Theoretical Seismic Behaviour of Steel Plate Shear Walls

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ABSTRACT:

In the last three decades, many researches have been carried out on steel plate shear walls, SPSWs, and they have been introduced as seismic load resisting systems. However, their non-linear behaviour complexities such as buckling, post buckling and tension field action of infill steel plate, in-elastic out-of-plane large deformation of steel plate, hysteresis loops, interaction with the surrounding frame, several configurations of SPSWs with thick plates, stiffened or un-stiffened thin steel plates, etc., have made them still attractive for more investigation. This paper presents shear strength capacities and proposes relevant formulas for SPSWs with and without stiffeners with considering effects of the relative stiffness and strength of the boundary elements on the shear capacities. Several analytical finite element models with different h/t of steel plate are considered and kinematic hardening model is used for steel material, non-linear large-displacement analysis under monotonic loading has been carried out and their push-over curves obtained. The results are compared with the experimental studies by other researchers and codes regulations. The theoretical and analytical studies indicate that the shear strength capacities of SPSW vary between results from using the plate girders simulation model for SPSW and the strip model, and depend on the relative stiffness and strength of the boundary elements as well as the slenderness of the infill steel plate.

KEYWORDS: Steel Plate Shear Walls, Plate Girders, Push-Over, Strip Model, Boundary Elements

1. INTRODUCTION

Recent researches have demonstrated that steel plate shear walls, SPSWs, can act as effective and economic seismic load resisting systems in the high risk zones. SPSWs have high elastic stiffness, large displacement ductility, and stable hysteretic behaviour and high energy dissipating capacity. They have been used in the structural design and retrofitting of existing buildings with different configurations, with thick steel plate, stiffened or un-stiffened thin steel plate. Use of thick steel plates or stiffened still plate in SPSW intends to conduct the steel plate to its full plastic strength prior to the elastic out-of-plane buckling of the plate and to prevent tension field action from developing in the infill plate , however, in un-stiffened SPSW with thin infill steel plate the post buckling strength of the infill steel plate due to tension field action development in the steel plate after the elastic out-of-plane buckling is main part of the shear strength capacity of the system.

The history of SPSWs returns to the plate girders, and the analogy that simulates the columns of a SPSW to the plate girders flanges and the beams to the stiffeners and the infill plate to the plate girder web is appropriate for general understanding of the system behaviour, however, it does not completely represent the behaviour of the SPSW, especially the non-linear and cyclic performance of the system. The differences mainly result from the boundary elements effects on the system responses. Many theoretical and experimental investigations have been carried out by researchers on SPSWs, which some of the relevant studies are summarized in the following. Wagner [1] is the first researcher who used a complete and uniform tension field to determine the shear strength of a panel with rigid flanges and very thin web, and inferred that the shear buckling of a thin aluminium plate supported adequately on its edges does not constitute failure, this idea has been recently developed for modelling of thin SPSWs. Basler et al. in [2] carried out shear tests on plate girders with vertical stiffeners and developed a failure theory based upon a theoretical model with a diagonal band of tension yield, they ignored effects and contribution of the flanges to shear strength of the plate girder, the web buckled at a predicted theoretically shear stress and subsequently due to stress distribution changing in the web, considerable post-buckling strength obtained.. Rocky and Skaloud [3] found that the rigidity of flanges has a strong influence upon the behaviour of panel and when the flanges are very light the collapse mechanism approximates to that assumed by Basler, but if they are heavy the plastic hinges form in the four corners of the panel, and for intermediate flanges two numbers of the plastic hinges will be located at the flanges and the remained two ones at the corners of the panel, Murray [4]. Other researches were also conducted on modelling of tension field action in the plate girders and collapse mechanism. In the following of these studies, Thorburn et al [5] developed an analytical method based on the Wagner's work to evaluate the shear resistance of un-stiffened SPSWs with thin steel plates and introduced the strip model to represent the shear panel as a series of inclined tensile strips, they assumed that the beams are rigid enough for developing the complete tension field action in the infill steel plate. Timler and Kulak [6] modified formula for angle of strips inclination with the column .Caccese et al.[7] presented the cyclic testing results of six 1:4 scale specimens with various plate thicknesses and moment resisting beam-to-column connections, and two specimens with shear beam-to column connections, the specimens with thicker plates showed an inelastic behaviour that was primarily governed by the columns, and the capacity of the specimen with the thickest plate was limited by the column instability. Lubell et al.[8] tested two single and one 4-story thin SPSWS under cyclic loading and compared the experimental results with the simplified tension field analytical models and found that the models can predict post-yield strength of the specimens well, with less satisfactory in the elastic stiffness results. Sabouri-Ghomi et al.[9] developed supporting theories based on the interactions between the system components for estimating the shear strength of SPSWs. Astaneh-Asl [10] provided a report including applications of steel plate shear walls in USA and Japan, and defined three regions compact , non-compact and slender of SPSWs behaviour in similarity with the plate girders. Driver et al [11] tested a 4-story large-scale specimen and developed its finite element model. Berman and Bruneau [12] presented a plastic analysis method for analysis and design of SPSWs based on the strip models assumptions. Alinia and Dastfan[13] studied the effects of boundary elements on behaviour of 1-story thin SPSWs and unlike to the present views, they concluded the flexural stiffness of surrounding members has no significant effects on the post-buckling behaviour of SPSWs, the result was not validated with the experimental method.

In this study, non-linear seismic behaviour of SPSWs has been investigated and the shear strength capacities of the un-stiffened and stiffened SPSWs are presented considering effects of the relative stiffness and strength of the boundary elements in the shear capacities and non-linear behaviour of the system. Several analytical finite element models of SPSWs, which have been validated with the experimental results of the other researchers and codes provisions, with various boundary conditions and slenderness of the infill steel plate, have been selected to verify and compare the analytical and theoretical outcomes. The kinematic hardening plasticity model is used for mild steel material modelling, and non-linear large-displacement analysis under monotonic loading are performed and the push-over curves obtained and the summarized results presented.

2. ANALYTICAL STUDY

2.1. Numerical Modelling and Analysis Method

Several numerical models of one-story stiffened and un-stiffened SPSWs are generated for evaluating the shear load capacity of SPSWs with various stiffness and strength of the boundary members and the infill plate slenderness h/t. All modelling are conducted using general-purpose nonlinear finite element program ANSYS, which is properly suited for the solution of highly nonlinear engineering problems like SPSWs. The numerical models have been validated with the available experimental data in the literature, and those models with not having the experimental data with the codes provisions. The kinematic hardening plasticity model has been utilized with multi-linear kinematic hardening material model for the mild steel materials. The Bauschinger effect is also included in the multi-linear kinematic hardening model and finite element geometrically nonlinear analysis by means of large deflection transient analysis has been executed; therefore the nonlinear buckling and post-buckling effects including local buckling of the steel elements have been incorporated into the results. The implicit solution method based on Newmark's algorithm is utilized, and 4-node plastic shell with six degrees of freedom at each node is employed for 3D-modelling of the shear walls, and appropriate time-stepping by the trial and error method is used to overcome to convergence problems.

2.2. Analytical method Verification

The analytical method has been validated with using the available experimental results in the literature; therefore the SPSW2 specimen from Lubell's work [14], Fig.1, is selected and modelled. The analytical load-displacement curves from the non-linear finite element modelling; FEM, analysis based on the described method is obtained and compared with the experimental results in Fig.2. It is inferred that the used analytical method has been successful to estimate the actual shear capacity of the system in comparison with the experimental results with good approximate precision (less than 5% deviation), moreover, it is observed in SPSW1 that has had one S75x8 beam instead of two beams at top the shear wall capacity has been less than SPSW2, which implicitly implies that the stiffness of the boundary elements will have affect on the strength of the system. Then, the verified analytical method has been extended and used for the other SPSWs samples.

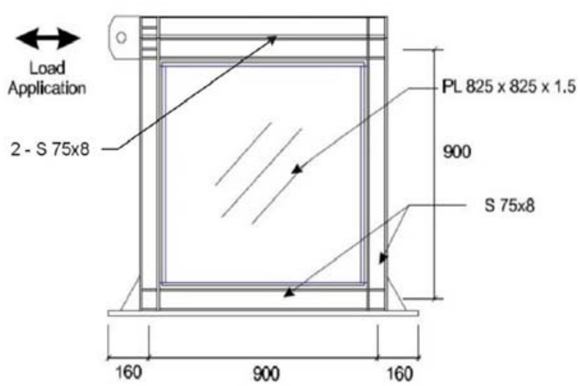


Fig.1. SPSW2 experimental model, Lubell-97

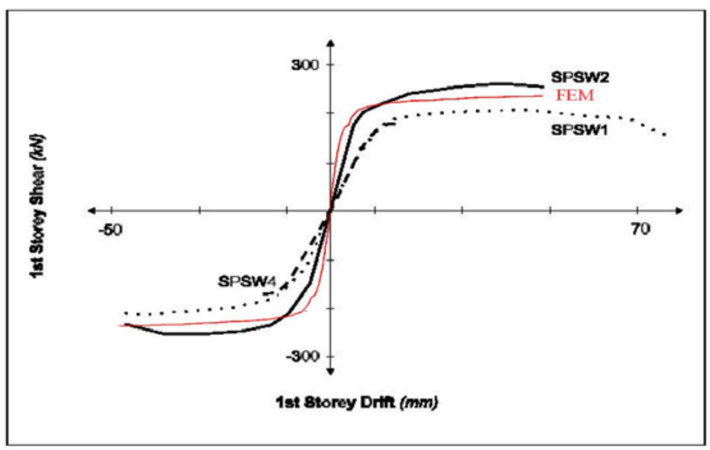


Fig.2. Envelope Curves for SPSW1, SPSW2, SPSW4 (Lubell-97) & Analytical model of SPSW2 (FEM)

2.3. Other Analytical models

Numbers of full scale models of one-story (3m x 3m) stiffened and un-stiffened SPSW with rigid beam-to-column connections and different slenderness of the infill plate are generated and the boundary elements are such designed to meet the preliminary requirements of steel plate shear wall and AISC 360-05 [15] provisions for the compact sections, then their flexural and shear stiffness have been changed from flexible to enough rigid for complete strip model formation in the infill plate. Details of the FEM models are given in Table 1; they are 4 shear walls of type SPSW (A) with 6 mm infill plate thickness and 4 shear walls of type SPSW (B) with 3 mm thickness of the infill plate, the vertical boundary elements, VBE, means the columns and the horizontal boundary elements, HBE, the beams and h / t is the slenderness of the infill plate between the columns clear span, I is the moment of inertia around x-x axis, and Mp is the plastic moment and Aw is the web cross area of the section.

Table 1. Details of the analytical models

Analytical Models	h / t	I (VBE) cm^4	Mp (VBE) KN.m	Aw (VBE) cm^2	I (HBE) cm^4	Mp (HBE) KN.m	Aw (HBE) cm^2	VBE (Fl.) mm	VBE (web) mm	HBE (Fl.) mm	HBE (web) mm
SPSW(A1)	450	24142.5	383.4	24	7478.3	174.0	16	PL300X15	PL300X8	PL200X15	PL200X8
SPSW(A2)		25717.5	421.2	45	7945.0	190.8	30	PL300X15	PL300X15	PL200X15	PL200X15
SPSW(A3)		52515.0	793.8	45	16960.0	367.2	30	PL300X30	PL300X15	PL200X30	PL200X15
SPSW(A4)		55890.0	874.8	90	17960.0	403.2	60	PL300X30	PL300X30	PL200X30	PL200X30
SPSW(B1)	900	24142.5	383.4	24	7478.3	174	16	PL300X15	PL300X8	PL200X15	PL200X8
SPSW(B2)		24142.5	383.4	24	7945.0	190.8	30	PL300X15	PL300X8	PL200X15	PL200X15
SPSW(B3)		25717.5	421.2	45	7478.3	174.0	16	PL300X15	PL300X15	PL200X15	PL200X8
SPSW(B4)		25717.5	421.2	45	7945.0	190.8	30	PL300X15	PL300X15	PL200X15	PL200X15

Moreover, some of the un-stiffened models have been stiffened with appropriate transverse stiffeners in accordance with AISC criteria for the plate girders in delimiting the slenderness of the web plates to prevent the shear buckling occurrence in the plate, and their non-linear behaviour has been studied. The mechanical properties of the used mild steel in the analytical models have been assumed as:

E=2.1E5 MPa ; Modulus of elasticity
Fy=240 MPa ; Yield stress
υ = 0.3 ; Poisson's ratio

2.3. Analytical results

some of the analytical results from non-linear analysis of the un-stiffened shear walls are presented in Fig. 3, it shows the developed stresses in the shear walls due to the monotonic loading till to reach 100 mm displacement on the top of the specimen corresponds to 3.3% drift, the loadings have been applied gradually by time sub-step controlling and in the ramped shape.

It can be observed that the distribution of the stresses based on the Von-Mises yield criterion in the specimens from SPSW(A1) , which has the most flexible boundary elements here with having less than half of the required moment of inertia of VBE specified by Kuhn et al.[16] , to SPSW(A4) that has the stiffest boundary elements has changed whereat the yielded areas in the infill plates have been extended by stiffening of the boundary elements, which implies that the shear capacities of the system has been consequently increased, the similar phenomenon was reported for the plate girders by Rocky and Skaloud [3]. The push-over curves of the un-stiffened samples series SPSW(A) and SPSW(B) are obtained and compared in Fig.4a ,Fig.4b, respectively. It has resulted that the push-over curves of the un-stiffened specimens with enough stiffness and strength of the boundary elements have had better curves than the specimens with flexible members, and just the shear walls with enough strong of the boundary elements have been able to reach the expected shear capacities based on the strip model method, and the shear capacities of the specimens with flexible boundary elements have been between the shear capacities of the similar plate girders with the slender web as a lower limit and the strip model result as an upper limit, the relevant formulations and discussion about the strengths of the boundary elements are presented in the next section. a sample result of the stiffened SPSW(B1) with horizontal transverse stiffeners, PL100*10 @ 200mm each side, is shown in Fig.5, and the push-over curves of the stiffened and un-stiffened SPSW(B1) and only the frame system have been represented in Fig.6.

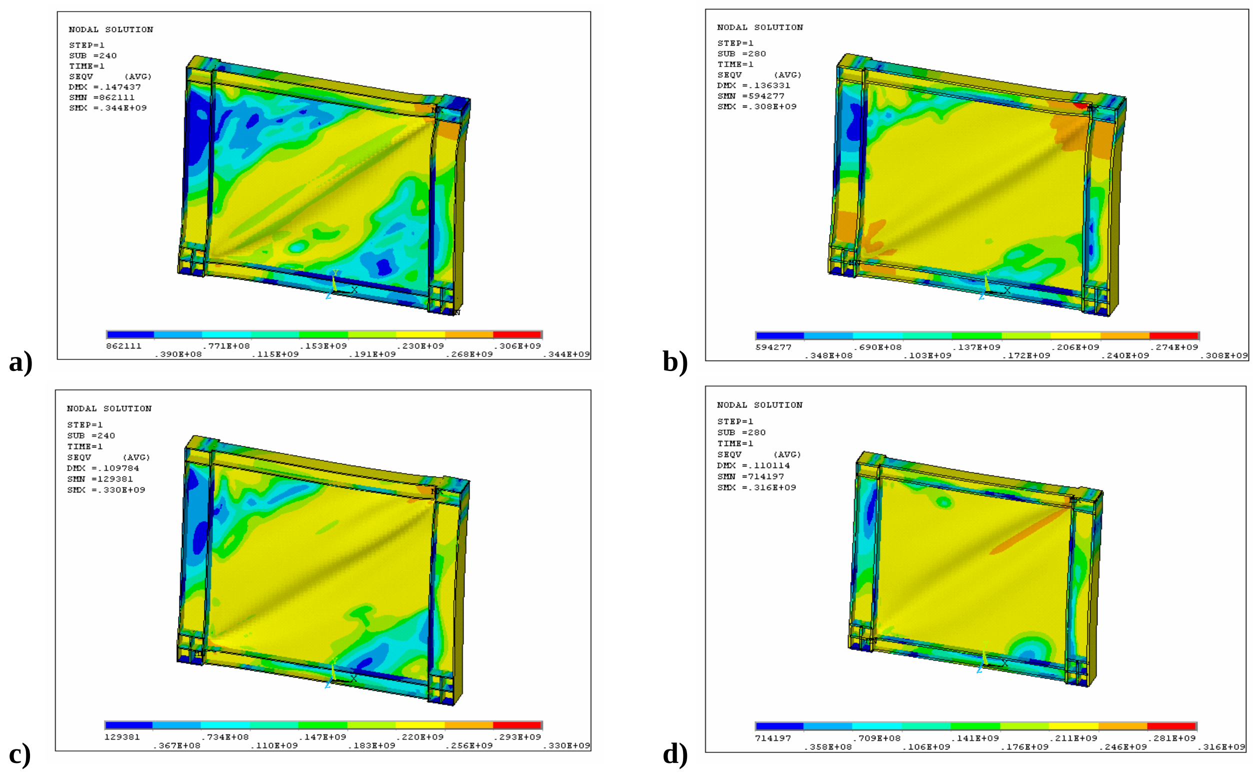
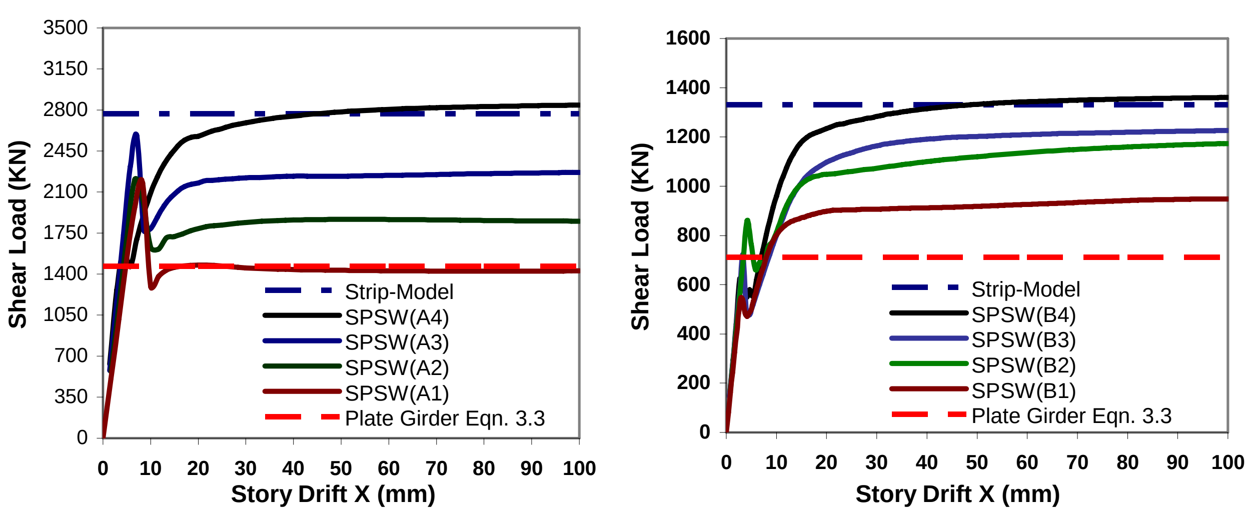


Fig.3. Von-Mises Stresses (Pa.) in 3.3% drift for a) SPSW(A1), b) SPSW(A2), c) SPSW(A3), d)SPSW(A4)



(a) (b)

Fig.4. Push-Over Curves of Steel Plate Shear Walls models for series a) SPSW(A), b) SPSW(B)

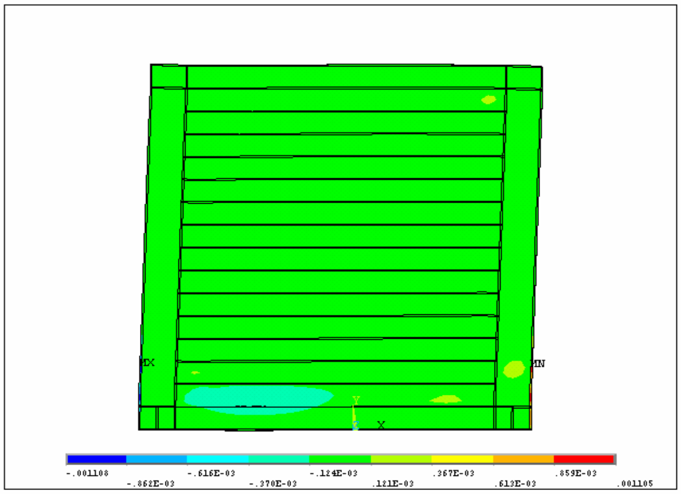


Fig.5. Stiffened SPSW(B1) Out-of-Plane Deflection(m.)

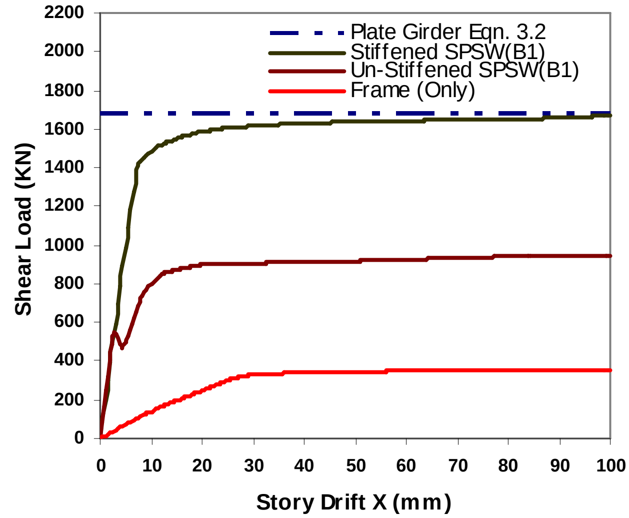


Fig.6. Push-Over Curves for stiffened SPSW(B1), un-stiffened SPSW(B1) and Frame (Only)

As a result, it can be inferred that by stiffening of the web plate with the transverse stiffeners in compliance with the plate girders requirements based on AISC-360-05 the shear buckling has not been occurred in the plate , and the shear wall has reached its full shear plastic capacity as propounded for the stiffened SPSW in the Eqn.3.2.

3. SHEAR STRENGTH CAPACITIES

3.1. Strip Model Method

The ultimate shear strength , V , of an un-stiffened SPSW has been evaluated by means of strip model method as an upper limit, this method assumes that the boundary elements have enough strength for complete tension field action development in the infill steel plate, its history has been briefly presented in the section 1, and the Eqn.3.1 has been concluded from the work of Berman and Bruneau [12] for one-story un-stiffened steel shear wall with the rigid-beam-to-column connections, and the analytical results as follows:

$$V = \frac{1}{2} F_y .b.t.\sin 2\alpha + \frac{2M_{pc}}{h_s} + \frac{2M_{pb}}{h_s} \quad (3.1)$$

where F_y and t, b , h_s, α , M_{pc} and M_{pb} are the tensile yield stress and the thickness of the infill steel plate, the span length, the storey height, the strips inclination with the vertical axis and the plastic moments of the column and beam, respectively. The second term is the frame shear capacity, which can be also estimated by push-over analysis. The calculated shear capacities using this method are represented in Fig. 4a, Fig.4b as an upper limit for the un-stiffened steel shear wall with strong enough boundary elements.

3.2. Plate Girder Method

The Simulation of the SPSW with plate girders has been used by the pioneers' designers of the steel shear walls, Astaneh-Asl [10] categorized the steel shear walls based on their slenderness and AISC-99 provisions for the plate girders, which a similar methodology based on AISC-360-05 has been used here for estimating an upper limit of the ultimate shear strength of the stiffened SPSW and a lower limit for the un-stiffened SPSW with flexible boundary elements. The calculated results for the sample SPSWs are shown in Fig.4a, Fig.4b from Eqn. 3.3 as the lower limit for the flexible boundary members, and from Eqn. 3.2 in Fig.6 as the upper limit for the stiffened steel shear walls based on the plate girders requirements, the equations have been presented as follows:

If $h/t \le 2.24\sqrt{\frac{E}{F_y}}$ (for stiffened SPSW with no tension field action and shear buckling), then

$$V = 0.6F_y.b.t + \frac{4M_{pc}}{h_s} \tag{3.2}$$

And if $h/t \ge 1.37\sqrt{K_V\frac{E}{F_y}}$ (for un-stiffened SPSW with flexible boundary elements), then

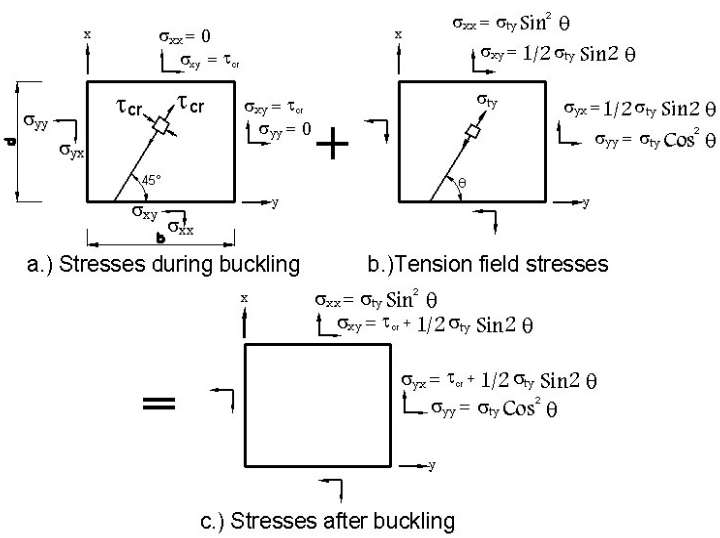
$$V = 0.6F_y.b.t.(C_V + \frac{1-C_V}{1.15\sqrt{1+(a/h)^2}}) \tag{3.3}$$

In which h is the clear distance between the columns or the stiffeners, a is the clear distance between the beams or the stiffeners, and the buckling coefficients are as:

$$C_V = \frac{1.51EK_V}{(h/t)^2 F_y} \quad (3.4) \quad ; \quad K_V = 5 + \frac{5}{(a/h)^2} \quad (3.5)$$

4. BOUNDARY ELEMENTS STRENGHTS

With respect to the boundary elements stiffness and strength to sustain the normal boundary stresses associated with the tension field action some equations were recommended in the literature, and in addition to them the obtained results from this study conduct that the shear strengths and stiffness of the boundary elements have effective influence upon the tension field action development in the infill steel plate, therefore the equations 4.1 and 4.2 have been extracted from the state of stresses in the steel shear walls shown in Fig.7 [9], and verified with the analytical results.



$$A_{bw} \ge \frac{b.t}{1.2}\sin^2\theta \tag{4.1}$$

$$A_{cw} \ge \frac{d.t}{1.2}\cos^2\theta \tag{4.2}$$

Fig.7. State of Stresses in the Steel Plate under Shear Load [9]

Where A_{bw} and A_{cw}, θ and d are beams and columns web areas, inclination of the tension field with the horizontal axis and height of the plate, respectively. The Eqn.4.1 need not be checked for the internal story beams.

5. CONCLUSIONS

The analytical and theoretical results of this study can be summarized as follows:

1-The shear strength capacities of the un-stiffened SPSWs depend on the relative stiffness and strengths of the boundary elements as well as the slenderness of the infill steel plate, wherein the boundary elements have been flexible the shear capacity of steel shear wall has been near to the plate girder result as a lower limit and if they have been strong enough the shear capacity has been near to the strip model method result as an upper limit.
2-The shear strength capacities of the stiffened SPSWs, which have been stiffened based on the plate girders requirements to preclude the shear buckling in the web plate, have been obtained near to the full plastic capacities of the system as an upper limit.
3-There are good agreements between the analytical and theoretical outcomes.
4-The shear strength and stiffness of the boundary elements have been also effective in tension field action development in the infill plate, the relevant equations are proposed for the minimum web cross areas of the boundary elements based on the theoretical and analytical results.

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