JANUARY 2017 M.Sc. in Civil Engineering DARA MOHAMMED MAHMOOD

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EFFECT OF ASPECT RATIO AND THICKNESS OF SPSW ON NONLINEAR RESPONSE OF STEEL BUILDINGS

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DARA MOHAMMED MAHMOOD

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Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By

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This is to certify that we have read this thesis and that in our consensus opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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Dara Mohammed MAHMOOD

ABSTRACT

EFFECT OF ASPECT RATIO AND THICKNESS OF SPSW ON NONLINEAR RESPONSE OF STEEL BUILDINGS

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The steel plate shear wall (SPSW) is considered as one of the effective and economical methods providing a good lateral load resisting system for the structures due to its ductility and high strength. However, several parameters such as the design properties of SPSW and its compatibility with the structure govern its structural performance. In this study, the behavior of different steel buildings employing with steel plates as infill panels was investigated numerically. To this aim, two cases were studied. In the first case, 2-story, 4-story, 6-story, and 8-story steel buildings with and without SPSWs were examined. The plate thickness in this case varied from 1 to 15 mm while the aspect ratio of the plate was kept constant. In the second case, 4-story steel building with SPSWs having different aspect ratios of 0.29 to 2.14 and various plate thicknesses of 1 to 15 mm were evaluated. For both cases, the nonlinear static (pushover) analysis was used to estimate the inelastic strength and deformation capacity. The strip model was adopted for modeling the steel plate. The numerical analysis developed and described in this study with different nonlinear properties for the infill panel and steel buildings were performed by means of the computer program SAP2000. The analysis of the results showed that the SPSWs caused a significant improvement in the performance of the buildings, depending mainly upon aspect ratio and thickness of the plate used as well as the properties of the frame structures.

Keywords: Aspect ratio; Lateral force; Plate thickness; Steel building; Steel plate shear wall.

ÖZET

ÇELİK PLAKALI PERDE DUVARLARIN EN-BOY ORANININ VE KALINLIĞININ ÇELİK BİNALARIN DOĞRUSAL OLMAYAN TEPKİSİNE ETKİSİ

MAHMOOD, Dara Mohammed
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Çelik plakalı perde duvar, süneklik ve yüksek dayanımlarından ötürü yapılar için iyi bir yanal yük direnç sistemi sağlayan etkili ve ekonomik yöntemlerden biri olarak düşünülür. Bununla birlikte, duvarın tasarım özellikleri ve yapı ile uyumluluğu gibi çesitli parametreler yapısal performansını etkiler. Bu çalışmada, çelik plakalarla dolgu yapılan farklı çelik yapıların davranışları sayısal olarak incelenmiştir. Bu amaçla iki durum çalışıldı. İlk durumda, çelik plakalı ve plakasız 2 katlı, 4 katlı, 6 katlı ve 8 katlı çelik binalar incelendi. Bu durumda plaka kalınlığı 1 ila 15 mm arasında değişirken, plakanın en-boy oranı sabit tutulmuştur. İkinci durumda, farklı en-boy oranları (0.29 ila 2.14) ve çeşitli kalınlıkları (1 ila 15 mm) olan çelik plakalı 4 katlı çelik bina değerlendirildi. Her iki durumda da, doğrusal olmayan statik (itme) analizi, inelastik kuvveti ve deformasyon kapasitesini hesaplamak için kullanılmıştır. Şerit modeli, çelik levhanın modellenmesi için kabul edildi. Bu çalışmada, dolgu paneli ve çelik binalardaki farklı doğrusal olmayan özellikler kullanılarak tanımlanan sayısal analiz, SAP2000 programı kullanılarak gerçekleştirilmiştir. Sonuçların analizi, plakaların en-boy oranı, kullanılan plakanın kalınlığı ve çerçeve yapılarının özelliklerine bağlı olarak binaların performansında belirgin bir iyileşmenin olduğunu göstermiştir.

Anahtar Kelimeler: En-boy oranı; Yanal kuvvet; Plaka kalınlığı; Çelik bina; Çelik plakalı perde duvar.

To my Families

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It is impossible to forget you my mother, I know what I have got in my studying skills is because of your advices and encouragements in all my life by your nice words and spiritual supports.

I remember you my dear wife and I want to acknowledge your tiredness with me because of my journey and study, I will not forget you forever.

Finally, thank you, my God, for this blessing and mercy.

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LIST OF SYMBOLS/ABBREVIATIONS

A_b	Cross-sectional area of beam
A_c	Cross-sectional area of column
F_y	Yield strength
I_b	Moment of inertia of beam section
I_c	Moment of inertia of column section
t_w	Thickness of plate
AISE	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
CSA	Canadian Standards Association
D	Dead load
FEA	Finite Element Analysis
FEM	Finite Element Model
FEMA	Federal Emergency Management Agency
Fu	Tensile strength
h	Story height
HBE	Horizontal Boundary Element
L	Center-to-center distance of columns

LL	Live load
LYP	Low Yield Point
Q	Story shear
RBS	Reduce Beam Section
SPSW	Steel Plate Shear Wall
VBE	Vertical Boundary Element
α	Angle of inclination
δ	Story deflection
φ	Inclination of equivalent brace

CHAPTER 1

INTRODUCTION

1.1 General background

The structural system of buildings must be designed to bear two types of loads, namely gravity, and lateral loads. Gravity loads depending on the self-weight of the structure and its contents, while lateral loads result from the action of the wind, seismic and blast forces on the building. Blast forces have incipient importance in the design of buildings with greater aspect ratio, where wind loads are significant, and also for buildings built in high seismic risk areas. Both of gravity loads and lateral loads requires various considerations in design. Gravity loads usually dictate the dimension of building such as columns and beams. On the other hand, lateral loads different structural elements have to be built in conjugation with the framing system designed to withstand gravity loads (Botros, 2006).

For lateral load resisting system purpose used moment resisting frames (rigid frames), shear walls and braced frames. In moment resisting frames system fully connection between beam and columns used to resist lateral loads. This system usually required big size member especially in bottom level in order to limit the horizontal sway. The rigid frame system is suitable and economical for structures up to twenty stories. They are two types of bracing or of diagonal members between beams and columns or k-bracing system (Figure 1.1). These two types of bracing frame vertical trusses to resist lateral loads. Past experimental improved that this system will be active for buildings up to forty stores. For over forty stories building shear walls most effective compared to rigid frames and bracing frames and most commonly used system (Botros, 2006).

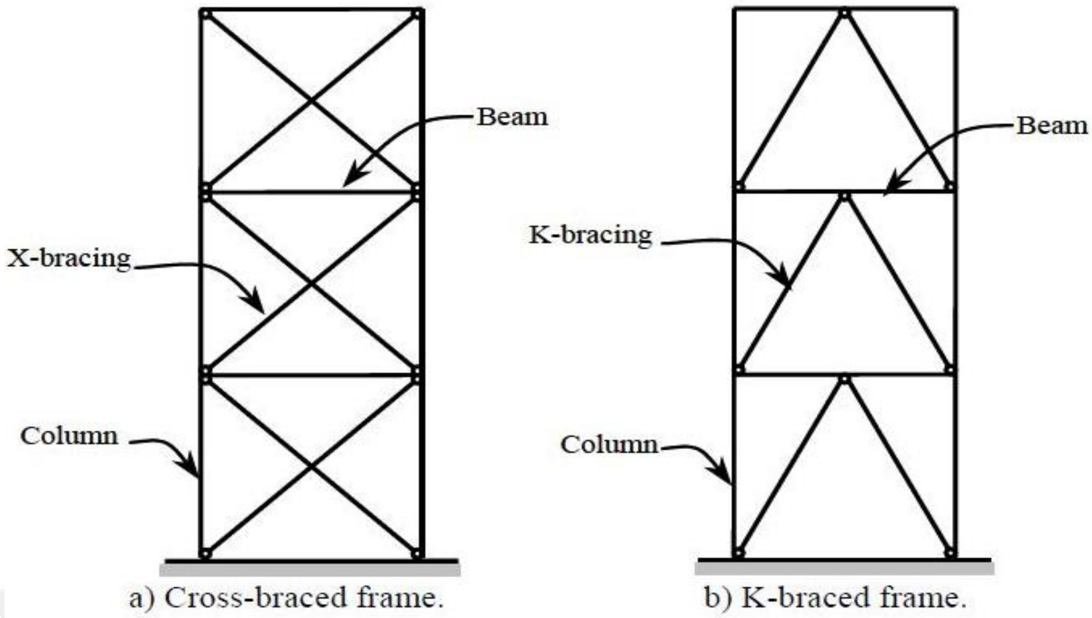


Figure 1.1 Bracing frame system (Botros, 2006)

Steel plate shear walls (SPSWs) consist of a various panel wall surrounding by horizontal boundary element (HBE) and vertical boundary element (VBE) as shown in Figure 1.2. When Lateral loads applied on the outside of the building are transferred through the floor, which acts as a diaphragm, to the shear wall and then transferred to the ground. The shear wall of a high-rise building subjected to wind loads can be idealized as a vertical beam loaded uniformly (Eatherton, 2006).

Figure 1.2 Steel plate shear wall element (Stankevicius, 2011)

There are several divergences of SPSWs with variables such as stiffened and unstiffened infill panels, moment-resisting and simple beam-to-column connections (i.e., the connection may transfer shear forces only or a combination of shear and bending moment, respectively), corrugated and non-corrugated, perforated and non-perforated infill panels, bolted and welded infill panels, and several others. Once the system has been built, it exhibits high initial stiffness for service wind loads and good energy absorption capacity for extreme loading cases such as earthquake events. Reinforced concrete shear walls have been exceedingly used in concrete as well as in steel buildings. Recently, new buildings have been built with steel shear walls made of flat steel plates. Because of their easy and fast building, flat plate shear walls have also been used in strengthening existing steel and concrete structures. In comparison with a rigid frame or reinforced concrete shear wall systems, using steel plate shear panels has shown (Troy and Richard, 1979; Timler, 1998) to be more cost-effective by:

- Reducing the amount of steel required.
- rising the available floor space.
- Increasing the rate of structural steel erection.
- Decrease foundation costs.
- Reducing dead weight while rising the overall stiffness and decreasing the drift index of the structure.

The behavior of the SPSW system under lateral loading is very similar to the behavior of the plate girders under transverse loading (Mahtab and Zahedi, 2008)

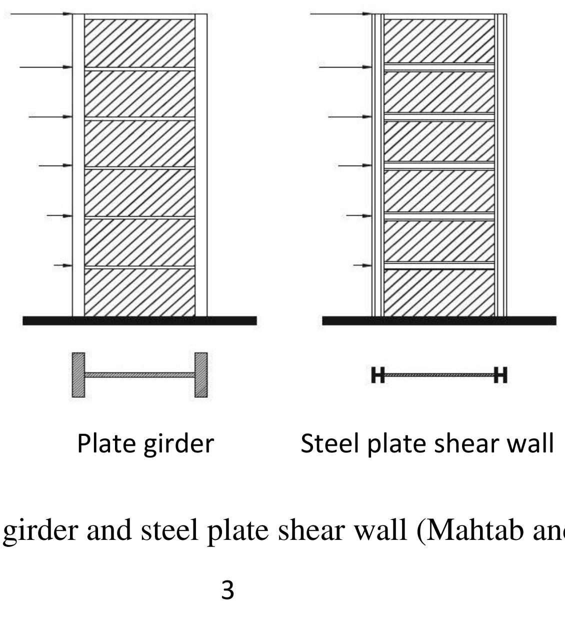


Figure 1.3 Typical plate girder and steel plate shear wall (Mahtab and Zahedi, 2008)

1.2 Types of Steel Plate Shear Wall Systems

Depending on connection beam-to-column, they are two different system types. If the connection between beam-to-column simple the system called standard. Then, steel plate shear walls assumed only lateral force resisting in the system. Otherwise, if the connection between beam-to-column moment-resisting the system called dual system. In this situation, special ductile frames work as a secondary or back-up system to primary lateral force resisting system. Both types of SPSW system shown in Figure 1.4 In addition, design philosophy can be identified for stiffened or unstiffened infill plate (Astaneh-Asl, 2001).

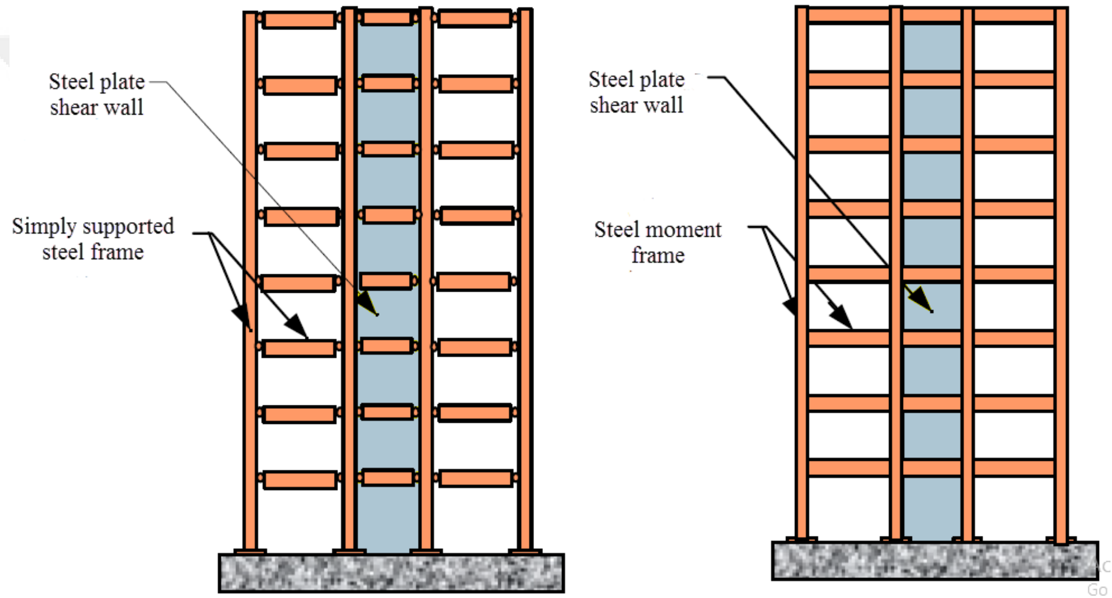


Figure 1.4 Two types of steel plate shear wall systems (Astaneh-Asl, 2001)

Astaneh-Asl (2001) identified that SPSWs in general may be classified into two essential types:

- Stiffened steel plate shear walls: the plate is prevented from buckling by stiffeners.
- Unstiffened steel plate shear walls: plate buckling occurs and system strength is relied on post-buckling strength of the panels by choosing an un-stiffened slender plate.

1.2.1 Stiffened Steel Plate Shear Walls

Until 1980s, the stiffened infill plate used for steel plate shear wall system as a shear panel to increase the shear yield strength and prevent out-of-plane buckling of the plate prior to shear yielding. This design exercise for steel plate shear wall systems is to border their capacity to the elastic buckling strength of the steel panels. During this time, SPSWs have been widely used as the main lateral load resisting system in Japan, United States, and Canada. Initially, until 1980.s, stiffened steel walls were used in Japan in new structure building and in the U.S. for seismic retrofit of the existing buildings as well as in new constructions (Ashish and Harshalata, 2014).

The performance of steel plate shear wall systems where plate panels are designed depending on the elastic buckling strength of the plate panels can be bordered by yielding and buckling of the columns prior to reaching a fraction of the post buckling strength of the steel plate panels. In addition, Basler (1961a) studied on stiffened plate girders in which it was illustrated that post-buckling tension field, along one of the plate panel diagonals, can provide substantial strength, stiffness, and ductility. Figure 1.5 showed a sample of stiffened steel panel with boundary element.

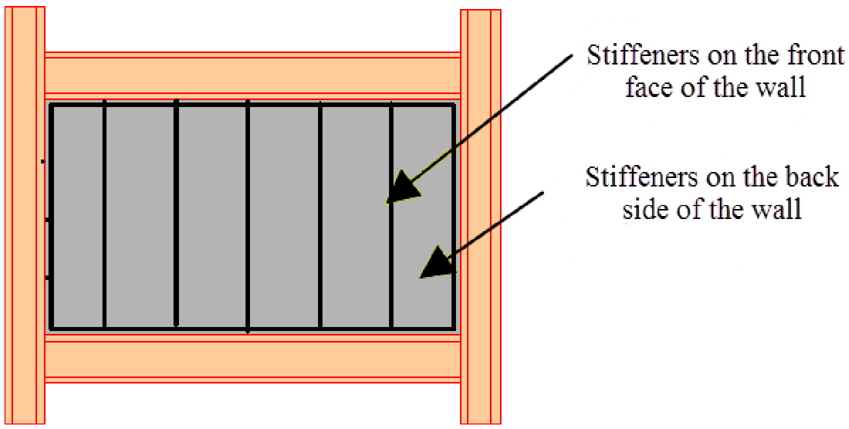


Figure 1.5 Stiffened steel plate shear walls (Astaneh-Asl, 2001)

1.2.2 Unstiffened Steel Plate Shear Walls

During 1980s, the, un-stiffened steel plate shear walls were used in constructions in the U.S. and Canada. Research on SPSWs has taken place over the last 30 years. The results of several static and quasi-static cyclic tests studied on models of changing scales and the analytical studies on the ultimate performance of such SPSWs have illustrated the steady energy absorption capacity of the infill panels and their capability as a main lateral load resisting system for structures. After using unstiffened steel plate as shown in Figure 1.6, an invention of a steel plate shear wall is comparatively simple because connection stiffeners to steel panels by welding is uneconomically and time-consuming. Astaneh-Asl (2001) recommended that stiffened steel shear walls are uneconomically compared to an unstiffened steel plate. Buckling of the plate is not synonymous with failure. The researchers illustrated that buckling does not necessarily represent the limit useful behavior and there is considerable post-buckling strength in an un-stiffened shear panel.

SPSW panels with their comparatively stiff boundary elements, which are horizontal boundary element and vertical boundary element, will have a higher post-buckling strength compare to plate girders with comparatively flexible flanges. Buckling happen after applied low load if use thin panel and the resistance of the panel is controlled by tension field action that grows in the infill plate equivalent to the directions of the principal tensile stresses. The combination of the shear plate tension field with the bending resistance of the surrounding frame elements proved to be an ideal system for achieving high stiffness and strength, while also maintaining a ductile system. Buckling happens at low loads while panel is thin and the resistance of the panel takes control by tension field action that develops in the steel plates parallel to the directions of the principal tensile stresses (Astaneh-Asl, 2001).

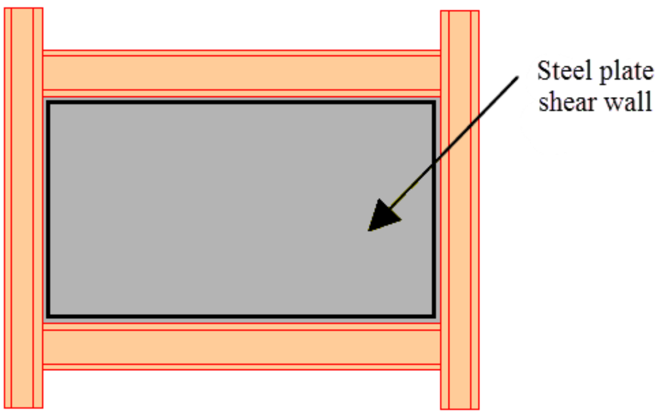


Figure 1.6 Unstiffened steel plate shear wall (Astaneh-Asl, 2001)

Depending on the thickness of the plate, unstiffened steel plate shear walls are classified to two type. If thickness thick called unstiffened thick plate, while called unstiffened thin if the thickness thin. In recent years, the researchers demonstrated that unstiffened thin SPSWs ownership characteristics that are fundamentally helpful in resisting seismically induced loads. These include superior ductility, robust resistance to degradation under cyclic loading, high initial stiffness, inherent redundancy and significant energy dissipation. As a result, in most applications of steel shear walls in recent years in the US and Canada, un-stiffened thin steel plates have been used efficiently and economically (Vatansever, Yorgun, and Özel, 2012)

1.3 Objectives and scope

The main objective of this thesis was to study the behavior of steel plate shear walls (SPSWs) under lateral loads. This research used the strip model, as identified by Thorbum et al. (1983), to obtain a more exact prediction of the inelastic behavior of SPSWs using a conventional structural engineering software program of SAP2000 version 18.2.0. To examine this behavior, a series of multi-story and various aspect ratios having different thicknesses of steel plate shear wall were considered. The study mainly focused on the pushover analysis method to obtained more realistic estimations of nonlinear response of the structures with SPSWs.

There are four specific objectives of the present research:

- Compare the behavior of structural frame without a plate and after using steel plate shear wall.
- Determine the effective thickness of steel plate on a structure by comparing the different sizes of small to large steel plates.
- Compare and determine different aspect ratios of steel plate.
- To discover the steel plate effectivity on the multi-story building as a resisting for lateral loads.

1.4 Outline of the thesis

Chapter 1-Introduction: This section presents a brief overview of the studies in the literature and objectives of this study are also introduced.

Chapter 2-Litreature review: A review of previous research on the historical background of the steel plate shear wall is given. The review includes a summary of relevant experimental and analytical work on SPSWs.

Chapter 3-Methodology: The methodology part covers the description of the different buildings and loading conditions of the buildings. Also, in this section, the details of the nonlinear analyses, aspect ratios and thicknesses of SPSWs are expressed.

Chapter 4-Results and discussion: In this section, the various models are compared and discussed to represent the results obtained from nonlinear static analysis. The structural performance and the response parameters of each frame system conducted in this study in terms of capacity, displacement, ultimate strength, initial stiffness, etc. are explained in details.

Chapter 5-Conclusions: The conclusions are given in the light of findings from the overall results of the study.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

After 1970s several analytical research programs and experimental have been worked on the behavior of Steel Plate Shear Walls (SPSWs) also its design procedure. Steel Plate Shear Walls come to light In Japan for the first time as a lateral resisting system in the building in the 1960s. During that time, designed SPSWs to resist lateral load by precluding shear buckling infill plate design seismic loads. Therefore, both of thin and thick stiffened plates were accepted and used. Over time, analytical and experimental studies have demonstrated that post-buckling strength is critical and thus, the use of thin-unstiffened infill plates in SPSWs has been improved by designers and researchers. Thin-infill plate in unstiffened SPSWs system wastes the main energy element, which is allowed to buckle out of panels incline tension field action advances the shear resistance (Ashish and Harshalata, 2014).

2.2 Development of steel plate shear walls

Takahashi et al. (1973) conducted the behavior of steel plate shear walls by using the inclusive experimental program, various of stiffened shear walls panel subjected to inelastic cyclic loading were worked on it to defined their resistance against lateral load as a resistance system for buildings. For this test, 12 panels were used all of the panels were 0.90 m in height and 2.10 m in width, and plate thicknesses (2.3 mm,3.2 mm, and 4.5 mm) and stiffener arrangement were tested, with one acting as a control specimen as it had no stiffener installed. High-strength bolts were connected each panel with a very strong rectangular pin-joined frame. From 4 to 6 complete cycles of shear loading were applied with rising displacements in all cycles. In addition, two full-scale tests of stiffened walls having two stories were conducted.

The researchers illustrated that the stiffened panel wasted significantly more main energy compare to an unstiffened panel, although stiffened panel and unstiffened panel mostly behaved in a stable and ductile manner. Takahashi et al. (1973) reported that stiffened SPSWs be designed so that the shear panel does not buckle elastically. Furthermore, it was recommended that must inelastic buckling happen, it is limited to local buckling between the stiffening elements, that is, overall buckling of the stiffened panel must not happen. Although more recent research has, for economic reasons, focused on SPSWs with un-stiffened panels that buckle elastically, the somewhat superior hysteretic behavior of stiffened panels, as illustrated by Takahashi et al. (1973).

A few years after Takahashi et al. (1973), Mimura and Akiyama (1977) sympathize the characteristic of the hysteresis behavior of un-stiffened SPSWs having infill plates that buckle prior to amounting to the shear yield load. They then expanded process for predicting this behavior. In order to demand sign to support theoretical model, Mimura and Akiyama (1977) studied a various of simply supported plate girder- type samples to single cyclic point load at mid-span. At load beyond the elastic buckling load, they expected that the steel panels would develop a tension field to resist the applied loads. The angle of inclination of tension field was identified with the derivation by Wagner (1931). All thing about panel aspect ratio, the number of panel per girder, the web thickness, the flange and stiffener member sizes were varied. Generally, just under two cycles of loading, although they illustrated moderate accord with the prognostic model.

Anglidis and Mansell (1982) studied of multi-stories building by conducting orthotopically stiffened SPSWs. A twenty stories office building was chosen, the building 30 m square in the plane with 10 m square central service core. The core consists of steel plate and vertical stiffeners in each orthogonal direction. Steel plate connected with vertical staffers in one direction and joined to two other cantilevers on the other direction. The thickness of steel plate 16 mm in the upper and 8mm in the lower level, also vertical stiffeners thickness 10 mm in the bottom and 4mm in the top. A two-dimensional linear plane-stress finite element analysis was carried out to acquire stresses and maximum deflections of the core under dead, live and wind load combinations. For plate steel design purpose was used compressive stresses in the web of the core at the bottom was a large number. After predicting the price of steel core was twice compare to the concrete core for this building. This included the price for steel panels, fabrication, erection and sprayed fire-proofing compared to the price for concrete, reinforcement steel, reusable framework and embedded predrilled plates to connect the floor beams.

Thorburn et al. (1983) recognized an analytical model to study the shear resistance of a thin unstiffened SPSWs depended on pure diagonal tension theory proposed by Wagner (1931). Steel plate shear walls was represented as a serious of pin-ended strips oriented in the analogous tendency as the main tensile stress in the infill plate is known as strip mode demonstrate in Figure 2.1.

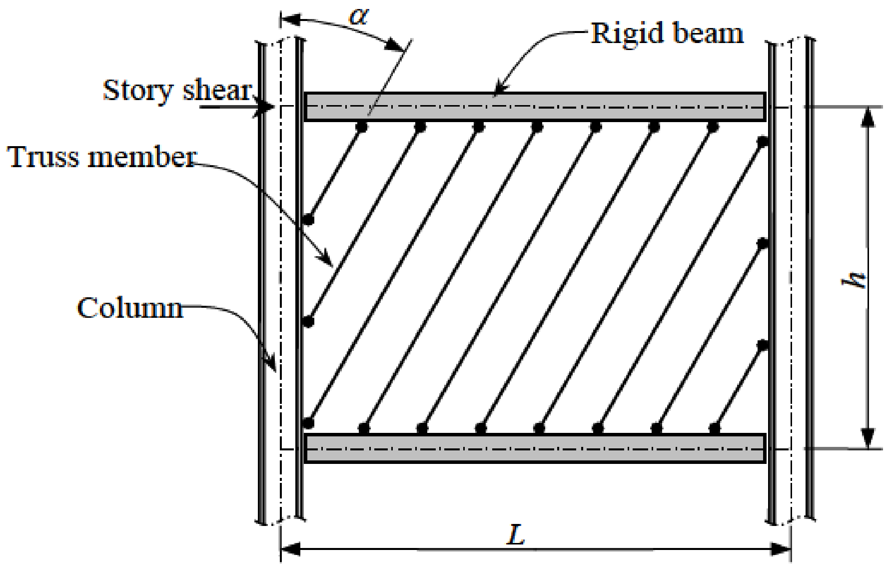


Figure 2.1 Strip model of a typical story (Thorburn et al., 1983)

Area of each one depending on the rectangular area equal thickness of plate time width of strips. This model is known as "strip-model" which has later been acknowledged by Canadian Steel Design Standards (CSA, 2009) later on. The angle of inclination of the tension field α was progressed as:

$$\alpha = \tan^{-1}\left[\frac{1+\dfrac{L\,t_w}{2A_C}}{1+\dfrac{h\,t_w}{A_b}}\right]^{\frac{1}{4}} \tag{2.1}$$

Where α is angle of inclination; t_w is steel plate thickness; L is length of panel; h is height of panel; A_C is area of column; A_b is area of beam. In addition to the strip-model, Thorburn et al. (1983) suggested a Pratt truss-model for analysis of unstiffened thin SPSW that is known as “equivalent brace model”. A diagonal brace was used to represent the infill plate and its equivalent stiffness was identified from the original strip-model. It can reduce a considerable amount of computational intricacy than original strip-model. The area of the brace (A) is calculated as follows:

$$A = \frac{t_{\omega} L \sin^2 2\alpha}{2 \sin \psi_b \sin 2\psi_b} \tag{2.2}$$

Where, ψ_b is the inclination of the brace; A is area of brace and all parameter identified above.

Timler and Kulak (1983) developed Thorburn et al. (1983) theory by reviewing a new element as one-story and one-bay SPSWs panel. Due to this purpose identified column as a horizontal element while for vertical worked in beam element. The research demonstrated that the value of the angle of inclination α affects by the flexural stiffness of columns. Thus, developed angle of inclination in Thorburn et al. (1983) equation was adapted as follows:

$$\tan^4{}_{\alpha} = \frac{1 + \frac{t_{\omega} L}{2A_C}}{1 + t_{\omega} h \left[\frac{1}{A_b} + \frac{h^3}{360 I_C L}\right]} \tag{2.3}$$

Where I_C is moment of inertia of column, and all parameter identified above. Also, found flexural stiffness of beam effect if beam has a steel plate just on one side and are consequently allow to bend, and at the top of shear wall have the beam. Thus, above equation re-derived to find angle of inclination as follows:

$$\tan^4{}_{\alpha} = \frac{1 + (\frac{1}{2A_C} + \frac{L^3}{120\, I_b\, h})}{1 + t_{\omega} h \left[\frac{1}{2A_b} + \frac{h^3}{320\, I_C L}\right]} \tag{2.4}$$

Where I_b is moment of inertia of beam. Timler and Kulak (1983) used strip model technique for analysis specimen also pin-ended connection used. In general, after analyzing discovered a good liaison between assumed value and real price of the steel pate axial strain, stresses, and the load vs. deflection response.

After a few years of test by Timler and Kulak (1983), Tromposch and Kulak (1987) used similar a large scale of steel plate shear walls. The chief difference between two tests was change in same detail like bay length and high, thinner plate 3.25 mm, beams stiffer and the connection between beams and column was weld and rather than use the bolted. Also, columns on the specimen under pretend gravity load were pre-stressed. The main aim of the test was to confirm the strip model recognized by Thorburn et al. (1983) also to identified the hysteretic property of the structure. Tromposch and Kulak also demonstrated that the strip model (Thorburn et al., 1983) gave conservative predicts of the ultimate strength and initial stiffness SPSWs. Further parametric researched illustrated use fix connection between beam to column in frame can save three-time energy compare to pin connection in a frame. In addition, Tromposch and Kulak (1987) showed that the strangeness of the fish plate with respect to the center of the vertical and horizontal members had not effected of the SPSWs specimen.

Chen (1991) studied an experimental program that consisted of ten-quarter scale, three-story, one- bay SPSWs that were applied by cyclic loading. The specimen used in the research can be looked in Figure. the thickness of plate ranged from 0.76 mm to 2.66 mm, also either simple or fixed connection was used for the beam to column connection. Aspect ratio and opening on plate wall was also studied. The loading procedure consisted of 24 fully reversed cycles of rising displacement up to a maximum specimen drift of 2%, or nearly 50 mm. Each frame under loading procedure was applied twice to define the ultimate capacity if possible under monotonic test. A structure without SPSW also tested as a lower bound comparison. The steel plate shear wall performed in a very ductile manner and exposed a high initial stiffness with high energy dissipation. The analysis discovered two failure modes, column buckling and plate yielding. Use very thin steel plates, the columns were comparatively stiff and provided fix boundary conditions, forcing the plate to yield before yielding of the columns. The initial stiffness was higher for thicker plate, but the relatively flexible boundary members were likely to yield before any plate yielding happened. The three story model is given in Figure 2.2.

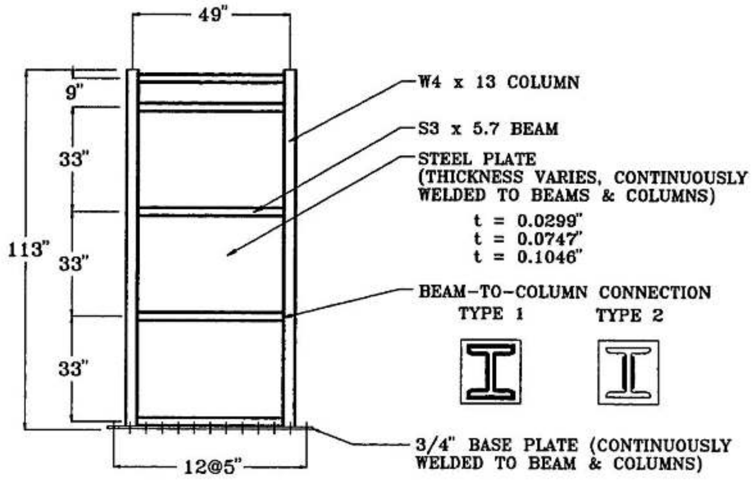


Figure 2.2 Three-story model used by Chen (1991)

It must be essentially select thickness of infill plate as it would influence the type of failure for a chosen size of column (Chen, 1991). The work of Chen (1991) also improved that simple or rigid connection had no effect on overall strength of SPSWs. The result observed that that the ultimate strength of the SPSW increase with increasing the aspect ratio, although the extent of the capacity increase was not thoroughly investigated.

Aoyama and Yamamoto (1984) used seven specimens to study methods for the concrete frame to identified supporting buildings by using stiffened thin-plate steel. Each test was just one bay panel, one-third scale one story, with using infill steel plate shear wall by joining to around of frame by using an amalgamation of headed studs and drilled resin anchors. Axial column load combined with three or four cycles of lateral to high story drift about 2% were loaded to the structure. The researchers identified SPSWs aid to permitting them to continue to backing axial loads, while even shear crack form in concrete columns.

Sabouri-Ghorai and Roberts (1991) suggested a method nonlinear dynamic analysis of thin SPSWs whereby the system was perfected as a vertical cantilever plate girder. The derive equation of motion for a continuous cantilever beam was discretized to a multi-story shear wall in which the associated story masses and the dynamic forces were concentrated on each floor. Initially, the equation predicted only for shear deformation and later more general information of both shear and bending. By using an approximate elastic-plastic hysteresis model for each panel of steel plate shear wall material nonlinear was combined into one body. The result of hysteresis model income into computation shear buckling and yielding of the web plate as well as HBE and VBE.

The behavior of hysteresis of the plate was calculated from a various of quasi-static tests small single panel stiffened plate and unstiffened, pin ended boundary elements (Roberts and Sabouri-Ghomi, 1991). An elastic-fully plastic material model was predicted for the lamming frame only, and plastic hinge assuming in the top of column and bottom. The hysteresis curve of the web plate and boundary columns presented the hysteresis curve of the entire shear wall. Sabouri-Ghonn and Roberts (1991) estimated their analytical model by analyzing specimen five-story and one bay SPSW and three periodic loading, elastic, resonance response and elastic-plastic were three different loading exanimated. After analytical technique illustrated by researchers has not been validated with any analysis test result.

Elgaaly et al. (1993) studied on SPSWs models depending on the multi-strip model suggested by Timler and Kulak (1983), to reproduce results experimentally attained by Caccese et al. (1993). Nonlinear material geometry and properties used in finite element model, 6 beam elements foe each member and plate represented by 6x6 mesh, thickness of plate was (1.9 mm and 2.7 mm) for experimentally work, also assumed beam-to-column connection moment resisting. Combined strips in each direction depended on the strip model was used in this model. Figure 2.3 shows the cyclic strip model.

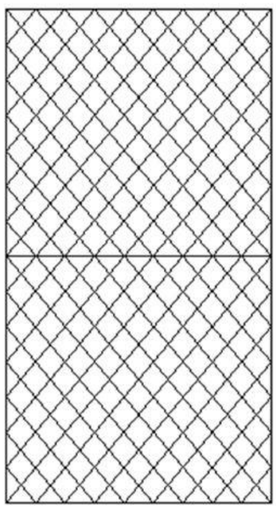


Figure 2.3 Cyclic strip model (Elgaaly et al., 1993)

The lateral load was monotonically loaded pending damage of stability developed due to column plastic hinging and flange local buckling. After testing illustrated that the SPSW with thicker infill plate has not been importantly sturdier because column yielding was the leading factor for each case. For experimental purpose used steel plate shear wall with (1.9 mm) and moment resistance for connection. Number if strips were equal to 12 for presenting plate in each story was modeled by the multi-strip method. The angle of inclination was discovered 42.8°, which agreed with principal strain was developed before between 40 and 50 ° with the columns. The results obtained that both of number of strips and angle pf inclination were agreement with the initial stiffness and maximum base shear and displacement at the maximum base shear. It was recommended that during the experimental, the hysteretic model complicated the use of an empirically hysteretic, hysteretic, strain stress association for the strips, and pretty good approval with investigational results was notified (Elgaaly et al., 1993).

Xue and Lu (1994) achieved an analytical study on a specimen 3 bay on multi-stories building (12 stories), thickness of plate (2.2 mm to 2.8 mm), with three bay wide and SPSWs. The purpose of this test to focus on beam-to-column and infill plate connection. Four states were well-thought-out:

a) Moment resisting beam-to-column connections and steel plate shear walls entirely attached to boundary elements.

b) Moment-resisting beam-to-column connections and the steel plate shear walls connected to only the beams.

c) Simple beam-to-column connections and entirely attached steel plate shear walls.

d) Simple beam-to-column connections with steel plate shear walls attached just to the beam.

After applying pushover with force applied at each story, result defined that the performance of force and deformation of the system significantly effect by connect between beam to column connection and connect column with the infilling steel plate shear walls has increase in the ultimate capacity of the system. Xue and Lu (1994) determined that connecting the steel plate shear walls just to the VBE and using simple connections in the interior bay was the best arrangement because this radically reduced the shear forces in the interior columns and assisted avert premature column failure.

Sugii and Yumada (1996) studied and analyzed of 23 specimens of SPSWs with and without concrete cover. The samples consisted of different aspect ratio, the thickness of plate and thickness of the concrete wall. For experimental purpose worked with a structure were one over ten scale models of a prototype and the high of specimen was equal to 3 m, the aspect ratios if 1:1 and 1:2, thickness of infill plate between 0.4 mm and 1.2 mm, and tested of fourteen specimens of SPSW without concrete covering. Each specimen has two adjacent congruous panels which were separated by a deep beam. The horizontal load was applied to the beam at mid-span in a manner similar to the set-up used by Timier and Kulak (1983). The two panels were thus loaded similarly. In all cases, the surrounding moment resisting composite frames consisted of wide flange steel sections covered by reinforced concrete.

They repeated that maximum resistance steel plate shear walls after tested monotonically was found story sway angle of 0.01 to 0.03 radians. The main factor to increase ultimate strength of specimen was thickness of steel plate shear wall under cyclic loading, and affecting the area under hysteresis curves. Steel plate shear wall panel from a tension brace changing to an equivalent effective width of 0.67 of the story height was suggested for an analytical method. Strain-stress relationship was predicted for the brace an elastic-perfectly plastic relationship was assumed for boundary elements. The calculated results, however, clearly over-predicted the elastic stiffness part of the load-deflection curves, but sensible captured the overall strength achieved in the tests.

Elgaaly and Liu (1997) discovered a modern modeling thin-plate desponding of a various of tension diagonal stress-strain relationship. The initial yielding and corresponding elongation of the strip-gusset elements together with post-initial yielding modulus were derived strip joined with vertical and horizontal frames by using gusset plate. In the technique plate represented by strips having angle of inclination 45° and an elastic and perfectly plastic semi-empirically. In general, by buckling shear stress of the equivalent square plate to the shear yield stress of the plate material may be found parameter detail of gusset plate. Depending on new technique were found could guess the pushover envelope and hysteresis curve.

To find the ductility, elastic stiffness and energy absorption capacity Driver et al. (1997; 1998) studied a sample single bay, four-story SPSW. To connect beam-to-column connection used moment resistance, also for connect steel plate was used fishplates welded connection. Depending of ATC-24 (Applied Technology Council 1992) cyclic lateral loads of equal magnitude for each floor level and the gravity loads for tops of the columns were applied. Result demonstrated that behavior of SPSW system is premium lateral load-resisting system for seismic loading.

Driver et al. (1997; 1998) also widened a finite element models to proposal the structural performance of the SPSW specimen. By using this analysis showed the premium convention with the investigational information, but was not able to get the full shear wall capacity. A full response analysis was also achieved and provided a brilliant assumption of maximum base shear but overrated the initial stiffness by around 15%. Figure 2.4 show the steel plate shear wall tested by Driver et al. (1997).

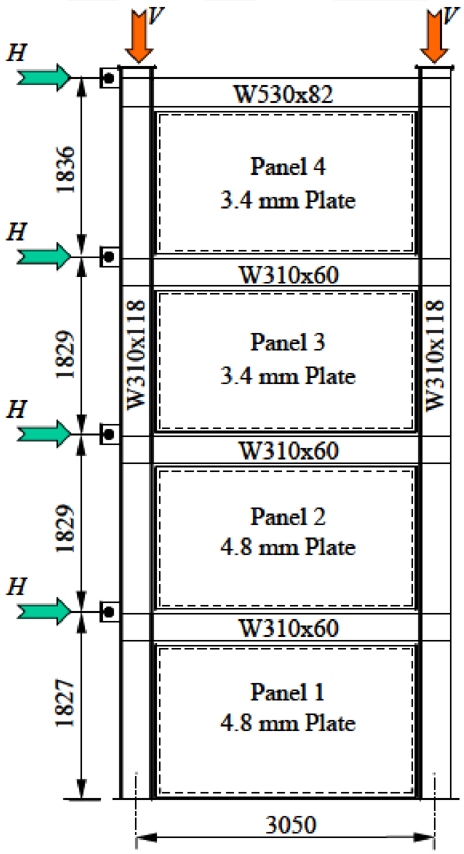


Figure 2.4 Four story steel plate shear wall tested by Driver et al. (1997)

Lubell et al. (2000) conducted research on 2-single story unstiffened plate steel shear walls as well as 4-story panel specimen. Quasi-static cyclic analysis was conducted on those specimens where ATC-24 (Applied Technology Council 1992) guidelines were followed for the loading history. The test was having aspect ratio of 1:1 to exhibit primarily flexural, rather than shear behavior specimen demonstrated in Figure 2.5. Loading were combination between fully reversed cyclic lateral load with gravity load. The objective of their study was:

- Stiffness and capacity of SPSW sharply raised when top beam can offer better anchorage for the tension field developed in the infill plate.
- Insufficient column flexural stiffness can produce significant "pull-in" in the column and undesirable yielding sequence which can create global instability of the system.
- Due to the high axial forces and moments in the columns the yielding sequence can be altered.

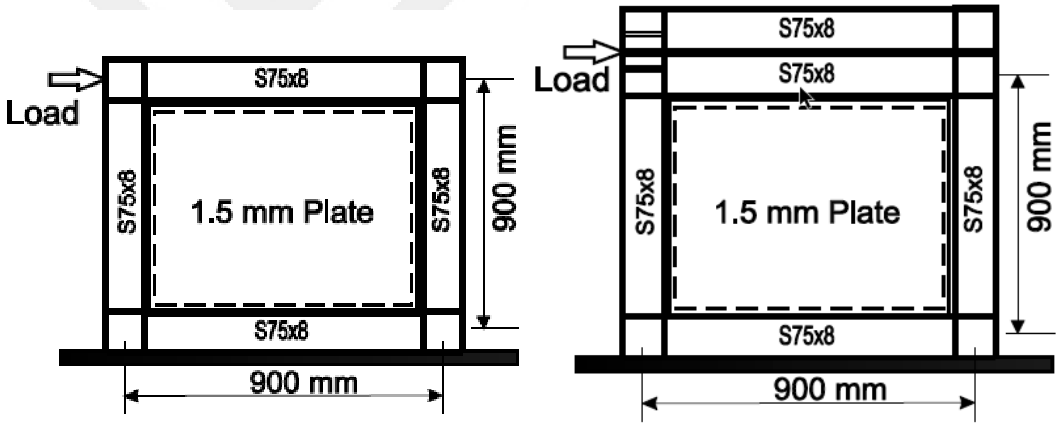


Figure 2.5 Two single-story test specimens of Lubell et al. (2000)

Shake table tested on steel plate shear wall for the first time was used by Rezai (1999). The same multi-story sample used it was tested before by Lubell et al. (2000) and masses were concentrated on each floor level. The elastic behavior concentrated on it because shake table was incapable of cause the shear wall to reach its ultimate capacity. In the University of Alberta, two research projects on SPSWs are presently underway in experimental and analytical work. One of the fulfillment of steel plate shear wall verified before by Driver et al. (1997) to improve the performance of shear wall that has previously been subjected to a seismic. the other fulfillment of the potential of shear wall walls as a means of rehabilitating deficient concrete frames.

For investigated seismic behavior of the system Kulak et al. (1999) studied of an eight-story building steel plate shear wall. Eight meter was the width of the panel and 4.5m high on the basement and 3.6 m of the all other stories. Both of base shear and the vertical distribution of lateral forces on steel plate shear were calculated based on National Building Code of Canada (NBCC, 1995). The precursory design was of steel plate shear wall was carried out by strut illustrated by Thorbum et al. (1983). This gave plate thickness of plate 3.33mm and 0.66mm from bottom to top, respectively to make more reality used the thickness of plate 4.8mm. Specimen was tested by mean of a linear static analysis program and for modeling they prefer to use stirp model only. The strength design contented both the wind and seismic drift limits set by NBCC (1995).

A response spectrum analysis was then done to predict the effect of higher modes of the vertical distribution of lateral forces. A pushover analysis of the system, experimented using the commercial software DRAIN-2D3C. Result illustrated that structure stronger than two times for base shear compare it has been described by NBCC. This rise of strength was largely because using steel plate shear wall of 4.8 mm, and it was significantly more than desired. In addition, the researchers also studied a nonlinear dynamic time history analysis of the tension-compression model by applying twenty scaled earthquake records to the structure. The result showed after applied 20 earthquakes maximum inter-story drift ratio did not pass 0.009, which is very small compared to a limit of 0.02 specified by NBCC. The maximum studied story shear varied from 2.07 to 2.97 times the prescribed NBCC (1995) values. In the most serious earthquake, a maximum ductility demand of 1.9δy was found, which was very small only about one-fifth of the ductility obtained from the pushover analysis. Although yielding happened in story one and three on the columns, the yielding did not proceed to create a soft story since the lateral deformation of the story (Kulak et al., 1999).

Schumacher et al. (1999) studied several of steel plate to surrounding frame connection details to test and compare their response under conditions similar to those existing in a shear wall subjected to a cyclic load. A close-up view of 4 corner details studied by Schumacher et al. (1999) is seen in Figure 2.6 and are as follows:

- Steel plate welded straight to the surrounding frame as presented in detail A.

- In detail B, fish plates welded to each of the surrounding frame and the steel plate lapped over the fish plates and welded.

- Fish plates with a corner cut-out welded to the boundary elements and the steel was then lapped over the fish plates and welded as seen in detail C.

- Fish plate welded to only one boundary element and then the infill plate welded directly to the other boundary element and lapped and welded onto the fish plate as exposed in detail D.

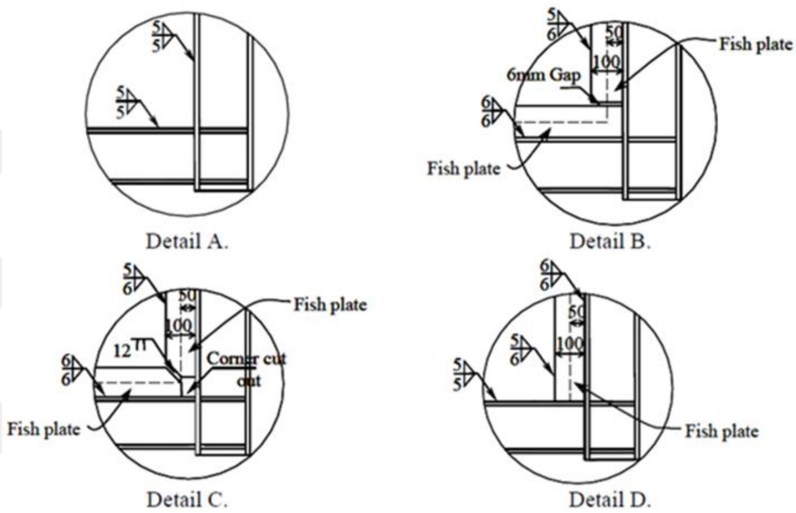


Figure 2.6 Corner details tested by Schumacher et al. (1999)

After applied cyclic loading to an unstiffened SPSW may be display two effects relevant to steel plate to boundary element corner details. In the first repeated opening and closing of the corners as the lateral loads on the specimen reverse. On the other hand, the second is the development of the tension field that forms diagonally in a panel after the thin plate buckles. The loading blueprint approved in the research program applied above effects to a secluded corner detail (see Figure 2.7). The results obtained displacement responses of each specimens illustrated comparable behavior for each detail, and they all displayed comparable energy dissipation capability (Schumacher et al., 1999).

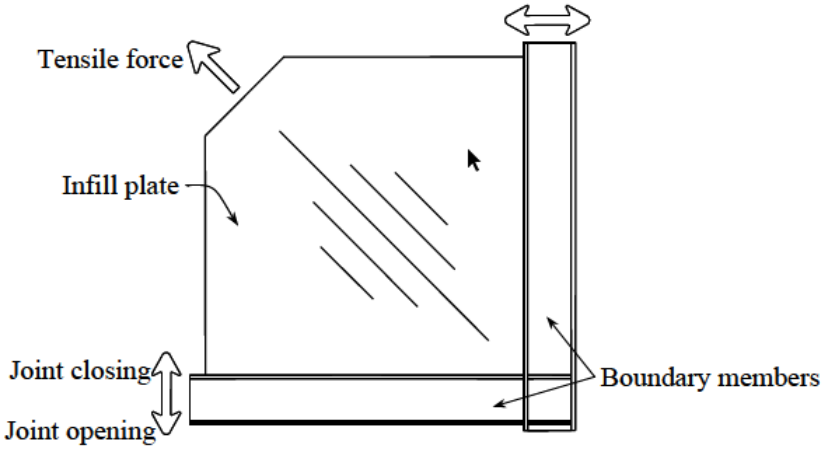


Figure 2.7 Corner specimen loading (Schumacher et al., 1999)

Astaneh-Asl (2001) conducted a research on two different types of steel plate shear wall used unstiffened with open and without it also, stiffened with and without opening for the specimen. In addition, research worked on the same different building in same places after using steel plate against the lateral load. After experimental recommended that if there is an opening in the SPSWs require using stiffened panel.

Bruneau and Bhagwagar (2002) studied two alternative perspective materials, namely low yield point steel (LYP) and shear fill (LYP) they were used in a special situation when steel plate waste energy recently developed LYP to enhance the energy was wasted in steel plate shear walls. Yamaguchi et al. (1998) demonstrated currently two type of LYP available LYP100 and LYP235, with the yield strength of 100 and 235 (MPa), respectively. The other one shear fill is a brand name fabric fabricated by Chemfab. In this anisotropic material, the fibers in one direction are called fill fibers, and the orthogonal fibers threaded between the fill fibers and are called wrap fibers. For this study used a Hospital building are two towers 20 stories, A572 Grade 50 steel and using SAP2000 program analysis. Conducted nonlinear inelastic dynamic analyses to examined how building performance influenced while used thin steel plate shear walls. Research recommended that use steel plate or any ductile material for infill panels may trim down story drifts also illustrated use LYP have bit effect better than standard constructional grade steel under seismic loading, but at the cost of some extra material. Bruneau and Bhagwagar (2002) recommended that using shear fill as infill indicated that, because of the low strength and stiffness of this material, the performance of the structure was not influenced entirely.

Behbahanifard et al. (2003) conducted a research and the same investigation of SPSWs, a large-scale test on a three-story SPSWs specimen was conducted under lateral quasi-static cyclic loading in the presence of gravity loads. Three top story SPSW, which had been tested before by Driver et al. (1997). This test was desponding of guidelines for simulating seismic loading (Applied Technology Council, 1992), for Standard fabrication techniques were used in the fabrication of the specimen and the loading history. In addition to this study, a nonlinear finite element model was developed in order to accurately simulate the monotonic and cyclic behaviors of thin unstiffened SPSWs. Behbahanifard et al. (2003) improved that ultimate capacity of the bottom story of the specimen was seven times of its yield displacement.

Berman and Bruneau (2003) calculated on equation for predicting maximum base shear of the one story and multi-story SPSWs with both of moment resistance and simple connection. Depending on two types of failure mechanisms that provide a rough range of ultimate strengths, soft story failure and uniform yielding of the steel plates, the equation was developed in a multi-story building. Using the assumed collapse mechanism shown in Figure 2.8, the story shear strength for a single-story steel plate shear wall with simple beam-to-column connections was found to be V, which is identical to the probable story shear strength given by CSA S16-01:

$$V = 0.5\, F_y\, t_w\, L\, sin\, 2\, \alpha \tag{2.5}$$

To provide a lower bound guess of capacity, simple beam-to-column was used for one-story steel plate shear walls to assume the capacity of a serious multi- and one-story SPSWs specimens, from the literature, having whichever or semi-rigid or pinned connections. This equation has illustrated an average between underestimate and overestimate about (6% and 9%), respectively. The equation developed for the soft story mechanism was demonstrated to overvalue the capacity of multi-story test specimens with rigid connections by around 17%. this model just gives the maximum base shear. The predicted equation does not characterize the ductility, the initial stiffness, the real failure mechanism nor do they give a means of defining the frame forces for use in design (Berman and Bruneau, 2003).

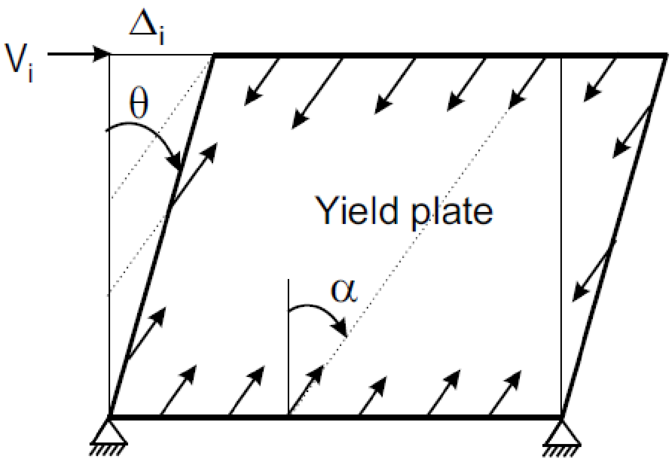


Figure 2.8 Single story collapse mechanism (Berman and Bruneau, 2003)

Kharrazi et al. (2004) discovered the design of steel plate shear walls systems in terms of the detach shear and bending deformations happening in the multi-story building. Kharrazi, Ventura, Prion, and Sabouri-Ghomi (2004) suggested modeling for the analytical of bending deformation and base shear and resulting forces in infill panel and it was a modified plate frame interaction. The purpose of this study to identified interaction between vertical boundary element and horizontal boundary element with SPSW and describe the respective contributions to deformations and strength. In addition, the behavior of bending was illustrated both of displacement and moment derived for a one-story panel at the crucial point at which place bucking happen, predicting a linear strain allocation across the wall cross-section. During experimental assuming, after occur buckling, the neutral axis will move to a column in tension, on the other hand, compressive stresses will be released on the web, alike to the neutral axis transmigration in a reinforced concrete beam following section cracking on the tension side. Equations for shear performance and bending performance are joint using interaction equations to complete the suggested method.

Mörel (2004) studied on unstiffened thin SPSWs. For this research purpose, four-story and one bay used, each panel was 3000 mm and 2000 mm for the width of the panel and high of the panel, respectively as shown in Figure 2.9, also used strip model for analysis purpose and SAP2000 program.

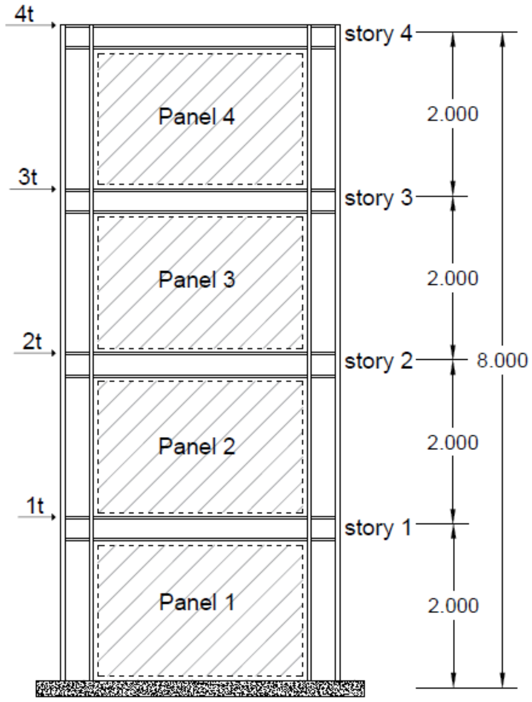


Figure 2.9 Four-story test specimens of Mörel (2004)

To identified behavior of steel frame, Mörel (2004) studied on two different part. In the first part, nonlinear static pushover analysis was performed by changing the angle of inclination α and a various of strip member, for this research about (36) model were used. In the second part, nonlinear static pushover analysis was performed by changing plate thickness, column and beam sections and beam to column totally about 112 model used.

Mörel (2004) recommended that using more member of the strips have more effective on nonlinear behavior. Also, illustrated that used different column and beam section were used with various steel plate shear wall effect on system behavior. Mörel (2004) improved that there is a finest thickness of plate for specific steel plate thickness, but when this value exceeds the only negligible rise in system strength is completed and any additional material used is lost.

Vian (2005) studied quasi-static cyclic tests on three structure, each structure has one-story, one-bay SPSW. The first structure had a moment resistance connection between beam-to-column with reduced beam section (RBS) and a solid steel plate of low yield strength(LYS). Other structures had the same boundary elements properties as the first one, and either various orderly spaced circular perforations in the steel plate or reinforced quarter-circle cut-outs in the upper corners of the steel plate. The last two structures were purposed to intake the requirement for interest systems to pass-through the infill plate.

Vian (2005) stated that the details of SPSW had a height of 2 m, and a width of 4 m, and a thickness of 2.6 mm with a minimum yield stress $\mathrm{Fy} = 165\ \mathrm{N/mm^2}$ and minimum tensile strength $\mathrm{Fu} = 305\ \mathrm{N/mm^2}$. The detail for perforated specimen suggested as demonstrated in Figure 2.10, used diameter of perforation was equal to 200 mm along the diagonal strip was equal to 4, used angle of inclination around at a 45° spaced at 0.3 m center-to-center along both the horizontal and vertical directions to provide a panel strip width, S diag, equal to 424 mm. For the cutout corner specimen, quarter-circle cut-outs of 0.5 m radius at the upper corners of the steel plate were used. To reinforce along the cut-out edges, arch sections 160 mm wide by 19 mm thick were selected. Vian (2005) observed substantial deficiencies in fabrication and inadequate overall quality of workmanship occurred.

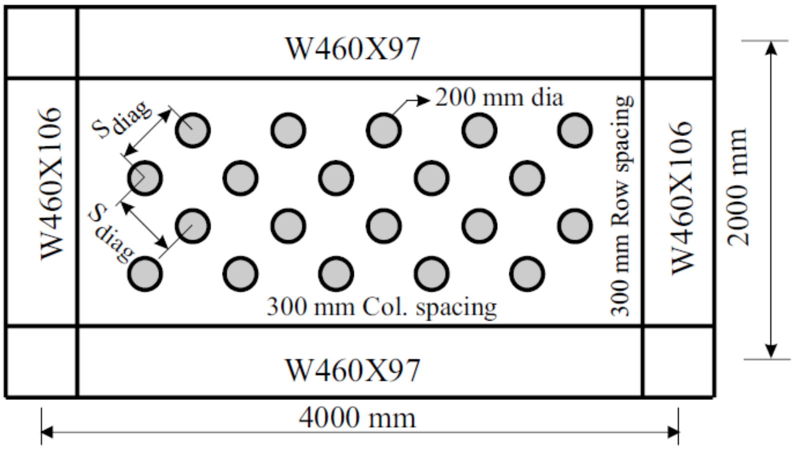


Figure 2.10 Perforated SPSW specimen (Vian, 2005)

Quasi-static cyclic loading was applied for each structure. During cyclic loading, all structures displayed behavior in which beam plastic hinging was located at the connection locations. This demonstrated the effectiveness of RBS connections in SPSW beams and was reported (Vian 2005) to control boundary frame yielding during a seismic occur. The perforated system was also reported for use in steel plate shear wall applications where the smallest available plate thickness is greater than required. The cut-out reinforced corner specimen also performed well during testing and appeared to be an effective solution for SPSWs to allow for the passage of utilities.

Finite element analysis conducted on each structure. The result identified good convention was between the test results and the hysteresis curves obtained from the analysis. This analysis was done with SPSW perforations in the steel plate with diameters of 0.1, 0.15, 0.2 m. the results obtained that the elongation assumed by the finite element model of a separate perforated strip and the full steel plate shear wall model, for monitored maximum strain, was significantly different and thus further research was recommended (Vian, 2005).

Purba (2006) conducted the behavior of SPSW unstiffened thin having opening in the steel plate shear wall by a various of finite element analyses. Similar to Vian (2005)'s work, each perforated strip having a size of 2 m by 0.40 m with 4 perforations along the strip length and perforation diameters, D, from 0.01 m to 0.3 m were first analyzed to develop an essential sympathetic of the performance of complete perforated steel plate shear wall. With the behavior of each perforated strip known, a series of 4 m by 2 m one-story perforated SPSWs having multiple perforations were modelled. For modelling used both of steel plate shear walls and boundary element depending on shell element. Nonlinear static pushover analyses subjected for structure perforated SPSWs. A lot of infill plate thickness and perforate diameter were considered in the analyses. Purba (2006) discovered that the result from each perforated strip analysis can strictly assume the performance of whole perforated steel plate shear wall provided the hole's diameter is less than 60% of the strip width (as presented in Eq.2.6).

$$\frac{\mathrm{D}}{\mathrm{S}_{\mathrm{diag}}} \leq 0.6 \tag{2.6}$$

Equation was improved that interaction exists between adjacent strips that could affect the stress distribution within each strip. Purba (2006) also developed the work proposed by Roberts and Sabouri-Ghomi (1992) to identified the strength of a perforated steel plate with multiple perforations. Depending on the analysis results, Purba (2006) found that shear strength of a steel plate of a steel plate shear wall having multiple regularly spaced circular perforations throughout the steel plate can be calculated by multiplying the shear strength of a solid infill plate by a factor (as seen in Eq. 2.7). Also, result it was observed that there was very small difference in the shear strength of the steel plate. Thus, the flat plate was deemed to be sufficient to reinforce the cut-out edges.

$$1 - 0.7 \frac{D}{S_{diag}} \tag{2.7}$$

Habashi and Alinia (2010) studied a various of SPSWs models for analysis. For this purpose, used single-story and single-bay, each panel parameter was 3000, 3000, and 3 mm for height, width, and thickness respectively of the plate as demonstrated in Figure 2.7, is design depending on AISC Design Guide (20) and AIS (341-05) rules and provisions. The main objective of this experimental was for find effectiveness of infill wall in resisting material loads and their performance on the frames. Result illustrated in the initial stage of loading and digest fundamental part of story shear steel plate shear walls word exactly very well. However once diagonal zones involve in the plate. They progressively miss their efficiency and frame member normally become more lively. In addition, research showed when drift angle between %0.5 to %1 plate become less active beyond the drift angle %1 any loading loaded basically carried by frame member.

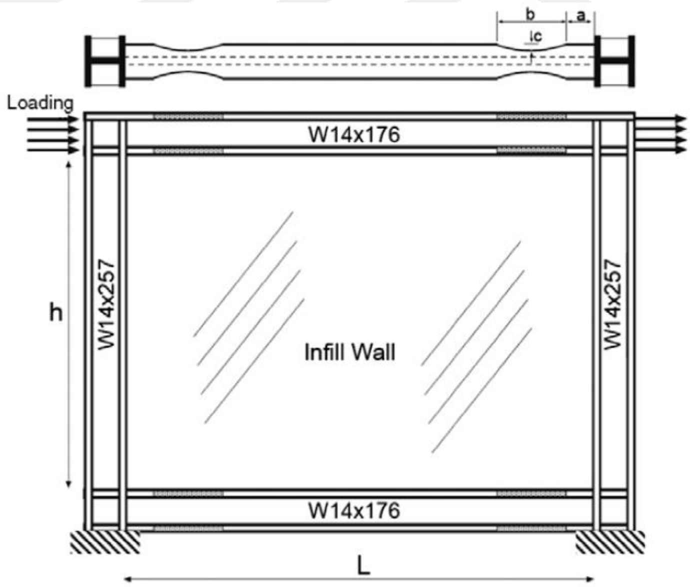


Figure 2.11 Specimen of Habashi and Alinia (2010)

Neilson et al. (2010) conducted a single-story SPSW have a high 1.9 m, and width 2.44 m using a thin gauge, cold rolled infill panel. The infill panel was included of two 20 Gauge A1008 CS (0.91 mm) cold rolled panels welded together in a lap splice. The experimental program studied methods of welding infill plate it was thin to boundary elements it was thick as well as the lap splice between two thin sheets. SPSWs applied ubder cyclic loading in conformity to the seismic testing guideline applied technology council-24 (ATC, 1992). The specimen was examined by mean of pushover analysis by using strip model proposed by Thorburn et al. (1983) and the analysis software SAP 2000.

Thin panels were selected for the test as thicker-than-required sections is not eligible when designing according to capacity design. The use of thicker than required plates in capacity design results in an over strength problem leading to exceedingly large boundary elements. ASTM A1008 CS 20-gauge plate was selected based on the low measured minimum yield stress $\mathrm{Fy} = 165\ \mathrm{N/mm^2}$ and minimum tensile strength $\mathrm{Fu} = 305\ \mathrm{N/mm^2}$. The low strength characteristic was eligible to ensure that the boundary elements designed according to capacity design principles remained small. As the plates choice were cold rolled, their width is usually too small to fit the bay using a single piece. Thus, two plates were used to give the best performance for this purpose it was discovered that a lap splice welded on both sides. A welding procedure was developed for the lap to minimize distortion and burn through of the thin plates. The lap was tack welded at regular intervals, and then stitch welds were placed between the tacks. This was done on both sides of the splice. A chill strip was used to reduce burn through and weights were placed on the infill panel to minimize welding distortions. A comparable procedure was used to connect the infill panel to the fish plates using a weld at the cut edge of the infill panel only (Neilson et al., 2010).

Based on applied technology council-24 (ATC, 1992), the SPSW was The SPSW was subject to a cyclic loading. Local buckling and lateral buckling of the beam near the loaded column occurred at about the same time. Several small imperfections were noted during the test including a 10 mm tear in the fish plate to infill panel connection which was a result of a poor quality weld. Also noted was the local buckling of the fish plate in the corners of the specimen resulting in a 15 mm tear. Despite some noted imperfections, there was no detectable loss of performance or integrity noted for the duration of the test. For modeling was used the strip model (Thorburn et al., 1983) in SAP2000 and examined by static nonlinear pushover analysis. For this purpose, was used 10 tension strips and the angle of inclination α used 40°. The model gave good agreement with the envelope of the hysteresis plot, underestimating the ultimate strength by 7.5% while the initial stiffness was predicted to within 8%. The moment frame, without the infill panel, was analyzed individually and it was seen that with the use of the infill panel, the model illustrated a rise in the initial elastic stiffness of 280% and a rise in the ultimate load capacity of 48% (Neilson et al., 2010).

Driver et al. (2014) studied of behavior of pushover analysis of perforated SPSWs. During this experimental a serious of single-story SPSWs, with eight different types of perforation were used depending of strip model. After analysis, all specimen illustrated by reducing shear strength of the solid plate can be calculated by the shear strength of the infill plate with circular perforation. Researchers recommended that using steel plate shear walls without hole more effective than perforated infill plate. However, using perforate infill plate effective than the frame structure only.

Narayanan and Rockey (1981) had conducted the openings in the web of plate girders. They had suggested a close method of analysis. Most precise methodology for calculating the ultimate shear capacity of plate girders with opening in webs were suggested by Narayanan (1983). The analytical models previously proposed by Porter, Rockey and Evans (1975), for predicting the ultimate capacity of plate girders were widened by Narayanan to the web plates with openings. Mo and Perng (2000) studied a specimen on frame shear walls using corrugated infill steel plate shear walls and illustrated an improved seismic. Berman and Bruneau (2005) studied an experimental study on one-story steel plate shear walls and light-gauge steel with infill plate and corrugated plate under quasi-static loading. Hong-Gun Park et al. (2010), depending on incline tension strips method with infill steel shear walls were reviewed the cyclic nonlinear analysis of thin SPSW.

Berman et al. (2011) discovered the seismic performance of infill plate designed depending on the performance. A various of SPSWs were studied and their behavior was calculated using nonlinear response history analysis for different ground motions. It is found that designs meeting current code specifications satisfy maximum inter-story drift requirements. Abhishek Verma et al. (2012) studied on static pushover analysis of unstiffened SPSWs. Also, illustrated that using larger aspect ratio exhibits the larger capacity and deformation compare to smaller aspect ratio. Abhishek Verma et al. (2012) improved with increasing thickness of plate shear walls ultimate load capacity increased. Gholizadeh and Yadollahi (2012) demonstrated that behavior of structure with using corrugated plate can be superior to that of a flat plate due to its higher loading capacity, ductility, and energy absorption capability.

Jian Guo Nie et al. (2013) studied a specimen stiffened SPSWs with lateral resistance capacity. Result illustrated that stiffened may be used for opening because steel plate shear walls have stiffness and stability. Emami and Mofid (2014) studied on trapezoidal-corrugated steel plate shear walls, an experimental investigated performance of specimen under monotonic and cyclic loads by using detailed numerical simulations.

The behavior of corrugated and perforated SPSWs were showed by Farzampour et al. (2015). Some of the advantages and disadvantages of perforated infill plates in terms of structural performance of SPSW systems were illustrated. Farzampour et al. (2015) focused on the cyclic performance and energy dissipation capability of corrugated infill plate in order to profit a best understanding of seismic performance of such lateral force-resisting systems.

2.3 Development of strip model for SPSWs

In the recent year, the strip model for both of a number of strips and angle of inclination has been developed. Past researches conducted by Driver (1997, 1998) and Elgaaly (1993) discovered that the angle of inclination and the number of stirp members were not significantly effect by the linear and nonlinear unstiffened thin SPSW. Driver (1998) shown that for 4-story study specimen, the load versus deformation performance is relatively insensitive to the angle of inclination choice approximately near 45°. Based on Driver’s work selected angle of inclination 42° and 50° gave fundamentally the same response. Also for infill plate presented by 10 strips and the load versus deflection envelope performance of the structure, and in particular its ultimate strength.

In Elgaaly’s study, the found angle of inclination from Kulak’s formulation for the test specimen was 43° and the selected angles were 38°, 43° and 45°. Elgaaly (1993) showed that the initial stiffness slim effect by small variation of angle of inclination and decrease the ultimate strength happen at the angle of inclination drop. In the worst case, the percent difference between the 38° case and 45° case was only 5.1%. The effect of the number of strips per panels was observed by changing their number from

4 to 12 in Elgaaly's analytical tests. Elgaaly (1993) recommended that for the shear wall systems use 1 to 3 strip with slenderness ratio more than 700, and for a slenderness ratio less than 700, 4-8 strip member must be used. The number of strip member in other various analytical studies are as follows. In experimental studied conducted by Berman (2003) to examine the plastic analysis of steel plate shear walls, for modeling used 11 strips for infill panels. In experimental analytical studies conducted by Bruneau (2002) to examine how building performance is affected when thin steel plate shear walls are used to seismically retrofit steel frames, used 11 or 15 strip members for each panel for modeling infill panels according to varying story heights.

In analytical studies conducted by Lubell (2000) to discover the performance of unstiffened SPSWs subjected by cyclic loading, 15 strip members were assumed for modeling infill panels. It was recommended that the numerical models were plausibly precise at assuming the post-yield strength and stiffness of the structures. A good prediction of the elastic stiffness was completed by using an angle of inclination much more severe than that discovered by Kulak's formulation. Parametric studies signalize that elastic stiffness and global yield strength is sparingly affected by the varying angle of inclination. In Canadian structural steel design code (CAN/CSA-S16.1, 2001), it was recommended for modeling steel infill panels at least need 10 strips per panel.

CHAPTER 3

METHODOLOGY

3.1 Analytical model of steel buildings

Through this work, two different cases are studied. The first case is the effect of plate thickness of steel plate shear walls (SPSWs) with a fixed aspect ratio on various multi-story steel buildings (2, 4, 6, and 8 story steel buildings) while in the second case, the influence of plate thickness of SPSWs with different aspect ratios on the 4-story steel building. Hence, these cases provide more insight into the effectiveness of SPSW applications in the steel structures.

The height of each story is 3.5 m, each panel space length is 7.5 m for frames in both directions, the length of infill plate changing according to model demonstrate later, all beams and columns and plate are grade S235 and using HE300A section for columns and IPE300 section for beams with a specified minimum yield stress $\mathrm{Fy} = 235\ \mathrm{N/mm^2}$ and minimum tensile strength $\mathrm{Fu} = 360\ \mathrm{N/mm^2}$, thickness of plate is 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm for both cases.

The uniform dead load and live load on each floor are taken individually as $5.5\ \mathrm{kN/m^2}$ and 5 $\mathrm{kN/m^2}$, respectively. The model consists of horizontal boundary element (HBE) and vertical boundary element (VBE) and plate, the nonlinear properties of columns and beams are assumed to be a fiber P-M2-M3 hinge and for the plate assumed to be P-axial (Purba and Bruneau, 2010).

It is assumed that the angle of inclination is equal to 45 degrees and number of strips per panel is equal to 13 strips based on the previous researchers and guidelines.

3.1.1 Case 1

The selected 2-story, 4-story, 6-story, and 8-story steel frame buildings with SPSWs having different plate thicknesses were used for this case. It has a plan dimension of

33.5 m in the X-direction and 24 m in the Y-direction as mentioned in Figure 3.1 for each building. Elevation of 2-story, 4-story, 6-story, and 8-story buildings are shown in Figures 3.2 - 3.5, respectively. It is noted that in this case the aspect ratio of SPSWs is kept constant as 1.0 but the plate thicknesses change as 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm. Moreover, the SPSW included buildings were compared with the ones without SPSW (bare models).

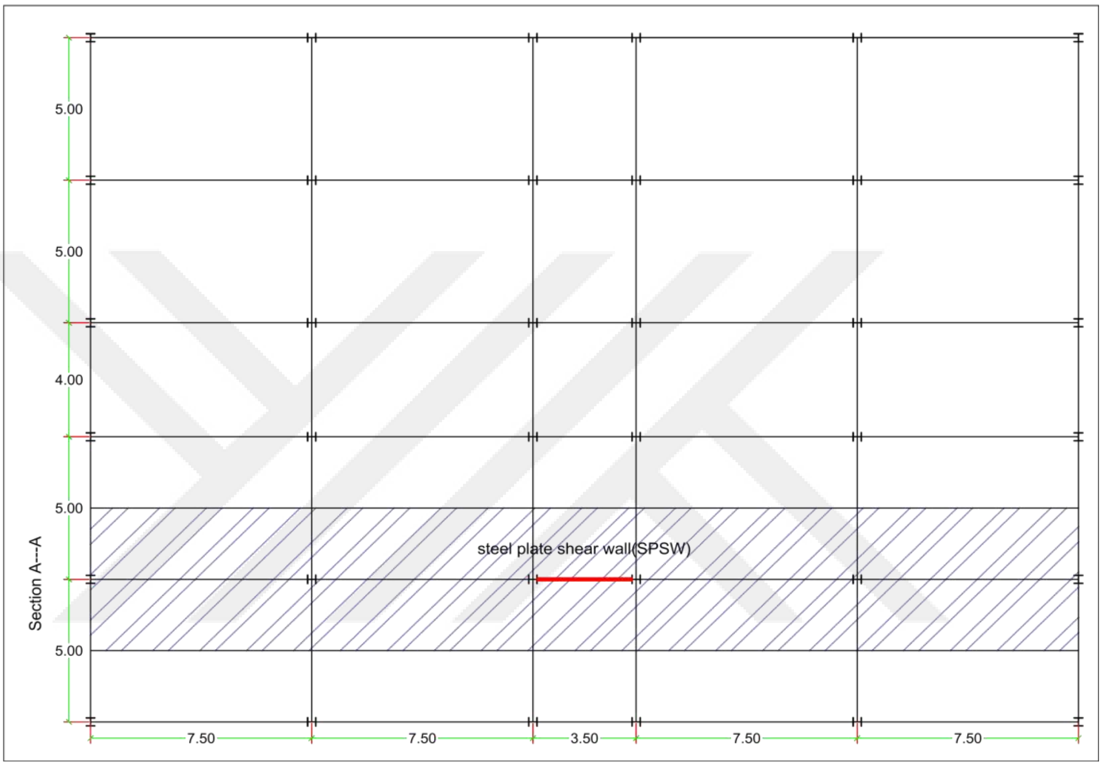


Figure 3.1 Plan of 2, 4, 6, and 8 story steel buildings with steel plate shear walls

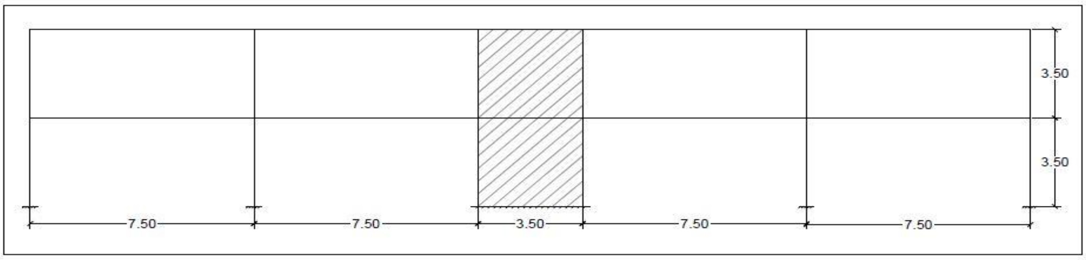


Figure 3.2 Elevation of 2-story building with steel plate shear walls having different plate thicknesses

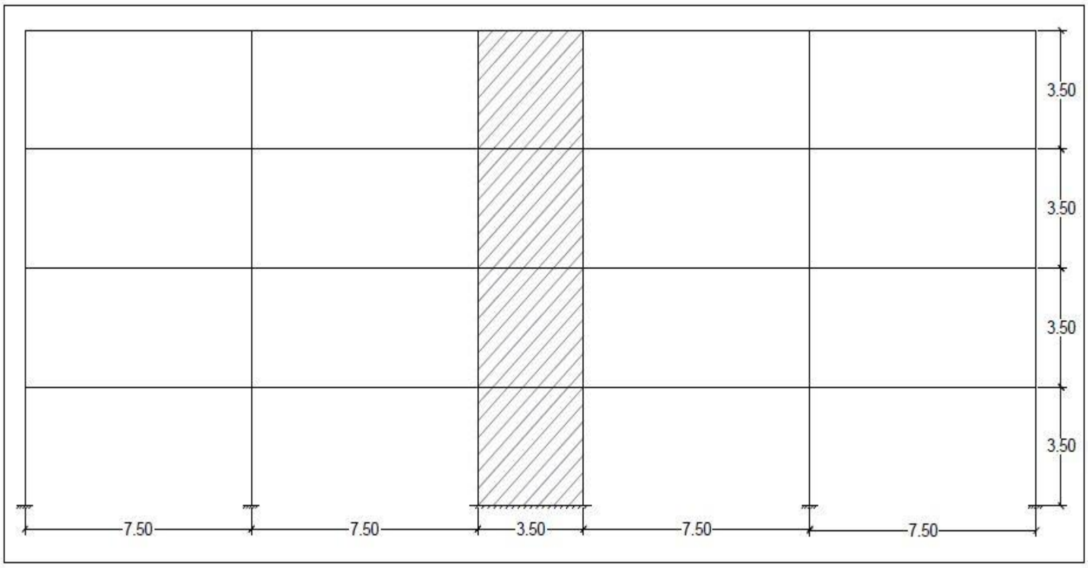


Figure 3.3 Elevation of 4-story building with steel plate shear walls having different plate thicknesses

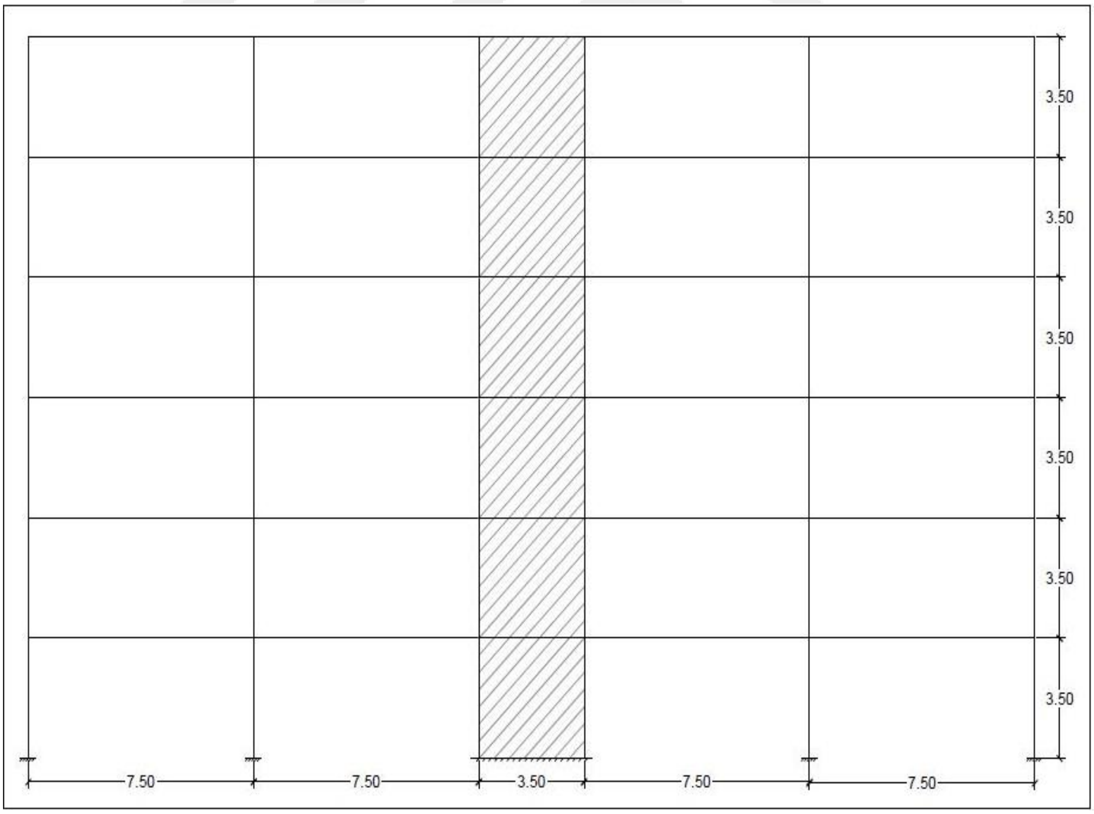


Figure 3.4 Elevation of 6-story building with steel plate shear walls having different plate thicknesses

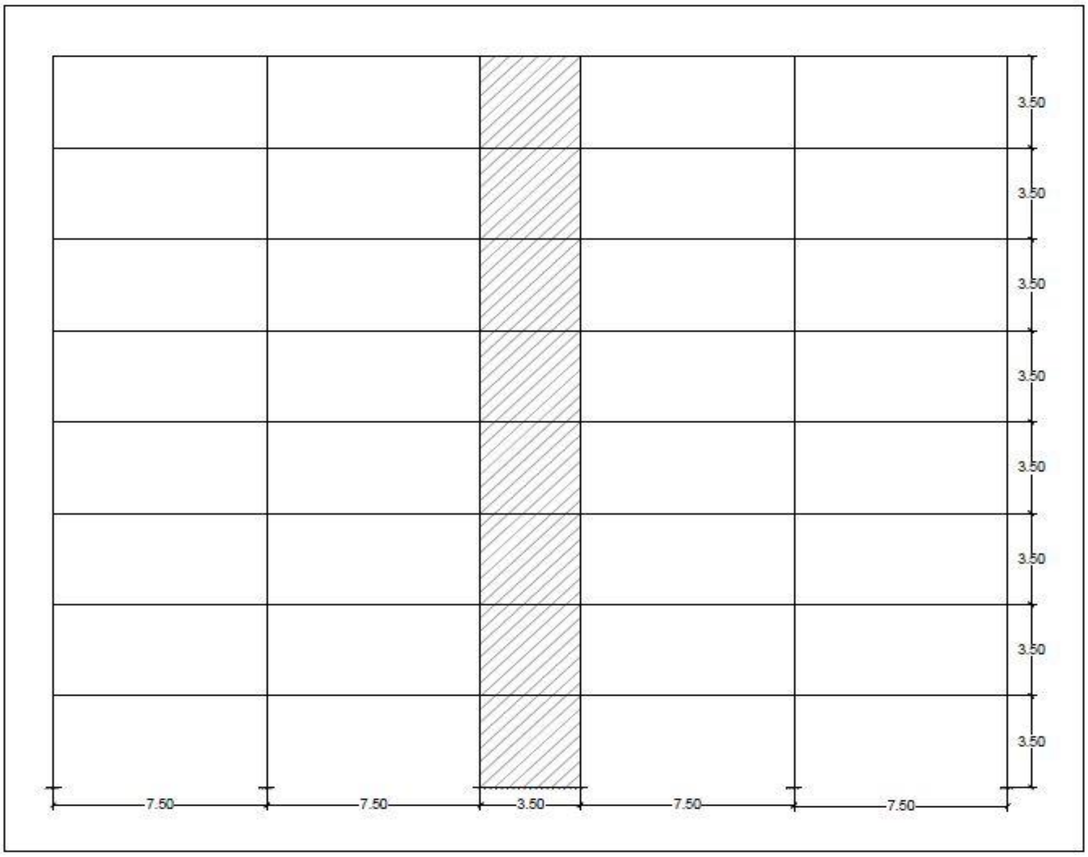


Figure 3.5 Elevation of 8-story building with steel plate shear walls having different plate thicknesses

3.1.2 Case 2

In this case, 4-story steel frame building with five different aspect ratios of 0.285, 0.570, 1.000, 1.430, and 2.142 and five different plate thicknesses of 1, 2.5, 5, 10, and 15 mm of SPSWs were studied. Plan dimension of X- direction is equal to 37.5 m, and that of Y-direction is equal to 24 m. They are totally five models having the length of infill plate panel equals to 1 m, 2 m, 3.5 m, 5 m, and 7.5 m as described below. The bare model is also considered for comparison purposes. The details of each model are given in Figures 3.6 - 3.15.

- Bare model: 4-story building without steel plate shear walls.
- Model 1: 4-story building with steel plate shear walls having aspect ratio of 0.285 and thickness of 1, 2.5, 5, 10 and 15 mm.

- Model 2: 4-story building with steel plate shear walls having aspect ratio of 0.570 and thickness of 1, 2.5, 5, 10 and 15 mm.

- Model 3: 4-story building with steel plate shear walls having aspect ratio of 1.000 and thickness of 1, 2.5, 5, 10 and 15 mm.

- Model 4: 4-story building with steel plate shear walls having aspect ratio of 1.430 and thickness of 1, 2.5, 5, 10 and 15 mm.

- Model 5: 4-story building with steel plate shear walls having aspect ratio of 2.142 and thickness of 1, 2.5, 5, 10 and 15 mm.

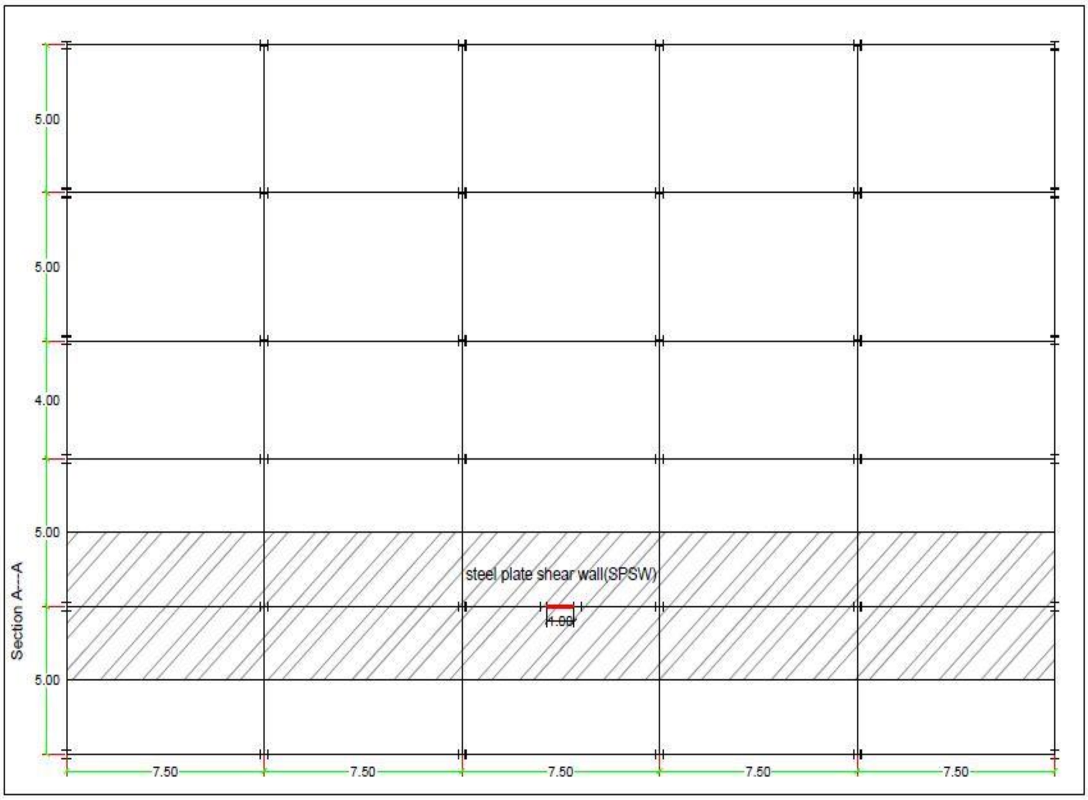


Figure 3.6 Plan of 4-story building with steel plate shear walls having aspect ratio of 0.285 (Model 1)

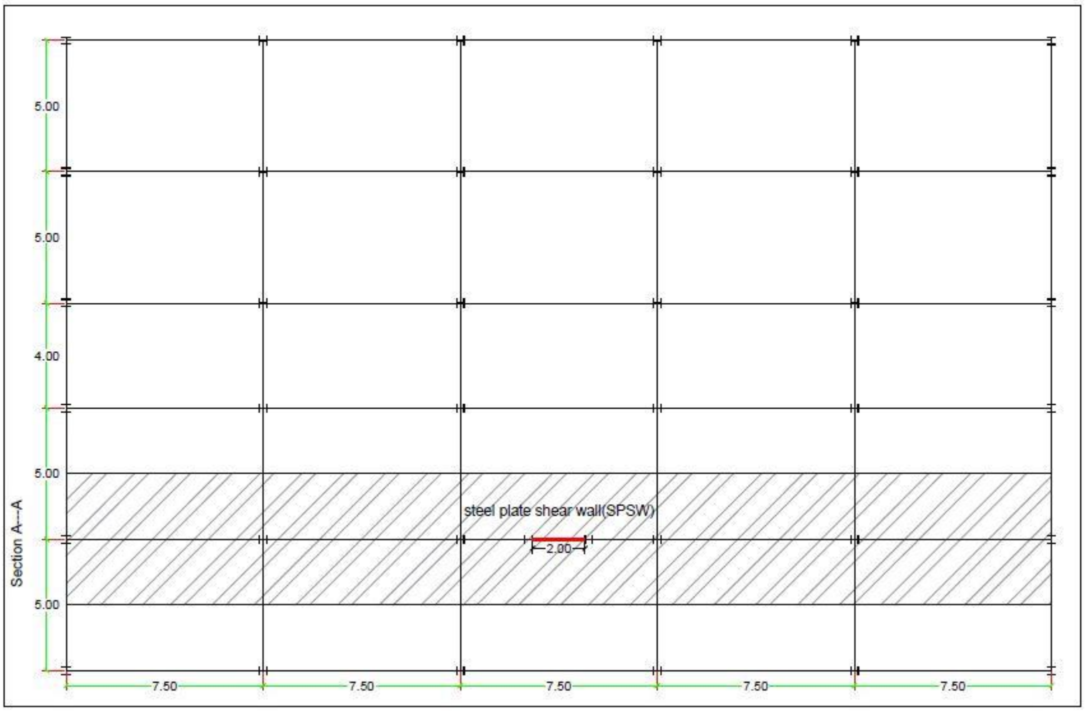


Figure 3.7 Plan of 4-story building with steel plate shear walls having aspect ratio of 0.570 (Model 2)

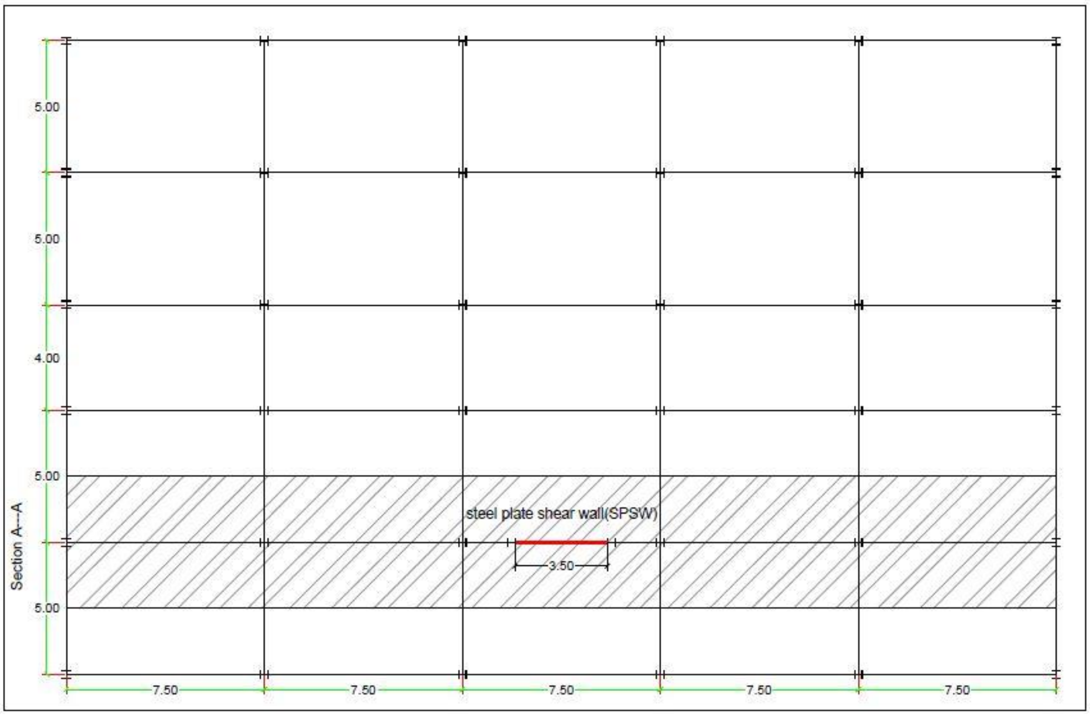


Figure 3.8 Plan of 4-story building with steel plate shear walls having aspect ratio of 1.000 (Model 3)

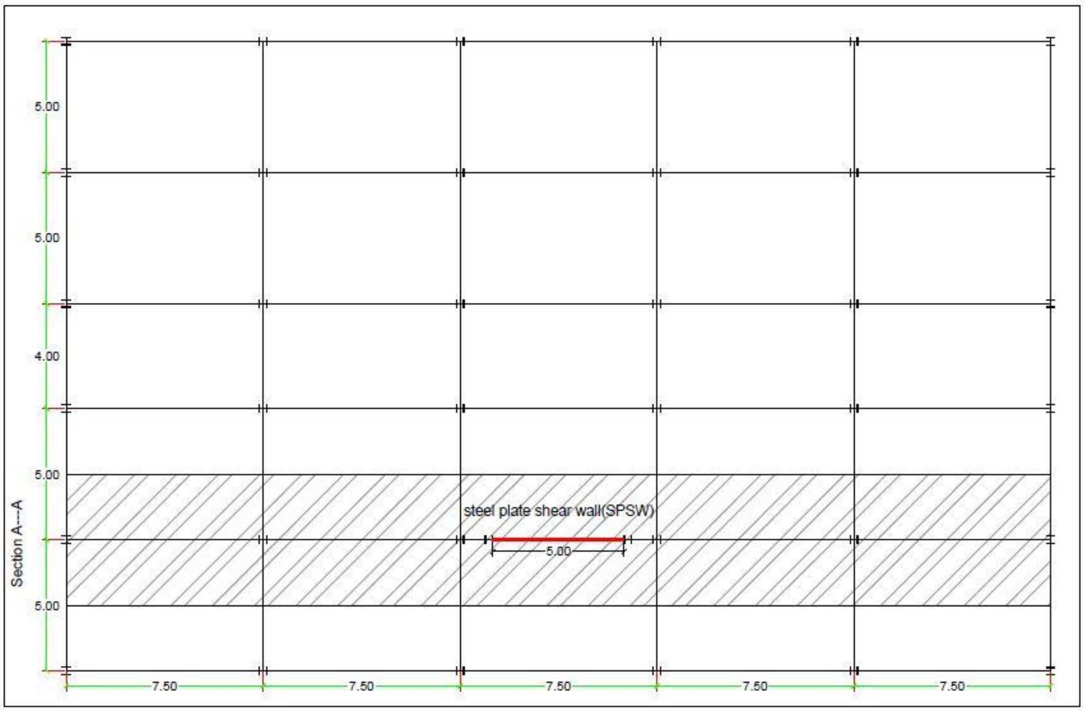


Figure 3.9 Plan of 4-story building with steel plate shear walls having aspect ratio of 1.430 (Model 4)

Figure 3.10 Plan of 4-story building with steel plate shear walls having aspect ratio of 2.142 (Model 5)

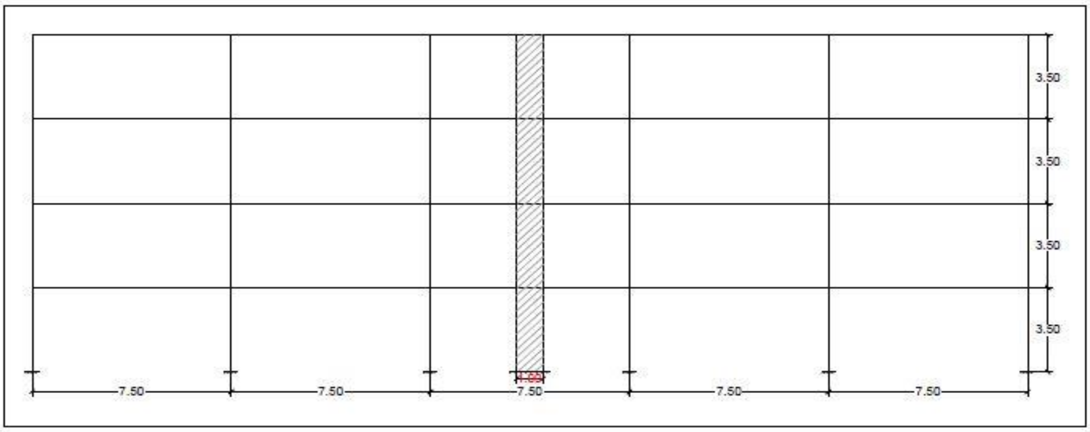


Figure 3.11 Elevation of 4-story building with steel plate shear walls having aspect ratio of 0.285 (Model 1)

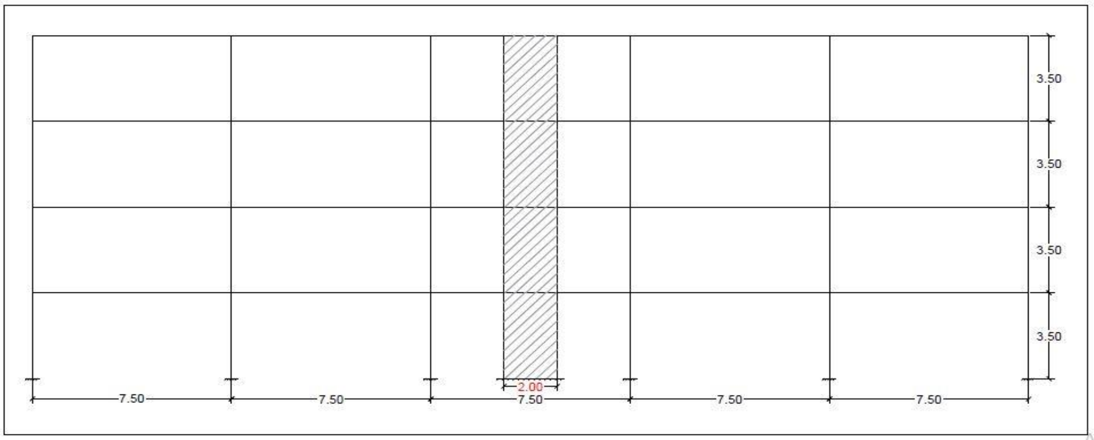


Figure 3.12 Elevation of 4-story building with steel plate shear walls having aspect ratio of 0.570 (Model 2)

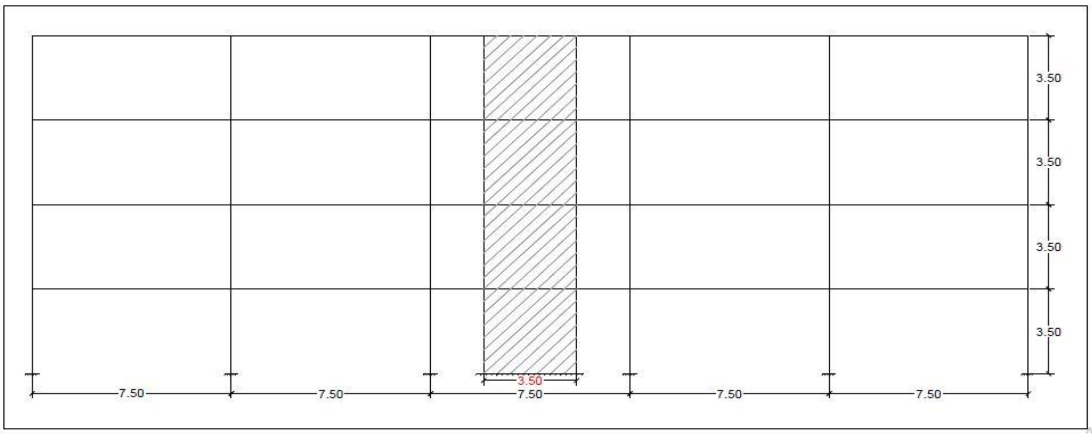


Figure 3.13 Elevation of 4-story building with steel plate shear walls having aspect ratio of 1.000 (Model 3)

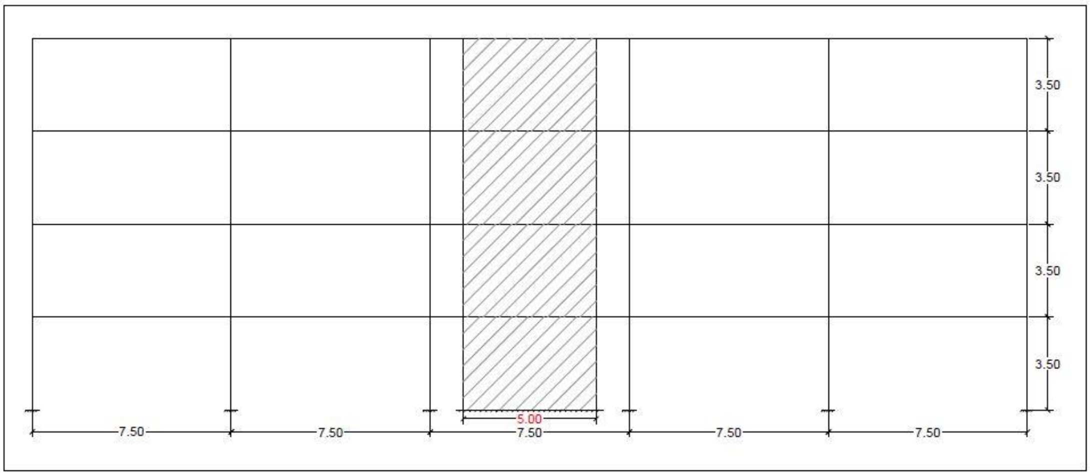


Figure 3.14 Elevation of 4-story building with steel plate shear walls having aspect ratio of 1.430 (Model 4)

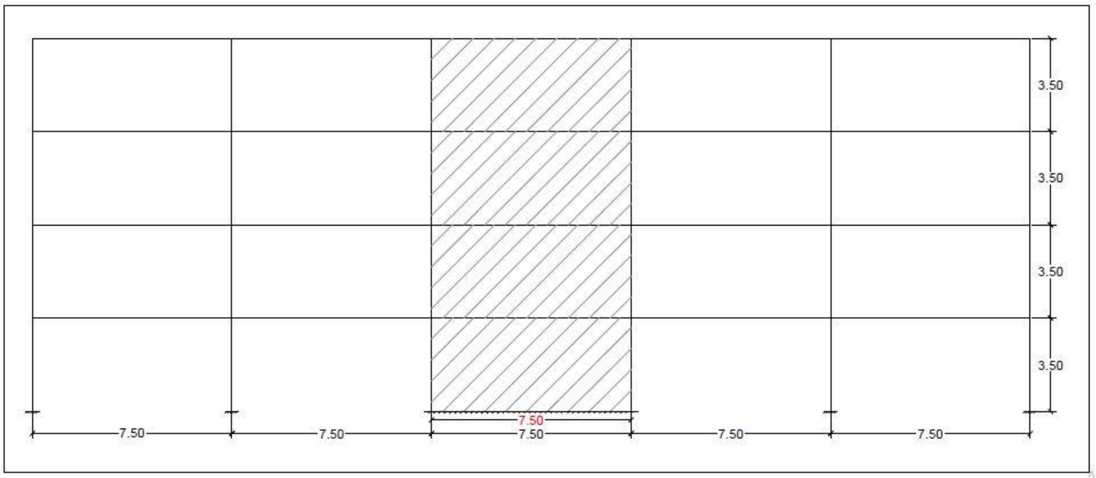


Figure 3.15 Elevation of 4-story building with steel plate shear walls having aspect ratio of 2.142 (Model 5)

3.2 Details of the analysis

A pushover analysis is a nonlinear analysis method that loads a structure monotonically to a specific load or displacement level while monitoring aspects of the response such as the deflection of a specified node the base shear. A pushover analysis takes into account material nonlinearities without this very difficult to engineer to calculate the behavior of post-yield and elastic. Geometric nonlinearities, such as P-A behavior, are mostly calculated for large deformations that can increase. Unless advanced finite element analysis package is used, the inelastic behavior would typically be concentrated in pre-defined locations where plastic hinges are expected to form. The lateral loads were created to approximate the relative inertial forces that are applied at specific places on the building with a specific substantial mass. Finally, the building has applied the load under the predefined lateral load patterns to a specified target roof displacement. Force in any member at the displacement level can estimate both of the strength and deformation demands (Shishkin, 2006).

In the pushover analysis, monotonically growing side powers are carried out to a nonlinear mathematical typical of the structuring up to the time that movement of the authority node at the surface level excels the aim displacement. The lateral forces must be carried out to the structuring using issues or portraits that concentrated, although closely it is predicted in the distribution of strong design deadlock earthquake. The presently NEHRP strategies like FEMA 356 (2000) and FEMA 273 (1997) show that, for an accurate earthquake, the building must have enough capability to resist a particularized roof shift. It is named the objective movement and is clarified as a predict of the expected building roof dislocation in the design earthquake. The purpose of the use of the pushover analysis is the value of the target displacement at which seismic behavior evaluation of the building is to be executed. Target displacement prepares as an approximation of the global displacement of the building is expected to occurrence within a project earthquake (FEMA 356 ,2000; FEMA 273 ,1997).

In the structural analysis program of SAP2000, the base shear and displacement results are defined at user-specified increments during a pushover analysis to take out the inelastic behavior of the modeled structure during loading. Steps during certain significant happens, such as yielding or the unloading of a hinge, that do not occur happen at the specified increment are also saved. Defining a small increment size can prevent the occurrence of anomalies, such as load spikes, from showing in the pushover curve. A load spike, which manifests itself as a vertical increase or decline in load on the pushover curve, may happen when the analysis goes from one specific equilibrium path to another. In addition, all parameter of pushover analysis must be set to include enough points are saved to represent the important features of the pushover curve sufficiently (Shishkin, 2006).

Krawinkler and Seneviratna (1997) state that the aim of the pushover analysis is to predict the probable behavior of a structural system by predicting its strength and displacement demands in design seismic by means of a static. Inelastic analysis, and likening these values to obtainable capacities at the behavior levels of interest the assessment is depending on an assessment of important performance parameters, such as global drift, inelastic element deformations, inter-story drift, deformations between elements, and element and connection forces. The inelastic static pushover analysis could be defined as a method for assuming seismic force and displacement demands, which calculates in a near method for the redeployment of internal forces happening when the building is exposed to inertia forces that no longer may be fought within the elastic range of structural performance. The pushover is anticipated to offer information on many response features that impossible attained from an elastic static or dynamic analysis. The following are examples of such response characteristics:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam-to-column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.

- Predicts of the displacement demand elements that have to deform in elastically in order to disintegrate the energy transported to the building by ground motions.

- Consequences of the strength deterioration of individual elements on the behavior of the structural system.

- Documentation of the critical areas in which the deformation demands are probable to be high and that have to become the emphasis of thorough specifying.

- Documentation of the strength interruptions in plan or elevation that will lead to changes in the dynamic physiognomies in the inelastic range.

- Guesses of the inter-story drifts that reason for strength or stiffness discontinuities and that might be used to control harm and to evaluate P-delta effects.

In modeling the steel plate shear walls (SPSWs), strip modeling, orthotropic modeling, and finite element modeling are used. Among them, the strip modeling is the commonly utilized model. The strip model represents the panels as a series of separated pin-ended diagonal strips, that are aligned with the average angle of inclination of the major principal stresses that develop under lateral loads, that are able to communicating just tension forces. A dimension of strips depending on a number of strips and thickness of the plate, an area of strips equal the width of strips multiple by the thickness of plate as shown in Figure 3.16. Lateral loads are then assigned to the model to define the performance of the structure. Thus, design and analysis of unstiffened steel plate shear walls are based on the post-buckling strength of the infill panels (Thorbum et al., 1983).

It is also reported that ten to fifteen strips are suitable to be provided per wall panel. Each strip is pinned at both of its ends to the surrounding beams or columns as per its location in the wall panel. A width of strips is equals to the center to center spacing of the consecutive strips. A thickness of strips is kept same as that of the plate. Area of strips is equals a width of strips multiplied by the thickness of the plate. The strips are normally inclined at 45° to the vertical. The value angle of inclination must be around 38° to 45° with the vertical. Small distinction in the angle of inclination does not affect significantly the behavior of the model, and axial-P hinge in the middle for all strips is used (Driver, 1998; Elgaaly, 1993; Berman, 2003; Bruneau, 2002; Bruneau, 2010; Lubeel, 2000; CAN/CSA S16-01, 2001).

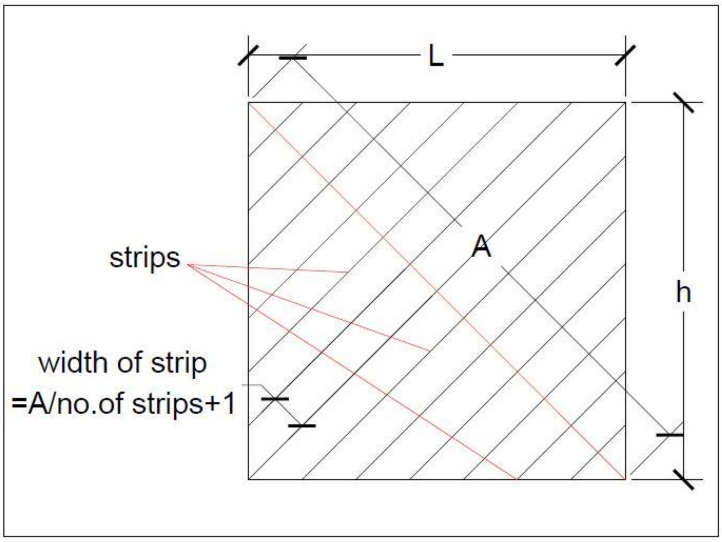


Figure 3.16 Detail of strip model according to the study of Thorbum et al. (1983)

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

In this section, the results obtained from two different cases under investigation were given and discussed comparatively. These results were achieved through the nonlinear pushover analysis. In the first case analysis, 2-story, 4-story, 6-story and 8-story steel buildings with and without SPSWs having different thicknesses and fixed aspect ratio were considered. In the second case analysis, 4-story steel building with and without SPSWs having different aspect ratios and different thicknesses were considered. As mentioned before, there are five models in second case (model 1, model 2, model 3, model 4, and model 5) having different aspect ratios of 0.285, 0,570, 1.000, 1.430 and 2.142, respectively. The structural response of different multi-story steel buildings were examined and the analysis of the results were discussed below.

4.2 Case 1

4.2.1 Pushover curve variation with plate thickness

The effect of using SPSWs on the lateral load carrying capacity of the multi-story buildings (2, 4, 6, and 8-story steel buildings) having different thicknesses of plate through the nonlinear pushover analysis are given in Figures 4.1 - 4.4. Assessment of the figures for all steel buildings indicated a substantial improvement in the seismic performance of the building after using SPSWs.

As seen in the figures, the buildings with SPSW had frequently greater capacity in comparison to the buildings without SPSW. For example, for the 2-story steel building as noted in Figure 4.1, the maximum base shear was equal to 1205 kN, but after using 1 mm of SPSW, the maximum base shear increased to 4902 kN. This implied about 4.06 times higher lateral load carrying capacity for the 2-story steel building with 1 mm of SPSW in comparison to the building without SPSW. Also after using 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the maximum base shear was gone up to 5895 kN, 6642 kN, 7235 kN, and 7776 kN, respectively. This indicated that the addition of 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate cases had about 4.89, 5.51, 6.00, and 6.45 times higher lateral load carrying capacity than the building without SPSW, respectively. Buildings with SPSWs were observed to be capable of more lateral load carrying and energy absorption capacities.

For 4-story steel building, as exposed in Figure 4.2, the maximum base shear was equal to 2986 kN, 3344 kN, 3615 kN, 3898 kN, and 4033 kN, when applying the steel building by addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate, respectively while before using SPSW the maximum base shear was equal to 847 kN. This implied about 3.52, 3.94, 4.26, 4.60, and 4.76 times higher lateral load carrying capacity for 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate case, respectively in comparison to the original building.

Another instance, for 6-story steel building as displayed in Figure 4.3, the maximum base shear was equal to 777 kN for the original building, but the maximum base shear was increased to 1839 kN, 2021 kN, 2261 kN, 2537 kN, and 2599 kN after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, respectively. Similar to 4- story steel building, in the 6-story steel building with plate thickness of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, about 2.36, 2.60, 2.90, 3.26, and 3.34 times increment in the lateral load carrying capacity was observed as compared to the normal building.

Also, as seen in Figure 4.4, the maximum base shear for 8- story steel building before adding infill plate was equal to 633 kN, but with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the maximum base shear rose to 1358 kN, 1479 kN, 1637 kN, 1842 kN, and 1951 kN, respectively. This indicated that the 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW case had about 2.14, 2.33, 2.58, 2.90, and 3.08 times higher lateral load carrying capacity than the building without SPSW.

The results demonstrated that the characteristic properties of ultimate strength and initial stiffness of shear wall were improved as the plate thickness increased. In addition, the SPSWs were more effective not only in a low-rise steel buildings but also in a high-rise steel buildings. For example, by the use of SPSWs, for the 2-story steel building, the maximum base shear was arrived to 7776 kN, but with increasing number of story directly the maximum base shear declined to 4033 kN for the 4-story steel building. For 6, 8-story steel buildings, the maximum base shear was gone down to 2599 kN and 1951 kN, respectively. As discussed in the above, the capacity of the bare building enhanced significantly by means of the SPSWs. However, the level of improvement was mainly governed by the plate thickness.

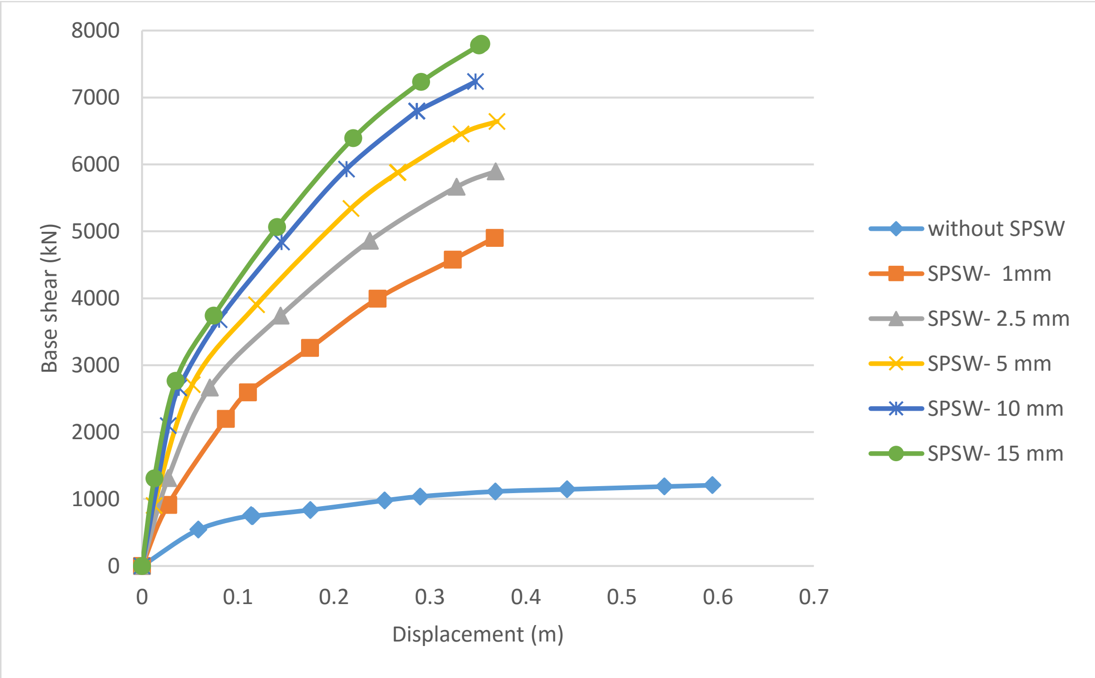


Figure 4.1 Pushover curves for 2-story steel building with and without steel plate shear walls having different thicknesses

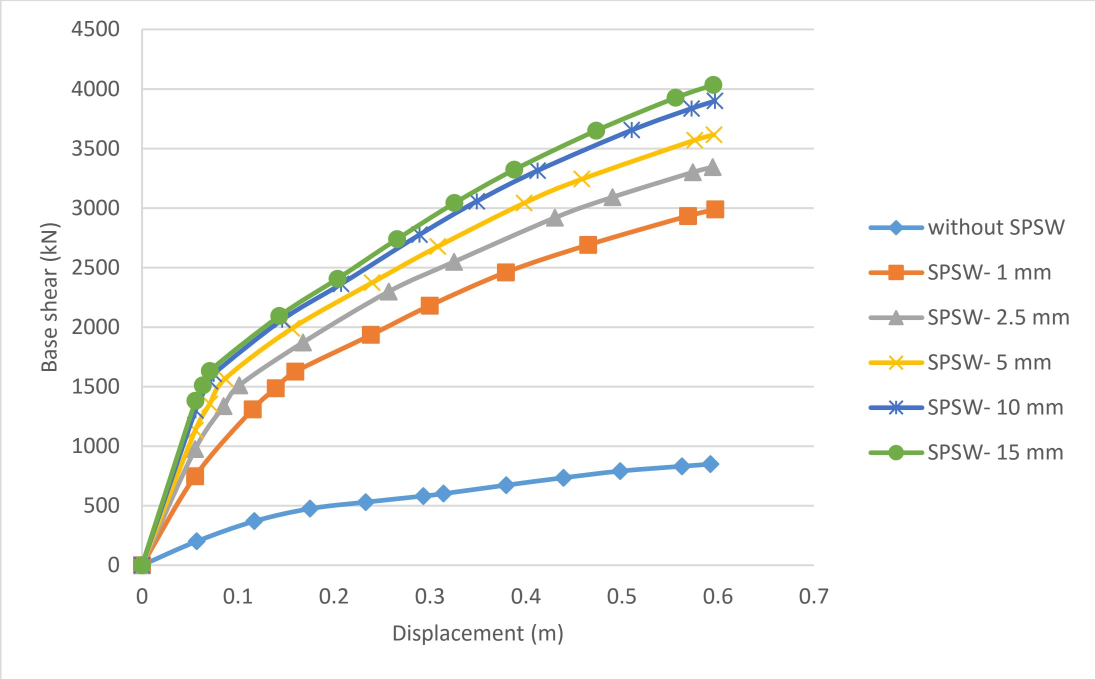


Figure 4.2 Pushover curves for 4-story steel building with and without steel plate shear walls having different thicknesses

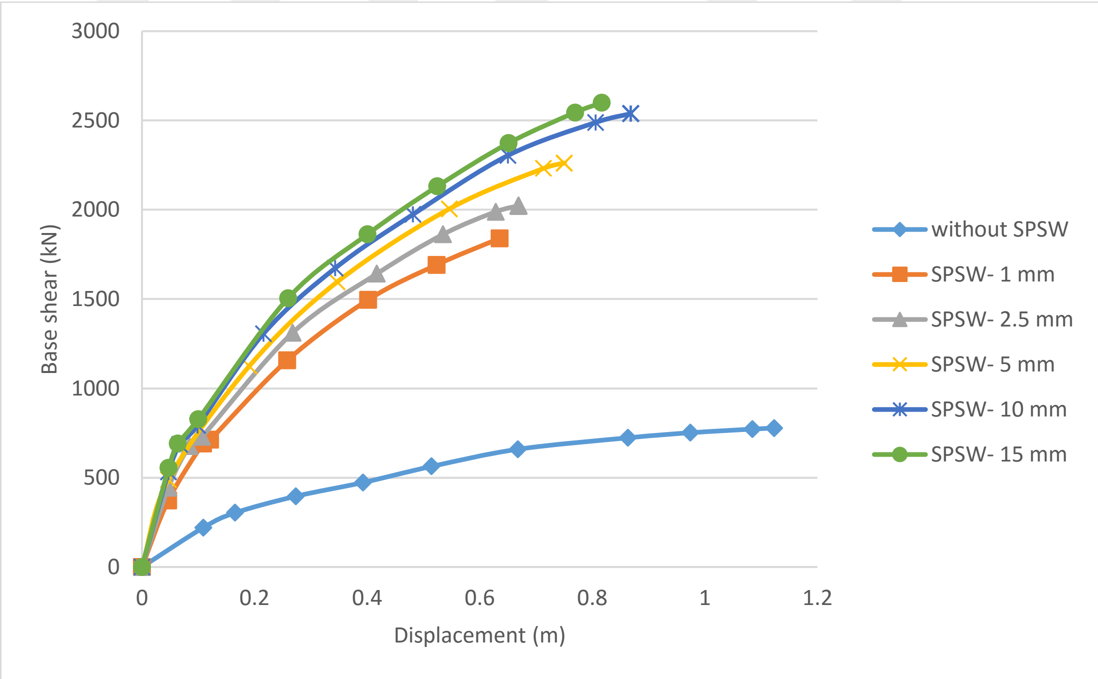


Figure 4.3 Pushover curves for 6-story steel building with and without steel plate shear walls having different thicknesses

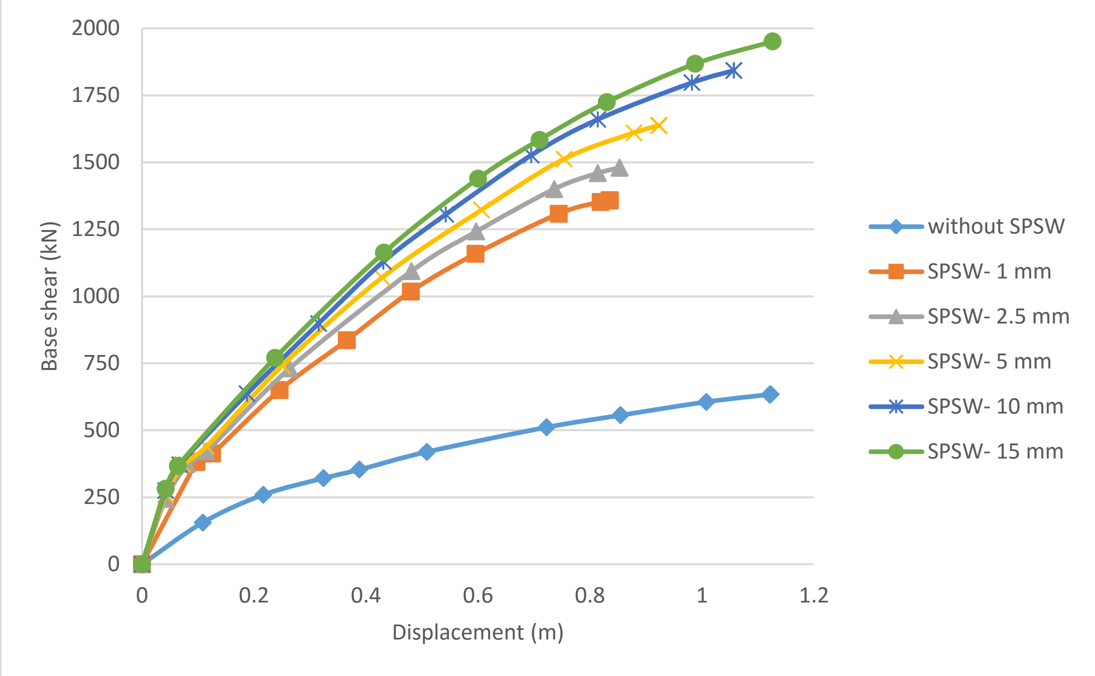


Figure 4.4 Pushover curves for 8-story steel building with and without steel plate shear walls having different thicknesses

4.2.2 Initial stiffness with plate thickness

The effect utilizing SPSWs on initial stiffness of the multi-story steel buildings having 2-story, 4-story, 6-story, and 8-story were studied for different thicknesses. The capacity curves give us valuable information on the stiffness characteristics of the examined structures. In order to see the contribution of SPSWs owing different thicknesses on the original stiffness of the building, the following curves were drawn as shown in Figures 4.5 - 4.8. It was observed that by increasing the thickness of plate, the initial stiffness of the system rose, viewing an enhancement in the total performance.

For 2- story steel building, according to Figure 4.5, the initial stiffness for the building without infill plate was equal to 9233 kN/m, but with plate thickness 1 mm, the initial stiffness increased to 33703 kN/m. This causes about 24470 kN/m increment. Also after using 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the initial stiffness was evaluated as 48459 kN/m, 71913 kN/m, 76831 kN/m, and 102665 kN/m, respectively. The result illustrated that after addition of 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, the initial stiffness has risen about 39226 kN/m, 62680 kN/m, 67598 kN/m, and 93432 kN/m in comparison to the building without SPSW.

Another instance, for 4-story steel building as observed in Figure 4.6, indicating the initial stiffness of the structure with and without SPSW. The initial stiffness was equal to 3516 kN/m for building without infill plate, but the initial stiffness increased to 13507 kN/m, 17614 kN/m, 20614 kN/m, 23339 kN/m, and 24784 kN/m, after supporting building by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15mm of SPSW, respectively. This means building with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, it has climbed about 9991 kN/m, 14098 kN/m, 16924 kN/m, 19823 kN/m and 21268 kN/m as compared with original building.

The initial stiffness for 6-story steel building without SPSWs was equal to 2022 kN/m, but after strengthened by the addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate. Ir was got up to 7979 kN/m, 9478 kN/m, 10411 kN/m, 11270 kN/m, and 11743 kN/m, respectively as displayed in Figure 4.7. The result demonstrated that the initial stiffness of the building increased about 5957 kN/m, 7456 kN/m, 8389 kN/m, 9248 kN/m and 9721 kN/m due to the use of the plate thickness of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively.

Futhermore, for 8-story steel building as governed in Figure 4.8, before adding SPSW to the structure, the initial stiffness was equal to 1432 kN/m, while after establishing the structure by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the initial stiffness was gone up to 3941 kN/m, 5774 kN/m, 6163 kN/m, 6493 kN/m, and 6673 kN/m, respectively. After looking in this figure, it was seen that the initial stiffness rose about 2509 kN/m, 4342 kN/m, 4731 kN/m, 5061 kN/m, and 5241 kN/m by using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate, respectively.

The figures indicated that the initial stiffness increased almost linearly by increasing the thickness of plate steel and hence the stability capacity of SPSW is enhanced. They are demonstrated that by changing the plate thickness from 1 to 15 mm, the initial stiffness of the shear wall rises, otherwise by increasing the number of story the initial stiffness declines. Obtained value for the initial stiffness was improved after establishing the steel building by SPSW, and eventually the performance of the structure could be going well against lateral load.

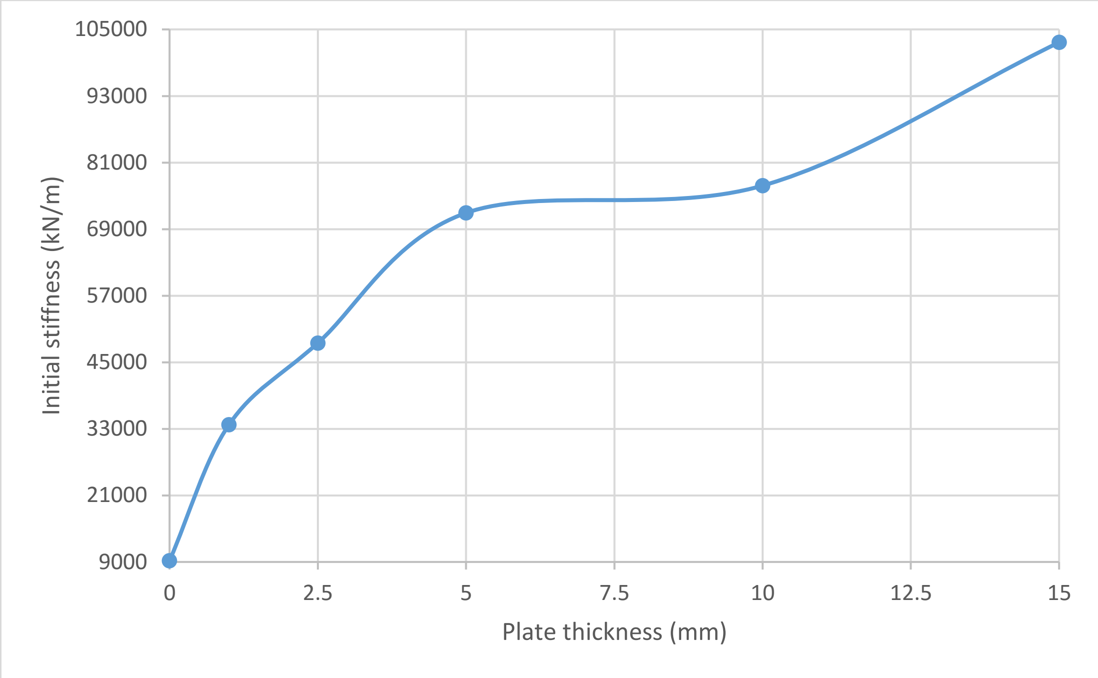


Figure 4.5 Effect of plate thickness on initial stiffness of the 2-story steel building with and without SPSW

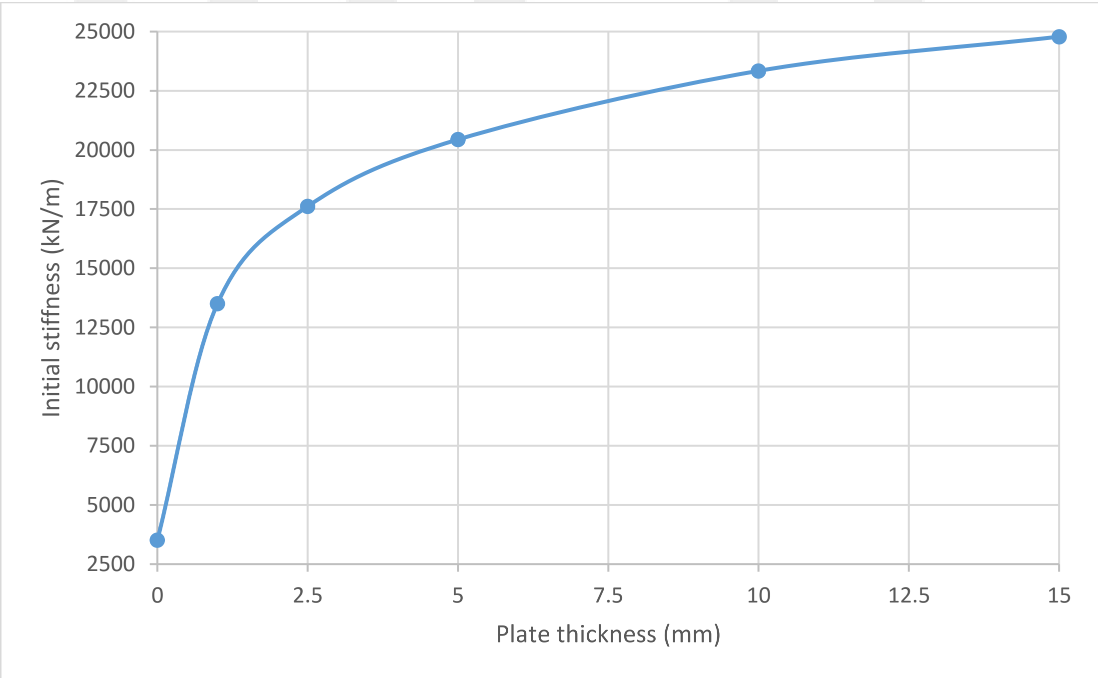


Figure 4.6 Effect of plate thickness on initial stiffness of the 4-story steel building with and without SPSW

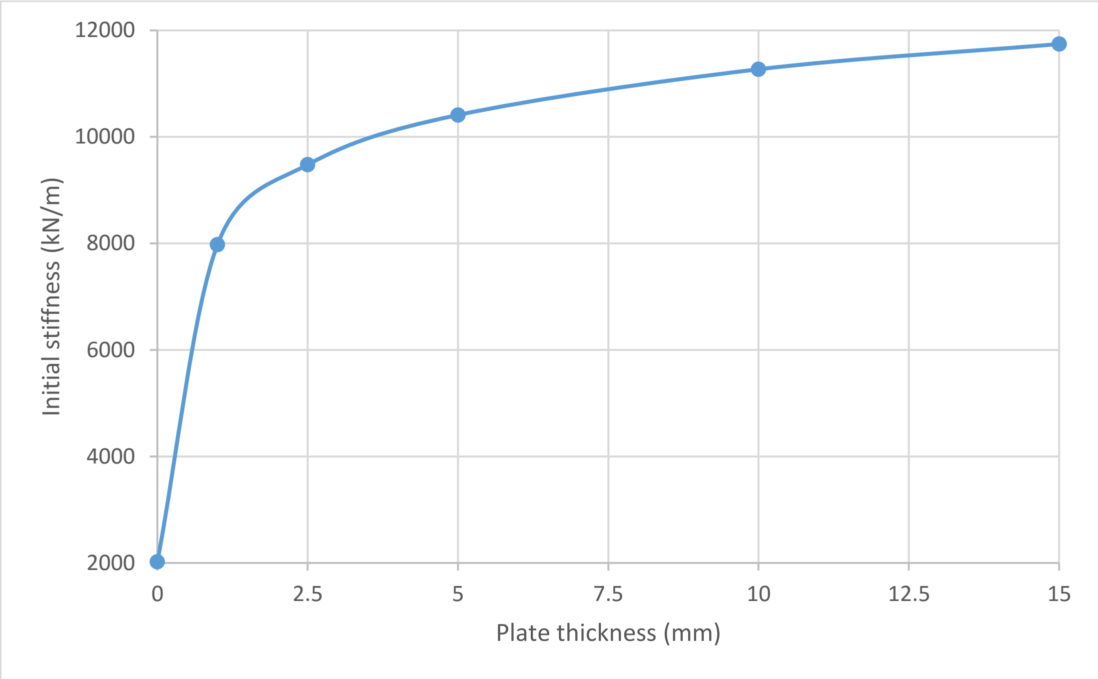


Figure 4.7 Effect of plate thickness on initial stiffness of the 6-story steel building with and without SPSW

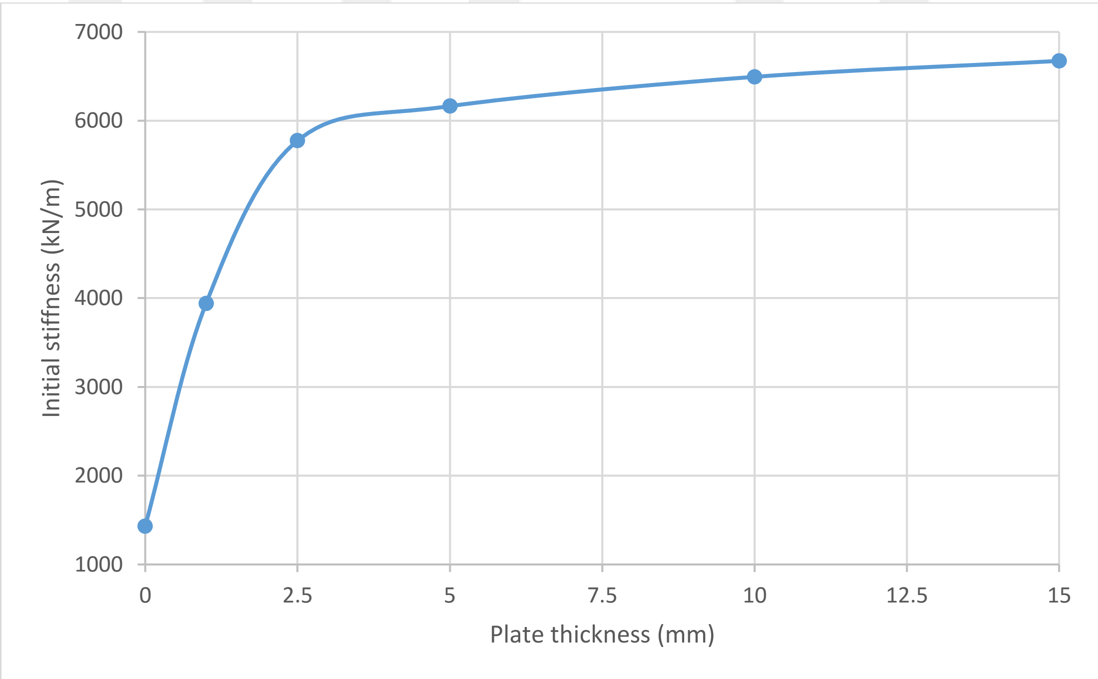


Figure 4.8 Effect of plate thickness on initial stiffness of the 8-story steel building with and without SPSW

4.2.3 Ultimate strength with plate thickness

The ultimate value of the base shear (Vmax) is obtained from the capacity curves of each building. In each building (2, 4, 6, 8-story steel building), by using SPSWs having different thicknesses, the ultimate strength also changes, increasing plate thickness results in grater strength. From Figures 4.9 - 4.12 the effect of the plate thickness on the ultimate strength of the structural system was illustrated. All figures confirmed that the ultimate strength increased significantly with increasing the plate thickness.

Figure 4.9 shows the ultimate strength variation of 2-story steel building without SPSW and after using SPSW having different thicknesses. The results demonstrated that the ultimate strength was equal to 1205 kN, but after using 1mm of infill plate the value of ultimate strength was gone up to 4902 kN. This indicated the ultimate strength increased about 3697 kN in comparison to the building without steel plate shear walls. Also after using 2.5 mm, 5 mm, 10 mm, and 15 mm of plate thickness, the ultimate strength rose to 5895 kN, 6642 kN, 7235 kN, and 7776 kN, respectively. The result illustrated that after the addition of 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, the ultimate strength has risen about 4690 kN, 5437 kN, 6030 kN, and 6571 kN, respectively in comparison to the building without SPSWs.

Moreover, for 4-story steel building it was observed from Figure 4.10, the ultimate strength without infill plate and after using infill plate, the ultimate strength for 1mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW was equal to 2986 kN, 3344 kN, 3615 kN, 3898 kN, and 4033 kN, respectively while before using SPSW it was equal to 847 kN. After looking it was found that the ultimate strength has increased about 2139 kN, 2497 kN, 2768 kN, 3051 kN, and 3186 kN, while using plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively as compared to the origin building.

For 6-story steel building, it was the value of the ultimate strength of 777 kN for the original building, this value was gone up to 1839 kN, 2021 kN, 2261 kN, 2537 kN, and 2599 kN, after strengthening by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, respectively. As well as, after strengthening by addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, the ultimate strength has climbed about 1062 kN, 1244 kN, 1484 kN, 1760 kN, and 1822 kN comparing with the building without infill plate. All above information was given in Figure 4.11.

For 8-story steel building, it was also evident in Figure 4.12 that the ultimate strength of the structure without SPSW and after using SPSW having different thicknesses was significantly affected. For the normal building, the ultimate strength was equal to 633 kN but with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the ultimate strength was increased to1358 kN, 1479 kN, 1637 kN, 1842 kN, and 1951 kN, respectively. The ultimate strength has risen about 725 kN, 846 kN, 1004 kN, 1209 kN, and 1318 kN in comparison to the building without SPSW while using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW.

In addition, each figure demonstrated that with changing thickness from small value to large the ultimate strength also changed and increased with increasing plate thickness. Observed results indicated that the strength rose roughly linearly by rising steel plate thickness and finally behavior of the building improved. All figures showed that by increasing plate thickness from 1 to 15 mm, the ultimate strength of the system was increased. Nevertheless, by increasing number of story strength decrease, for example, after using 15 mm of infill plate the ultimate strength for 2-story steel building was equal to 7799 kN, but by increasing the number of story to 4-story steel building, the ultimate strength declined to 4033 kN and also it was equal to 2599 kN and 1951 kN for 6, 8-story steel buildings, respectively and also the same for all another plate thicknesses.

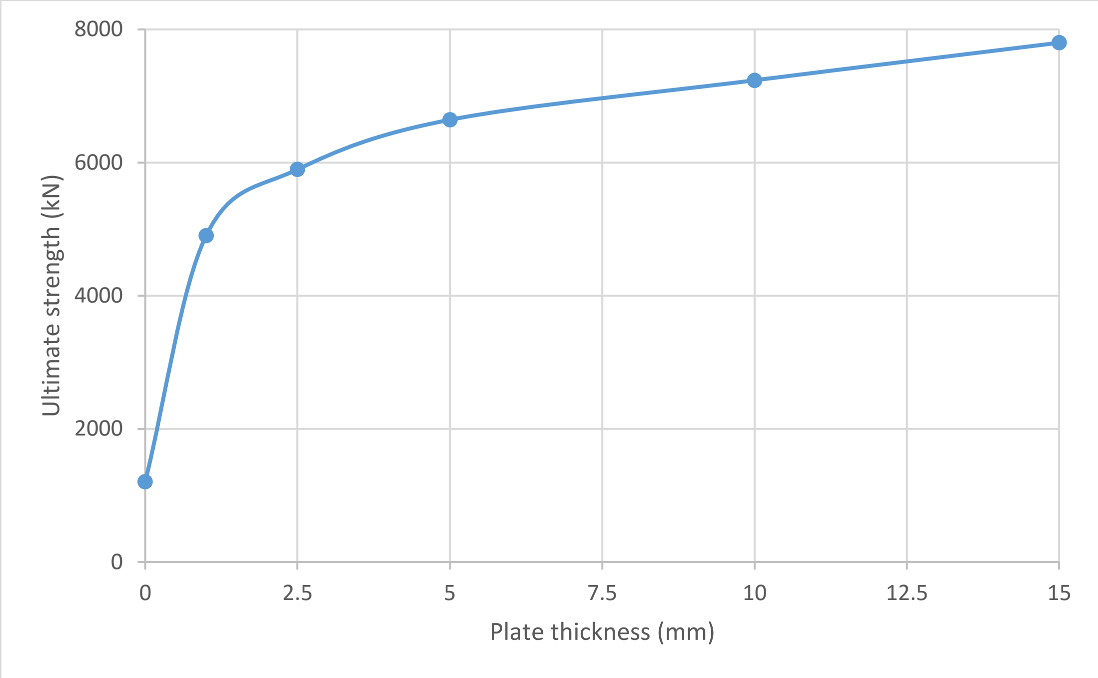


Figure 4.9 Effect of plate thickness on ultimate strength of 2-story steel building with and without SPSW

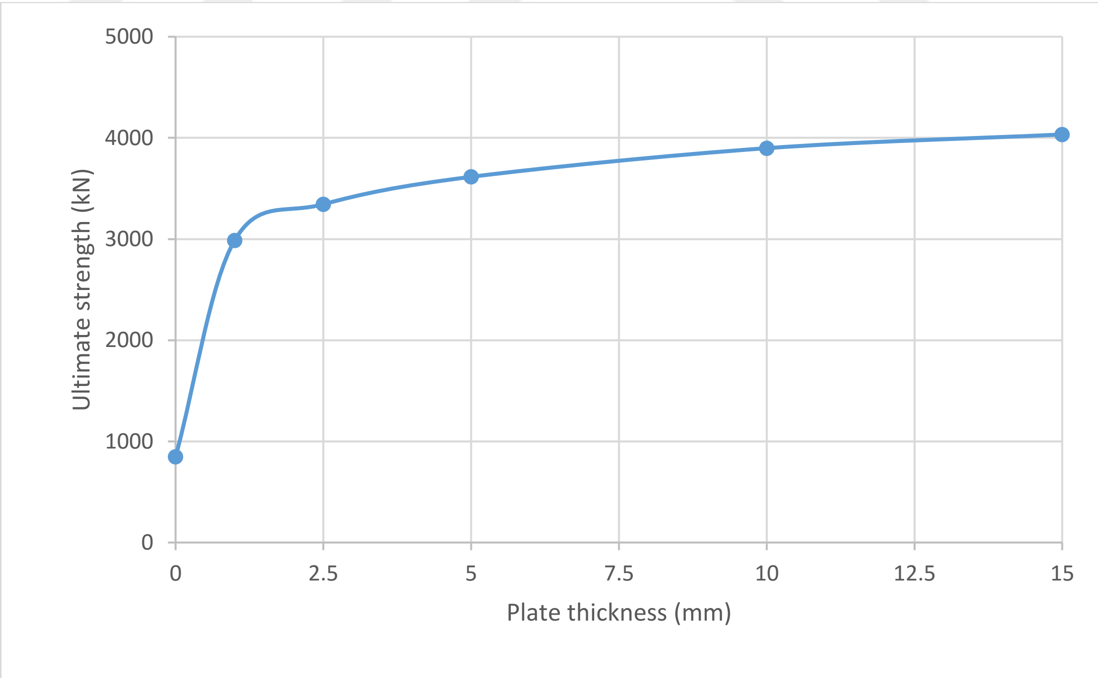


Figure 4.10 Effect of plate thickness on ultimate strength of 4-story steel building with and without SPSW

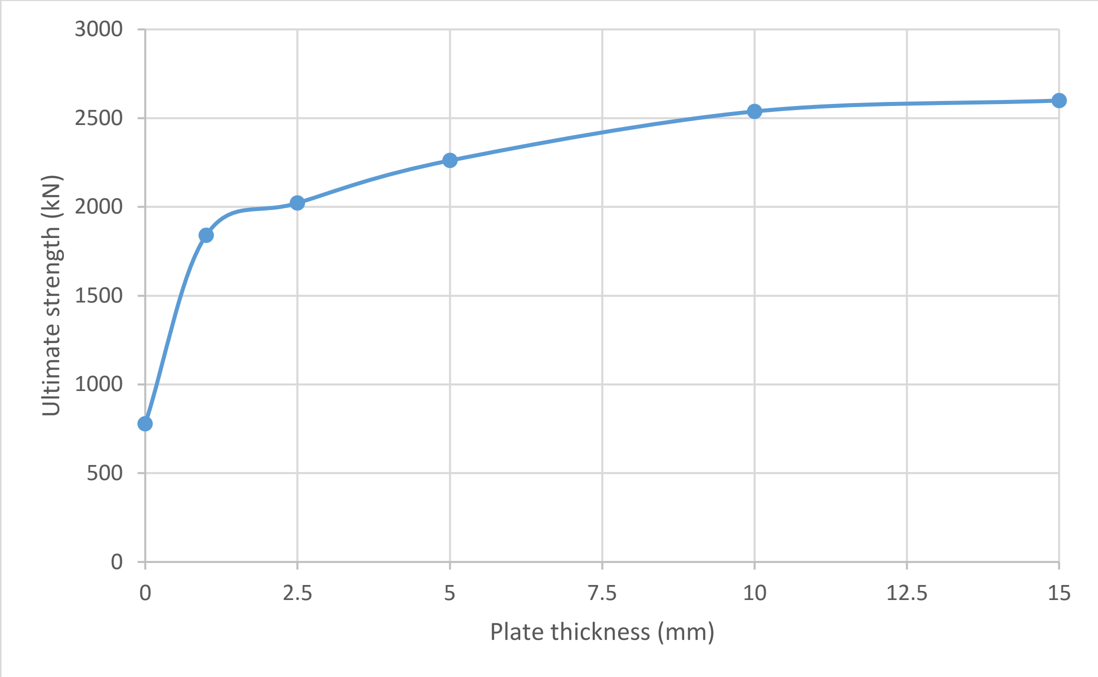


Figure 4.11 Effect of plate thickness on ultimate strength of 6-story steel building with and without SPSW

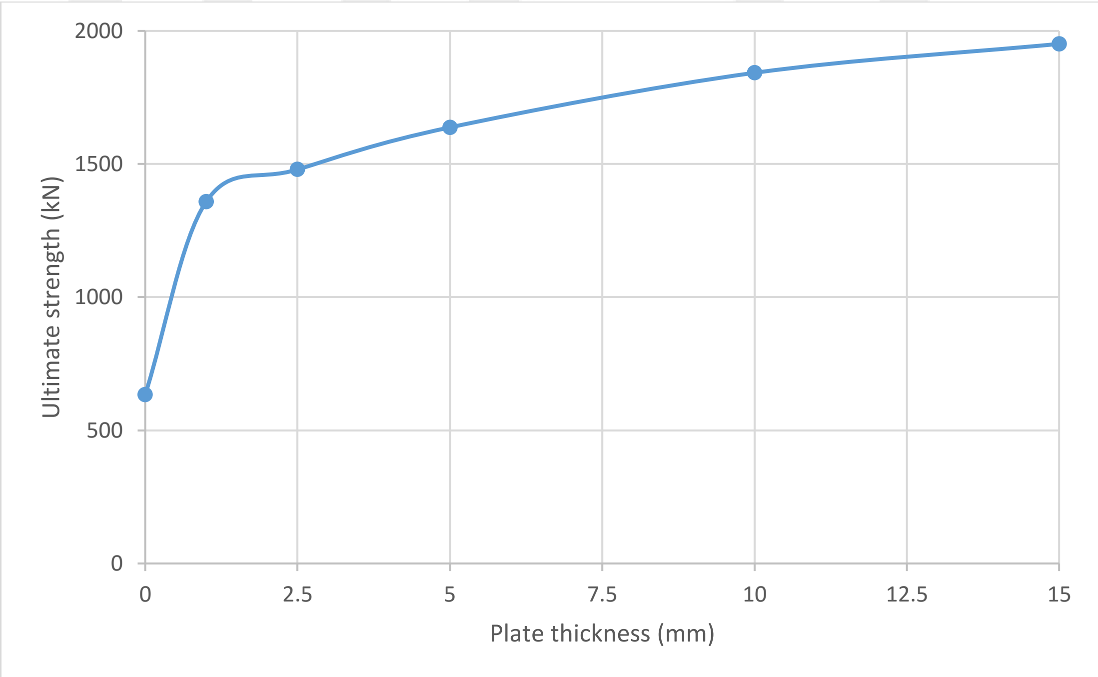


Figure 4.12 Effect of plate thickness on ultimate strength of 8-story steel building with and without SPSW

4.3 Case 2

4.3.1 Pushover curve variation with plate thickness

The effect of SPSW on the lateral load carrying capacity of the 4-story building having different aspect ratios of 0.285 to 2.142 were investigated under different plate thicknesses of 1 to 15 mm through the nonlinear pushover analysis. Five models were studied in this case. Model 1 considers the steel plate shear walls having the aspect ratio of 0.285, model 2 has steel plate shear walls having the aspect ratio of 0.570, model 3 covers steel plate shear walls having the aspect ratio of 1.000, model 4 takes into accost steel plate shear walls having the aspect ratio of 1.430, and model 5 has steel plate shear walls having the aspect ratio of 2.142. Each model has also five different plate thicknesses of 1, 2.5, 5, 10, and 15 mm. Thus, a total of 25 different model cases were studied in the mentioned case 2. The bare model (without SPSWs) was also compared with the others. When the pushover curves shown in Figures 4.13 - 4.17 were assessed for all models, a substantial improvement was observed in the seismic performance of the building after using SPSW. For all different aspect ratios with 4-story steel building after using SPSW increased the capacity curve with increasing thickness of steel plate shear walls.

For example, for Model 1 as shown in Figure 4.13, the maximum base shear was equal to 1230 kN, but after using 1 mm of SPSW led to increase the maximum base shear to 1505 kN. This implied about 1.22 times higher lateral load carrying capacity for the building with 1 mm of SPSW in comparison to the building without SPSWs. Also after using of 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, the maximum base shear was gone up to 1558 kN, 1613 kN, 1684 kN, and 1731 kN, respectively. This indicated about 1.26, 1.31, 1.37, and 1.40 times higher lateral load carrying capacity for the building with plate thickness of 2.5 mm, 5 mm, 10 mm, and 15 mm in comparison to the building without SPSW.

As exposed in Figure 4.14 for Model 2 by changing plate thickness from 1 mm to 15 mm, also the behavior of structure has changed, after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate, the maximum base shear was equal to 2146 kN, 2183 kN, 2422 kN, 2514 kN, and 2539 kN, respectively while the maximum base shear for the building without plate was equal to 1242 kN. The behavior of strength of the steel building going well against lateral load about 1.72, 1.75, 1.95, 2.02, and 2.04 times higher, after strengthening by applied with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm in comparing with the origin building.

For Model 3 after looking at Figure 4.15, the maximum base shear for the building without SPSW was equal to 847 kN, also the maximum base shear was equal to 2986 kN, 3344 kN, 4412 kN, 4874 kN, and 5278 kN, after strengthened the structures by using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, respectively. This implied about 3.52, 3.94, 5.20, 5.75, and 6.23 times higher lateral load carrying capacity for the building after addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm as compared with normal building.

Another instance, according to Figure 4.16, the maximum base shear augmented from 1216 kN to 4153 kN, 4512 kN, 5025 kN, 5477 kN, and 5743 kN for Model 4, after applied steel plate for the building by thicknesses equal to 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively. The result illustrated that the behavior of structure improved and rose about 3.41, 3.71, 4.13, 4.50, and 4.72 times with the building without plate after addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively.

Furthermore, as seen in Figure 4.17, the maximum base shear was equal to 828 kN for Model 5 and without adding plate thickness, but with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the maximum base shear got up to 4039 kN, 4238 kN, 5410 kN, 6516 kN, and 6962 kN, respectively. The maximum base shear of the structure after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW was significantly higher than the structure without SPSW, about 4.87, 5.11, 6.53, 7.86, and 8.40 times increment was observed.

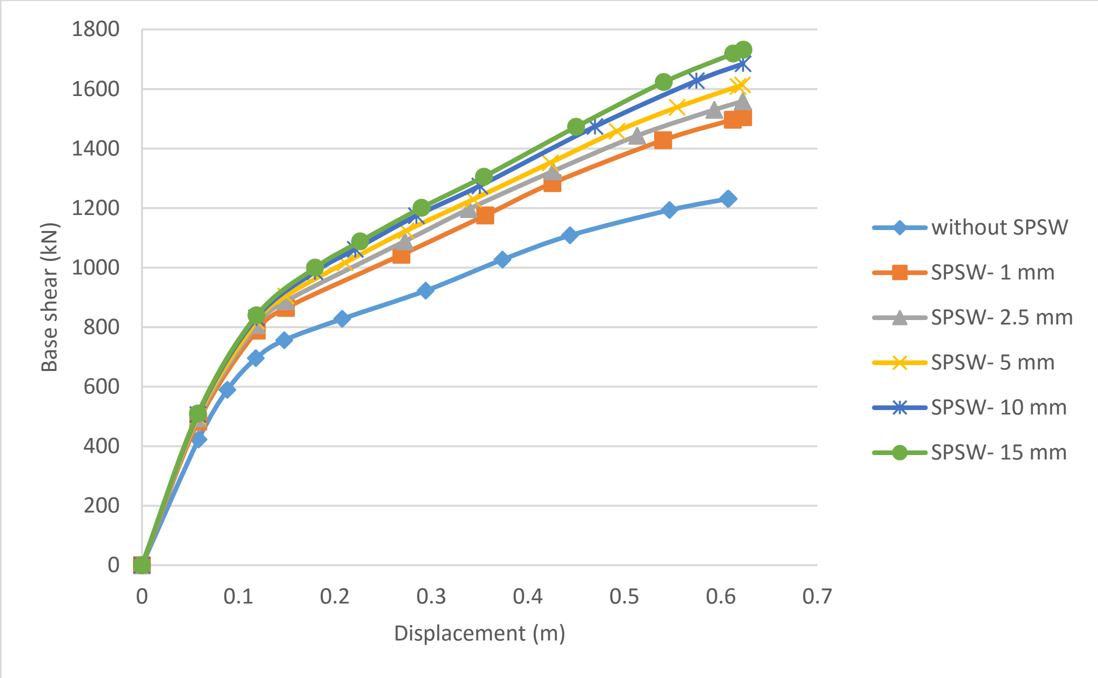


Figure 4.13 Pushover curves for bare model and model 1 with SPSWs having aspect ratio of 0.285 and different thicknesses

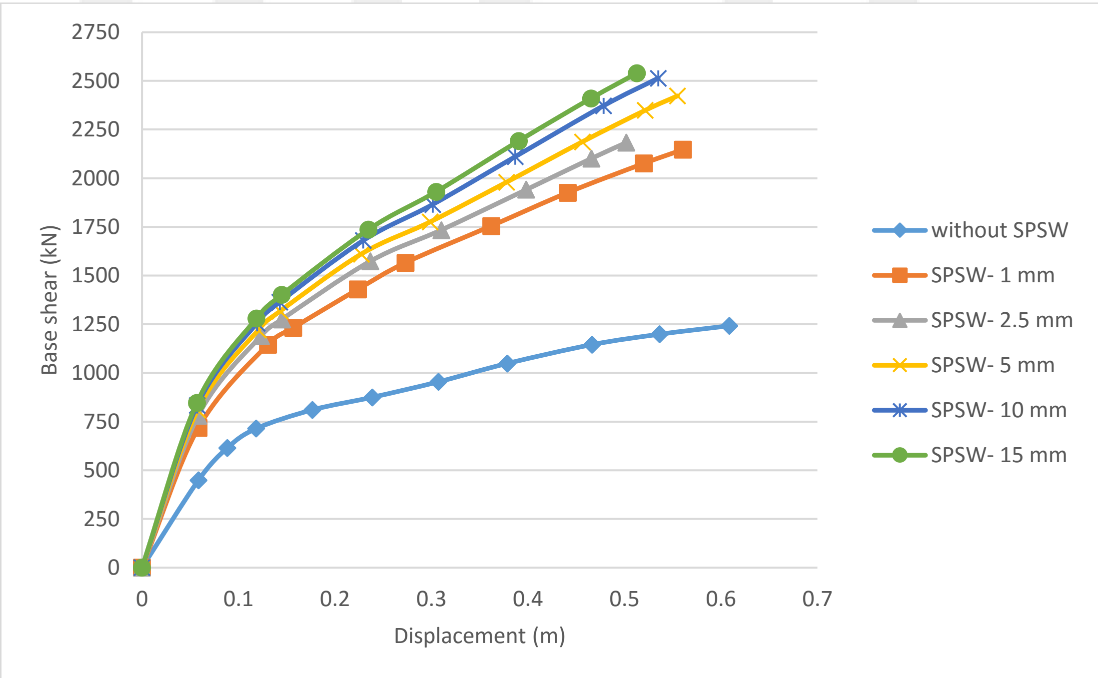


Figure 4.14 Pushover curves for bare model and model 2 with SPSWs having aspect ratio of 0.570 and different thicknesses

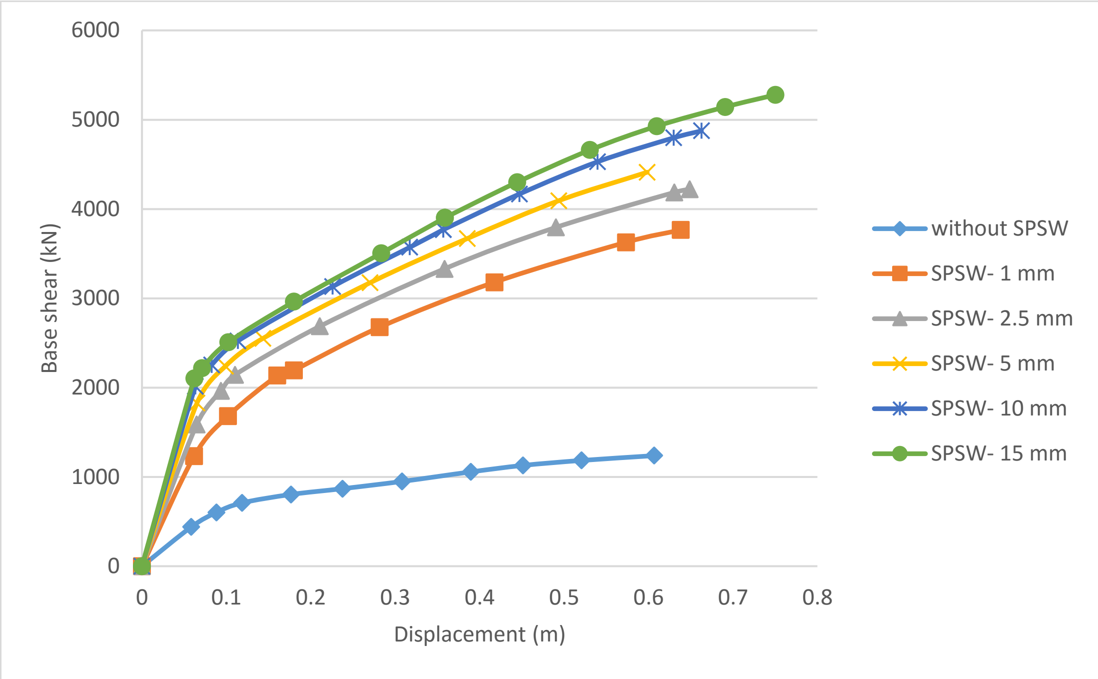


Figure 4.15 Pushover curves for bare model and model 3 with SPSWs having aspect ratio of 1.000 and different thicknesses

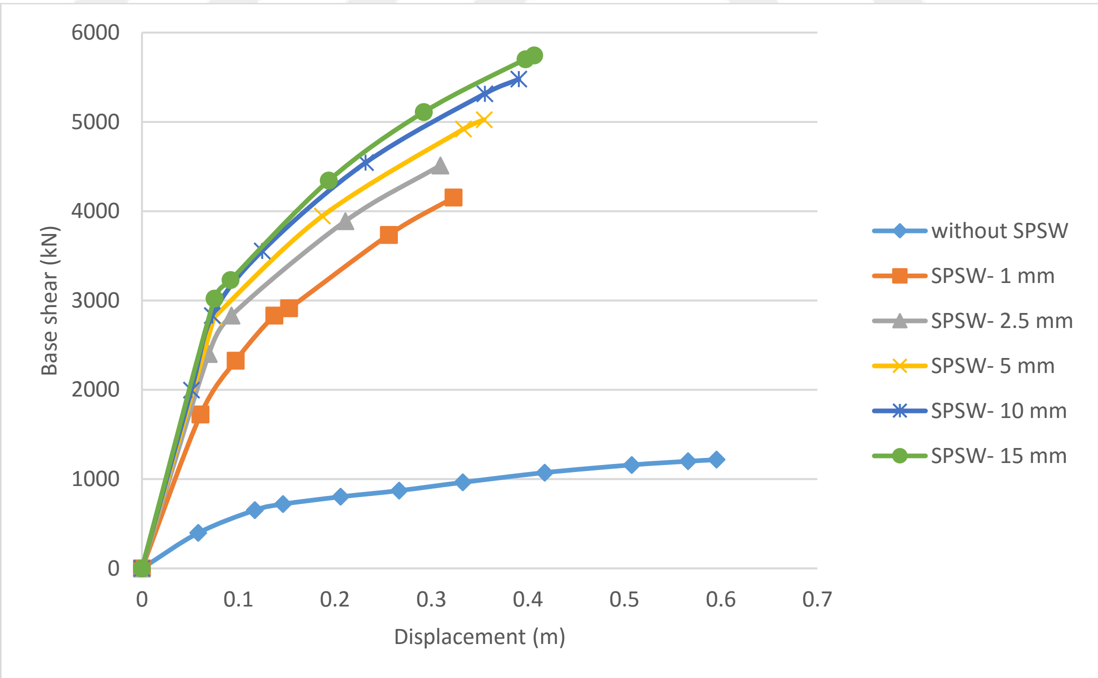


Figure 4.16 Pushover curves for bare model and model 4 with SPSWs having aspect ratio of 1.430 and different thicknesses

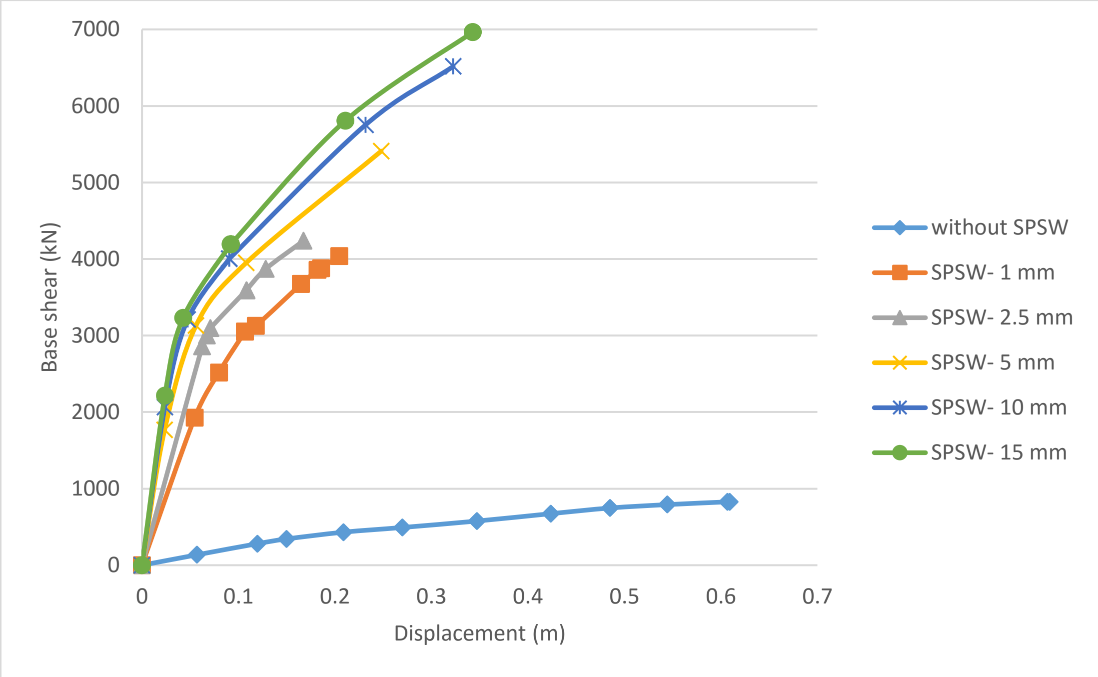


Figure 4.17 Pushover curves for bare model and model 5 with SPSWs having aspect ratio of 2.142 and different thicknesses

4.3.2 Initial stiffness with plate thickness

The effect of SPSWs on the initial stiffness of the 4-story steel building having different aspect ratios from 0.285 to 2.142 were studied considered different thicknesses of plate through the nonlinear pushover analysis. The capacity curves gave us valuable information on the stiffness characteristics of the examined structures. In order to see the influence of SPSW with different aspect ratios and thicknesses on the original stiffness of the building, the following plots given in Figures 4.18 - 4.22 were illustrated.

For example, for Model 1 as observed in Figure 4.18, the effect of the thickness of plate on initial stiffness for 4- story steel building with SPSWs having aspect ratio of 0.285 was very clear. The figure illustrated that the initial stiffness for the building without SPSW was equal to 7211 kN/m, but after using 1 mm steel plate shear walls, the initial stiffness increased to 8290 kN/m. This resulted in an increment of the initial stiffness about 1079 kN/m. This also implied about 1.15 times higher the initial stiffness for the building with plate thickness 1 mm in comparison to the building without SPSW. Also after using 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the initial stiffness went up to 8499 kN/m, 8622 kN/m, 8733 kN/m, and 8801 kN/m, respectively. The result illustrated that with plate thickness 2.5 mm, 5 mm, 10 mm, and 15 mm, the initial stiffness has risen about 1288 kN/m, 1411 kN/m, 1522 kN/m, and 1590 kN/m in comparison to the building without infill plate. This implied about 1.24, 1.27, 1.29, and 1.30 times higher the initial stiffness for the building with plate thickness 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively in comparison to the building without SPSW.

Furthermore, as shown in Figure 4.19 for Model 2, the initial stiffness was equal to 7678 kN/m before using SPSW, but this value ascended to 12292 kN/m, 13407 kN/m, 14048 kN/m, 14582 kN/m, and 14876 kN/m, while using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, respectively. In comparison of the initial stiffness between structures with infill plate and without infill plate, Figure 4.19 shows that it has augmented about 4614 kN/m, 5729 kN/m, 6370 kN/m, 6904 kN/m, and 7198 kN/m after addition of a 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, respectively.

Also, as seen in Figure 4.20, the initial stiffness was equal to 13507 kN/m, 17614 kN/m, 28033 kN/m, 31767 kN/m, and 33975 kN/m, they are achieved with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm individually, whilst it was equal to 3516 kN/m before supporting by infill plate. Result demonstrated that after reinforcing the building by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the initial stiffness was evaluated about 9991 kN/m, 14098 kN/m, 24517 kN/m, 28251 kN/m, and 30459 kN/m, respectively as compared to original building.

Figure 4.21 illustrates that the initial stiffness for the building without SPSW was equal to 6810 kN/m, but after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of steel plate shear walls, the initial stiffness increased to 28352 kN/m, 34671 kN/m, 37631 kN/m, 39009 kN/m, and 40193 kN/m, respectively for Model 4. The result illustrated that after addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate, the initial stiffness has risen about 21542 kN/m, 27861 kN/m, 30821 kN/m, 32199 kN/m and 33383 kN/m, respectively in comparison to the building before adding plate.

The value of the initial stiffness for Model 5 without SPSWs was about 2384 kN/m while supporting the building by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSWs this value sharply rose to 35257 kN/m, 45828 kN/m, 75444 kN/m, 87213 kN/m, and 93642 kN/m, respectively as seen in Figure 4.22. This means that the initial stiffness significantly risen about 32873 kN/m, 43444 kN/m, 73060 kN/m, 84829 kN/m and 91258 kN/m with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, respectively.

The results acquired during this study indicated that the initial stiffness grew approximately linearly by increasing the plate thickness and thus improved the ability and stability of the structure SPSW. All figures discovered that by changing the thickness of plate from 1 to 15 mm, the initial stiffness of the shear wall increased. Likewise, the initial stiffness due to high aspect ratio was being very high amount, when aspect ratio of 0.285, the initial stiffness was equal to 8290 kN/m for 1 mm of SPSW but by changing aspect ratio of 0.570 also the initial stiffness increased to 12292 kN/m for the same thickness. This implied about 1.48 times higher the initial stiffness for the building without SPSW. As well as, after looking for all figures discovered that for the same thickness of infill plate the value of the initial stiffness increase with increasing aspect ratio. The results showed that SPSW was more effective for structures having high aspect ratio than low aspect ratio.

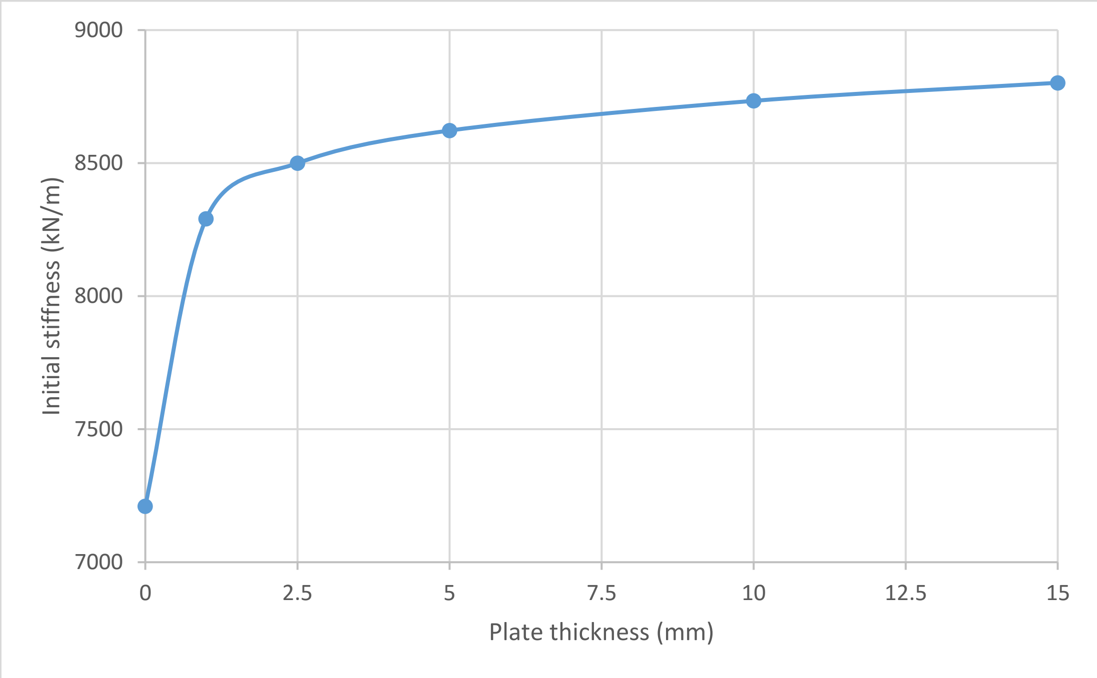


Figure 4.18 Effect of plate thickness on initial stiffness of 4-story steel building (Model 1) with and without SPSW

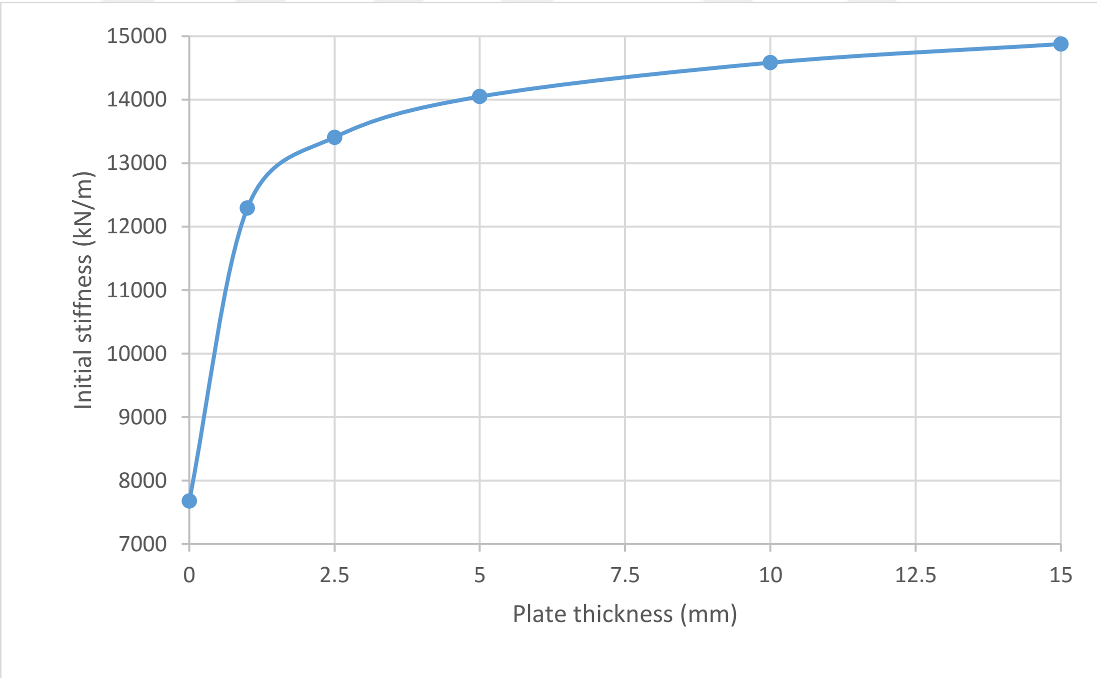


Figure 4.19 Effect of plate thickness on initial stiffness of 4-story steel building (Model 2) with and without SPSW

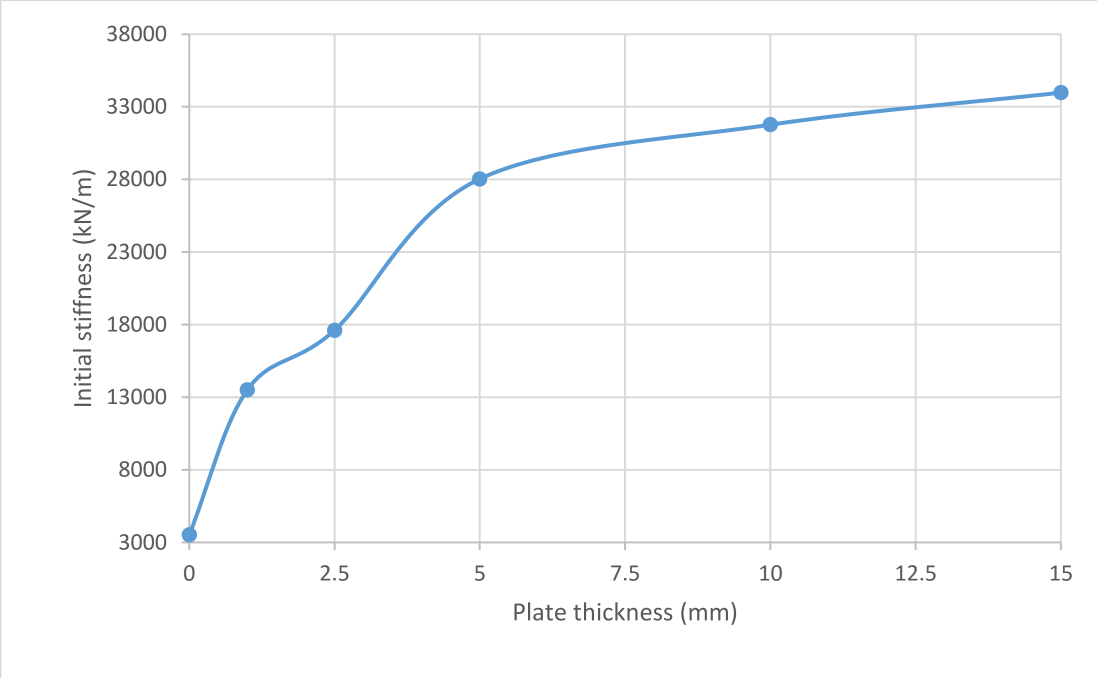


Figure 4.20 Effect of plate thickness on initial stiffness of 4-story steel building (Model 3) with and without SPSW

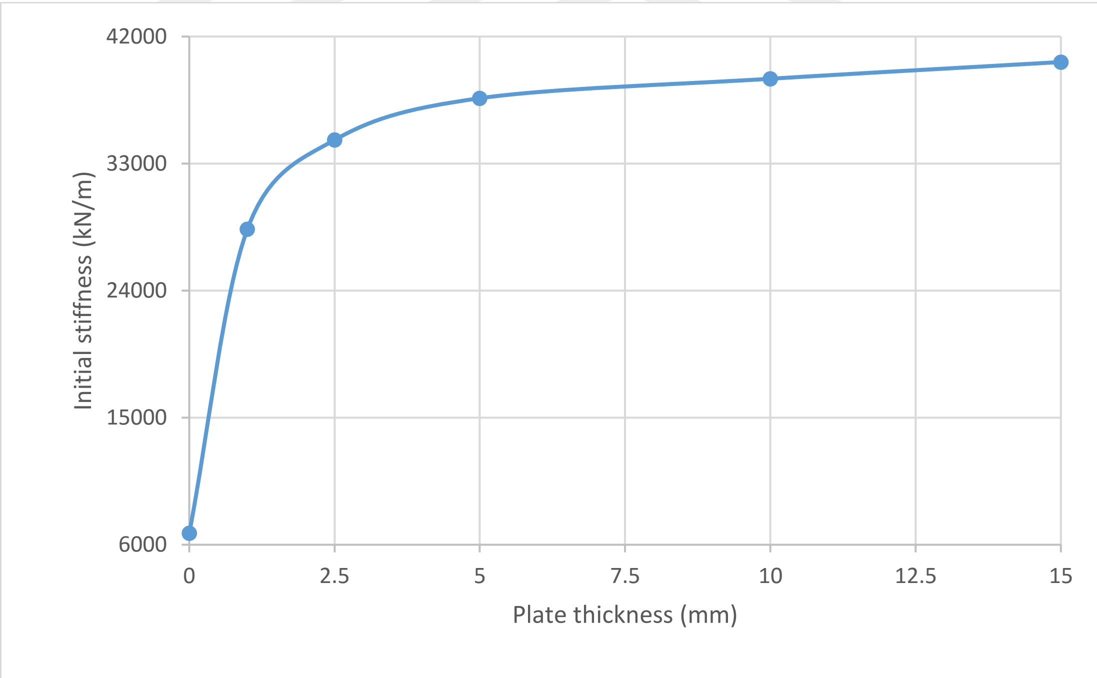


Figure 4.21 Effect of plate thickness on initial stiffness of 4-story steel building (Model 4) with and without SPSW

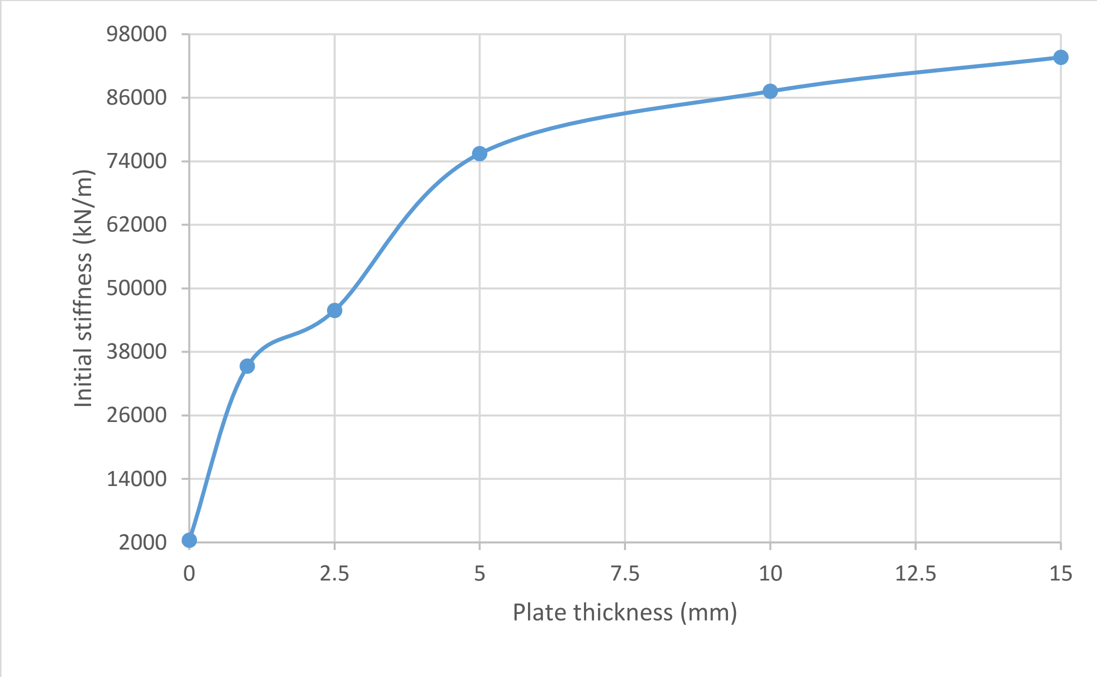


Figure 4.22 Effect of plate thickness on initial stiffness of 4-story steel building (Model 5) with and without SPSW

4.3.3 Ultimate strength with plate thickness

The ultimate strength values are obtained from the capacity curves of each models. In 4-story steel building illustrated as Model 1 – 5, by using SPSW having different thicknesses, the ultimate strength also changed. By increasing the plate thickness, the ultimate strength of the system also increased, demonstrating an enhancement in the overall behavior. This effect was clearly observed from the analysis of the results as given in Figures 4.23 – 4.27.

For Model 1, Figure 4.23 displays the ultimate strength for 4-story steel building without SPSW and after using SPSW having different thicknesses. This figure demonstrated that the ultimate strength was equal to 1230 kN for origin building, but after using with plate thickness 1 mm, the value of ultimate strength was calculated as 1505 kN. This meant that the ultimate strength increased about 275 kN in comparison to building without steel plate shear walls. Also after using 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate the ultimate strength was equal to 1558 kN, 1613 kN, 1684 kN, and 1731 kN, respectively. The result illustrated that after addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the ultimate strength has risen about 328 kN, 383 kN, 454 kN, and 501 kN in comparison to the building without SPSW, respectively.

For Model 2, as exposed in Figure 4.24, the ultimate strength for building without SPSW was equal to 1242 kN, but the ultimate strength determined as 2146 kN, 2183 kN, 2422 kN, 2514 kN, and 2539 kN after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, respectively. Supporting building by addition of using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm infill plate made the value of the ultimate strength greater about 904 kN, 941 kN, 1180 kN, 1272 kN, and 1297 kN as compared with building before adding plate.

For Model 3, Figure 4.25 gives the ultimate strength variation for 4-story building without SPSW and after using SPSW having different thicknesses. The result demonstrated that before supporting the building the ultimate strength was equal to 847 kN, but after supporting the steel building with plate thickness 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the ultimate strength increased to 2986 kN, 3344 kN, 4412 kN, 4874 kN, and 5278 kN, respectively. As well as, after strengthening the building by addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the ultimate strength has climbed about 1062 kN, 1244 kN, 1484 kN, 1760 kN, and 1822 kN in comparison to the origin building.

Figure 4.26 shows the ultimate strength variation Model 4 before and after using steel plate shear wall with different thicknesses. The result revealed that the ultimate strength was equal to 1216 kN but after using after using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of SPSW, the ultimate strength was equal to 4153 kN, 4512 kN, 5025 kN, 5477 kN, and 5743 kN, respectively. The result illustrated that after supporting the steel building by 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm of infill plate the ultimate strength has risen about 2937 kN, 3296 kN, 3809 kN, 4261 kN, and 4527 kN, respectively as comparing with normal building.

Another instance, as seen in Figure 4.27, the ultimate strength for Model 5 was about 4039 kN, 4238 kN, 5410 kN, 6516 kN, and 6962 kN in the cases of using 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm steel plate shear wall, respectively. While, before reinforcing the origin building by steel plate shear wall, the ultimate strength value was equal to 828 kN. The obtained results identified that after reinforcing origin building by addition of 1 mm, 2.5 mm, 5 mm, 10 mm, and 15 mm, the ultimate strength has increased about 3211 kN, 3410 kN, 4582 kN, 5688 kN, and 6134 kN, respectively comparing with steel building before supporting by infill plate.

It was clearly understood that for all models the ultimate strength of the building significantly grew after supporting the origin building by SPSW, also results confirmed that by increasing the plate thickness, the ultimate strength of the system was improvement. In addition, looking in figures discovered that the infill plate was more effective for building having large aspect ratio.

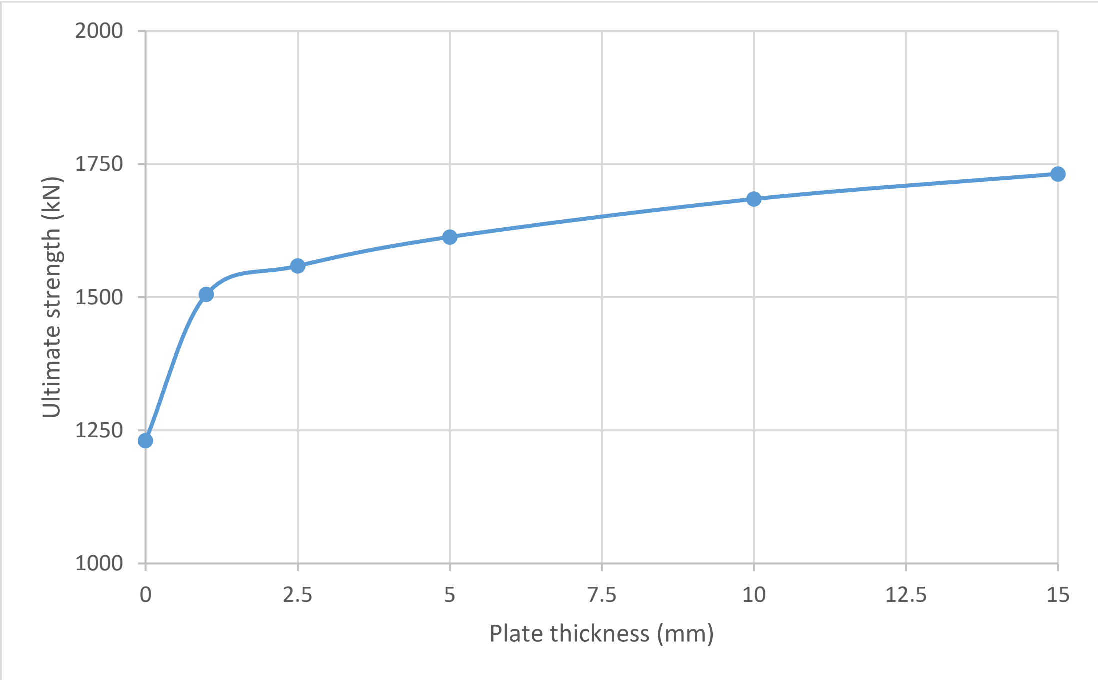


Figure 4.23 Effect of plate thickness on ultimate strength of 4-story steel building (Model 1) with and without SPSW

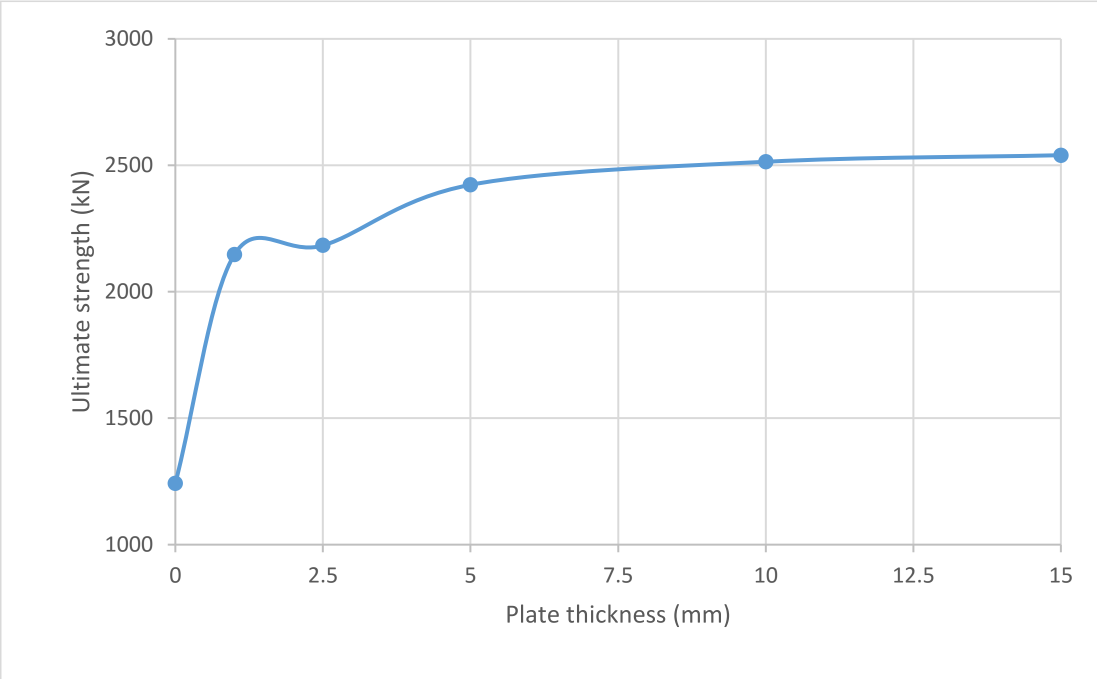


Figure 4.24 Effect of plate thickness on ultimate strength of 4-story steel building (Model 2) with and without SPSW

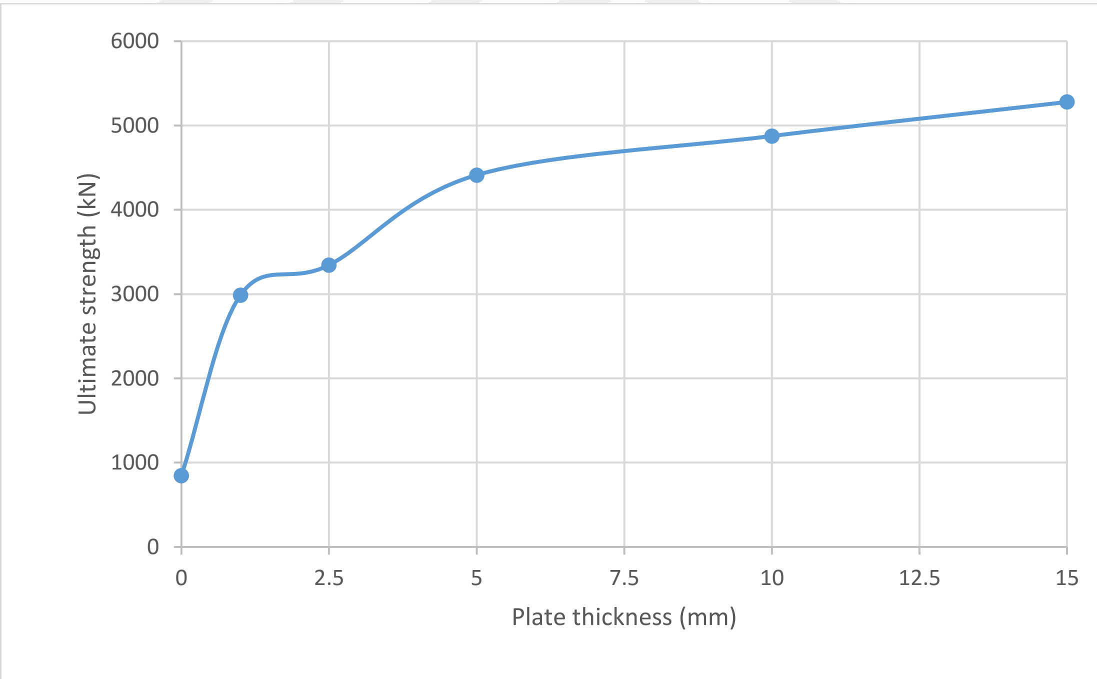


Figure 4.25 Effect of plate thickness on ultimate strength of 4-story steel building (Model 3) with and without SPSW

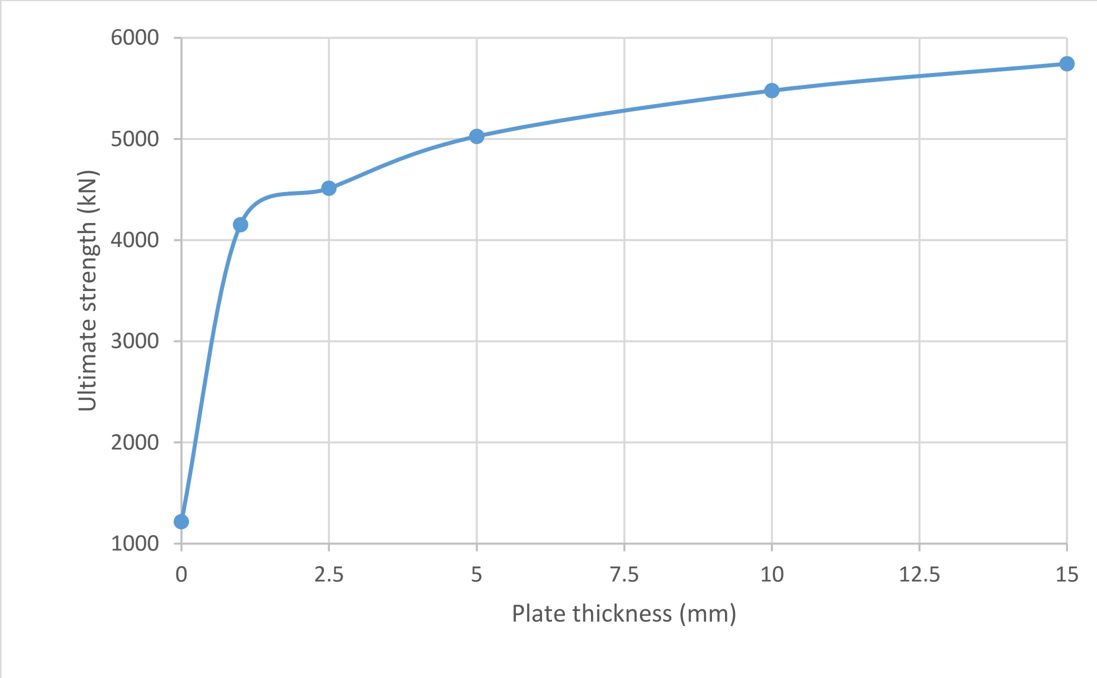


Figure 4.26 Effect of plate thickness on ultimate strength of 4-story steel building (Model 4) with and without SPSW

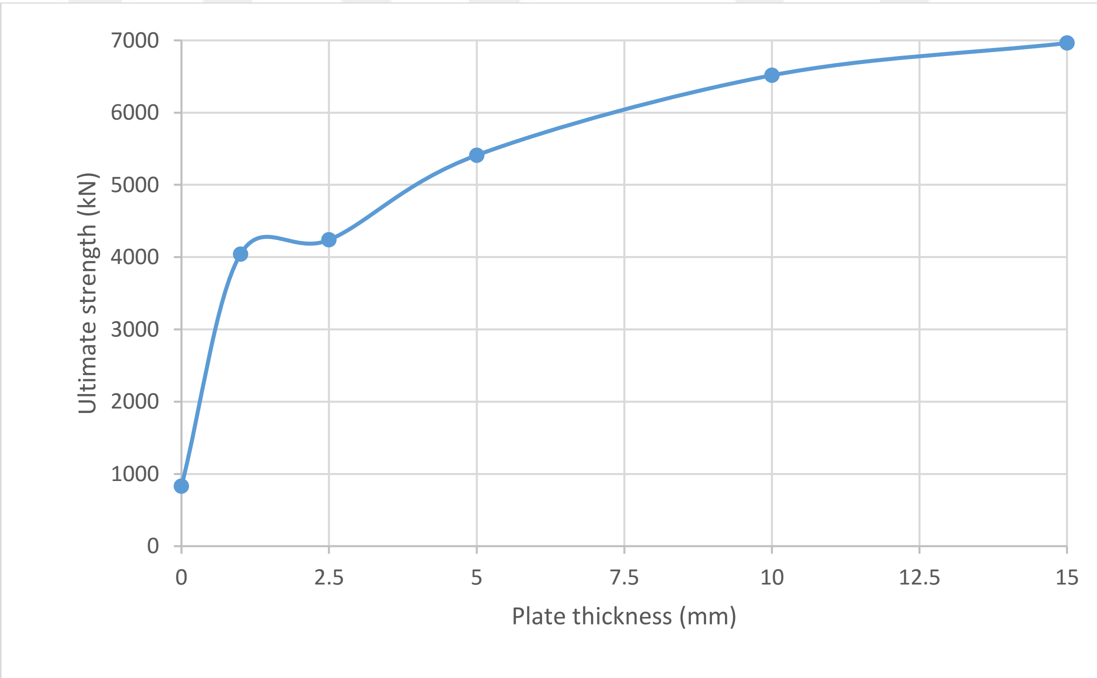


Figure 4.27 Effect of plate thickness on ultimate strength of 4-story steel building (Model 5) with and without SPSW

4.3.4 Effect of aspect ratio on pushover response

The aspect ratio of the SPSW system, which is the ratio of the panel width to the panel height, is an important feature in the design of the wall. In order to investigate the effect of the aspect ratio on the overall performance of the structure with wall system, the story's height is held constant and only the width and thickness of the infill plates vary.

For this purpose, five different aspect ratios studied with five models. Model 1 had steel plate shear walls having the aspect ratio of 0.285, Model 2 contained steel plate shear walls having the aspect ratio of 0.570, Model 3 composed of steel plate shear walls having the aspect ratio of 1.000, Model 4 had steel plate shear walls having the aspect ratio of 1.430, and Model 5 included steel plate shear walls having the aspect ratio of 2.142.

Figure 4.28 shows the effect of the aspect ratio on the capacity curve with using plate thickness 1 mm, the figure illustrated that the maximum base shear was equal to 1505 kN for Model 1 and was equal to 2146 kN, 3765 kN, 4153 kN, and 4039 kN for Model 2, Model 3, Model 4, and Model 5, respectively. The result demonstrated that the SPSW was more effect in Models 4, and 5 where the value of aspect ratio was larger compared to the others.

Model 1 - 5 as displayed in Figure 4.29, based on models, the amount of the base shear changed with changing aspect ratio under the constant value of 2.5 mm infill plate. For example, the maximum base shear for Model 1, Model 2, Model 3, Model 4, and Model 5 was equal to 1558 kN, 2183 kN, 4222 kN, 4512 kN, and 4238 kN, respectively. Similarly, to the previous figure, after looking at this curve identified the performance of the steel building improved with increasing value of aspect ratio.

Appling 5 mm of steel plate shear wall of the origin structure as exposed in Figure 4.30, the maximum base shear was equal to 1613 kN , 2422 kN, 4412 kN, 5025 kN, and 5410 kN, for Model 1, Model 2, Model 3, Model 4, and Model 5, respectively. Governed by the highest value of the base shear was equal to 5410 kN, this is obtained when the aspect ratio was equal to 2.142 and this is largest aspect ratio in comparison to the other.

The maximum base shear for Model 1, Model 2, Model 3, Model 4, and Model 5 was equal to 1684 kN, 2514 kN, 4874 kN, 5477 kN, and 6516 kN, respectively. The smallest value come from Model 1 when aspect ratio was equal to 0.285 which is the smallest aspect ratio and with rising aspect ratio for the same infill plate the maximum base shear increased, finally got the maximum value from Model 5 which is the maximum aspect ratio. That information is given in Figure 4.31 for 10 mm thickness of SPSW.

Furthermore, as seen in Figure 4.32, it was found that the effect of aspect ratio on the capacity curve with plate thickness 15 mm was very clear and illustrated that the maximum base shear was equal to 1731 kN , 2539 kN, 5278 kN, 5743 kN, and 6962 kN for Model 1, Model 2, Model 3, Model 4, and Model 5, respectively. Noting information given in Figure 4.32, it was observed that Model 5 had the largest value of the maximum base shear of 6962 kN, also Model 1 had the smallest value of the maximum base shear of 1731 kN.

Finally, according to the figures, the results indicated that the influence of aspect ratio on the behavior of the structure with SPSW was significant. In other words, all figures demonstrated that if the amount of steel plate and plate thickness remained constant, then aspect ratio had an important effect on the behavior of a system. In that situation, the most important to use SPSW in longer spans to avoid having big-size columns.

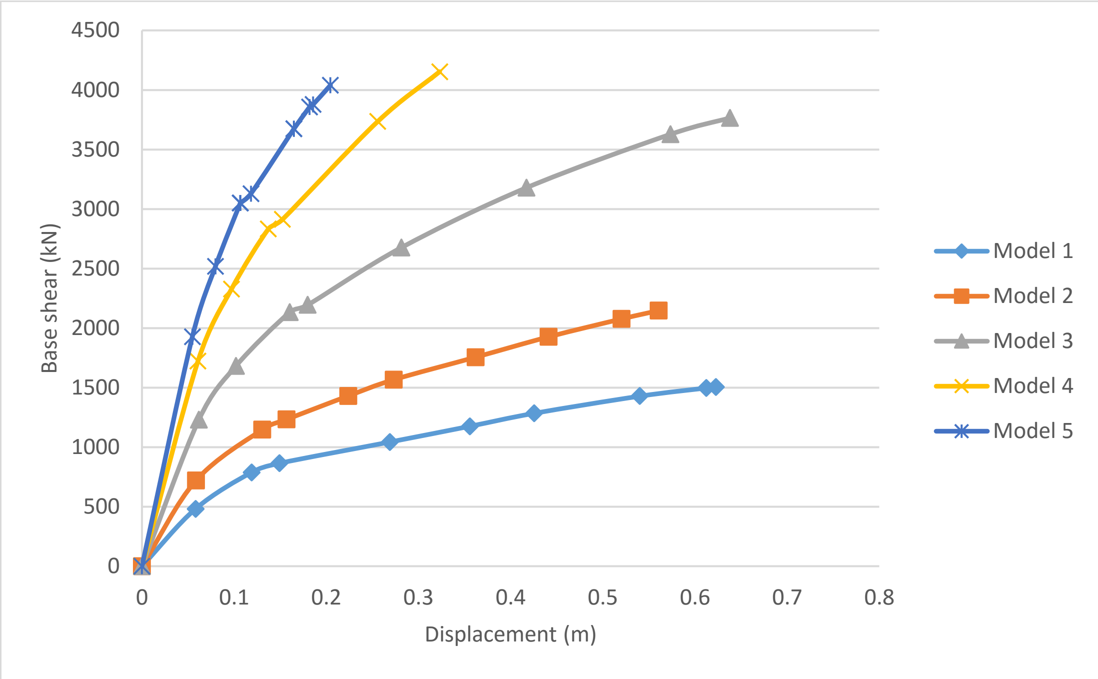


Figure 4.28 Figure 4.28 Effect aspect ratio on capacity curves of 4-story steel building (all models) with 1 mm steel plate shear wall

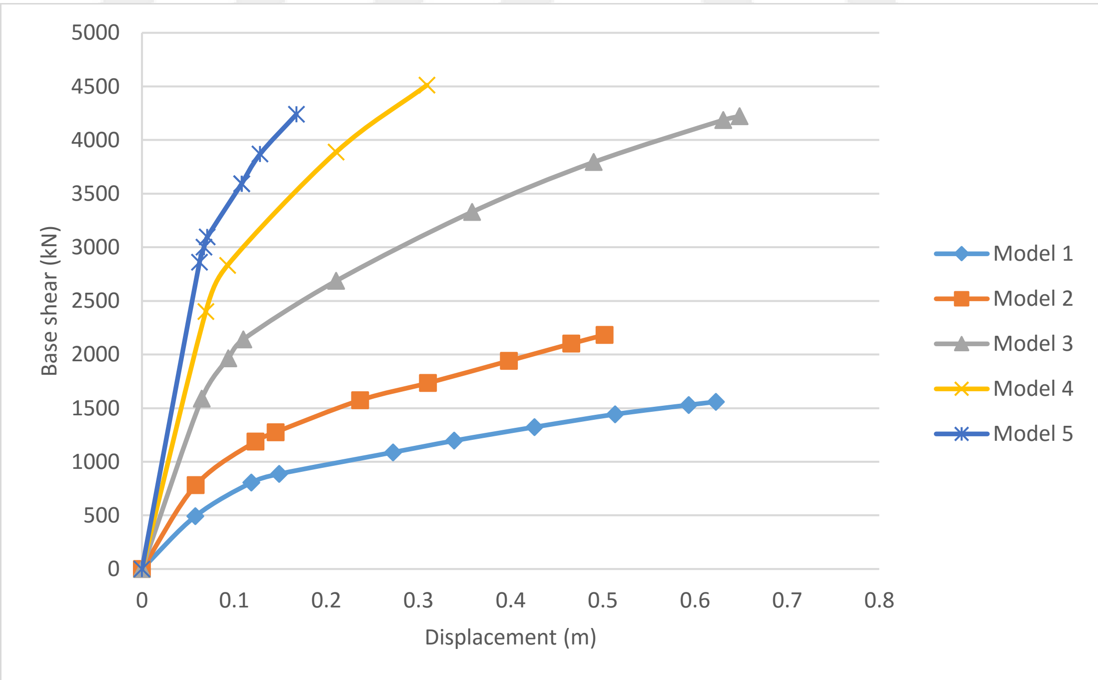


Figure 4.29 Effect aspect ratio on capacity curve of 4-story steel building (all models) with 2.5 mm steel plate shear wall

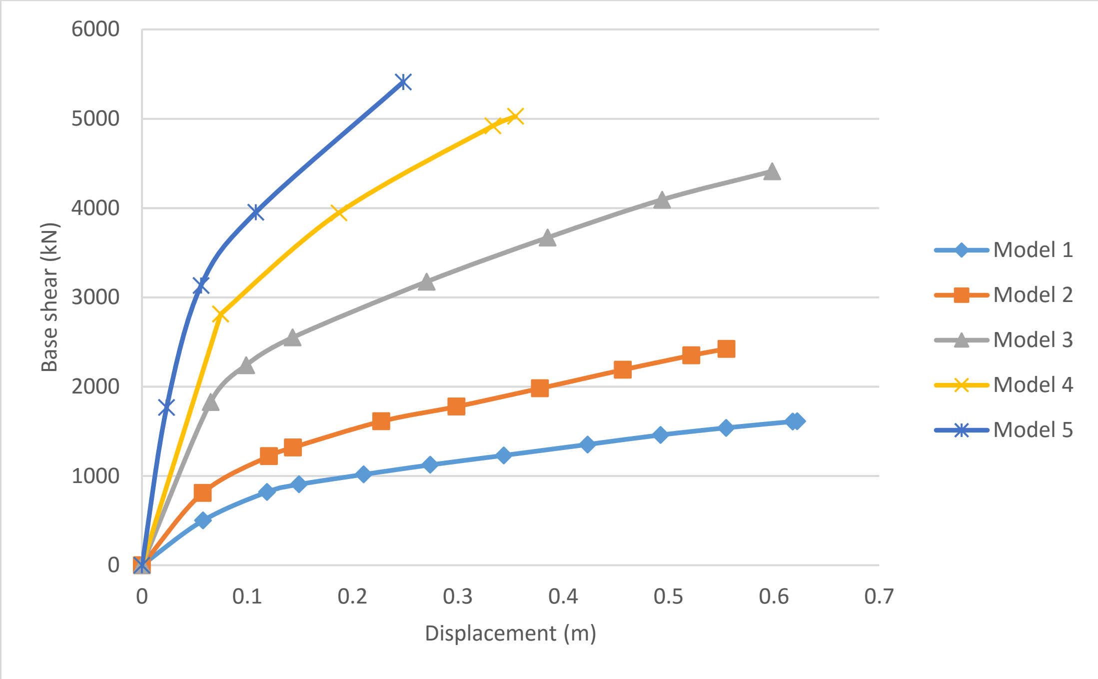


Figure 4.30 Effect aspect ratio on capacity curve of 4-story steel building (all models) with 5 mm steel plate shear wall

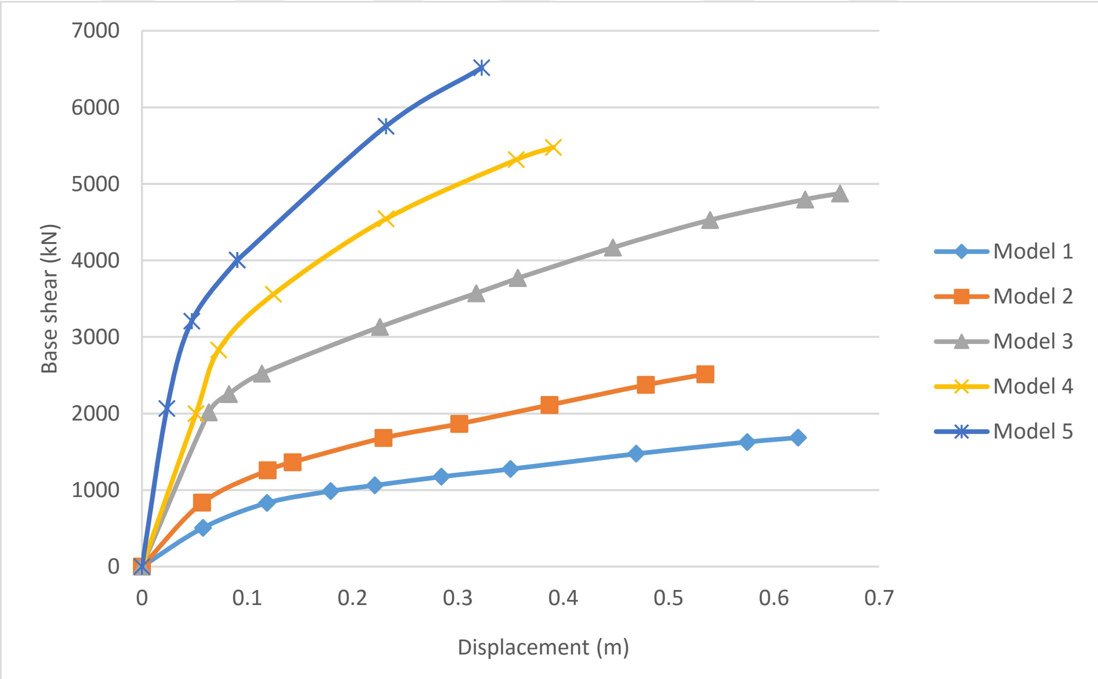


Figure 4.31 Effect aspect ratio on capacity curve of 4-story steel building (all models) with 10 mm steel plate shear wall

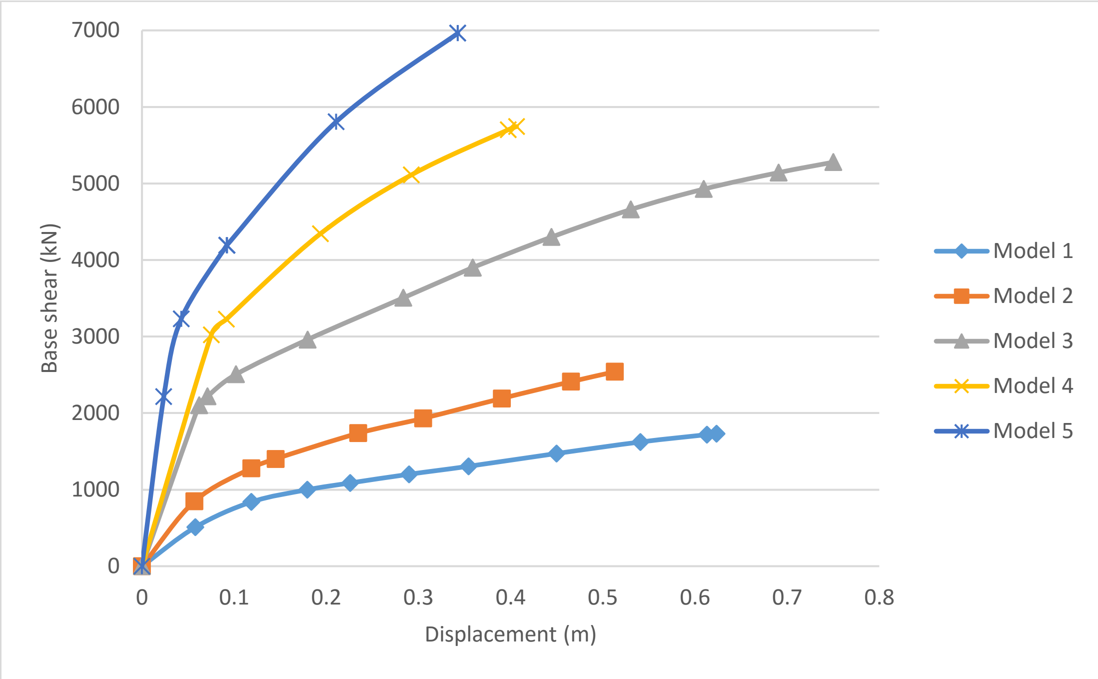


Figure 4.32 Effect aspect ratio on capacity curve of 4-story steel building (all models) with 15 mm steel plate shear wall

4.3.5 Effect of aspect ratio on strength and stiffness

In this study, five different models with five different aspect ratios were used as mentioned before. Figures 4.33 - 4.37 illustrates the effect of aspect ratio on both of ultimate strength and initial stiffness. For variation steel plate thicknesses of 1 to 15 mm. In general, it was pointed out that mostly steel plate shear wall was more effective with large span in compassion to small span. In other words, increasing the aspect ratio caused considerable enhancement not only in the strength but also in the stiffness of the case study structure.

Figure 4.33 (a and b) demonstrates the effect of aspect ratio on ultimate strength and initial stiffness of the structure with plate thickness 1 mm. As seen in Figure 4.33 (a), the ultimate strength was equal to 1505 kN when aspect ratio was equal to 0.285, but by increasing aspect ratio to 0.570, also the ultimate strength became 2146 kN. Also by rising aspect ratio to 1.000, 1.430, and 2.142, the ultimate strength calculated as 3765 kN, 4153 kN, and 4039 kN, respectively. In addition, as observed in Figure 4.33 (b), the initial stiffness was equal to 8290 kN/m when aspect ratio was equal to 0.285, but by increasing aspect ratio to 0.570 also initial stiffness increased to 12292 kN/m.

Also by rising aspect ratio to 1.000, 1.430, and 2.142, the initial stiffness increased to 19929 kN/m, 28352 kN/m, and 35257 kN/m, respectively.

Both of ultimate strength and initial stiffness for all models illustrated in Figure 4.34 (a and b) after addition of 2.5 mm infill plate. Depending on Figure 4.34 (a) when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, the ultimate strength corresponded to 1558 kN, 2183 kN, 4222 kN, 4512 kN, and 4238 kN, respectively. On the other hand, the initial stiffness was equal to 8499 kN/m, 13407 kN/m, 24484 kN/m, 34671 kN/m, and 45828 kN/m, when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, respectively as given in Figure 4.34 (b).

In Figure 4.35 (a and b), the variation in the initial stiffness and ultimate strength with the aspect ratio for the to 5 mm thickness of SPSW is given. For example, as exposed in Figure 4.35 (a) when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, the ultimate strength was equal to 1613 kN, 2422 kN, 4412 kN, 5025 kN, and 5410 kN, respectively after supporting building by 5 mm of infill plate. Moreover, as demonstrated in Figure 4.35 (b), the initial stiffness was equal to 8622 kN/m, 14048 kN/m, 28033 kN/m, 37631 kN/m, and 75444 kN/m when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, respectively.

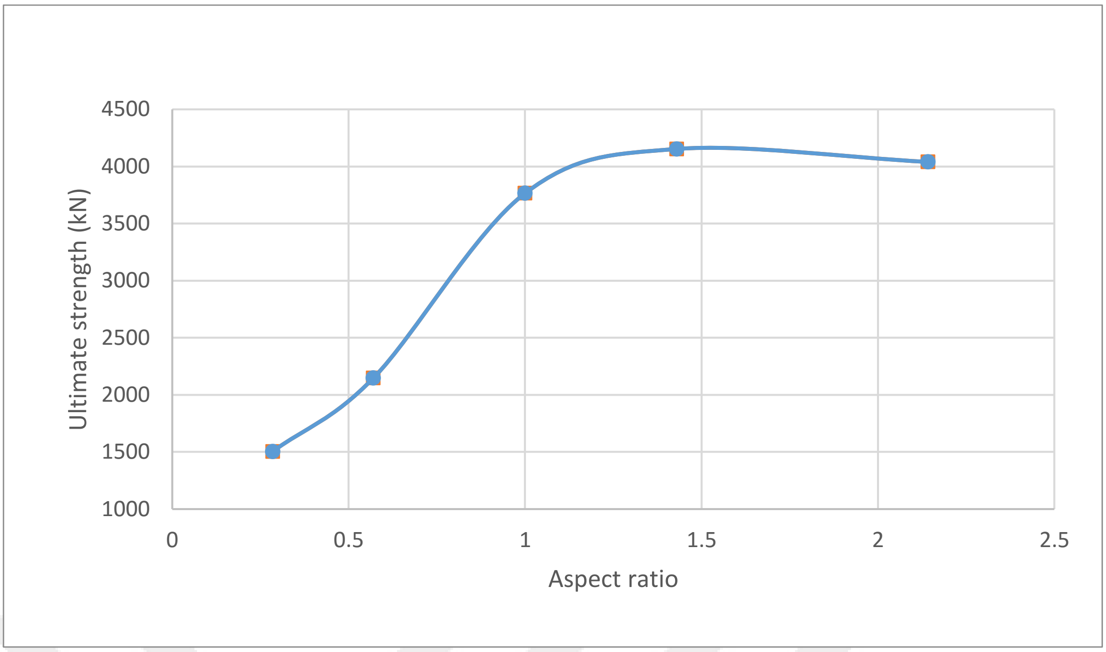
Another instance, as exposed in Figure 4.36 (a and b) for the structure with plate thickness 1 mm, the ultimate strength was equal to 1684 kN, 2514 kN, 4874 kN, 5477 kN, and 6516 kN, also the initial stiffness was equal to 8733 kN/m, 14582 kN/m, 31767 kN/m, 39009 kN/m, and 87213 kN/m, while aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, respectively.

Figure 4.37 (a and b) illustrates the effect of the aspect ratio on ultimate strength and initial stiffness for 15 mm thickness of SPSW. For example, Figure 4.37 (a) shows that the ultimate strength was equal to 1731 kN, 2539 kN, 5278 kN, 5743 kN, and 6962 kN when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, respectively. Also, Figure 4.37 (b) demonstrates that the initial stiffness was equal to 8801 kN/m, 14876 kN/m, 33975 kN/m, 40193 kN/m, and 93642 kN/m when aspect ratio was equal to 0.285, 0.570, 1.000, 1.430, and 2.142, respectively.

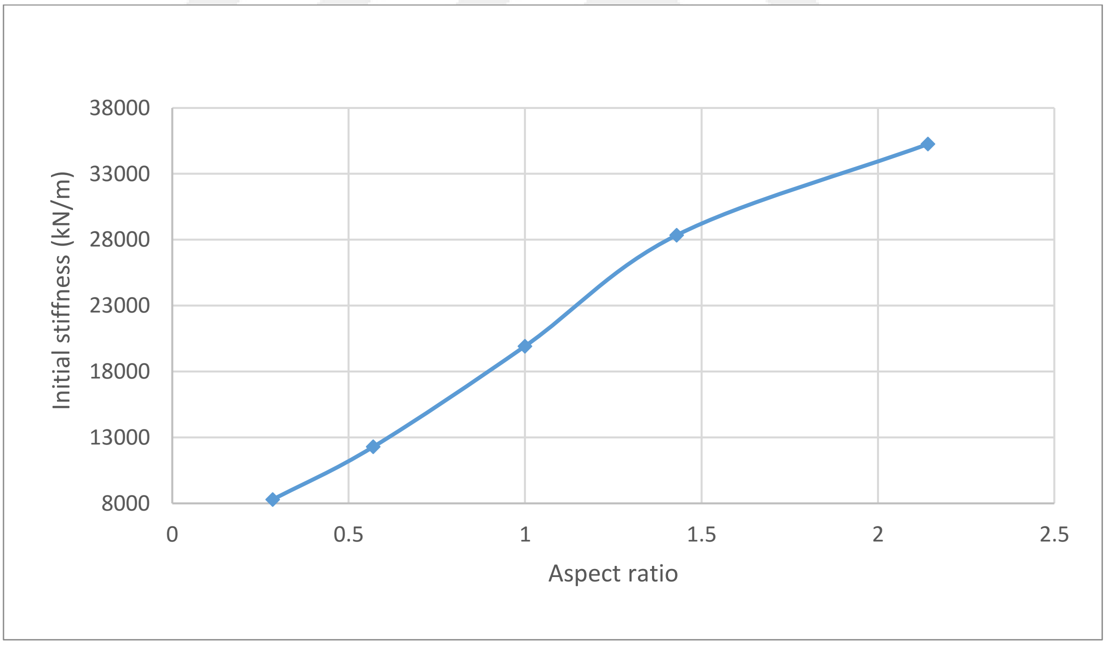
The figures clearly showed that the initial stiffness increased almost linearly by increasing the steel plate thickness and hence the stability capacity of the structure with

SPSW is improved. The analysis of the result demonstrated that by changing the aspect ratios from 0.285 to 2.142, the initial stiffness of the shear wall rose for the constant plate thickness. The initial stiffness nearly was equal to 35257 kN/m for aspect ratio of 0.285, but by increasing aspect ratio to 2.142 the initial stiffness also increased approximately to 93642 kN/m for 15 mm infill plate. Looking for each figure demonstrated for the same SPSW the initial stiffness increased with increasing aspect ratio.

According to the findings obtained for the ultimate strength that, mostly after supporting the origin building by SPSW the value of the ultimate strength has risen at the same times. Figures showed that for the constant plate thickness, the ultimate strength had a tendency to increase by changing the aspect ratio from 0.285 to 2.142. The result indicated that the ultimate strength for Model 1 and 2 was close to each other after using 1 mm and 2.5 mm of plate thickness, but when aspect ratio was equal to 1.430 and 2.142 with plate thickness 5 mm, 10 mm, and 15 mm, significantly strength increased. For all models except Model 1 and Model 2 the maximum strength linearly increased with increasing the aspect ratio of steel plate shear wall.

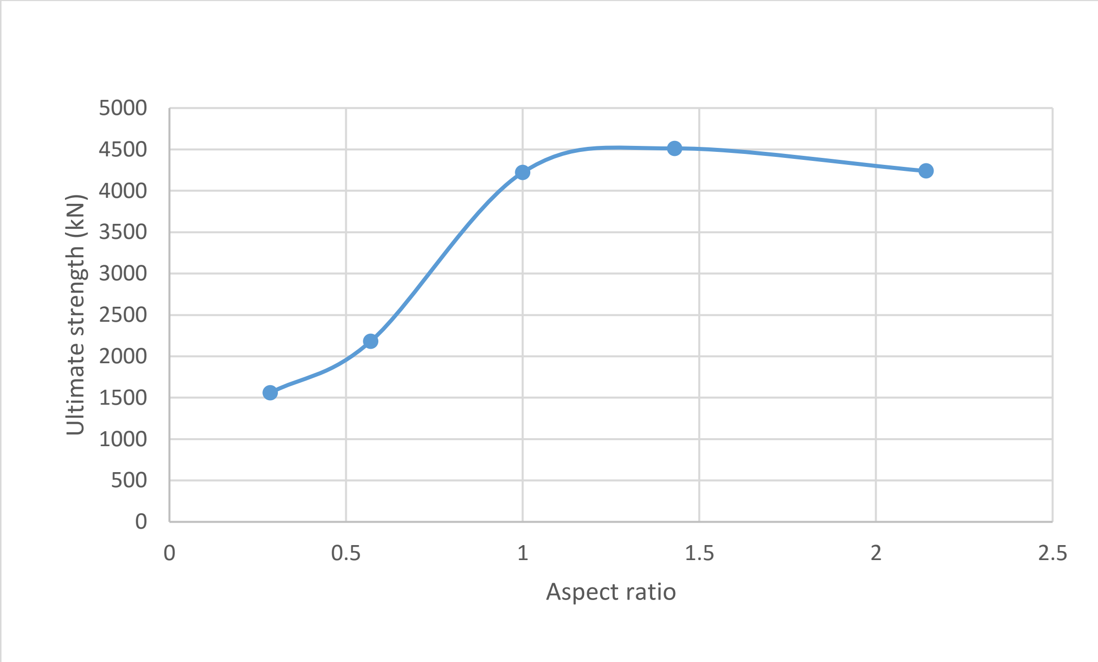


(a)

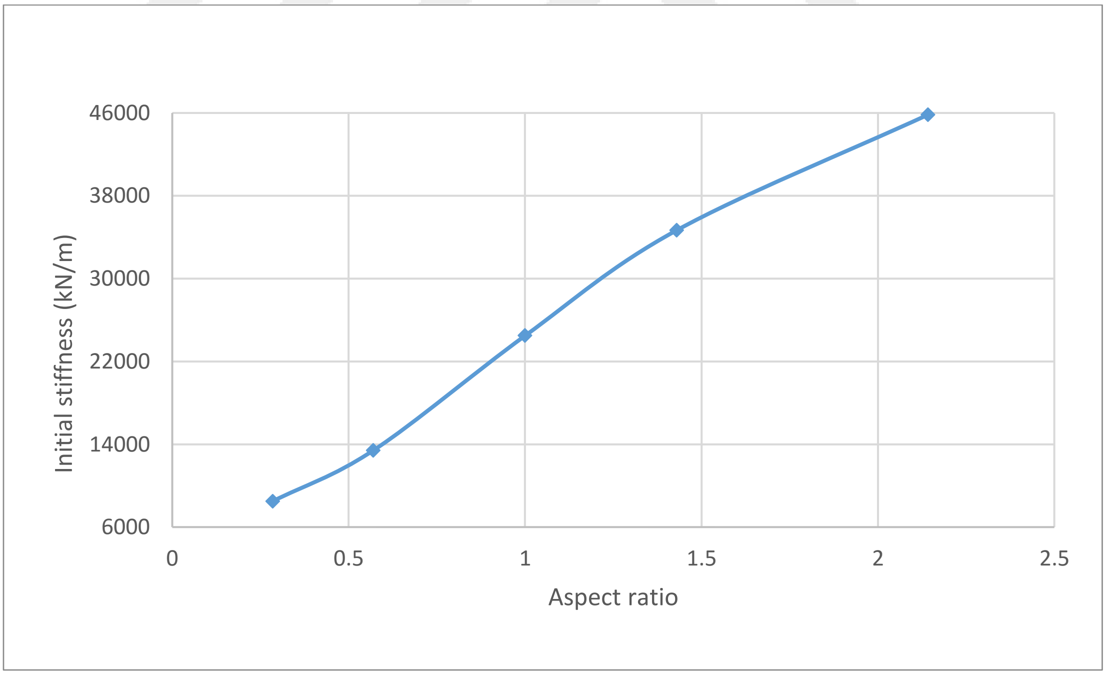


(b)

Figure 4.33 Effect aspect ratio on strength and stiffness of 4-story steel building (all models) with 1mm plate thickness (a: ultimate strength, b: initial stiffness)

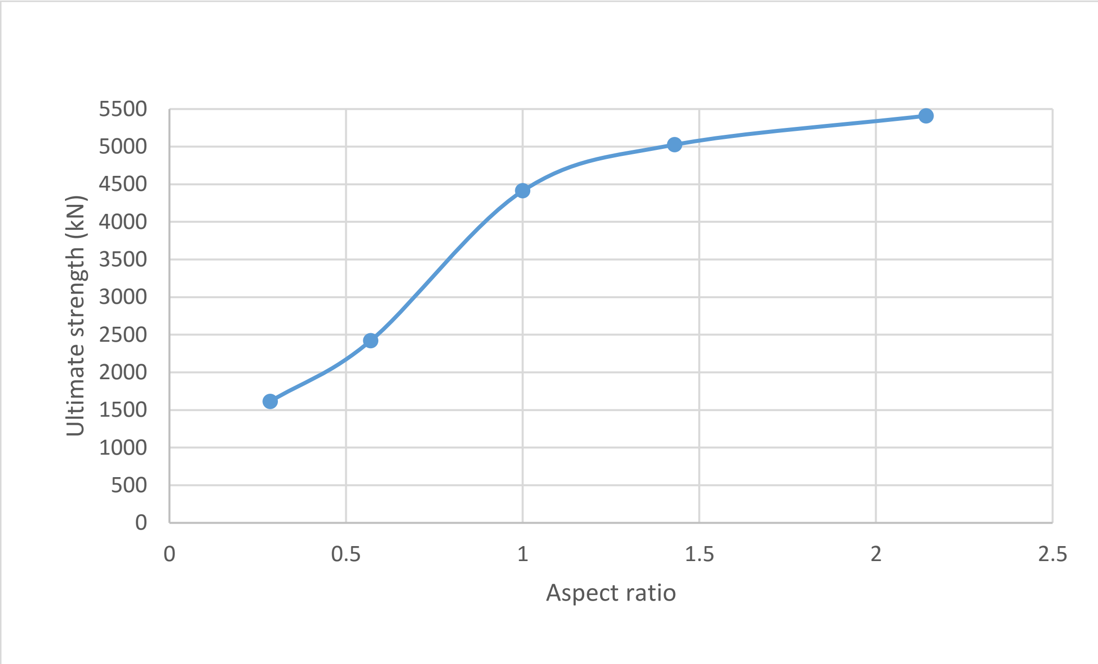


(a)

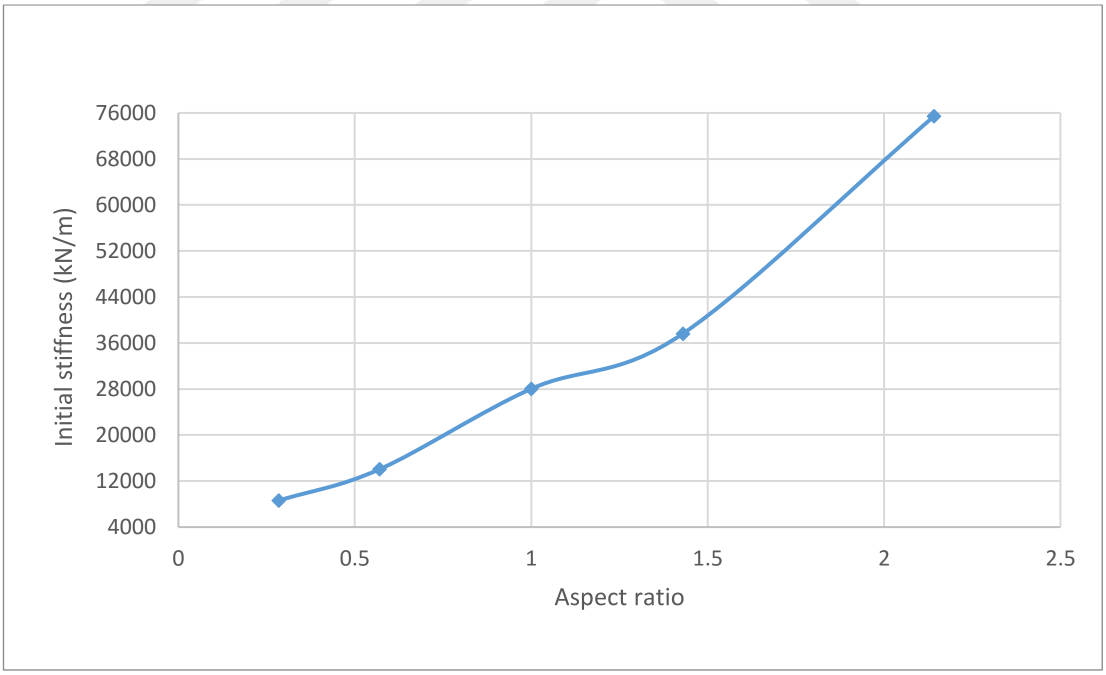


(b)

Figure 4.34 Effect aspect ratio on strength and stiffness of 4-story steel building (all models) with 2.5mm plate thickness (a: ultimate strength, b: initial stiffness)

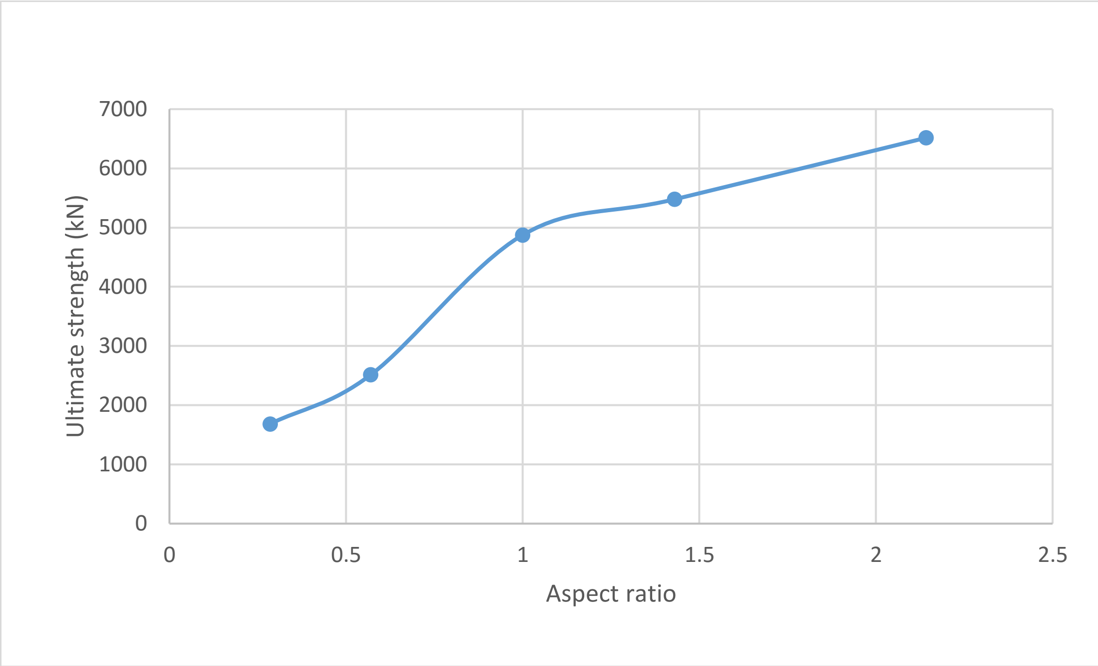


(a)

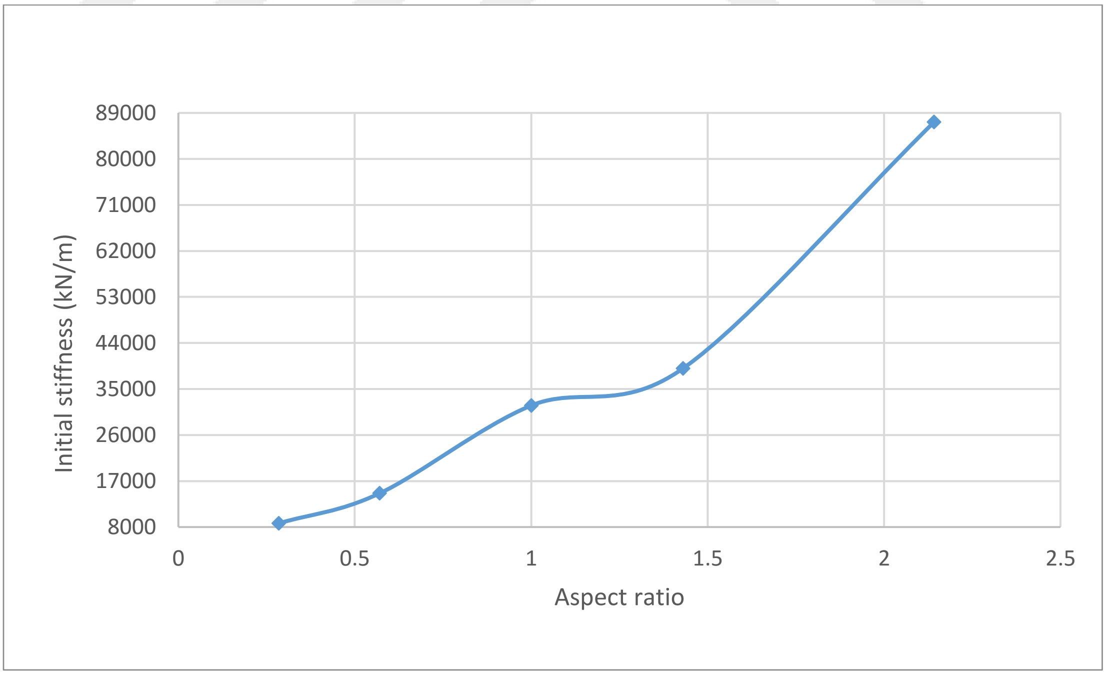


(b)

Figure 4.35 Effect aspect ratio on strength and stiffness of 4-story steel building (all models) with 5mm plate thickness (a: ultimate strength, b: initial stiffness)

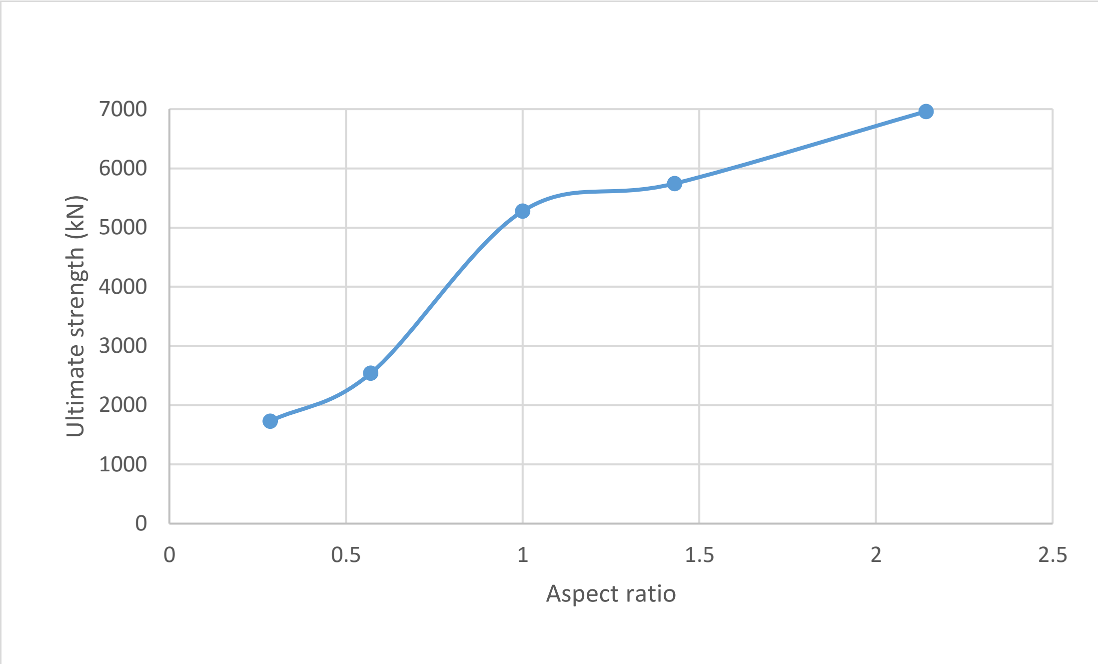


(a)

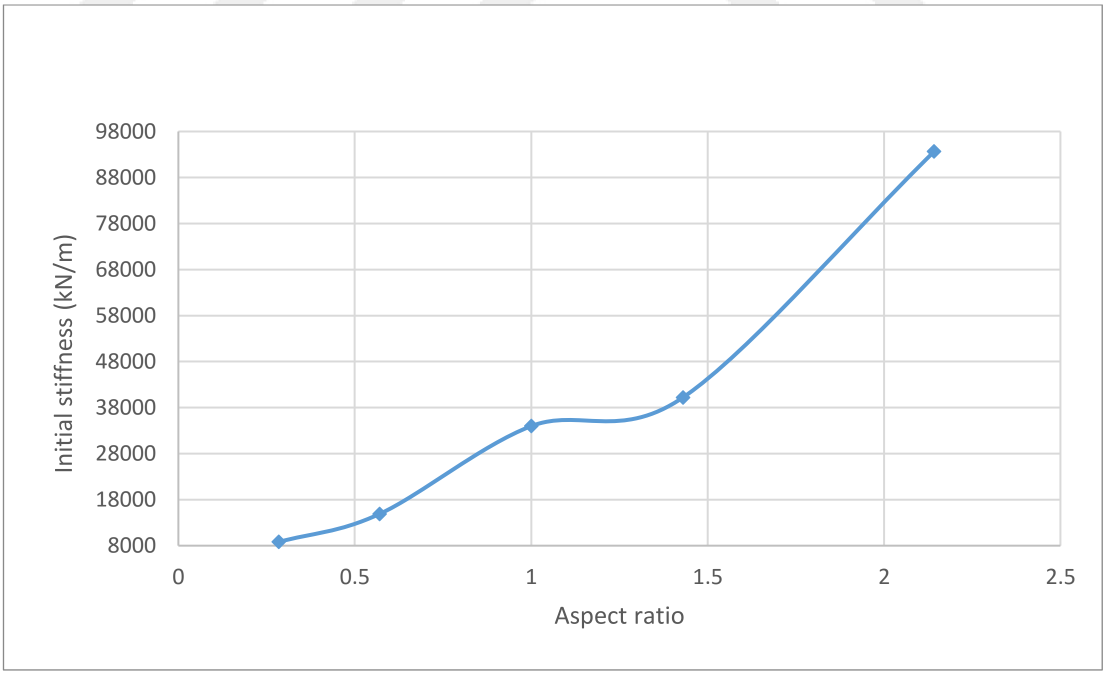


(b)

Figure 4.36 Effect aspect ratio on strength and stiffness of 4-story steel building (all models) with 10mm plate thickness (a: ultimate strength, b: initial stiffness)



(a)



(b)

Figure 4.37 Effect aspect ratio on strength and stiffness of 4-story steel building (all models) with 15mm plate thickness (a: ultimate strength, b: initial stiffness)

CHAPTER 5

CONCLUSIONS

The effectiveness of steel plate shear walls (SPSWs) for upgrading the seismic behavior of the steel structures was investigated as a case study. The performance properties were assessed based on the nonlinear pushover analysis considering the two cases. The investigated parameters covered the steel building with different number of story, aspect ratio and thickness of SPSWs. Comparing the performance of the structures with and without SPSWs having different properties, the following conclusions can be made:

- The results from the capacity curves obtained that the base shear, which is the ability of the structure to resist lateral loads, was remarkably improved in the case of the frames due to the use of SPSWs. The buildings with steel plate shear walls had the significantly greater load carrying capacity as compared to the original buildings. By changing the thickness of plate from 1 to 15 mm, the capacity curve also increased directly.
- The capacity curves for the (2, 4, 6, and 8-story steel buildings) indicated that the building with 1 mm of SPSW had about 4.06, 3.52, 2.36, and 2.14 times higher load carrying capacity than the building without SPSW, respectively. In the case of 4-story steel building having different aspect ratios of 0.285, 0.570, 1.000, 1.430, and 2.142, these values were 1.22, 1.72, 3.52, 3.41, and 4.87 for the same SPSW thickness. For all other thicknesses of plate, approximately the same improvement was observed.
- From the results of nonlinear pushover analysis, it was pointed out that the plate thickness of the shear wall was the most effective on the performance of the frame structures and behavior of the SPSW system. It was clearly understood that the initial stiffness and the ultimate strength were increased almost linearly by increasing steel plate thickness and hence the stability capacity of steel plate

shear walls improved. For all models, with increasing of plate thickness both of stiffness and strength increased.

- Analysis of the results indicated that the initial stiffness and the ultimate strength were also affected by the number of the story for the same thickness of the plate. Generally, the effectiveness of SPSW depended on the number of the story of the structure.
- The comparison of the ultimate strength for 2, 4, 6, and 8-story steel buildings with 1 mm of SPSW and without SPSW showed that after supporting the structure it has increased about 3697 kN, 2986 kN, 1062 kN, and 725 kN in comparison of the original building, respectively. Also, the initial stiffness after using 1 mm of SPSW has increased about 24470 kN/m, 9991 kN/m, 5957 kN/m, and 2509 kN/m in comparison to the building without SPSW, respectively. With increasing the plate thickness from 1 to 15 mm, more strength and stiffness enhancement was widest.
- For all buildings with different aspect ratio, it was discovered that the effect of aspect ratio on the behavior of the frame with SPSW was significant. Based on the capacity curve, the largest value of base shear was of the greatest value of aspect ratio if the amount of steel plate thickness remained constant. In addition, it’s the most important to use SPSWs in longer spans to avoid having big-size columns.
- After looking for all the buildings with different aspect ratio, the result obtained in terms of the ultimate strength and initial stiffness had a mostly similar trend. Both of them had larger value when aspect ratio was greater, conversely.
- The comparison of the ultimate strength for Model 1, 2, 3, 4, and 5 with 15 mm of SPSW showed that by changing aspect ratios from 0.285 to 0.570, 1.000, 1.430, and 2.142 it has increased about 1.46, 3.04, 3.31, and 4.02 respectively. The initial stiffness has increased also about 1.69, 3.86, 4.56, and 10.63 respectively.

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