Theoretical and Experimental studying on simply supported Steel shear wall

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ABSTRACT :

In this study, the behavior of simply supported steel shear walls under monotonic and also cyclic loading has been investigated. Initially a small specimen was loaded with diagonal direct tensile loading and its behavior compared with theoretical results. Then six specimens with various dimensions went through cyclic loading. The effects of height, width and thickness have been studied and it is concluded that the drift enhanced with increasing height while a small reduction on the shear strength was observed. Comparison of test and theoretical results confirms the validity of the models.

KEYWORDS: Steel Shear Wall, Experimental, Theoretical

1. INTRODUTIN

In the past two decades, considerable research has been conducted on steel shear walls. Due to the analogy of the shear strength in these systems and the plate girders, similar behavior has been used in the study of these systems. Advantages include cost reduction, weight minimization and desirable absorption of the plastic energy reveals the significance of studies on this type of structures. In comparison to the similar systems, we lead to simpler fabrication and erection in the steel shear walls. Also due to stress concentration in those structures, it is not necessary to contract the weld quality.

Post-buckling behavior of the shear panels has been initially studied by Wagner in 1931. He formulated his tension field theory after his through experiments on the thin shear aluminum panels [8]. Since then, several researchers like Kohen, Basler, Rackie, Porter studied tension field of plate girders following Wagner's studies and as a result, the effect of flanges stiffness were considered in the calculation of ultimate strength of panels .

Essentially most studies in the recent 20 years employ the assumption of diagonal tension field development after the steel plate buckling. According to the results of plate girders theory, Kulak et al were the first researchers who proposed the application of thin steel shear walls in University of Alberta on 1989. They concentrated their research on these steel shear walls and after several experimental investigations, suggest replacement of thin web plate by series of diagonal tensile bars.

Elgaaly has studied steel shear walls in general and because of the extreme strain at the end of the corresponding plate, he replaced the plate with virtual strips accompanied a gusset plate at the ends. Then he evaluated the stress and strain in the strip and gusset plate. His computational modeling indicates good agreement with experimental bolted and welded specimens. [3]

Berman and bruneau developed an effective idea on justifying the strip bars[2]. They divided the behavior of steel shear walls into three parts. Then shear strength of each part was calculated and superimposed together. In spite of their innovation, the ultimate strengths of these panels showed partial error in comparison with Sabouri's theoretical relationships. [2]

2. STUDYING OF THEORETICAL BEHAVIOR

As mentioned earlier, the behavior of a plate can be analyzed in three regions where shear strength and

strain can be studied separately:
1.Pre-buckling behavior
2.Elasto – plastic behavior after buckling until panel yields
3.Post – buckling behavior after yielding until rupture stress attained by adopting proper plastic constitute law

2.1. Pre-buckling behavior

In this region, the shear force increases until buckling occurs. Panel behavior is governed by linear plate equation. Margins of this area are slim compared to the others, and it can be ignored if the panel thickness is very small (e.g. less than 1/500) with respect to its other dimensions. Critical stress obtained based on classic plate theory by:

$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\upsilon^2)} \times (\frac{t}{b})^2 \qquad (2.1)$$

Where k is defined as:

$k = 5.35 + 4(\frac{b}{d})^2$ for b/d > 1 $\quad k = 5.35(\frac{b}{d})^2 + 4$ for b/d <1

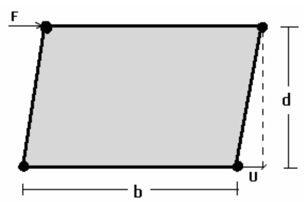

Figure 1: steel shear wall panel

In the above relations b, d, t are width, height and thickness of the plate respectively (See figure1)

2.2. Elasto-plastic behavior after buckling until panel yields

In this phase, which starts after buckling and continues to yield stress of the plate, we can replace the plate by diagonal strips with angle of 450 [2] as shown in figure 2.

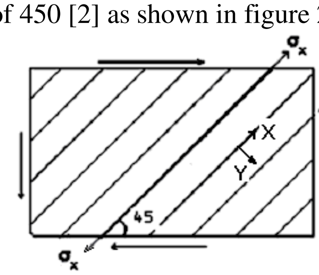

Figure 2: Replacing the plate with diagonal strips

According to Elgaaly's tests, the strain distribution on a strip element is not constant and can be varied as shown in figure 3: [3]

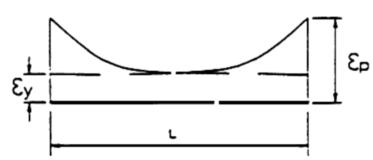

Figure 3: The strain distribution on the strip elements

$$\varepsilon_p = \alpha\varepsilon_y \qquad (2.2)$$

$$\Delta_y = \varepsilon_y L + (\varepsilon_p - \varepsilon_y)\frac{L}{3} = (2+\alpha)\varepsilon_y \frac{L}{3} \qquad (2.3)$$

$$\beta = \frac{2+\alpha}{3} \qquad (2.4)$$

$$\varepsilon = \frac{\beta\sigma_e}{E} \qquad (2.5)$$

Deformation coefficient has been represented by α, which varies between 5 and 20. When the panel thickness is small and boundary elements have sufficient rigidity, α is about 20. On the other hand, when the panel is thick and boundary elements are flexible, the coefficient α will reduce to 5. Hence:

$$5 \le \alpha \le 20 \qquad\qquad 2.32 \le \beta \le 7.33$$

Now, defining X axis on the strip direction, the strain distribution on complete yielding of strip will be:

$$\varepsilon_{xe} = \frac{(1+\upsilon)}{E} \times \tau_{cr} + \frac{\beta\sigma_e}{E} \qquad (2.6)$$

$$\varepsilon_{ye} = -\frac{(1+\upsilon)}{E} \times \tau_{cr} - \upsilon' \frac{\beta\sigma_e}{E} \qquad (2.7)$$

Where $\upsilon' \cong 0.5$ is the plastic passion's ratio and σ_e is the yield stress of plate which can be obtained by von-misses criteria as:

$$\sigma_e \cong F_y - \sqrt{3} \times \tau_{cr} \qquad (2.8)$$

Transforming these strains to the principle direction, the shear strain when the panel reaches to yield stress is:

$$\gamma_E = \frac{2(1+\upsilon)}{E} \times \tau_{cr} + (1+\upsilon')(\frac{\beta\sigma_e}{E}) \qquad (2.9)$$

Substituting $G = \frac{E}{2(1+\upsilon)}$, we can calculate panel drift as:

$$U_e = \left(\frac{\tau_{cr}}{G} + (1+\upsilon')(\frac{\beta\sigma_e}{E})\right) \times d \qquad (2.10)$$

When the plate reaches to its yield stress, the total shear strength can be obtained using equilibrium method as:

$$F_e = (\tau_{cr} + 1/2\sigma_e k \ \sin 2\theta) bt \qquad (2.11)$$

Where k is the modified coefficient of shear strength and varies as a function of aspect ratio (b/h). It means that for different heights, the formula calculates several shear strength. Figure 4 shows that by increasing the height of panel, its strength is reduced. This figure is also verified during this investigation.

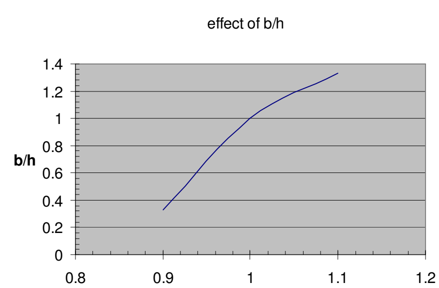

Figure 4: Effect of b/h on the strength of panel

The angle θ can be assumed by the code of Canada (CAN/CSA-S16.1-94) as follows:

$$\tan^4\theta = \frac{1+\frac{tl}{2A_c}}{1+th_s(\frac{1}{A_b}+\frac{h_s^3}{360I_cL})} \tag{2.12}$$

Where Ac is cross section of the column and I_c its moment of inertia. Also h_s is the story height and A_b is the cross section of beam. It must be noted that the error of θ in the calculation of ultimate strength is negligible. (Less than 2%)

2.3. Plastic behavior after yielding until rupture stress

We must note that the concepts of elasticity are not true in this region but we can assume the plate as the strip elements. The only difference in this formulation is replacement of the elastic module which will reduce. So, by assuming constant value for strip section during the formulation, we use Et instead of E. Because of very small value of the thin plate, it can be justified. Representing σ_p for the stress value in the beginning of the plastic region until its ultimate value, the ultimate stress and strain on X and Y direction are:

$$\sigma_u = \sigma_e + \sigma_p \tag{2.13}$$

$$\varepsilon_{xp} = \frac{(1+\upsilon)}{E}\times\tau_{cr} + \frac{\beta\sigma_e}{E} + \frac{\sigma_p}{E_t} \tag{2.14}$$

$$\varepsilon_{yp} = -\frac{(1+\upsilon)}{E}\times\tau_{cr} - \upsilon'\frac{\beta\sigma_e}{E} - \upsilon'\frac{\sigma_p}{E_t} \tag{2.15}$$

In which, the negative sign indicates reduction of the strain. When the panel reaches its ultimate stress, the shear strain and drift are as follow:

$$\gamma_P = \frac{2(1+\upsilon)}{E}\times\tau_{cr} + (1+\upsilon')\left(\frac{\beta\sigma_e}{E} + \frac{\sigma_p}{E_t}\right) \tag{2.16}$$

$$U_p = \left(\frac{\tau_{cr}}{G} + (1+\upsilon')(\frac{\beta\sigma_e}{E} + \frac{\sigma_p}{E_t})\right)\times d \tag{2.17}$$

In this case the panel strength is also:

$$F_p = (\tau_{cr} + 1/2\sigma_u \sin 2\theta)bt \tag{2.18}$$

3.COMPARING OF THEORITICALCONCEPS AND TEST RESULTS

Seven specimens of simply supported steel shear walls were fabricated and tested under monotonic and cyclic loading. The results were broadly agreed with theoretical concepts and were described in separated sections.

3.1. A steel shear wall under monotonic loads

Dimension of this specimen were 55×30×0.06 (all in centimeter). It was tested under diagonal tensile test until failure. Table 1 shows its characteristics:

Table 3.1 Specimen characters

Name	width	length	thickness	Fy kg/cm²	Fu kg/cm²	E	E_t
Test 1	55	30	0.06	2209	3306	2.1E6	63045

Using relations (12) and (19), we have:

$$\gamma = (1+\upsilon')\beta(\frac{\sigma_e}{E}) = (1.5)(7.33)(\frac{2209}{E}) = 0.01156$$

$$\Delta Ue = \frac{0.01156\times 300}{\cos 28.62} = 3.95mm$$

$$\Delta Up = \langle\frac{\tau_{cr}}{G} + (1+\upsilon')(\frac{\beta\sigma_e}{E} + \frac{\sigma_P}{E_t})\rangle\frac{d}{\cos 28.62} = 12.87mm$$

$$F_e = \left(\tau_{cr}bt + 1/2(\sigma_y - \sqrt{3}\tau_{cr})bt\right)\frac{\sin 70}{\cos 28.62}$$

$$=\left(49.62\times55\times0.06+1/2(2209-\sqrt{3}\times49.62)\times55\times0.06\right)\frac{\sin 70}{\cos 28.62}=3913.7kgf$$

From the test results, k is about 1.076. Figure 5 shows test mechanism and the specimen after failure has been shown in figure6.

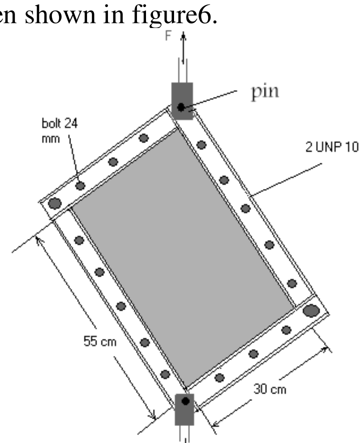

Figure5: Schematic of the tests

Figure 6: the specimen after loading

Figure 7 shows comparison of the experimental and theoretical results.

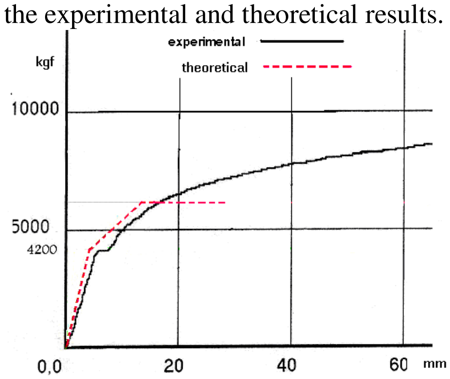

Figure7: comparing of test and theoretical results

3.2. Six steel plate shear walls under cyclic loading

In this test, six steel shear walls went through cyclic loading and their seismic response were investigated for various aspect ratio and thickness values. Characteristics of these specimens are summarized in table 2.

Cyclic loading were applied using hydraulic jacks to a couple of steel welded brackets on the top of story beam. Figure 8 show schematic of test setup.

Table3. 2: specifications of specimens

Name	B(cm)	H	t(cm)	F_Y(kg/cm²)	τ_{cr} (kg/cm²)	Fe kgf	Ue(mm)	error (mm)
307	92	92	0.07	2663	10.27	8583	17.5	2.5
308	92	92	0.1	2283	20	10524	14.1	0.1
309	92	142	0.07	2663	7. 34	7836	24.78	0.2
310	92	142	0.1	2283	15	9619	22	1.9
311	142	92	0.07	2663	6.55	13894	17.5	0
312	142	92	0.1	2283	13/4	17031	14.2	0

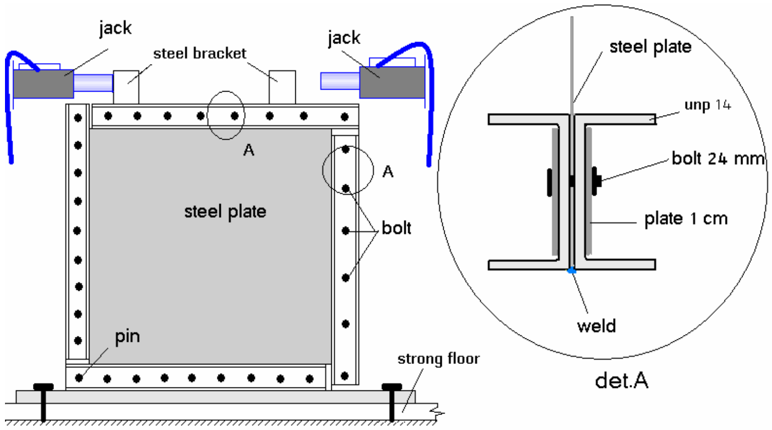

Figure8: test elevation

Figure9, shows coincidence of the theoretical relationships and experimental hysteresis loops for four specimens referred as 309, 312, 308, 310. For the rest of specimens, good agreements have also been observed.

Using above relationships, we can predict the behavior of steel shear walls under monotonic or cyclic loadings. Because of the strain hardening, hysteresis loops gradually come apart of the theoretical line as a result of steel plate behavior.

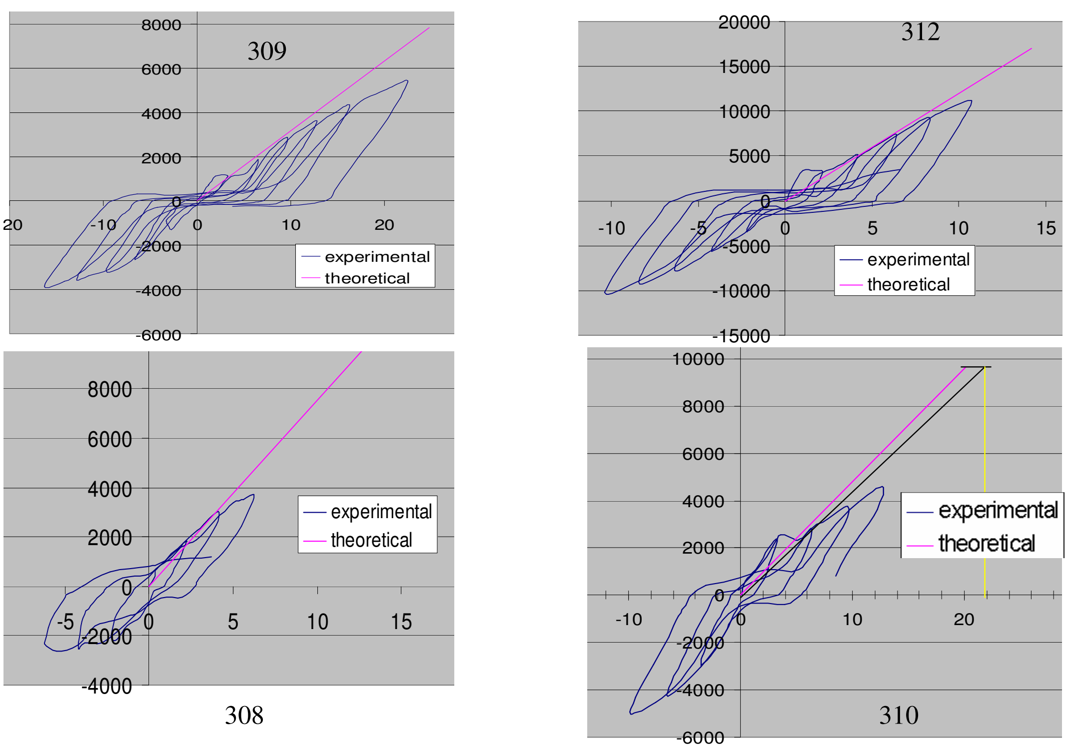

Figure9: comparison of hysteresis loops and theoretic relation ships

Figure 10 shows a specimen under test which illustrates the application of lateral loads on the steel brackets using hydraulic jacks. Buckling waves are clearly shown in the figure. In addition, out of plane bracing and fixing of the specimen are also demonstrated in the figure.

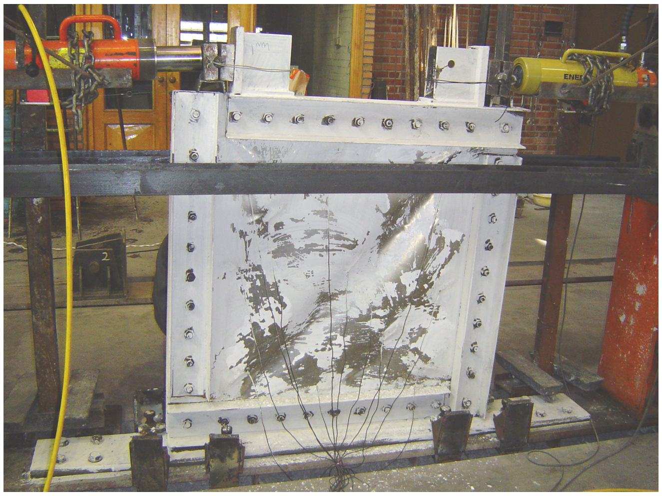
Figure10: view of the test

3.3.Effect of height on the shear strength

Generally, it seems that with height reduction in constant width, the stiffness increased after post-buckling. The reason is effective length reduction in the parallel bars. In such case, the beam experiences a little drop and can operate as a stiffener as shown in figure 11.

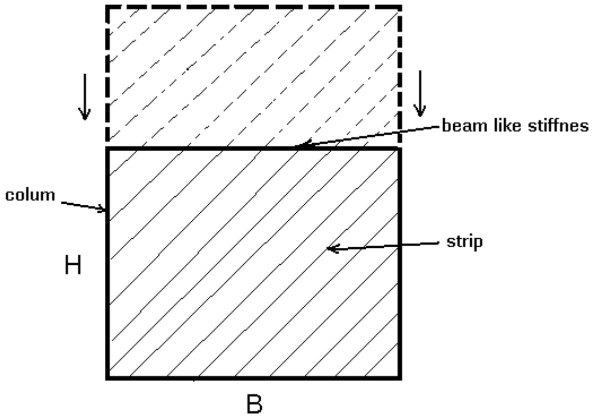

Figure 11: Beam drop as a stiffener for the panel

As shown in figure 11, using the strip model of steel shear wall, reduction of height reduces the length of strips. Hence it can function as a stiffener on the panel. Clearly, introduction of a stiffener on the panel enhances its shear strength. This concept is observed in eq. (2.11).

In figure 12 we can see the shear strength of two panels with equal width but different height. As shown in the figure, height reduction results in the enhancement of shear strength and also reduction of drift which can be resulted by eq. (2.10).

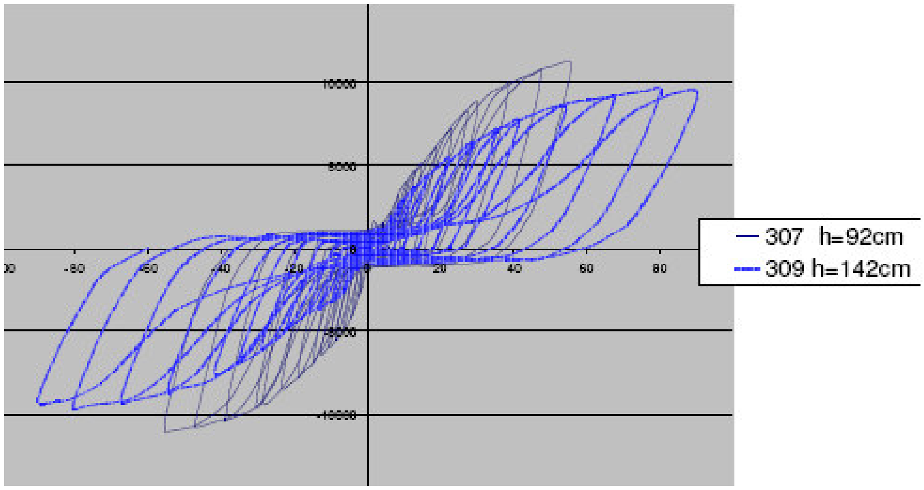

Figure 12: The effect of height on shear strength

4.CONCLUSION

1. Our studies modified displacement- shear strength relationships of simply supported steel shear walls.
2. Comparison of theoretical relationships and experimental results under monotonic and cyclic loadings shows a good prediction of the behavior of steel shear wall systems.
3.All specimens reached to the ultimate strength on relative drift of 1.7%~2% and failed on 5%
4.Reduction of height of shear panel reaches to reducing the drift and enhancement of shear strength
5. Long height of the panel results in the enhancement of panel drift and absorbs a significant plastic energy and it leads to small reduction on shear strength.
6.Enhancement of width reaches to significant increasing on shear strength of panel and drift reduction

REFRENCE

1- Astaneh-Asl, A. (2000) “Steel plate shear walls” .US. -Japan workshop on seismic fracture issues in steel structure February. San Francisco.,

2- Berman.J, M.ASCE, and Bruneau.M, M.ASCE(2003) “Plastic Analysis and Design of Steel Plate Shear Walls”. Journal of Structural Engineering, 129(11), November 1

3- Elgaaly.M (1998) “Thin steel plate shear walls behavior and analysis” Thin-Walled Structures 32 pp 151–180

4- Sabouri-Ghomi, S., and Roberts, T. M. (1991)"Nonlinear dynamic analyses of thin steel plate shear walls." Compo and Struct., 39(112), pp 121-127,

5- Sabouri-Ghomi.S; Ventura.C, M.ASCE; and Mehdi H. K. Kharrazi, (2005) “Shear Analysis and Design of Ductile Steel Plate Walls” Journal of structural engineering ASCE / JUNE

6-Takahashi, Y.. Takemoto, y" Takeda, T., and Takagi, M.. "Experimental study on thin steel shear walls and particular bracings under alternative horizontal load." Preliminary Rep., ABSE Symp. on Resistance and Ultimate Deformability of Struct. Acted on by Well-Defined Repeated Loads, International Association

7- Tyan Chen.S , Jeng.V, (2001) “Seismic Assessment and Strengthening Method of Existing RC Buildings in Response to Code Revision” Earthquake Engineering and Engineering Seismology 67, 3(1), pp 67–77, March

8-Wagner.H and Ballerstedt.W (1935) “Tension fields in originally curved thin sheets during Shearing stresses” NACA