LATERAL STIFFNESS OF UNSTIFFENED STEEL PLATE SHEAR WALL SYSTEMS

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LATERAL STIFFNESS OF UNSTIFFENED STEEL PLATE SHEAR WALL SYSTEMS

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ABSTRACT

LATERAL STIFFNESS OF UNSTIFFENED STEEL PLATE SHEAR WALL SYSTEMS

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Finite element method and strip method are two widely used techniques for analyzing steel plate shear wall (SPSW) systems. Past research mostly focused on the prediction of lateral load capacity of these systems using these numerical methods. Apart from the lateral load carrying capacity, the lateral stiffness of the wall system needs to be determined for a satisfactory design. Lateral displacements and the fundamental natural frequency of the SPSW system are directly influenced by the lateral stiffness. In this study the accuracy of the finite element method and strip method of analysis are assessed by making comparisons with experimental findings. Comparisons revealed that both methods provide in general solutions with acceptable accuracy. While both methods offer acceptable solutions sophisticated computer models need to be generated. In this study two alternative methods are developed. The first one is an approximate hand method based on the deep beam theory. The classical deep beam theory is modified in the light of parametric studies performed on restrained thin plates under pure shear and pure bending. The second one is a computer method based on truss analogy. Stiffness predictions using the two alternative methods are found to compare well with the experimental findings. In addition, lateral stiffness predictions of the alternate methods are compared against the solutions provided using finite element and strip method of analysis for a class of test structures. These comparisons revealed that the developed methods provide estimates with acceptable accuracy and are simpler than the traditional analysis techniques.

Keywords: Steel plate shear wall, stiffness, finite element, strip method.

ÖZ

ÇELİK PLAKALI PERDE DUVARLI SİSTEMLERİN YATAY RİJİTLİĞİ

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Sonlu elemanlar metodu ve şerit (strip) metodu, çelik plakalı perde duvarlı sistemlerin analizleri için yaygın olarak kullanılan tekniklerdir. Daha önceki yapılan çalışmalar çoğunlukla bu tarz nümerik metotları kullanılarak bu tür sistemlerin yatay yük taşıma kapasitelerinin tahmini üzerine odaklanmıştır. Perde duvarlı sistemin yatay yük taşıma kapasitesinin dışında başarılı bir tasarım için yatay rijitliğinin belirlenmesine ihtiyacı vardır. Çelik plakalı perde duvarlı sistemlerin yatay yer değiştirmeleri ve doğal frekansı yatay rijitlikleri tarafından doğrudan etkilenmektedir. Bu tezde sonlu elemanlar metodu ve şerit metodunun doğruluğu deneysel bulgular ile karşılaştırmalar yapılarak değerlendirilmiştir. Karşılaştırmalar her iki metodun da genel sonuçlar içinde kabul edilebilinir doğrulukları sağladığını göstermiştir. Her iki metot da kabul edilebilinir sonuçlar sunarken karmaşık bilgisayar modellerinin oluşturulmasına da ihtiyaç vardır. Bu tezde de iki alternatif metot geliştirilmiştir. Bunlardan birincisi derin kirişlerin eğilme teorisi üzerine kurulmuş yaklaşık el metodudur. Klasik eğilme teorisi yalnız kesme ve eğilme altındaki tutulmuş plakalar üzerinde gerçekleştirilmiş parametrik çalışmaların ışığında yeniden düzenlenmiştir. İkinci alternatif metot ise kafes kiriş sistem mantığına dayanan bir bilgisayar modeli metodudur. İki alternatif metodu kullanılarak rijitlik tahminleri deneysel bulgular ile karşılaştırılmıştır. Ayrıca alternatif metotların yatay rijitlik tahminleri, test edilen yapıların bir bölümü için sonlu elemanlar ve şerit metotları kullanılarak elde edilen analiz sonuçları ile de karşılaştırılmıştır. Bu karşılaştırmalar geliştirilen metotların kabul edilir doğrulukta tahminleri sağladığını ve geleneksel analiz tekniklerinden daha basit olduğunu göstermiştir.

Anahtar Kelimeler: Çelik plakalı perde duvar, rijitlik, sonlu eleman, şerit (strip) metot.

To My Parents

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I would like to express my deepest gratitude to my supervisor Assoc. Prof. Dr. Cem TOPKAYA for his guidance, suggestions, encouragements and support throughout the development and writing of this thesis.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Steel plate shear walls (SPSWs) can be used as a primary lateral load resisting system capable of effectively and economically bracing against both wind and earthquake forces for buildings. A conventional steel plate shear wall consists of steel plates bounded by steel columns and beams. Steel infill plates one story high and one bay wide can be connected to the surrounding beams and columns by using either welded or bolted connections. The resulting system is a stiff cantilever wall which resembles a vertical plate girder. Columns are analogous to the flanges of the girder while beams are analogous to the stiffeners. Slender infill plates are susceptible to buckling when subjected to shear stresses. To prevent buckling at low lateral loads either thick or stiffened plates can be used with the outcome of increased expenses. On the other hand, more recent approaches rely on the post buckling strength and stiffness.

Based on the work of Wagner (1931), it has been known that buckling does not necessarily represent the limit of structural usefulness and there is considerable post buckling strength possessed by the restrained unstiffened thin plates. At the onset of buckling, the load carrying mechanism changes from in-plane shear to an inclined tension field. For a thin panel, the shear buckling strength is low and, as a result, the tension field action is the main mechanism for carrying the

applied shear forces. In order for the tension field to fully develop, the boundary members should have sufficient bending stiffness.
The consideration of the post-buckling strength of plates has been accepted in the design of plate girder webs for many years based largely on the work of Basler (1961). Experimental studies (Timler and Kulak 1983; Tromposch and Kulak 1987; Caccese et al. 1993; Driver et al. 1998a; Lubell et al. 2000; Park et al. 2007) reported to date revealed that steel plate shear walls possess properties that are fundamentally beneficial in resisting seismically-induced loads and unstiffened SPSW systems have high stiffness, excellent energy absorption capacity and stable hysteresis characteristics.

In the past strip method (Thornburn et al. 1983) was developed for the analysis of SPSW systems. An analytical model termed the strip model was developed to simulate the tension field behaviour, wherein the infill plate is modeled as a series of tension only strips oriented at the same angle of inclination, α, as the tension field. The strip model assumes that the boundary beams are infinitely stiff in order to reflect the presence of opposing tension fields above and below the modeled panel. The model studied in Thornburn's research program used hinged connections at the beam ends (Fig. 1.1 and Fig. 1.2a).

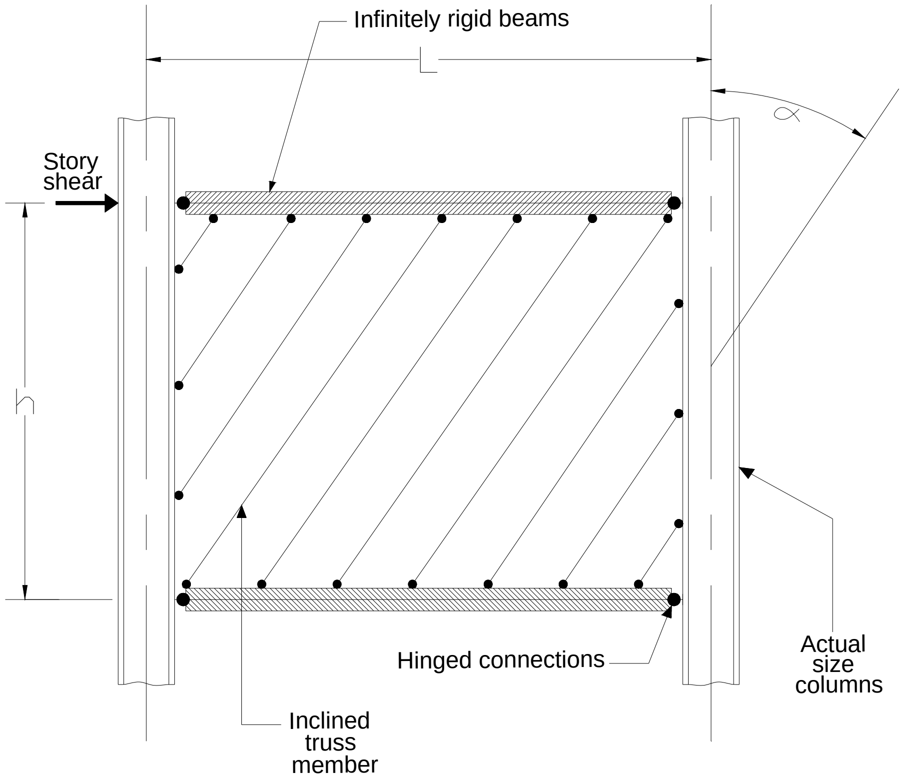


Figure 1.1 Development of Strip Model

Equations for calculating the inclination of pin ended strips were developed (Thornburn et al. 1983; Tromposch and Kulak 1987) based on the energy principals. Using the principle of least work, Thorburn et al. (1983) derived an equation for α that takes the following form:

$$\alpha = \tan^{-1} \sqrt[4]{\frac{1 + \frac{tL}{2A_c}}{1 + \frac{th}{A_b}}} \tag{1.1}$$

where t is the thickness of the infill plate, h is the story height, L is the bay width, A_c is the cross-sectional area of the vertical boundary element, and A_b is the cross-sectional area of the horizontal boundary element. The derivation included the effect of the axial stiffness of the boundary members, but not the flexural stiffness.

To verify the analytical method developed by Thorburn et al. (1983), Timler and Kulak (1983) tested a full-scale specimen that represented two single–storey, one–bay steel plate shear wall elements. These researchers recognized that the flexural stiffness of the columns affects the value of α. Thus, the equation for α, originally developed by Thorburn et al. (1983), was modified as follows:

$$\alpha = \tan^{-1} \sqrt[4]{\frac{1 + \frac{tL}{2A_c}}{1 + th\left(\frac{1}{A_b} + \frac{h^3}{360 I_c L}\right)}} \tag{1.2}$$

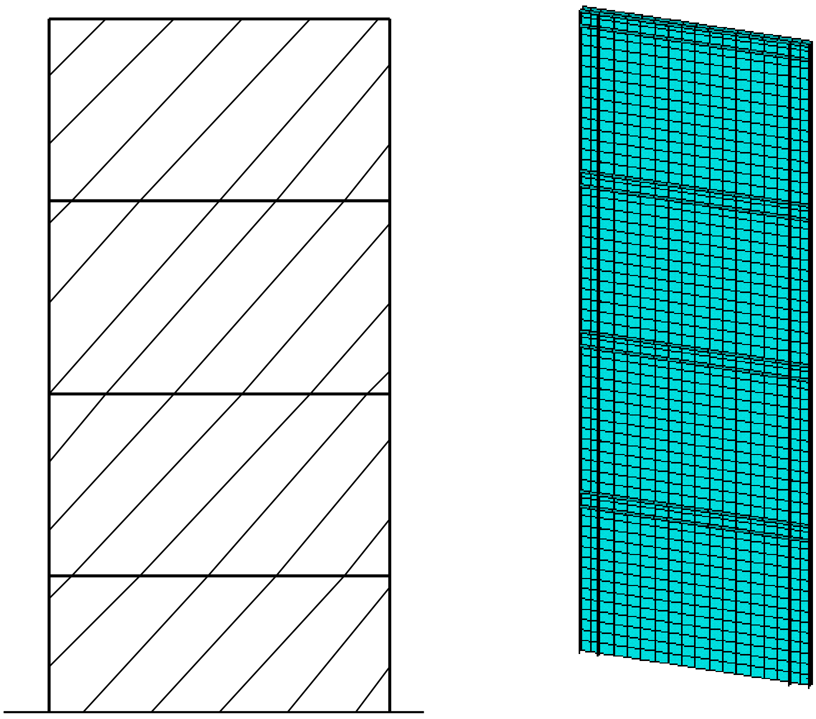
where I_c is the moment of inertia of the vertical boundary element.

The stiffness and capacity of an unstiffened steel plate shear wall system mainly depends on the development of the tension field in the infill plate. In order to develop a uniform tension field, the boundary members should have enough flexural stiffness to anchor the tension field. In-plane transverse deformations of the boundary members release the tension field in the infill plate and reduce the effectiveness of the system. For interior beams or columns of a steel plate shear wall, the presence of equal and opposite tension fields usually keeps the flexural deformations small. At the top and bottom panels, however, enough rigidity should be provided to anchor the tension field. Similarly, in order to increase the effectiveness of a steel plate shear wall, enough flexural stiffness should be provided by the columns on the perimeter of the shear wall. To prevent excessive deformation leading to premature buckling under the pulling action of the plates, the following equation is used to calculate the required minimum moment of inertia of the columns;

$$I_c \geq \frac{0.00307 w h^4}{L} \tag{1.3}$$

where w is the panel thickness.

In recent years due to advances in hardware technology finite element method is employed more frequently for the analysis of these systems. Finite element models with differing complexity were used in the past. Elgaaly et al. (1993) and Driver et al. (1998b) modeled the plate with shell elements and beams/columns with frame elements. Behbahanifard (2003) modeled the whole structure using shell elements (Fig. 1.2b). Although three dimensional finite element modeling provides a rigorous analysis methodology its use in routine design environment is limited. The complexities in modeling with finite elements and the strip method presented the need for simpler yet accurate approaches. Recently some hand techniques were developed (Berman and Bruneau 2003; Sabouri-Ghomi et al. 2005; Kharrazi 2005) to calculate the lateral capacity of SPSW systems.



(a) (b)

Figure 1.2 a) Strip Model b) Three Dimensional Finite Element Modeling

Most of the previous research work has focused on the lateral capacity of SPSW systems with little emphasis on the lateral stiffness. During the design of an SPSW system deflections under lateral forces are calculated and these values are compared against the drift limitations dictated by the design codes. In addition, fundamental period of vibration of the SPSW needs to be calculated reliably to be able to estimate the amount of lateral forces. Based on this discussion it is apparent that accurate computation of lateral stiffness of an SPWS has paramount importance during the design.

1.2 OBJECTIVES AND SCOPE

There are two main objectives of this thesis. First the accuracy of existing methods of analysis is evaluated. For this evaluation experimental studies on multistory steel plate shear walls are reviewed. Experimented specimens are analyzed with finite element method and strip method to estimate the stiffness of the system. Chapter 2 focuses on the literature review of experimental studies on multistory systems and modeling with existing analysis methods.

Second, two new techniques are developed to predict the lateral stiffness of the steel plate shear wall system. One of these new techniques is an approximate hand method and the other one is a computer method based on truss analogy. In Chapter 3 detailed formulation for these new techniques are presented. The predictions of these developed methods are compared with experimental observations and with the predictions of existing analysis methods in Chapter 4. Finally conclusions are given in Chapter 5.

CHAPTER 2

EVALUATION OF EXISTING METHODS

2.1 OVERVIEW OF EXPERIMENTAL STUDIES ON MULTISTORY STEEL PLATE SHEAR WALLS (SPSWS)

In this chapter, in order to make an assessment of the existing numerical methods and the methods proposed in this thesis, experimental studies of four independent research teams are considered. Emphasis is given on the multistory specimens rather than the single story specimens because these are the ones that represent the real practice. Following is an overview of the properties of the specimens tested by the research groups.

2.1.1 Caccese, Elgaaly, Chen (1993)

Caccese et al. studied the cyclic behavior of steel plate shear wall systems and demonstrated the effective use of the post-buckling strength of steel plate panels in SPSWs. Six 1:4 scale specimens of different plate thickness and beam-to-column connections were tested at roof level under cyclic horizontal loading and monotonic loading. The tests were performed without any dead load applied to the columns. Two important parameters under study were the effects of beam-to-column connections and the panel slenderness ratio on overall behaviour of SPSW. One of the six specimens was used for the moment frame. Three specimens were built with moment resisting beam-to-column connections and infill plate thickness of 0.76 mm, 1.90 mm and 2.66 mm. The last two specimens were constructed with shear beam-two-column connections and infill plate thicknesses of 0.76 mm and 1.90 mm.

Three specimens which had moment resisting connections are considered in the numerical investigations. These three specimens namely M22, M14, M12 had 22 gage (0.76 mm), 14 gage (1.9 mm), 12 gage (2.66 mm) thick specified steel plate panels, respectively. The layout of the three story one bay specimens is given in Figure 2.1. The S3x5.7 beams and W4x13 columns made up of ASTM A36 steel is used in the frames. The center-to-center spacing and storey height are 1245 mm and 838 mm, respectively. In the frames there is a stiff panel at the top to anchor the tension field in the upper story. The specimens were attached by welding application to a base plate at the bottom which is fastened to the floor by high strength bolts. Table 2.1 shows the geometrical properties of the beam, column and plate sections.

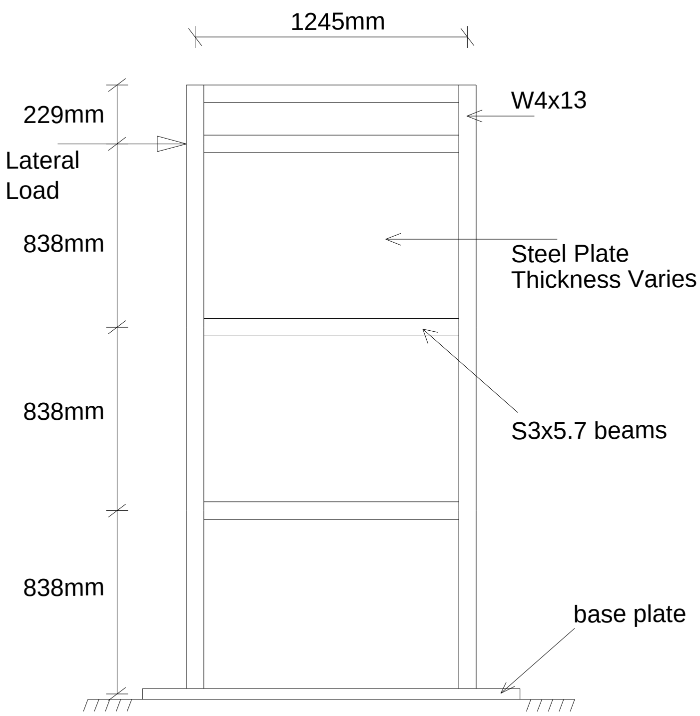


Figure 2.1 Details of Specimens Tested by Caccese et al. (1993)

Table 2.1 Geometrical Properties of Frame Components.

Section	Depth (d) mm	Flange Width (b_f) mm	Flange Thickness (t_f) mm	Web Thickness (t_w) mm	Area mm^2	Moment of Inertia (I_x) mm^4
S3x5.7	76.2	59.1	6.6	4.3	1077	$1.04x10^6$
W4x13	105.6	103.1	8.8	7.1	2470	$4.7x10^6$
	M22	M14	M12			
Plate thickness mm	0.76	1.87	2.65			

By using an actuator the test specimens were loaded with a single in-plane horizontal load at the top of the third story. After that the displacements and strains were recorded. Each specimen was loaded cyclically with gradually increasing deflections up to a maximum of 51 mm (2% drift) measured at the top of the shear wall.

2.1.2 Driver, Kulak, Kennedy, Elwi (1998)

Driver et al. (1998) tested a large-scale four-story and one bay steel plate shear wall system to evaluate the performance of this type of structure under severe cyclic loading. The test specimen had unstiffened panels and moment-resisting beam-to-column connections. Geometrical properties of the experimented specimen are given in Figure 2.2.

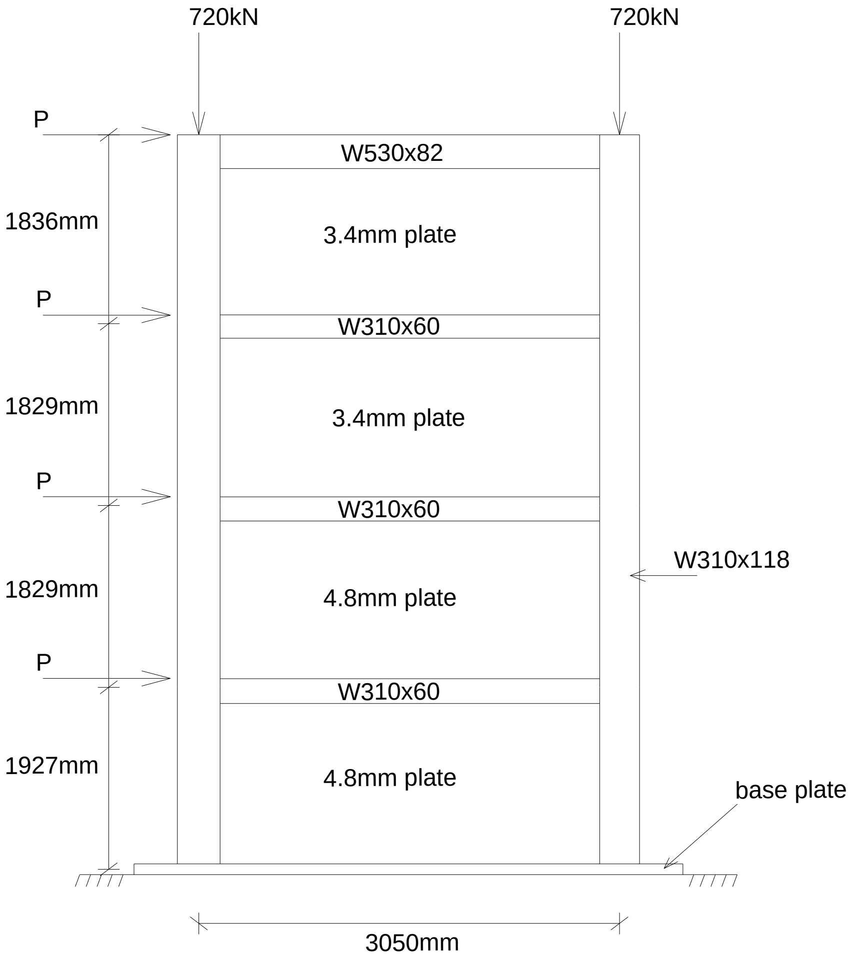


Figure 2.2 Details of Specimen Tested by Driver et al. (1998)

As it can be seen from Figure 2.2, the overall height of the specimen is 7.421 m, the typical story height is 1.83 m in the top three stories and the first story has a height of 1.93 m. Columns are 3.05 m apart from center-to-center. These dimensions are representative of a shear wall at half scale of an office building of 3.60 m typical story height, or about 60% scale for a residential building. The infill plates for the first and second storey were 4.8 mm thick (the mean measured thicknesses for panels are 4.54 mm and 4.65 mm). The third and fourth storey panels were filled with 3.4 mm thick plates (the mean measured thicknesses for panels are 3.35 mm and 3.40 mm). The four stories have two columns with a W310x118 section on both sides without splices. Beam sections at first, second and third floor levels are W310x60 and the beam section at fourth floor level is W530x82. Connection of beam flanges to the columns was made using complete penetration groove welds. Geometrical properties of the beam and columns cross sections are given in Table 2.2.

Table 2.2 Geometrical Properties of Frame Components

Section	Depth (d) mm	Flange Width (b_f) mm	Flange thickness (t_f) mm	Web thickness (t_w) mm	Area mm^2	Moment of Inertia (I_x) mm^4
W310x118	314	307	18.7	11.9	14966	$275x10^6$
W310x60	303	203	13.1	7.5	7590	$129x10^6$
W530x82	528	209	13.2	9.6	10473	$475x10^6$

2.1.3 Lubell, Prion, Ventura, Rezai (2000)

Lubell et al. (2000) tested a single four story steel plate shear wall (SPSW4). The specimen represented 25% scale model of one bay of a steel framed office building core. Geometrical properties of the experimented specimen are given in Figure 2.3. The panel aspect ratio of the specimen was 1.0 and plate thickness was 1.5 mm for all stories. The specimen has S75x8 columns and horizontal members. At the top story S200x34 section was used to anchor the tension field. All beam to column connections were moment connections. Equal horizontal loads were applied at each floor level. Before lateral loads were applied vertical load of 13.5 kN was applied to each story using steel masses.

Geometrical properties of the beam and columns cross sections are also given in Table 2.3.

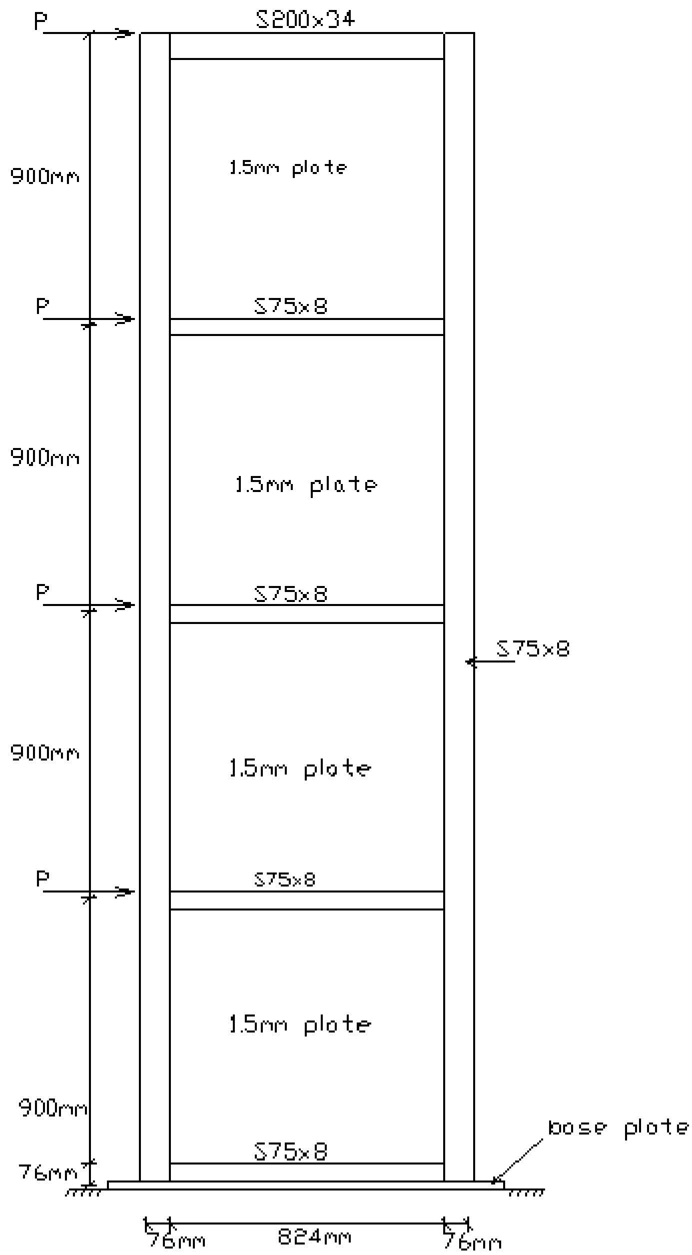


Figure 2.3 Details of Specimen Tested by Lubell et al. (2000)

Table 2.3 Geometrical Properties of Frame Components

Section	Depth (d) mm	Flange Width (b_f) mm	Flange thickness (t_f) mm	Web thickness (t_w) mm	Area mm^2	Moment of Inertia (I_x) mm^4
S75x8	76	64	6.6	8.9	1403.72	1203973
S200x34	203	106	10.8	11.2	4321.28	26738390

2.1.4 Park, Kwack, Jeon, Kim, Choi (2007)

Park et al. (2007) tested five three story steel plate shear walls. Test specimens were one-third models of three story prototype walls and an experimental study was performed to investigate the cyclic behavior of these walls. For all specimens plate thickness was constant along the height of the wall. Two different built-up column sections were considered. Specimens SC2T, SC4T, and SC6T had H-250x250x20x20 column sections while specimens WC4T, and WC6T had H-250x250x9x12 column sections. For all specimens beams at first and second stories were H-200x200x16x16 built-up wide flange sections and the top beam was made up of H-400x200x16x16 section. Lateral loading was applied at the top story and no axial load was used in testing. Specimen SC2T had a plate thickness of 2.42 mm, and specimens SC4T and WC4T had a plate thickness of 4.49 mm, and specimens SC6T and WC6T had a plate thickness of 6.5 mm. Aspect ratio of the infill plate was 1.5. Geometrical properties of the beam and columns cross sections are given in Table 2.4.

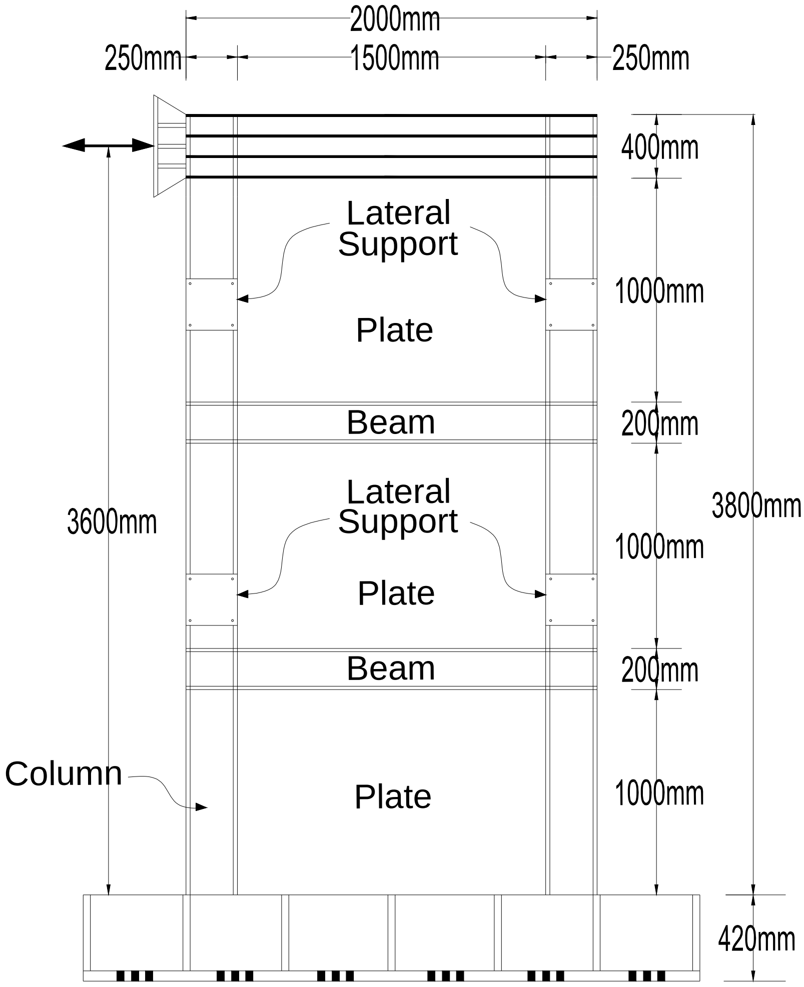


Figure 2.4 Details of Specimen Tested by Park et al. (2007)

Table 2.4 Geometrical Properties of Specimens

Specimen	Column section	Plate thickness mm	Depth (d) mm	Flange Width (b_f) mm	Flange thickness (t_f) mm	Web thickness (t_w) mm	Area mm^2	Moment of Inertia (I_x) mm^4
SC2T	H-250X250X20X20	2.42	250	250	20	20	14200	148018333
SC4T	H-250X250X20X20	4.49	250	250	20	20	14200	148018333
SC6T	H-250X250X20X20	6.50	250	250	20	20	14200	148018333
WC4T	H-250X250X9X12	4.49	250	250	12	9	8034	93695382
WC6T	H-250X250X9X12	6.50	250	250	12	9	8034	93695382
	Beam section		Depth (d) mm	Flange Width (bf) mm	Flange thickness (tf) mm	Web thickness (tw) mm	Area mm^2	Moment of Inertia (I_x) mm^4
	H-200X200X16X16		200	200	16	16	9088	60628309
	H-400X200X16X16		400	200	16	16	12288	3.03×10^8

2.2 ANALYSIS OF EXPERIMENTED SPECIMENS USING EXISTING METHODS

Lateral stiffness of ten specimens is computed using strip and finite element method of analysis. Details of the specimens are given in Table 2.5. A commercially available finite element program ANSYS (2006) is used to conduct the analysis. For specimens tested by Lubell et al. (2000) and Driver et al. (1998a) researchers reported deflections at the first story and top story. Therefore, for these specimens two stiffness values (Total lateral load divided by the deflection at a particular story level) are considered.

Table 2.5 Lateral Stiffness of Experimented Specimens and Analysis Results

Case	Study	Specimen	# of story	Load Application	Axial Load	Stiffness based on displacement at	Experimental Stiffness (kN/mm)	FEM/Experimental	Strip/Experimental
1	Lubell and others	SPSW4	4	Equal Lateral	Yes	FS	17	2.84	2.10
						TS	4.2	2.10	1.64
2	Driver and others	------	4	Equal Lateral	Yes	FS	425	1.00	0.88
						TS	96	1.06	0.82
3	Caccese and others	M22	3	Top Story	No	TS	14.22	1.32	0.88
4		M14	3		No	TS	22.06	1.57	0.84
5		M12	3		No	TS	26.61	1.67	0.77
6	Park and others	SC2T	3	Top Story	No	TS	83	1.04	0.74
7		SC4T	3		No	TS	111	1.11	0.74
8		SC6T	3		No	TS	120.5	1.31	0.79
9		WC4T	3		No	TS	92.5	1.00	0.68
10		WC6T	3		No	TS	98	1.18	0.73
					Average (w/o case1)			1.23	0.79
					Standard Deviation (w/o case1)			0.24	0.07

FS: first story, TS: top story

In all cases 12 strips are used for every panel and strip angle is found using the recommendations given in AISC Seismic Provisions for Structural Steel Buildings (2005) (Eqn. 1.2). In the strip method of analysis horizontal and vertical boundary elements are modeled with frame (beam3) elements and inclined strips are modeled with truss (link8) elements. A geometrically linear analysis is conducted and the analysis results are given in Table 2.5.

In the finite element method of analysis SPSW systems are modeled with shell (shell93) elements. In order to simulate the post buckling response geometrical imperfections have to be introduced into the finite element model. Previous studies (Behbahanifard 2003) revealed that the magnitude of initial imperfection does not have a major effect on the capacity but slightly affects the stiffness of the system. However, it was found out (Behbahanifard 2003) that for imperfection sizes larger that 1% of $\sqrt{Lh}$ (L and h represent the side lengths of the infill plate) stiffness reduction is noticeable. This value of imperfection is significantly higher than the ones encountered in practice. Therefore, imperfection values taken within normal fabrication tolerances do not have a significant effect on the analysis results. For all the three dimensional models a center imperfection of 3 mm is considered for the infill panels. This value is well within the limit recommended by Behbahanifard (2003). A representative deflected shape of Driver's specimen is given in Fig. 2.5. In addition, during the early stages of research other imperfection values in the vicinity of 3mm were used and were found not to significantly affect the analysis results. A geometrically nonlinear analysis is conducted and SPSW systems are subjected to a lateral drift equal to the drift at first yield observed during the experiments. In computing the lateral stiffness secant stiffness at first yield is considered and the analysis results are presented in Table 2.5.

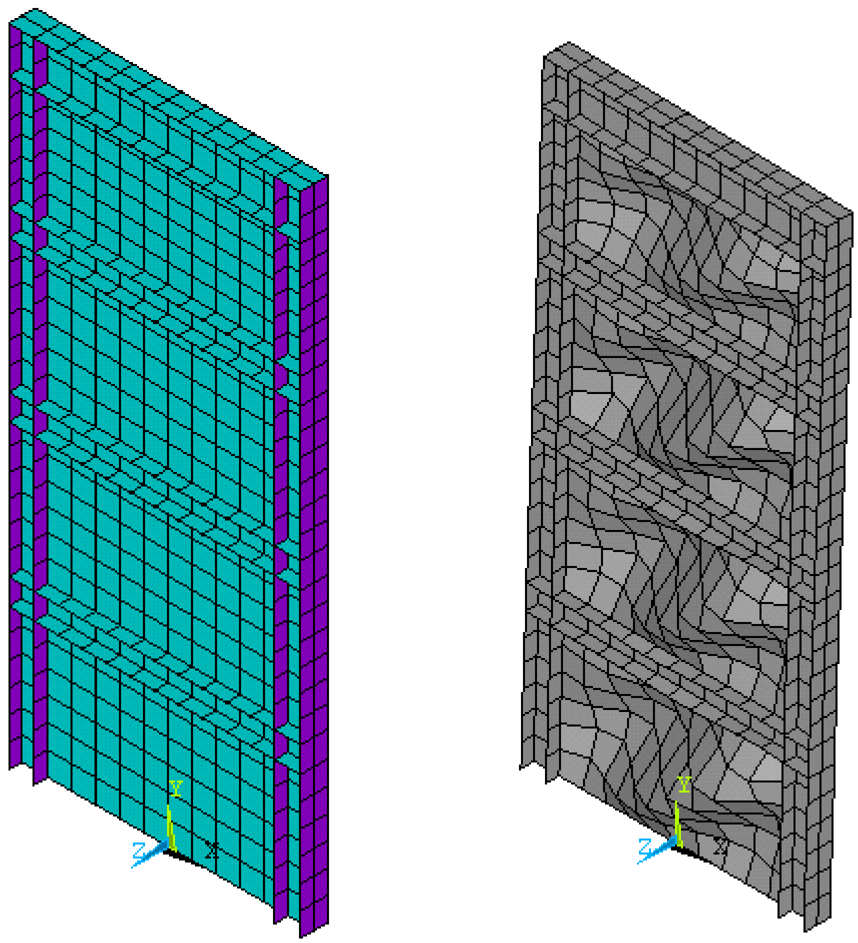


Figure 2.5 Finite Element Model and Deflected Shape of Driver Specimen

When analysis results are examined it is observed that the stiffness if specimen SPSW4 tested by Lubell et al. (2000) is significantly overpredicted by both methods. Same observation was pointed out by Rezai et al. (2000) and these researchers proposed a multi angle strip model to better predict the lateral stiffness of this specimen. When analysis results for other specimens are considered it is found that both methods provide predictions with acceptable accuracy. At this point the significant overprediction of stiffness for SPSW4 is inconclusive and the data related to this specimen is excluded from the statistical analysis presented in this thesis. Analysis results revealed that three dimensional finite element method overpredicts the stiffness while strip method of analysis provide underpredictions. The FE results is on average 23 percent higher than the experimental results while the same figure is 21 percent lower for the strip method results.

In general finite element method offers stiffer solutions due to the restricted displacement fields assumed in the solution process. In addition it is very difficult to input the complex deformed shape of the infill plate into the analysis model.

Strip method was developed to find out the lateral capacity of SPSW systems and can underpredict the lateral stiffness. This phenomenon was observed by Driver (1997) in the past. In fact Driver (1997) developed an effective infill plate thickness concept based on energy theorems to better predict the lateral stiffness of SPSW systems. For the usual range of tension field angle an effective thickness equal to 1.55 times the actual thickness was proposed by Driver (1997). This concept was applied to the analysis of specimen tested by Driver et al. (1998a). It was found that even with this increase in stiffness in the model, the behavior is still less stiff than that of the specimen.

At this point it can be concluded that the actual value of stiffness lies between the predictions offered by the strip and finite element methods. The finite element method solutions can be considered as an upper bound while strip method solutions can be regarded as a lower bound estimate. It is observed from Table 2.5 that the maximum amount of overprediction is much more pronounced as compared to the maximum amount of underprediction (excluding the SPSW4 specimen). Stiffness of specimens M14 and M12 are overpredicted by 57% and 67%, respectively using finite element method. In author's opinion this level of overprediction is due to the unavoidable out of plane deflection of the specimen as reported by the researchers (Elgaaly et al. 1993). In fact a similar specimen (S14) with pinned beam to column connections exhibited 21% higher stiffness than the one with rigid beam to column connections (M14). While this observation is counterintuitive it strengthens the assertion that testing conditions played an important role on the experimental results.

CHAPTER 3

DEVELOPMENT OF ALTERNATIVE TECHNIQUES FOR CALCULATING LATERAL STIFFNESS

Calculating lateral stiffness of steel plate shear wall systems presents a variety of challenges. Geometrical nonlinearity due to post buckling of infill plates has to be taken into account during the computations. An approximate hand method and a computer method are developed as a part of this study to calculate the stiffness of the wall system.

3.1 APPROXIMATE HAND METHOD

The approximate hand method is based on the observation that the steel plate shear wall is analogous to a vertical plate girder. First, the mechanics of the problem is studied for the case where geometrical nonlinearity is excluded from the analysis. Second, geometrical nonlinearity is included in the analysis and a simple way of considering the effect of post buckling behavior is developed. Following sections present the details of the development.

3.1.1 Geometrically Linear Analysis

If geometrical nonlinearities due to post buckling of infill plates are excluded from the behavior then lateral deflections of steel plate shear walls can be calculated from the elementary beam theory. As mentioned before steel plate shear walls are actually vertical plate girders where columns act as the flanges, infill plates act as the web and beams act as the stiffeners. In calculating lateral deflections both shear and bending deformations need to be included in the calculations. Castigliano's second theorem is one of the most widely used methods for calculating beam deflections (Ugural and Fenster 2003). The strain energy must be represented as a function of external loads. The total strain energy (*U*) stored in the system is composed of strain energy stored in the beam due to bending (U_b) and shear (U_s) as given in Eqn. 3.1.

$$U = U_b + U_s \tag{3.1}$$

Strain energy due to bending (U_b) can be represented as a function of cross sectional moment (*M(x)*) along the wall height as in Eqn. 3.2.

$$U_b = \int_0^H \frac{(M(x))^2}{2EI} dx \tag{3.2}$$

where; *E*: modulus of elasticity, *I*: moment of inertia, *H*: height of the steel plate shear wall system.

Strain energy due to shear (U_s) can be represented as a function of cross sectional shear (*V(x)*) along the wall height as given in Eqn. 3.3.

$$U_s = \int_0^H \frac{(V(x))^2 \beta}{2GI^2} dx \tag{3.3}$$

where; *G*: shear modulus of elasticity.

The β factor in Eqn. 3.3 is dependent on the cross sectional properties and can be found as follows:

$$\beta = \int_A \frac{Q^2}{b^2} dA \tag{3.4}$$

where; *Q*: statical moment of the area with respect to neutral axis, *b*: width of the section, *A*: area of steel plate wall.

If a typical steel plate shear wall cross section is examined the qualitative variation of shear stresses on the section which is a function of the ratio of *Q/b* can be depicted as in Fig. 3.1.

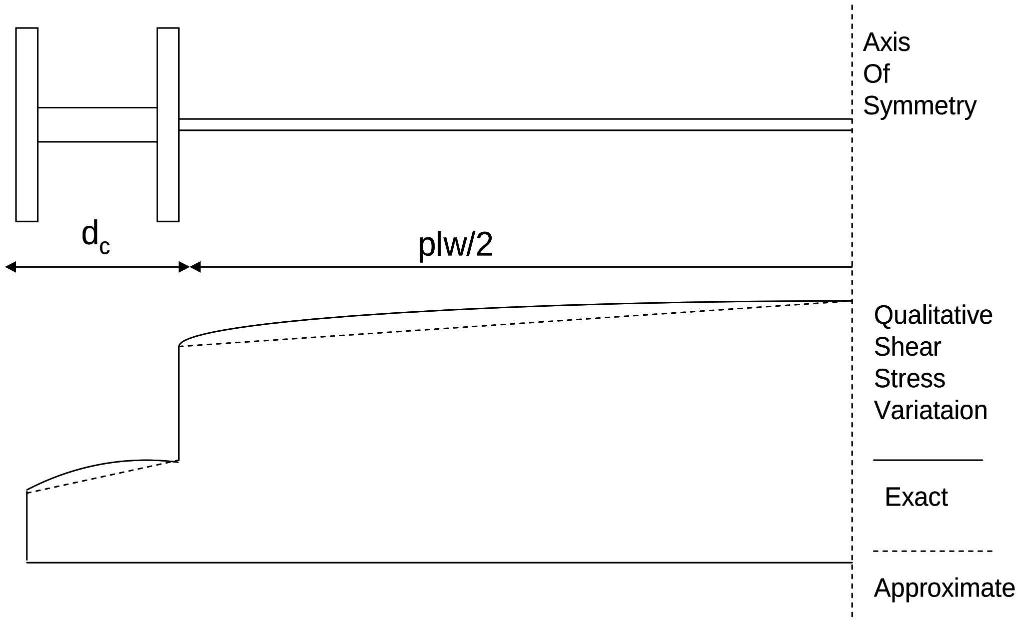


Figure 3.1 Shear Stress Variation for a Steel Plate Shear Wall System

The exact calculation of β requires the integration of fourth order polynomials that might be cumbersome during routine design practice. However, observing that a linear variation assumption along the continuity regions lend itself to a much simpler β equation that can be expressed as follows:

$$\beta = \beta_1 + \beta_2$$

$$\beta_1 = \frac{Q_1^2 + Q_2^2}{t_{cw}} d_c \qquad \beta_2 = \frac{Q_3^2 + Q_4^2}{2ptk} plw$$

$$Q_1 = A_{cf}(0.5plw + d_c)$$

$$Q_2 = Q_1 + A_{cw} 0.5(plw + d_c)$$

$$Q_3 = A_c 0.5(plw + d_c)$$

$$Q_4 = Q_3 + \frac{(plw)^2}{8} ptk \tag{3.5}$$

where; plw: width of the infill plate, ptk: thickness of the infill plate, d_c: depth of column section, t_{cw}: thickness of column web, A_{cf}: area of column flange, A_{cw}: area of column web, A_c: area of column.

In Eqn. 3.5 β_1 represents the contribution of shear stresses in the column section and β_2 represents the contribution of shear stresses in the infill plate. Among the two β_2 is much larger compared to β_1 for typical steel plate shear wall geometries.

After obtaining geometrical constants the displacement at a given location can be found by using Castigliano's second theorem as follows:

$$\Delta = \frac{\partial U}{\partial P} = \frac{\partial U_b}{\partial P} + \frac{\partial U_s}{\partial P} \tag{3.6}$$

$$\Delta = \int_0^H \frac{M(x)}{EI}\left(\frac{\partial M(x)}{\partial P}\right)dx + \int_0^H \frac{V(x)\beta}{GI^2}\left(\frac{\partial V(x)}{\partial P}\right)dx \tag{3.7}$$

where; P: force acting at the point where displacement is sought.

The procedure listed above requires that the moment and shear are written in terms of the distance along the height of the wall and the applied load P. Rather than using energy methods, virtual work method can also be used in the same fashion to come up with the same conclusion.

For the case of a steel plate wall with constant inertia under the action of a lateral load P acting at the top of the wall, the tip deflection reduces to:

$$\Delta = \Delta_b + \Delta_s$$

$$\Delta_b = \frac{PH^3}{3EI} \qquad \Delta_s = \frac{PH\beta}{GI^2} \tag{3.8}$$

where; Δ_b: deflection due to bending, Δ_s: deflection due to shear.

The general expression (Eqn. 3.7) can be used to calculate deflections of SPSW systems with variable infill plate thickness and subjected to different lateral load variations along the height. Solutions for some typical cases are given in Eqn. 3.9.

Tip deflection of an SPSW system with variable infill thickness can be calculated as follows:

$$\Delta_b = \sum_{i=1}^{n} \frac{C_{bi} P h^3}{E I_i} \qquad \Delta_s = \sum_{i=1}^{n} \frac{C_{si} P h \beta_i}{G I_i^2}$$

$$C_{si} = 1 \qquad C_{bi} = 0.5(n-i+2/3) + (n-i)(n-i+0.5) \quad \text{Tip Load} \tag{3.9}$$

$$C_{si} = n-i+1 \quad C_{bi} = \left(\sum_{k=1}^{n-i} k \right)(n-i+0.5) + 0.5(n-i+1)(n-i+2/3) \quad \text{Equal Lateral Load}$$

where; n: number of storeys, i: i^th^ panel (story number counted from the base), C_{si}, C_{bi}: shear and bending coefficients, β_i: β factor for i^th^ panel, I_i: moment of inertia for the ith panel, P: tip load or story load, h: story height.

These equations are verified for a class of SPSW systems details of which are given in later sections of the thesis and are analyzed with linear finite element analysis. Comparisons revealed that these expressions are satisfactory in finding lateral stiffness of systems solved under geometrically linear assumption (Fig. 3.2). In general, stiffness values obtained using the hand method are lower than the ones obtained from finite element analysis. Maximum difference between the results is less than 15 percent for the test structures that are considered.

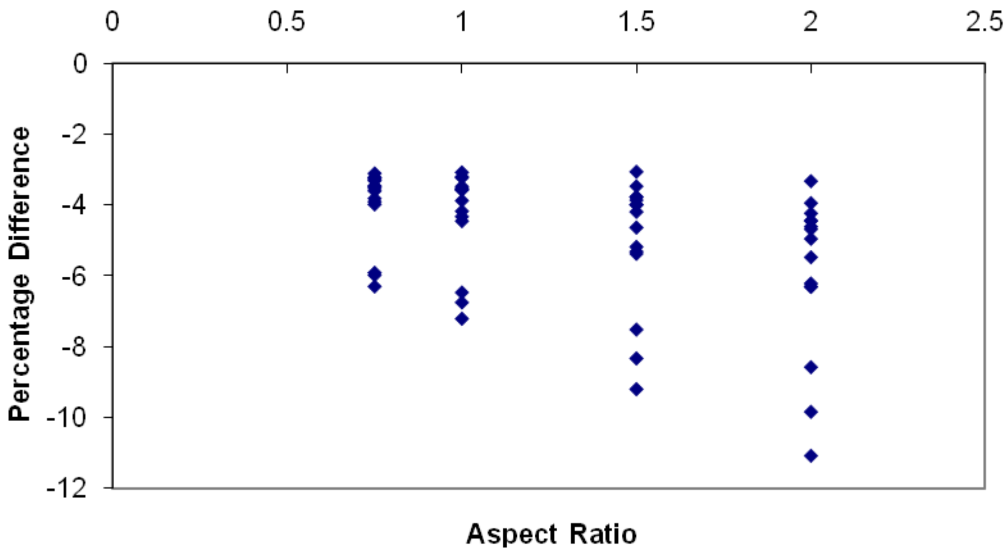


Figure 3.2 Comparison of Finite Element Method (FEM) and Hand Method under Geometrically Linear

3.1.2 Geometrically Nonlinear Analysis

In the expressions developed so far post buckling response of infill plates is not considered. In reality however, the infill plates buckle at very low lateral load levels and the post buckling stiffness contributes to the overall stiffness of the system. Expressions developed on the basis of geometrically linear theory need to be modified to account for the loss of stiffness due to buckling of infill plates. Buckling of restrained thin plates is not synonymous with failure. Plate has significant post buckling strength and stiffness after the critical buckling load is reached. Up till now most of the research work has focused on the reserve strength possessed by restrained thin plates. In this study the post buckling stiffness of plates are studied for two main deformation modes namely shear and bending.

3.1.2.1 Post Buckling Stiffness of Plates under Shear

Post buckling of plates under shear is studied through a finite element parametric study. In the parametric study restrained plates with different aspect ratios and slenderness values are subjected to shear. A typical loading pattern and a finite element mesh are given in Fig. 3.3.

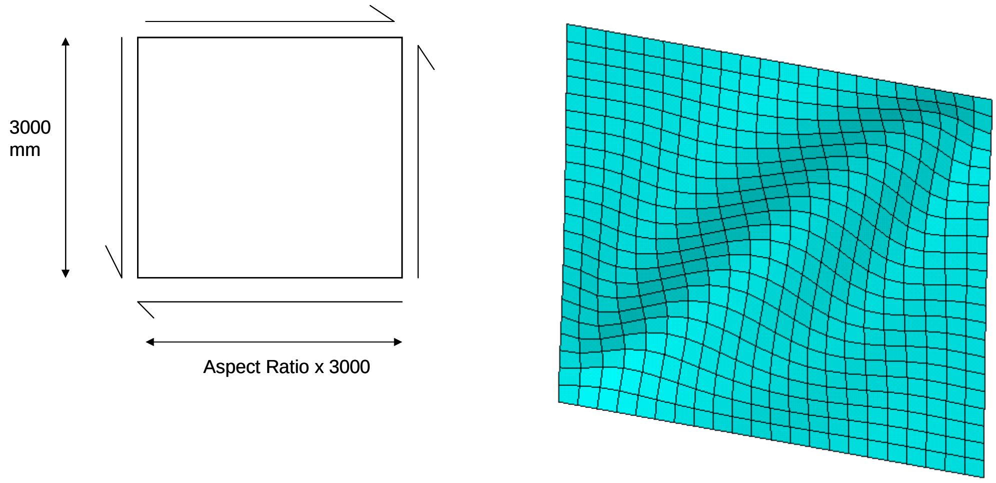


Figure 3.3 Restrained Thin Plate under Shear

Actual boundary conditions around the plate boundaries are quite complex. The unstiffened thin plate is surrounded by beams and columns that have finite stiffness. For accurate modeling of the physical system the flexibility of the boundary elements need to be taken into account. However, determining the flexibility of boundary members can be quite complex and can adversely affect the simplicity of the approach. Beams that are placed on top and bottom of the plate are subjected to opposing tension field forces. These opposing forces reduce the bending in the beams. For the columns, there is a minimum stiffness requirement given in the design specifications (AISC Seismic Provisions 2005) to ensure the development of tension field action. In this study the boundary conditions are considered as fully restrained and bending of beam and column members are neglected. In addition, rotations of plate boundaries are restrained during the analysis. Preliminary investigations revealed that allowing rotations at the boundaries does not significantly influence the results. In reality the stiff boundary members around plate boundaries provide restraint against rotation.

For the parametric study one of the dimensions is considered to be 3000 mm and the other dimension is changed according to the aspect ratio of the plate. For each aspect ratio six different plate thickness values are considered that results in plate slenderness values of 250, 375, 500, 667, 750, and 1000. It should be mentioned that in calculating the slenderness of plates under shear the shortest edge dimension needs to be considered. For all geometries two analyses are conducted where in one the geometrical nonlinearities are excluded and in the other one included. Fig. 3.4 shows a representative load-displacement behavior of a plate solved under geometrically linear and nonlinear assumptions. Finite element analysis results are presented in Table 3.1 where the results are given in the form of α_s factor which is the ratio of the post buckled stiffness of the plate to the pre buckled original stiffness. As can be seen from this table, α_s values change with the slenderness and the aspect ratio of the plate.

Table 3.1 α_s Factor Values for Plates with Different Aspect Ratios and Slenderness

Aspect Ratio (AR)	**Slenderness**					
	250	**375**	**500**	**667**	**750**	**1000**
0.75	0.70	0.57	0.50	0.45	0.43	0.40
1.0	0.79	0.67	0.60	0.57	0.52	0.48
1.5	0.79	0.68	0.62	0.60	0.57	0.54
2.0	0.79	0.70	0.70	0.61	0.60	0.59

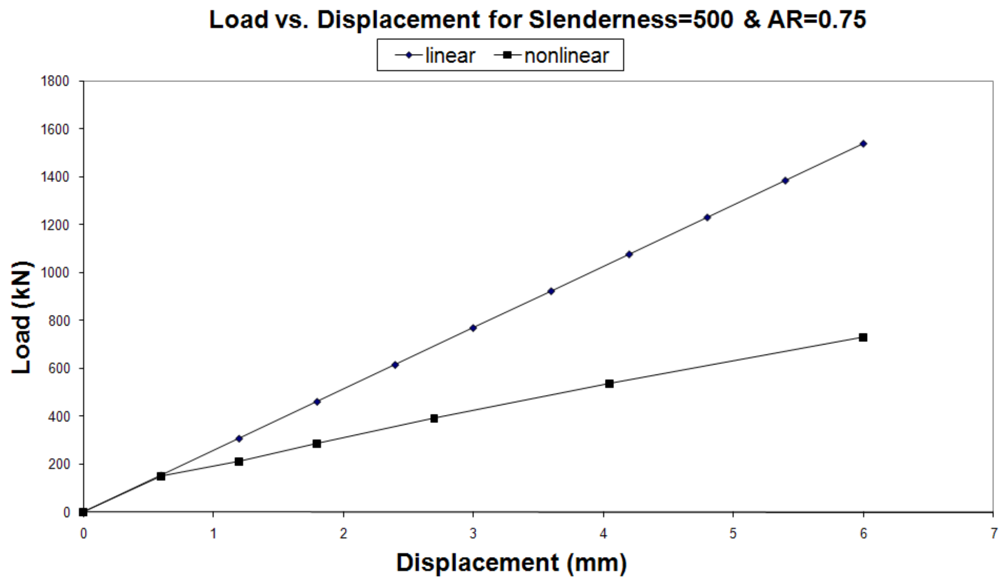


Figure 3.4 Load versus Deformation for Slenderness=500 & AR=0.75

3.1.2.2 Post Buckling Stiffness of Plates under Bending

Bending response is studied in a similar way as the study of post buckling stiffness of restrained plates under shear. Plates having different geometries are subjected to pure bending according to the loading pattern given in Fig. 3.5. Owing to the symmetry, only half of the plate is considered. Plate dimensions are changed to obtain aspect ratios of 0.75, 1.0, 1.5, and 2.0. For each aspect ratio eight different plate thickness values are considered that results in plate slenderness values of 375, 500, 667, 750, 1000, 1333, 1500, and 2000. Slenderness of the plates is determined by dividing the width to the plate thickness. In terms of vertical boundary conditions, truss elements are placed at the plate edges to represent the columns. Area of the truss elements are changed in the parametric study to observe its effect on the post buckling behavior of plates under bending. The inertia of the infill plate and that of the columns contribute to the total moment of inertia of the SPSW system. The ratio of the inertia contributions can be represented as a nondimensional factor, γ, where

$$\gamma = \frac{\text{Inertias of columns with respect to centerline}}{\text{Inertia of infill plate}} = \frac{0.5A_c(plw + d_c)^2 + 2I_c}{I_{pl}} \quad (3.10)$$

where; I_{pl}: moment of inertia of the infill plate, I_c: moment of inertia of the column section.

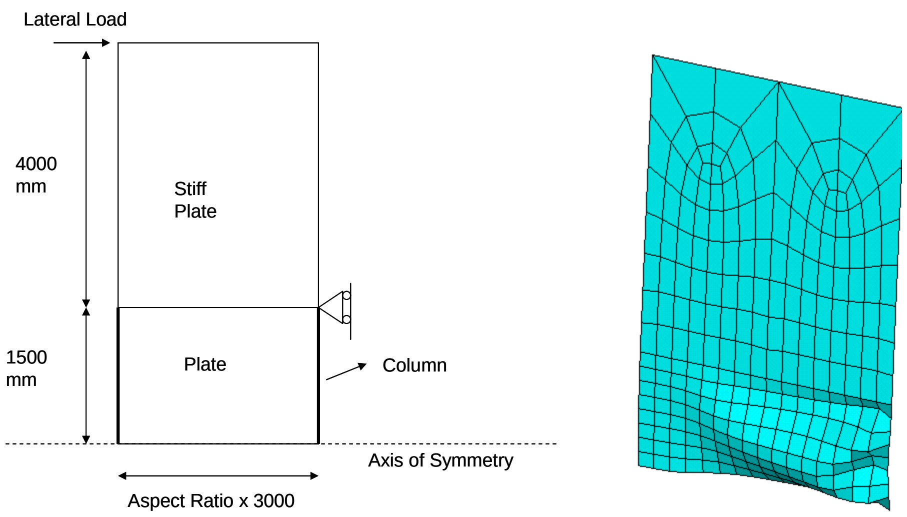


Figure 3.5 Restrained Thin Plate Under Bending

In the parametric study area of the columns are changed to obtain different values of γ between zero and 15. For each geometry and column area, a geometrically linear and a geometrically nonlinear analysis are conducted. The results are converted to an α_b factor which is the ratio of post buckled stiffness of the plate under bending to the original pre-buckled stiffness and are presented in Table 3.2. Analysis results showed that α_b factor is mostly influenced by the plate slenderness (plw/ptk) and the γ factor. The influence of aspect ratio is not much pronounced.

Table 3.2 α_b Factor Values for Plates with Different γ, Aspect Ratios and Slenderness

					γ	γ	γ	γ	γ	γ	γ
					0	**1**	**2.5**	**5**	**7.5**	**10**	**15**
Plate Height mm	**Aspect Ratio**	**Plate Width mm**	**Plate Thickness (t_w) mm**	**Slenderness**	α_b	α_b	α_b	α_b	α_b	α_b	α_b
3000	0.75	2250	6	375	0.761	0.811	0.835	0.853	0.863	0.871	0.882
3000	0.75	2250	4.5	500	0.682	0.747	0.781	0.781	0.819	0.828	0.818
3000	0.75	2250	3.375	666.6666667	0.572	0.659	0.702	0.725	0.736	0.764	0.768
3000	0.75	2250	3	750	0.536	0.632	0.672	0.710	0.726	0.738	0.749
3000	0.75	2250	2.25	1000	0.456	0.573	0.619	0.656	0.672	0.686	0.702
3000	0.75	2250	1.6875	1333.333333	0.400	0.526	0.581	0.619	0.640	0.653	0.667
3000	0.75	2250	1.5	1500	0.382	0.507	0.567	0.608	0.628	0.641	0.654
3000	0.75	2250	1.125	2000	0.345	0.482	0.538	0.574	0.596	0.618	0.628
3000	1	3000	8	375	0.686	0.751	0.785	0.812	0.828	0.840	0.858
3000	1	3000	6	500	0.610	0.692	0.735	0.767	0.787	0.800	0.820
3000	1	3000	4.5	666.6666667	0.511	0.616	0.671	0.716	0.740	0.756	0.779
3000	1	3000	4	750	0.477	0.589	0.647	0.690	0.715	0.733	0.758
3000	1	3000	3	1000	0.401	0.534	0.597	0.642	0.668	0.687	0.707
3000	1	3000	2.25	1333.333333	0.343	0.483	0.560	0.603	0.630	0.649	0.674

Table 3.2 (continued)

3000	1	3000	2	1500	0.326	0.477	0.540	0.592	0.619	0.637	0.663
3000	1	3000	1.5	2000	0.290	0.446	0.520	0.572	0.597	0.617	0.638
3000	1.5	4500	12	375	0.598	0.692	0.748	0.798	0.829	0.851	0.880
3000	1.5	4500	9	500	0.531	0.645	0.709	0.760	0.798	0.819	0.851
3000	1.5	4500	6.75	666.6666667	0.453	0.590	0.665	0.729	0.767	0.791	0.829
3000	1.5	4500	6	750	0.413	0.571	0.649	0.713	0.756	0.792	0.819
3000	1.5	4500	4.5	1000	0.340	0.508	0.597	0.674	0.719	0.750	0.789
3000	1.5	4500	3.375	1333.333333	0.285	0.470	0.565	0.643	0.691	0.724	0.767
3000	1.5	4500	3	1500	0.266	0.458	0.555	0.632	0.681	0.715	0.757
3000	1.5	4500	2.25	2000	0.229	0.430	0.534	0.613	0.662	0.696	0.738
3000	2	6000	16	375	0.554	0.683	0.764	0.832	0.868	0.888	0.902
3000	2	6000	12	500	0.480	0.633	0.729	0.800	0.841	0.864	0.883
3000	2	6000	9	666.6666667	0.450	0.594	0.718	0.800	0.843	0.846	0.867
3000	2	6000	8	750	0.390	0.577	0.698	0.784	0.829	0.841	0.862
3000	2	6000	6	1000	0.316	0.529	0.650	0.753	0.796	0.825	0.851
3000	2	6000	4.5	1333.333333	0.261	0.494	0.621	0.723	0.778	0.811	0.840
3000	2	6000	4	1500	0.242	0.481	0.611	0.716	0.771	0.805	0.835
3000	2	6000	3	2000	0.207	0.458	0.592	0.701	0.758	0.792	0.826

The reduction in the plate stiffness does not have a significant effect on the total moment of inertia for cases where γ is greater than 15. Based on this observation and a statistical analysis of the data the following equation is developed to predict the α_b values as a function of the geometric quantities.

$$\alpha_b = 4(\gamma)^{-0.3} (\frac{plw}{ptk})^{[0.06 \ln \gamma - 0.28]} \qquad \text{If } \gamma > 15 \text{ use } \gamma = 15 \qquad (3.11)$$

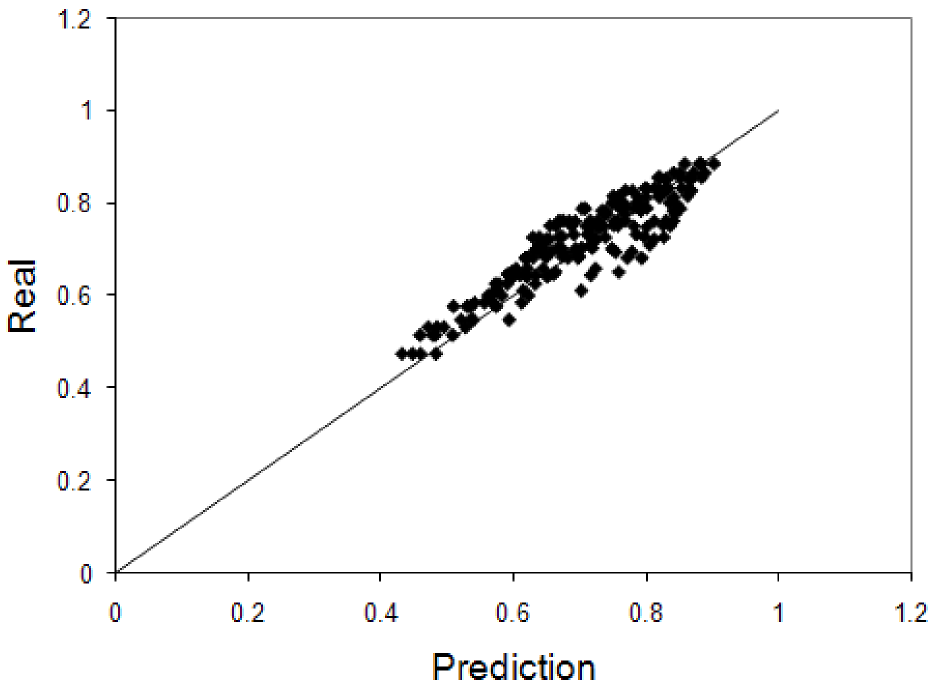


Figure 3.6 Shows the Comparison for the Predicted and Observed α_b Values

In the range of γ between zero and 15 the ratio of the predicted values from Eqn. 3.11 versus observed values has a mean of 1.026, a standard deviation of 0.06, a maximum ratio of 1.157 and a minimum ratio of 0.860. When bending and shear response are compared it is found that the change in stiffness is much more pronounced for the case of shear. Due to the presence of stiff boundary members, buckling of the plate under bending does not significantly influence the overall inertia.

3.1.3 Modified Expressions

Based on the study of individual plates under shear and bending it can be concluded that there is reduction in stiffness due to shear and bending actions. In reality infill plate is subjected to a combination of bending moment and shear. At this point an approximate procedure for deflection calculations is developed. This approximate hand method is based on the premise that buckled infill plates provide a reduced stiffness to the wall system when compared with the plates that are not buckled. Shear and bending response are treated separately in the calculation procedure. New expressions are based on the previously presented ones except that the geometrical factors for the wall system are modified. It is observed that the contribution of plate stiffness to the overall shear stiffness is going to be reduced and this can be taken into account by using a modified shear factor, β_m, calculated as follows:

$$\beta_m = \beta_1 + \frac{\beta_2}{\alpha_s} \tag{3.12}$$

In a similar fashion the contribution of the plate stiffness to the overall bending stiffness is going to be reduced and this can be taken into account by using a modified inertia, I_m, calculated as follows:

$$I_m = 2I_c + 0.5A_c(plw + d_c)^2 + \alpha_b I_{pl} \tag{3.13}$$

Deflections of the wall system can be calculated using Eqn. 3.7 and utilizing the modified expressions presented in Eqns. 3.12 and 3.13. It should be mentioned that modified inertia is used only in the bending deflection (Δ_b) computations not in the shear deflection (Δ_s) computations. Verification of these modified expressions with experimental results, geometrically nonlinear finite element analysis and strip method of analysis is presented in Chapter 4.

3.2 TRUSS MODEL

The drawback of the developed hand method is that it cannot be used directly to estimate the lateral stiffness of steel plate shear walls that are a part of a frame system. However, the findings presented in the development of the hand method can be used to develop a new computer method that is suitable for conventional structural analysis software. As a part of this study a simple computer method is developed to predict the lateral stiffness of SPSW systems. This method is based on the observation that the SPSW system is analogous to a truss given in Fig. 3.7. Vertical members of the truss are used to simulate the bending stiffness and diagonals are used to simulate the shear stiffness. All members are pin connected and horizontal members are provided for stability of the truss. A similar yet different equivalent truss approach was proposed earlier by Thornburn et al. (1983).

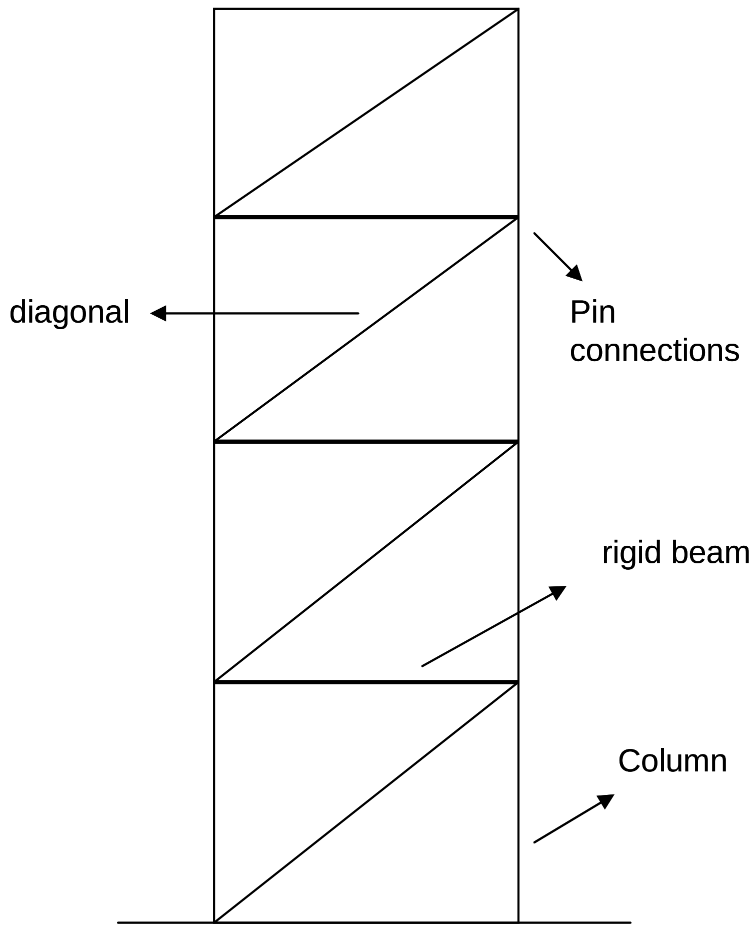


Figure 3.7 Truss Model

Truss model is very convenient in terms of modeling the wall system using traditional structural analysis software. Engineers need to input the truss elements based on the wall geometry and using centerline dimensions. Horizontal members are assumed to be axially rigid. Areas of vertical and diagonal members are determined using the expressions developed for the hand method. Vertical members should have an area (A_{ver}) that provides an inertia with respect to the centerline of the wall equivalent to the modified inertia, I_m. Based on this observation area of vertical members is calculated as follows:

$$A_{ver} = \frac{I_m}{0.5(plw + d_c)^2} \tag{3.14}$$

Diagonals are provided to represent the infill plate. Area of diagonals (A_d) should be selected such that the lateral stiffness provided by the diagonal is equivalent to the lateral stiffness of the infill plate. Area of diagonals can be simply calculated as follows:

$$A_d = \frac{L_d^3 (I^2 / \beta_m)}{2.6h(plw + d_c)^2} \tag{3.15}$$

where L_d: length of diagonal.

It should be mentioned that for SPSW systems with variable infill plate thickness vertical and diagonal member sizes for every story need to be computed and input into the analysis module. Verification of truss model with experimental results, geometrically nonlinear finite element analysis and strip method of analysis is presented in Chapter 4.

CHAPTER 4

VERIFICATION OF ALTERNATIVE TECHNIQUES

The accuracy of the developed methods is assessed by making comparisons with experimental findings and with existing methods of analyses. Stiffness values are compared against the ones obtained from experimented specimens. Due to the limited number of experiments conducted on multistory SPSW systems there is a need to further verify these new methods with the existing ones. In order to accomplish this task a parametric study consisting of 80 SPSW is conducted.

4.1. VERIFICATION WITH EXPERIMENTAL FINDINGS

Lateral stiffness of ten experimented specimens is calculated and the results are given in Table 4.1 in normalized form. It is evident from the results that both of the developed methods in general provide solutions with acceptable accuracy. When compared with the experimental results the new methods offer stiffer solutions except a few cases. This is natural because the stiffness reduction due to buckling of infill plates is tuned according to finite element analysis results. The average overestimation for the seven specimens is 11% for the hand method and 12% for the computer method (truss model). From these observations it can be concluded that the new methods can be alternatives to more traditional computer based methods. The excessive stiffness calculated for specimens M12 and M14 are considered to be due to the unreliable nature of measurements during experimentation as explained earlier.

Table 4.1 Lateral Stiffness of Experimented Specimens and Analysis Results

Case	Study	Specimen	# of story	Stiffness based on displacement at	Experimental Stiffness (kN/mm)	FEM/Experimental	Strip/Experimental	Hand Method/Experimental	Truss Model/Experimental
1	Lubell and others	SPSW4	4	FS	17	2.84	2.10	2.75	2.40
				TS	4.2	2.10	1.64	2.07	1.94
2	Driver and others	------	4	FS	425	1.00	0.88	0.89	0.86
				TS	96	1.06	0.82	0.98	0.95
3	Caccese and others	M22	3	TS	14.22	1.32	0.88	1.18	1.01
4		M14	3	TS	22.06	1.57	0.84	1.64	1.40
5		M12	3	TS	26.61	1.67	0.77	1.76	1.50
6	Park and others	SC2T	3	TS	83	1.04	0.74	0.70	0.73
7		SC4T	3	TS	111	1.11	0.74	0.94	0.98
8		SC6T	3	TS	120.5	1.31	0.79	1.12	1.17
9		WC4T	3	TS	92.5	1.00	0.68	0.90	0.94
10		WC6T	3	TS	98	1.18	0.73	1.06	1.10
			Average (w/o case1)			1.23	0.79	1.11	1.06
			Standard Deviation (w/o case1)			0.24	0.07	0.34	0.24

FS: first story, TS: top story

4.2. VERIFICATION WITH EXISTING METHODS OF ANALYSIS

A parametric study is conducted to investigate the validity of the developed methods for SPSW systems possessing different geometrical characteristics. In the verification wall systems with different heights are subjected to a tip load. A total of 80 SPSW systems are considered in the parametric study. Among these systems 60 of them have a constant infill plate thickness along the height and 20 of the have variable infill plate thickness. Infill plate height is considered to be 3000 mm for all systems. Two, four, six, eight, and ten storey wall systems with plate aspect ratios of 0.75, 1.0, 1.5, and 2.0 are analyzed. These aspect ratios are in agreement with the limitations presented by the AISC Seismic Provisions for Structural Steel Buildings (2005). Infill plate thickness of 3.0 mm, 4.5 mm, and 6.0 mm are considered to cover a wide range of slenderness ratios.

Capacity design principles are used to size the columns. In the design of plate walls a structural steel grade of S235 is considered for infill plates and S355 is used for beams and columns. First, depending on the plate width and plate thickness shear capacity of the infill plate is calculated by using the AISC Specification equation. Then, this shear capacity is amplified by 2.4 to account for the overstrength of the system. The amplified shear capacity is used to determine the maximum amount of lateral force on the system. This lateral load is distributed in an inverted triangular pattern over the height of the wall. Based on this lateral load, column forces are determined and these forces are used in the design of these elements.

Columns are selected among European HD rolled sections for each set of plate thickness and total number of stories. In addition, beam sections are selected among HEA rolled sections based on the aspect ratio. The sections used in the test cases are summarized in Table 4.2.

Table 4.2 Column and Beam Sections for the Parametric Study

COLUMNS					
Plate Thickness	2 storey	4 storey	6 storey	8 storey	10 storey
3.0 mm	HD 320x158	HD 400x287	HD 400x421	HD 400x509	HD 400x634
4.5 mm	HD 400x216	HD 400x421	HD 400x592	HD 400x818	HD 400x990
6.0 mm	HD 400x287	HD 400x551	HD 400x744	HD 400x990	HD 400x1086

BEAMS				
Aspect Ratio	0.75	1.0	1.5	2.0
	HEA 140	HEA 180	HEA 240	HEA 300

For the cases with variable plate thickness a bottom story infill plate thickness of 6 mm is considered and the plate thickness is reduced along the height of the wall. Two storey wall systems have the first story infill plate thickness of 6 mm and the second story infill plate thickness of 3 mm while four storey wall systems have the first and second story infill plate thickness of 6 mm and the third and forth story infill plate thickness of 3 mm. For six storey wall systems the first and second story infill plate thickness of 6 mm, the third and forth story infill plate thickness of 4.5 mm, the fifth and sixth story infill plate thickness of 3 mm are considered. In eight storey wall systems the first three stories have infill plate thickness of 6 mm, the next three stories have infill plate thickness of 4.5 mm and the last two stories have infill plate thickness of 3 mm. As a final, in ten storey wall systems the first four stories have infill plate thickness of 6 mm, the next three stories have infill plate thickness of 4.5 mm and the last three stories have infill plate thickness of 3 mm.

A finite element mesh is prepared for all 80 wall systems. A center imperfection of 3mm is modeled for all panels. A geometrically linear and a geometrically nonlinear analysis are performed to obtain the tip deflection. All wall systems are subjected to a top drift of 0.3 percent. This value of drift is conformable with the yield drift values (between 0.17% and 0.43%) obtained in the experiments by Park et al. (2007). Usually in an event of an earthquake structural systems can undergo larger drift values (more than 2 percent) due to the inelasticity that is present. However, during the design process a linear analysis is conducted up until the yield point. Later the drift values found using linear analysis are magnified by a displacement amplification factor. The recommended displacement amplification factor for SPSW systems in AISC Seismic Provisions (2005) is 6. Widely used design specifications such as International Building Code (2003) limits the allowable drift on structures. Usually maximum drift of 2 percent is used. When this value is divided by the recommended displacement amplification factor than the order of magnitude for the considered drift value (0.3 percent) is justified. A typical load deflection response for a wall system solved under geometrically linear and nonlinear assumptions is given in Fig. 4.1. Similar response curves are obtained for all test structures. The lateral stiffness for the geometrically nonlinear case is calculated from the secant stiffness at 0.3 percent top story drift. As mentioned before the results of the linear finite element analysis is compared with the results from geometrically linear theory.

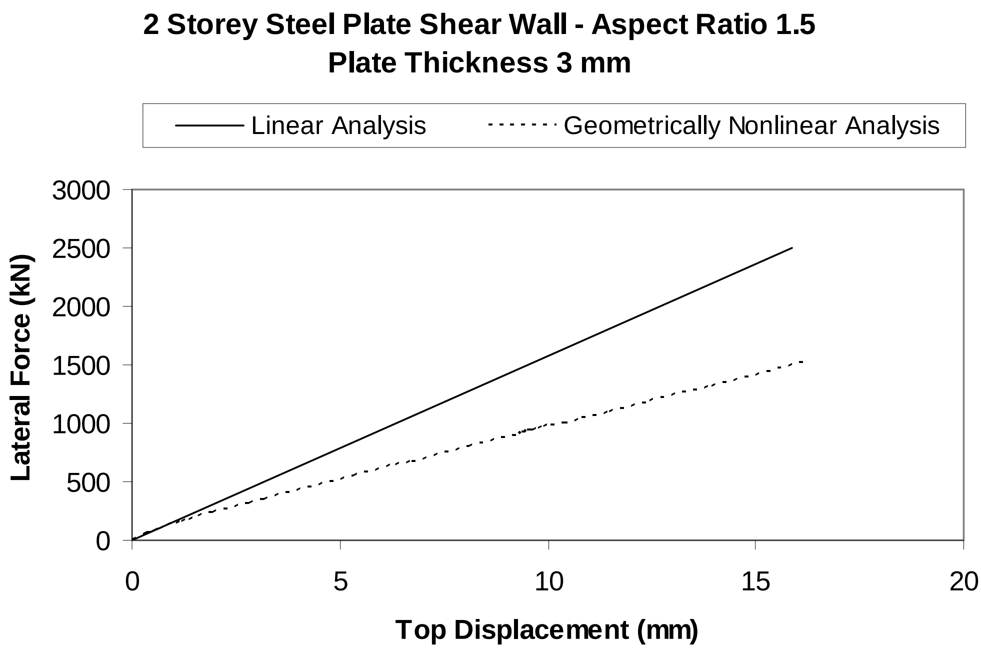


Figure 4.1 A Typical Load-Displacement Response of a Steel Plate Shear Wall

In addition to the finite element analysis strip method of analysis is conducted for the 80 structures. Strip angles for these structures vary between 35 and 54 degrees and a total of 12 strips are used for each panel.

Lateral stiffness of 80 test structures are computed using the two methods developed as a part of this study. Stiffness values obtained using these methods are compared against finite element analysis results and strip method of analysis results in Fig. 4.2. In this figure the stiffness values obtained from the developed methods are normalized by the stiffness value obtained by averaging the ones of strip method and finite element method of analysis. It is observed that most of the data points are below 1.2 and there are a few outliers. The outliers belong to cases where there is large difference between finite element and strip method of analysis results. When compared to the average of the results of two existing methods for the 80 structures analyzed hand method and truss method provide predictions that are on average 12% and 13% stiffer, respectively. In general the new methods provide flexible solutions when compared with the finite element analysis results. On the contrary these methods provide stiffer solutions when compared with the strip method of analysis results. Comparisons revealed that the solutions provided by the developed methods have acceptable accuracy. These new methods are much simpler to use and provide solutions that fall within the solutions provided by the existing methods of analysis.

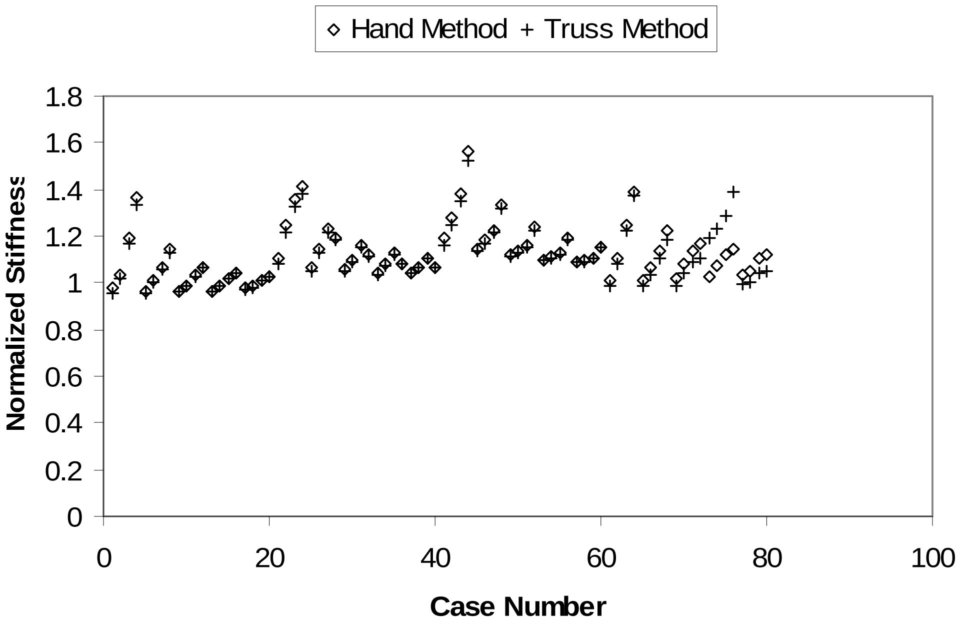


Figure 4.2 Normalized Stiffness Values of Two Methods

CHAPTER 5

SUMMARY AND CONCLUSIONS

Accuracy of finite element method of analysis and strip method of analysis in predicting lateral stiffness of SPSW systems are assessed by making comparisons with the experimental findings. Two new methods are developed as a part of this study to calculate lateral stiffness of SPSW systems. The first one is a hand method based on modified deep beam theory. The second one is a computer method based on truss analogy. Lateral stiffness values computed using the new methods are compared against the ones from experimental results and analysis results obtained using existing methods.

The following can be concluded from this study:

- Finite element method offers stiffer solutions for lateral stiffness when compared with experimental results and can be considered as an upper bound solution.

- Strip method of analysis offers flexible solutions for lateral stiffness when compared with experimental results and can be considered as a lower bound solution.

- Expressions developed using the deep beam theory can be used to study the response of SPSW systems solved under geometrically linear assumption. However, these expressions need to be modified for cases where geometrical nonlinearities are included.

- Study of individual restrained plates under shear revealed that the post buckling stiffness of the plate is influenced by the aspect ratio and plate slenderness. Values presented in Table 3.1 can be used to estimate the ratio of the post buckled stiffness to the pre-buckled one.

- Study of individual restrained plates under bending revealed that the post buckling stiffness of the plate is mostly influenced by the plate slenderness and stiffness of boundary elements. Based on the parametric studies Eqn. 3.11 is developed to estimate the ratio of the post buckled stiffness to the pre-buckled one.

- Stiffness values computed using the approximate hand method and the computer method developed in this study compare well with the experimental findings.

- For the class of test structures considered stiffness values computed using the new methods are in agreement with the ones computed using the existing methods of analysis.

- The new methods are simple to use. The hand method can be implemented using a pocket calculator and the truss model requires minimal effort in terms of structural modeling. These methods can be alternatives to the existing ones.

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