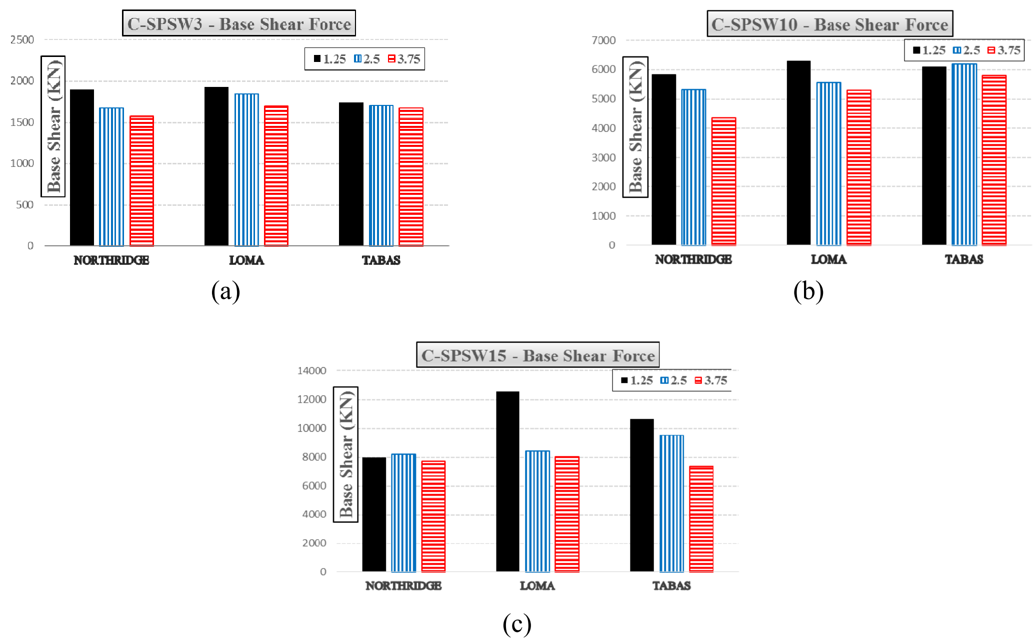
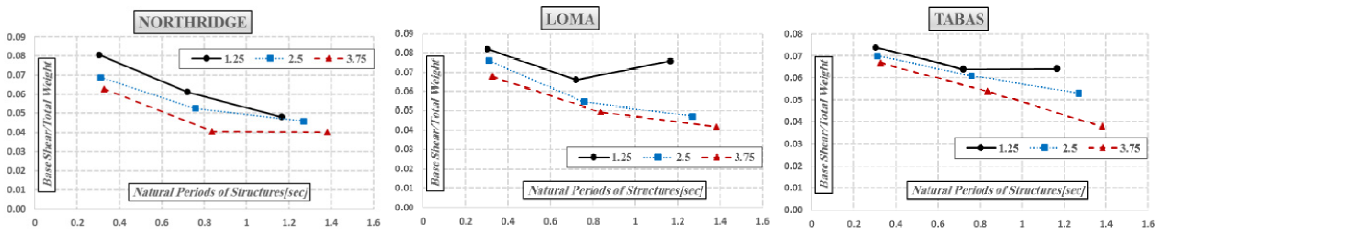


Fig. 9 Diagram of maximum base shear based on accelerograms: (a) 3-story model; (b) 10-story model; (c) 15-story model



(a) (b) (c)

Fig. 10 Diagram of base shear to the total weight of the structure against the natural period of structure with respect to the earthquakes in 3, 10 and 15 stories models: (a) Northridge; (b) Loma Prieta; (c) Tabas

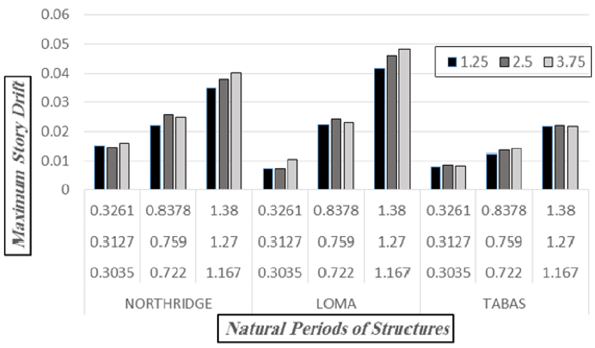


Fig. 11 Diagram of maximum drift of story at each period of structure based on accelerograms

1.25 m link beam is more than the 2.5 and 3.75 meters, in other words in all models, regardless of the accelerograms type; increased period of structures (due to the increase in the length of link beam) decreases the C coefficient.

6.2 Drift

Fig. 11 shows the maximum drift of floors in each period of structure separately for each earthquake accelerograms. Drift as a significant factor has many applications in the design and evaluation of structure. According to Fig. 11, the maximum drift values are storyified in different frames of accelerograms at the interval period 0.3035 to 1.38 sec that is the natural period of coupled steel plate shear wall frames and as it can be seen with the increase in construction period (by increasing the length of link beam), and increasing the number of floors, drift increases at coupled steel plate shear wall.

6.3 Degree of coupling (*DC*)

Using the study of Borello and Fahnestock Degree of coupling is expressed according to Eq. (3).

$$DC = \frac{M_{COUP}}{M_{TOTAL}} = \frac{M_{COUP}}{\sum M_{VBE} + \sum M_{PIER} + M_{COUP}} \tag{3}$$

In which:

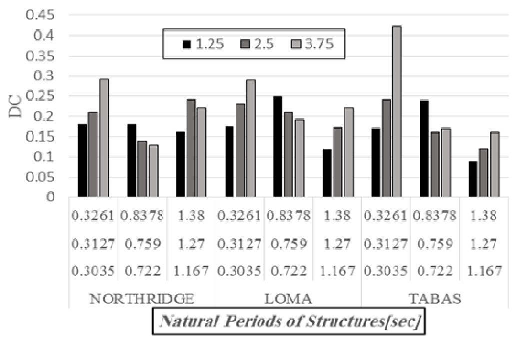


Fig. 12 The degree of coupling diagram in each structure period based on accelerograms

M_{COUP}: Link beam-induced coupled moment (N.m)
M_{VEB}: Moment of each pier (N.m)
M_{PIER}: Coupled moment at each panel of SPSW (N.m)

Fig. 12 shows the maximum degree of coupling based on the period of structure for different accelerograms and values of basic coupling degree (DC) in different frames of accelerograms according to the periodic interval 0.3035 to 1.38 second (the main period of Coupled steel plate shear wall). As can be seen in the periods of 0.3035, 0.3127 and 0.3261 second which represent 3-story frame, the link beam length are 1.25, 2.5 and 3.75 meters and in the periods of 1.167, 1.27 and 1.38 second that represents the 15-story frame, respectively, the length of link beam are 1.25, 2.5 and 3.75 meters, in the all existing mappings with increasing the link beam the degree of coupling increases, while in the periods of 0.722, 0.759 and 0.8378 second, which indicates that the 10-story frame, with the length of link beams of 1.25, 2.5 and 3.75 meters respectively, through increasing link beam the degree of coupling reduces. Accordingly, increasing the length of the link beam in coupled steel plate shear wall has various positive and negative effects on the degree of coupling.

7. The height distribution of structural response parameters

Relative height was used to compare the effect of height on the structural response parameters of all three structures in various link beam length. Figs. 13 and 14 show the height distribution of maximum shear and drift models respectively, and Fig. 15 shows the height distribution of base DC.

As can be seen in the diagrams of Fig. 13, the shear distribution of story to the effective weight of structure will be increased continually in higher floors. Fig. 13 shows the ratio of maximum shear of story to the cumulative weight of structure induced by nonlinear dynamic analysis for three coordinated accelerograms against the percentage of structure height in 3, 10, and 15-story models. According to the Fig. 13 in 3, 10, and 15-story model the maximum shear story to the cumulative weight in all the mappings with link beam length of 1.25 meter is more than 2.5 meter and 3.75 meter, also at link beam of 1.25 meter compared to 2.5 and 3.75 meters with relatively parallel to the slope will also increase in which constant slope of these diagrams shows the shear consistent distribution of floors to the ratio of effective weight in Coupled steel plate shear wall with this height.

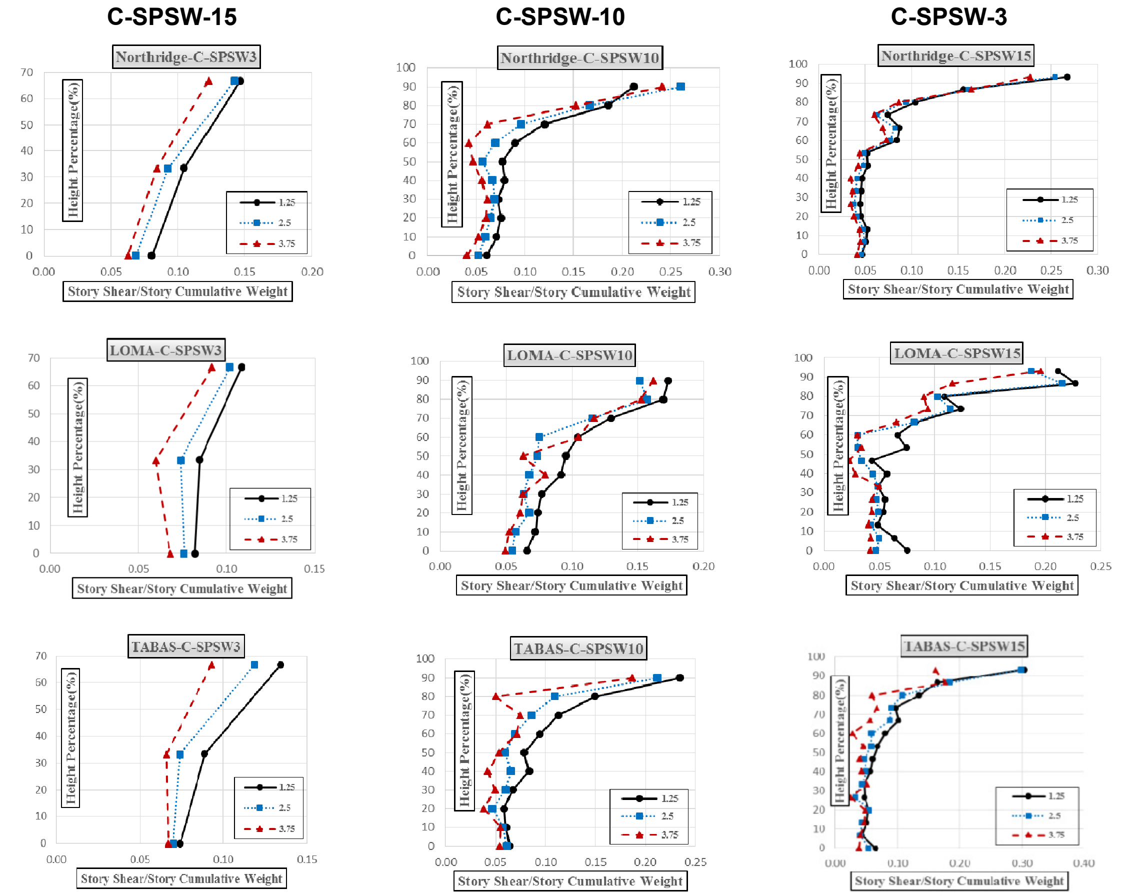


Fig. 13 The shear distribution diagram of floors based on the effective weight of structure in 3, 10 and 15-story models according to their accelerograms

As can be seen in the diagrams of Fig. 14, the increase in the height increases the drift of nonlinear dynamic analysis. In Northridge earthquake in 65% to 80% by changing the height of structure from 3 to 10 story drift lowers, and in 40% to 50% by changing the structure height from10 to 15 the drift increases.

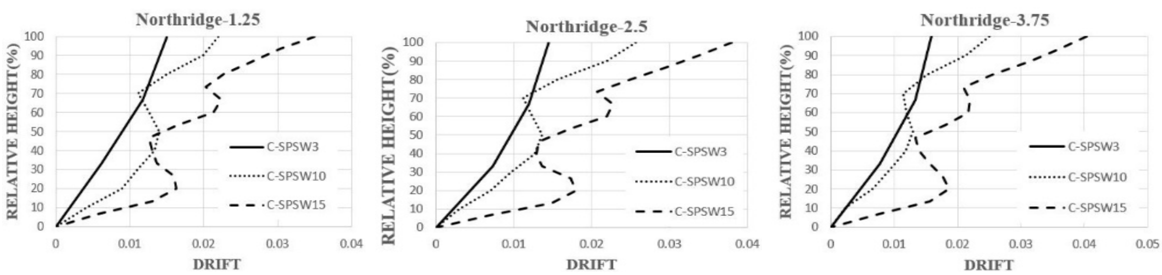


Fig. 14 The diagram of height distribution of drift 3, 10 and 15-story models based on accelerograms

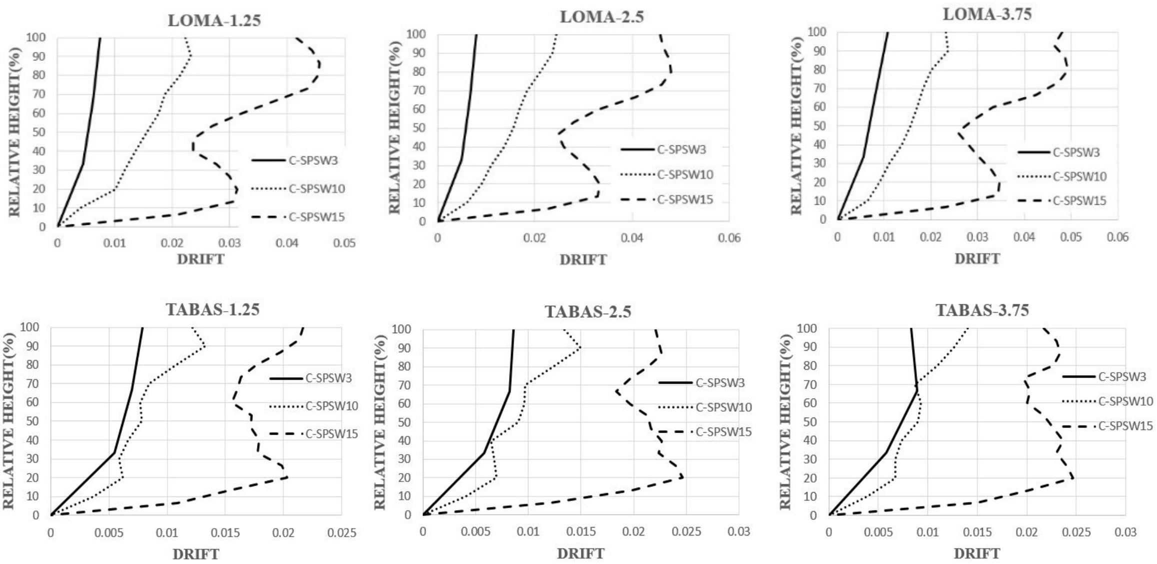


Fig. 14 Continued

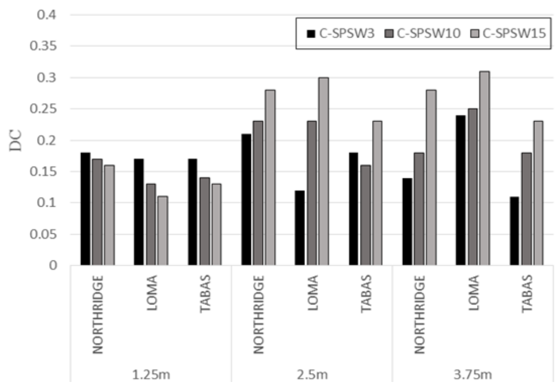


Fig. 15 The diagram of height distribution of basic degree of coupling at the span length of 1.25, 2.5 and 3.75 meters regarding the accelerograms

As can be seen in Fig. 15, the degree of coupling resulted from the nonlinear dynamic analysis at various span length of link beam by increasing the number of floors decreases for the 1.25 meter and increases for the length of 2.5 and 3.75 meters.

8. Nonlinear behavior of the link beams

Relative height was used to compare the effect of demand / capacity ratio based on the shear and moment of all three structures in various link beam length. Figs. 16 and 17 show the height distribution of the demand / capacity ratio of the link beam based on the shear and moment models, respectively and Fig. 18 show plastic mechanism of C-SPSW.

The demand / capacity ratio of link beams based on the shear distribution of story are shown in Fig. 16 and will be decreased continually in higher floors that induced by nonlinear dynamic analysis for three coordinated accelerograms against the percentage of structure height in 3, 10, and 15-story models.

According to the Fig. 16 in 3, 10, and 15-story model, the demand / capacity ratio of link beams based on the shear in all the mappings with link beam length of 1.25 meter is more than 2.5 meter and 3.75 meter, also at link beam of 1.25 meter compared to 2.5 and 3.75 meters with relatively parallel to the slope will also decrease in higher floors in which constant slope of these diagrams shows the shear demand / capacity ratio consistent distribution of link beam of floors in Coupled steel plate shear wall with this height. As can be seen, at 3, 10 and 15-story model, the shear ratio during Northridge, Loma Prieta and Tabas earthquakes at 1.25 m link beam was 10% and 25% respectively, greater than the length of link beam 2.5 and 3.75 meters.

Fig. 17 shows the demand / capacity ratio of link beams based on the moment distribution of story will be decreased continually in higher floors that induced by nonlinear dynamic analysis for three coordinated accelerograms against the percentage of structure height in 3, 10, and 15-story models.

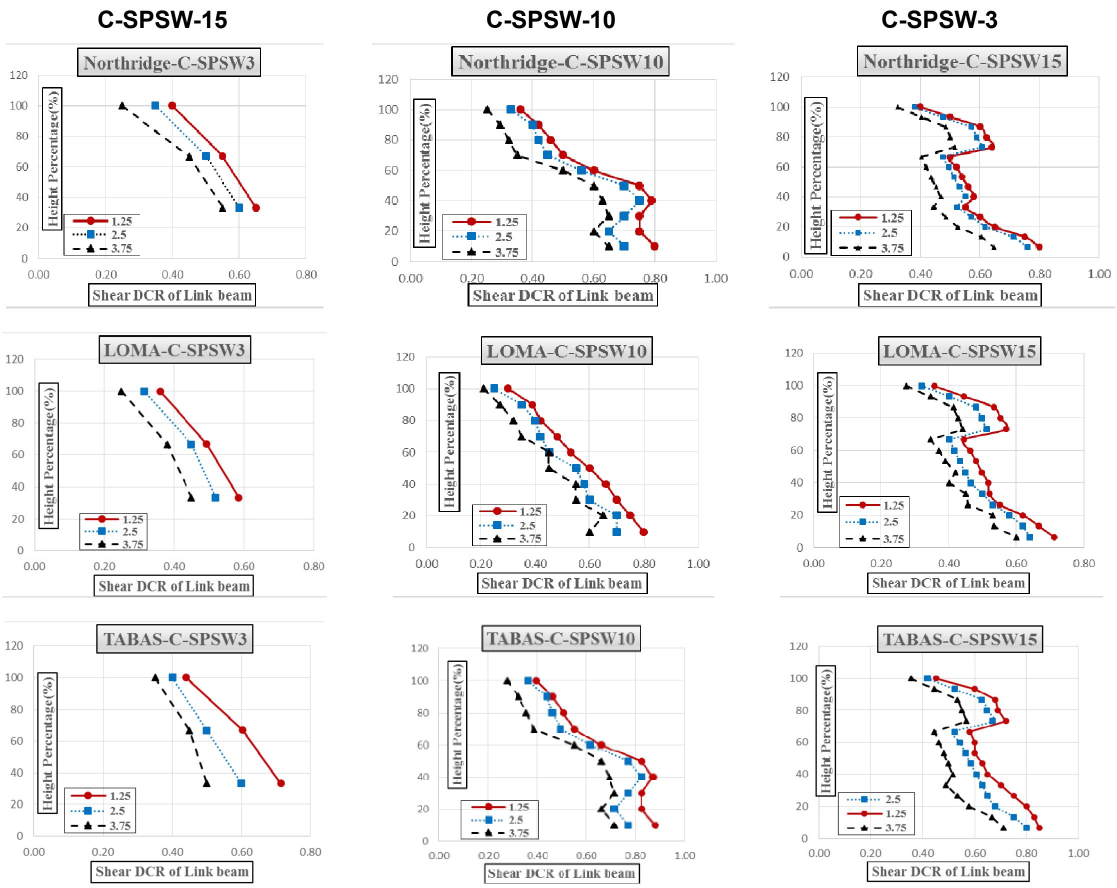


Fig. 16 The diagram of height distribution of demand / capacity ratios of the link beams based on shear at the link beam length of 1.25, 2.5 and 3.75 meters regarding the accelerograms

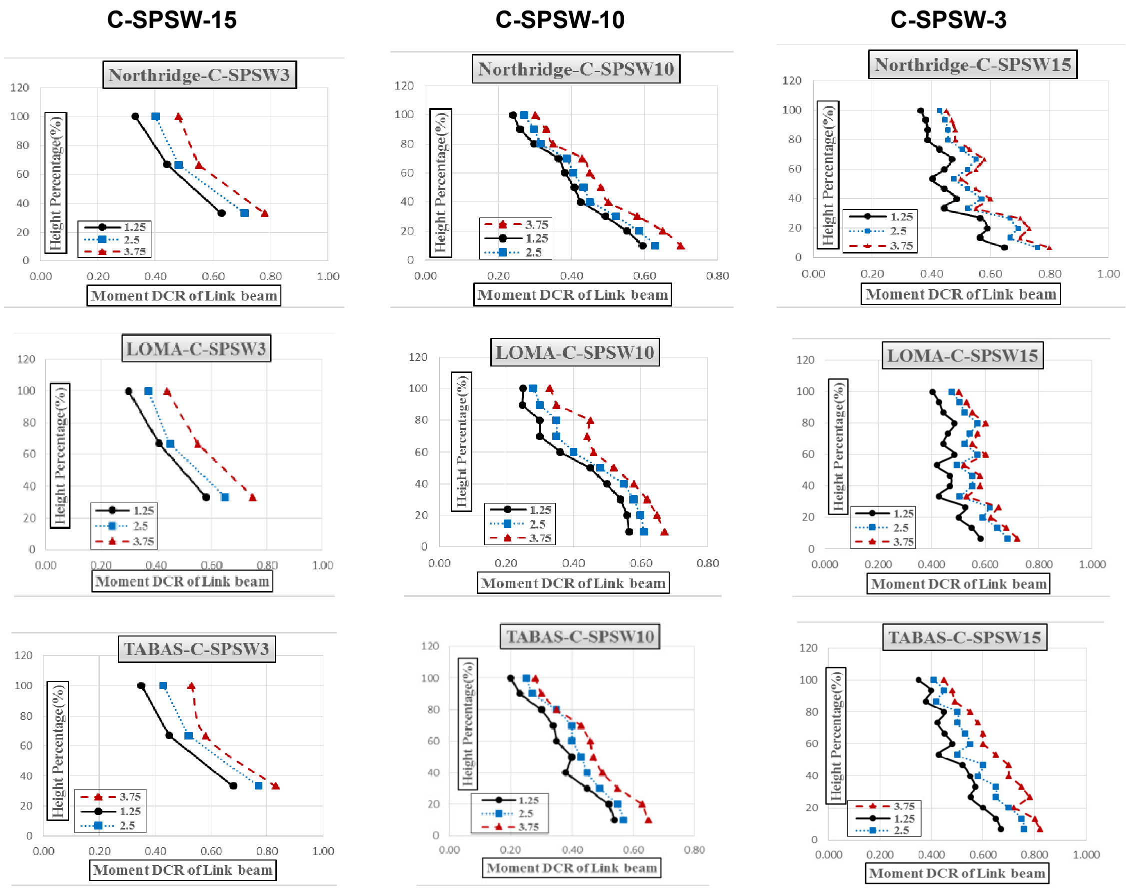


Fig. 17 The diagram of height distribution of demand / capacity ratios of the link beams based on moment at the link beam length of 1.25, 2.5 and 3.75 meters regarding the accelerograms

According to the Fig. 17 in 3, 10, and 15-story model, the demand / capacity ratio of link beams based on the moment in all the mappings with link beam length of 3.75 meter is more than 2.5 meter and 1.25 meter, also at link beam of 1.25 meter compared to 2.5 and 3.75 meters with relatively parallel to the slope will also decrease in higher floors in which constant slope of these diagrams shows the shear demand / capacity ratio consistent distribution of link beam of floors in Coupled steel plate shear wall with this height. As can be seen, at 3, 10 and 15-story model, the shear ratio during Northridge, Loma Prieta and Tabas earthquakes at 1.25 m link beam was 14% and 22% respectively, greater than the length of link beam 2.5 and 3.75 meters.

By extending the provisions in the current U.S. building code (AISC 2005) for the link beam of the eccentrically braced frame (EBF) to the prediction for the yielding mechanism of the coupling beam, the coupling beam will yield in shear when its length, e, is smaller than 1.6 Mp/Vp, where Mp and Vp are the plastic flexural strength and plastic shear strength of the coupling beam, respectively; and the coupling beam will develop flexural plastic hinges at its both ends when its length, e, bigger than 2.6 Mp/Vp.

Based on the classification used in the Seismic Provisions for eccentrically braced frame (EBF) links, the plastic mechanism results have been shown in the Table 8-10.

Table 8 Plastic mechanism of the link beam length of 1.25, 2.5 and 3.75 meters in 3, story

Story	Link beam sections	Plastic mechanism		
		1.25 m	2.5 m	3.75 m
1	Pl30×0.8-15×0.8	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
2	Pl30×0.8-15×0.8	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
3	Pl30×0.8-15×0.8	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge

Table 9 Plastic mechanism of the link beam length of 1.25, 2.5 and 3.75 meters in 10 story

Story	Link beam sections	Plastic mechanism		
		1.25 m	2.5 m	3.75 m
1	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
2	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
3	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
4	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
5	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
6	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
7	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
8	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
9	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
10	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge

Table 10 Plastic mechanism of the link beam length of 1.25, 2.5 and 3.75 meters in 15 story

Story	Link beam sections	Plastic mechanism		
		1.25 m	2.5 m	3.75 m
1	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
2	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
3	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
4	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
5	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
6	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
7	Pl42.8×2.3-28.9×4	Yield in Shear	Shear & Flexural	Flexural plastic hinge
8	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
9	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
10	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
11	Pl42.7×1.2-28.2×2	Yield in Shear	Shear & Flexural	Flexural plastic hinge
12	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
13	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
14	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge
15	Pl34×2-15×2	Flexural plastic hinge	Flexural plastic hinge	Flexural plastic hinge

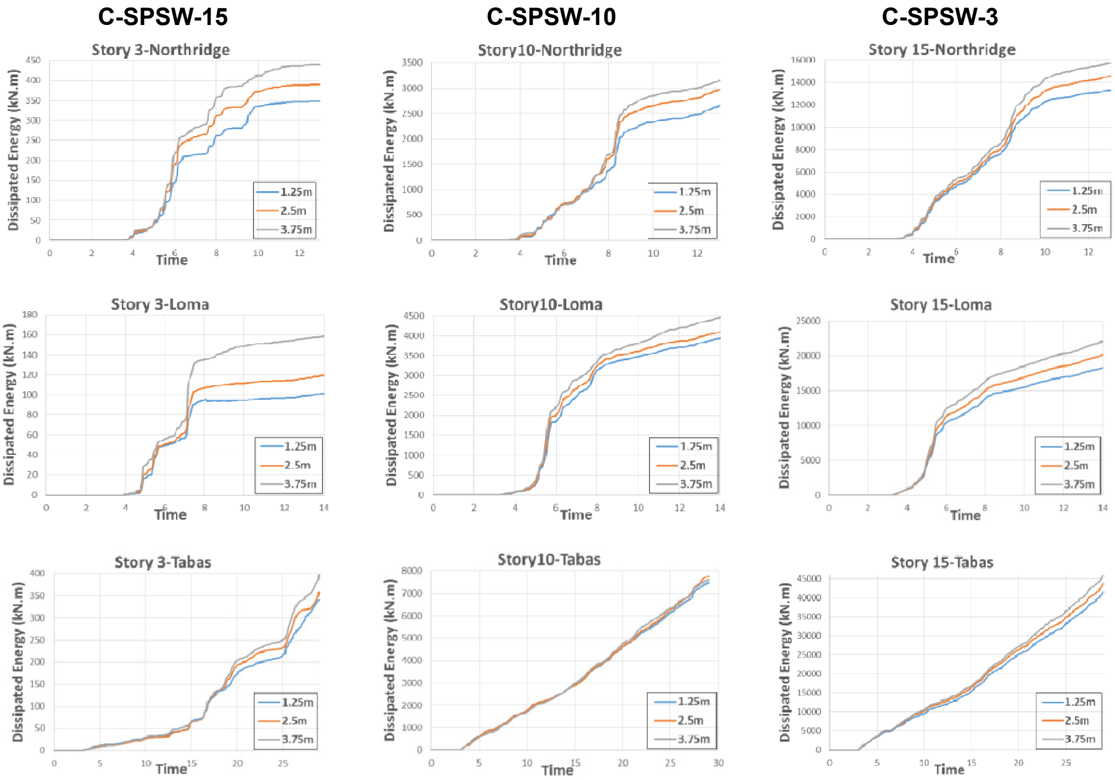


Fig. 18 The diagram of energy dissipated of the link beams length of 1.25, 2.5 and 3.75 meters regarding the accelerograms

9. Energy dissipated of link beam

Values of dissipated energy were calculated during excitation, effect of link beam length in samples of 3, 10 and 15-story on energy dissipation were investigated, in Fig. 18. As illustrated in Fig. 18 dissipated energy was almost zero during primary and weak motions. As the level of excitation suddenly increases, dissipated energy increase considerably. These results indicate that regardless to the type of accelerograms, with an increase in the link beam length, the dissipated energy is increased in all models.

10. Conclusions

In this study the behavior of Coupled steel plate shear wall with beam connections to the rigid column was investigated by the use of nonlinear dynamic analysis. Thus, three C-SPSWs with 3, 10 and 15 story, were examined in models at three link beam length of 1.25, 2.5 and 3.75 meters. The samples of Northridge, Loma Prieta and Tabas earthquakes were analyzed and their base shear, drift, structure period, the degree of coupling, demand/capacity ratio, plastic mechanism and dissipated energy were evaluated. Results showed that increasing the length of the link beam in coupled steel plate shear wall increases structure period, drift and reduces the base shear. The results also showed that increasing the length of link beam had additive and subtractive effects on the degree of coupling and the dissipated energy is increased. In the 1.25 m link beam, with increasing the number of floors, degree of coupling reduces, in the 2.5 m and 3.75 m with increasing the number of floors, degree of coupling increases.

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