Experimental and Theoretical Investigation of Simply Supported Steel Shear Walls

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Abstract: In this work, our purpose is investigating the behavior of simply supported steel shear walls under the monotonic and cyclic loading. First, we applied diagonal direct tensile loading on a small specimen and its behavior was compared with theoretical results. After that, we applied cyclic loading on six specimens with various dimensions and the effects of thickness, height and width have been investigated. Results have shown that, with increasing height, the drift was enhanced but a small reduction on shear strength was observed. The comparison of our results with theoretical results confirmed the validity of model.
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1. Introduction

In the recent years, considerable amount of researches have been investigated the steel shear walls. Because of the similarity of shear strength in these systems and plate girders, a similar method has been used in to investigate these systems. Benefits such as cost reduction, weight minimization and desirable absorption of the plastic energy indicate the importance of studies on these structures. Our method results in a simpler fabrication and erection of the steel shear walls compared the similar methods. Wagner (1931) first studied post-buckling behavior of the shear panels. He formulated the tension field theory by conducting experiments on the thin shear aluminum panels (Wagner H. and Ballerstedt W., 1935). Later, tension field of plate girders was investigated by researchers such as Kohen, Basler, Rackie, and Porter following Wagner's works. By these efforts, the effect of flanges stiffness was included in the calculation of ultimate strength of panels.

Generally, in the past two decades, most studies adopted the development of diagonal tension field assumption after steel plate buckling. Concerning the results of plate girders theory, Kulak et al (1989) in the University of Alberta, Canada first proposed the use of thin steel shear walls. They were focused on steel shear walls and finally proposed the replacement of thin web plates by series of diagonal tensile bars. Elgaaly (1998) studied steel shear walls and due to the very high strain at the end of the corresponding plate, replaced the plate with virtual strips as well as a gusset plate at the ends and examined the stress and strain in the strip and gusset plate. The computational modeling introduced by him showed a good agreement with the experimentally bolted and welded specimens (Elgaaly, 1998).

Berman and Bruneau (2003) have established a well-formulated method for justifying the strip bars. They considered behavior of steel shear walls in three divisions. Then the shear strength of each part was calculated and laid over together. Although their method was innovative, the ultimate strengths of these panels, compared to Sabouri's theoretical relationships, had a partial error (Berman and Bruneau, 2003).

2. Material and Methods

As mentioned above, to investigate the behavior of a plate, we can divide this behavior to three sections, as follows, and examine shear strength and strain separately:

- Pre-buckling behavior;
- Elastic-plastic behavior after buckling until the panel yields and;
- Post-buckling behavior after yielding until reaching rupture stress

2.1. Pre-buckling behavior

In this section, the sheer force increases to initiate buckling. As we know, linear plate equation governs the panel behavior. Compared to the other parts, margins of this area are thin and could be ignored if the panel thickness, concerning the other dimensions, is very small (e. g. less than 1/500). Based on classic plate theory, the critical stress is:

$$\tau = \frac{k\pi^2 E}{12(1-\nu^2)} \times \left(\frac{t}{b}\right)^2 \qquad (1)$$

Where is:

$k = 5.35 + 4\,\frac{b^2}{d}$ (For b/d > 1) and is $k = 5.35 + 4\,\frac{b^2}{d} + 4$ (For b/d < 1)

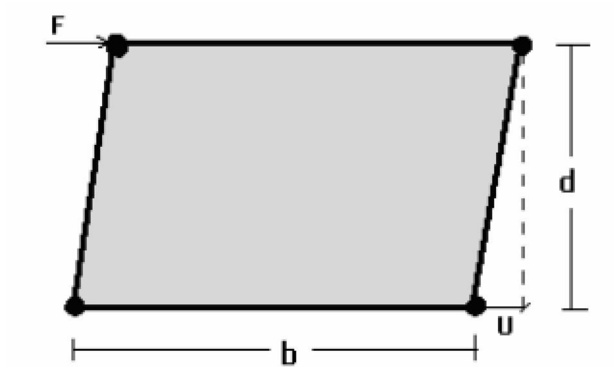

Figure -1: panel of steel shear wall

In the above relations, b, d, and t stand for width, height and thickness of the plate, respectively (Figure1).

2.2. Elastic-plastic behavior after buckling until panel yields

This phase starts after buckling and continues to yield stress of the plate. In this phase, we can replace the plate by diagonal strips with angle of 450 [Berman and Bruneau, 2003) as shown in Figure 2.

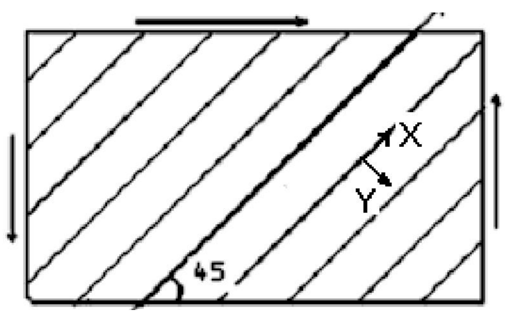

Figure 2: Replacing the plate with diagonal strips

According to Elgaaly (1998),distribution of strain on a strip element is not constant and can be varied as shown in figure 3.

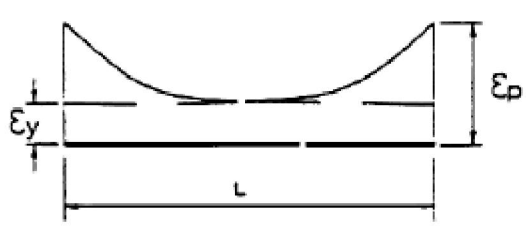

Figure 3: The strain distribution on the strip elements

$$\varepsilon_p = \alpha\varepsilon_y \quad (2)$$

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

$$\beta = \frac{2+\alpha}{3} \quad (4)$$

$$\varepsilon = \frac{\beta\sigma_e}{E} \quad (5)$$

Deformation coefficient is represented by α, which ranges from 5 to 20. In the case that, the panel is thin and boundary elements have appropriate rigidity, α is about 20. In addition, when the panel is thick and boundary elements are flexible, α coefficient will be reduced to 5. Hence: 5≤ α ≤ 20 and 2. 32≤ β ≤ 7. 33

Now, by defining X-axis on the strip direction, the strain distribution on complete yielding of strip is:

$$\varepsilon_{Xe} = \frac{(1+\nu)}{E} \times \tau_{cr} \times \frac{\beta\sigma_e}{E} \quad (6)$$

$$\varepsilon_{Ye} = -\frac{(1+\nu)}{E} \times \tau_{cr} - \nu' \frac{\beta\sigma_e}{E} \quad (7)$$

Where ν ≡ 0. 5 is the Poisson's ratio of plastic and σ_e is the yield stress of plate, which can be obtained the criteria of von-misses:

$$\sigma_e \equiv F_y - \sqrt{3} \times \tau_{cr} \quad (8)$$

By transforming these strains to the principle direction, the shear strain at yield stress is equal to:

$$\gamma_E = \frac{2(1+\nu)}{E} \times \tau_{cr} + (1+\nu')\frac{\beta\sigma_e}{E} \quad (9)$$

By substituting $G = \frac{E}{2(1+\nu)}$,the panel drift can be calculated as:

$$U_e = \left(\frac{\tau_{cr}}{G} + (1+\nu)(\frac{\beta\sigma_e}{E})\right) \times d \quad (10)$$

When a plate reaches its yield stress, the equilibrium method can be used to calculate total shear strength:

$$F_e = (\tau_{cr} + \frac{1}{2}\sigma_e K \sin 2\theta)bt \quad (11)$$

Where, k is modified coefficient of shear strength and varies as a function of aspect ratio (b/h) indicating that, for different heights, the formula gives several shear strengths. By increasing the height of panel its strength will be reduced, as shown in Figure 4.

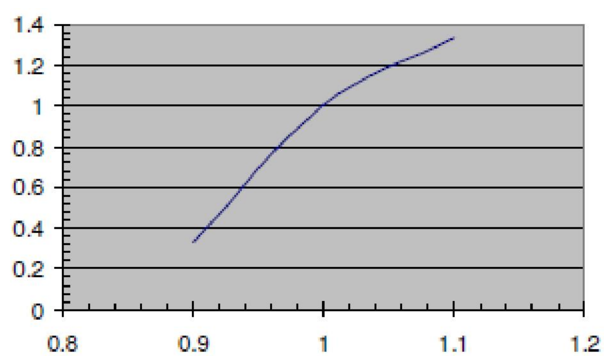

Figure 4: Effect of b/h on the strength of panel

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

$$\tan^4\theta = \frac{1+\frac{tl}{2A_c}}{1+th_s\left(\frac{1}{A_b}+\frac{h_s^3}{360I_cL}\right)} \qquad (12)$$

Where A_c, is cross-sectional area of the column and, Ic, is moment of inertia. In addition, h_s, is the story height and A_b is the cross-sectional are of the beam. It is worth mentioning that, the error of θ in calculating ultimate strength is less than 2% and can be neglected.

2.3. Post-buckling behavior after yielding until reaching rupture stress

We should consider that, the elasticity is not true in this region but we may assume plate as the strip elements. The only difference in this formula will be the replacement of the elastic module, which will be reduced. Therefore, during the formulation, by adopting constant value for strip section, we can use, E_t, instead of E. Due to the very small value for thin plate, it can be justified. By representing σ_p for the stress, from the beginning of the plastic region until its ultimate value, the ultimate stresses and strains on X and Y directions are:

$$\sigma_u = \sigma_e + \sigma_p \qquad (13)$$

$$\varepsilon_{xp} = \frac{(1+\nu)}{E}\times\tau_{cr} + \frac{\beta\sigma_e}{E} + \frac{\sigma_e}{E_t} \qquad (14)$$

$$\varepsilon_{yp} = -\frac{(1+\nu)}{E}\times\tau_{cr} - \nu'\frac{\beta\sigma_e}{E} - \nu'\frac{\sigma_p}{E_t} \qquad (15)$$

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

$$\gamma_p = \frac{2(1+\nu)}{E}\times\tau_{cr} + (1+\nu')\left(\frac{\beta\sigma_e}{E}+\frac{\sigma_p}{E_t}\right) \qquad (16)$$

$$U_p = \left(\frac{\tau_{cr}}{G} + (1+\nu')\left(\frac{\beta\sigma_e}{E}+\frac{\sigma_p}{E_t}\right)\right)\times d \qquad (17)$$

In addition, the panel strength is equal to:

$$F_p = (\tau_{cr} + \tfrac{1}{2}\sigma_u \sin 2\theta)bt \qquad (18)$$

3. Results

For experimental tests, seven specimens of simply supported steel shear walls were fabricated and subjected to monotonic and cyclic loading. The results were in agreement with theories.

3.1. Steel shear walls under the monotonic loading

The dimensions of this specimen were 55×30×0. 06 cm. Diagonal tensile loading was applied on this sample until failure. Table 1 shows the characteristics of this sample.

Table 1: Specimen characters

Name	Width	Length	Thickness	E_t
Test 1	55	30	0. 06	63045

Using relations (12) and (19):

According to our results, k is 1. 076. Figure 5 shows the schematics of the experimental setup and Figure 6 shows the specimen after failure.

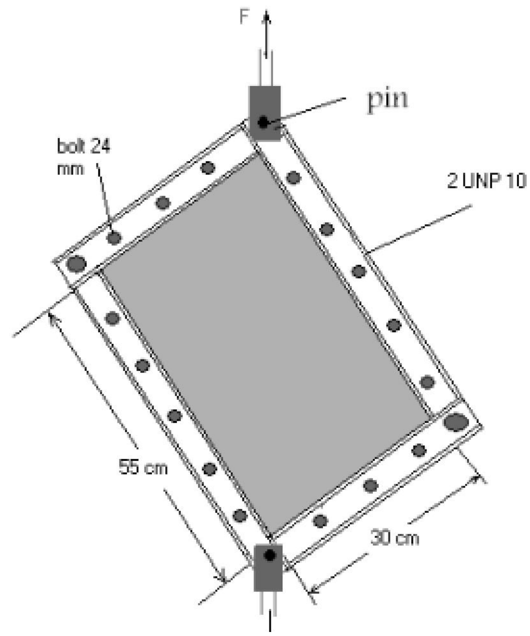

Figure 5: Schematic of the experimental setup

Figure 6 shows the comparison of the experimental and theoretical results.

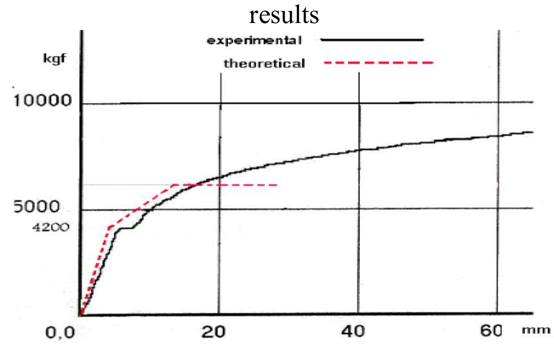

Figure 6: comparison of experimental and theoretical

3.2. Applying cyclic loading on six steel plate shear walls

In this section, cyclic loading was applied to six samples of steel shear walls and their seismic responses, for various aspect ratios and thicknesses, were investigated. Table 2 shows the characteristics of these specimens.

Table 2: specifications of samples

Name	B(cm)	H	T(cm)	Fy (kg/cm^2)
307	92	92	0. 07	2663
308	92	92	0. 1	2283
309	92	142	0. 07	2663
310	92	142	0. 1	2283
311	142	92	0. 07	2663
312	142	92	0. 1	2283

Hydraulic jacks were used to provide cyclic loading on a couple of steel welded brackets on the top of story beam. Schematic of experimental setup is shown in Figure 7.

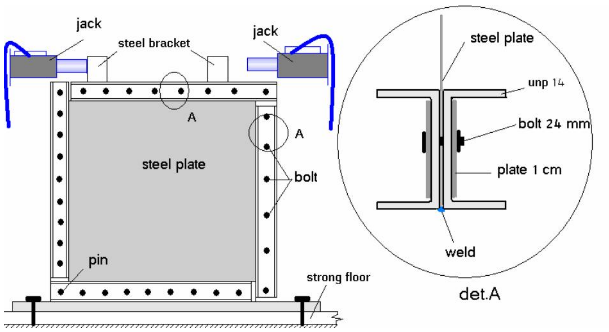

Figure 7: experimental elevation

Figure 7 indicates that, there is a full agreement between theoretical relationships and experimental hysteresis loops for four samples namely,309,312,308,and 310; the results of the rest samples showed good agreements with theories.

The above relationships help us to predict the behavior of steel shear walls under the monotonic or cyclic loading. Hysteresis loops gradually fall apart of the theoretical results due to the strain hardening resulted from steel plate behavior.

Figure 10 shows a specimen under the application of lateral loads on the steel brackets provided by hydraulic jacks. Moreover, this figure shows out of plane bracing and fixing of the samples.

3.3. Impact of height on the shear strength

In general, it appears that, reducing height while width is constant, the stiffness will be increased after post-buckling. This is the result of the reduction of effective length in the parallel bars. In this situation, the beam experiences a little drop and operates as a stiffener (Figure 9).

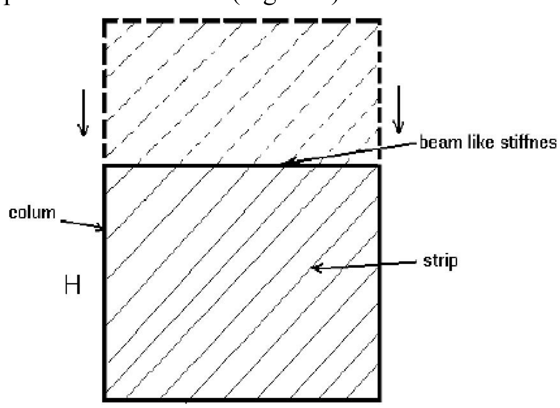

Figure 8. The impact of height on the shear strength

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Figure 8 shows that, via using the strip model for steel shear wall, reduction of height will reduce the length of the strips, therefore, it can be considered as a stiffener for the panel. It is obvious that, introducing a stiffener on the panel enhances its shear strength. Eq. 11 shows this idea.

Figure 9 depicts the shear strength of two panels with identical widths but different heights. As it can be seen, reducing height results in enhancement of shear strength and a drift. Eq. 10 shows this idea.

4. Discussions

- This work modified displacement- shear strength relationships for simply supported steel shear walls.
- A good prediction for the behavior of steel shear wall systems can be derived from comparing theoretical relationships and experimental results under monotonic and cyclic loadings.
- All samples reached ultimate strength at relative drift of 1. 7% to 2% and failed at 5%.
- Reducing the height of shear panel reduces drift and improves shear strength.
- Increasing the height of the panel enhances panel drift and through which a significant amount of plastic energy will be absorbed shear strength will be relatively reduced.
- Increasing width results in significant increase of shear strength of the panel and reducing the drift.

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