NUMERICAL INVESTIGATION OF STEEL PLATE SHEAR WALL UNDER CYCLIC LATERAL LOAD

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Declaration

Except for the contents where specific references have been made to the work of others, the studies embodied in this dissertation are outcome of the research conducted by the author.

Md. Raqubul Hasib

Table of Contents

List of figures

List of tables

List of Symbols and Abbreviations

A_b	Cross section area of beam
A_c	Cross section area of column
b_f	Width of flange in W section
d	Web depth in W section
E	Modulus of Elasticity
G	Elastic Shear Modulus
H	Height of shear wall panel
I_c	Moment of inertia of column
L	Length dimension
B	Width of infill panel
V	Shear force in one storey model
A	Inclination of tension field from vertical
Aspect ratio	L/h (aspect ratio)
σ_{true}	nominal (engineering) stress
ε_{nom}	nominal (engineering) strain
$\Delta\varepsilon$	mechanical strain increment
$\Delta\varepsilon_{el}$	mechanical strain increment (elastic part)
$\Delta\varepsilon_{pl}$	mechanical strain increment (plastic part)
SPSW	Steel plate shear wall.
HBE	Horizontal Boundary Element
VBE	Vertical Boundary Element

Acknowledgement

A research work cannot truly completed by one person alone. As such, I want to thank a varied group of individuals who, whether they knew it or not, all came together to encourage and support this effort.

At first I want to thank my supervisor for her valuable guidance and helpful review to successfully complete this work. I also want to thank my parents for continuous supporting and encourages in my study.

At last I want to thank my employer and colleagues to give me time for continuing M.Sc study in the middle of professional work.

ABSTRACT

Steel plate shear wall is an efficient lateral load resisting system. It provides good energy dissipation capacity in medium and high rise buildings which is very important for high seismic areas. The system consists of steel infill plates connected to the surrounding horizontal beam and vertical column. Steel plate shear wall is well suited for new construction and also used for seismic upgrading of existing structure. Although significant number buildings are now constructed using this technology; the design tend to be very conservative as the behavior of this system is not easily understood. Though lots of analytical and experimental investigations are performed, required parametric enquiries are not fully available, hence appropriate design recommendations are limited.

The aim of the research is to investigate the effect of key parameters on the behavior of steel plate shear wall. Prior to parametric analysis a three storied reference experimental model was modelled with commercial finite element software ABAQUS using static stress displacement formulation. Kinematic hardening material model was used to incorporate material nonlinearity. The performance of the model was evaluated by comparing its monotonic and cyclic prediction with experimental results.

The parametric study of steel plate examines the effect of varying shear wall aspect ratio, shear wall thickness and varying stiffness of vertical boundary element. It has been observed that, aspect ratio and the plate thickness have mutual effect on steel plate shear wall system. Capacity and the stiffness of the system can be improved significantly by increasing the thickness of the plate. However, beyond the certain thickness (10 mm) capacity increasing becomes saturated (wide shear wall). Study finds that, relationship between the column flexibility parameter and the capacity of the shear wall is linear. Numerical study is also conducted for horizontally stiffened steel plate shear wall, it is evident form the analysis that, stiffener application is effective only for narrow tall shear wall.

CHAPTER 1
INTRODUCTION

1.1 General

Steel plate shear wall (SPSW) is a lateral load resisting system which resists the horizontal story shear of a building. In general, it consists of a steel plate wall, boundary columns and horizontal floor beams. Steel plate shear walls (SPSW) have several benefits over traditional concrete shear wall. Compared to reinforced concrete shear walls, SPWs are much lighter, which ultimately reduces the demand on columns and foundations, and reduces the seismic load. The steel infill panels in SPSW system can be either stiffened or unstiffened. Use of horizontal and vertical struts in stiffened SPSW systems can increase the capacity and therefore also reduce the section of horizontal beam and vertical column. A properly designed steel plate shear wall has superior ductility, high initial stiffness, stable hysteresis loops, inherent redundancy, and good energy absorption capacity. These characteristics make the system attractive in high-risk seismic regions.

1.2 Background

In the past two decades the steel plate shear wall (SPSW), also known as the steel plate wall (SPW), has been used in a number of buildings in Japan and North America as part of the lateral force resisting system. Early steel plate shear wall buildings in japan include the Nippon steel building (20 storey) and Shinjuku Nomura building (51 storey), both in Tokyo both built in the 1970s (Seilie et al. 2005). The 35-storey Kobe City Hall tower has stiffened steel plate shear wall from third floor and above. The structure has been subjected to the severe earthquake in 1995 (Seilie et al. 2005). Fujitani et al. (1996) reported minor local buckling on the stiffened steel plate shear wall on the 26th storey and residual building drift. Stiffened steel plate shear wall are most common in Japan. The Saitama Joint agency Buildings, 31 and 26 stories (Minami et al. 1998), provide examples of implementation of stiffened steel plate shear wall.

Steel plate shear walls have been used in United States since 1970's when initially they were used for seismic retrofit of low and medium-rise existing hospitals and other structures. These initial shear walls were designed with relatively closely spaced horizontal and vertical stiffeners. For example, SPSW was used in wind controlled design of Hyatt regency hotel in Dallas. The system was also used in Olive view medical center at Sylmar in California’s San Fernando Valley (Astaneh et al. 2001). Stiffened steel shear wall was used for the seismic retrofit of the Veterans Administrations Medical Center in Charlestown, South Carolina. In addition to providing in-plane resistance to wind load or seismic loads, SPSW is also used for blast-resistant design on the basis of their out of plane strength (Innovation, 2002).

Since early 1980s, unstiffened steel plate shear has been constructed in Canada. An eight storey building was constructed in Vancouver, British Columbia, to provide adequate seismic performance in the short building direction. Another example of a modern steel structure that features steel plate shear walls is the recently completed head office expansion of the Canam Manac Group located in St.-Georges de Beauce, Quebec located in seismic zone 3. The six-story expansion adds over 3,700m^2 (39,830 sq. ft.) of office space to the building, plus an extension to an underlying hotel. The sylmer hospital building at LA, California is potential example of unstiffened steel plate shear wall, which is shaken by 1994 Northridge earthquake (Celebei et al. 1997). The building has reinforced concrete shear walls in the first two stories and steel plate shear walls in the upper four stories. The steel shear wall panels in this building are 25 feet wide and 15.5 feet high with thickness of wall plate being 5/8” and ¾”.

From a designer's point of view, steel plate shear walls have become a very attractive alternative to other steel systems, or to replace reinforced concrete elevator cores and shear walls. In comparative studies it has been shown that the overall costs of a building can be reduced significantly when considering the following advantages:

I. An SPSW system, when designed and detailed properly, has relatively large energy dissipation capability with stable hysteretic behavior, thus being very attractive for high risk earthquake zones.

II. An SPSW system has relatively high initial stiffness, and is thus very effective in limiting wind drift.

III. Compared to reinforced concrete shear walls, SPSWs are much lighter, which ultimately reduces the demand on columns and foundations, and reduces the seismic load, which is proportional to the mass of the structure.

IV. Compared to reinforced concrete construction, the erection process of an all-steel building is significantly faster, thus reducing the construction duration, which is an important factor affecting the overall cost of a project.

V. By using shop-welded, field-bolted SPWs, field inspection is improved and a high level of quality control can be achieved.

VI. For architects, the increased versatility and space savings because of the smaller cross-section of SPSWs, compared to reinforced concrete shear walls, is a distinct benefit, especially in high-rise buildings, where reinforced concrete shear walls in lower floors become very thick and occupy a large proportion of the floor plan.

VII. All-steel construction with SPWs is a practical and efficient solution for cold regions where concrete construction may not be feasible, as very low temperatures complicate construction and freeze-thaw cycles can result in durability problems.

VIII. In seismic retrofit applications, SPSWs are typically much easier and faster to install than reinforced concrete shear walls, which is a critical issue when building occupancy needs to be maintained throughout the construction time.

IX. In the event of inelastic response, steel panels are more readily replaced, and repairs are otherwise simpler than for equivalent reinforced-concrete systems.

The failure mechanism has been recognized as early as in the 1930s in aerospace engineering (Wagner, 1931), and as early as in the 1960s in steel building construction, when it was incorporated into the design process of plate girders (Basler, 1961). Research on unstiffened steel plate shear walls has investigated the effect of simple versus rigid beam-to-column connections on the overall behavior (Caccese et al., 1993), the dynamic response of steel plate shear walls (Sabouri-Ghomi et al. 1992; Rezai, 1999), the effects of holes in the infill plates (Roberts et al., 1992; Vian et al., 2004), the use of low-yield-point steel and fight-gauge steel (Vian et al., 2004; Berman et al., 2005).

The appropriateness of post-buckling stiffness and strength characteristics of SPW to resist service lateral loads was analytically predicted by Thornburn et al. (1983) and experimentally confirmed by Timler et al. (1983). In addition to the strip model, researchers have used continuum-type finite element analysis to determine and investigate local response mechanisms in SPSWs. Asteneh-Asl et al. (2002) employed orthotropic plate elements to simulate the response of infill panels. This approach has also been used in the design of SPSW buildings (Seilie et al. 2005). While practical, orthotropic plate models are limited to elastic behavior and thus can only be used in the equivalent lateral force or elastic time history analysis procedures. Driver et al. (1998) and Behbahanifard et al. (2003) tested a 4-story and 3-storey large-scale specimen respectively and developed its finite element model. Mortazavi et al. (2013), Kurban (2009), Behbahanifard et al. (2003) and other researchers performed limited parametric study.

However, still there needs serious parametric study for better understanding the behavior of steel plate shear wall.

1.2 Objectives of the study

The objectives of this study are,

1. To conduct a nonlinear finite element analysis of steel plate shear walls under cyclic lateral loading and validate it with experimental results (Study did not consider stiffness degradation of the system).
2. To carry out a parametric study on the behavior of unstiffened SPSW under cyclic loading. The parameter to be varied are aspect ratio, plate thickness, column flexibility parameter and grade of steel
3. To investigate the effect of horizontal struts on the performance of SPSW.

1.3 Scope of the study

Under the current research, there is scope to find out the key parameters, and to investigate their likely effect on steel plate shear wall system by numerical modeling. Analytical study of geometric parameters in larger scale also provide the opportunity to propose some design recommendations. A design recommendation was proposed to improve the capacity of SPSW system using horizontal strut, however effect of vertical strut was not studied.

1.4 Methodology

A 3D nonlinear finite element analysis of a three storey steel plate shear wall was conducted under quasi-static and cyclic loading conditions using ABAQUS (Hibbit et al. 2001) finite element code. Experimental results of a single bay three storied shear wall (tested by Behbahanifard et al. 2003) was used for verification of FEM model. Both material and geometric nonlinearities was included in the model. Effect of residual stress originated from the steel welding is not considered. A bi-linear true stress strain curve has been defined to simulate the material behavior of steel. Non-linear static solution was implemented to trace the load deflection behavior of the SPSW up to failure. After validation of finite element model, an extensive parametric study will be conducted on SPSW under cyclic loading. The variable parameters are aspect ratio, plate slenderness ratio, and yield strength of the steel. The effect of horizontal stiffener for improving the capacity of the system was also explored for narrow tall steel plate shear wall system.

1.5 Outline of the thesis

The contents of the thesis has been organized in six chapters.
Chapter 1 includes the background of the research along with its objectives and scope

Chapter 2 provides a sequential review of earlier research on steel plate shear walls. The review includes a summary of both the experimental and analytical investigations available in the published literature.

A detailed description of the finite element model that has been developed in this study for the analysis of steel plate shear wall has been presented in chapter 3. The description of the reference test specimen that is used for validation of finite element model is also included in this chapter.

Chapter 4 demonstrates the performance of finite element model of SPSW system developed in this study. The proposed finite element model is used to predict the behavior of the three-story steel plate shear wall. Pushover analysis, cyclic behavior, energy dissipation, and inclination of the tension field are obtained and compared with the test results from the published literature.

Chapter 5 of the thesis presents a detailed parametric study which includes the effects of two geometric and one material parameter on the behavior of unstiffened SPSW system. The effects of additional stiffeners has also been included.

Finally, the summary and major findings of the current research along with the recommendations for future work has been presented in chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Research on steel plate shear walls (SPSW) was begun in the early 1970's. Experimental and numerical studies that have been conducted so far have all established that a properly designed steel plate shear wall is the most effective and economical lateral load resisting system, especially for application in severe earthquake regions. SPSW are widely used in Japan, Canada, Mexico and USA, as a low rise residential building to high rise commercial building and also to retrofit existing frame structure to increase strength and stiffness. Structural engineers prefers to design SPSW with closely spaced horizontal and vertical stiffener at the early stage. This design practice was followed in order to increase shear strength and to prevent out of plane buckling of the plate prior to shear yielding.

Stiffened steel plate shear wall is not a cost effective option. Unstiffened steel plate shear wall is more popular. It is desirable to achieve desirable strength and stiffness by using an unstiffened slender web plate rather than a stiffened web plate. High capacity can be achieved by increasing the thickness of web plate or using rigid beam to column connections in the frame of the shear wall.

2.1.1 Types of steel plate shear wall

Conceptually, steel plate shear walls can be classified in two fundamental types (Figure 2. 1).

a) Un-stiffened steel plate shear walls.

b) Stiffened steel plate shear walls.

(a) Un-stiffened steel plate shear wall

In unstiffened steel plate shear wall system the web plate has negligible compressive strength, thus shear buckling occurs at smaller magnitude loading. Lateral loads are resisted through diagonal tension in the web plate. The post-buckling behavior of plates is stable and plates will continue to carry higher loads beyond their elastic critical loads. If we draw imaginary diagonals on the plate, the diagonal which gets loaded in compression, buckles and cannot support additional load on the other hand, the diagonal in tension continues to take more load and the plate becomes like a triangular truss with only tension diagonals, which is known as tension field action.

Boundary elements, in this kind of system, are designed to permit the web plates to develop significant diagonal tension.

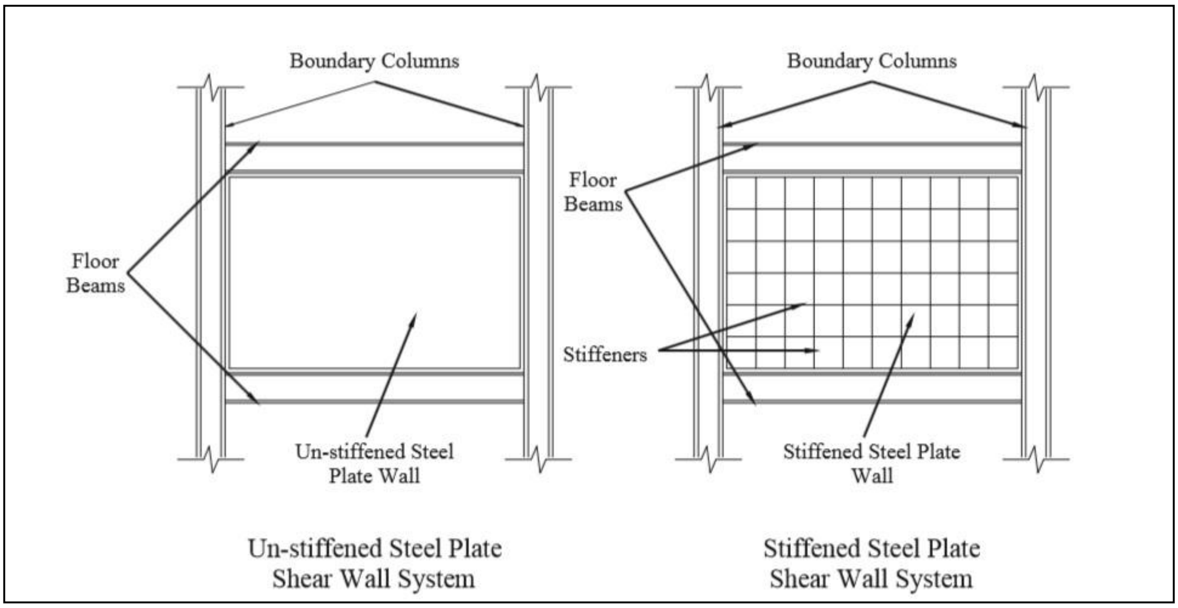


Figure 2. 1: Stiffened and Un-stiffened steel plate shear wall systems (Osman et al. 2004).

(b) Un-stiffened steel plate shear wall

Stiffened steel plate shear walls are also used frequently. Stiffening increases the shear buckling strength of the web plate. Sufficient stiffening to permit the web plate to develop its full shear yield strength.

Stiffening of the web plate has moderate effect on the strength and stiffness of the wall. It tends to reduce the strength and stiffness requirement of boundary element. Stiffening also results in hysteresis behavior that is significantly less pinched (Figure 2. 3). However, the system substantially increase the cost of construction.

2.1.2 Uses of steel plate shear wall in structural system

There are various ways to incorporate Steel plate shear wall in structural system (Figure 2. 2). Following systems are commonly used in current civil engineering practice.

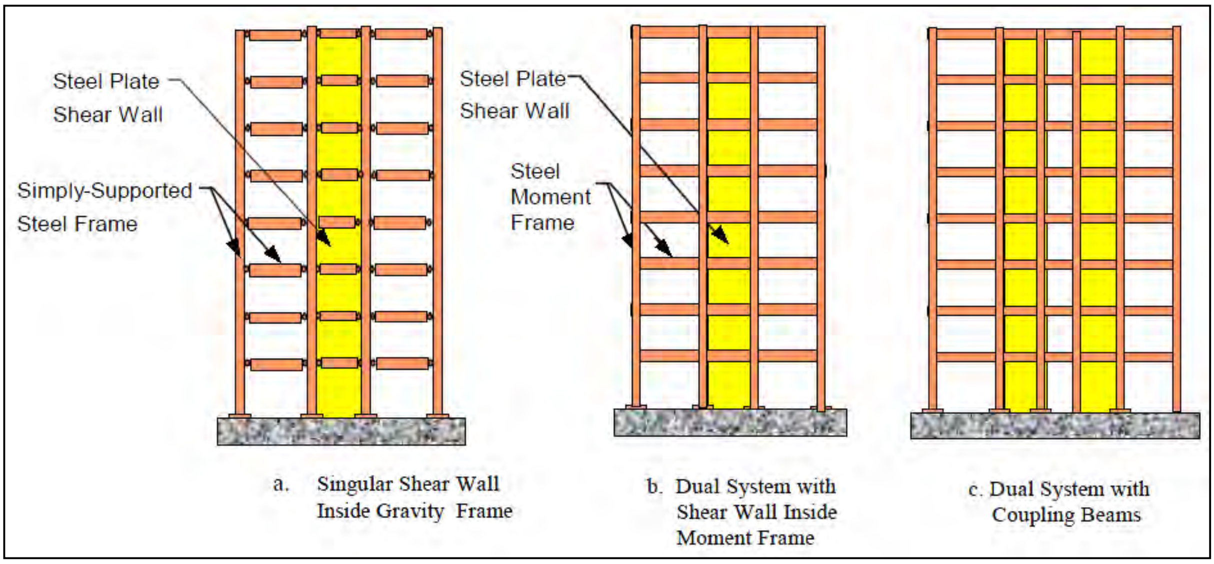


Figure 2. 2 Use of steel shear wall in different structural system (Astaneh et al. 2001).

(a) “Singular” shear wall system where a steel shear wall is placed inside gravity frame and shear wall is the only element resisting story shear.
(b) “Dual” shear wall system where steel shear wall is placed either inside a special moment frame or is parallel to it.
(c) Coupled Shear wall system where a coupling beam connects two shear wall bays. The frame or portion of it that contains the shear walls and coupling beams is special moment frame.

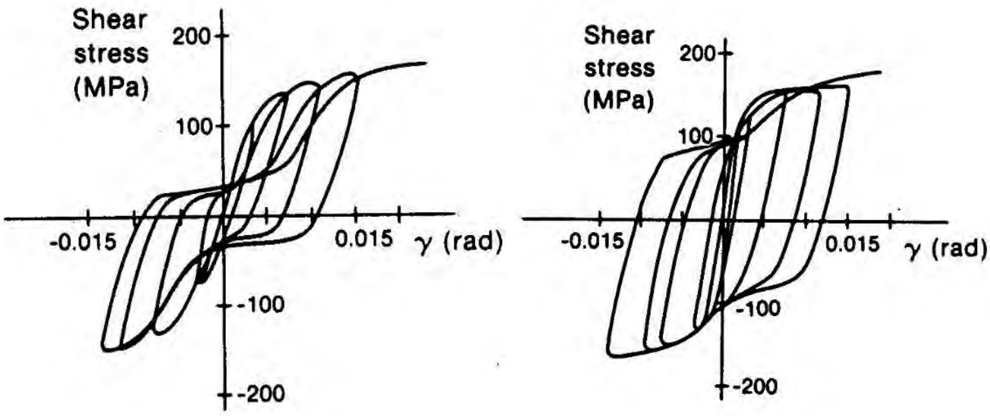
2.2 Major Historical research on steel plate shear wall

Researchers has performed numerous study to fathom the behavior and performance of steel plate shear wall. A number of experimental and numerical investigation has been conducted on stiffened and unstiffened steel plate shear wall system. A sequential review of previous steel plate shear wall research is presented below:

2.2.1 Takahashi et al. (1973)

Japan appears to be the first country to have extensively designed, tested, and constructed buildings using steel plate shear walls. Takahashi et al. (1973) conducted a series of 12 single panel tests and two single bay, two-storey full-scale stiffened steel plate shear walls with and without reinforced openings. The single panel specimens were fabricated with and without stiffeners in a pin jointed frame. The series of tests on single panels indicated that panels stiffened to prevent buckling of the infill plate showed excellent behavior under cyclic loading with hardly any pinching. In contrast, the unstiffened panel showed significant pinching of the hysteresis loops (see Figure 2. 3). The full-scale two-storey test specimens, designed to behave plastically, also showed good behavior under cyclic loading. Takahashi et al. (1973) developed guidelines for the design of stiffened steel plate shear walls to prevent elastic buckling and a finite element model for the in-plane inelastic behavior of stiffened steel plate shear walls (the out–of–plane buckling of the infill plate was not considered in their finite element analysis). The finite element model was able to trace accurately the envelope of the hysteresis loops.

Based on their test results, Takahashi et al. (1973) recommended that stiffened steel plate shear panels be designed so that the panel does not buckle elastically and if inelastic buckling occurs, it should be limited to local buckling between the stiffeners. They also concluded that the classical shear theory, wherein the horizontal shear is transferred by beam action alone, can be used to calculate the stiffness and yield strength of the stiffened shear panels.



(a) Unstiffened (b) Heavily stiffened

Figure 2. 3: Hysteresis behaviour of steel plate shear walls (Takahashi et al., 1973)

2.2.2 Mimura et al. (1977)

Mimura et al. (1977) developed a general method for predicting the monotonic and cyclic behavior of unstiffened steel plate shear panels through a series of experimental and analytical studies. The monotonic behavior of a shear wall panel was obtained by superimposing the behavior of the infill plate and the frame separately.

Classical plate theory was used to predict the infill plate buckling capacity and a diagonal tension field action was assumed in the post-buckling range. The contribution of the moment resisting frame was obtained from an elastic–plastic frame analysis.

Mimura et al. (1977) proposed a model to predict the cyclic behavior based on their monotonic behavior model and a number of simplifying assumptions. The main assumption was that after plastic deformation of the panel in one direction the amount of deformation required to develop the tension field in the opposite direction is one half of the permanent plastic deformation during the previous loading cycle. This statement is based on the assumptions of inelastic Poisson's ratio of 0.5 and an angle of inclination of the tension field of 45°. The stiffness of the frame during the redevelopment of the tension field was neglected

Mimura et al. (1977) conducted a series of tests to validate their proposed model. The tests were conducted on small-scale simply supported stiffened plate girders subjected to a single cyclic point load at mid-span. The test results were in good agreement with their proposed model except in the redevelopment phase of the tension field where stiffness of the frame was neglected.

2.2.3 Timler et al. (1983)

In order to verify the strip model proposed by Thorburn et al. (1983), Timler et al. (1983) conducted a large-scale, single-storey steel plate shear wall test. The major areas of interest were the study of the tension field development in the infill plate, the out–of– plane behavior of the plate under service load reversals (quasi-wind cyclic loading), and the ultimate load behavior of the system. The test specimen consisted of a pair of single-storey, one-bay, and shear wall with pinned joints at the four extreme corners.

Timler et al. (1983) modified the angle of inclination of tension field proposed by Thorburn et al. (1983). The method and the basic model used by Timler and Kulak was the same as used by Thorburn et al. (1983) except that the bending strain energy of the columns was added to the energy calculation. The revised equation for the angle of inclination of the tension field takes the following form:

$$\alpha = \tan^{-1} \sqrt[4]{\frac{1+\frac{tL}{2A_c}}{1+th\left(\frac{1}{A_b}+\frac{h^3}{360 I_c L}\right)}} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(2.1)}$$

Where, I_c = moment of inertia of the boundary column.

A_c = Cross section area of column

A_b= Cross section area of beam

t= thickness of steel plate

L = Distance between VBE centerline

h= Distance between HBE centerline

Although the formulation of Equation 2.2 involved a number of simplifying assumptions, Timler et al. (1983) showed a reasonable agreement between the predicted value of the angle of inclination of the tension field and the angle measured during the test. The measured value for α , as obtained from the strain gauge reading, was between 47° and 53° in the lower portion of the panel as compared to the predicted value of 51°.

A comparison between the test results and the predicted behavior using the strip model of Thorburn et al. (1983) with the angle of the tension field given by Equation 2.1 showed good agreement. The measured axial strains in the columns were also in good agreement with the predicted values, but the bending strains were over predicted by the analysis.

The strip model proposed by Thorburn et al. (1983) and the modified equation for the angle of inclination of tension field proposed by Timler et al. (1983) have been adopted by the Canadian Standard CSA–S16–01 as a simple approach for the analysis of unstiffened steel plate shear walls.

2.2.4 Thorburn et al. (1983)

Thorburn et al. (1983) developed a simple analytical model to study the shear behavior of thin unstiffened steel plate shear walls. The model was based on the pure diagonal tension field introduced originally by Wagner (1931). The shear strength of the panel prior to buckling was neglected, leaving only the tension field action as the load carrying mechanism. In this model, referred to as the strip model, the action of the tension field was modelled by a series of pin-ended inclined tension-only members. These strips were oriented parallel to the direction of the tension field. Each strip was assigned an area equal to the width of the strip times the plate thickness. The strip model for a typical interior panel is shown in Figure 2. 4. In this model the interior beams are assumed to be infinitely rigid in bending. The angle of inclination of the tension field was obtained using the principle of least work and considering only the energy of the tension field and axial energy in the beams and columns. The proposed equation for angle of inclination of the tension field by Thorburn et al. (1983) takes the following form:

$$\tan\alpha = 4\sqrt{\frac{1+\frac{Lt_p}{2A_c}}{1+\frac{ht_p}{A_b}}} \quad \text{(2.2)}$$

Where, α is the angle of inclination of tension field,

t_p = infill plate thickness

L = distance between VBE centerline

h = distance between HBE centerline

And, A_b and A_c = cross-sectional area of the beam and an individual column, respectively.

By using a plane frame program and the strip representation of the infill plate a steel plate shear wall system can be analyzed. The beams and columns are assigned their actual stiffness. The researchers also studied the use of a single equivalent diagonal brace suitable for preliminary analysis of multi-storey shear walls. The area of the brace is obtained in such a way that the stiffness of the panel is equivalent to that derived from the strip model.

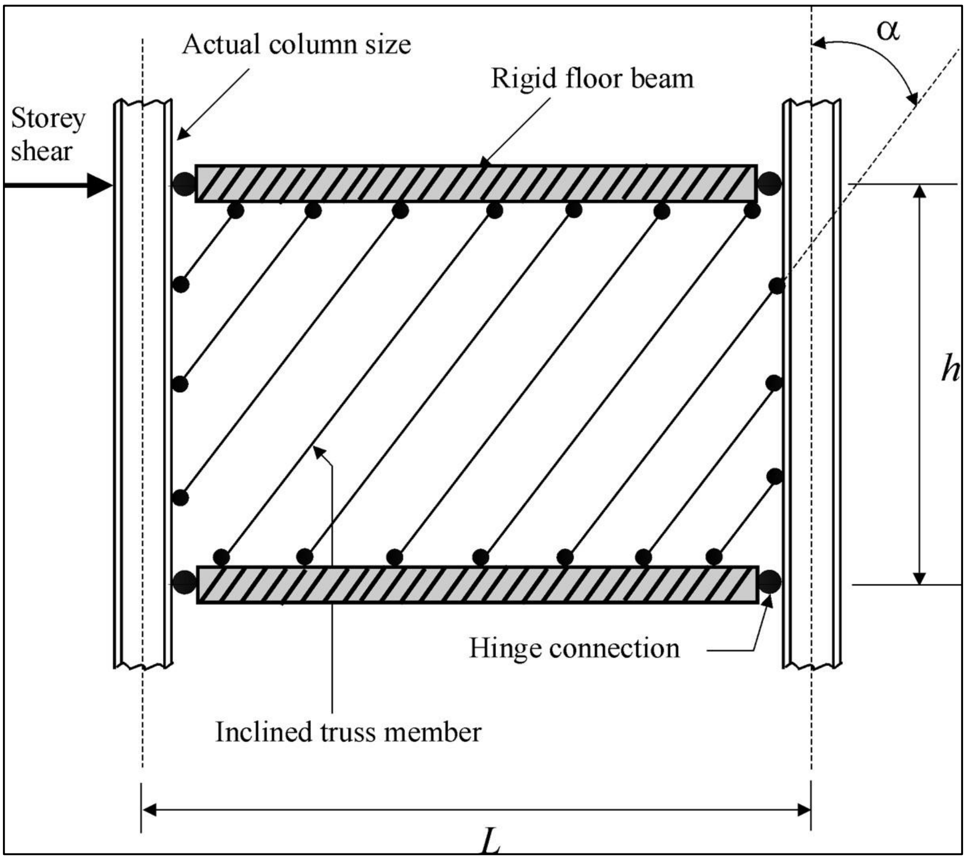


Figure 2. 4: Strip model proposed by Thorburn and Kulak (1983)

2.2.5 Tromposch et al. (1987)

Tromposch et al. (1987) conducted a large-scale test similar to the one conducted by Timler et al (1983). The test specimen, shown in Figure 2. 5, was different from Timler’s specimen in two respects: it used typical bolted shear beam–to–column connections and gravity loads were applied to the columns. Cyclic loading was applied to the test specimen, with gradually increasing displacements in a quasi-static condition, up to the limit of the loading device at 67% of the ultimate load. The test was followed by monotonic loading up to the ultimate capacity of the specimen. As shown in Figure 2. 6, the specimen showed very ductile and stable behavior, but the hysteresis loops were severely pinched due to use of very thin plate and flexible boundary frame. By using the strip model developed by Thorburn et al. (1983), the researchers conducted a pushover analysis of the specimen. Good agreement was found between the analysis and the envelope of the hysteresis loops obtained in the test.

Tromposch et al. (1987) proposed a model for predicting the hysteresis behavior of unstiffened steel plate shear walls. Similar to the model proposed by Mimura et al. (1977), the hysteresis loops were generated using a monotonic load versus deflection curve (obtained from a strip analysis) and assumptions about the hysteresis behavior of the shear panel. The model incorporated the effect of frame stiffness and the effect of low panel buckling strength.

Tromposch et al. (1987) demonstrated that the proposed model was able to predict reasonably well the experimentally observed hysteresis behavior of their test specimen.

A parametric study showed the significance of connection type on the stiffness and energy absorption capacity of steel plate shear walls. Changing simple beam–to–column connections to rigid beam–to–column connections can increase significantly the energy absorption capacity of the system.

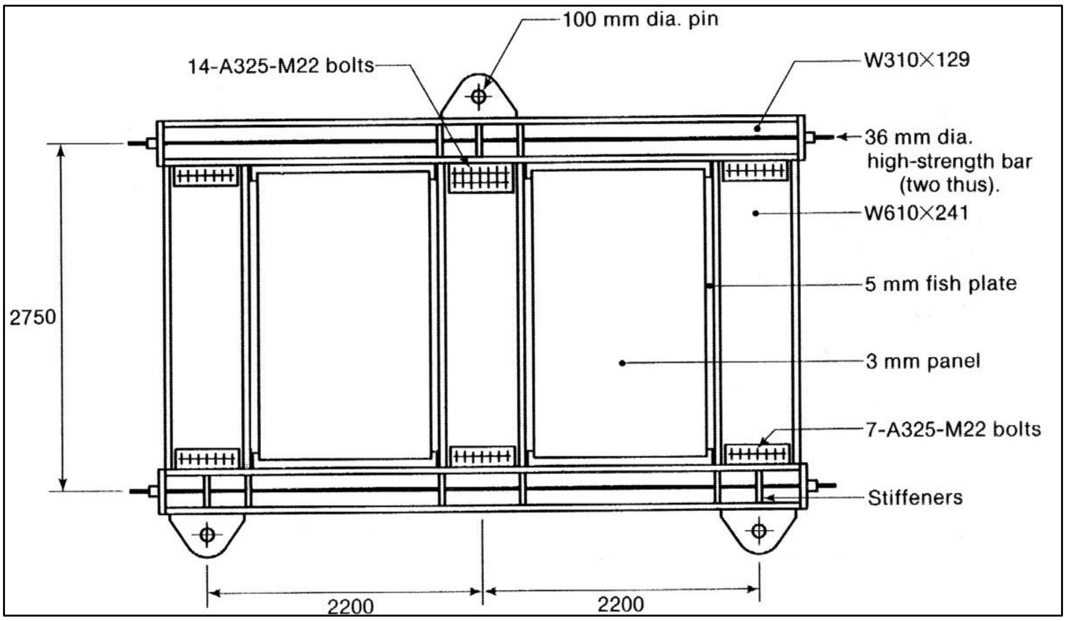


Figure 2. 5: Schematic of specimen tested by Tromposch and Kulak (1987)

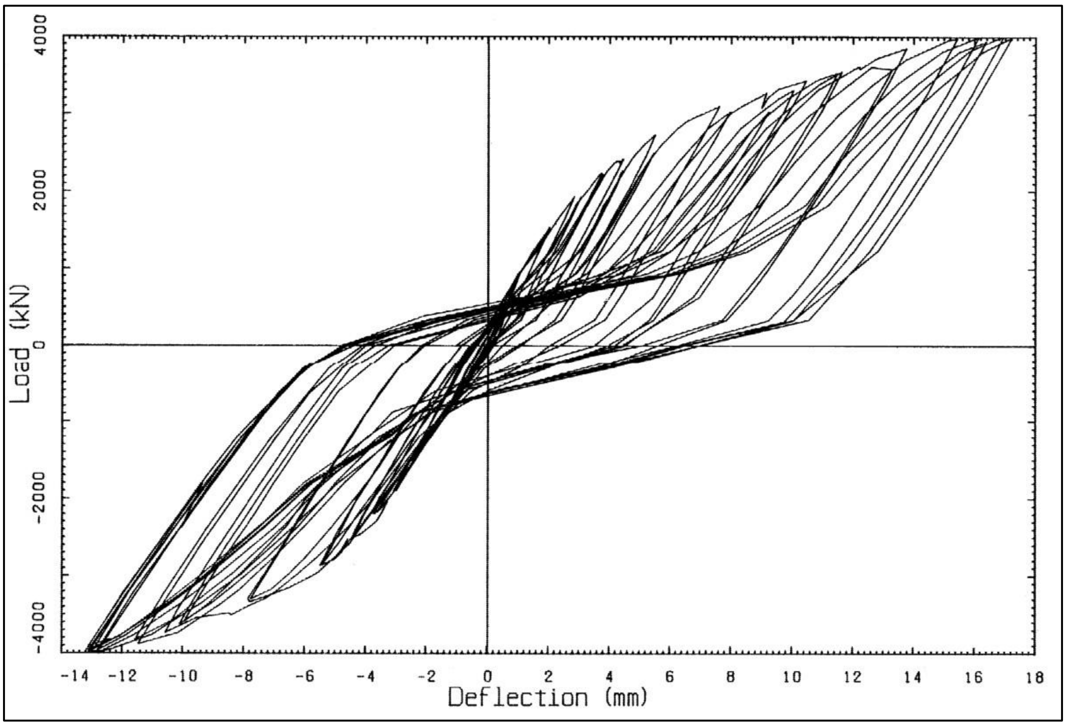


Figure 2. 6: Hysteresis behaviour of specimen tested by Tromposch and Kulak (1987)

2.2.6 Sabouri-Ghomi et al. (1992)

Sabouri-Ghomi et al. (1992) proposed a method for nonlinear dynamic analysis of thin steel plate shear walls whereby the system was idealized as a vertical cantilever plate girder. The governing differential equation of motion for a continuous cantilever beam was discretized to a multi-storey shear wall in which the associated storey masses and the dynamic forces were concentrated at each floor. Initially, the governing equations were formulated assuming only shear deformation and later, a more general formulation that included both bending and shear deformations was presented. The governing differential equations were solved using a finite difference time stepping technique.

Material nonlinearity was incorporated in the analysis by using an approximate elastic–plastic hysteresis model for each panel of the shear wall. The hysteresis model took into account the shear buckling and yielding of the web plate as well as the boundary members. The hysteresis behavior of the web plate was obtained from a series of quasi-static tests on small-scale single panel unstiffened plates with stiff, pin-ended boundary frames. An elastic–perfectly plastic material model was assumed for the boundary frame alone, assuming plastic hinges at the top and bottom of the columns. The hysteresis curve for the entire shear wall panel was defined by superposition of hysteresis curves for the web plate and the boundary columns. The theoretical model was in reasonable agreement with the test results.

Sabouri-Ghomi et al. (1992) evaluated their analytical model by analyzing a five-storey single bay steel plate shear wall subjected to three different periodic loadings. The loads were selected in such a way as to examine the elastic, elastic–plastic, and resonance response of the model. The results were interpreted only by engineering judgment. The analytical technique developed by the researchers has not been validated with any experimental test results.

2.2.7 Caccese et al. (1993)

To assess the effectiveness of using the thin-plate shear wall system in seismic zones Caccese et al. (1993) conducted quasi-static cyclic tests on six quarter-scale, single-bay three-storey unstiffened steel plate shear walls. Beam–to–column connection type (simple and rigid) and panel width–to–thickness ratio were the parameters that were investigated.

The experimental program included cyclic and monotonic tests. The specimens were loaded with a single horizontal load at the top of the shear walls. The load history, similar to that proposed in ATC-24 (Applied Technology Council, 1992), consisted of displacement peaks that were increased in eight increments up to 2% drift measured at the top of the shear walls.

The test results demonstrated that addition of an unstiffened thin steel plate to a steel frame results in a system with a substantial increase in stiffness, capacity, and energy absorption. The researchers concluded that the beam–to–column connection type has a minor effect in the behavior of a steel plate shear wall system. This conclusion was discussed by Kulak et al. (1994) who pointed out that because of different plate thickness and material properties among the test specimens, plus a failed weld in one of the tests, made a direct comparison of the test results impossible. Their assessment of the effect of connection type was therefore rejected. Caccese et al. (1993) also concluded that when a slender plate is used as an infill, inelastic behaviour is initiated by yielding of the plate and the strength of the system is governed by the formation of plastic hinges in the columns.

When the infill plate thickness is increased the failure mode is governed by column instability and only a negligible increase in system capacity is achieved.

Following their experimental study, Elgaaly et al. (1993) carried out numerical investigations of the test specimens under monotonic loading. Two numerical models were considered. First, a nonlinear finite element model, including material and geometric nonlinearity, was used. The infill plates were modelled with shell elements and beam elements were used to model the boundary members. The finite element model greatly overestimated both the stiffness and the capacity of the test specimens.

In the second study, the simple model developed by Thorburn et al. (1983) was used. In this model the infill plates were replaced by a perpendicular grid of tension members oriented in the direction of the principal tensile and compressive stresses. By using an elastic–perfectly plastic material model for the strips, only the initial slope of the response and the capacity of the specimens were predicted accurately. Based on the observed test behavior, a bilinear elastic–plastic stress versus strain curve was proposed for the infill plate and the parameters of the model obtained empirically, which resulted in a better fit of the test results. The parameters used in the model were a linear function of the ratio between the buckling and the yield strength of the infill plate. An empirical model was also developed for predicting the hysteretic behavior of the specimens. This model predicted the behavior of the test specimens reasonably well. The researchers indicated that their empirical formula was valid within a specific range of the parameters and should not be applied outside that range without further test results (the ratio of the buckling to yield strength of the infill plates used in their experimental program varied between 0.0098 and 0.123). Although the influence of the number of truss elements used in the strip model was found to be important for an accurate calculation of internal forces in the boundary members, the variation of the angle of inclination of tension field was found to have only a small effect on the predicted capacity of the steel plate shear wall.

2.2.8 Xue and Lu (1994)

Xue and Lu (1994a) conducted a numerical investigation of the effect of different arrangements for connecting the infill plate to the boundary members and the effect of beam–to–column connection type on the behavior of unstiffened steel plate shear walls. A three-bay, 12-storey frame with moment resisting beam–to–column connections in the two exterior bays and with steel infill plate in the middle bay was used for their investigation. The system was designed to resist the earthquake loads specified in the Uniform Building Code (UBC, 1988). Based on two different beam–to–column connection types (rigid for all connections or shear type at intermediate bay and rigid for the exterior bays) and two different arrangements for connecting the infill plate to the boundary members in a panel (connecting to both girders and columns, GC, or connecting only to the girders, a total of four frame–wall combinations were considered, namely, **F-GC**, **F-G**, **P-GC**, and **P-G**. Lower bound and upper bound solutions were also produced for comparison with the numerical analysis results. The upper bound solution consisted of a frame with all moment-resisting connections, infill plates connected along all four edges, and infill plates assumed not to buckle under load. The lower bound was a frame with simple beam–to–column connections in the interior bay, and no infill plate. The primary parameter investigated in this study was the lateral stiffness of the system, since drift control is often a major design consideration. A total of six frame–wall structures were modelled using the finite element method. Elastic beam elements were used to model the frame members and 4-node shell elements with large deformation capability were used for the infill plates. The initial imperfections introduced in the panels consisted of the superposition of several shear buckling modes of the infill plates. A bi-linear stress versus strain curve with kinematic hardening model was used for the infill plates. Vertical distribution of the lateral loads at each floor was based on UBC (1988). The lateral loads were applied monotonically at each floor and no gravity loads were applied. The base shear versus top storey displacement obtained from the analysis demonstrated that the infill plates increased significantly the stiffness of the system, but the type of beam–to–column connection in the in-filled bay had a negligible effect on lateral stiffness. The stiffness of the systems with infill plates connected to both girders and columns (**GC**) were as high as the stiffness predicted using the upper bound solution and were only slightly higher than the stiffness of the systems with infill plates connected to girders only (**G**). A number of factors led to the conclusion that the **P-G** system (simple beam– to–column connections in the infilled bay and infill plates connected to the girders only) has the best performance. Xue and Lu (1994b) also conducted a parametric study to investigate the load versus deformation characteristics of the frame–wall system consisting of a panel of steel plate shear wall with simple beam–to–column connections and infill plate connected to beams only. Rigid boundary members were used in the analysis. The width–to–thickness ratio of the infill plate and the panel aspect ratio (width/height) were investigated by finite element analysis of 20 different cases. The researchers found that the width–to–thickness ratio has no significant effect on the response of the system while the aspect ratio of the panel had a significant effect on the panel behavior. The load at significant yield increased significantly as the aspect ratio increased while the post-buckling stiffness remained almost the same. From the results of their parametric study, Xue and Lu proposed a simplified empirical equation to predict the yield strength, yield displacement, and the post-yield stiffness of the system.

Xue and Lu (1994b) also conducted a cyclic analysis on a single panel of a twelve-storey three-bay structure described by Xue and Lu (1994a). Although the researchers used a simple panel with infill plate connected to girders only and neglected the deformation of the boundary members, they reported numerical difficulties in the analysis due to snap- through behavior of the infill plate in the intermediate deformation range. Six cycles of gradually increasing displacements were applied up to a storey drift of 1.68%. The panel demonstrated significant energy dissipation capacity even with some pinching. The pinching became relatively less severe as the shear deformation increased. The merit of the approach proposed by these researchers, compared to the traditional approach for a frame–wall system, should be further investigated. A comparative study should highlight the differences of the two systems in terms of cyclic behavior and failure mode in severe earthquake simulations. No experiment is available to confirm the cyclic behavior of the proposed system.

2.2.9 Driver et al. (1997, 1998)

Driver et al. (1997, 1998a) conducted a quasi-static cyclic test on a half-scale four-storey unstiffened steel plate shear wall. The main objective of the test was to evaluate the overall in-plane performance of the shear wall under extreme cyclic loading. The specimen, shown in Figure 2. 7, had rigid beam–to–column connections and the infill plates were welded to the boundary members through fish plates. Gravity loads were applied at the top of the columns and were kept constant during the test. Equal horizontal loads were applied cyclically at each floor under quasi-static condition. The load and deflection sequences were selected based on recommendations by Applied Technology Council (1992). The storey shear versus storey deformation of the first panel was used to control the test. A total of 30 load cycles were applied to the specimen and 20 of those cycles were in the inelastic range. The shear wall specimen was found to be initially stiff, very ductile, and it exhibited hysteresis behavior with significant energy absorption. In the final cycle the panel had reached a deformation of nine times the yield deformation. The post-ultimate degradation was slow and controlled. The moment resisting boundary frame used in the test specimen improved the behavior and prevented the severe pinching of the hysteresis loops that was seen in the shear walls with shear type beam–to–column connections (Tromposch et al. 1987). Driver et al.

(1997, 1998b) developed a finite element model for the analysis of their test specimen. Beams and columns were modeled with beam elements and the infill plate was modelled with shell elements. Initial imperfections based on the first buckling mode of the plate were incorporated in the model and residual stresses were included in the boundary members. A bilinear stress versus strain curve, along with a kinematic hardening model, was used for the material modelling. Because of convergence problems, geometric nonlinearity could not be included up to the ultimate load. The model was loaded both monotonically and cyclically. The analysis conducted with monotonic loading gave a good prediction of the capacity but overestimated the stiffness of the specimen. The analysis under cyclic loading was not able to capture the important feature of the system, namely, the pinching of the hysteresis loops due to buckling and redevelopment of the tension field. The researchers recommended that more research be conducted to improve the finite element model. Driver et al. (1997) also analyzed their test specimen using the strip model proposed by Thorburn et al. (1987). Infill plates in each panel was replaced by 10 pin-ended diagonal tension strips. The angle of inclination of the tension field was obtained from equation (2.2). Using a plane frame analysis program capable of only elastic analysis, an incremental analysis was conducted up to the ultimate strength. As yielding of the strips was detected in the elastic analysis, the yielded strips were removed. The strip model gave a good prediction of the ultimate strength, but it underestimated the initial stiffness of the specimen.

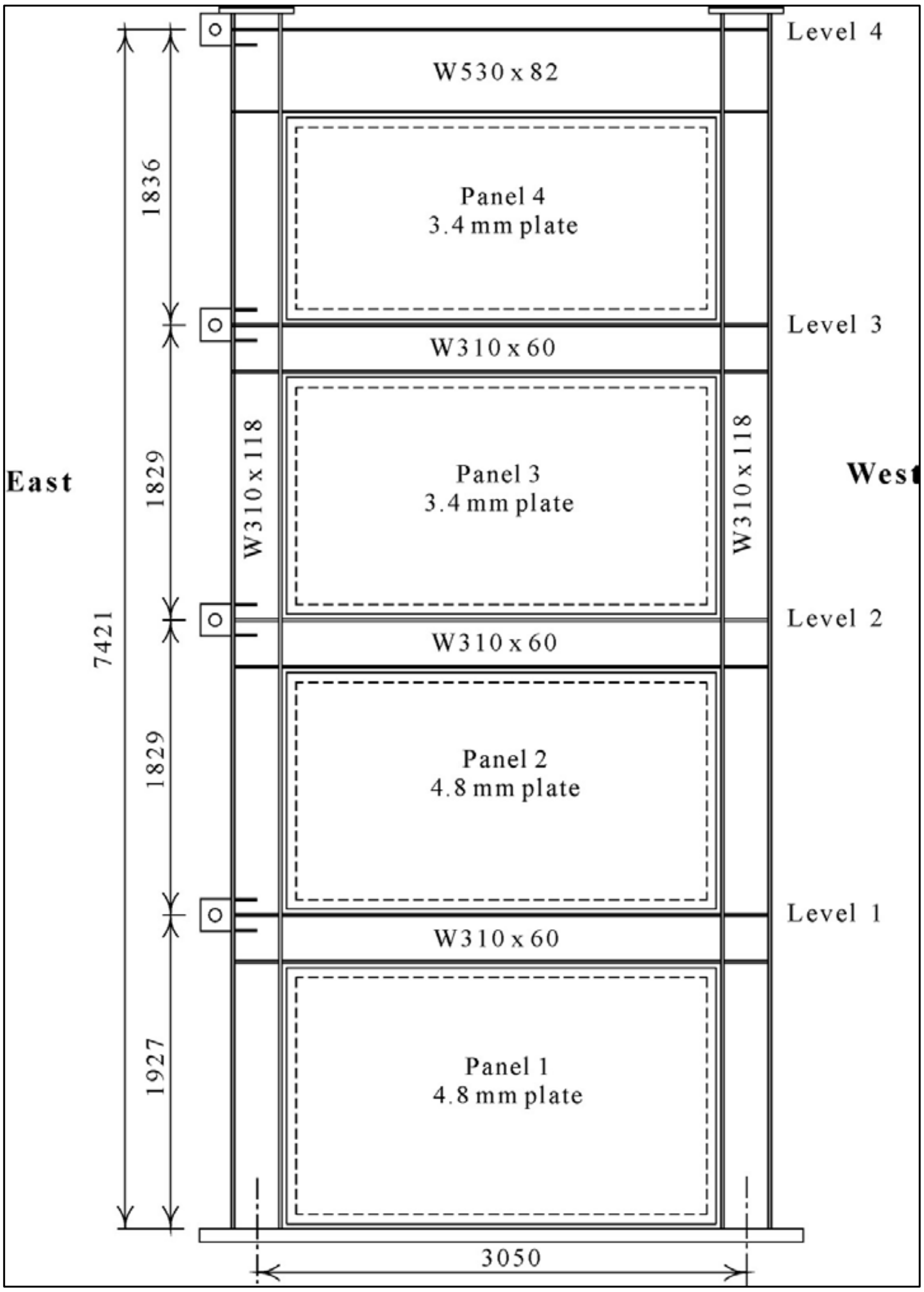


Figure 2. 7: Four-storey steel plate shear wall tested by Driver et al. (1997)

2.2.10 Elgaaly et al. (1997)

Earlier research by Elgaaly et al. (1993) showed that the strain distribution in the infill plate along a diagonal tension strip is not uniform (higher near the boundary members). As a result, yielding of the tension strips starts at the boundaries and then gradually extends towards the center of the strips. Based on the model originally developed by Thorburn et al. (1983), Elgaaly et al. (1997) introduced the concept of strip–gusset elements in order to simulate the non-uniform distribution of strain along the length of the tension strips. In this concept the strips are connected through square gusset plates at both ends to the boundary members. The dimensions of the gussets are determined by equating the buckling shear stress of the equivalent square plate to the shear yield stress of the plate material. The gusset area represents the shear zone near the boundary members that yield in shear before buckling. To simplify the analysis, the strip–gusset elements were replaced by equivalent truss elements at an inclination of 45^0. The researchers assumed that the stress versus strain relationship for the equivalent truss element is elastic, elasto–plastic, and perfectly plastic (i.e., a tri-linear behavior). The initial yielding and the post-initial yielding modulus of the model were obtained from the strip–gusset element. The equivalent truss element was developed for both welded and bolted connections of infill plate to the boundary members. The numerical model was implemented on some of the specimens tested by Elgaaly et al. (1993). The numerical model was able to simulate the test results accurately. Comparing the bolted shear wall with welded shear walls, the researchers stated that, because of slippage and local deformation at the connections, a bolted shear wall can have a lower stiffness and initial yielding but the ultimate capacity is comparable provided that no premature failure of the columns or connections occurs.

2.2.11 Kulak et al. (1999)

Kulak et al. (1999) conducted a numerical study of an eight-storey steel plate shear wall building to investigate the seismic performance of the system. The shear wall had a width of 8 m and storey height of 4.5 m in the first panel and 3.6 m in the remaining stories. The design base shear and the vertical distribution of lateral forces on the shear wall were obtained from the National Building Code of Canada (NBCC, 1995). The preliminary design of the shear wall was carried out by single strut idealization of the system as proposed by Thorburn et al. (1983). This gave a panel thickness ranging from 3.33 mm for the bottom panel to 0.66 mm for the top panel. In the order to make the plate thickness more realistic, a thickness of 4.8 mm was used for all panels. A detailed design was then carried out by using a linear static analysis program and the tension-only strip model proposed by Thorburn et al. (1983). The strength design satisfied both wind and seismic drift limits set by NBCC (1995). A response spectrum analysis was then carried out to estimate the effect of higher modes on the vertical distribution of lateral forces. A pushover analysis of the system, conducted using the commercial software DRAIN-2DX, demonstrated that the structure could resist up to two times the NBCC prescribed base shear. This over-strength was largely due to using infill plate thickness of 4.8 mm, which was significantly greater than required. Although most of the plastic deformations occurred in a column at the third storey, a ductility ratio, $\delta / \delta y$, greater than 10 was still obtained for this storey with the structure still carrying more than the NBCC shear for that panel. Therefore, the system showed significant robustness in the storey that was deforming. The researchers also conducted a nonlinear dynamic time history analysis of the tension-compression model by applying 20 scaled earthquake records to the structure. The maximum inter-storey drift ratio in any storey for the suite of 20 earthquakes considered did not exceed 0.009 (compared to a limit of 0.02 specified by NBCC). This small inter storey drift provides protection to the structural and non-structural elements of the building. The maximum calculated storey shear varied from 2.07 to 2.97 times the prescribed NBCC (1995) values. In the most severe earthquake a maximum ductility demand of 1.9 δ_y was calculated, which was only one-fifth of the ductility obtained from the pushover analysis. This demonstrated that a large reserve of energy dissipation exists in the system. Although significant yielding occurred in one of the columns in stories 1 and 3, the yielding did not progress to create a soft storey since the lateral deformation of the storey as well as strains were stabilized by the elastic tension field.

2.2.12 Rezai(1999)

Rezai (1999) conducted the first shaking table test, using a 25% scale model of a 4-storey unstiffened steel plate shear wall. The main objective of the shaking table test was to obtain more information regarding seismic performance of the system. A similar specimen was also tested under quasi-static loading by Lubell (1997). The test specimen consisted of a four-storey one-bay steel plate shear wall with typical storey height of 900 mm and center-to-center column spacing of 920 mm. The beams in the lower three stories were made from S75 8 sections and a stiff S200 34 beam was used at the top storey. The columns were made of B100 9 sections over the full height of the test specimens. The column sizes differed slightly from quasi-static test specimen (a B100 9 section was used instead of a S75 8 section for the quasi-static test specimen) to increase the out–of–plane buckling strength of the columns and to accommodate the installation of lateral braces. Full moment connections were provided at all joints. An infill plate thickness of 1.5 mm was used for all panels. The infill plates were welded to the boundary frame using fish plate connections similar to those used in the Driver et al. (1998) investigation. Stacks of steel plates were mounted to the test specimen at each storey level to provide a 1700 kg dead load at each storey. The specimen was braced in the out–of–plane direction. The fundamental frequency of the shake table specimen with surrounding support frame was obtained as 6.1 Hz in the longitudinal direction. Four different types of earthquake time histories at various intensities were selected as an input to the shake table test. The limited capacity of the shake table prevented attainment of the significant inelastic response in the specimen. The maximum computed tensile principal strain in the infill plate was 65% of the yield strain. Compressive principal strains were about one-third of the tensile principal strains at the center of infill plates. The specimen deformed mainly in the first mode and the contribution of the higher mode was very small. The inter storey drift observed in the specimen during the tests demonstrated the domination of the flexural mode at the top panels. The researchers found that the first natural frequency of the specimen decreased as the intensity of the shaking increased. By comparing the measured displacements during the test with the maximum inter storey drift limitation prescribed in the National Building Code of Canada (NBCC, 1995), the researchers concluded that the design of test specimen would be governed by the drift limitation and not by strength, which is an undesirable situation. Based on this test, the researchers emphasized the importance of accurately estimating the stiffness of steel plate shear wall systems and the need for a reliable analytical tool.

2.2.13 Lubell et al. (2000)

Lubell et al. (2000) conducted a series of experimental and numerical investigations on quarter-scale models of unstiffened steel plate shear walls. The experimental program consisted of two single-storey (SPSW1 and SPSW2) and one four-storey specimen (SPSW4) under cyclic quasi-static loading. The single storey specimens represented the bottom storey panel of the four-storey specimen. Column-to-column spacing and beam-to-beam dimensions were 900 mm, resulting in an aspect ratio of 1 for all the panels. All material was hot-rolled. Beam and columns were S75 8 sections and the infill plate thickness was 1.5 mm. In specimen SPSW2 an additional S75 8 top beam was welded along adjoining flange tips to better anchor the tension field at the top of the panel whereas in SPSW4 specimen a S200 34 beam was used at the top. Rigid beam–to–column connections were provided for all joints. Specimen SPSW1 was fabricated with no special precaution to eliminate frame and plate distortion due to welding and, as a result, initial out–of–plane deformations up to 26 mm (15 times the plate thickness) were measured in the infill plate. All the specimens were tested under quasi-static cyclic conditions. The load history followed the procedures recommended in Applied Technology Council (1992) guidelines. The single storey specimens were loaded with a horizontal cyclic load at the top of the panel. Specimen SPSW4 was loaded with equal horizontal cyclic loads at each floor level and a constant gravity load of 13.5 kN used at each floor. Gravity loads were applied using by steel masses attached to the test specimen. Well defined elastic–plastic load deformation envelopes, high initial stiffness, good displacement ductility, and stable S-shape hysteresis behaviour were observed in the experiments. Specimen SPSW2 showed significant improvement in stiffness and capacity relative to SPSW1, mainly due to the stiffer storey beam and, to some extent, the reduction in the out–of–plane imperfections in the infill plate. The sequence of yielding in the single-panel specimens was yielding of the infill plate followed by yielding of boundary frames whereas in specimen SPSW4 the columns yielded before significant yielding in the infill plates. The less desirable behaviour observed in SPSW4 was attributed to influence of overturning moments and the small aspect ratio of the panels, which resulted in a state of global instability and termination of the test at a ductility ratio of $1.5\delta_y$.The researchers noticed significant "pull-in" of the columns in all specimens. The inward deformation of the columns reduces the magnitude of the tension field stress near the mid-height of the storey and increases the stress near the horizontal beams at the top and bottom of the panel. In specimen SPSW2, inward column deformation resulted in the formation of plastic hinges at the top and bottom of the columns, with the specimen taking on an "hourglass" shape at the end of test. The "pull-in" effect, which was observed in these series of tests, was discussed by Montgomery and Medhekar (2001). The discussers believed that the specimens tested by Lubell et al. (2000) had inadequate column stiffness and unusual geometric characteristics.

Lubell et al. (2000) also conducted a series of numerical studies to assess the ability of the current simplified analysis technique presented in the Canadian steel design standard, CAN/CSA-S16.1-94, to accurately simulate the behaviour of their test specimens. The simplified model is basically the model proposed by Thorburn et al. (1983), the investigated numerical models were developed using the recommendations in the design standard and were analyzed using nonlinear frame analysis software. Rigid beams were used to simulate rigid floor action.

The capacity of all the test specimens was predicted reasonably well by the numerical models. However, the elastic stiffness was significantly over predicted for SPSW1 and SPSW4 specimens. The researchers argued that the presence of flexural modes caused by the specimen height and the small panel aspect ratio influenced significantly the behavior of the system. An increased overturning moment in the multi-storey specimen resulted in high axial and flexural force effects in the columns and, therefore, altered the inelastic deformation characteristics of the system by changing the yielding sequence in the shear wall (columns yielding prior to the infill plate). The researchers stated that as the height of the steel plate shear wall is increased while keeping the other parameters constant, the flexural action caused by the overturning moment will dominate at the upper stories where the story shear is low. This leads to a condition that is not consistent with the panel shear mechanism assumed by Thorburn et al. (1983). As a result of this investigation the researchers recommended that design standards should require steel plate shear walls to be analyzed as a whole since the analysis of single panel behaviour is significantly different from the multiple panel behaviour. The researchers concluded that the current design guidelines contained in the CSA–S16–01 (CSA, 2001) may not be directly applicable to some steel plate shear walls. Although the recommended procedure in the standard shows a good correlation with the specimen post-yield strength, it may significantly overestimate the elastic stiffness under certain conditions. They also stated that current design standard provisions do not adequately address design issues related to multi-storey shear walls, including the effect of large overturning moments, influence of aspect ratio, and the potential for undesirable yielding sequences of the shear wall components.

2.2.14 Astaneh-Asl et al. (2002)

Astaneh-Asl et al. (2002) conducted two half-scale tests to investigate the cyclic behaviour of a steel plate shear wall system developed by Skilling Ward Magnusson Barkshire of Seattle. The system, which is shown in Figure 2. 8, is a dual system where a coupled unstiffened steel plate shear wall is the primary lateral load resisting system with a ductile moment frame being used as a backup system. Large steel tubes filled with high strength concrete are used for the exterior columns whereas rolled wide flange sections are used for the interior beams and columns. The exterior columns carry a major portion of the gravity loads and contribute significantly to the storey shear resistance. Astaneh-Asl and Zhao tested two specimens. The specimens, shown in Figure 2. 9, were half-scale and representative of a two-storey (specimen 1) and a three-storey (specimen 2) portion of this system. Specimen 1 had an aspect ratio (width–to–height ratio) of 0.67 while the aspect ratio of the panels in the second specimen was 1.0. More details about the specimens can be found elsewhere (Astaneh-Asl et al. 2002). To simulate the boundary condition existing at mid length of a coupling beam, the test specimens were supported on sliding load cells. The specimens were subjected to fully reverse cyclic displacements by applying a single horizontal load at the top level, which increased in a controlled manner in each cycle. Both specimens showed a very ductile behaviour and resisted a large number of inelastic cycles.

Specimen 1 resisted a total of 79 cycles, of which 39 cycles were in the inelastic range. Up to an inter-storey drift of 0.7% the behaviour was elastic. At an inter-storey drift of 2.2% local buckling occurred in the interior columns. At an inter-storey drift of 3.3% and base shear of about 4000 kN, the upper floor-coupling beam fractured at the face of the column. The fracture, which was a result of low cycle fatigue, resulted in a loss of 40% of the load and the test was terminated at this point.

Specimen 2 resisted 29 cycles of loading, of which 15 cycles were in the inelastic range. As for specimen 1, specimen 2 showed an elastic behaviour up to an inter-storey drift of about 0.7%. At an inter-storey drift of 2.2%, when the specimen had reached a base shear of 5451 kN, the upper floor-coupling beam fractured at the face of the column due to low cycle fatigue. At this point the load dropped by about 25% and the test was terminated. Both test specimens demonstrated large ductility. Yielding of the infill plates, beams, and interior columns was found to be the main contributing factor to energy dissipation. In both specimens the coupling beams developed plastic hinges at the face of the columns and fracture occurred only after a large number of inelastic cycles. The concrete filled steel tube column behaved elastically during both tests. The performance of bolted splices was very good and, although they were slipping during the later cycles of the test, they did not fracture. The beam-to-tubular column connections also performed in a ductile manner.

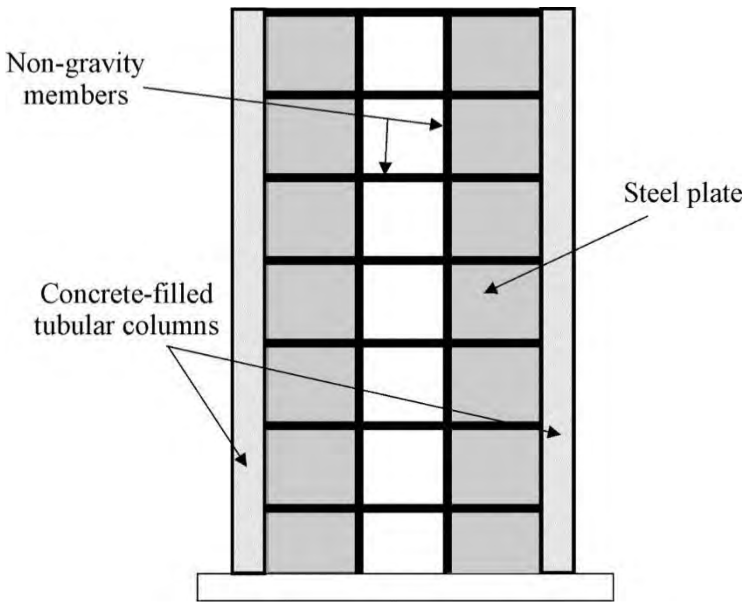


Figure 2. 8: Steel plate shear wall system studied by Astaneh-Asl and Zhao (2002)

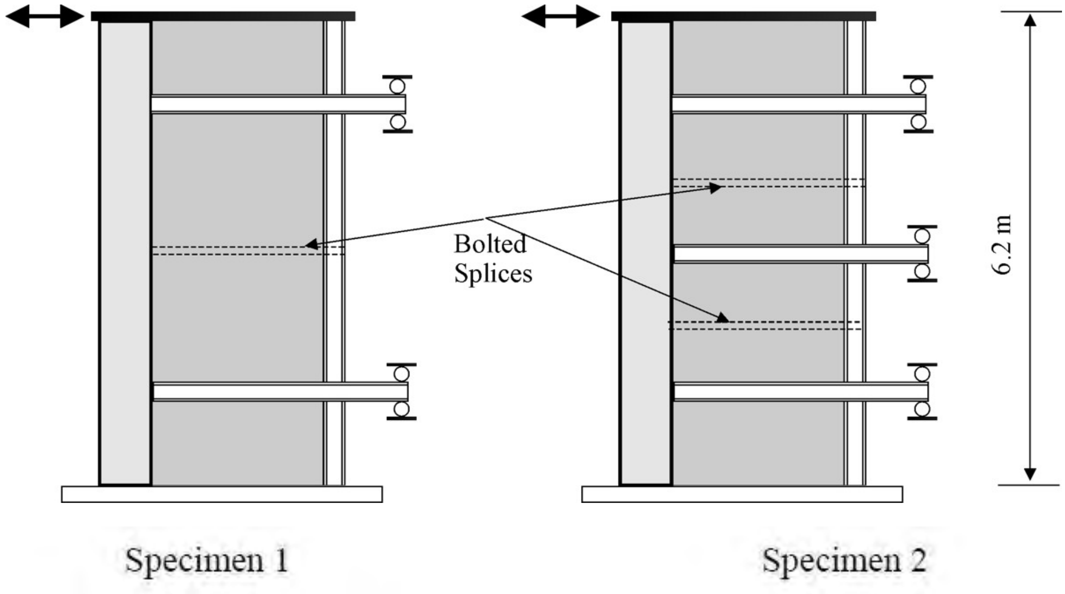
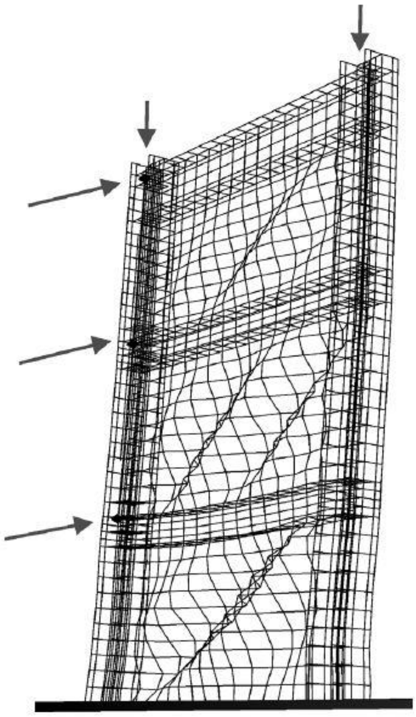


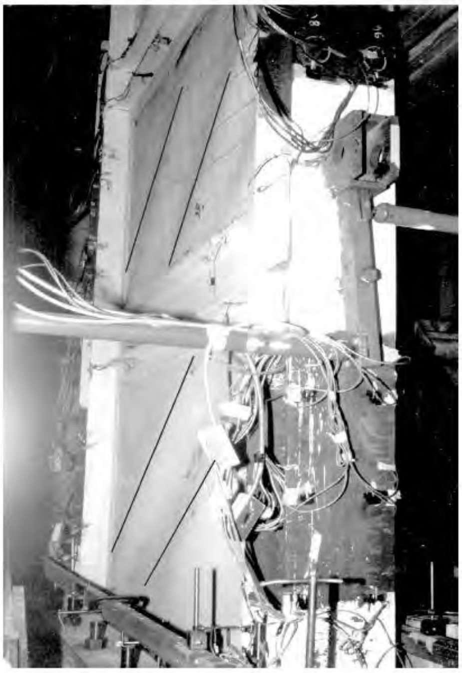
Figure 2. 9: Specimens tested by Astaneh-Asl and Zhao (2002)

2.2.15 Behbahanifard et al. (2003)

Behbahanifard et al. (2003) conducted a test on a steel plate shear wall specimen, which was taken straight from the one tested by Driver et al. (1998a) with the bottom panel removed. Again, excellent ductility, high energy dissipation capacity, stable hysteresis loops and a high degree of redundancy were observed in the test specimen. To further study the behaviour of steel plate shear walls, a finite element model was developed for both monotonic and cyclic response. Behaviour of finite element model and test specimen shows a nice conformity (in Figure 2. 10) .The model result was compared with the test results from both Behbahanifard et al. (2003) and Driver et al. (1998a) and good agreement was found, with a slight underestimation of the predicted capacity. A parametric study was conducted after the validation of the finite element model. Negligible effect was found of varying the aspect ratio of the panel from 1.0 to 2.0 on the behaviour of the panel. The inward displacement of the column was found to be induced by the tension field and then resulted in a non-uniform tension field. It was also found that the imperfections in the panel could have a significant influence on the stiffness of the panel with little effect on the capacity of the panel.



(a) Finite Element model



(b) First and second panels of test specimen

Figure 2. 10: Deformed shape of steel plate shear in finite element model (a) and test specimen (b)

2.2.16 Choi, I and Park, H. (2008)

Choi, I and Park, H. (2008) conducted tests on three steel plate shear walls, one moment-resisting frame (MRF) and one centrically braced frame (CBF) to investigate the ductility and energy dissipation capacities of steel plate shear walls with thin infill plates. Ductile details were used in the specimen to maximize the potential ductility, including full penetration welded connections at beam-to-column joints, ductile fish plate details, and 26 seismic compact column sections. Columns with only 60% of the shear strength for resisting tension field action of the infill panel were used in one of three steel plate shear walls to study the effect of the shear capacity of the columns on the ductility of the steel plate walls, while the MRF and CBF were tested to be compared with the steel plate shear walls. Excellent ductility and great energy dissipation capacity were exhibited in the steel plate shear walls when ductile details were used. The test showed that columns with adequate shear capacity must be designed to resist the tension field action of the infill panel. The research recommended that an idealized tension strip model can be used to estimate the energy dissipation capacity of the steel plate walls.

2.2.17 Dastfan, M (2011)

Dastfan, M (2011) performed experimental and numerical study on steel plate shear wall with PEC (partially encased composite column). Since the boundary frame members play an important role in steel plate shear wall systems by providing proper anchorage to the infill plate, hence their research was focused on the column flexibility parameter, ω_h, and the development of a suitable flexibility parameter to determine the minimum required flexural stiffness of end beams in the top and bottom stories of steel plate shear walls. Dastfan, M (2011) performed an analytical study to develop the end-panel flexibility parameter, ω_L. In order to determine the upper limit of ω_L, an extensive parametric numerical study was conducted. The numerical study covered various parameters like panel aspect ratio (L / h), infill plate thickness and size effect. In order to investigate the effect of the beam to- column connection rigidity on both ω_h and ω_L, a numerical study was conducted on full-scale models with various panel aspect ratios and infill plate thicknesses. Dastfan et al. (2011) also performed an experimental study on the behavior of two large-scale two-story steel plate shear walls with partially encased composite (PEC) columns with built-up H-shaped steel sections. The test specimens were subjected to both gravity loads and lateral loads. Both specimens exhibited large initial stiffness and good ductility and energy dissipation characteristics. In the post-peak stage, the strength degradation was gradual and the behavior of both specimens at large deformations was stable. The PEC columns in both specimens failed at their base in a ductile manner. Another objective of the experimental program was to examine the modular construction method in one of the specimens and the Reduced Beam Section (RBS) connections in the other specimen. No severe problem was detected in the connections and infill plate until the last cycles of the modular test. The presence of the shear connections in the modular test specimen helped the infill plate of the second story enter the inelastic range and thus engaged more in the energy dissipation of the specimen. The fish plates were not damaged, which means that the infill plate could be replaced easily after the occurrence of a severe earthquake. The presence of the RBS connections in the RBS test specimen improved the overall behavior of the specimen as it postponed the formation of plastic hinges at the top of the PEC columns in the first story and thus the formation of a soft story mechanism. The location of the plastic hinge center within the cut was monitored. It was observed that a plastic hinge formed within the RBS cut closer to the column than the cut centerline.

2.2.18 Deng, X. (2012)

Deng, X (2012) conducted experimental and numerical study of a half-size two-storey one-bay steel plate shear wall specimen (Figure 2. 11), with PEC columns as the boundary elements, was tested under vertical and cyclic lateral loads to study its behavior, ductility and performance. Deng, X (2012) developed a finite element model of the specimen loaded in a push-over analysis with a dynamic explicit solution strategy to help study the behavior of PEC columns and the whole system. The failure mode of the test specimen was the initiation of tears at the outside column flange tips at the bottom of the columns during the formation of plastic hinges. The specimen behaved in a ductile manner with no rapid drop of the specimen strength after the ultimate capacity was reached. Compared with steel plate shear walls with a steel frame, more nonlinear behaviors were observed in the specimen due to the existence of the concrete, which led to severely pinched hysteresis curves (Figure 2. 12) without a clear yield portion. Although the energy dissipation capacity did not keep increasing until the end of the test, it did increase beyond the value observed when the ultimate capacity was reached. In general, the model gave good predictions of the overall specimen behavior and internal frame forces.

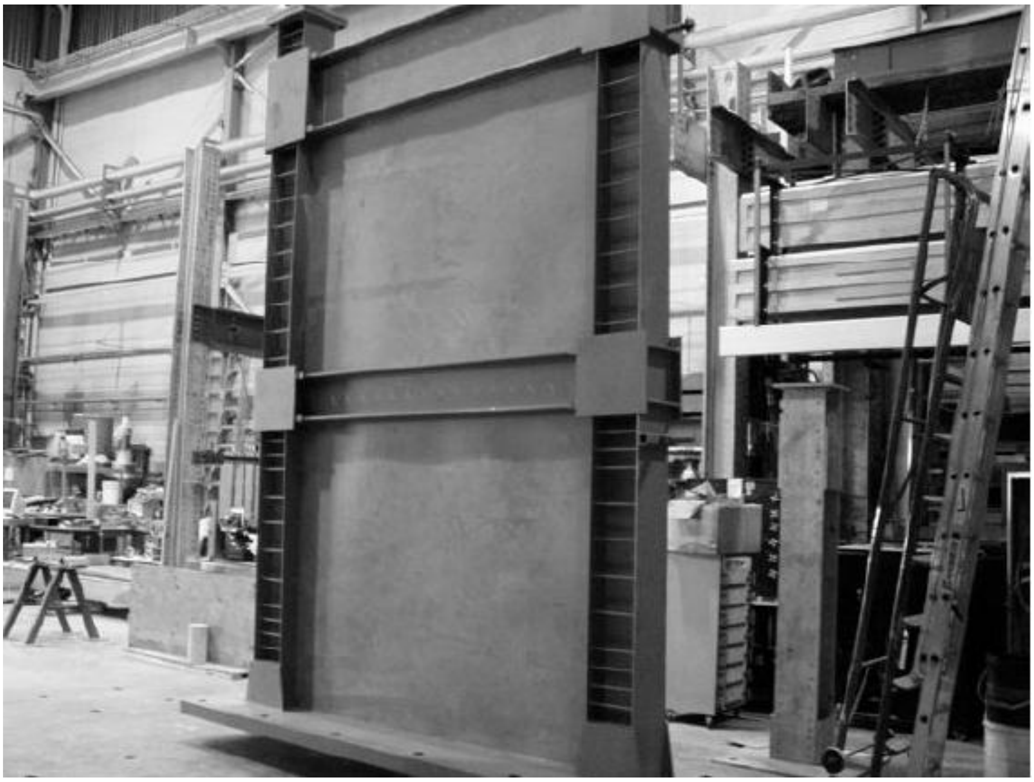


Figure 2. 11: Test specimen of Deng et al (2007).

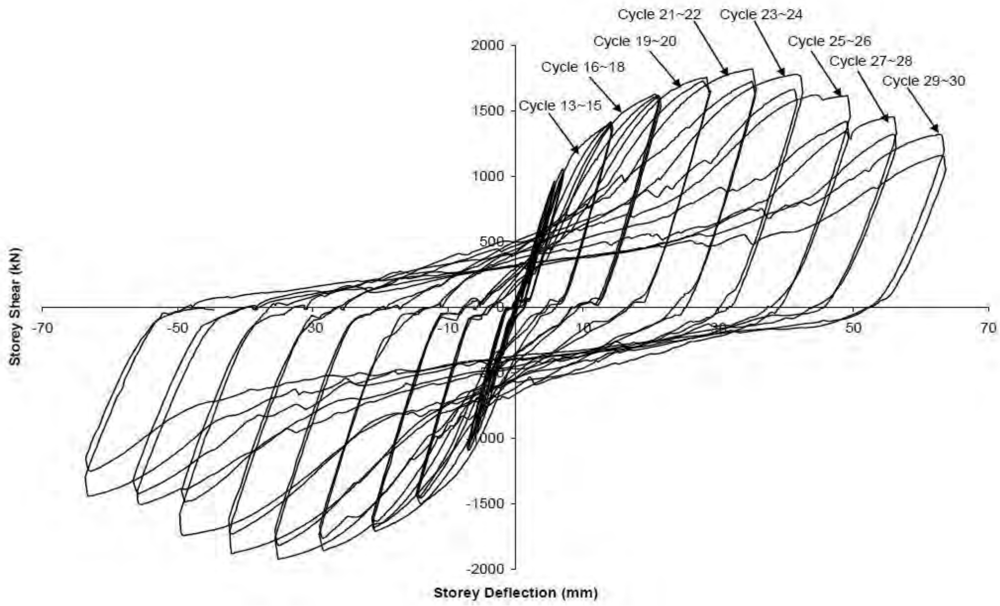


Figure 2. 12: Hysteretic loops of story shear versus story deflection of first story (Deng et al. 2008)

2.2.19 AISC design guide (2007)

This technical design guide instructions the design of web plate, boundary elements (Horizontal beam and vertical column) and shear wall opening.

According to AISC 341 equation 17-1, nominal strength of web plate (in-filled plate of shear wall) can be calculated as,

$$V_n = 0.42 F_y t_w L_{cf} \operatorname{Sin}(2\alpha) \quad \text{.......... (2.1)}$$

Thickness of the web plate can also be calculated from equation (2.1)

Angle of inclination can be calculated from AISC 341 equation 17-2

$$\alpha = \tan^{-1} \sqrt[4]{\frac{1+\frac{tl}{2A_c}}{1+th\left(\frac{1}{A_b}+\frac{h^3}{360 I_c L}\right)}} \quad \text{...................... (2.2)}$$

Once web plate design is finalized then design of boundary member needs to be accomplished.

Primary selection of VBE can be made on stiffness requirement given in AISC 341 section 17.4g

$I_c > 0.00307 \frac{t_w h^4}{L}$ ………………………….. (2.3)

On the other hand, recommended minimum stiffness of HBE is same as VBE

$I_{HBE} > 0.003 \frac{\Delta t_w L^4}{h}$ …………………… (2.4)

Here,

Δt_w = Difference in web plate below and above HBE

2.3 Summary

Chronological review on steel plate shear wall shows that extensive experimental and numerical study has been conducted on SPSW system. Large scale experimental study has been conducted by Takahashi et al. (1973), Caccese et al. (1993), Kulak et al. (1994) on both stiffened and unstiffened steel plate shear wall.

A number of analytical study are also conducted. Modified strip model was proposed by Thorburn et al. (1983). Driver et al. (1998) and Behbahinifard et al. (2003) developed finite element model that can accurately predict strength of SPSW system. Behbahinifard et al. (2003) conducted parametric studies on unstiffened SPSW system, his research shows that aspect ratio and plate thickness has negligible effect but column flexibility parameter and initial imperfection has significant effect on steel plate shear wall system.

In spite of extensive research work carried out on stiffened and unstiffened SPSW system, there are some code specific guidelines, (effect of plate slenderness ratio, estimation of inclination angle) on which investigations should carried out such as, and these issues are highly focused on parametric study.

CHAPTER 3

FINITE ELEMENT MODEL OF STEEL PLATE SHEAR WALL

3.1 Introduction

Experimental studies do not cover the full range of cases that might be encountered in practice as it is too much expensive. It is therefore, necessary to develop analytical tools to investigate the behavior of steel plate shear walls with different geometry and loading conditions, thus avoiding the large expense of performing additional tests. The chapter describes the development of a finite element model that can simulate the behavior of steel plate shear walls under cyclic loading. Experimental model of Behbahanifard et al (2003) was adopted as a reference test specimen for finite element modeling with the help of commercial general-purpose nonlinear finite element program ABAQUS (Hibbitt et al. , 2001). This software is well suited for the solution of highly nonlinear engineering problems. It contains an extensive library of elements that can model virtually all geometric boundary conditions.

3.2 Description of Reference Test specimen Behbahanifard et al. (2003)

A schematic of the experimental model of Behbahanifard et al. (2003) are shown in Figure 3. 1. The columns spaced at 3050 mm, center-to-center. The overall height of the shear wall is 5497 mm, with a typical storey height of 1830 mm. The test specimen consisted of the top three storey's of four-storey steel plate shear wall specimen of Driver et al. (1998). The columns were connected to a 3800×800×90 mm steel base plate. Beam and column section used in the experimental models are provided Figure 3. 1.

The infill plates of the experimental system were connected to the boundary member by fish plate connection as shown in Figure 3. 2.

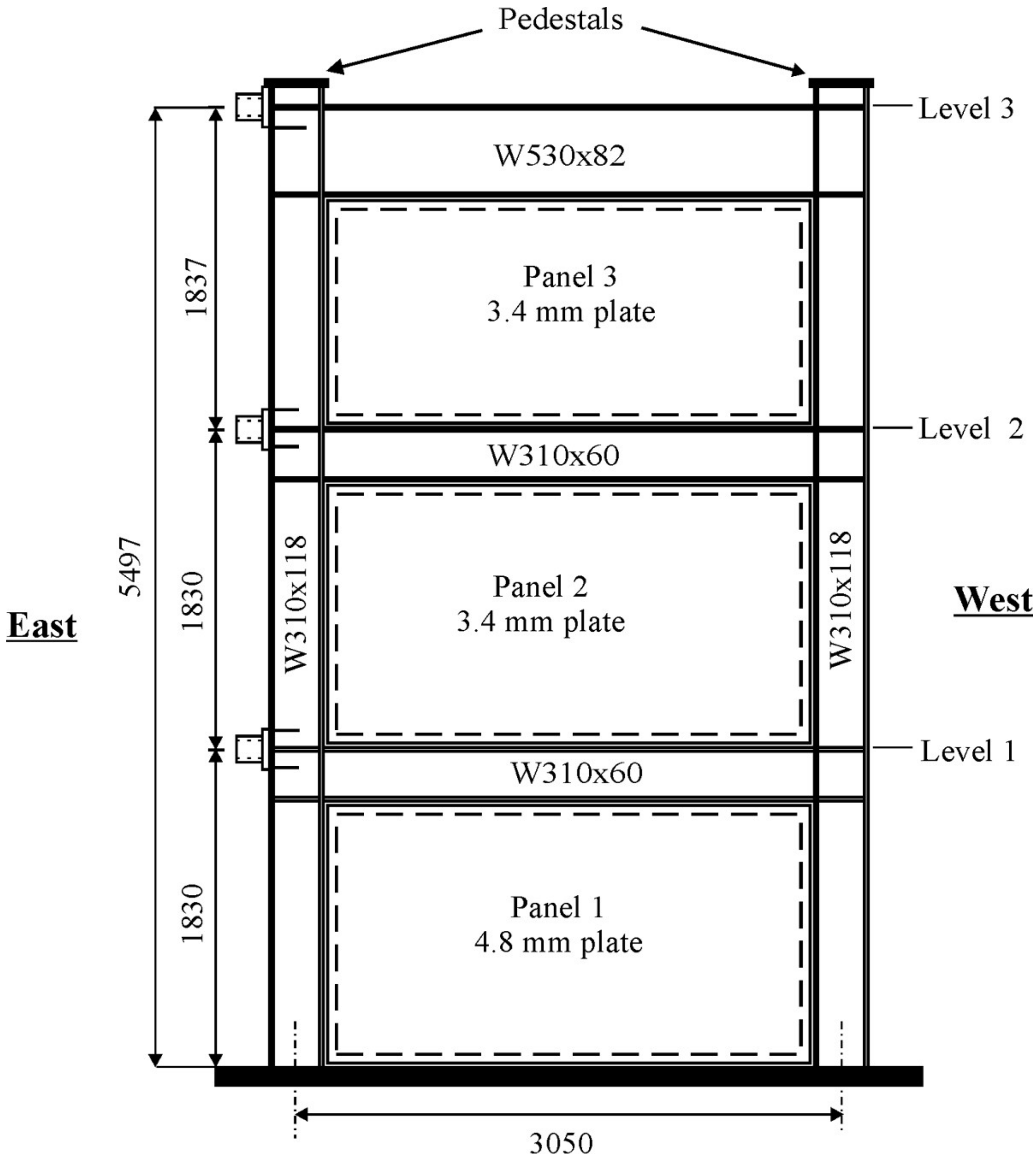


Figure 3. 1: Schematic of three-storey steel plate shear wall (Behbahanifard et al., 2003)

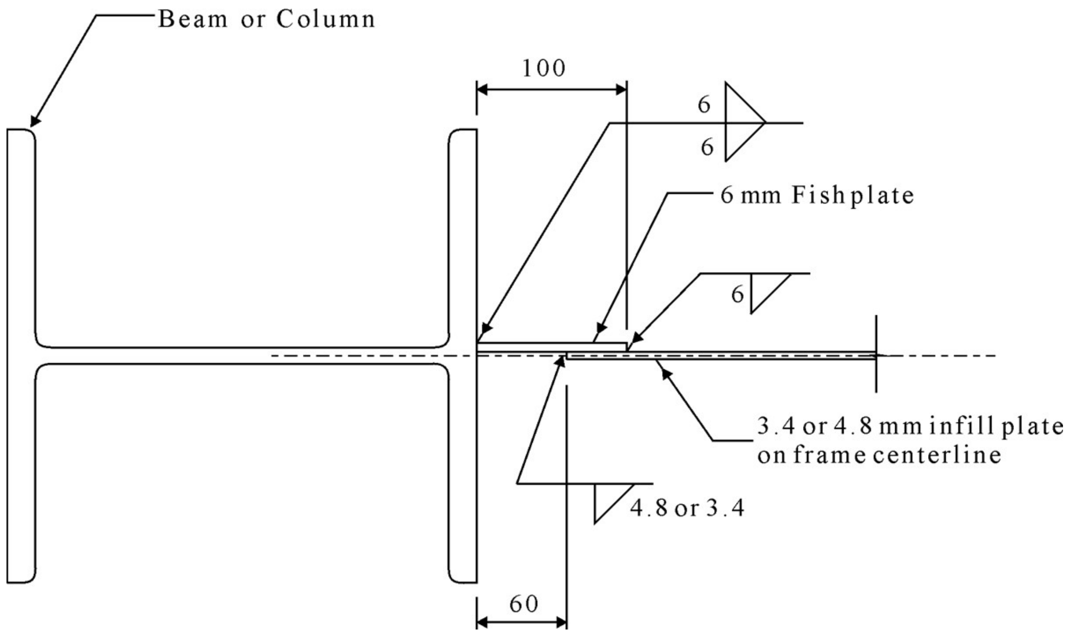


Figure 3. 2: Fish plate detail used for connection of infill plate to the frame
(Behbahanifard et al., 2003)

Gravity loads of a magnitude representing reasonable un-factored values for a typical building were applied at the top of the columns. The gravity load applied to each column was kept constant at 540 kN throughout the test, except during the first three lateral load cycles during which the load was 400 kN. Out–of–plane bracing was provided at the ends of each beam (six locations) as well as at each end of the distributing beam.

Since the friction between the specimen and the concrete floor was not sufficient to prevent in-plane slippage of the shear wall during the test (as reported by Driver et al. (1997), a heavy in-plane bracing member was designed and welded to the specimen at the base plate and connected to the strong wall.

The test was conducted under fully reversed cyclic loading based on the recommendations outlined in ATC-24 (Applied Technology Council, 1992). The document provides guidance on loading history and the presentation of the results for slow cyclic loading tests. Based on ATC-24 a “deformation control parameter” (taken here as some parameter related to inter storey drift) was selected for controlling the test in the inelastic range. In multi-storey buildings, usually the majority of deformation and energy absorption takes place in the bottom storey. The deformation control parameter selected for this test was the inter storey drift in the second panel. It is observed that buckling and yielding of the second panel would initiate slightly before the first panel due to the smaller thickness and lower yield strength of the panel in the second storey. Lateral loads were applied to the test specimen very slowly in order to simulate a quasi-static condition. In each plastic cycle and before recording the data, the target displacement at both excursions were maintained for a while to allow yielding and plastic deformation take place in the specimen. In average about one cycle per day were applied to the test specimen.

Before application of the lateral loads, a gravity load of 400 kN was applied at the top of each column. This load was maintained constant for the first three cycles while the lateral loads were applied to the test specimen. After the third loading cycle, the gravity load in the columns was increased to 540 kN and kept constant for the remainder of the test. The application of the gravity loads did not cause any visible signs of distress anywhere in the test specimen.

Prior to the point of significant yielding, the test was conducted under load control condition. Single loading cycles, resulting in a base shear of ±200 kN, ±400 kN, ±600 kN and three blocks of cycles with ±1000 kN and ±2020 kN were applied to investigate the elastic and early inelastic behavior, which constituted cycles 1 to 9. After that, from cycle 10 to cycle 24 the test was displacement control. In cycle 21, an unexpected failure occurs at the beam column joint. After completing that cycle, the connection was repaired and the remaining cycles were conducted to the full stroke of the hydraulic jacks to obtain the capacity and investigate the behavior of the specimen beyond the peak load. The test was ended after cycle 24. Although the specimen had not yet failed at this time, severe local buckling deformations in the beam at level 1 and in the columns at the base, and the rapid growth of plate tears in panel 1, indicated potential rapid deterioration of the test specimen.

The loading protocol used to conduct the test is mentioned in the study of Behbahanifard et al. (2003). Cycles designated as + or – refer to loading in the west and in the east directions (away from or towards the reaction wall), respectively.

Table 3. 1: Material properties used for analysis of the three-storey steel plate shear wall (taken from Driver et al., 1997)

`component of test specimen	Elastic Modulus (MPa)	Static Yield (MPa)	Static Ultimate (MPa)	Yield Strain %	Hardening Strain %	Ultimate Strain %	Rupture Strain %
W310x118	203000	313	482	0.17	1.41	15.5	26.3
W310x60	203900	332	478	0.19	1.76	16.8	26.2
W530x82	206100	349	493	0.2	1.85	15.5	28.2
Panel 1	208800	341	456	0.18	2.62	20.1	34.2
Panel 2	210900	257	344	0.13	2.44	20	42.5
Panel 3	203100	262	375	0.15	1.53	17.7	34.1

3.3 Description of the finite element model

3.3.1 Element selection

A steel plate shear wall system typically consists of beams and columns with thin steel plate in filled in the openings delineated by the columns and beams. In order to capture local buckling of beam and column flanges, the infill plate and the boundary members were discretized with solid elements. Most of the continuum and plate elements in ABAQUS are based on an updated Lagrangian formulation (Bathe, 1996). This means that at the beginning of each increment the nodal coordinates are updated to reflect current positions in space and all the shape functions and derivatives are re-evaluated using these updated nodal coordinates. This formulation is useful since the deformation magnitude and strains in the infill plates after many cycles are so large that the shape of the shear wall, especially in the first panel, is changed considerably.

The C3D8R solid element was selected from the ABAQUS library of elements to model the steel plate shear wall system. Finite element mesh of steel plate shear wall system is shown in Figure 3. 5. This element is a general-purpose 8-node doubly curved solid element.

The modeling was created in three dimensions using the finite element software ABAQUS 6.7. In the following article the element type used was described.

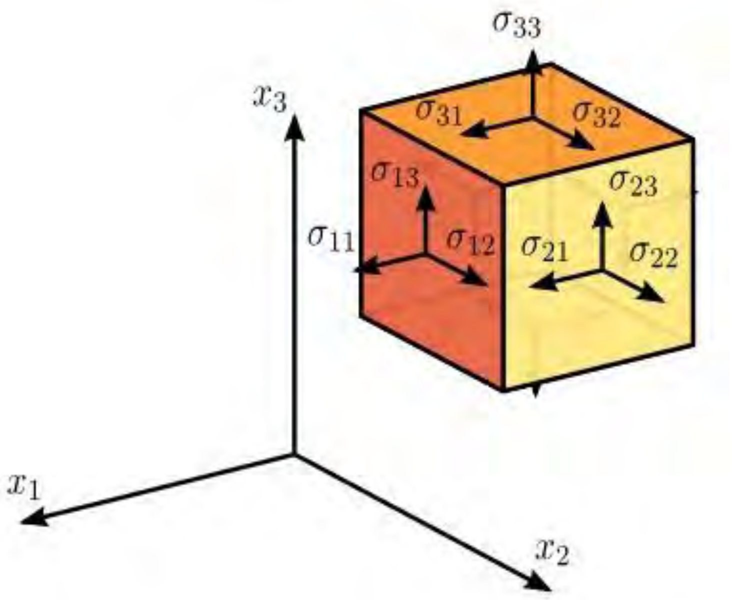


Figure 3. 3: Element used in Finite element analysis.

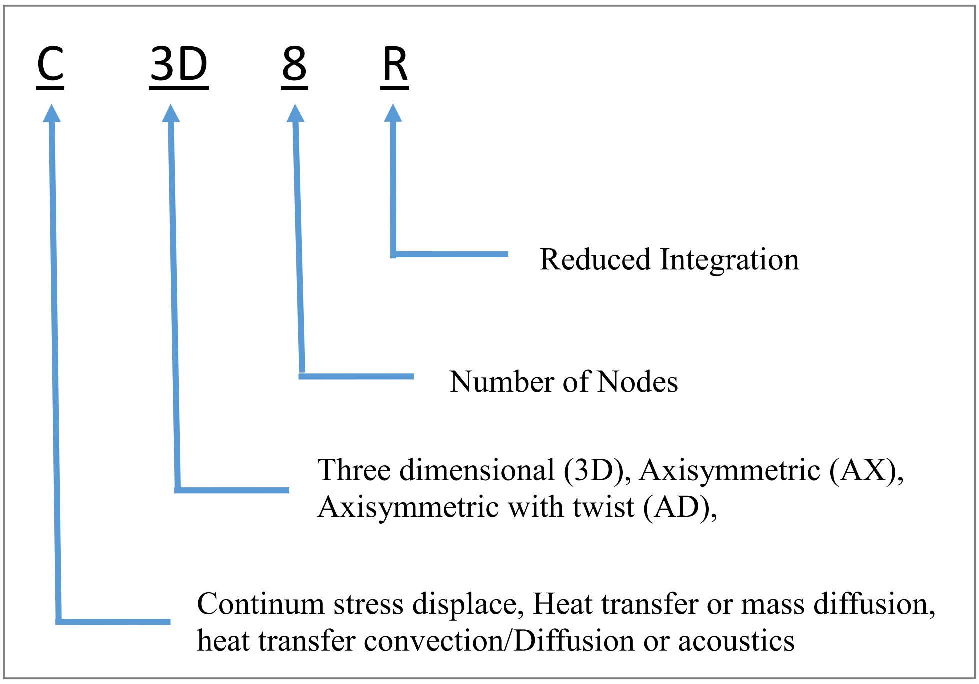


Figure 3. 4: Short overview of C3D8 solid element.

The solid element has eight nodes and six degrees of freedom at each node: Three translation (U_x, U_y and U_z). Hence it has 24 degrees of freedom per element.

The element provide output for six strain components (E_{XX}, E_{YY}, E_{ZZ}, E_{XY}, E_{YZ}, E_{ZX}) and six stress components (S_{XX}, S_{YY}, S_{ZZ}, S_{XY}, S_{YZ}, S_{ZX}). Strain function available for providing output of strain components are 3 principal strains, volumetric strain, max shear strain and octahedral shear strain. Stress function available for providing output of stress components are 3 principal stresses,von mises effective stress, max shear stress and octahedral shear stress. Solid element supports isotropic, orthotropic and laminated composite material model. The 3D Solid (8 node) can be loaded with surface pressures on any or all six faces. Pressure is in units of force/area and may be constant over the element surface or variable and interpolated from values at the corner nodes. A consistent pressure vector is created, which converts the pressure loads into equivalent nodal forces. In addition to surface pressure solid element can take nodal forces, nodal temperatures, body forces (linear and angular acceleration) and prescribed displacements.

Solid element supports static, modal, buckling, steady-state heat transfer, nonlinear static, transient dynamic, and frequency response and shock spectrum

However the 3D solid element is used to model steel plate, flange plate and web of horizontal beam and vertical column of steel plate shear wall.

3.3.2 Mesh Description

Reference test specimen of three storied Steel plate shear system consists of Shear wall, column, Beam and stiffener (at beam column joint). Mesh size of the element should be optimized so that, nodes of SPSW components at the connections are concurs with each other. For example, at beam column joint, element node of the beam end should be connected with element node of the column flange. For this purpose, column flange are partitioned according to the cross section of horizontal beam, along the x-section beam flange is divided into four equal division. Along the portioned face, equal number node is created to ensure proper transfer of between beam column joint in finite element model. However, the approximate smallest element size of the finite element mesh for beam, column steel plate and the stiffener is 0.05 m. The total number of element in the finite element mesh is 23154.

A sensitivity analysis was performed with C3D8R elements to optimize the mesh in order to produce proper representations of local buckling of the steel flange, while maintaining reasonable computing economies.

An eight node linear solid element (Figure 3. 3) is used for modeling steel plate shear wall system. The element can realistically model the connection interface, and allows the application of surface pressure across the cross section.

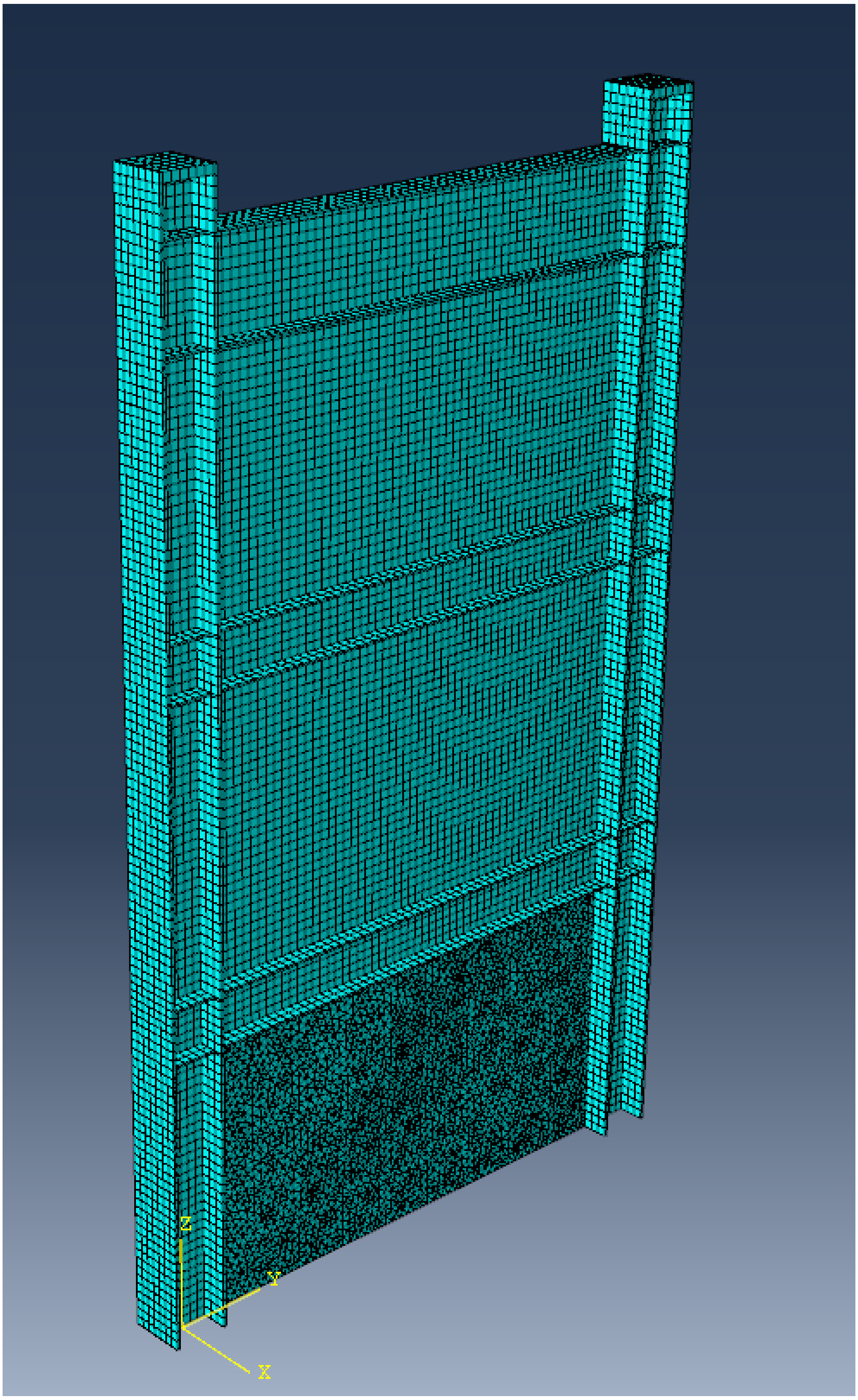


Figure 3. 5: Finite element mesh.

3.3.3 Geometry and initial imperfections

Before conducting the shear wall test, the specimen was measured to determine the as-built dimensions required for the finite element analysis. The imperfections can be categorized as camber and sweep of beams and columns and out– of–flatness of the plate. The camber and sweep of the beams and columns and the column out–of–plumb were considered small and were neglected in the formulation of the finite element model.

The fish plate connection tabs were not considered in the finite element analysis. The assumption that neglecting the fish plate will not affect the overall behavior of steel plate shear wall was shown to be adequate by Driver et al. (1997).

Geometric non-linearity was incorporated to the finite element analysis. The geometric nonlinearity is calculated considering the Lagrangian finite element formulation for large deformations,

The *X*-direction Lagrange normal strain can be expressed as

$$\epsilon_X = \frac{\partial u}{\partial X} + \frac{1}{2}\left(\frac{\partial u}{\partial X}\right)^2 + \frac{1}{2}\left(\frac{\partial v}{\partial X}\right)^2 + \frac{1}{2}\left(\frac{\partial w}{\partial X}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.5)}$$

The behavior of thin plates subjected to in-plane membrane stresses is affected by initial out–of–plane deformations. The stiffness of a perfectly flat plate is very high under in-plane-shear forces, but slight initial imperfections will substantially reduce the in-plane shear stiffness of the plate. In the study, initial imperfections of the infill plates, was considered in the finite element model. The maximum value of the out–of–plane initial imperfection was measured to be 39 mm in the first panel. This pattern was considered as an initial imperfection pattern for the present study. The measured out–of–plane displacement pattern was then mapped onto the finite element mesh in order to get a finite element mesh that accurately modelled the tested steel plate shear wall. Detailed description of measure initial imperfection was described at Behbahanifard et al. (2003).

3.3.4 Material properties

The constitutive relationship in the analysis is based on stress versus strain responses obtained from tension coupon tests of different parts of the steel plate shear wall.

The results of the tension tests from Driver et al. (1997) were used in the numerical model of three storied steel plate shear wall. The steel used in all parts of the shear wall exhibited the classical stress versus strain behavior of hot rolled ductile steel with a well-defined yield plateau. A simple rate independent constitutive behavior that is identical in tension and compression is used. The elasto–plastic kinematic hardening material modelling.

Kinematic hardening model is suitable for simulating Bauschinger effect, and similar responses, where a hardening in tension will lead to a softening in a subsequent compression.

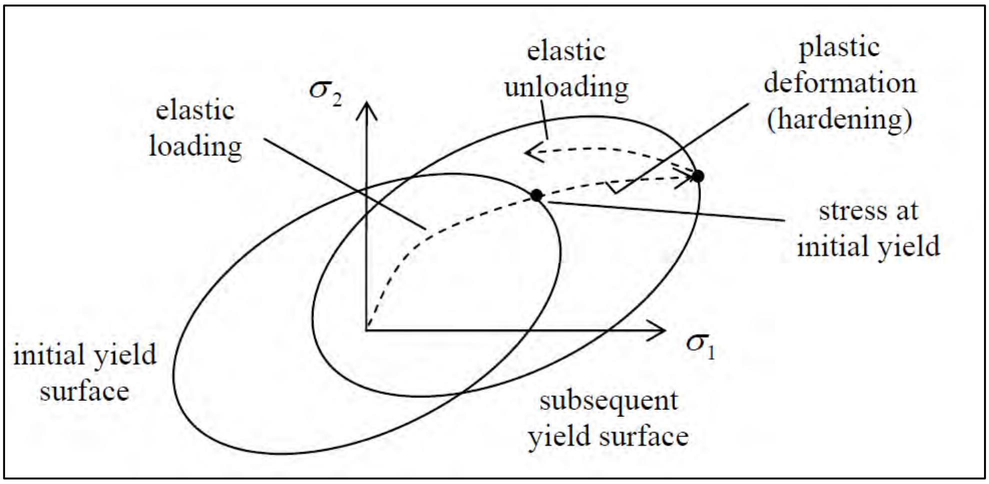


Figure 3. 6: Kinematic Hardening

This is where the yield surface remains the same shape and size but merely translates in stress space Figure 3. 6 . The yield function now takes the general form,

$$f\left(\sigma_{ij}, K_i\right) = f_0\left(\sigma_{ij} - \alpha_{ij}\right) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

The hardening parameter here is the stress α_{ij} , known as the **back-stress** or **shift stress**; the yield surface is shifted relative to the stress-space axes by α_{ij} (Figure 3. 7).

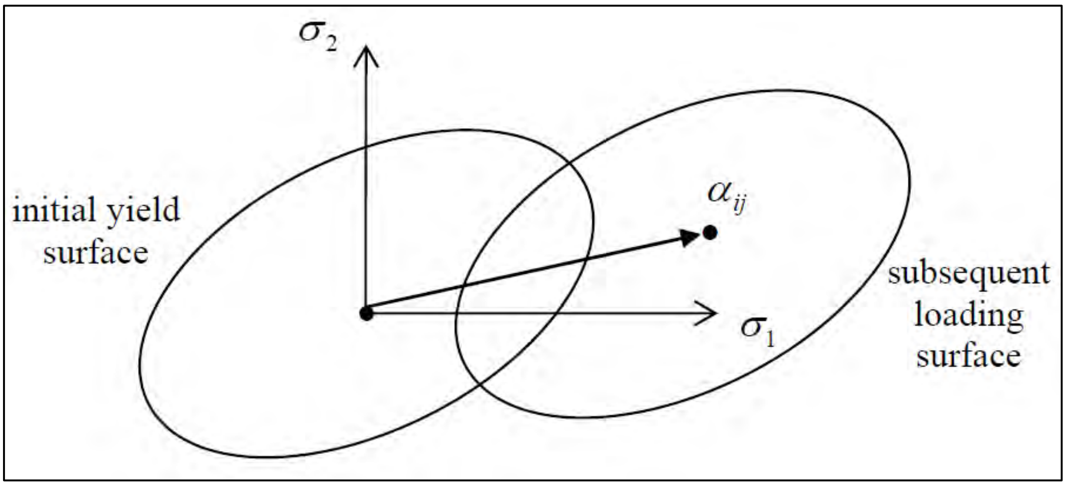


Figure 3. 7: Kinematic hardening; a shift by the back-stress.

Considering the Von Mises yield surface without incorporating the Bauschinger effect. At initial yield,

$$f_0(\sigma_{ij}) = \frac{1}{\sqrt{2}}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2++(\sigma_3-\sigma_1)^2} - Y$$

$$= \sqrt{3J_2} - Y$$

$$= \sqrt{\frac{3}{2}S_{ij}S_{ij}} - Y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.2)$$

Here,

$\sigma_1, \sigma_2, \sigma_3 = Principal\ stress$

S_{ij} = component of deviator stress invariant.

Y = the yield stress in uniaxial tension.

Using the deviatoric part of $\sigma - \alpha$ rather than the deviatoric part of σ,

$$f_0(\sigma_{ij}) = \sqrt{\frac{3}{2}(S_{ij}-\alpha_{ij}^{d})(S_{ij}-\alpha_{ij}^{d})} - Y = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

The bilinear stress versus strain curve is obtained by extending a line from the origin to the mean value of static yield point (the slope is equal to the mean modulus of elasticity) and then to the mean static ultimate stress and corresponding strain. The material properties obtained from a tension coupon test are nominal values, i.e., engineering stress and engineering strain, which are defined in terms of an initial gauge length and initial cross sectional area of the coupon. The finite element analysis uses true stress (Cauchy stress) and logarithmic strain as stress and strain measures regardless of the type of analysis.

To obtain the true stress (σ_{true}) and logarithmic plastic strain (ε_{ln}^{pl}) the following transformations are applied to the tension coupon data (Lubliner, 1990):

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad (3.3)$$

And

$$\epsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (3.4)$$

Where, E is the modulus of elasticity σ_{nom} is the nominal (engineering) stress and ε_{nom} is the nominal (engineering) strain obtained from material tests.

The material models in ABAQUS are based on "incremental" theories in which the mechanical strain increment, $\Delta\varepsilon$, is decomposed into an elastic part, $\Delta\varepsilon_{el}$ and a plastic part $\Delta\varepsilon_{pl}$. An incremental plasticity model usually is formulated in terms of a yield surface, flow rule, and a hardening model. The von Mises yield surface is used in ABAQUS to specify the state of multi-axial stress corresponding to start of plastic flow. This yield surface assumes that yielding of metals is independent of the hydrostatic stress and has the form of a cylinder that is centered on the hydrostatic axis in a three-dimensional principal stress space.

A hardening rule specifies the evolution of the yield surface during plastic flow. In ABAQUS three types of work hardening models are provided for metals: a perfectly plastic model, an isotropic hardening model, and the Johnston-Cook hardening model. In the perfectly plastic model the yield stress does not change with plastic strain and, as a result, no hardening or softening occurs in the material. This model was tried for the pushover analysis as well as for cyclic analysis of the three-storey steel plate shear wall, but was not successful in predicting the post-yielding behavior of the specimen. In the isotropic hardening model the size of the yield surface changes (increases or decreases) uniformly in all directions as plastic straining occurs. The isotropic hardening model in ABAQUS is nonlinear and a full range of effective plastic stress versus effective plastic strain can be defined. The Johnston-Cook hardening model is a particular type of isotropic model. In this model the yield stress is defined as an analytical function of effective plastic strain, strain rate, and temperature. This hardening rule is suitable for modelling monotonic high rate deformations of most metals. The isotropic hardening model was used only for the pushover analysis of the shear wall. However, cyclic loading of the test specimen implies many strain and stress reversals occur during the process. The Bauschinger effect becomes important and should be considered in the model. The kinematic hardening flow rule is intended to simulate the behavior of metals subjected to cyclic loading and is typically applied to studies of low cycle fatigue. In this model the basic concept is that the yield surface translates in stress space without any rotation or changes in size. This means that yielding in one direction reduces the yield stress in the opposite direction, thus simulating the Bauschinger effect and anisotropy by work hardening.

3.3.5 Solution strategy adopted in the finite element model

Finite element model has been developed using two different solution strategy, the first one is non-linear static analysis which is used to for simulating non-linear cyclic load deflection response and the second is modified riks analysis, this solution technique allows to generate pushover curve which is actually envelope of hysteresis curve. A brief description of these solution strategy are presented in short.

3.3.5.1 Nonlinear static analysis

For the purpose of cyclic analysis of three storied SPSW test specimen, "Nonlinear static" solution technique is adopted. A static stress analysis, is used when inertia effects can be neglected; can be linear or nonlinear; and ignores time-dependent material effects (creep, swelling, viscoelasticity) but takes rate-dependent plasticity and hysteretic behavior for hyper elastic materials into account. However, creep (result of long-term exposure to high levels of stress that are still below the yield strength of the material) is not important dynamic loading and visco - elasticity of steel is not predominant in room temperature.
Again, the solution technique supports, geometric, material and boundary non-linearity which is able to capture the large- displacement effect of steel plate shear wall.

3.3.5.2 Static riks analysis

Riks method is suitable for predicting buckling, post-buckling, or collapse of certain types of structures, materials, or loading conditions, where linear or eigenvalue method will become inadequate or incapable, especially when nonlinear material, such as plasticity, is present, or post-buckling behavior is of interest.

In nonlinear static analysis for buckling, post-buckling, or collapse behavior, the tangent stiffness from the load-displacement response curve could change signs when system changes its stability status as shown in Figure 3. 8. The classical Newton's method will not work in this situation because the corrections for approaching equilibrium solutions during iterations may become difficult to determine when the tangent stiffness is close to null. There are different approaches to solve such problems, such as switching to dynamic analysis, using displacement controlled static analysis, or adding dashpots for stabilization during sudden strain energy release. But those methods are not without limitations in such aspects as high computational cost, non-unique responses due to jump phenomenon. Alternatively, static equilibrium states during the unstable phase of the response can be found by using the "Riks method" developed by Riks (1979) later modified by Ramn (1981). This method is used for cases where the loading is proportional; that is, where the load magnitudes are governed by a single scalar parameter. The basic Riks algorithm is essentially Newton's method with load magnitude as an additional unknown to solve simultaneously for loads and displacements, thus, can provide solutions even in cases of complex and unstable response

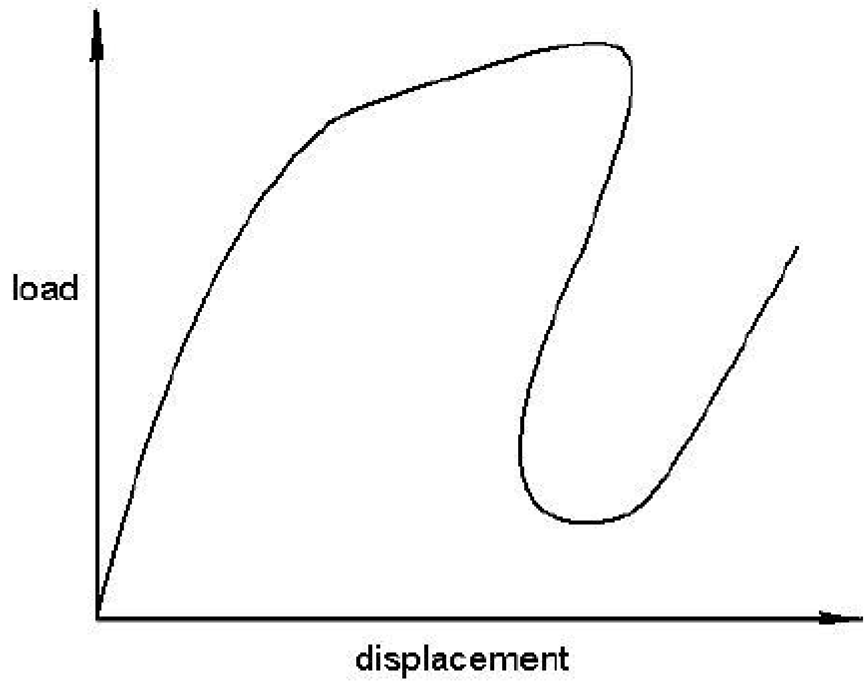


Figure 3. 8: A typical unstable response curve.

3.3.6 Boundary conditions and loading of finite element model in static cyclic analysis

In order to simulate the rigid boundary at the base of the shear wall the nodes at the base of the steel plate shear wall model are made fully fixed. Additional Boundary conditions are provided at the column to resist the out of plane buckling. However the boundary condition and the applied loads are provide along the surface to prevent hourglass deformation.

Gravity load is made active throughout the cyclic loading. A constant axial load of 400 kN magnitude is applied at the column at the first two cycle, however from the third cycle it is increased to 530 kN.

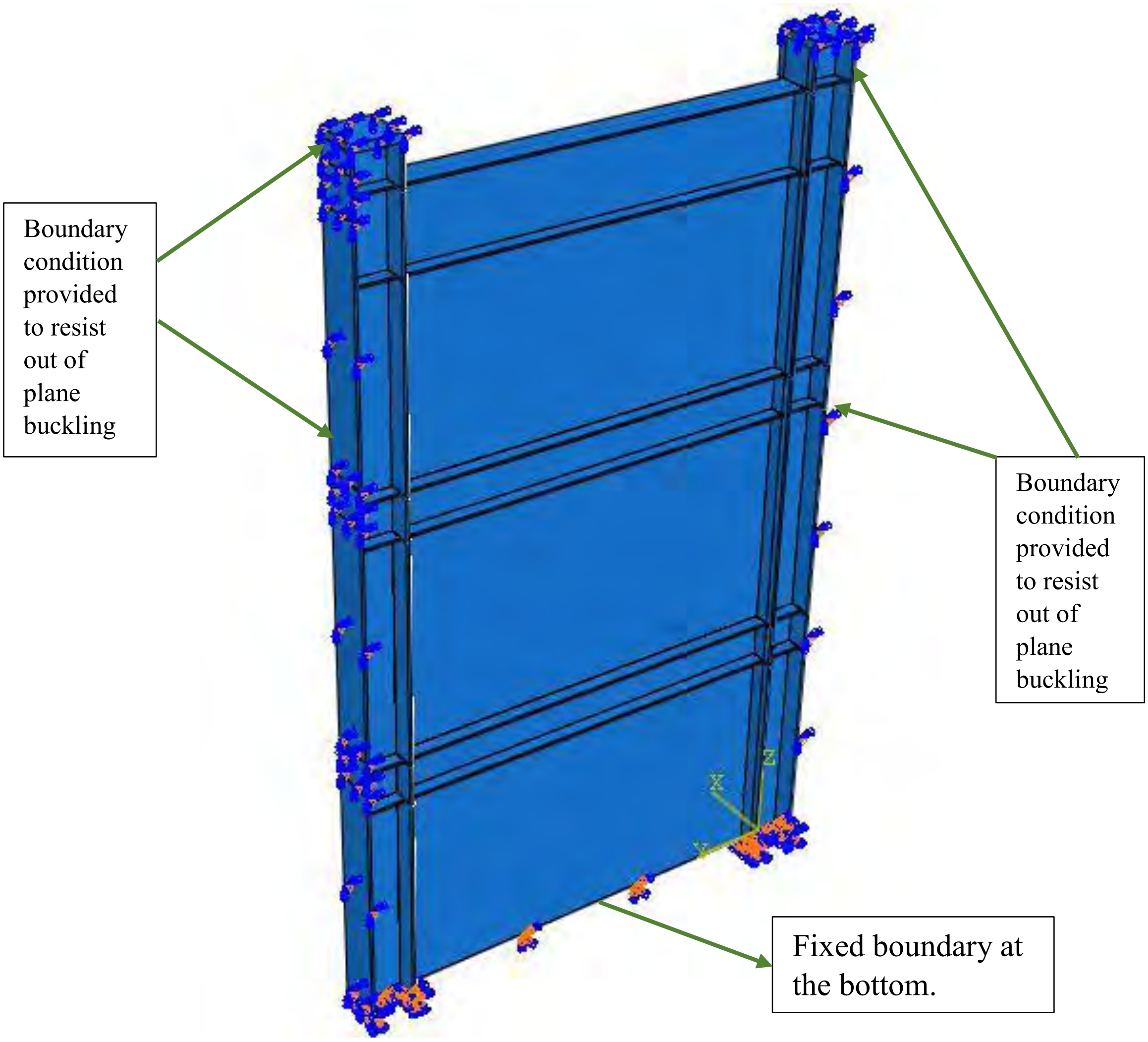


Figure 3. 9: Boundary Condition applied to the finite element.

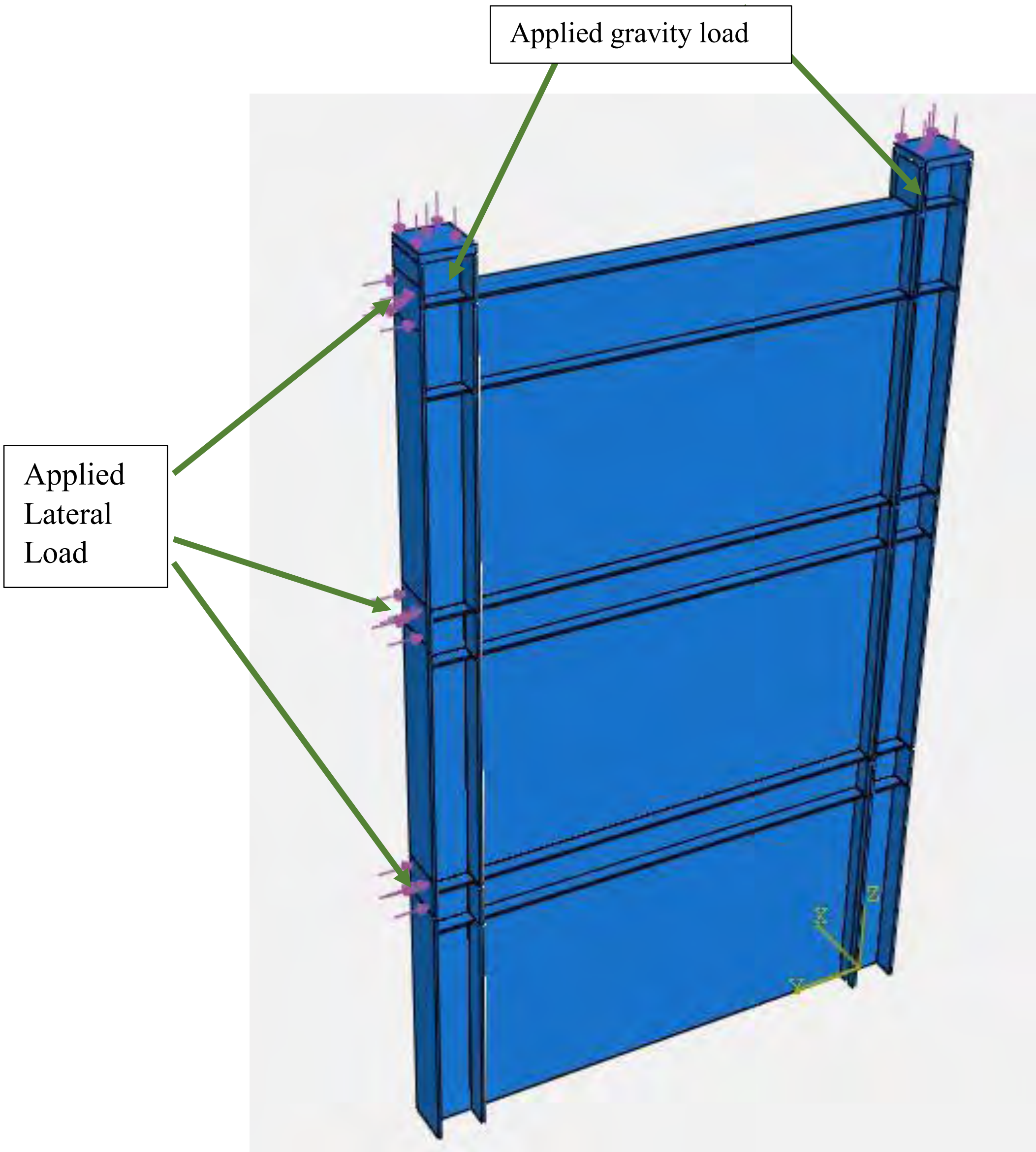


Figure 3. 10: Applied load on the finite element model.

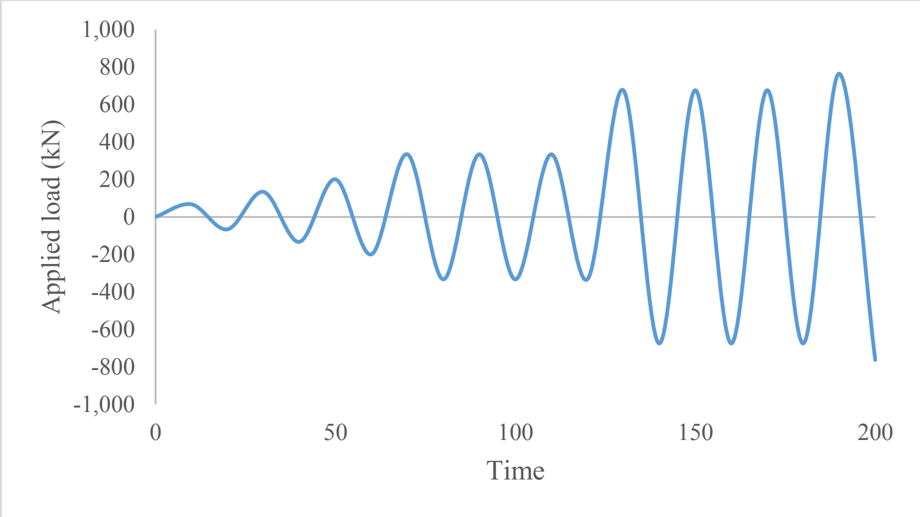


Figure 3. 11: Applied time history of load in each storey level.

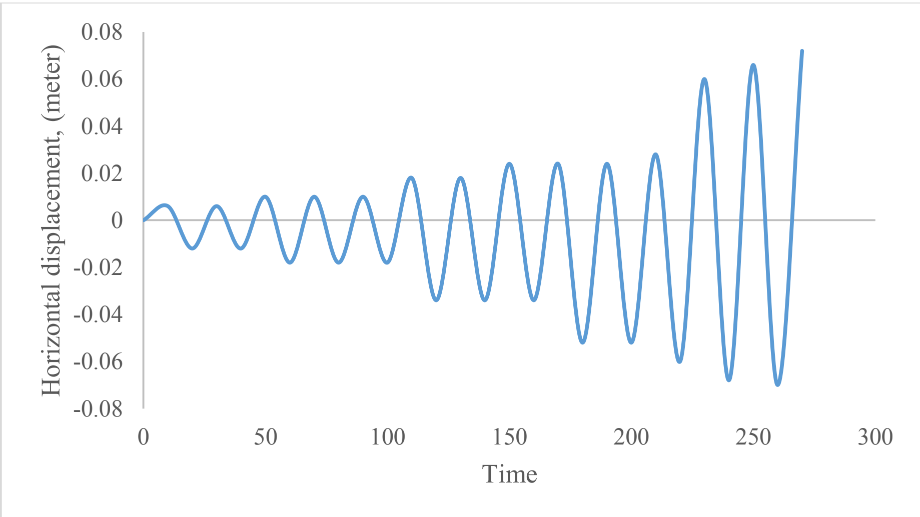


Figure 3. 12: Applied history of horizontal displacement in first storey.

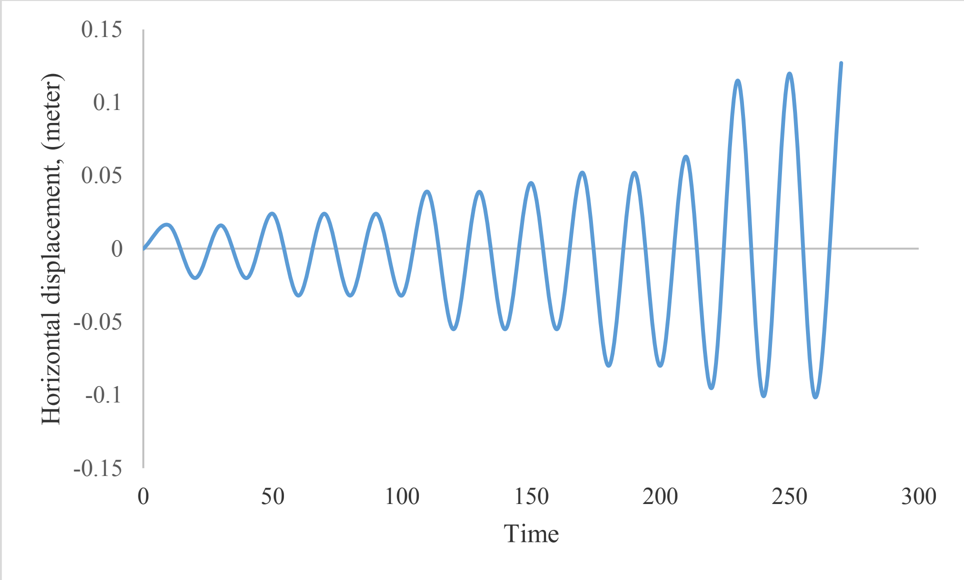


Figure 3. 13: Applied history of horizontal displacement in second storey.

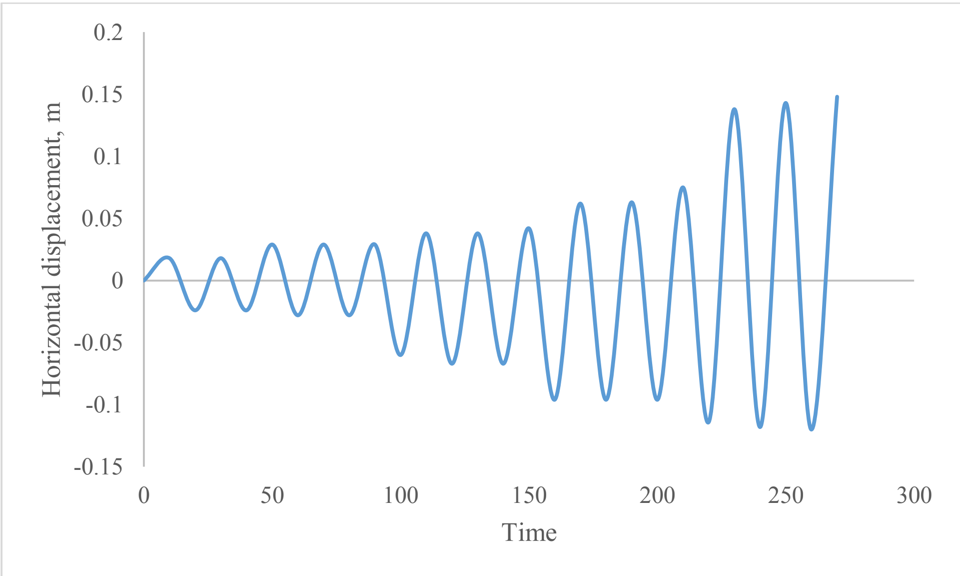


Figure 3. 14: Applied history of horizontal displacement in third storey.

The total analysis step time is 470. Likewise the experimental setup the cyclic load application in the finite element model is incorporated in two steps. The first step (step time allocated 200) contains nine cycles and it is load controlled. Resulting base shear is equally divided and applied in three different stories (Figure 3. 10 and Figure 3. 11). Load was applied in each storey level as a surface pressure.

Again the second step is deflection control and 270 step time is allocated. Applied deflection cycle in three different storey is shown in Figure 3. 12, Figure 3. 13 and Figure 3. 14.

3.3.7 Boundary conditions and loading of finite element model in static riks analysis

Ultimate base shear sustained by test specimen (three storied steel plate shear wall) is 3500 kN. The base shear is divided is three equal surface pressure of 9.9×10^6 N/m^2. With constant gravity load applied at the column, the finite element model is analyzed under modified riks strategy. From modified riks analysis load vs. deflection response is obtained, which is actually envelope of hysteresis analysis. A gravity load of 540 kN was applied to the top of each column in the first load step. This magnitude is equal to the target gravity load used in the physical test and was kept constant for the remainder of the analysis.

CHAPTER 4

PERFORMANCE EVALUATION OF FINITE ELEMENT MODEL

4.1 Introduction

A perfect analytical model of steel plate shear wall must be able to predict the capacity of the system, its behavior under loading conditions and its failure pattern. This chapter hereby describes the efficiency of finite element model of SPSW to response the cyclic hysteresis and static pushover behavior. Therefore a direct comparison of cyclic and static load deflection response of the experimental system and the numerical model are presented. Failure behavior of finite element model is also compared against the damage of the reference test specimen.

4.2 Finite element analysis of the three-storey steel plate shear wall

4.2.1 Pushover analysis

To determine how accurately the proposed finite element model is able to predict the stiffness and the capacity of the three-storey steel plate shear wall specimen, a pushover analysis was carried out using the finite element model of the specimen described in Chapter 3. For pushover analysis procedure static riks solution strategy is adopted. The behavior of simulated test specimen is characteristics of thin unstiffened steel plate shear walls and it was observed during the test when the development of the tension field was accompanied by loud reports and rapid out–of–plane deformations in the infill plates.

The base shear versus horizontal displacement at the top storey is shown in Figure 4. 1. Figure 4. 1 to Figure 4. 2 all indicate that the finite element model predicts the stiffness of the shear wall very well in storey levels. The higher predicted stiffness in the third panels is attributed to the fact that the applied loads were maintained horizontal in the finite element model, whereas the loads applied to the test specimens rotated slightly as the test specimen deformed, and reduces the stiffness of the panel. Another reason for the slight overestimation of the wall stiffness could be the effect of residual stresses, which are ignored in the finite element model. In addition to the response of the steel plate shear wall, the response of the bare frame is also shown in Figure 4. 1 to Figure 4. 4. The gradual post-ultimate strength degradation exhibited by the test specimen is not observed in the finite element model because the cracks and tearing of the shear wall were not included in the model.

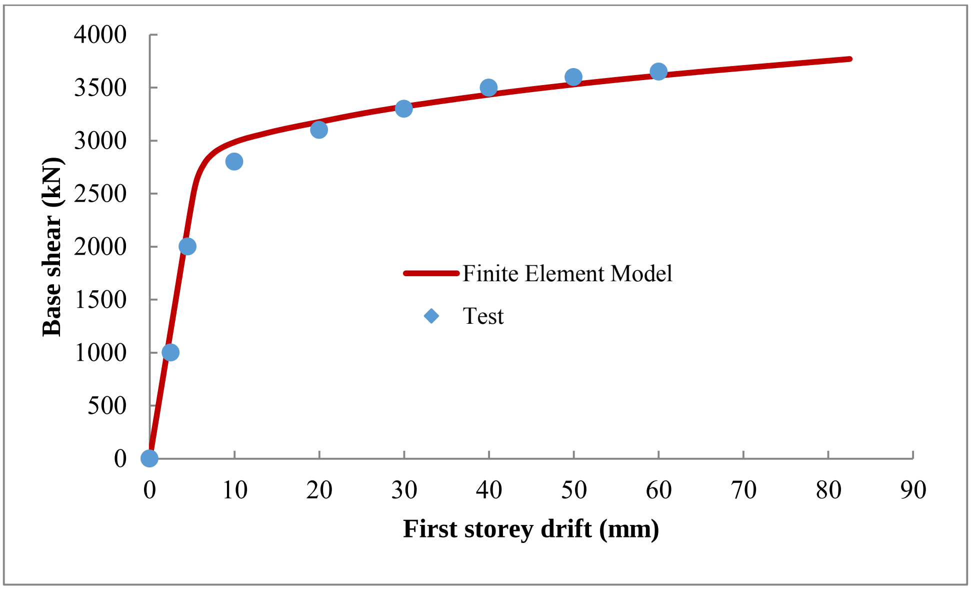


Figure 4. 1: Monotonic finite element analysis compared with the envelope of test cyclic response (Base shear vs. First storey drift)

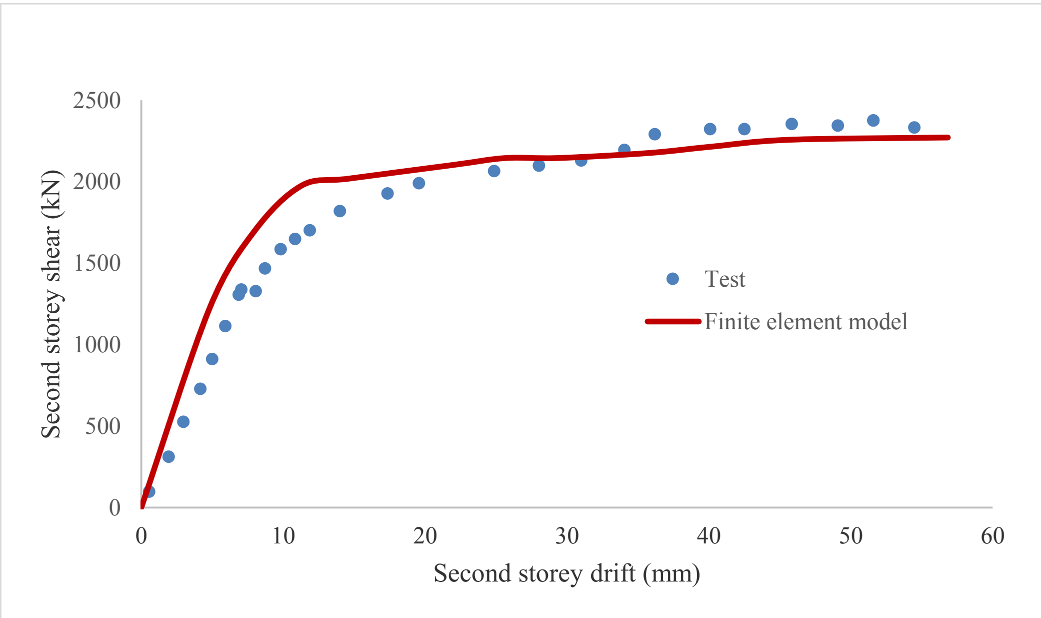


Figure 4. 2: Monotonic finite element analysis compared with the envelope of test cyclic response (second storey shear vs. second storey drift)

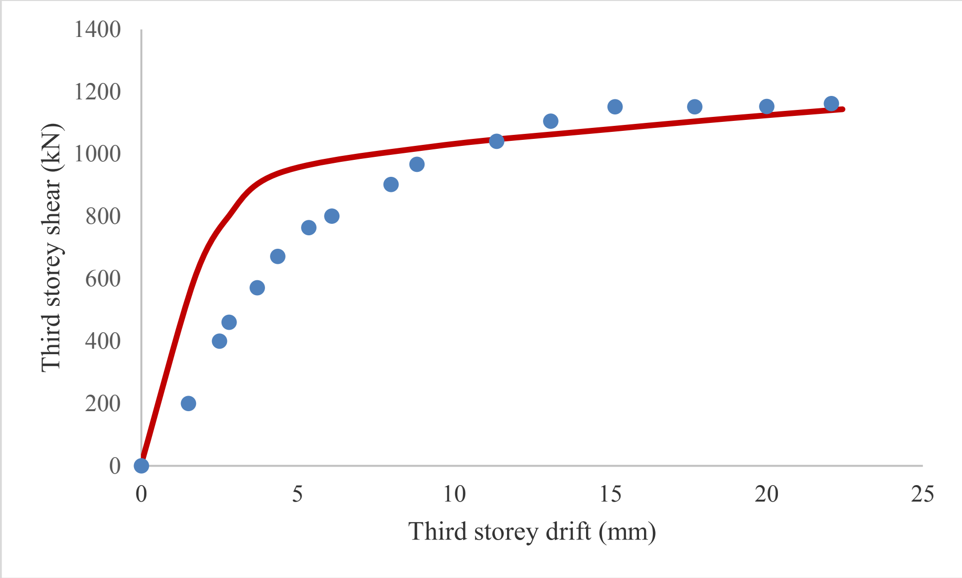


Figure 4. 3: Monotonic finite element analysis compared with the envelope of test cyclic response (third storey shear vs. third storey drift).

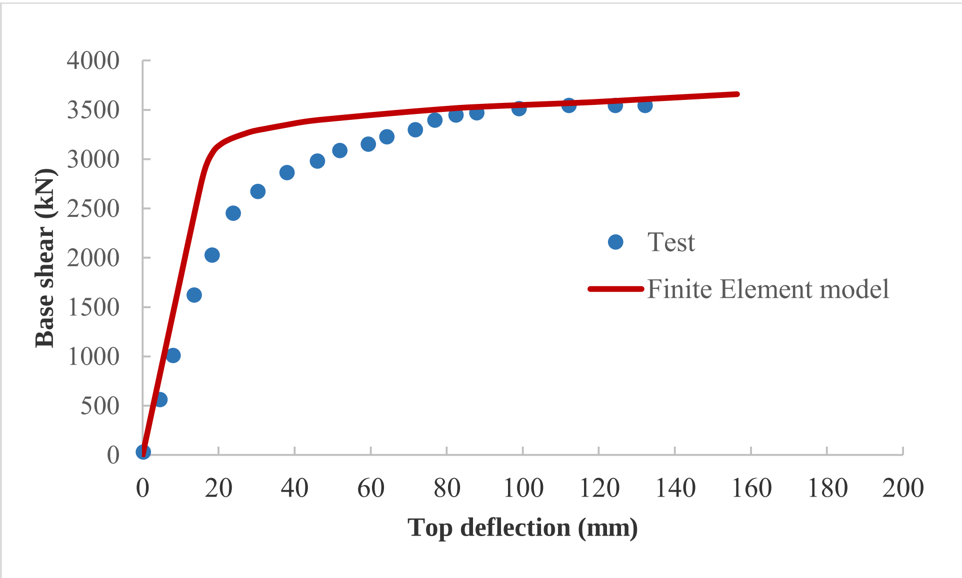


Figure 4. 4: Monotonic finite element analysis compared with the envelope of test cyclic response (Base shear vs. Top storey displacement).

4.2.3 Cyclic analysis

A pushover analysis provides an estimate of the stiffness and capacity of a steel plate shear wall as it captures closely the envelope of cyclic response of a system. However, to evaluate the energy dissipation characteristics and the efficiency of a steel plate shear wall under cyclic loading, the finite element model should be able to simulate accurately the cyclic response of the system. Hysteresis loops obtained from the finite element analysis of the three-storey steel plate shear wall are compared with the hysteresis loops obtained from the test results in Figure 4. 5 to Figure 4. 8 for each panel and the displacement at the top of the shear wall. In general, there is good agreement between the test and the finite element analysis.

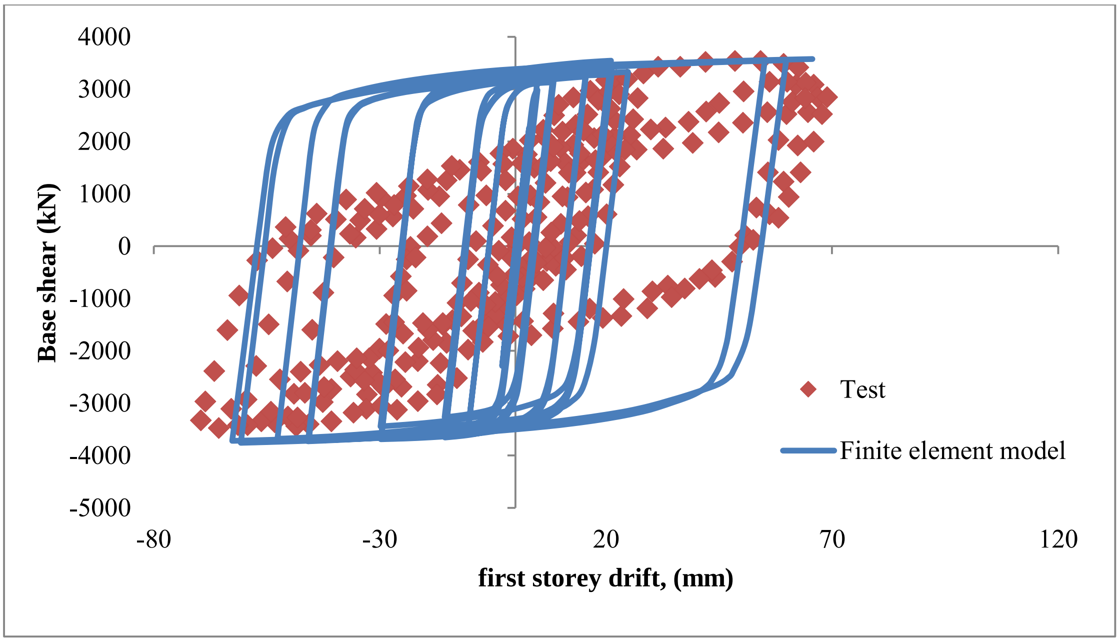


Figure 4. 5: Comparison of finite element hysteresis with test results (Base shear vs. First storey drift).

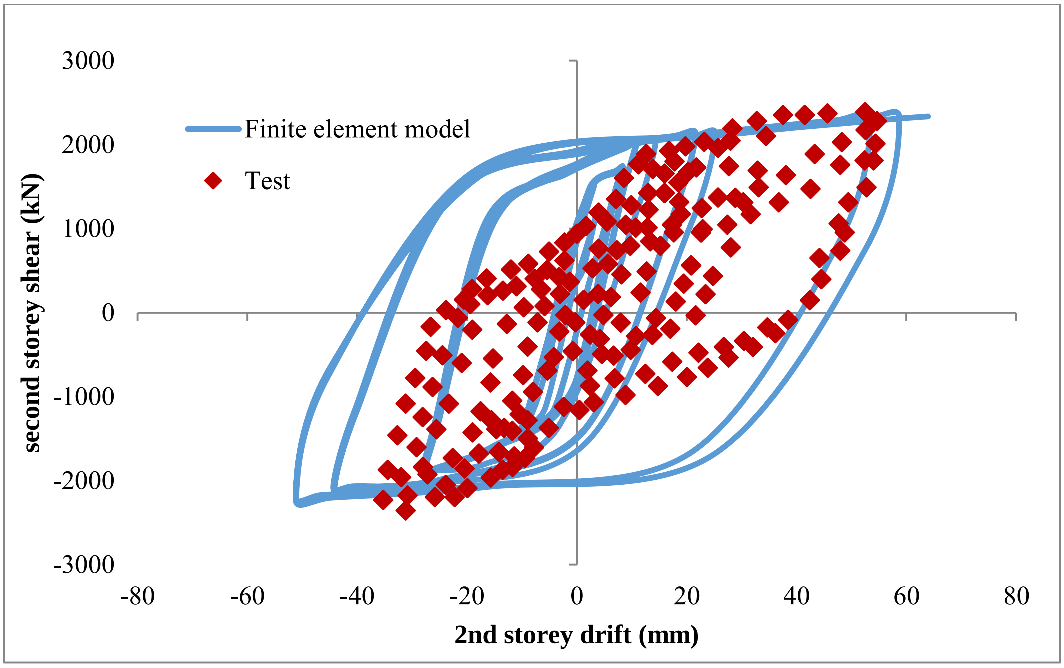


Figure 4. 6: second storey shear vs. second storey drift

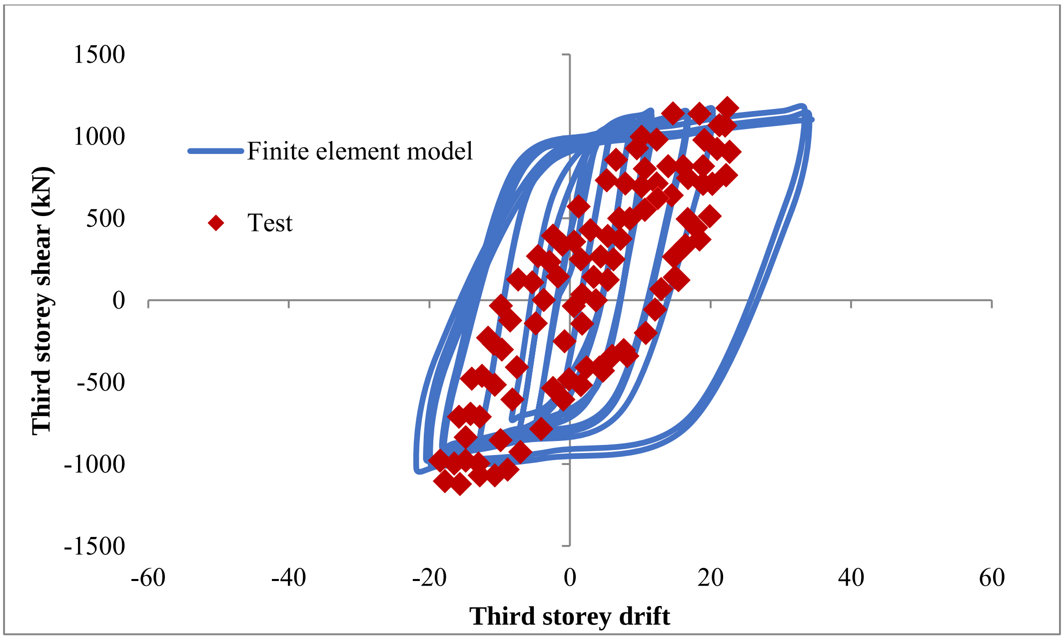


Figure 4. 7: Third storey shear vs. third storey drift

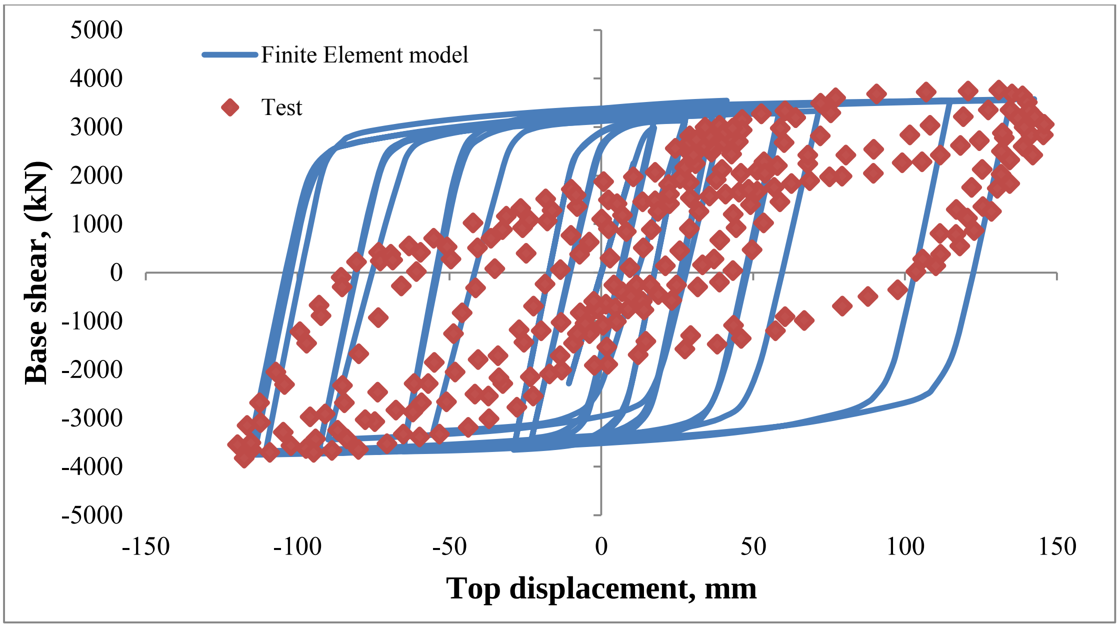
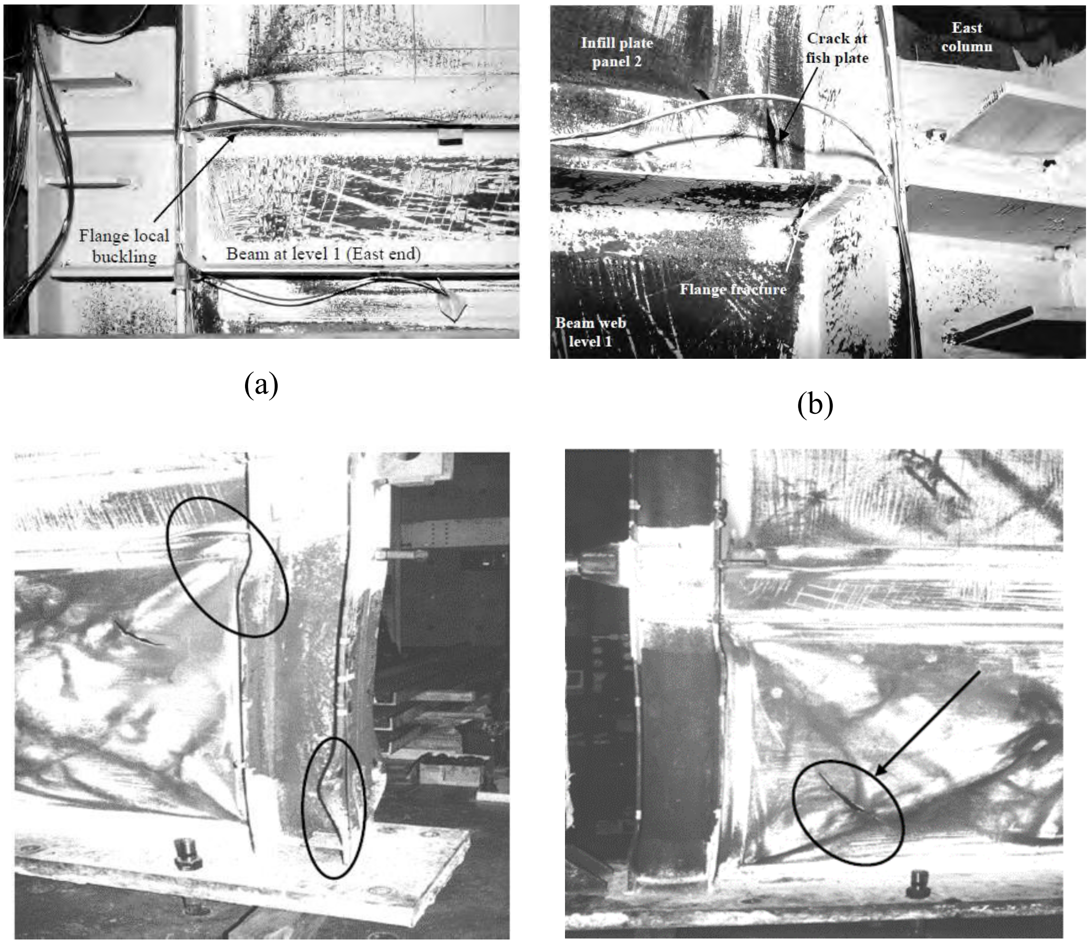


Figure 4. 8: Comparison of finite element hysteresis with test results (Base shear vs. Top storey displacement).

4.2.4 Failure pattern comparison of finite element model and experimental model

This section makes a qualitative comparison between the failure pattern of experimental and finite element model. Failure in the experimental model is initiated with yielding of in-filled steel plate shear wall. Shear wall yielding first initiated from cycle 7, after that minor local buckling is observed in the web of horizontal beam at cycle 10. Significant flange buckling is observed at the beam end during cycle 20 [Figure 4. 9 (a)]. Fracture at the beam column joint is visible right after the 20th cycle [Figure 4. 9 (b)]. At the end of the test severe local buckling is noticed near the base of the column in addition to that tearing is observed at the shear wall [Figure 4. 9 (c) and (d)].



(a) (b)

(c) (d)

Figure 4. 9: Failure pattern of experimental model (Behbahanifard et al. 2003).

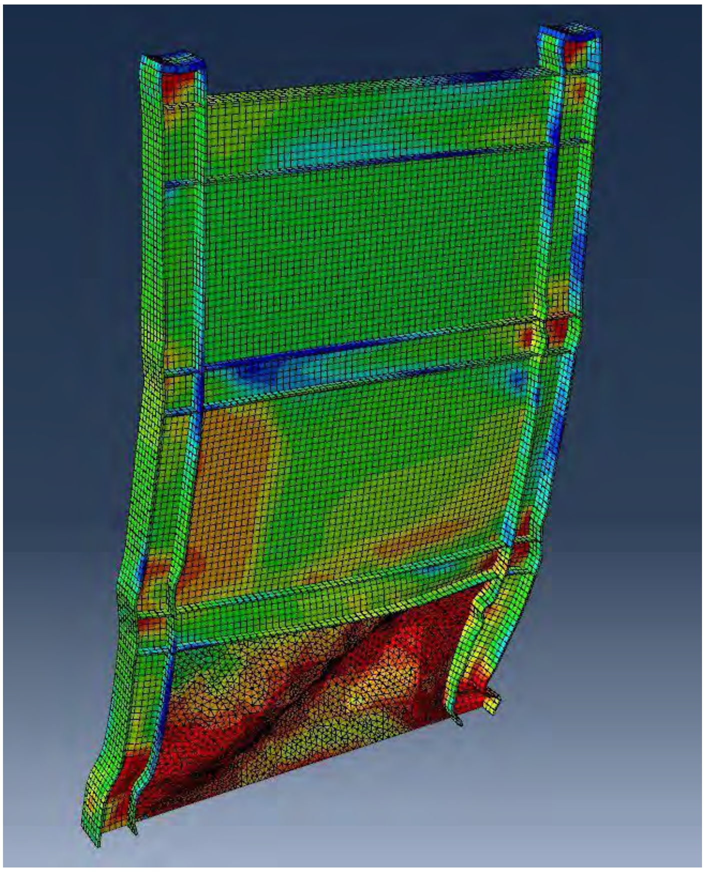


Figure 4. 10: Three- dimensional Finite element model of steel plate shear wall.

Finite element model of three storied steel plate shear wall also shows the similar patterns of failure. Excessive local buckling is observed at the column in the lower portion of the First level and near the beam column joint. On the other hand local buckling at the horizontal beam is observed at the beam end especially on the first level and the second level.

Plate shear buckling is most prominent in the first panel and second panel, however in the first panel it is more severe. Significant plate buckling is not observed in third panel.

CHAPTER 5

PARAMETRIC STUDY OF STEEL PLATE SHEAR WALL

5. Introduction

The finite element model developed in the previous chapter has been found to predict accurately both the monotonic and cyclic capacity of steel plate shear wall system. This chapter presents a detailed parametric study on SPSW system using the developed finite element model. Previous researchers conducted numerous parametric studies. For example, Caccese et al. (1993) derived the failure pattern thin and thick SPSW system. Xue and Lu (1994) investigated the effect of width–to–thickness ratio of the infill plate and the panel aspect ratio (width/height) by finite element analysis of 20 different cases and remarked that, aspect ratio is also a key parameter, and it governs the capacity of SPSW system. Wagner (1931) first introduced flexibility parameter of a boundary member around plate girder, his idea was later incorporated in AISC (2005) and Canadian (CSA S16-01) building code. Behbahaniferd et al. (2003), Mortazavi et al. (2013), Heidari et al. (2014) performed limited numerical studies on geometric parameters.

However present study, conducted parametric studies in wider ranges. The research also highlighted sensitivity of selected geometric parameters and their correlations (mutual effect of aspect ratio and plate thickness; to find out the effect of plate slenderness on the strength of steel plate shear wall)

The model selected for is a single steel plate shear wall panel with Horizontal beam and vertical column with fixed base, subjected to shear force and constant gravity load. A set of geometric and material parameters are identified, that define the behavior of the model. The steel plate shear wall model selected for the parametric study is presented in Figure 5. 1.

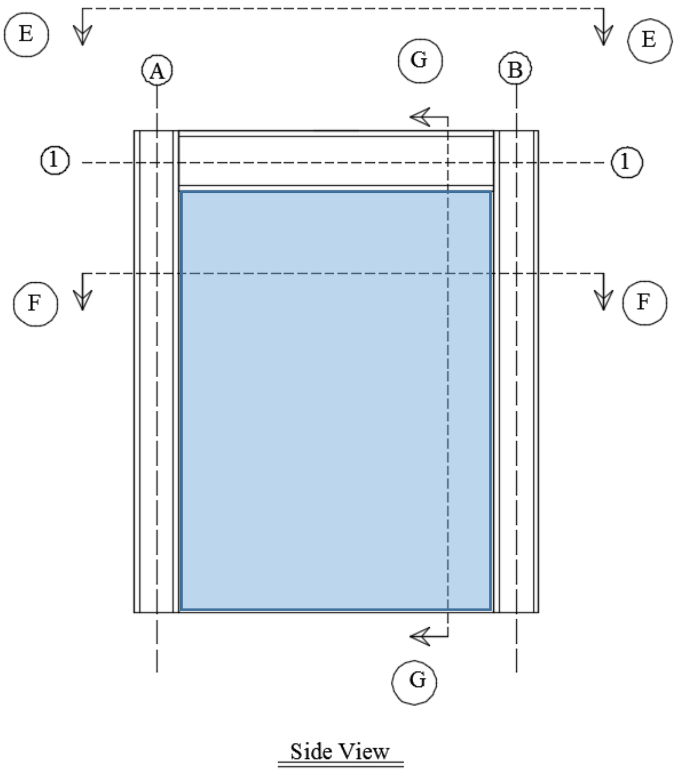
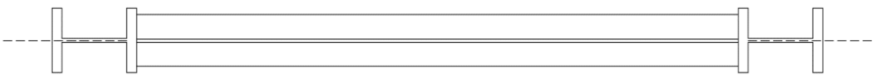
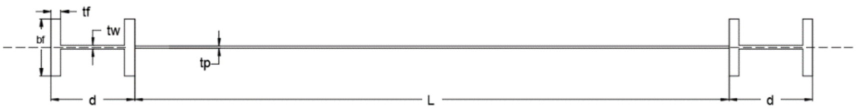


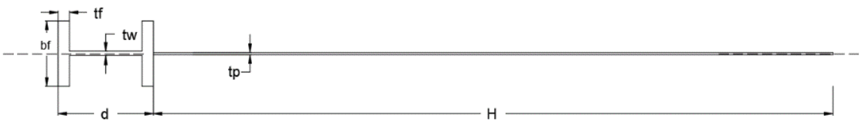
Figure 5. 1: Elevation of one storied steel plate shear wall system.



(a) Section E-E



(b) Section F-F



(c) Section G-G

Figure 5. 2: Different sectional view of single storied steel plate shear wall system as shown in Figure 5. 1

5.1 Selection of parameters

5.1.1 Shear wall aspect ratio ($\frac{L}{H}$)

Aspect ratio (L/H) of steel plate shear wall can be defined as the ratio of width (L) to height (H). This parameter has significant effect on SPSW system, it strongly influences the inclination of the tension field and the strength of the steel plate shear wall.

According to AISC seismic provision 2005 the limit of aspect ratio is $0.8 < L/H < 2.5$, in this study, the aspect ratio was varied from 0.80 to 1.40 with an intermediate value of 1.0

5.1.2 Shear wall thickness (t_p) / plate slenderness effect ($\frac{h}{t_p}$)

For a steel plate, the plate thickness is a significant parameter which is governed by plate slenderness ratio. Plate thickness govern the failure mode of SPSW system. Hence, it was incorporated in parametric study. The slenderness limit of steel shear wall is $\frac{\text{min of (L,H)}}{t_p} = 25\sqrt{\frac{E}{F_y}}$ suggested by AISC 2005. In this study, plate slenderness ratio is calculated with respect to the height of the infill plate.

Effect of this parameter is illustrated on section 5.3.2

5.1.3 Column flexibility parameter ($0.7\sqrt[4]{\frac{h^4 t_p}{2LI_c}}$)

Column flexibility parameter is defined as ($0.7\sqrt[4]{\frac{h^4 t_p}{2LI_c}}$). It is proportional to the ratio of in-plane bending flexibility of column ($\frac{h^3}{EI_c}$) to the in-plane flexibility of infill plate ($\frac{L}{Et_p h}$) in horizontal direction.

The parameter was originally introduced by Wagner (1931) in order to study the effect of flexibility of the plate girder flange on the redistribution of the tension field in a plate girder web. The code (CSA S16 committee) maximum limit for column flexibility parameter is 2.5, effect of this parameter is investigated in section 5.3.3. Values of column flexibility parameters selected for the study are 1.65, 1.82, 2.02, 2.52 and 3.16.

5.1.4 Yield strength of material (σ_y)

The yield strength of the steel plate shear wall system affect the load displacement behavior of the system. In current study, the yield strength was varied from 250 MPa (A36 steel) to 550 MPa (A706 Grade 80 steel) with an intermediate value of 345 MPa (A572 Grade 50 steel) and 440 MPa (A706 Grade 60 steel).

5.2 Effect of the parameters on the behavior steel plate shear wall system

In this section only four parameters are investigated in depth in the context of the single storey model. These are the aspect ratio, the plate thickness, the column flexibility parameter, the beam stiffness and the effect of plate yield strength respectively. The effect of these parameters on the behavior and strength of SPSW system was studied and the discussion of the analysis results are presented below.

5.2.1 Effect of aspect ratio

The effect of infill plate aspect ratio on the behavior of steel plate shear wall has been discussed in this section. Aspect ratios adopted in the numerical investigation are 1.40, 1.0, and 0.80. An aspect ratio of 1.40 represents a wide and short shear wall panel whereas an aspect ratio of 0.80 represents a narrow and tall shear wall. Cross sectional dimension of Horizontal beam and vertical column are kept constant and their dimensions are depicted in Figure 5. 3.

The effect of aspect ratio on the strength and stiffness of SPSW system are presented in Table 5. 2 for two different plate thickness. The base shear vs. displacement curve for SPSW for selected ranges of aspect ratio are shown in Figure 5. 4 and Figure 5. 5 respectively for t_p = 10 mm ($\frac{H}{t_p}$=330) and t_p =5 mm ($\frac{H}{t_p}$=650).

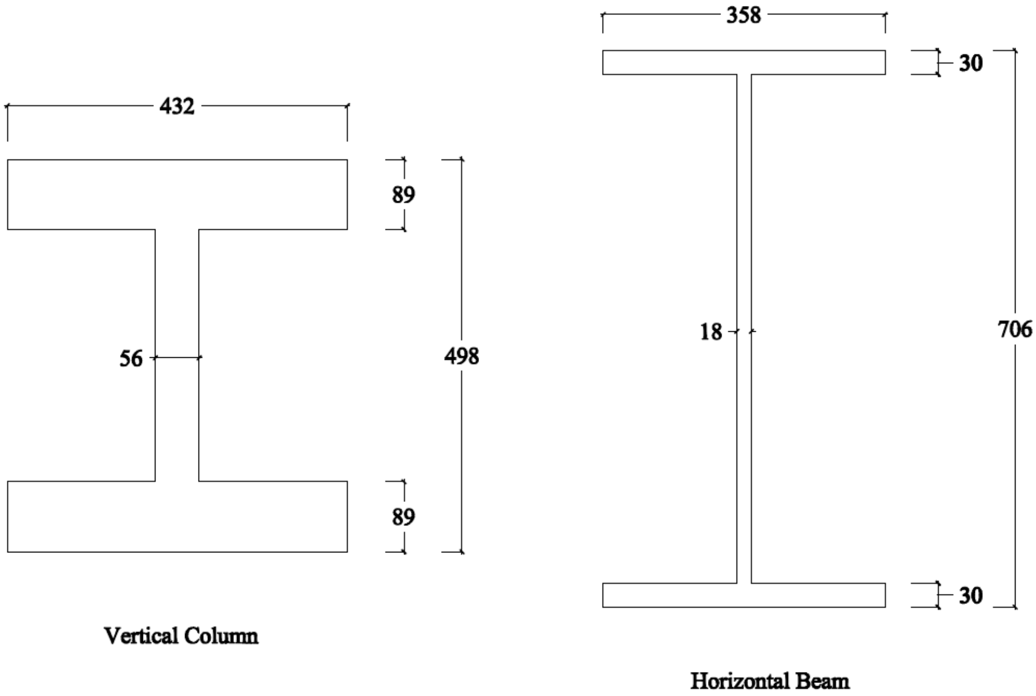


Figure 5. 3: Cross section of Horizontal beam and Vertical column fixed for parametric study (all dimensions are in mm).

Table 5. 1: Shear wall specimen selected for investigation of aspect ratio (by changing width)

SI No.	Specimen Designation	Height, H (mm)	Width, L (mm)	Aspect ratio, L/H	thickness of plate, t_p (mm)	Plate slenderness ratio ($\frac{H}{t_p}$)	Steel yield strength, f_y (MPa)
1	SW_330_1.40_3.5	3260	4580	1.40	3.5	930	345
2	SW_330_1.40_5	3260	4580		5.0	650	345
3	SW_330_1.40_10	3260	4580		10.0	330	345
4	SW_330_1.40_15	3260	4580		15.0	220	345
5	SW_330_1_3.5	3260	3260	1.0	3.5	930	345
6	SW_330_1_5	3260	3260		5.0	650	345
7	SW_330_1_10	3260	3260		10.0	330	345
8	SW_330_1_15	3260	3260		15.0	220	345
9	SW_330_0.80_3.5	3260	2600	0.80	3.5	930	345
10	SW_330_0.80_5	3260	2600		5.0	650	345
11	SW_330_0.80_10	3260	2600		10.0	330	345
12	SW_330_0.80_15	3260	2600		15.0	220	345

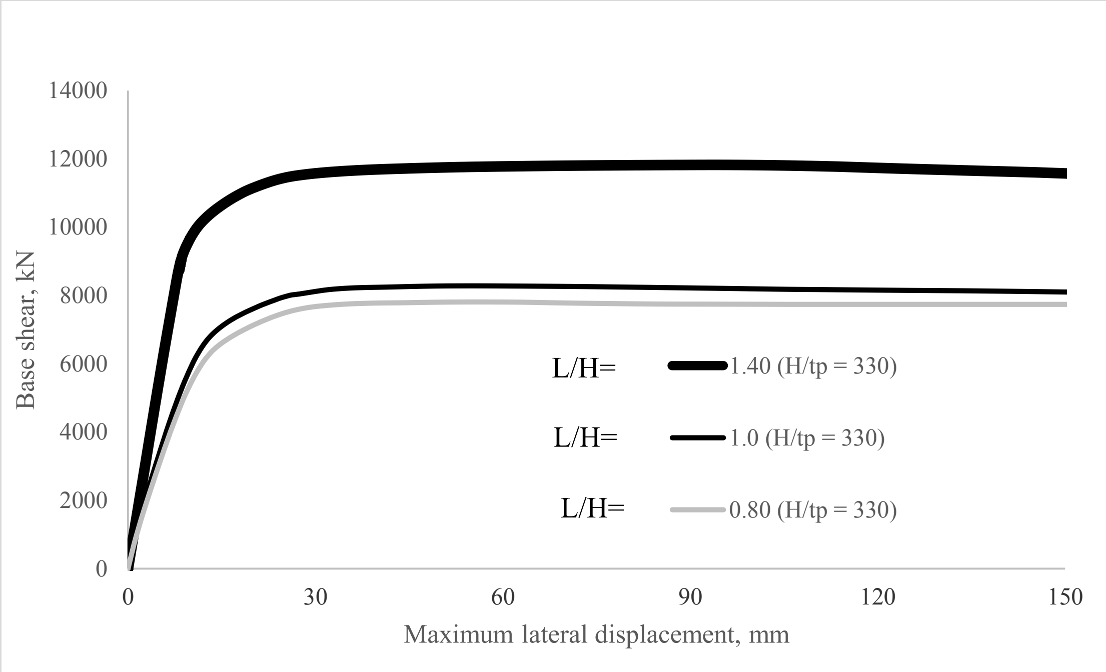


Figure 5. 4: Effect of aspect ratio on base shear vs. lateral deflection response ($\mathbf{t_p}$ =10 mm; $\frac{H}{t_p}$ =330).

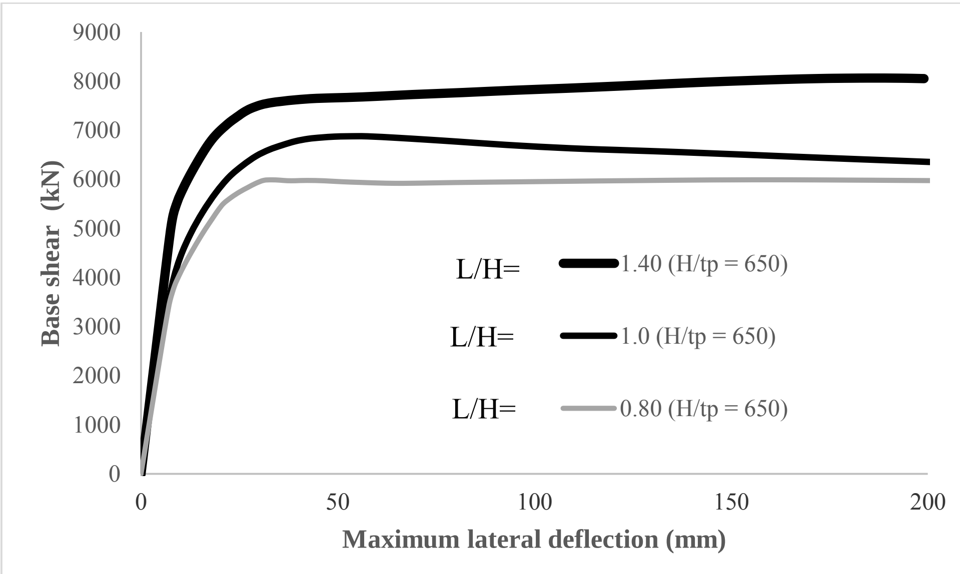


Figure 5. 5: Effect of aspect ratio on base shear vs. lateral deflection response ($\mathbf{t_p}$ =5 mm; $\frac{H}{t_p}$ =650).

Table 5. 2: Effect of aspect ratio on strength and stiffness of steel plate shear wall (for t_p = 5 mm; $\frac{H}{t_p}$=650 and 10 mm; $\frac{H}{t_p}$=330)

plate thickness	Aspect ratio /plate slenderness ratio	stiffness (kN/mm)	% difference of strength (w.r.t. aspect ratio 1.0)	Maximum capacity (kN)	% difference of stiffness
10 mm	1.40 ($\frac{H}{t_p}$=330)	1100	59.4	11820	42.8
	1.0 ($\frac{H}{t_p}$ = 330)	690		8280	
	0.80 ($\frac{H}{t_p}$ = 330)	610	-11.6	7800	-5.8
5 mm	1.40 ($\frac{H}{t_p}$=650)	670	19.6	8060	17.2
	1.0 ($\frac{H}{t_p}$ = 650)	560		6875	
	0.80 ($\frac{H}{t_p}$ = 650)	490	-12.5	6000	-12.7

Referring to Table 5. 2, for 10 mm ($\frac{H}{t_p}$=330) thick shear decreasing the aspect ratio from 1.0 to 0.80 decrease the strength and stiffness of shear wall 11.6% and 6% respectively. While on the other hand, increasing the aspect ratio from 1 to 1.40 results in increase in the strength and stiffness 59.4% and 42.8% respectively.

Again, for 5 mm ($\frac{H}{t_p}$=650) thick plate the increase in strength due to increase in aspect ratio from 1.0 to 1.40 is 17%. Decreasing the aspect ratio from 1.0 to 0.80 decreases the strength by 13%. The effect of aspect ratio on strength and stiffness is found to be more prominent for 10 mm ($\frac{H}{t_p}$=330) thick plate. It can be concluded from the analysis, aspect ratio has significant on the thick shear wall system.

Current study varied the aspect ratio by increasing the shear wall width. Wider shear wall provide greater shear resistance than narrower one. This is why the strength of SPSW system increases with an increase in aspect ratio.

Aspect ratio, also impacts the post-buckling development of diagonal tensile stresses in slender plate-girder web panels. Correlation between aspect ratio and inclination angle is shown in Figure 5. 6. Referring to Figure 5. 6, for aspect ratio 1.40 Angle of inclination is $57^{\mathbf{0}}$, for square shear wall inclination angle is $45^{\mathbf{0}}$ and for narrow-tall shear wall (aspect ratio 0.80) inclination angle is $43^{\mathbf{0}}$.

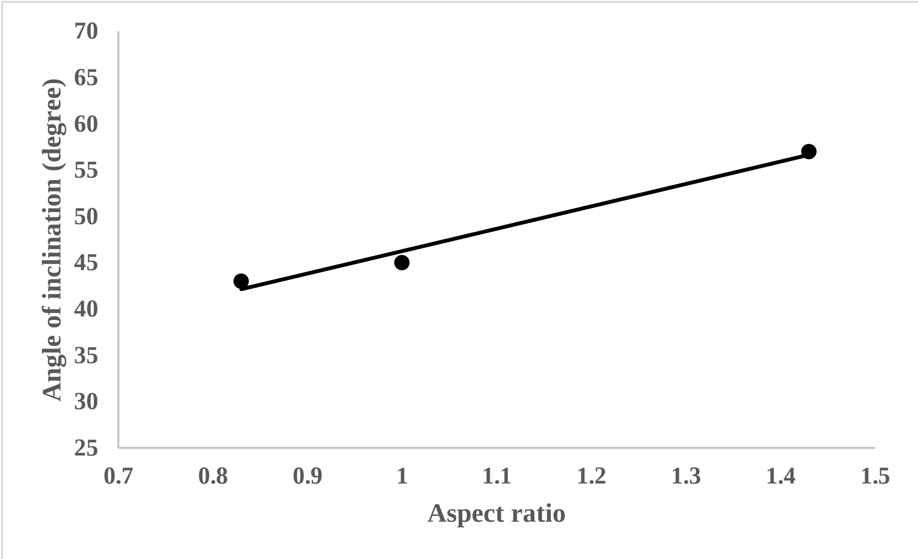


Figure 5. 6: Correlation between angle of inclination and aspect ratio.

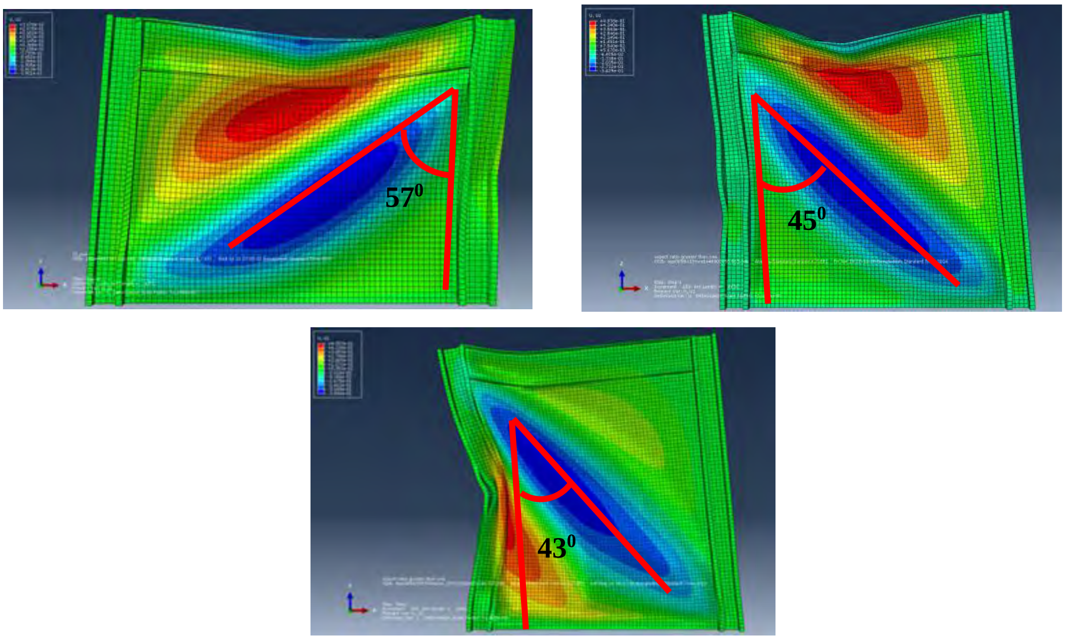


Figure 5. 7: Changes of inclination angle with aspect ratio.

5.2.2 Effect of plate thickness/ plate slenderness ratio

The effect of the steel plate thickness in the overall behavior of the SPSW has been discussed in this section. The thickness of the plate has been varied between 3.5 mm ($\frac{H}{t_p}$=930), 5 mm ($\frac{H}{t_p}$=650), 10 mm ($\frac{H}{t_p}$=330) and 15 mm ($\frac{H}{t_p}$=220) for three different aspect ratio, which was mentioned in the previous section. Therefore, twelve sets of SPSW finite element model has been developed (Table 5. 1). The stiffness of the boundary members are kept constant in these models (Figure 5. 3). Current study, calculates the plate slenderness from the ratio of shear wall thickness (**t_p**) and height of shear wall (**H**) rather than, width of shear wall (**L**); because the aspect ratio of the system is changed by changing width (**L**). Therefore, plate slenderness considering the height of infill plate is considered to be consistent.

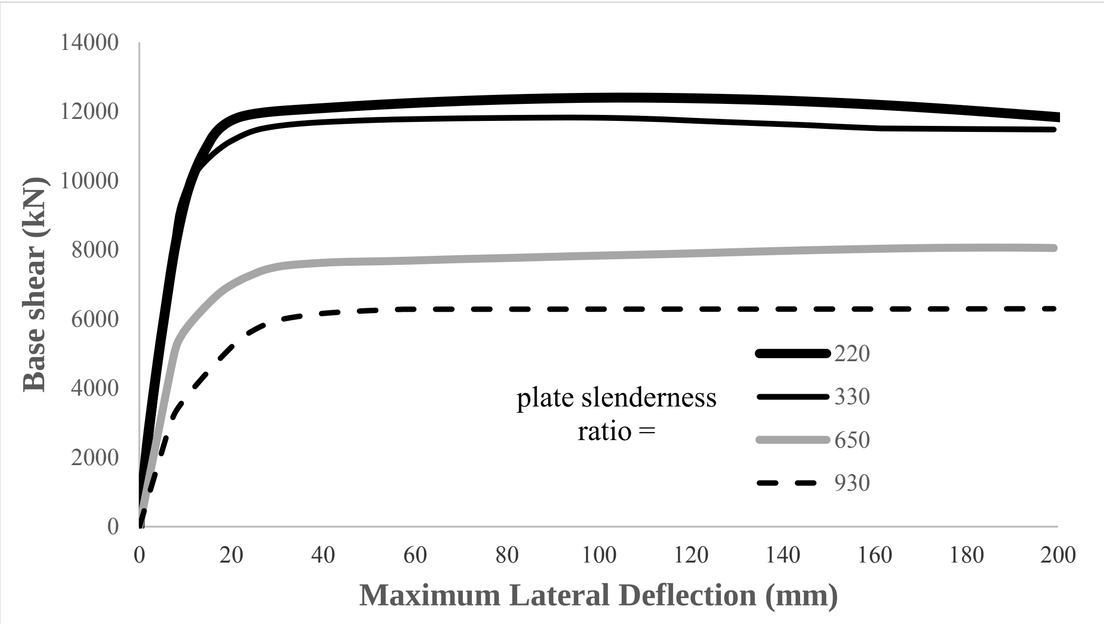


Figure 5. 8: Effect of plate thickness on base shear vs. lateral deflection response (aspect ratio 1.4)

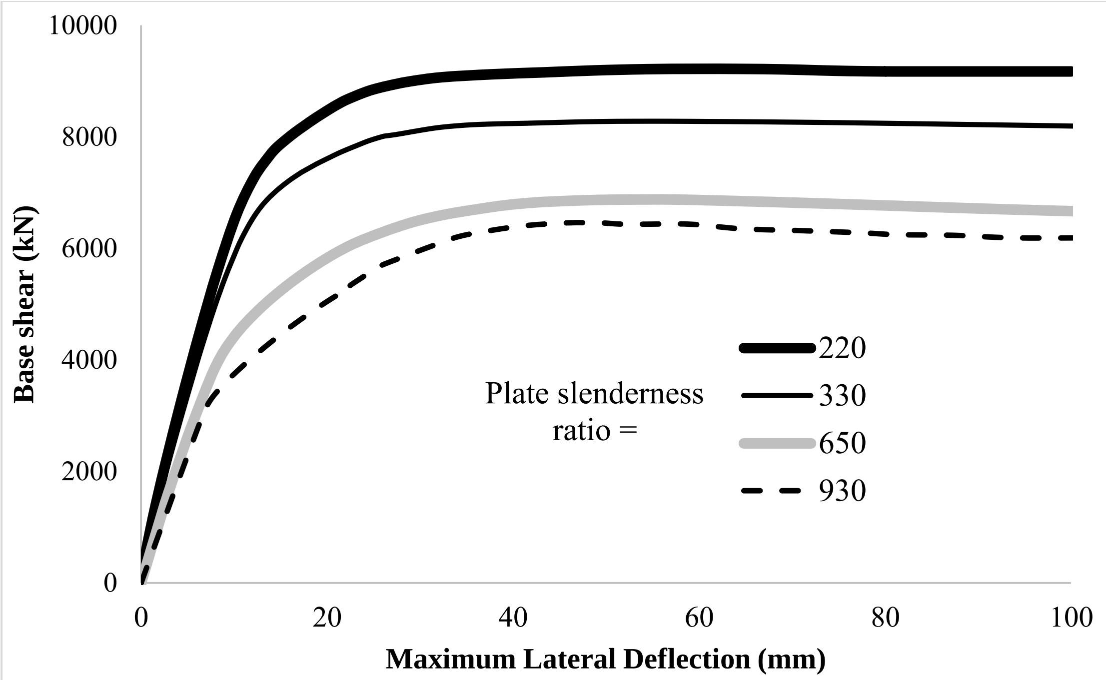


Figure 5. 9: Effect of plate thickness on base shear vs. lateral deflection response (aspect ratio 1.0)

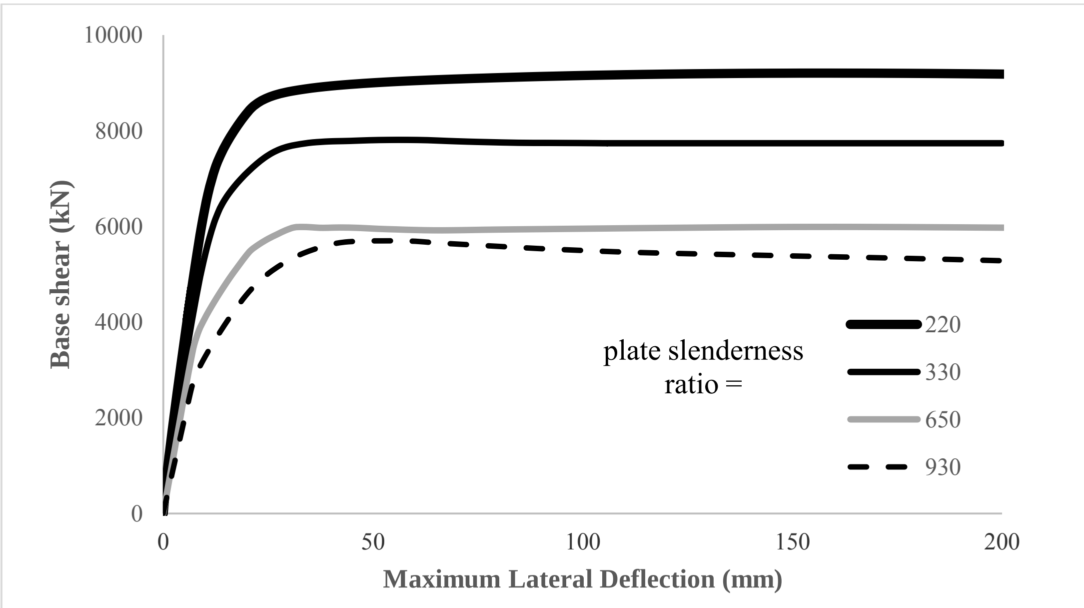


Figure 5. 10: Effect of plate thickness on base shear vs. lateral deflection response (aspect ratio 0.80)

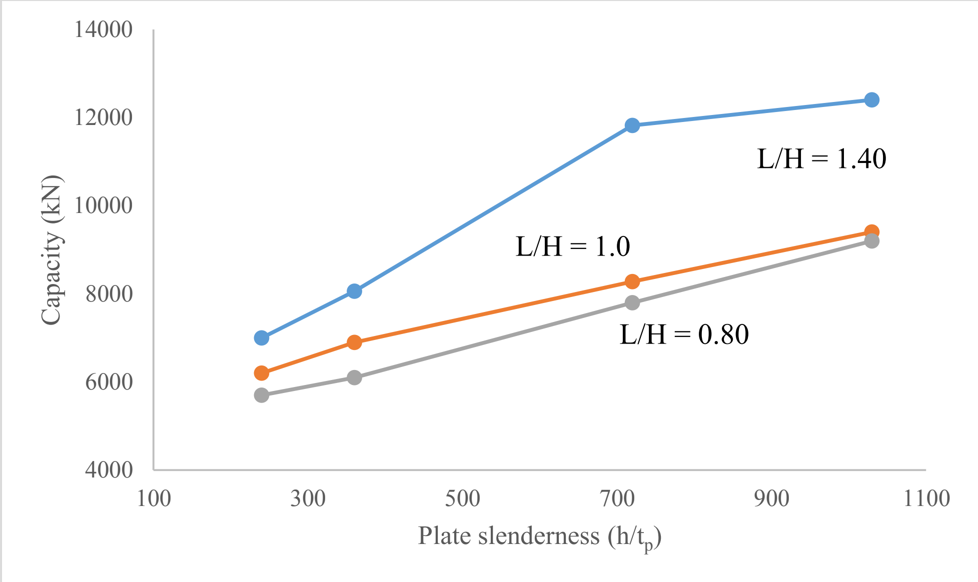


Figure 5. 11: Variation of SPSW capacity with variable **plate thickness** and Variable **aspect ratio**.

Figure 5. 8 presents the base shear vs. lateral deflection curve for aspect ratio 1.40 for the selected values of plate thickness. This figure demonstrates that the capacity of the shear wall increases by increasing the thickness from 3.5 mm ($\frac{H}{t_p}$=930) to 15 mm ($\frac{H}{t_p}$=220). Increasing the thickness from 3.5 mm ($\frac{H}{t_p}$=930) to 5 mm ($\frac{H}{t_p}$=650) results in an increase in capacity by 15%, and increasing 5mm ($\frac{H}{t_p}$=650) to 10mm ($\frac{H}{t_p}$=330) enhance the capacity by 46% on the other hand, increasing the thickness from 10 mm ($\frac{H}{t_p}$=330) to 15 mm ($\frac{H}{t_p}$=220) increases the capacity by 5% only. This is due to the fact that, the system becomes compact, when plate thickness exceeds 10 mm ($\frac{H}{t_p}$=330).

The effect of plate thickness on shear walls with aspect ratio 1.0 and 0.80 are shown in Figure 5. 9 and Figure 5. 10 respectively. For aspect ratio 1.0, increasing the thickness from 3.5 mm ($\frac{H}{t_p}$=930) to 5 mm ($\frac{H}{t_p}$=650) results in an increase in capacity by 11%, and increasing thickness from 5 mm ($\frac{H}{t_p}$=650) to 10 mm ($\frac{H}{t_p}$=330) enhances the capacity by 20%. Further, increasing the thickness from 10 mm ($\frac{H}{t_p}$=330) to 15 mm ($\frac{H}{t_p}$=220) increases the capacity by 13%. Again for aspect ratio 0.80, increasing the thickness from 3.5 mm ($\frac{H}{t_p}$=930) to 5 mm ($\frac{H}{t_p}$=650) results in an increase in capacity by 7%, and increasing 5 mm ($\frac{H}{t_p}$=650) to 10 mm ($\frac{H}{t_p}$=330) enhance the capacity by 28% further, increasing the thickness from 10 mm ($\frac{H}{t_p}$=330) to 15 mm ($\frac{H}{t_p}$=220) increases the capacity by 18%. Capacity increment of steel plate shear wall system (aspect ratio 1.0 and aspect ratio 0.80) with respect to plate thickness is found to be linear. Which indicates that the system still remains non - compact at comparatively higher thickness. Capacity variation of steel plate shear wall system with respect to aspect ratio and plate thickness is shown in Figure 5. 11. However it can be concluded that, shear capacity of narrow tall shear increases linearly with respect to thickness of in filled panel. This condition is also true for square panel. However, on the other hand, for wide and short shear walls, capacity increases linearly up to a thickness of 10 mm ($\frac{H}{t_p}$=330), beyond this limit; rate of capacity increment ceases considerably.

5.2.3 Effect of column flexibility parameter

In order to investigate the effect of this parameter on the behavior of a steel plate shear wall, three models with column flexibility ratios of 3.16, 2.52, 2.02, 1.82, and 1.65 (Figure 5. 13) were analyzed. Aspect ratio of shear wall panel is 1.0 with a plate thickness of 5.0 mm ($\frac{H}{t_p}$=650), the dimension of horizontal beam is kept constant. Figure 5. 12 show the load deflection response of the five steel plate shear walls with varying column flexibility values. For instance by reducing the column flexibility of the model from 2.02 to 1.65 the capacity of the steel plate shear wall is increased by 17%. Ultimate capacity of steel plate shear wall and column flexibility parameter shows a linear relationship which is observed from (Figure 5. 14). On the other hand, the relationship between the column flexibility parameter and the stiffness of the shear wall is nonlinear (Figure 5. 15).

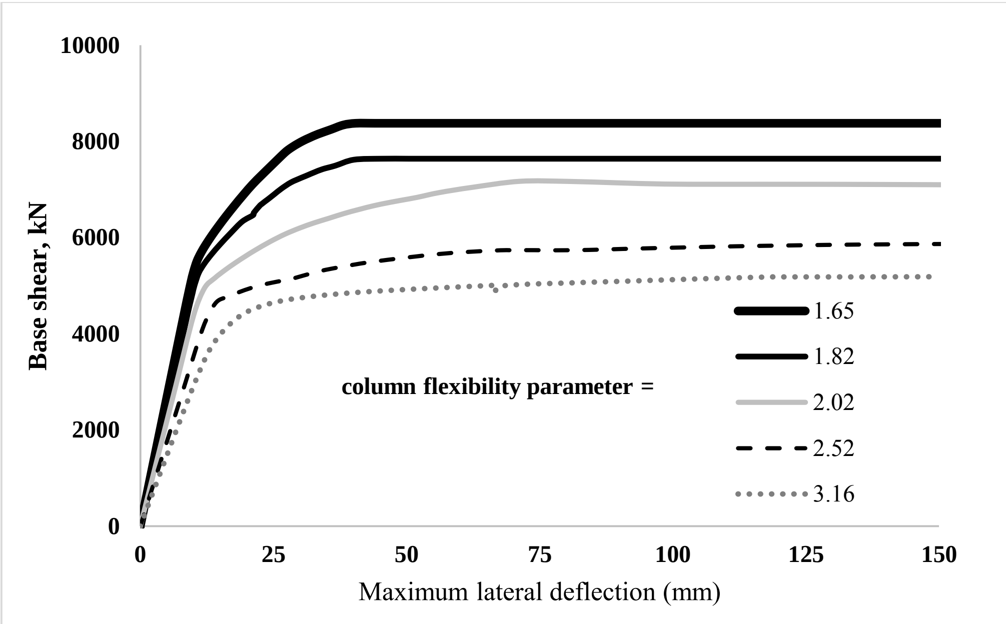
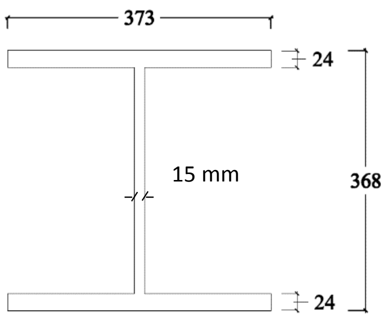
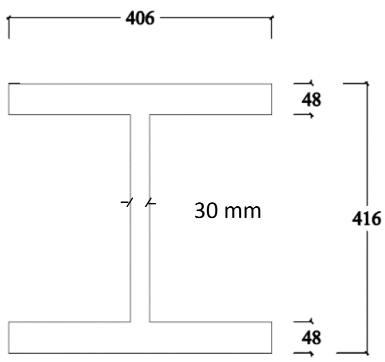


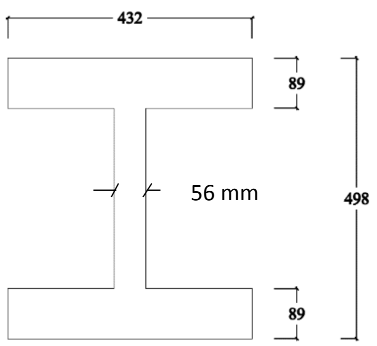
Figure 5. 12: Effect of column flexibility parameter on base shear vs. lateral deflection response (Aspect ratio 1; $t_p = 5$ mm; $\frac{H}{t_p}$=650)



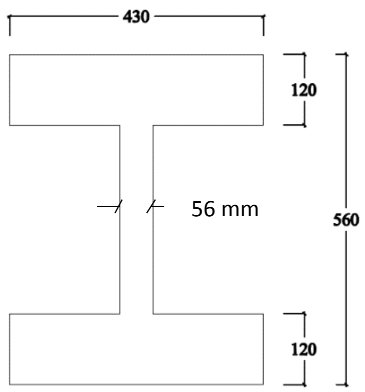
Column flexibility parameter 3.16



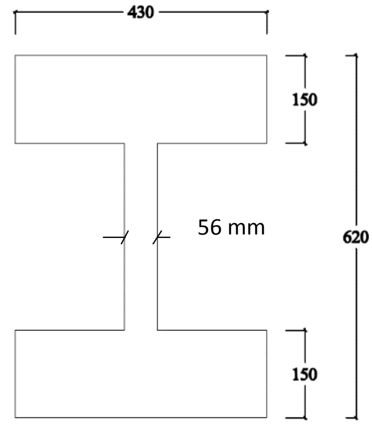
Column flexibility parameter 2.56



Column flexibility parameter 2.02



Column flexibility parameter 1.82



Column flexibility
parameter 1.65

Figure 5. 13: Selected column dimension to see the effect of column flexibility parameter

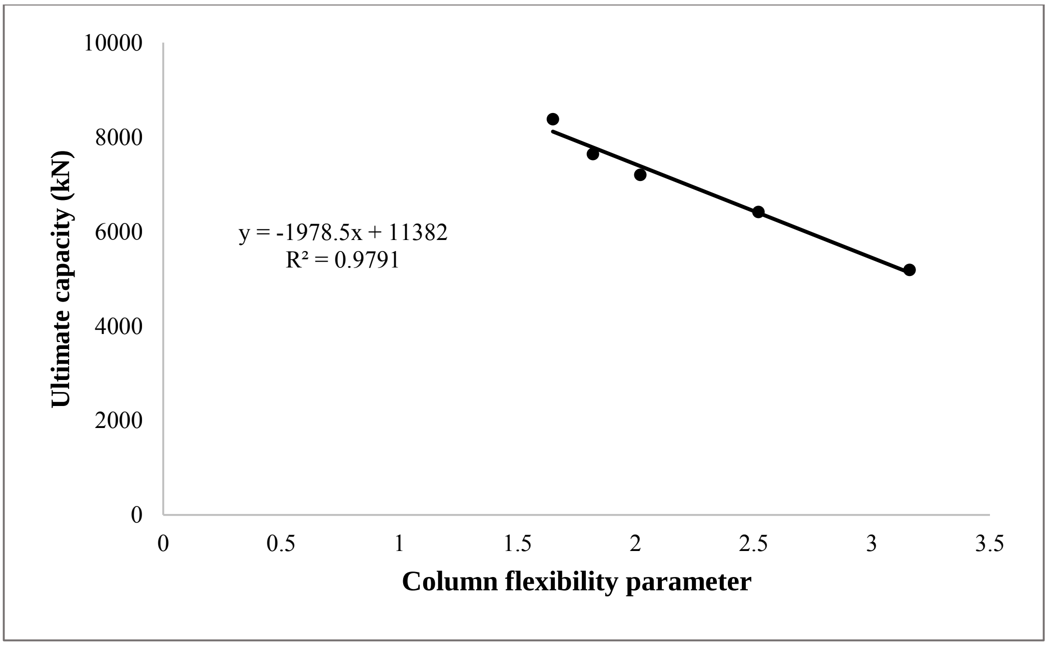


Figure 5. 14: Relationship between column flexibility parameter and capacity of SPSW system.

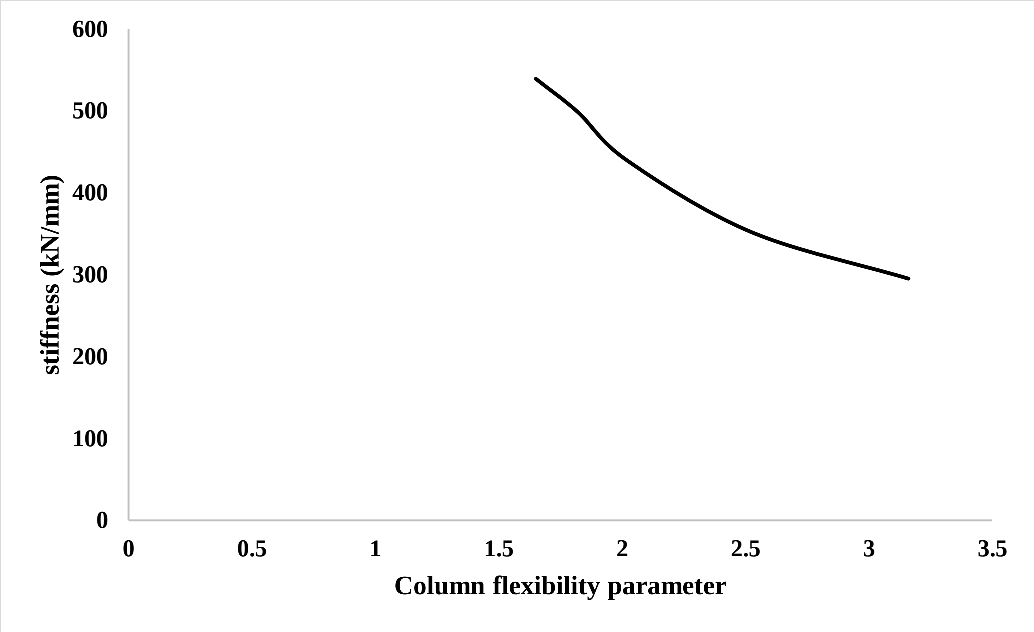


Figure 5. 15: Relationship between column flexibility and shear wall stiffness.

5.2.4 Influence of Material property on shear wall capacity

The effect of yield strength of the steel plate on its ultimate capacity has been investigated for four different ASTM steel grades (A36, A572 Grade 50, A706 Grade 60 and A706 Grade 80 steel). Stress strain curve of A36 and A572 grade 50 steel is obtained from Johnston et al. 1968. On the other hand, nonlinear curve of A706 grade 60 and A706 grade 80 steel are obtained from NIST GCR 14-917-30 Report, 2014. Nonlinear stress strain curve of the ASTM steel's that are used on finite element are shown in

Figure 5. *16*.

The base shear vs. lateral deflection response of SPSW for the selected grades of steel are shown on Figure 5. 17. In this parametric study, aspect ratio (=1.0) and plate thickness (= 10 mm; $\frac{H}{t_p}$=330) were kept constant.

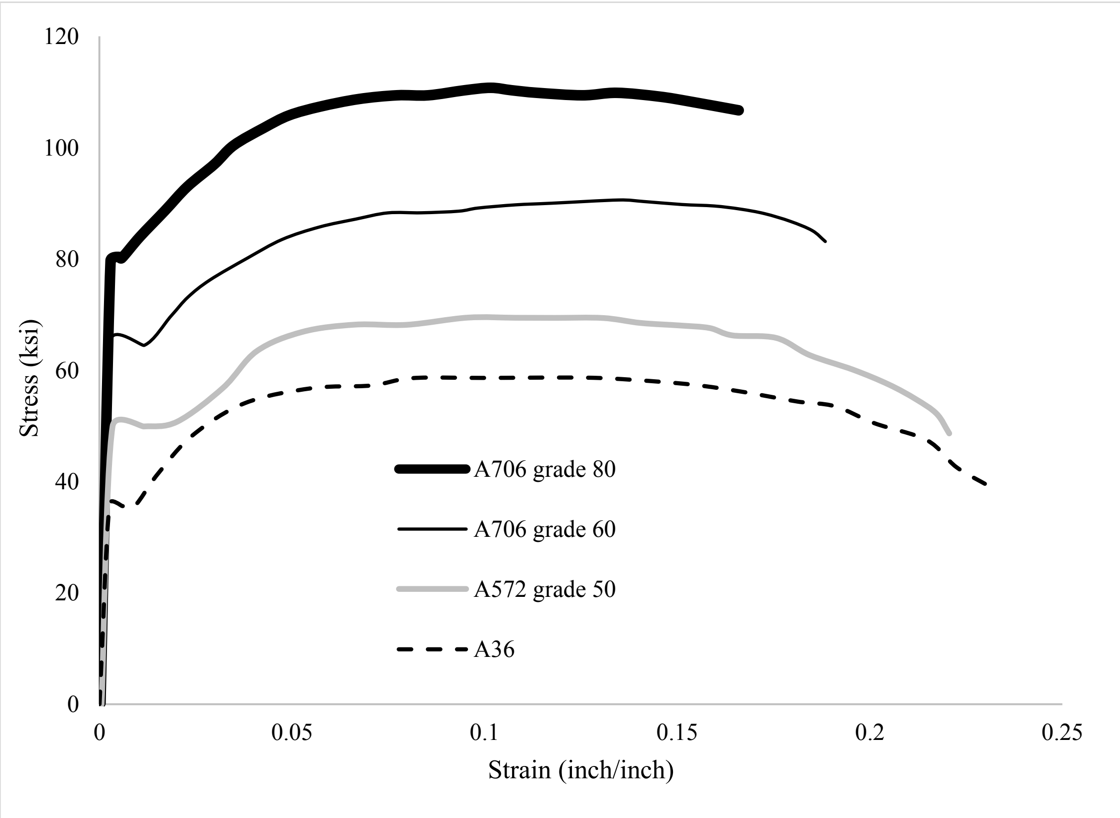


Figure 5. 16: Non-linear stress strain curve of standard ASTM steel used Finite element simulation for parametric study.

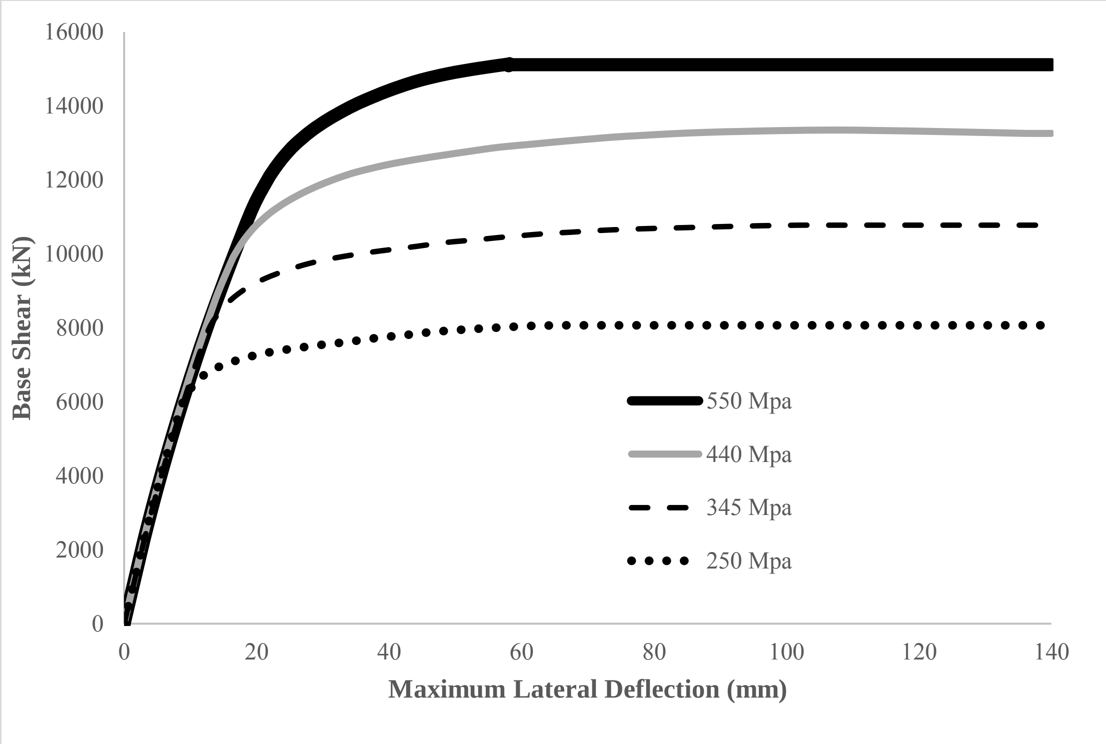


Figure 5. 17: Effect of yield parameter on base shear vs. lateral deflection response (aspect ratio 1; plate thickness 10 mm; $\frac{H}{t_p}$=330).

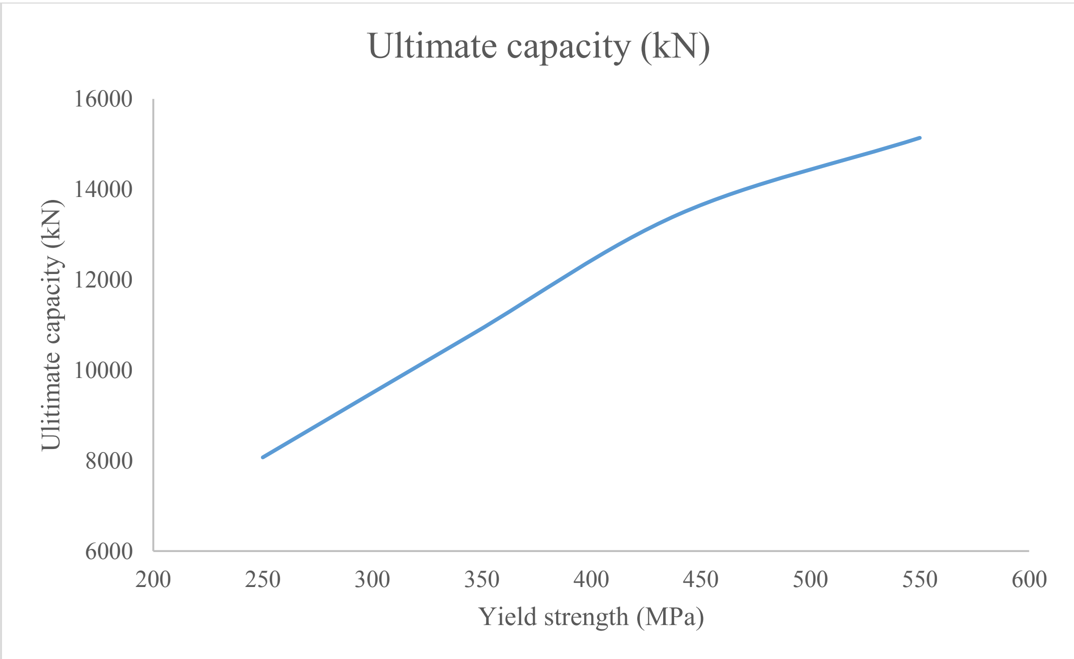


Figure 5. 18: Variation of ultimate capacity with strength of material (aspect ratio 1; plate thickness 10 mm; $\frac{H}{t_p}$=330)

From this figure it is obvious that the capacity is greatly affected by increasing the yield strength of steel. Increasing the yield strength from 250 MPa to 345 MPa results in increase in capacity by 32%, on the other hand, increase the yield strength from 345 MPa to 440 MPa results in increase the capacity by 24%, again increase the yield strength from 440 MPa to 550 MPa results in increase the capacity by 16%.

5.2.5 Load shared by shear wall with compared to Boundary column

In steel plate shear wall system, shear wall and boundary column together resist lateral load. Normally shear wall takes the major portion of lateral load. Column and the shear wall shares the lateral forces according to their stiffness ratio. This section discusses the scenario of energy sharing between vertical column and steel wall. To do so, the stiffness of shear wall is varied by changing the thickness (Table 5. 3). Energy absorption of steel plate shear wall system is calculated from the area under the pushover curve. Individual energy absorption of column and shear wall is calculated separately, from where the load sharing between shear wall and column is computed.

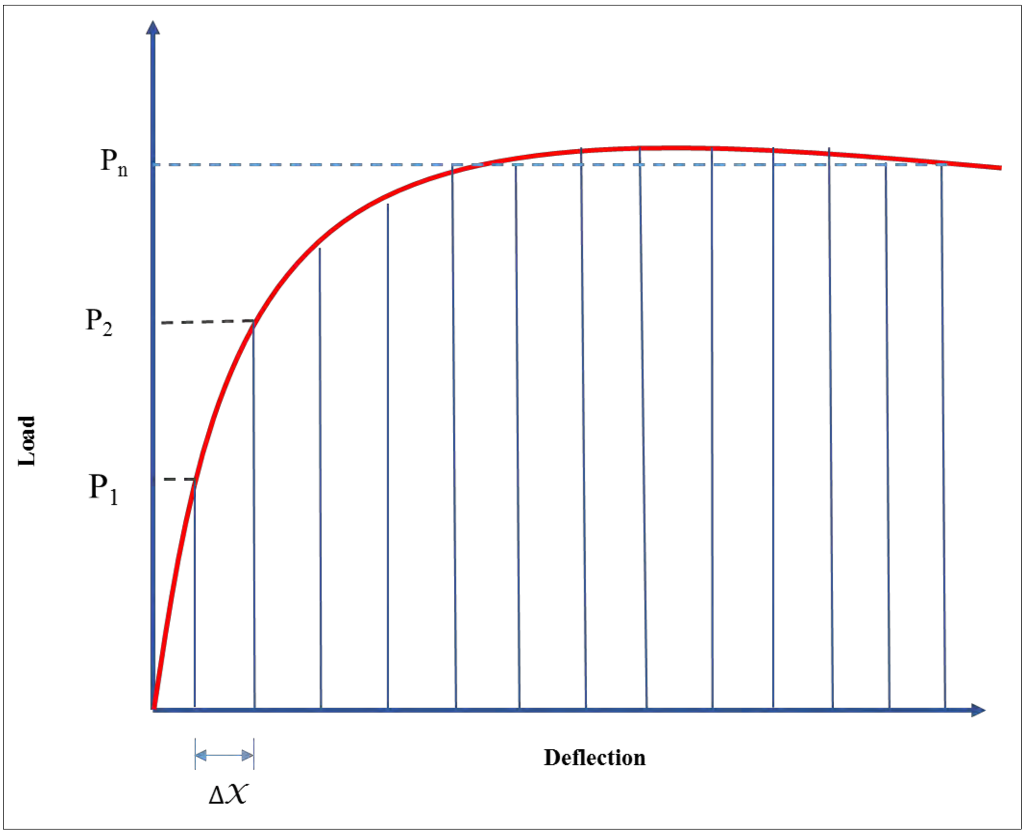


Figure 5. 19: Calculation of Energy consumption by steel plate shear wall system.

Energy consumption can be calculated as,

$$E = \sum_{n=0}^{n} \frac{1}{2} * (P_n + P_{n+1}) * \Delta x$$

When plate thickness is 3.5 mm ($\frac{H}{t_p}$=930) energy dissipated by shear wall is 35%, when thickness is 5 mm ($\frac{H}{t_p}$=650), energy shared by shear wall is 50%, for 10 mm ($\frac{H}{t_p}$=330) thick plate 75% of system's energy is shear wall, and for 15 mm ($\frac{H}{t_p}$=220) plate thickness 78% of the energy is shared by shear wall.

It is evident from the analysis that, increasing the shear thickness results in increasing the energy dissipation capacity of shear wall. If the plate thickness exceeds 10 mm ($\frac{H}{t_p}$=330) the dissipation capacity does not improve significantly.

Beam and column dimensions used in SPSW system are showed in Table 5. 3

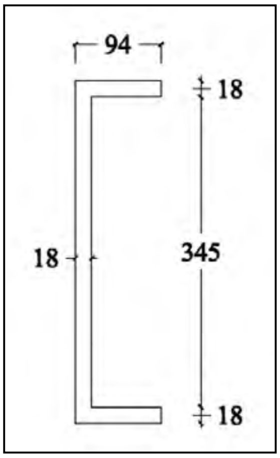
Table 5. 3: Load shared by steel plate shear wall and boundary column

Panel thickness	Energy dissipated by SPSW system (kN.m)	% Energy shared by column (kN.m)	% Energy shared by shear wall
3.5 mm ($\frac{H}{t_p}$=930)	290	65	35
5 mm ($\frac{H}{t_p}$=650)	1500	50	50
10 mm ($\frac{H}{t_p}$=330)	1800	25	75
15 mm ($\frac{H}{t_p}$=220)	2400	22	78

5.3 Effect of shear wall Strut

5.3.1 Effect of Horizontal Strut on steel plate shear wall

It is not always possible to increase the dimension of shear wall and supporting column. It is then logical to introduce horizontal or vertical stiffener. In these study capacity of shear wall is investigated by incorporating horizontal strut only. Present study checked the effectiveness of strut for strength increment of SPSW system. Horizontal strut (Figure 5. 20) is applied to short wide (aspect ratio 1.40), square (aspect ratio 1.0) and narrow tall (aspect ratio 0.80). A comparison was made between the unstiffened and stiffened SPSW system (Figure 5. 21).



Horizontal strut section 01
(C380 X 74)

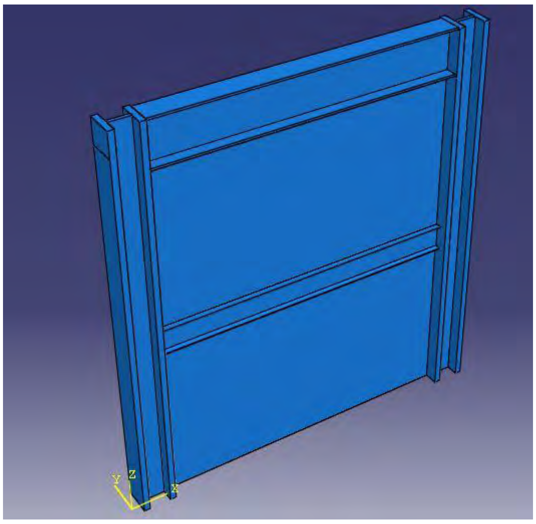


Figure 5. 20: Variation of ultimate capacity with strength of material (aspect ratio 1, plate thickness 10 mm; $\frac{H}{t_p}$ =330)

Table 5. 4: Aspect ratio used to find out optimum situation, suitable for horizontal strut.

Aspect ratio	Height (H), mm	Width (L), mm	Plate thickness t_p (mm)	Plate slenderness ratio $(\frac{H}{t_p})$
1.40	3260	4580	10	330
1.0	3260	3260	10	
0.80	3260	2600	10	

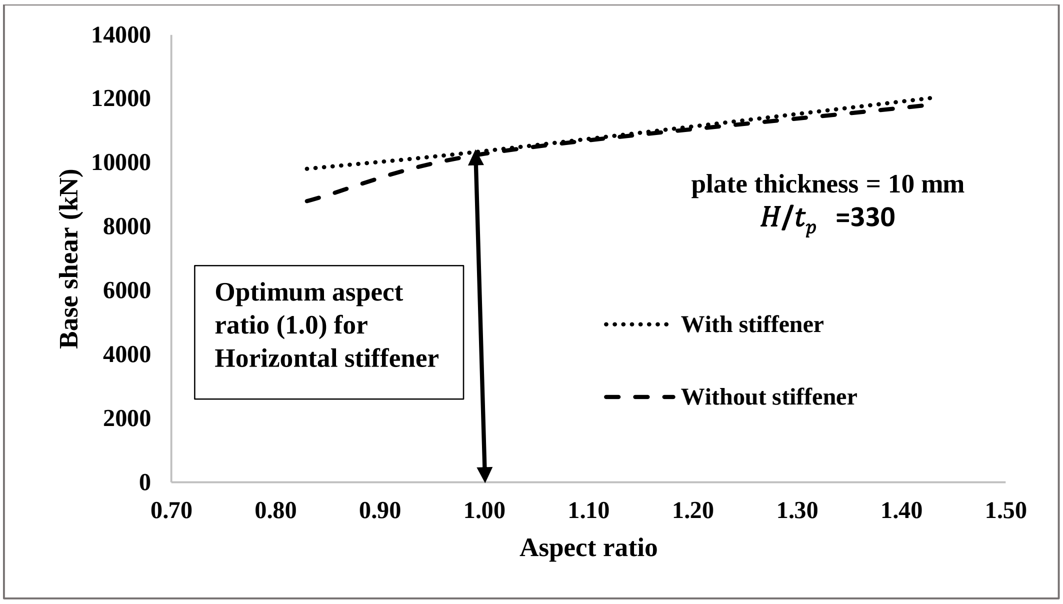


Figure 5. 21: Minimum aspect ratio requirement for strut (Horizontal strut) application.

It is eminent from the analysis that, application of horizontal strut is effective if aspect ratio is greater than one. For aspect ratio 0.80; the difference between stiffened and unstiffened shear wall is 17.8%.; hence it can be concluded from the analysis that, strength of slender (tall and narrow) shear wall can be enhanced by applying horizontal strut.

CHAPTER 6

CONCLUSION AND RECOMMENDATION

6.1 General

A finite element model based on static formulation was developed for the analysis of steel plate shear walls. Material and geometric nonlinearity, were included in the model. Appropriate loads and boundary conditions applied in the finite element model are analogous to the laboratory setup. A smooth cyclic amplitude are defined to simulate the cyclic load application. Load application on finite element model was performed in two different steps. Total 23 cycles are applied in static analysis. Among them first nine cycles are load controlled and rest of the cycles are deflection controlled. Tied constrained are applied between members to represent the structural connections. Initial imperfection of structural member was included but residual thermal stress is ignored in the analysis. Analytical model shows good performance to predict the ultimate capacity of the system. The stiffness of the system is not well predicted by the numerical model as the residual stress is ignored.

In general, the hysteresis loops generated by the finite element model were in good agreement with those generated during the test. However, Key geometric parameters are identified by studying the existing literature. To investigate the effects of parameters, a single panel of steel plate shear wall with rigid floor beams subjected to shear and gravity loads. The capacity of the system is investigated with varying parameters.

6.2 Conclusion

Finite element model, developed by using a nonlinear static formulation, gave an excellent prediction of the monotonic and cyclic behavior of three unstiffened steel plate shear walls without any solution difficulty. The proposed model is able to capture all essential features of the behavior of steel plate shear walls.

A parametric study of some of the primary variables was conducted and the following conclusions can be drawn from this analysis:

a. Changing the aspect ratio (= L / h) of the shear wall panels within the range of 0.8 to 1.4 has significant effect (50% increase for 10 mm steel plate) on the strength of a shear wall panel.

b. Increasing the plate thickness (t_p) of steel shear wall leads to the increasing capacity and stiffness of the system. However, for wide (aspect ratio 1.4) shear wall beyond the certain thickness (10 mm; $\frac{H}{t_p}$ =330) capacity increasing becomes saturated (Only 4% increment when increasing the plate thickness from 10mm to 15mm).

c. Lateral base shear is partially shared by column and steel plate. It is observed that thick shear wall plate (>10 mm; $\frac{H}{t_p}$ =330) takes almost 75% of the total energy.

d. Column flexibility parameter, has a significant effect (38% increase of capacity by changing the parameter from 3.16 to 2.02 and 18% increase of the capacity by changing the parameter from 2.02 to 1.65) on the behavior of steel shear walls. Relationship between column flexibility parameter and capacity of shear wall is linear and shows a good correlation.

e. Capacity augmentation of steel plate shear wall has been investigated using horizontal strut. It is observed that, Horizontal strut can improve capacity (13% increment of the capacity for 10mm steel plane) of narrow, tall shear wall (aspect ratio 0.80).

6.3 Future Recommendation

For future research on steel plate shear wall system, several recommendations are proposed. They are given below:

a. The research focused on the effectiveness of horizontal strut; further research can be performed for vertical stiffener/strut.

b. Extensive parametric study can be performed using composite column (with variable concrete strength, variable link spacing and variable rebar diameter), which will be more cost effective than steel column.

c. Studies should be performed for usage of steel plate shear wall in building system. Different types of structural systems are available for steel plate shear wall usage such as shear wall with opening, dual steel plate shear wall system.

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