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Dear Mr. Iman TAHERIAN

I am pleased to inform you that your paper entitled "ANALYTICAL STUDY ON COMPOSITE STEEL PLATE WALLS USING A MODIFIED STRIP MODEL " written by Iman TAHERIAN, Mansour GHALEHNOVI and Hashem JAHANGIR has been accepted to be presented in the 7th international conference of seismology and earthquake engineering (see7).

Thank you for your contribution and on behalf of the Conference committee, I look forward to see you in Tehran during the conference time (18-21 May, 2015).

Best regards

Kambod Amini Hosseini

Associate Professor and SEE7 Co-Chair
Risk Management Research Center
International Institute of Earthquake Engineering and Seismology (IIEES) Tehran / Iran

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Conference on
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ANALYTICAL STUDY ON COMPOSITE STEEL PLATE WALLS USING A MODIFIED STRIP MODEL

Iman TAHERIAN
M.Sc. Student of structural engineering, Ferdowsi University of Mashh Mashhad, Iran

E-mail address: imantaherian@yahoo.com

Mansour GHALEHNOVI
Associate Professor of civil engineering, Ferdowsi university of Mashhad, Mashhad, Iran
E-mail address: ghalehnovi@um.ac.ir

Hashem JAHANGIR
Ph.D. Student of structural engineering, Ferdowsi University of Mashhad, Mashhad, Iran
E-mail address: jahangir.farhad@gmail.com

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ABSTRACT

A composite steel plate shear wall (CSPSW) system consists of a steel plate shear wall with reinforced concrete panels attached to one side or both sides by bolts or connectors. This arrangement restrains the possible occurrence of out-of-plane buckling of thin-walled steel plate, thus significantly increasing the load-carrying capacity and ductility of the overall wall. In this paper, a new analytical model for the CSPSW-Strip Model was proposed based upon the mechanism and failure mode of CSPSW. The cross sectional properties and hysteretic model for the cross strips in the model were determined with theoretical analysis. Comparison with experimental results showed that the proposed model was able to capture accurately nonlinear behavior of CSPSW under monotonic and cyclic loading.

1. INTRODUCTION

Shear walls have been widely used as lateral load resisting system in concrete buildings in the past, especially in high-rise buildings. In steel buildings, in many cases concrete shear walls are used with a boundary steel frame to resist seismic effects. However, there are several disadvantages for using a concrete shear wall in this case. The most important one is the development of tension cracks and localized compressive crushing during large cyclic displacement, which can result in spalling and splitting failure of the wall and lead to serious deterioration of stiffness and reduction in strength. Also, reinforced concrete shear walls used in tall buildings tend to develop relatively large shear forces during seismic events due to their relatively large lateral stiffness, but the high weight to strength ratio of concrete material will make the use of reinforced concrete shear walls impractical for this case. In addition, the casting and curing of reinforced concrete walls in a steel building makes the construction not so efficient compared to other systems such as braced frames or moment frames.

In recent years, steel plate shear wall has been used in a number of buildings and achieved satisfactory results regarding construction efficiency and economy. However, overall buckling of the steel plate shear wall will result in reduction of shear strength, stiffness and energy dissipation capacity of the whole system (Zhao and Astaneh-Asl, 2004). It could be prevented by adding stiffeners to the steel plate, which, however, will result in additional fabrication costs (Astaneh-Asl, 2001). In addition, in structures with steel shear walls, due to relatively large inelastic deformations of the panel, the connections of the boundary frame can undergo relatively large cyclic rotations as well as somewhat larger inter-story drifts (Allen and Bulson, 1980).

Composite shear walls, on the other hand, might combine the advantages of the reinforced concrete shear wall and steel shear wall together and promote the usage of shear wall systems in steel buildings. The composite shear walls have been used in a few modern buildings in recent years (Dean et al., 1977), but not as frequently as the other lateral load resisting systems. Seismic behavior of this system and the design guidelines are therefore of high interest.

2. COMPOSITE SHEAR WALL SYSTEMS

Two configurations of composite shear wall system denoted as “traditional” and “innovative” have been introduced. The innovative system was proposed and designed by Astaneh (2002). The difference between the two configurations is that in the traditional system the reinforced concrete wall is in direct contact with the steel boundary frame but in the innovative system there is a gap, as shown in Figure 1. The gap can be left empty or filled with soft material. If one desires to add to energy dissipation capacity of the structure at additional cost, the gap can be filled with visco-elastic material (Zhao and Astaneh-Asl, 2003).

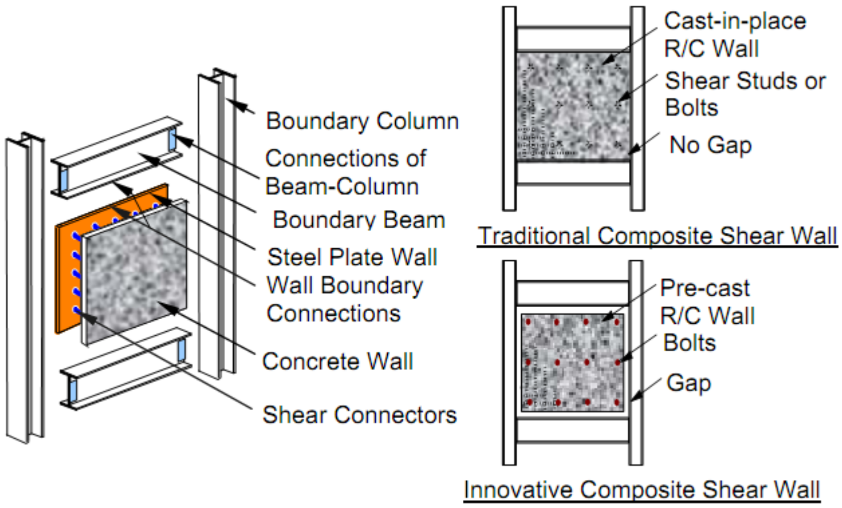

Figure 1. main components of composite shear wall (Zhao and Astaneh-Asl, 2003)

It is anticipated that the existence of the gap in the innovative composite shear wall system will result in reduction of the system lateral stiffness and change of the reinforced concrete wall behavior under severer seismic events. In the traditional system, both steel and concrete walls are active and provide stiffness and strength from beginning of loading. As a result, not only large forces can be attracted to the structure due to relatively large stiffness of the combined system, but the reinforced concrete wall could be damaged under relatively small lateral displacement. In the innovative system, however, due to the existence of the gap, the concrete wall will not get involved in resisting lateral loads until the inter-story drift has reached certain value, as shown in Figure 2 (Zhao and Astaneh-Asl, 2003).

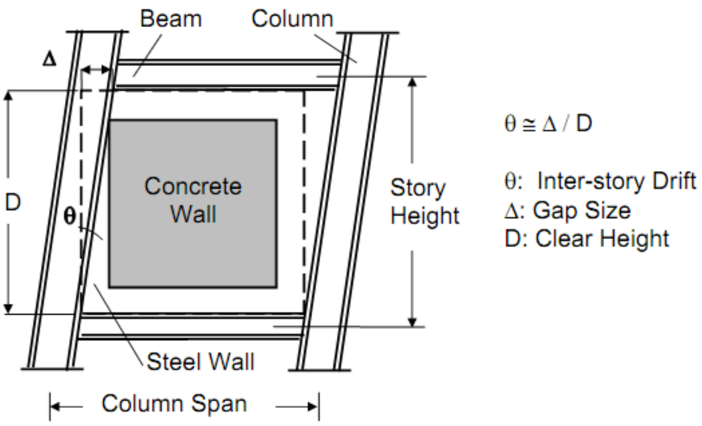

Figure 2. function of gap in the innovative composite shear wall system (Zhao and Astaneh-Asl, 2003)

When the drift is under the specified value, only steel shear wall and the boundary moment frame provide strength, stiffness and ductility, and the role of reinforced concrete wall is to provide out-of-plane bracing for the steel plate. When the drift is over the specified value, the gap is closed at corners and both steel and concrete walls become active and provide strength, stiffness and ductility. Then the participation of reinforced concrete wall brings in the much needed extra stiffness to help reduce the drift and P-Δ effects, compensates for loss of stiffness of steel shear wall due to yielding, and helps in preventing lateral creep and collapse failure of the structure due to P-Δ effects (Zhao and Astaneh-Asl, 2003).

Comparing steel plate wall (SPW) and composite steel plate wall (CSPW), as indicated in Figure 3, SPW will buckle under very low compressive stress and hence resist lateral force by means of diagonal tensile action. In the case of CSPW, the steel plate will develop pure shear stress if the out-of-plane restraint provided by the concrete panel is ideal and sufficient. However, according to experimental results (Gao, 2007), the steel plates in CSPW exhibited diagonal tensile action similar to that of thin SPWs. This is due to the gaps between the concrete panel and the boundary members, where the steel plates do not have out-of-plane restraint. Thus the diagonal residual deformations occurred in this region. Based upon the similarity between the mechanisms of the CSPW and the SPW, the strip model (Driver et al., 1998) suitable for the latter case will be extended to the former one.


Figure 3. stress status in thin SPW and CSPW

In the strip model for SPW as shown in Figure 4, a group of parallel strips are employed to represent the tensile diagonal action, while in the compressive diagonal direction, no components are present as no significant compressive stress can exist due to buckling. In comparison, a large amount of compressive stresses will develop in the steel plate of a CSPW resulting from the protection of the concrete panel. Thus, a second group of parallel strips in the compressive diagonal direction will be adopted for CSPW, in addition to the tensile strips. This introduces a model named as Modified Strip Model shown in Figure 4.

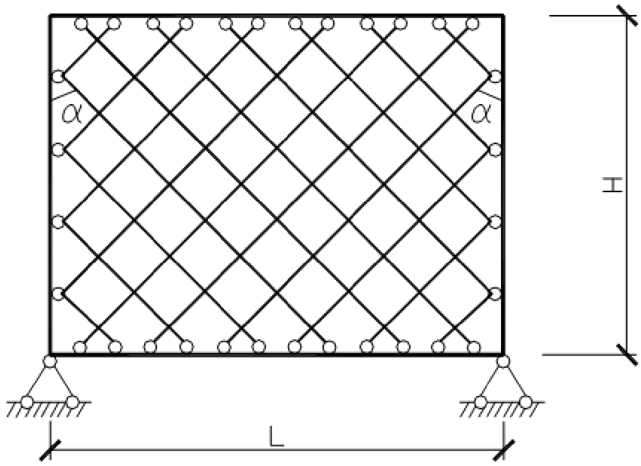

Figure 4. Modified Strip model for CSPW

3. MODIFIED STRIP MODEL

To construct a modified strip model in Figure 4, two groups of diagonal parallel strips are defined to simulate the steel plate in a CSPW while the concrete panel will not appear in the model, as it does not resist lateral force, bearing in mind the fact that there are gaps between the concrete panel and the boundary members. The effect of the panel to restrain the out-of-plane deformation of the steel plate is modeled by allowing the compressive action of the strips. Each strip is pin-connected to the boundary members and all the strips have a same cross sectional area and are placed with equal spacing. Number of each strip group is suggested not less than 10. The cross-sectional area of each strip, A_s, can be determined according to the spacing as follows,

$$A_S = \frac{t(L\cos\alpha + H\sin\alpha)}{n} \tag{1}$$

Where t, L and H are the thickness, width and height of the steel plate, respectively. α is the angle of inclination between the strip and the vertical line and n is the number of strips in one diagonal direction. The determination of α may make use of the formula in the theory of strip model for SPW as follows (Driver et al., 1998),

$$\tan^4\alpha = \frac{1 + tL/2A_C}{1 + th\left(\dfrac{1}{A_b} + \dfrac{h^3}{360 I_C L}\right)} \tag{2}$$

Where h is story height, A_c and A_b are cross-sectional area of boundary columns and boundary beams, respectively and I_c is the cross-sectional moment of inertia of boundary columns.

Driver and his group (Driver et al., 1998) demonstrated with their experimental results that the inclination angle of SPW varied from 42° to 50°, which covers the inclination of residual plastic deformation of the steel plate of the CSPW. Numerical parametric analysis showed that the variation of the inclination angle in the above range made little influence on the shear force and story drift of SPW. Hence, α is assigned with a constant value of 45° for simplicity, resulting in a simpler formula for Equation (1) as follows (Driver et al., 1998),

$$A_S = \frac{\sqrt{2}}{2}\frac{t(L+H)}{n} \tag{3}$$

In the strip model for SPW, the strip is a tension-only element. When extended to the modified strip model for CSPW, the strips in two diagonal groups are in tension and compression respectively. In the case of monotonic loading, a tension-only element and a compression-only one can be used, while for cyclic loading case, it is better to use the same axial bar element capable of resisting both tension and compression for each strip. The yielding and ultimate tensile stress of each strip can be simply defined as those of steel, as no interaction between the stresses in the tensile and compressive strips has been considered in the model. In this way, the compressive strength of the strips must be different from that of steel, even there is no buckling in the steel plate. This treatment may lead to errors in stress distributions in the members, but will play little effect on global behavior.

In order to determine the compressive strength, both the tensile and compressive behavior are assumed elasto-plastic. Considering the kinematic and equilibrium conditions for a hinged frame with rigid boundary columns and beams filled with a CSPW at limit state, the portion of a horizontal point load at the beam level resisted by all the tensile strips is as follows according to Berman and Bruneau (Berman and Bruneau, 2003),

$$V_T = 0.5 f_y Lt \sin 2\alpha \tag{4}$$

Which becomes

$$V_T = 0.5 f_y Lt \tag{5}$$

With the assumption of α=45°. Similarly the other portion resisted by all the compressive ones is the following

$$V_C = 0.5 f_y' Lt \tag{6}$$

Where f_y and f'_y are tensile and compressive strength of the strips, respectively. Thus the total capacity of the above system reads

$$V = V_T + V_C = 0.5\left(f_y + f_y'\right)Lt \tag{7}$$

According to AISC Seismic Provisions (2005), the capacity of a CSPW can be evaluated as

$$V = 0.6 f_y Lt \tag{8}$$

Comparing Equations (7) and (8) gives

$$f_y' = 0.2 f_y \tag{9}$$

Which means the compressive strength of the strip can be taken as 20% of the tensile one of steel.

4. NUMERICAL ANALYSIS

The experiment done by Gao research group is used to validate the proposed model (Gao, 2007). One of the specimen is modeled with four-sided of CPSW are connected to the boundary members. The specimen was installed in a pin-jointed frame with span of 2480mm and height of 1300mm (Figure 5). The height, width and thickness of the steel plate was 900mm, 1800mm and 2mm, respectively. The material properties of the steel plate of CW4 are listed in Table 1 (Gao, 2007).

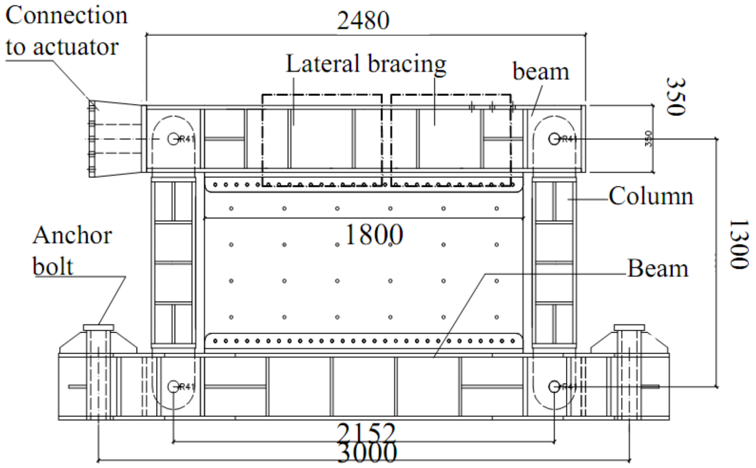

Figure 5. Test setup (Gao, 2007)

Table 1- Material properties of the steel plate of experimental specimen (Gao, 2007)

Plate thickness (mm)	Tensile strength (N/mm^2)	yield strength (N/mm^2)	Elastic modulus (N/mm^2)	Elongation (%)
2	362.9	287.0	174499.4	36.2

Software SAP2000 is used to simulate the test. The modified strip model for specimen is shown in Figure 6.

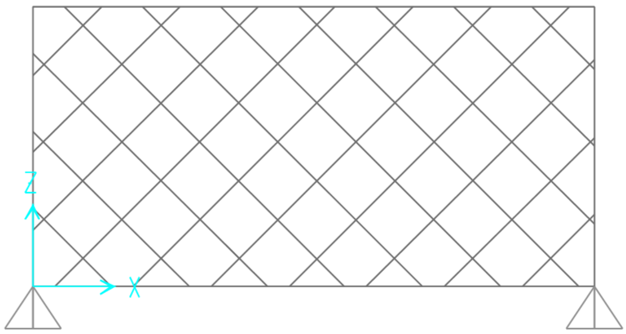

Figure 6. Model for experimental specimen with SAP2000

The boundary members are modeled with elastic beam element as they remained elastic during the test. For each strip, a fiber hinge is constructed to introduce material nonlinearity. In either diagonal direction, 10 parallel strips are constructed. The cross-sectional area of each strip is obtained with Equation (1) as follows,

$$A_s = \frac{0.707 \times 2 \times (1800 + 900)}{10} = 381.78 mm^2 \tag{10}$$

In order to verify Equation (9), the ratio f'_y/f_y is taken as primary parameter, R, here. Five cases with R = 0.0, 0.1, 0.2, 0.3 and 0.4 are considered. Pushover curves are shown in Figure 7. When R = 0.0, all the compressive strips are actually inactive, thus the initial stiffness in this case is only a half of all other cases, as indicated both in Figure 7. According to experimental results, the concrete panel of specimen contacted the fish plates after a considerably large displacement, due to out-of-plane displacement of the panel (Gao, 2007). The experimental pushover curves in Figure 7 are actually the skeleton of the original experimental hysteretic curve, which corresponds to the ultimate capacity of 792kN.

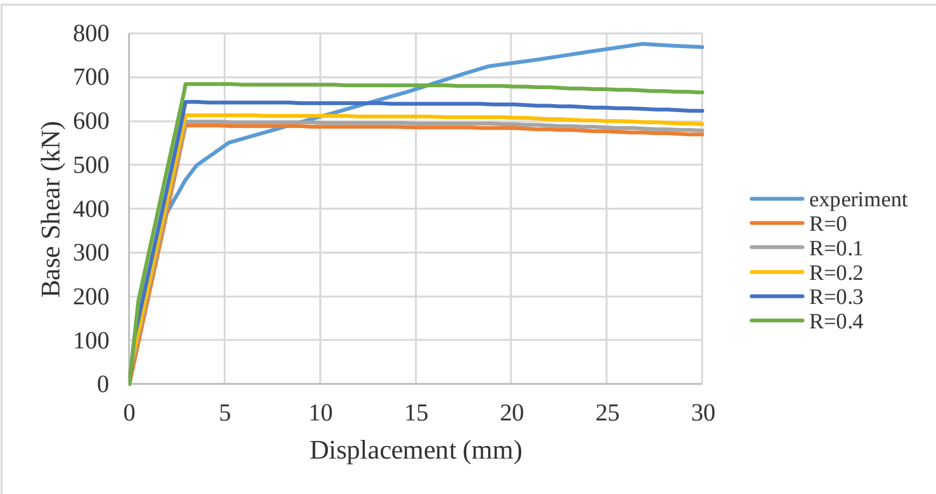

Figure 7. The effect of variation of the compressive strength on pushover results

For more accurate verifying Equation (9) and select the proper R parameter, initial stiffness of pushover curves are compared more precisely. As can be seen in Figure 8, the predicted initial stiffness in the case of R = 0.2 is most close to the test one, among other cases, which validates Equation (9).

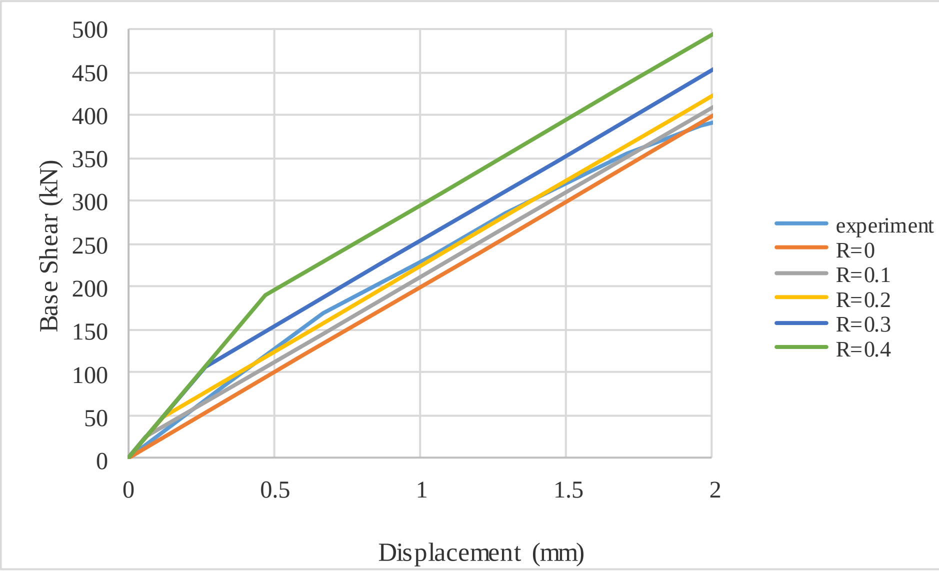

Figure 8. The effect of variation of the compressive strength on pushover results

In addition to R parameter, the ratio of post-yield stiffness to initial one, a, is chosen as the key parameter to perform parametric analysis. Five stress-strain curves with ratios of a = 0%, 0.5%, 1.0%, 1.5% and 2% are adopted and the relevant pushover results are plotted in Figure 9. The ratio of 2% introduces an increase of horizontal force by 20% at the displacement of 30mm, compared with the case of 0% (i.e. elastic-ideally plastic case). This case is most proper post-yield parameter comparing to experimental results.

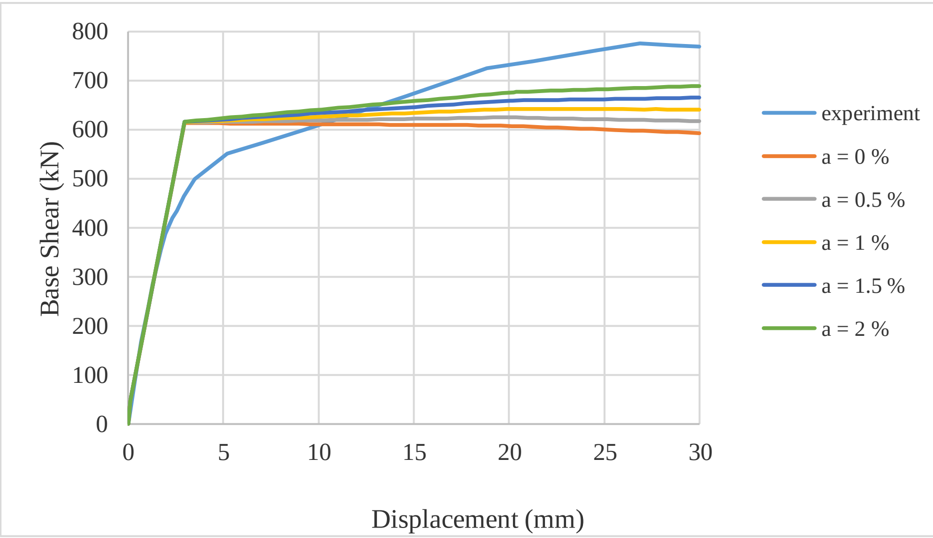

Figure 9. Effect of post-yield stiffness on pushover results

With the analysis above, the bi-linear model with R= f'_y/f_y = 0.2 and post-yield stiffness to initial parameter, a = 2% is used to simulate hysteretic behavior of experimental specimen. As can be seen in Figure 10, the comparison between the experimental hysteretic curve and numerical pushover curve validates that the accuracy of the proposed cross-strip model is acceptable.

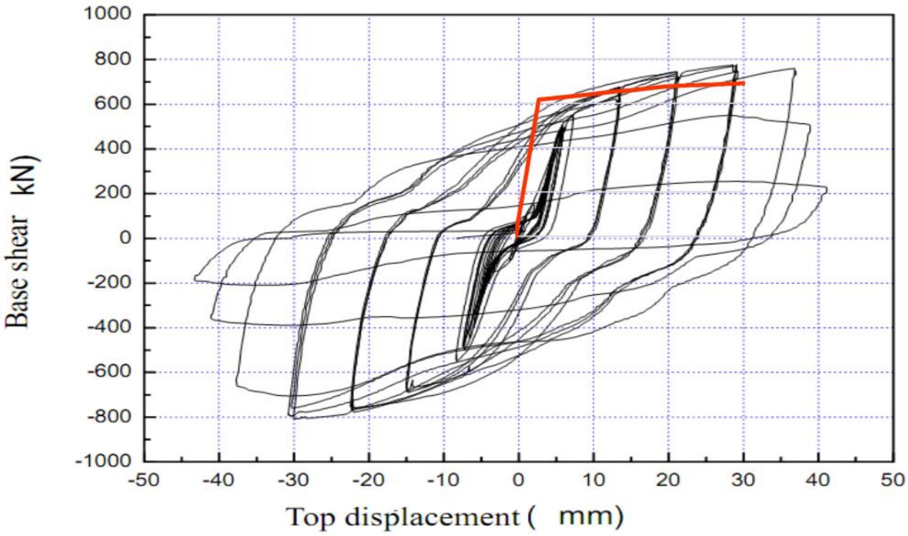

Figure 10. Validation of modified strip model with hysteretic curves

5. CONCLUSIONS

In this paper, based on the mechanism and failure mode of composite steel plate wall, a new modified strip model was proposed. Verification of this model is done by considering parameter R, the ratio f'_y/f_y, and parameter a, the ratio of post-yield stiffness to initial one. Comparison with experimental results showed that the proposed model was able to capture accurately nonlinear behavior of CSPSW under cyclic loading. The accuracy of the proposed model needs to be further refined especially for the initial compressive stiffness of the strips, as very few tests have been done.

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