Chapter 2. Literature Review

2.1 General Overview

Research on steel plate shear walls has been going on for the last forty years. Around 1960s Japan first introduced steel plates to use in buildings as shear walls. At that time owing to the limitation of research in this new area, highly stiffened steel plates were used since under design loads buckling was considered as failure of the plates. With time several analytical and experimental studies have been carried out to investigate the behavior of Steel Plate Shear Wall (SPSW) systems. With development in research it was concluded that buckling is not the ultimate failure of plates. The tension field developed in plates after bucking is also capable to resist shear. This usable post-buckling strength widened the applicability of SPSWs and soon SPSW became a more popular structural system in construction particularly in USA and Japan. Particularly places where dissipation of lateral forces is a concern, SPSW made its way. For high seismic region, where higher ductility is the prime requirement for the lateral load resisting system use of SPSW in structures can be at a very effective and economic solution.

Proper technique for utilizing the post buckling strength and a methodical approach for analysis of SPSW was first suggested by Thorburn et al. (1983). When buckling starts in steel plate the in plane tension field becomes inclined tension field i.e. from linear behavior the plate behavior changes to geometrically non-linear behavior. The concept and formulation introduced by Thorburn has later been acknowledged by Canadian Steel Design Standards (CAN/CSA-S16-01) and was accepted as a standard method for analysis. This concept of diagonal tension field being formed in buckled plates under shear was first introduced by Wagner (1931). Wagner's theory indicated that the capacity of thin plate being supported by relatively stiff boundary members depend largely on the parallel tension fields being developed. Other researchers like Kuhn et al. (1952) tried to establish a proper relation on how the flexibility of boundary members limited the complete development of tension fields in plates. Their work was mainly on plate girders. Research indicated similarity in behavior of plate girders and SPSW systems. Initial proposal for design technique of SPSW was based on the assumption that columns of SPSW system behave as flanges of a plate girder and the infill plate as web of the girder. Also, beams in SPSW system act as stiffener plates connecting the two flanges and attached to web of girder. The modern design technique is an evolution from this theory. However, with time even more complicated models of Finite Element Method (FEM) came up (like Elgaaly et al., 1993, Driver et.al, 1997) which are normally more accurate and reliable but time expensive technique. For cyclic load test or test of structure with several ground motions where performance based design philosophy is involved, none of the models described so far (Thorburn et al. (1983), Elgaaly (1998), Mohammad et al. (2003), Bhowmick et al. (2010), Kharrazi et al. (2004), Topkaya and Atasoy (2009)) in this area of research is very effective.

Berman et al. (2005) indicated that for low rise structures and for the upper stories of high rise structures the infill plate thickness required for the seismic design loads is less than 1mm. Practically, achieving this thickness with hot rolled steel becomes impossible. Also, handling and welding demands a higher thickness. If the thickness of infill plate is increased, then instead of plates yielding first, the boundary members start to yield when design loads are exceeded. This contradicts the capacity design philosophy. Vian et al., (2005) proposed a solution to this problem by introducing areas of weakness within the plates like quarter circle corner cutoffs and openings. Using cold rolled steel is another solution to the thickness problem, since cold rolled steel can be made thinner. The main problem faced with cold rolled steel was welding of such thin plates. Neilson (2010) documented the welding requirements while using cold-rolled SPSW of thickness less than 1mm. There have been some experimental tests on light gauge thin-walled SPSW systems (Kharrazi (2005), Neilson (2010)) but hardly any analytical study to determine the acceptability of light gauge infill plate in SPSW system design is available. If a convenient, fast, reliable and easy to use modeling technique is available to determine the acceptability of SPSW systems on case specific basis then its industrial acceptability is expected to widen up. A brief review relating to research works done in relation to modeling of SPSW systems analytically and use of light gauge thin plate shear wall, around the world has been presented in the following.

2.2 **Establishing property of SPSW systems**

To develop any new modeling technique or comment on the acceptability of available design techniques for steel plate shear walls, it is necessary that a detailed study on all the parameters responsible for the complete behavior of SPSW systems is carried out. Analyzing individual parametric properties of SPSW systems as reported through earlier studies by various authors on basis of experimental or numerical studies creates the background for the objective of this research.

Thorburn et al. (1983) works can be regarded as one of the first to give a comprehensive estimate on the behavior of unstiffened steel plate shear wall systems. Through this research it was indicated clearly that buckling does not indicate the ultimate failure of infill plates in SPSW system. Wagner's (1931) theory in development of tension field was utilized to explain the tension strip development in panels. Strength of infill plates prior buckling was considered negligible in comparison to the strength that the tension strips can provide. In other words, rather than considering the buckling as failure of plates, shear strength prior to buckling of plate was neglected. The tension fields that dominate the post-buckling strength were considered to be the only load transferring path and thus came up a strip model that can estimate the SPSW behavior. A parametric study with the basic parameters like plate thickness, aspect ratio, column flexibility, etc. was conducted. It was concluded that the parameters are interdependent on one another and their interactions are complex. This exposed bigger challenges for upcoming researchers through next few decades. Following the works of Thorburn et al. (1983), several other attempts were made to establish a set of independent parameters that can predict the behavior of SPSW systems. Tromposch and Kulak (1987) made an important conclusion based on their experimental work that eccentricity involved in fixing fish plate had no significant effect on performance of SPSW specimen. They also attempted to investigate the effect of beam column connection and concluded that the experimental model was somewhere in between fixed and pinned connection. It was also concluded that increasing the connections to rigid beam-column has a significant increase in energy absorption capacity of the system.

With the objective of evaluating the overall in-plane performance of shear wall under extreme cyclic loading Driver et al. (1997, 1998) tested a half-scale four-storey unstiffened steel plate shear wall (Figure 2.2). All connections in the specimen were made rigid and the infill plate was welded to the boundary framing members using a fish plate. For pre-stressing the members constant gravity load was applied on top of columns. Cyclic load of constant magnitude was applied at every floor level. Also, the cyclic test was carried out according to the requirements of ATC-24 (Applied Technology Council 1992). First storey displacement was used as control point for the loading. Initially the first yield displacement load was applied and then in consecutive cycles the yield displacement was increased. Up to a maximum of five times the first yield displacement the structure could resist increasing loads. Gradual and stable strength reduction was observed after ultimate strength (3080 KN) was achieved. The maximum deflection attained by the lowest storey, beyond which the structure failed, was nine times the yield deflection. A total of 30 cycles of load were applied out of which almost 20 were in the inelastic range. Hysteretic curves are also observed to be very stable throughout the experiment (Figure 2.1). It was concluded from the experiment that rigid beam column connections are capable of dissipating more energy than that of shear beam to column connections (Thromposch and Kulak 1987), since severe shear pinching of hysteretic loops were significantly less with rigid connections. This research also inferred that for specimens tested the angle of inclination of tension field strips ranged between 42^0 and 50^0. In that short range of inclination angles little effect on the final push-over curves so, a parametric study to observe the behavior further was suggested.

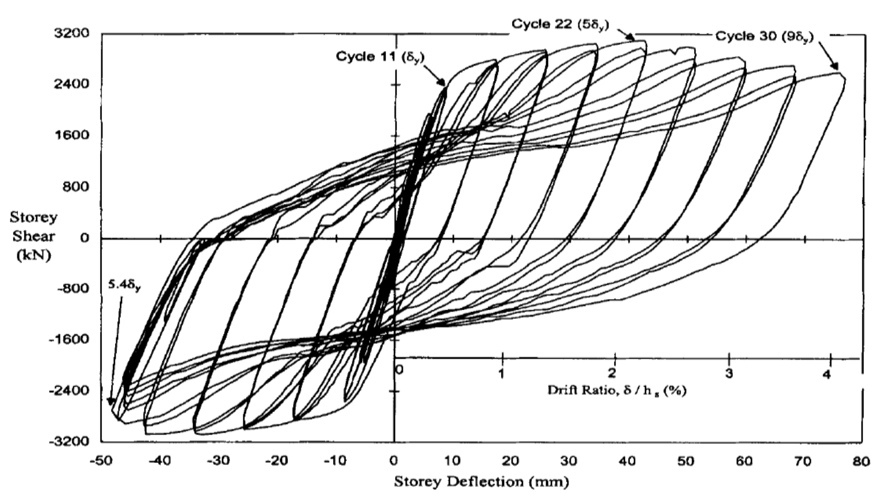


Figure 2.1: Storey shear vs storey deflection of panel-1 (Driver et al., 1998)

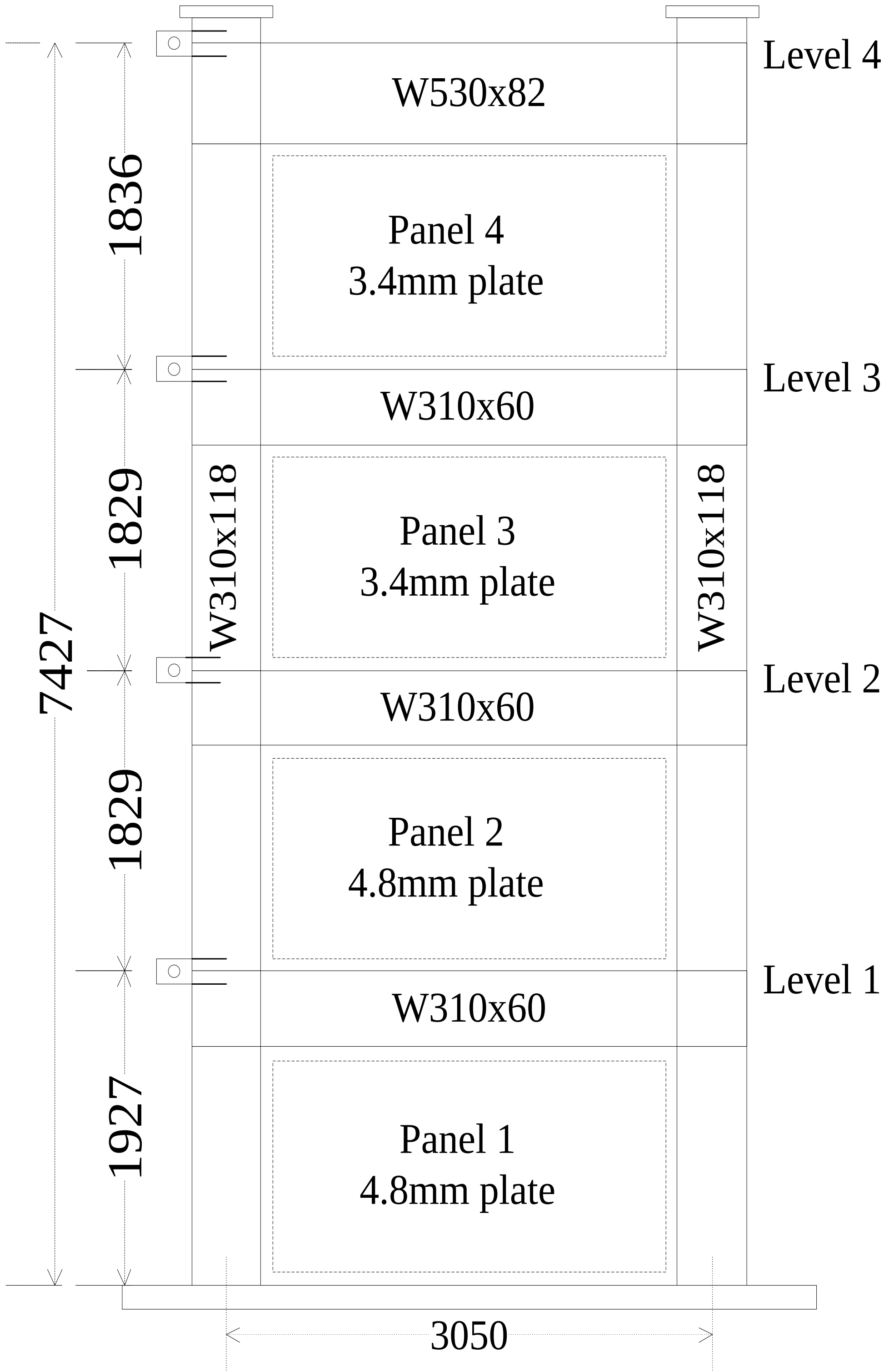
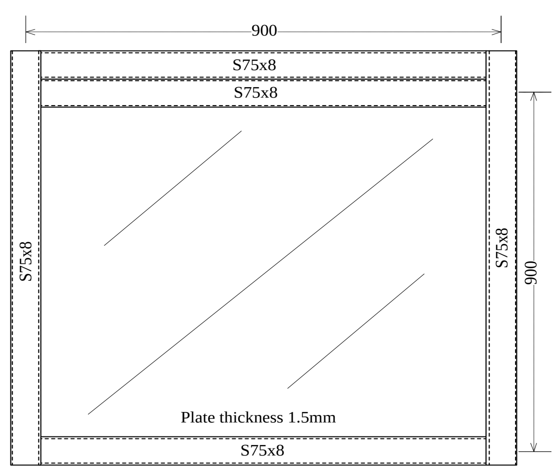
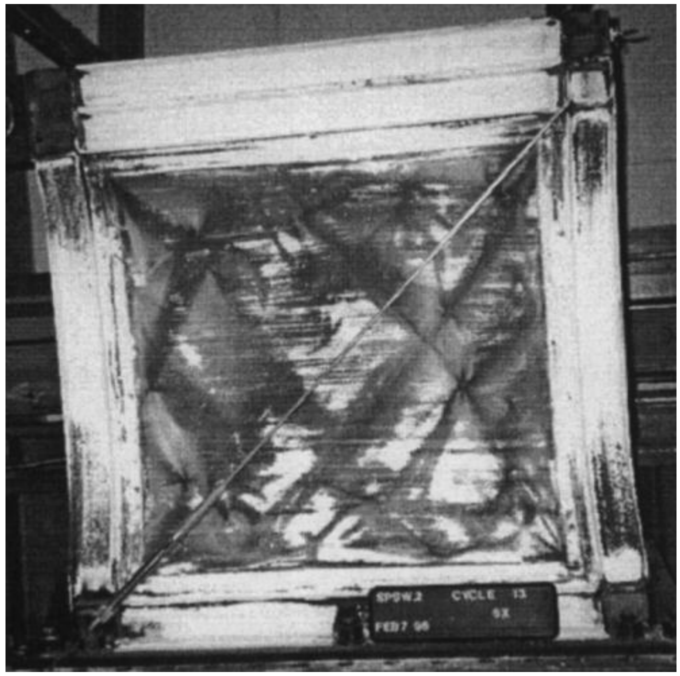


Figure 2.2: Four Storey specimen tested by Driver et al. (1998)

Lubell et al. (2000) tested three quarter scale unstiffened SPSW specimens. Two single storey (SPSW-1 and SPSW-2) and one four storey (SPSW-4) SPSW system were tested under quasi-static cyclic loading. An aspect ratio of one was maintained all through the design. The center line spacing in horizontal and vertical directions was 900mm. S75x8 section for both beams and columns and a plate thickness of 1.5mm was chosen. All materials were hot rolled. Specimen one (SPSW-1) represented the first storey of the four storey structure. Specimen two (SPSW-2) was basically specimen one with just one more top beam of same S75x8 section above the existing frame (Figure 2.3). The purpose of an additional beam was to allow better tension field development in the specimen. All connections were rigid weld connection. In the four storey specimen (SPSW4) the top beam was S200x34. An initial out of plane deformation i.e. initial imperfection up to 26mm has been reported for the first specimen (SPSW1). Quasi-static loading cycles following the guidelines given by Applied Technology Council (ATC-1992) were applied to all three specimens. The single storeys had their load control points at the top of the storey and for the four storey, all the storey were loaded together with the same load as in case of single storey. A gravity load, created by additional steel masses attached at desired portions, of 13.5 KN was maintained at all floor levels.



(a) Schematic diagram of SPSW-2



(b) Deformation and yield pattern for specimen SPSW-2

Figure 2.3: Specimen SPSW-2 tested by Lubell et al. (2000)

Stable S-shaped hysteresis was observed from the test (Figure 2.4). From envelope of hysteresis curves it was concluded that all the structures had sufficient initial stiffness and good displacement ductility. Comparing SPSW2 with that of SPSW1, a significant increase in stiffness and capacity has been reported in SPSW2. This was expectedly because of the stiffer beams and less imperfection involved in specimen SPSW2. For SPSW1 and SPSW2 the plate yielded significantly before the boundary members started to yield but for SPSW4 it was the columns where yielding started before any significantly noticeable yielding of plates. The yield sequence of SPSW4 is not desirable in practical design. The possible justification for it was the influence of overturning moments and small aspect ratio of panels. This yielding of columns caused instability and restricted the experiment to a ductility ratio of about one and half times the yield displacement. The boundary members considered through the set of experiments were significantly light and places of incomplete tension field development have been reported. Specimen SPSW2 inward column deformation resulted in the formation of plastic hinges at top and bottom of column. The experimental outcomes have been discussed in more details by other research groups like Montgomery and Medhekar (2001) and reported that these experiments had inadequate column stiffness. Thus, the importance of strength of boundary members for acceptable behavior of SPSW systems has been indicated through the experimental work of Lubell et al. (2000).

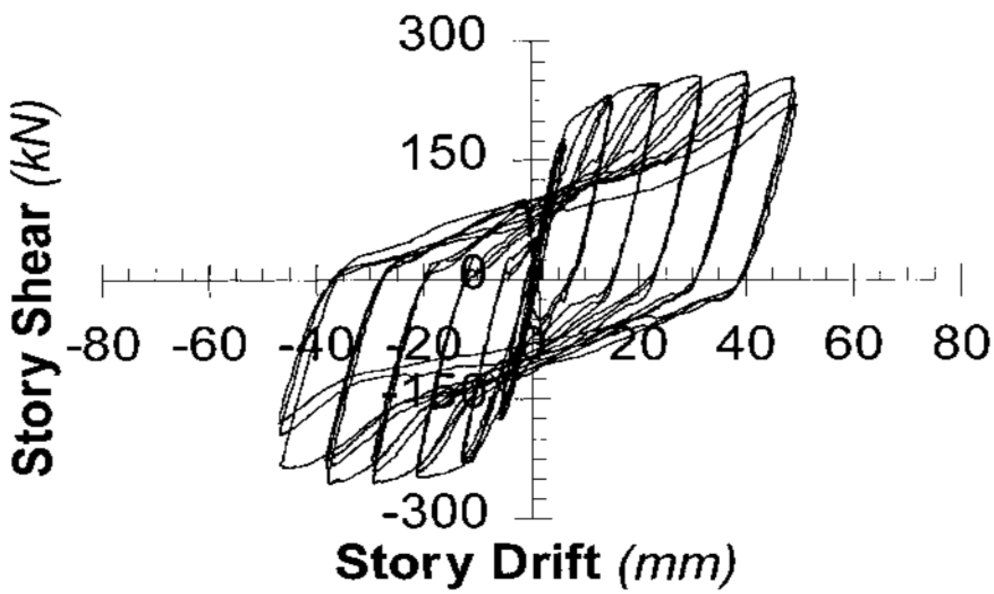


Figure 2.4: Hysteretic curves generated with specimen SPSW2 by Lubell et al. (2000)

Lubell et al. (2000) also concluded that with increase of panel height while all other parameters remain constant, the influence of flexural behavior increases and over turning moment dominated mainly the upper stories in tall buildings. Through this observation it was concluded that in case of SPSW systems the full structure needs to be analyzed as a whole and not as single panel, since the single panel behavior differed significantly from the behavior of the full multi-panel structure. Modification to the then available design guidelines were indicated through this work by concluding that the available design guidelines may show good co-relation in post yielding character of the structure but may significantly over estimate the elastic stiffness under certain conditions. Also, inefficiency in available design provisions for multi-storey structures was highlighted. Particularly the possible large over turning moment created undesirable yielding sequence in SPSW components, as observed with their experimental study.

Mohammad et al. (2003) carried out experimental and numerical study on unstiffened SPSW. They summarized a set of ten independent parameters which could be used to characterize the behavior of SPSW systems. The specimen test by Mohammad et al. (2003) was very similar to the one tested by Driver at al. (1998a), only with bottom panel removed. Thus, full scale single bay three storey sample was used for experimental test (Figure 2.5). The material properties were assumed to be the same as in case of Driver at al. (1998a), since there was no additional scope to test the material properties of a fabricated sample. Large Initial imperfection (maximum of 39mm) has been reported through this study. All the stories were pushed laterally by hydraulic jacks with same force and second storey displacement was the control point for the setup. The cyclic loading sequence followed ATC-24 guidelines. Rupture in the first level beam at top flange and web of the beam-to-column connection was observed even before the ultimate strength was reached. To achieve the ultimate capacity of plates and to observe the behavior of boundary members under extreme loading, the rapture was fixed and the experiment continued. The hysteric curves indicated a stable behavior. Ultimate strength of the specimen was observed when the maximum second storey displacement was seven times the yield displacement. Beyond this displacement limit, the lower-storey infill plate started to show tears and thus gradual strength degradation was noticed. Like most other experimental research work, this one also showed excellent ductility and stable hysteric loops (Figure 2.6). High initial stiffness, high degree of redundancy and also reported through this study.

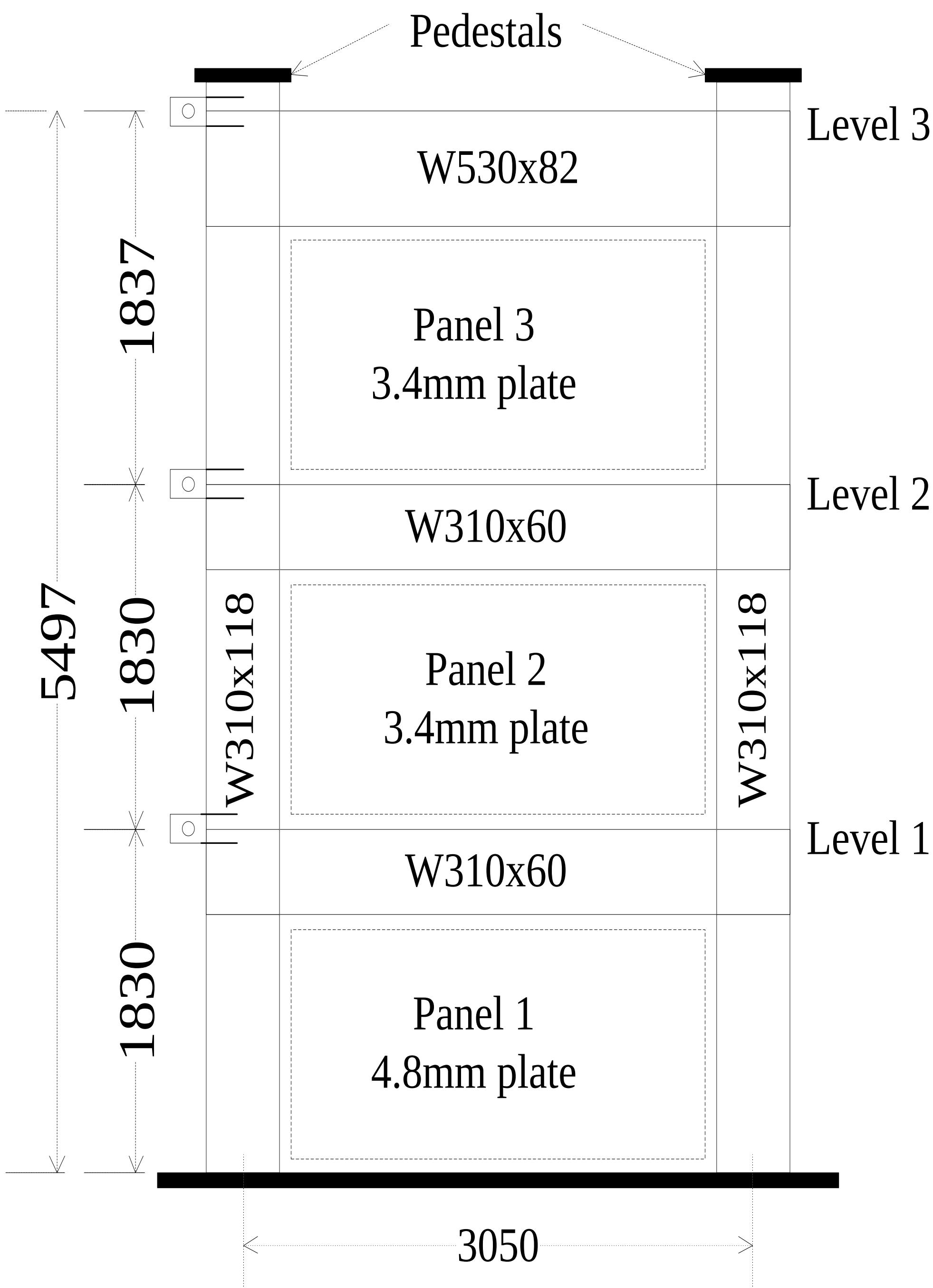


Figure 2.5: Three Storey specimen tested by Mohammad et al. (2003)

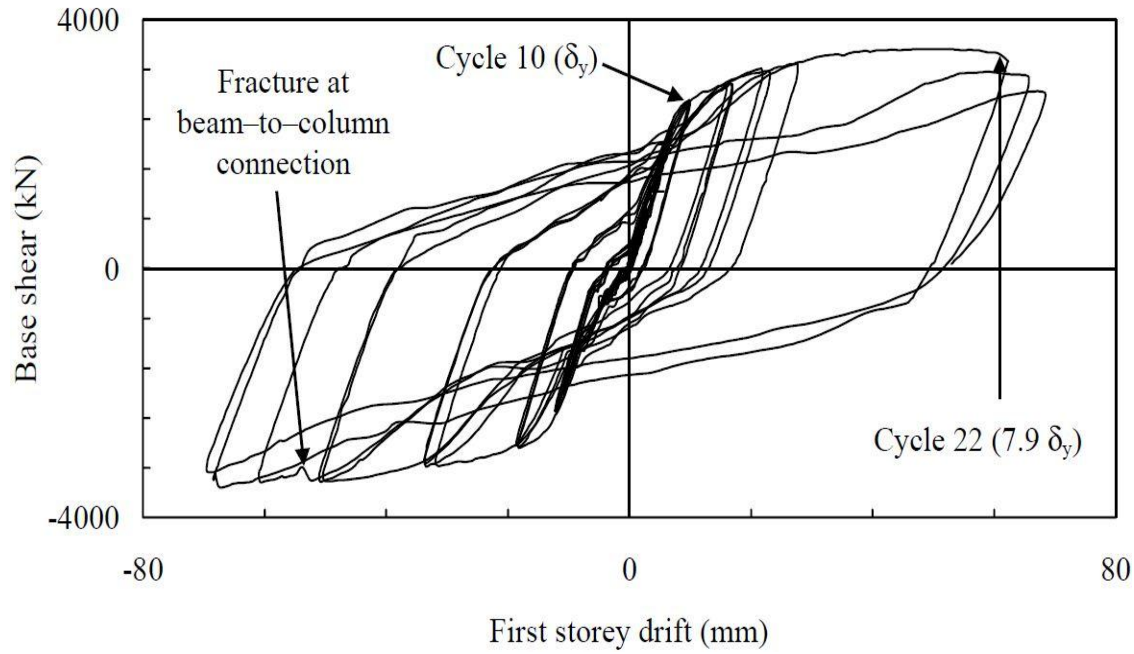


Figure 2.6: Base Shear versus first storey drift (Mohammad et al. 2003)

A numerical model based on Finite Element Method (FEM) was used by Mohammad to validate the experimental work and carry out parametric study. A set of ten non-dimensional parameters were shorted out for parametric study (Table 2.1). Keeping one parameter variable and all others constant several numerical analyses was carried out using a single storey FE model. Effect of parameters like column flexibility, aspect ratio, ratio of axial stiffness of infill plate to that of column, etc. on the overall performance of the SPSW system was indicated. Things like increase of column flexibility will affect the bending deformation of the columns and will introduce non-uniform tension field, resulting in reduced stiffness and capacity was one of the conclusions derived from the parametric study. Imperfection was also one such parameter that affects the capacity and stiffness of SPSW systems, but with imperfection less than $0.01\sqrt{(lh)}$, the effect is negligible. Gravity load and over turning moment is also observed to have a negative effect in the elastic stiffness, normalized capacity and ductility of SPSW systems.

With this parametric study the gross behavior of SPSW was summarized into the influential variables.

Table 2.1: Dimensionless parameters responsible for SPSW behavior

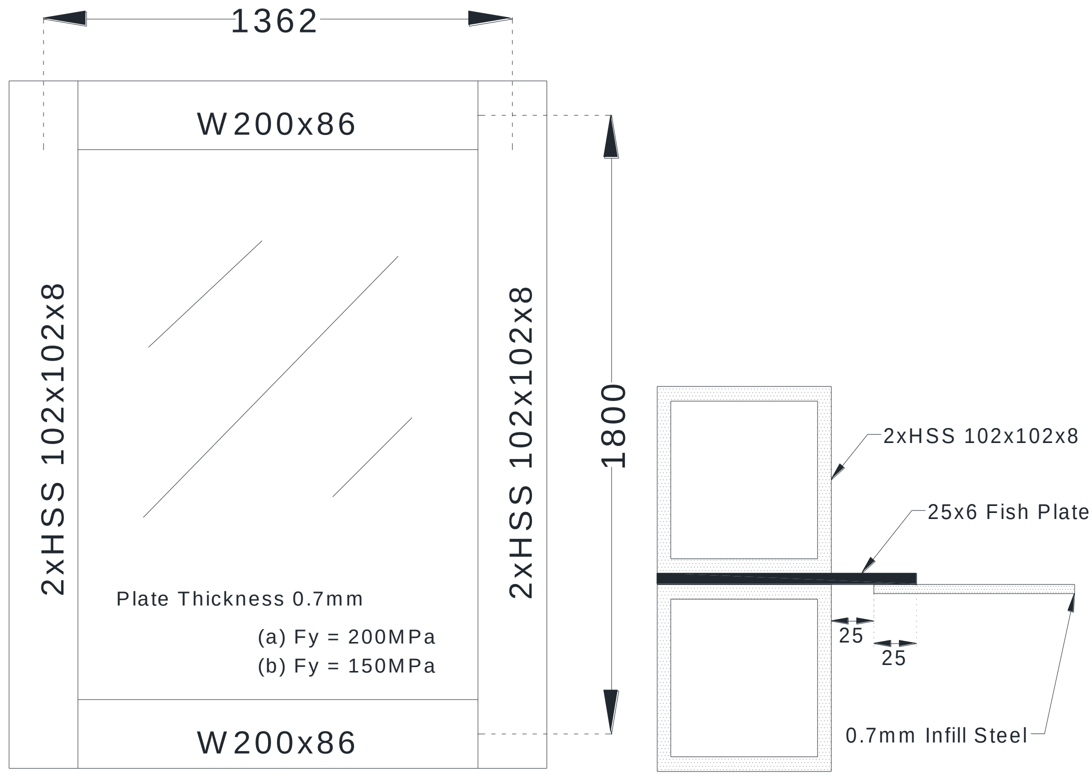
No.	Parameter	Details
β_1'	L_1/H	Aspect ratio
β_2'	$\frac{b*L_1}{2A_c}$	Ratio of axial stiffness of plate to that of columns
β_3'	$0.7*\sqrt[4]{\frac{H^4*b}{2L_1*I_c}}$	Column flexibility parameter
β_4'	W/W_y	Ratio of gravity load (W) to axial yield load (W_y) or normalized gravity load
β_5'	δ/H	Drift index, δ being the top displacement
β_6'	V/V_y	Ratio of shear load (V) to the shear yield capacity (V_y) or normalized base shear
β_7'	$\frac{\sigma_{yc}}{E}=\varepsilon_{yc}$	Column yield strain, σ_{yc}, ε_{yc} being stress and strain of column material
β_8'	$\frac{\sigma_y}{E}=\varepsilon_y$	Plate yield strain, σ_y, ε_y being stress and strain of plate material
β_9'	$\frac{\Delta_{imp}}{\sqrt{L_1*H}}$	Imperfection ratio, Δ_{imp} being the maximum pre-existing imperfection in plate
β_{10}'	$\frac{(A_c)^2}{I_c}$	Local buckling index

To establish any simplified model or determine the workability of SPSW system all properties of SPSW systems indicated through the selected reports presented above has to be kept in mind. Though there are several other researches that have been reported relating to the SPSW systems, only the ones related to this thesis were shorted out.

2.3 Works on Light-gauge SPSW

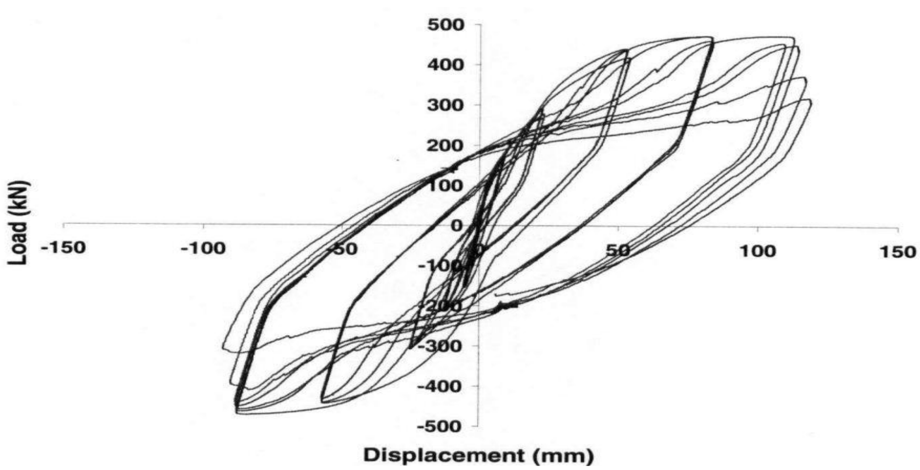
All experimental and numerical works mentioned so far used to obtain behavior of SPSW systems, was based on hot rolled steel. There is very little research that has been carried out with cold rolled steel as infill panel. Kharrazi (2005) is one of those who performed experimental works with light-gauge SPSW systems. He conducted quasi-static and dynamic test on two light gauge thin walled single storey SPSW specimens (DSPW-1 and DSPW-2) and a moment resisting frame (SF-1). The moment resisting frame had identical boundary frame as in case of DSPW-1 and DSPW-2. To avoid local effect on column, HSS sections were used to design columns but the beams were chosen as W-shapes (Figure 2.7(a)). A 22-gauge cold rolled steel sheet was used for infill plates. The only difference in DSPW-1 and DSPW-2 was in the material property of the plates. DSPW-1 had tensile yield strength of 200MPa whereas DSPW-2 has tensile yield strength of 150MPa. The two HSS 102x102x8 sections used in columns were connected to the light gauge infill plate by an intermediate fishplate (Figure 2.7(b)).

Three cycle cyclic loading was applied with increasing storey drift, unless the load carrying capacity deteriorated significantly. At around 4% drift plastic hinges were observed at top and bottom of columns. At 5.8% drift for specimen DSPW-1, fracture along the weld line of fish late and infill plate was observed. Bram column connections showed to have fracture at around 7.5% drift. The experiment ended with complete separation of fish plate to infill plate along with complete rupture of beam-column connection. Up to top displacement of twelve times the yield displacement, the specimen showed ductile behavior but beyond this limit the rupture has been reported to be brittle. DSPW-2 had very close behavior to that of DSPW-1 but the weld tearing at fish plate to infill plate was earlier in DSPW-2. At nearly 4% drift, the crack stretched around 400mm and more crack were significantly noticeable. Beyond 6% drift the strength degradation was even more significant. The cyclic load – displacement curves generated from the three samples are shown in Figure 2.8. Energy dissipation in the inelastic region and ductile behavior is well observed. Through conclusion of the research, a comparison amongst the three samples has been done in regards to energy absorption capacity in each cycle of load (Figure 2.9). Dynamic shake table test was performed on another set of sample specimen DSPW-3 (identical to DSPW-1) and SF-2 (identical to SF-1). The shake table test could hardly pass the elastic range owing to the huge capacity of the specimen. The overall behavior reported through this study was not observed to be very different from the one expected in hot-rolled steel infill panel. Berman and Bruneau (2005) also performed similar experiments with light gauge SPSW systems. Their results also indicated a very close behavior as one would expect in use of hot-rolled steel infill plates. Notably, both the experiments with light gauge steel infill panel were single storey experiment. No attempt was made to test the performance in case of multi-storey structures. Also, from practical point of view one major difficulty in use of light-gauge infill plate is welding such a thin plate to the boundary members.

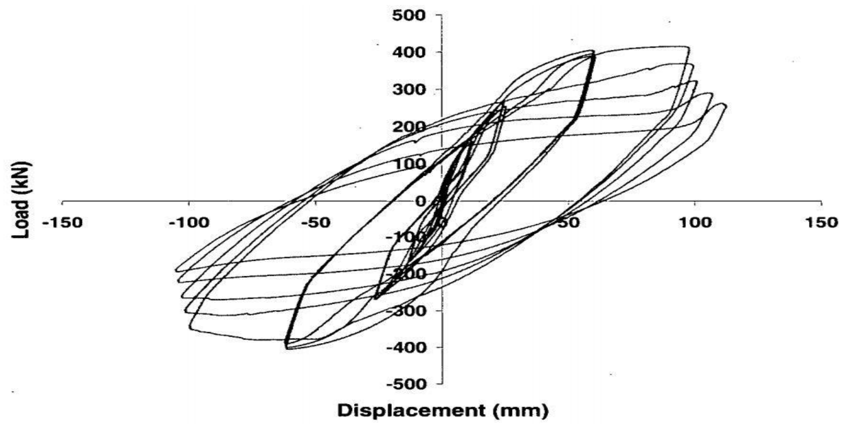


(a) (b)

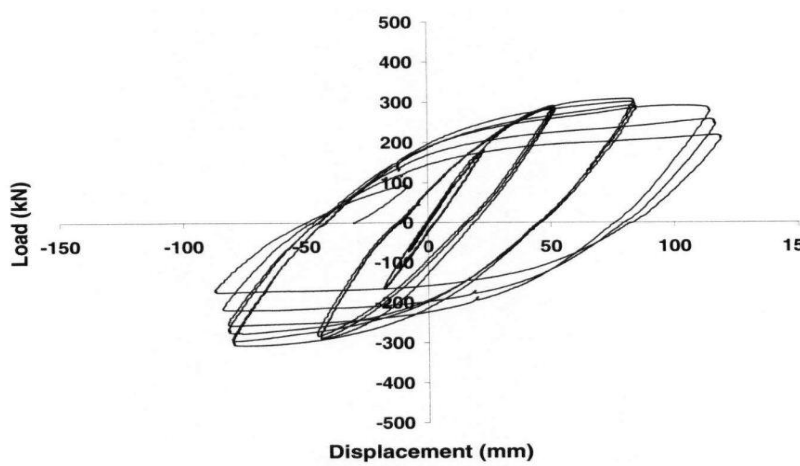
Figure 2.7: (a) Sketch of single storey specimen (b) Sectional view of column plate arrangement (Kharrazi 2005)



(a) DSPW-1



(b) DSPW-2



(c) SF-1

Figure 2.8: Load-displacement cycles for samples tested by Kharrazi, 2005

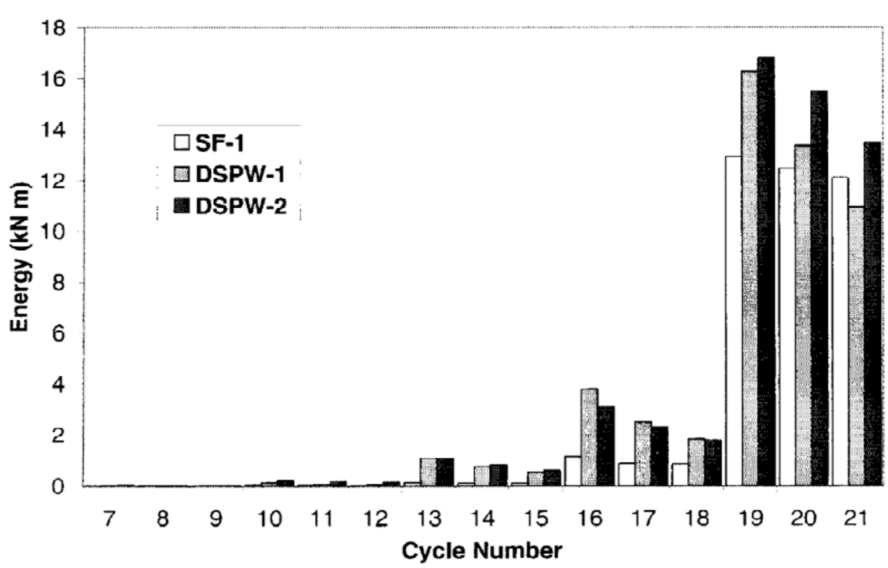


Figure 2.9: Hysteretic energy dissipation of DSPW-1, DSPW-2, SF-1 (Kharrazi, 2005)

Neilson (2010) studied the connection of light gauge steel plate shear wall with its boundary members. The main objective of the work was to develop welding procedure that will be simple to fabricate and at the same time can achieve good cyclic performance. Parameters such as joint geometry, material properties, welding process, electrode and shielding gas were selected for study. Arc welding was selected in making the experimental specimen. An ER70S-6 electrode was selected due to its strength, high toughness and deoxidizer content. Two shielding gases were used, namely, 75Ar – 25CO_2 and pure CO_2. Pure CO_2 was selected as it produced welds with the best arcing, wetting, and profile characteristics. Four possible configurations for the infill panel-to-boundary element connection and two possible configurations for the infill panel splice test have been reported. Configurations differed depending on whether one or two welds were used in the lap joint, and whether a chill strip was placed behind the thin sheet steel during welding to reduce the probability of burn-through and magnitude of distortion. All the configurations were subjected to a set of three quasi-static monotonic load and three cyclic loadings. Once the best configuration was shorted out, it was implemented on a single-storey large scale SPSW specimen (Figure 2.10). Loading of specimen was done based on ATC-24 (ATC, 1992) standards and a peak load of 630KN was achieved in 16 cycles. At the 17^{th} cycle storey drift of 3.5% and a fracture at the model base was reported. The infill panel to fish plate welding has been reported to be stable throughout the test out than some out of plane displacements. No detectable loss of integrity or strength degradation for loss of connection has been reported through the SPSW test. Finally, care on alignment of the weld has been indicated through the study. Thus, the limitation on use of light-gauge infill panel in SPSW system was successfully resolved.

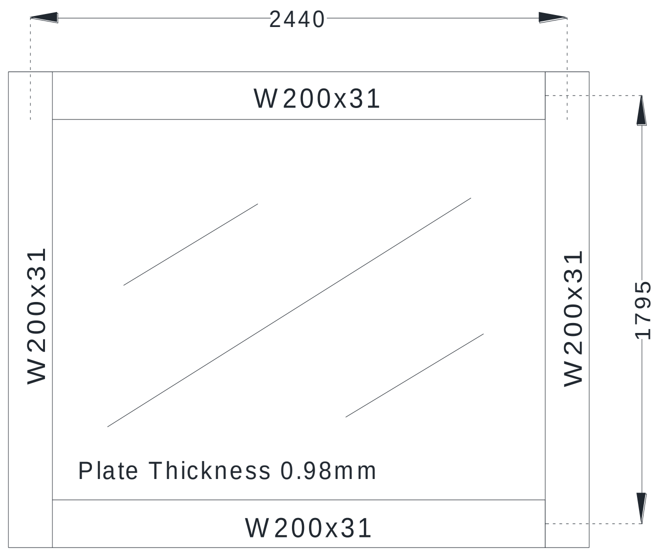


Figure 2.10: Schematic diagram of sample tested by Neilson (2010)

Almost all the experimental research in SPSW systems have attempted to validate with a numerical model. Numerical models are really convenient and cost effective way of carrying out analysis when large scale study with repeated analysis is involved. Several modeling technique has been discussed by researchers so far each having their own set of advantages and disadvantages. The most popular two methods worth mentioning are the strip model and the detailed Finite Element model. Some of these popular models have been discussed through this study.

2.4 **Strip model**

Though SPSWs are being used for decades, the consideration on contribution of post buckling strength in its design and thus modeling accordingly is relatively new. Thorburn et al. (1983) introduced a design technique where the post buckling strength was used. They introduced "strip model", which proved to be a reasonably acceptable method for analysis of

SPSW systems. In this model, the plate has been analyzed as a number of pin-ended discrete strips capable of taking tension only and oriented along the principal tension direction, which is found by the principle of the least work. Each strip was assigned an area equal to the width of the strip times the thickness of the plate. Inclination of strips with vertical was calculated based on principle of least work (Equation 2.1). For the development of the strip model, the beams were assumed to be infinitely rigid in bending in order to reflect the presence of opposing tension fields above and below the modeled panel. A possible configuration of strips in an interior panel is shown in Figure 2.11. Material properties were assumed to be same as that of original infill plate, like the tensile yield stress in plates is same as that of strips. Since, only post-buckling behavior was considered effective shear strength of plate prior buckling was neglected. This research also indicated a minimum of ten strips per panel is required for analysis. For cyclic and dynamic analysis where the load is not unidirectional the strips need to be oriented in both the directions, this made the modeling technique a bit more complex. Also, this model has been later criticized for under-estimating the stiffness (Driver, 1997). However, owing to its reliability Canadian steel design standard, CAN/CSA-S16-09 (CSA, 2009) accepted this model as a design tool for SPSW systems. A normal plane frame analysis using Finite Element (FE) can be carried out to get the relevant outputs from strip model. However, this method becomes more complicated when cyclic push-pull load (like earthquake load) are applied to the structure. For cyclic loadings, the orientation of strips in both tension and compression direction are necessary. This makes the model more complex and time consuming when repeated analysis is necessary. Elgaaly (1998) modified the strip-model by introducing gusset plates which connect the boundary element with the strips (Figure 2.12). In this modified strip model, orientation of truss strips were assumed to be 45^0 and the truss material was assumed to be elastic, elastic-plastic and perfectly plastic. Empirical relations were used to compute the reduced young's modulus and second yield stress after first yielding has occurred. A significant observation was reported that the strains at the ends of the diagonal strips near the supported boundaries are higher than the strains at the middle near the center of the plate panel. It was based on this observation that use of gusset plate in modeling technique was recommended. Since the gusset plate is expected to yield in shear before buckling, the dimension of square gusset was derived by equating shear yield stress of plate material with buckling shear stress of the equivalent square plate. Though this model was more accurate than strip model but it was by far more complicated.

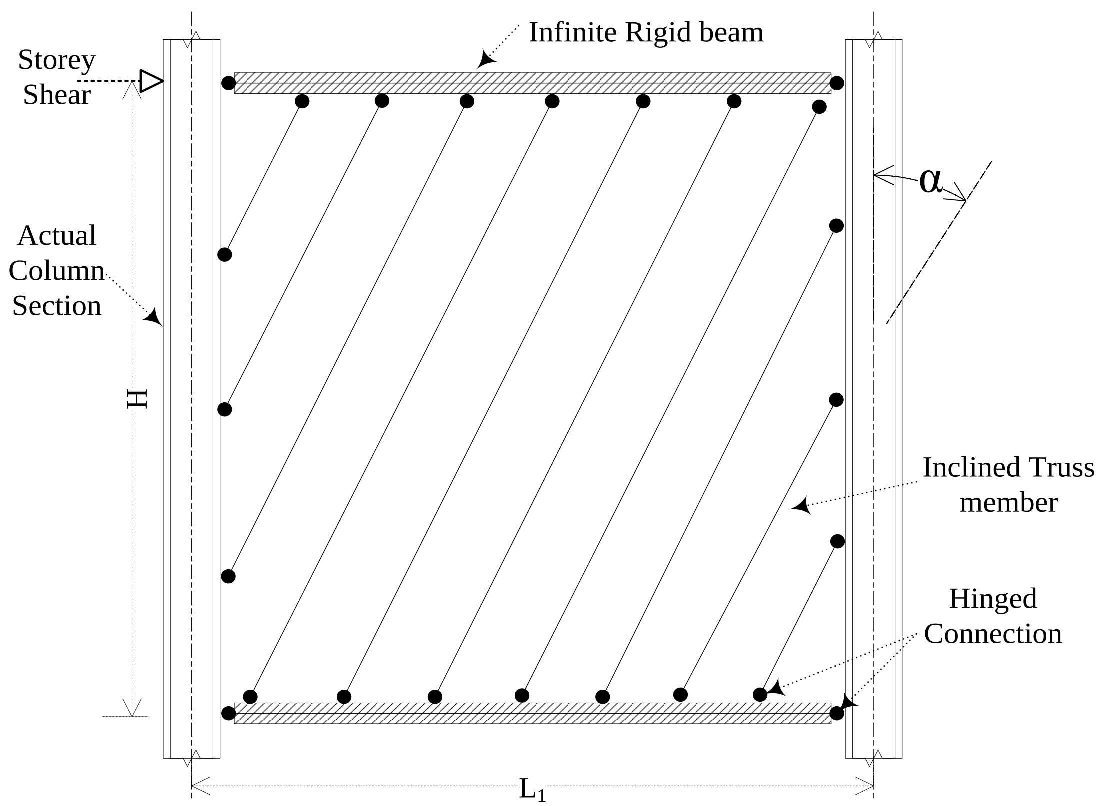


Figure 2.11: Strip model (Thorburn at al. 1983)

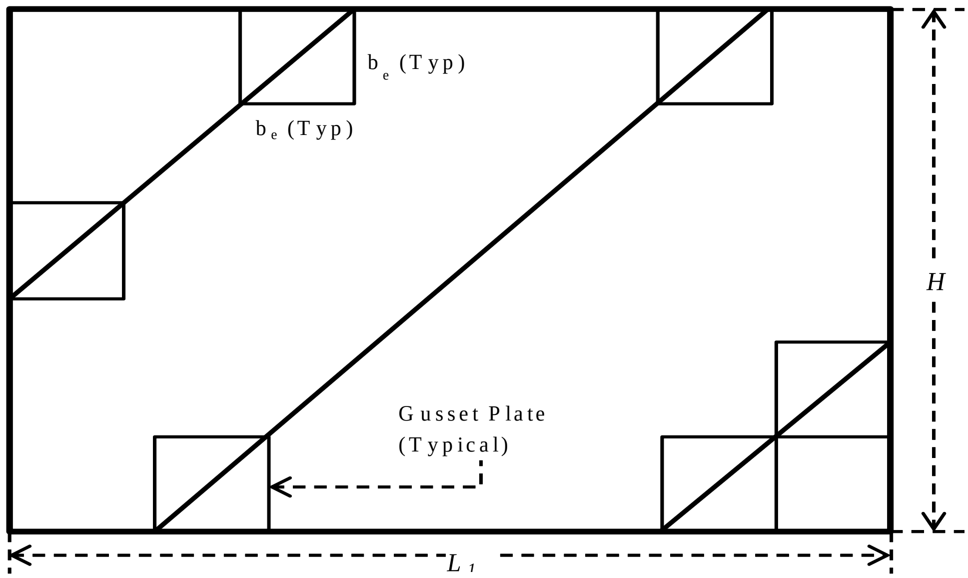


Figure 2.12: Model proposed by Mohamed Elgaaly (1998)

$$tan\alpha = \sqrt[4]{\frac{1+\frac{L_1 * b}{2A_c}}{1+\frac{H * b}{A_b}}} \tag{2.1}$$

where, α is the angle of inclination of tension field (Figure 2.11), b is infill plate thickness, L_1 and H are the width and height of panel, A_b and A_c are the cross-sectional area of the beam and column, respectively. The relation of $tan\alpha$ was later modified by Timler and Kulak (1983) as in Equation 2.2.

$$tan\alpha = \sqrt[4]{\frac{1 + \frac{L_1 * b}{2A_c}}{1 + b * H * \left(\frac{1}{A_b} + \frac{H^3}{360 * L_1 * I_c}\right)}} \tag{2.2}$$

where, I_c is the moment of Inertia of the columns and other symbols are as introduced before.

There have been several experimental tests to evaluate the performance of strip-model like Tromposch and Kulak (1987) conducted large scale test with two-storey structure laid horizontally on supports (Figure 2.13 and Figure 2.14). The beam to column connections were bolted connections. Before applying a cyclic load test, gravity load was applied to generated pre-existing stresses on the structure. Owing to the limitation of the loading machine, only 67% of the ultimate load was the maximum applied quasi-static load in cyclic test. This cyclic test was followed by a monotonic loading test where load up to the ultimate capacity of the specimen was applied. The main objective of the test was to validate the strip model proposed by Thorburn et al. (1983). Pinching effect for the presence of 3.25mm plate and flexible boundary elements was significantly noticeable. Also, the ductile behavior of SPSW system was indicated through the experimental study. Nonlinear pushover analysis was conducted using the strip model. The pushover curve had good agreement with envelope of hysteresis loops from test (Figure 2.15). It was also concluded that the strip model gave conservative estimates of both initial stiffness and ultimate capacity of Steel Plate Shear Walls.

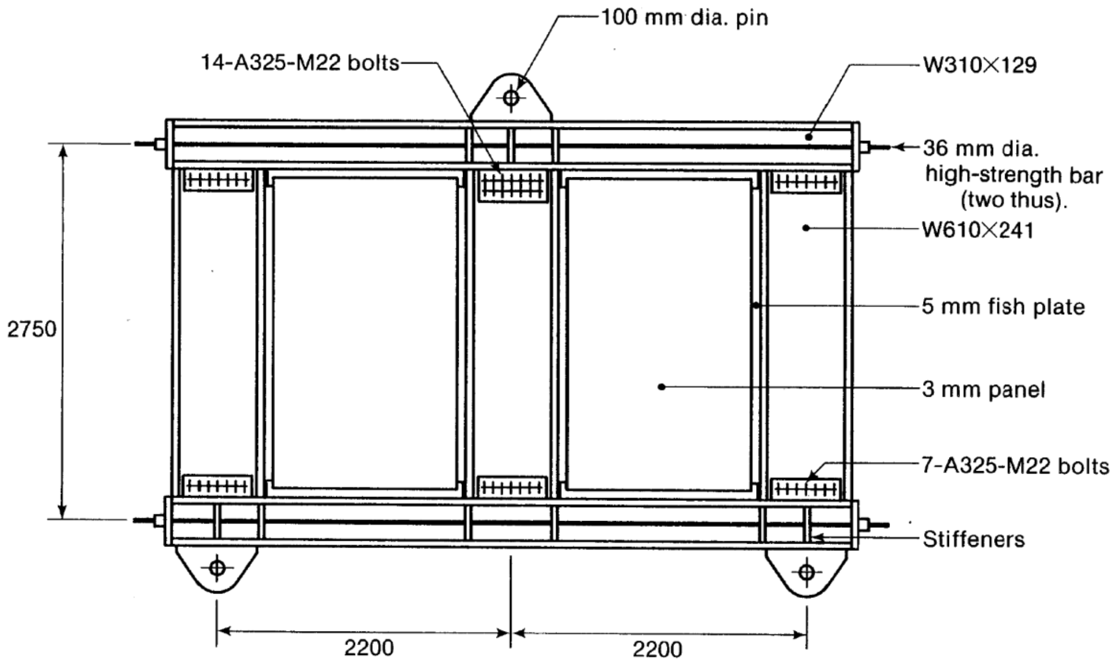


Figure 2.13: Sample tested by Tromposch and Kulak (1987).

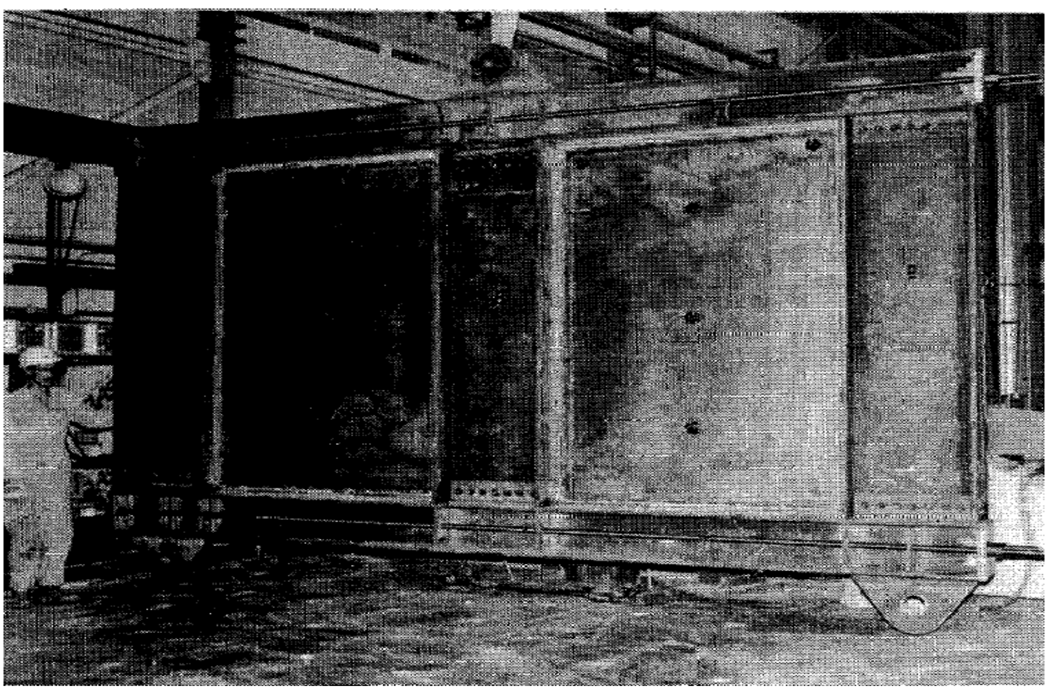


Figure 2.14: Experimental setup of Tromposch and Kulak (1987).

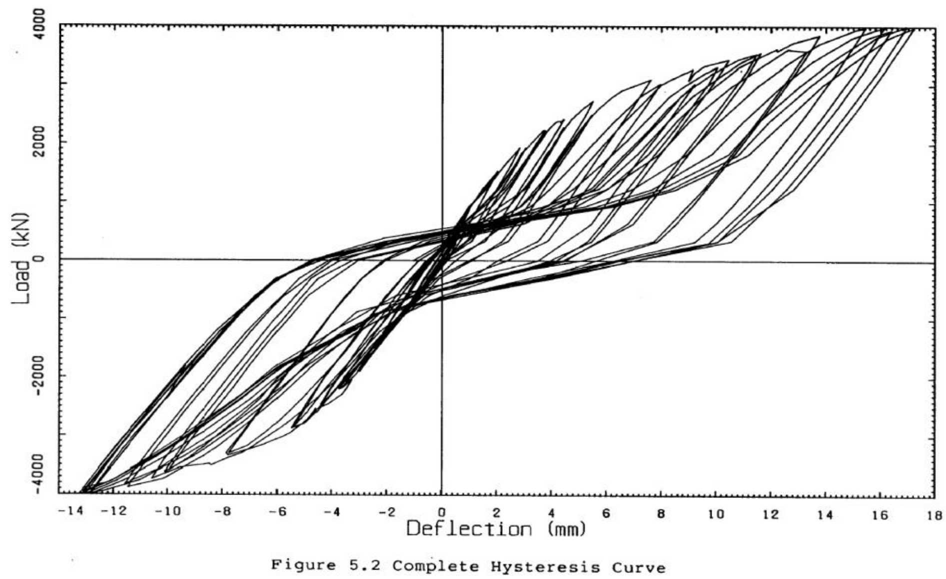


Figure 2.15: Hysteretic curves generated by Tromposch and Kulak (1987).

Driver et al. (1997, 1998) also used the strip model to get the pushover curve and compare with that of experimentally obtained hysteric loops. Ten pin ended strips with area as suggested by Thorburn et al. (1983) was used to replace the infill plate. For calculating the inclination of angle of tension field in the infill plate, the modified relation suggested by Timler and Kulak (1983) was used (Equation 2.2). The strip model was analyzed by a plane frame model using elastic analysis. Incremental load was applied and the strips yielded were removed from the model and tensile yield force from the strip was applied in the direction of the strip at the point where the strip was connected. When the column and beam reached its plastic moment capacity, a true hinge was placed with a constant moment at hinge joint. So, the analysis was carried out in several steps. Gravity load and P-Δ effects were also introduced into the model for analysis. Finally, it was concluded that though the ultimate strength is well estimated but the initial stiffness is slightly under estimated. The underestimation of stiffness was justified by several possible reasons like formation of localized compression field in the diagonally opposite corners of the frame that form acute angles in the deformed structure where compressed length of plate is short. Other possible reasons can be additional axial stiffness of the tension column arising from the presence of infill plate connected to it. It was also reported that increasing the number of strips from ten to twenty had no significant influence in the final outcome. A new hysteric model was proposed, based on the strip model that explicitly divides the SPSW into two components (the moment resisting frame and the infill panel). This hysteric model has shown good predictions for cyclic behavior.

Following the recommendations in design standard for strip model, Lubell et al. (2000) analyzed his test specimens using non-linear frame analysis software.. Unlike their real experiment the analytical models had rigid beams to simulate floor action. For samples SPSW1 and SPSW4 in their experiment the initial stiffness was significantly over predicted by the analytical study using strip model. However, the ultimate strength in all the models was close enough. It was justified that the flexural modes caused by columns of low stiffness, aspect ratio and panel height significantly influenced the system behavior. The yielding sequence and inelastic characteristics was influenced by high axial and flexural co-efficient developed for excess over turning moment in columns. Thus, presence of excess over-turning moment in columns affects the accuracy in results that can be obtained from strip-model.

As explained before, strip model has some limitations. However, it still remains as one of the most widely accepted method for analysis of SPSW systems. A modeling technique simpler than strip model and yet equally reliable (if not more) is still in nascent stage amongst researchers.

2.5 **Detailed finite element model**

Finite Element Analysis (FEA) proved to be the acceptably accurate way of modeling SPSW (Elgaaly et al., 1993, Driver et.al, 1997). Strictly speaking strip-model and equivalent braced model are also finite element models. But those are simplified model made with several assumptions. Under the topic of detailed FE model, complex behavior of shell-plate element can be discussed. Through FEA it is possible to introduce both geometric non-linearity and material non-linearity. Shell elements representing the plate gave a better estimate of the stiffness and strength. However, the accuracy of FEA model depends greatly on the choice of modeling techniques used. Several modeling FEA modeling techniques like Mohammad et al. (2003), Bhowmick et al. (2010) have been suggested in past, where the efforts were mostly directed to developing a method for predicting more accurate behavior of SPSWs.

Successful finite element modeling technique was introduced by Driver et al. (1997, 1998b) to predict the behavior of SPSW system and compare the model with their experimental results. Quadratic beam elements were used to model the boundary members and quadratic shell element for the plate. Initial imperfections were introduced in the model based on first buckling mode and experimentally obtained residual stresses were included in the boundary members. The dimensions and material properties in FE model was specified as is in case of experimental specimen. Elastic perfectly plastic material curve with kinematic hardening model introduced material non-linearity in the model along with geometric non-linearity for initial imperfection. With a monotonic load pushover curve was generated which gave correct estimate of the ultimate strength but slightly over estimated the initial stiffness of the specimen. Cyclic load analysis failed to capture the pinching effect and re-distribution of tension fields. More research in FE modeling was recommended by authors.

Analytical study was carried out by Mohammad et al. (2003) using FE software package (Abaqus/Explicit). Material and geometric non-linearity included in the FE model made the model more robust. A kinematic hardening material model was used to simulate Bauschinger effect for cyclic analysis of the SPSW. Also, to make the model displacement control, the concept of loading frame was used. Monotonic and cyclic load test was carried out using the FE model to validate both the experimented three-storey model and the previous four-storey model tested by Driver et al. (1998a). To avoid numerical instability dynamic explicit analysis was carried out with sufficiently small time step so that the final results remain reliable. The FE model and the experimental ones showed good match (12% under estimate in three-storey and 7.8% under estimate in four storey capacity), even the pinching effect was almost accurately captured.

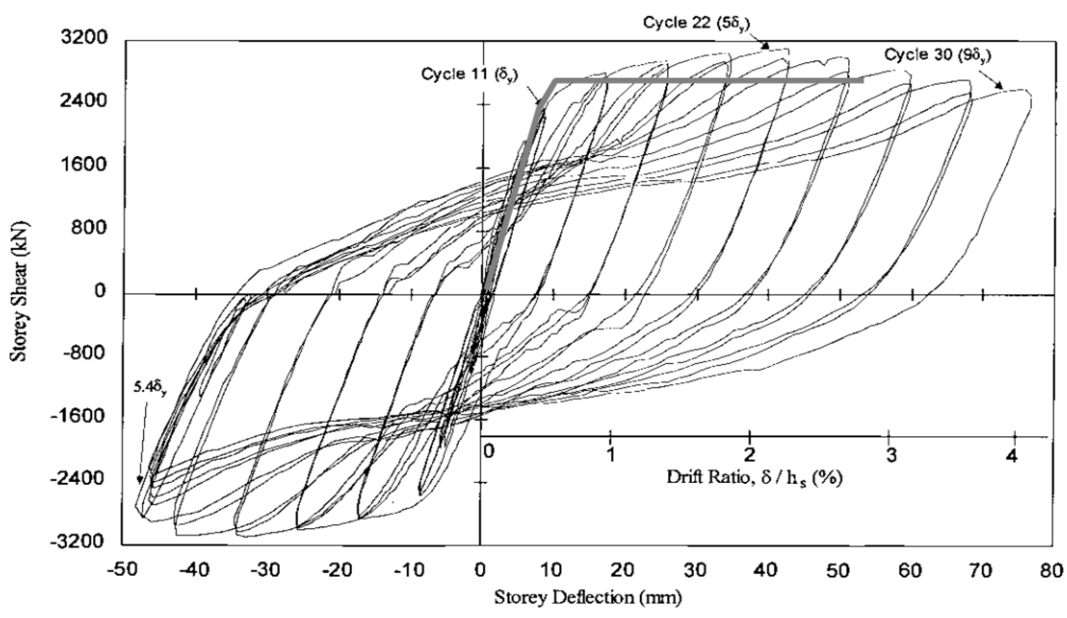
FE modeling is considered to be the acceptably accurate from of analysis for SPSW systems. Wherever highly accurate results are demanded, a detailed FE analysis with shell-plate element representing the infill should be used. However, research have indicated that even with this complicated method the pinching effect is at times not correctly estimated (Driver et al. 1998b). The reason FEA with shell elements is commercially not a practical choice is for it’s over complicacy in modeling, particularly for high rise buildings huge time is required for analysis. Speeding up the analysis without compromising accuracy significantly is particularly important when a multi-storey model is analyzed under cyclic or dynamic loading due to earthquakes. Such structures may need to be analyzed for a suite of seismic ground motions in order to carry out a performance-based design.

2.6 M-PFI model

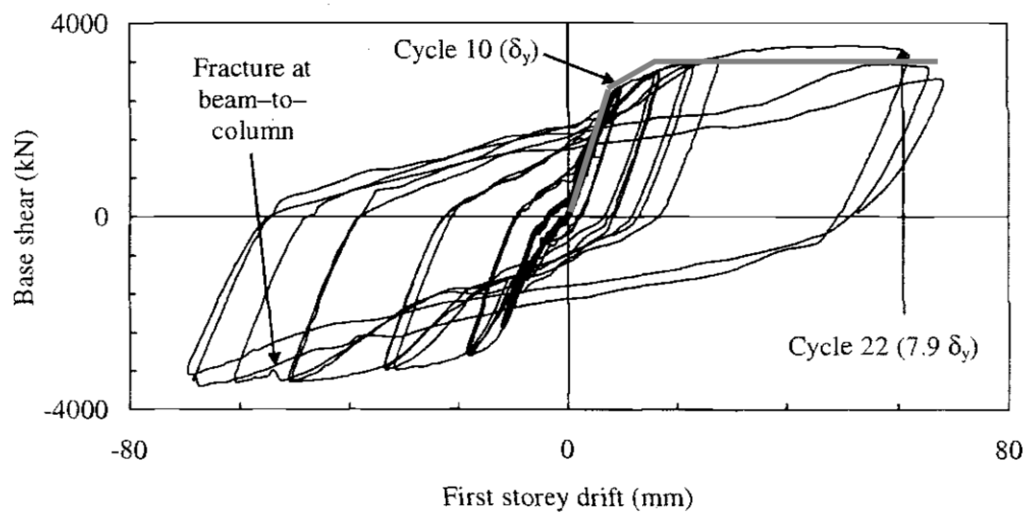
Very few attempts have been made in developing a technique where analysis can be made faster and reasonably accurate. Modified Plate-Frame Interaction method (M-PFI) is one such method developed by Kharrazi et al. (2004). The model was based on the critical observation of the ductile load-displacement curve of SPSW systems. The three main parts of the load-displacement curve, namely elastic buckling, post-buckling and yielding, is treated separated and then combined into the M-PFI model. Initial steps of the model involved the shear analysis of infill plate and frame separately, through which a relation of shear and load-displacement for each infill plate and bounding frame were obtained. These individual relations were super imposed to obtain the shear behavior of SPSW system. In the next step, bending analysis was conducted where the infill plate and frame was considered as a single structural unit. Finally, the interaction of bending and shear was developed and that concluded the M-PFI model. Through the set of equations described in M-PFI model, certain points on the load-displacement curve can be obtained, thus indicating the behavior of the ductile SPSW system. Through this model, with sufficient hand calculations, it is possible to evaluate design parameters (such as the shear load-displacement values, strength, stiffness and limiting elastic displacement for the steel plate and plate-frame interaction) and their effect on the overall SPSW capacity.

To establish the numerical model with proper validation, Kharrazi et al. (2004) used the experimental results from Driver et al. (1998a). For simplicity the model assumed the tension field developed has an inclination of 45^0. The push-over curve obtained from M-PFI model gave a good estimate of the envelope of cyclic test (Figure 2.16). However, it was reported that the approximate model overestimated the initial stiffness by 5% and under estimated he ultimate capacity by 10%. The main limitation of this model is in describing the ductility of SPSW specimen or the actual failure mechanism. Also, nothing about obtaining the member forces was mentioned through this study.

The main advantage of this model is in its incorporation of bending effect for high rise buildings and is suitable incorporating practical seismic design provisions. However, cyclic load test or time history analysis is not possible through this model. This restricts the applicability of M-PFI method in performance based design.



(a) Driver et al., 2000



(b) Mohammad et al., 2003 by Kharrazi (2004)

Figure 2.16: Test of M-PFI model with experimental results from

2.7 Existing braced models

In this modeling technique truss members representing non-concentric braces along with beam member representing the boundary frame is modeled. Property of the braced and bounding members are so established that as a whole the braced frame represents the original SPSW system. The main advantage with this model over other modeling technique is that it takes the accuracy of FEA and yet very simple to model and most importantly requires very less time for analysis. Also, real time dynamic analysis can be easily performed. However, the accuracy depends greatly on how the properties of the members are established. Thus, the challenge in establishing the material and geometric property of truss braces such that they are capable of representing the behavior of SPSW systems opens up an area of interest amongst researchers. From the very beginning of research with SPSW, attempt has been made to develop an equivalent truss model that can accurately predict the behavior of SPSW. Thorburn et al. (1983) was one of the first to attempt it. Diagonal braces connected at beam column joints by pin connection, capable of taking tensile force and remaining frame members were same as in actual SPSW was used to develop the truss model. Thorburn's formulation had the objective of determining the area of the diagonal braces (A_d) such that the truss model and their strip model gave same top displacement under shear. They assumed the boundary members to be rigid for the calculation of A_d. For modeling in finite element beam is always considered rigid and actual member dimensions are used as columns. By principle of least work and equating the stiffness of the plate with that of the brace in tension, they came up with significant area of the plate through which the effective tension field works (Figure 2.17), which can ultimately be related to Ad (Equation 2.3). Most of Thorburn's work was based on geometric distribution of tension field. This method of formulation does not necessarily always yield accurate results, thus making the model unreliable.

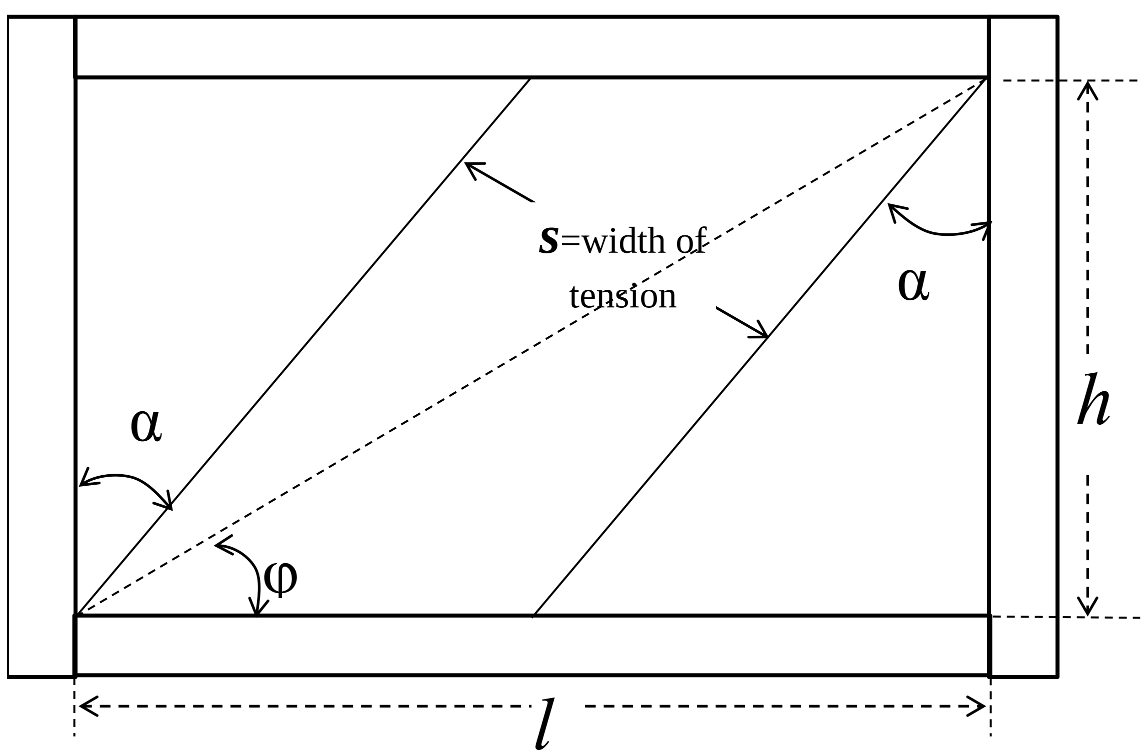


Figure 2.17: Truss model by Thorburn et al. (1983)

$$A_d = \frac{l * b * \sin^2(2 * \alpha)}{2 * \sin(90 - \varphi) * \sin(180 - 2\varphi))} \tag{2.3}$$

where, l is the width of plate, b is thickness of the plate and φ is the angle of brace with the beam (as in Figure 2.17)

Topkaya and Atasoy (2009) came up with a new method for computing the area of equivalent brace model. In their model the beams are the only rigid members. Also, the beams are assumed to be connected to columns by pinned joints (Figure 2.18). By using some empirical and analytical equations primarily developed for hand calculation method, they could come up with an area for the truss braces (A_d). An enhancement of area of the vertical boundary member (A_{ver}) was also recommended thought their study (Equation 2.4 and Equation 2.5). This recommendation for increasing the boundary member's area gave a good estimate for the initial stiffness of SPSW systems. All their computations are restricted within elastic limit. For tall multi-storey structures, the braced model is observed to overestimate the initial stiffness as well. Their work also introduced a parameter α_s which is the ratio of the post-buckled stiffness of the plate to the pre-buckled original stiffness (Figure 2.19). Unlike other parameter, a representative table indicating the possible values of α_s based on slenderness and aspect ratio of plates has also been provided. However, no proper statistically developed mathematical relation has been indicated to estimate the value of α_s. The truss model is workable on computers allowing repeated analysis with cyclic loadings. Validation with acceptable range of accuracy, on the stiffness predicted from both these two models, has been done with some available experimental results, some Finite Element models and some strip model results as well. Through their calculations and study it has also been indicated that due to presence of stiff boundary members, the buckling of plate under bending does not significantly influence the overall inertia. Thus, for development of a simplified approximate model the bending effect can be safely neglected.

$$A_d = \frac{L_d^3\left({I_{pl}}^2/\beta_m\right)}{2.6h * (l + d_c)^2} \tag{2.4}$$

$$A_{ver} = \frac{I_m}{0.5 * (l + d_c)^2} \tag{2.5}$$

where, h and l are height and width of plate respectively, L_d is diagonal length of plate, d_c is the depth of column section, I_{pl} is the moment of inertia of the plate and I_m is the modified moment of inertia of the plate given by Equation 2.6. β_m represents the sum of contribution of shear stresses in column (β_1) and infill plate (β_2) as represented by approximate Equation 2.7. Also, the contribution of shear stress from plate is enhanced by coefficient α_s for considering geometric non-linearity.

$$I_m = 2I_c + 0.5A_c(l + d_c)^2 + \alpha_b I_{pl} \tag{2.6}$$

$$\beta_m = \int \frac{Q^2}{b^2} dA_{pl} \approx \beta_1 + \left({}^{\beta_2}/_{\alpha_s}\right) \tag{2.7}$$

where, Q is static moment of the area with respect to neutral axis, b is width of section, I_{pl} and A_{pl} are second moment of inertia and area of steel plate wall respectively, I_c and A_c are second moment of inertia and area of cross section for column respectively. α_b is the ratio of post-buckled stiffness of the plate under bending to the original pre-buckled stiffness. Significance of α_b is also reported to negligible in most cases.

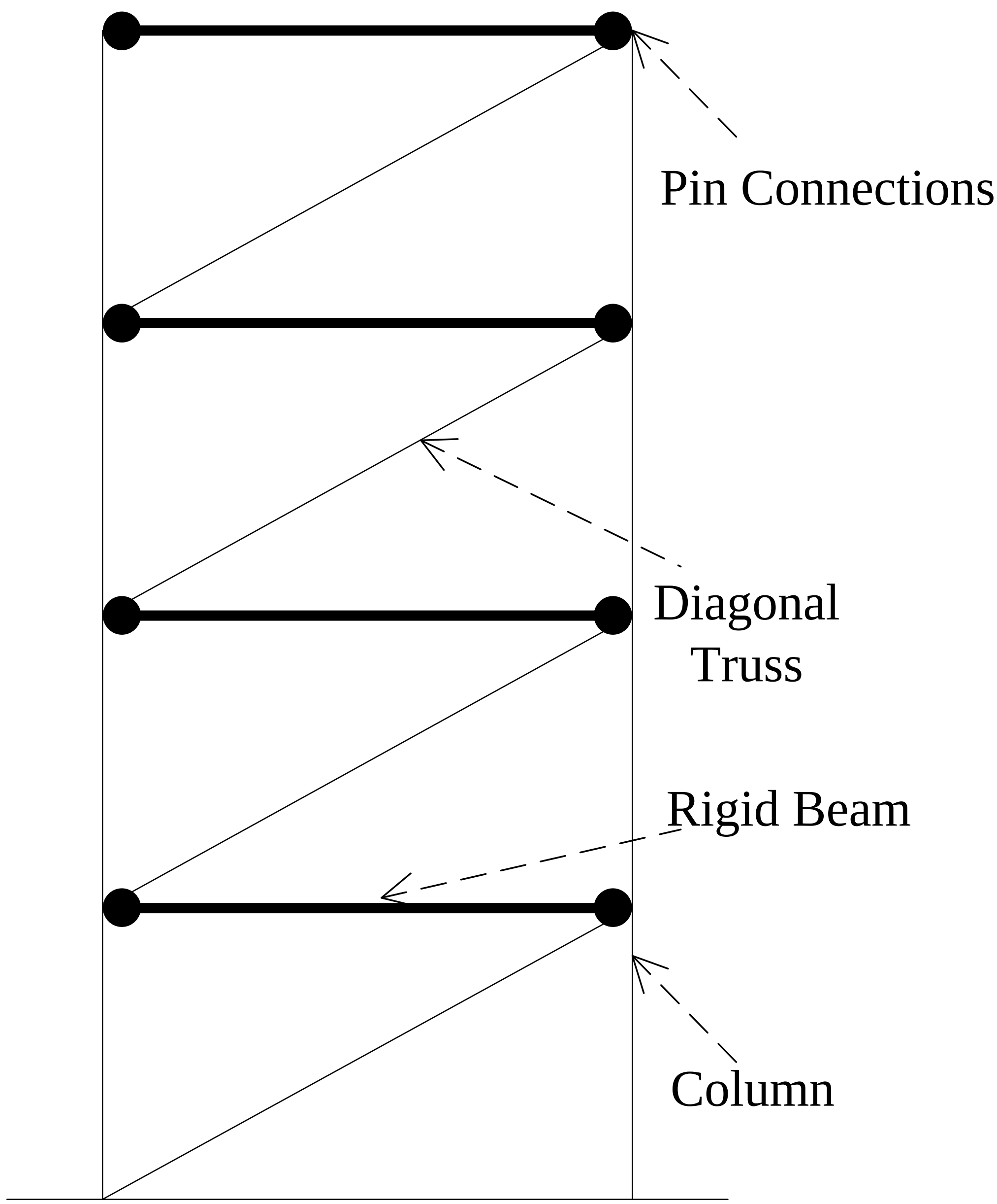


Figure 2.18: Diagonal truss model proposed by Topkaya and Atasoy (2009)

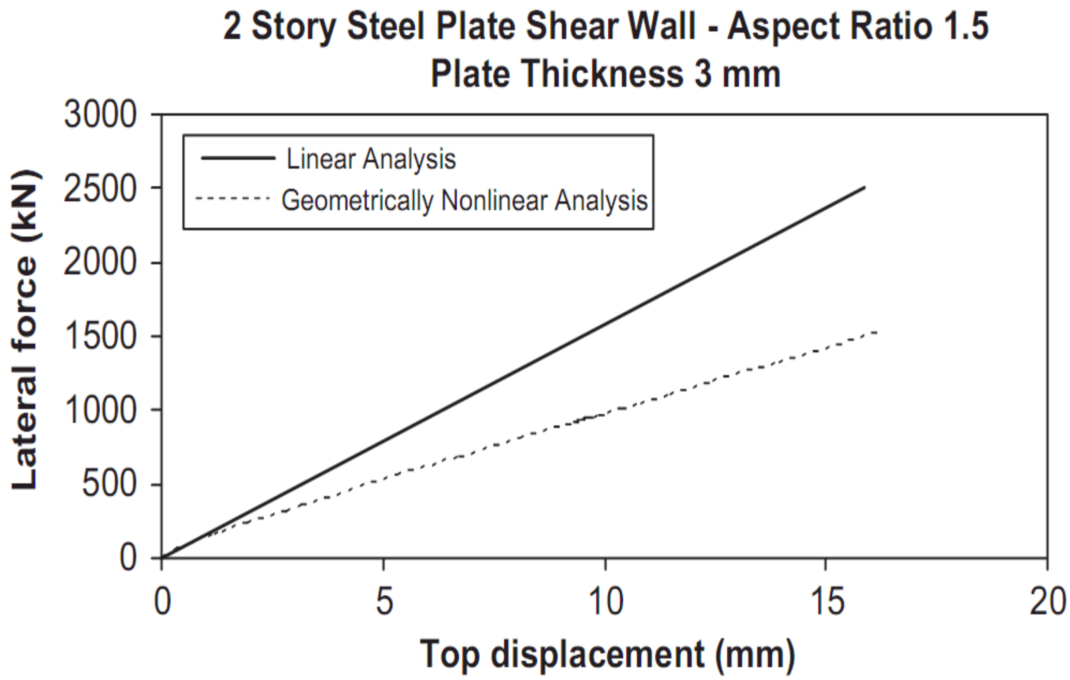


Figure 2.19: A typical Load displacement response of SPSW comparing the change of stiffness for buckling of infill plates by Topkaya and Atasoy (2009)

2.8 Some other modeling technique

Tromposch and Kulak (1987) also developed hysteretic model based on research by Mimura and Akiyama (1977) (Figure 2.20). Frame stiffness effect of low panel buckling strength was incorporated in their model. This was one of the beginning level models that could estimate the cyclic load-displacement curve. However, owing to its approximation and complicacy in computation it was not observed to be that popular amongst researchers in this area.

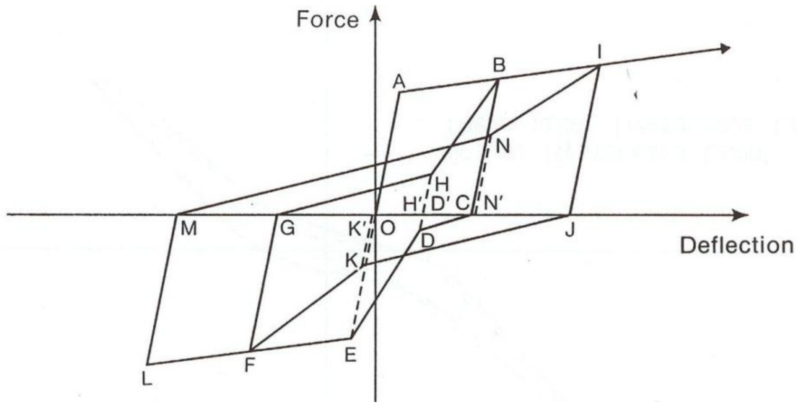


Figure 2.20: Hysteresis model proposed by Tromposh and Kulak (1987)

Topkaya and Atasoy (2009) performed a series of numerical study to develop an approximate hand calculation method to estimate the top displacement of SPSW systems. Modifying the classical deep beam theory based on some parameters as found suitable for the SPSW system under pure shear and pure bending action. Parameters like geometric non-linearity has been implemented into the model by repeated analysis with varying geometric properties in Finite element model. This hand calculation model incorporated some analytical system of equations merged with empirical relations developed by parametric study using FEA model. An example solution for hand calculation method has also been presented by Topkaya and Atasoy. The hand calculation and truss model methods have reported an average of 8% and 6% stiffer in comparison to strip model. A comparison of normalized stiffness for different cases of samples tested by the author has been indicated (Figure 2.21). The model is good for making prior estimate on expected top deflection of SPSW before carrying out final design. Though, this model gives a reasonably accurate estimate of stiffness but only within linear elastic limit of the material. Also, for repeated analysis this model is cumbersome and time consuming. Restrictions with cyclic load test and dynamic tests like time history analysis is also a significant limitation.

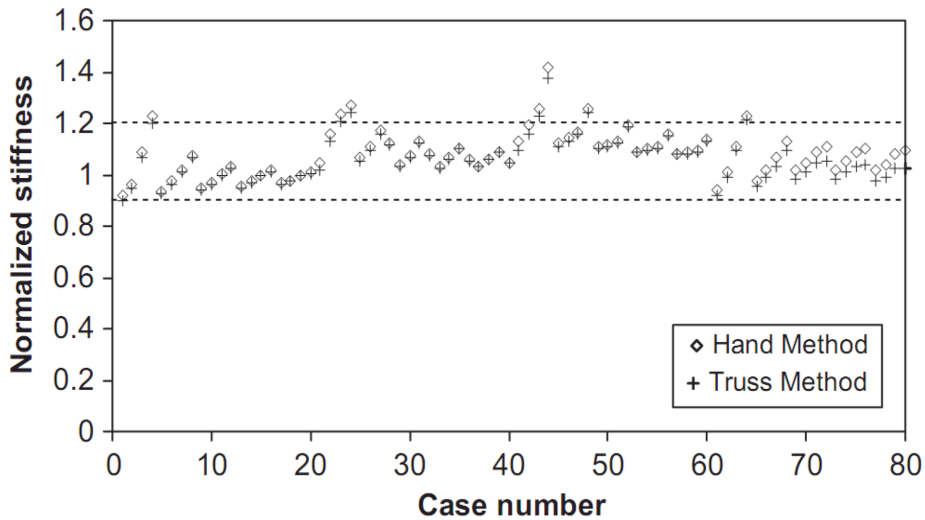


Figure 2.21: Comparison of results for hand and strip methods of analysis with existing methods reported by Topkaya and Atasoy (2009)

2.9 Summary

Research on Steel Plate Shear Wall system has been continuing through decades. Almost all literatures, indicated its effectiveness in overall increased structural ductility, stable hysteric behavior, enhanced stiffness and ability to dissipate energy during earthquakes. Thus, steel plate shear wall is gaining more popularity in construction industries specially, in zones of high seismicity where ductility demand of structure is very high. This opens up a demand on more sophisticated and precise design techniques. Involvement of light-gauge infill panels in modern SPSW system is also expected to have more importance in near future. As indicated through this study, research on light gauge SPSW has been very limited (Kharrazi (2005), Berman and Breunau (2003), Neilson (2010)). Even with the welding technique being properly established (Neilson, 2010), all it remains is a proper analytical study. It should be noted here that currently no design guidelines are available for seismic design of light-gauge SPSWs. Thus, a numerical study will be conducted in this research to assess the applicability of the capacity based design approach, currently used for design of ductile SPSWs, for seismic design of light gauge SPSW systems.

While carrying out this objective with repeated analysis of several multi-storey structures, a bigger realization comes to play i.e. a demand for simplified modeling technique need to be established. From the study of literatures done so far it can be inferred that there is a deficiency in an established simplified model that is reliable and at the same time very fast computationally. Also, the new modeling technique to be established should be capable of carrying out real time time history analysis very fast. It has also been observed that capacity based design technique has already been discussed by several researchers (Bhowmick et al. (2009); Berman and Bruneau (2008)), no study on performance based design is currently available. This is probably due to the limitation of available modeling techniques. Presently available models are either not reliable or far too complex for repeated dynamic analysis. Thus attempt will be made to establish simplified and reliable modeling technique using equivalent bracing system that can be used for performance based seismic design of steel plate shear walls.