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Comparative Study on Shear Strength of Corrugated Steel Plate Shear Walls

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Abstract

Utilizing corrugated web plates in steel shear walls (CSPSWs) has led to the introduction of a novel lateral load bearing system in recent years. Greater shear buckling strength compared with flat plates, considerable in-plan and out-of-plane stiffnesses, enhancement of ductility and energy dissipation capacity of the system and turning up a smooth hysteresis behavior without any pinching are some the merits that come up with using corrugated plates in steel shear walls. Despite conducting thorough investigations upon the shear strength of corrugated steel plates utilized in the web of girders regarding theoretical and dimensional conditions of such a structural element, accuracy and reliability of these formulas have been unclear when used for CSPSWs. This paper presents an exhaustive study on formulas employed for determining the shear strength of corrugated steel plate girders if implemented for CSPSWs. To this end, following experimental verification of two finite element models of vertical and horizontal CSPSWs (V-CSPSW and H-CSPSW) subjected to a complete cyclic loading protocol, a comparison among H-CSPSW's shear buckling stress values acquired by previously developed formulas that are suitable for corrugated web plate girders, with the numerical and experimental shear buckling stress results will be drawn, so as to study the correctness and applicability of these formulas when utilized for shear walls. Obtained results reveal that due to fundamental differences in theoretical assumptions, dimensional and boundary conditions of girders with shear walls, these formulas underestimate the shear strength of H-CSPSWs. Consequently, developing new shear strength formulas for CSPSWs, regarding their theoretical and dimensional characteristics, should be sought in the future for the cost-effective design of shear walls.

Keywords:
Steel plate shear wall, Corrugated web plate, Shear strength, Cyclic loading

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Introduction

Steel plate shear walls (SPSWs), have been employed in a variety of countries as an effectual lateral load bearing system for buildings by the reason of providing some advantages such as adequate lateral stiffness and strength, acceptable energy dissipation, and having remarkable post-buckling resistance. Along with the mentioned advantages, there are some disadvantages mainly related to the sudden drop of the stiffness and strength in post-buckling stage or negligible buckling strength in these systems that led researchers towards adopting strengthening methods such as the use of stiffener steel plates. These stiffener plates can bring about inevitable initial imperfections and residual stresses in the web plate of a steel shear wall as well as increasing construction cost and process. Given these findings, using corrugated web plates have been recently proposed as a well-suited alternative to flat web plates in steel shear walls with aim of obviating some of the shortcomings of these plates and improving the structural and seismic performance of the system. Corrugated steel plate shear walls have captured researchers' interest due to its unique buckling mode, having further initial lateral stiffness, providing notable out-of-plane stability and reducing construction costs (Emami et al., 2013). Corrugating steel plates generates a sort of specific geometry by which the axial and out-of-plane stiffnesses along the direction parallel to the corrugations are remarkably increased, while the stiffness in the direction perpendicular to the corrugations is significantly decreased, that is recognized as "Accordion Effects" (Zhao et al., 2017). Consequently, the corrugated steel web plates are capable of being in pure shear state and do not suffer gravity loads if they are horizontally placed in the web plate of shear walls. Stability characteristics and extra deformation capacity attributed to the corrugations in corrugated plates provide unstiffened corrugated steel plate shear walls with the capability of absorbing and dissipating more input energy, thereby maintaining the overall stability of the system even in the high drift ratios.

By conducting an experimental research upon half-scale steel shear walls with corrugated and flat web plates, Emami et al. (2013) made a comparison among their stiffness, ductility, cumulative dissipated energy, and strength. Bahrebar et al. (2016) demonstrated the influence of the angle of waves, the thickness of web plate, and central square opening size on the structural performance of steel shear walls through finite element simulation. Zhao et al. (2017) carried out a set of cyclic and pushover analyses upon finite element models including flat steel plate shear wall as well as trapezoidally and sinusoidally corrugated steel plate shear walls with 2 distinct wave directions while the effects of gravity loads were accounted as well. As reported, the overall behavior of corrugated steel plate shear walls is slightly sensitive to the gravity loads and destruction of boundary members than flat steel plate shear walls.

In general, extensive studies have been conducted to determine the shear strength of corrugated web plate girders, but little information is available about the accuracy of the proposed equations when applied to the corrugated steel plate shear walls. Easley and McFarland (1969) developed a formula for calculation of the global shear buckling capacity in lightweight corrugated plates by doing experiments on corrugated diaphragms. In addition, a set of laboratory tests along with numerical nonlinear analyses were performed by Elgaaly et al. (1996) to scrutinize responses of trapezoidally corrugated web plate beams and propose a formula for calculation of the inelastic shear buckling stresses in them. El-Metwally and Loov (2003) investigated composite girders made of pre-stressed concrete flanges which were connected to a corrugated steel web plate, and provided a formula to determine the shear strength of corrugated steel webs. Driver et al. (2006), also Yi et al. (2008) executed a series of laboratory tests and numerical modeling on the full-scale trapezoidally corrugated web plate girders typically utilized in steel bridges, to determine types and amount of shear buckling capacities in corrugated steel plates. Based on achieved results, each of them proposed their own lower bound formula by taking the effects of geometric imperfection and residual stresses of the construction into account. Through collecting a set of information consisting of more than

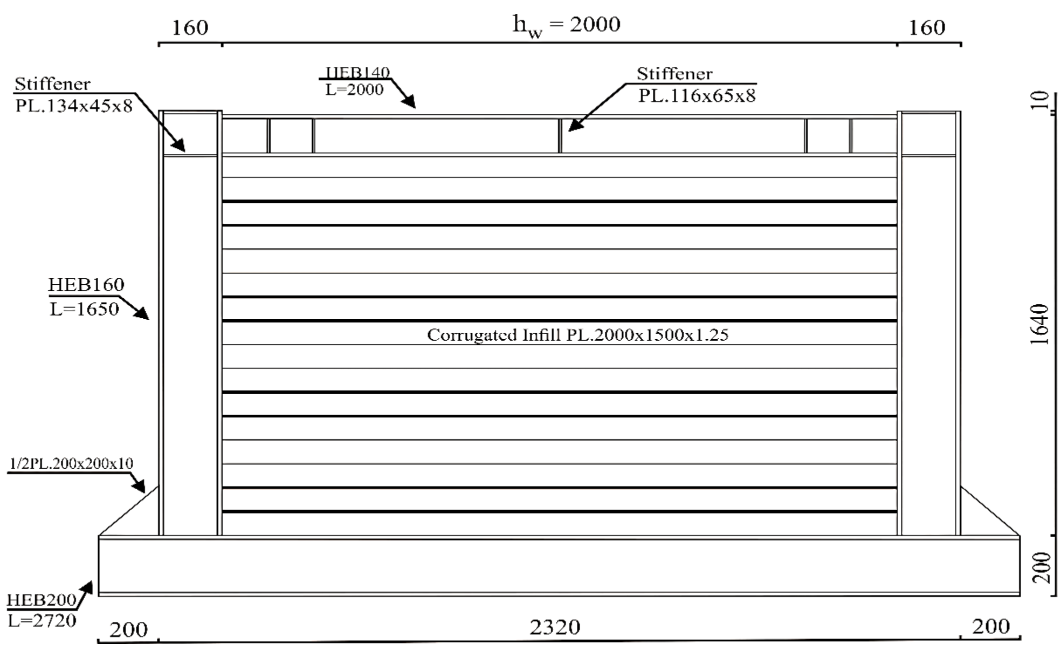
100 experiments which had been carried out upon corrugated web girders by various researchers, Sause and Braxtan (2011) examined the shear buckling strength of trapezoidally corrugated plates. The authors mentioned that due to the inconsistency between dimensions of tested girders with the theoretical conditions assumed in developing previous shear strength formulas, only a subset with 22 samples out of the 102 existing samples can fulfill the theoretical requirements. Hence, they proposed a formula for computing the shear strength of corrugated plates regarding only these 22 samples.

EN-1993-1-5 code (2006) has suggested a formula to estimate the shear strength of steel girders with trapezoidally corrugated web plates in terms of reduction factors corresponding to the global and local buckling stresses and shear yield stress of the web plate. Since the previously proposed formulas may underestimate the trapezoidally corrugated steel plates' shear strength, Leblouba et al. (2017) presented a formula based on a modified hyperbolic model and verified it with the Sause and Braxtan's database (2011) and their experimental data obtained from laboratory tests. It should be noted that in all of the aforementioned formulas the effect of post-buckling strength is neglected.

This study outlines the prediction accuracy of previously derived corrugated steel web girders' shear strength formulas in the case of applying to a trapezoidally horizontal corrugated steel plate shear wall as well as precise finite element modeling of corrugated steel plate shear walls tested by Emami et al. (2013).

Specifications and Details of CSPSW Specimens

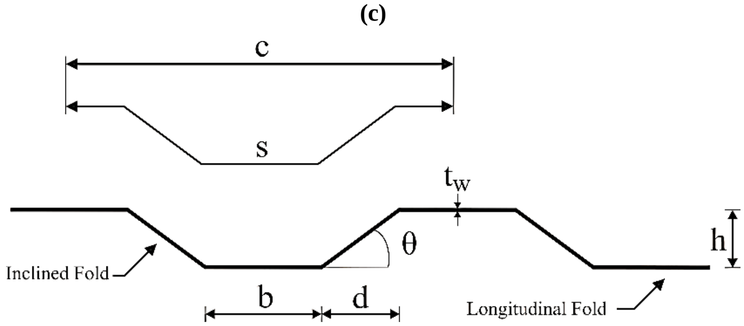
The selected CSPSW specimens for numerical simulation are two half-scale single-span one-story trapezoidally vertical and horizontal corrugated steel plate shear walls, tested by Emami et al. (2013). Geometric details of tested panels and the test setup are illustrated in Figure 1.



(a)

(b)

(c)



(d)

Figure 1. Geometric details of CSPSW specimens tested by Emami et al. (2013): a) Geometric sketch of panels; b) Tested V-CSPSW; c) Tested H-CSPSW; d) Geometric notations of trapezoidal waves.

The beam-to-column connections are considered rigid during the test process since complete joint penetration groove welds and fillet welds were used in beams' flanges and webs, respectively. The bottom parts of the panels were completely fixed through the laboratory floor. Furthermore, two transverse beams were fastened to the upper part of each panel so as to provide lateral support for specimens. The details of corrugations parameters and mechanical properties of used steels (measured values by Emami et al. (2013)) are tabulated in Tables 1 and 2, respectively.

Table 1. Trapezoidal waves parameters.

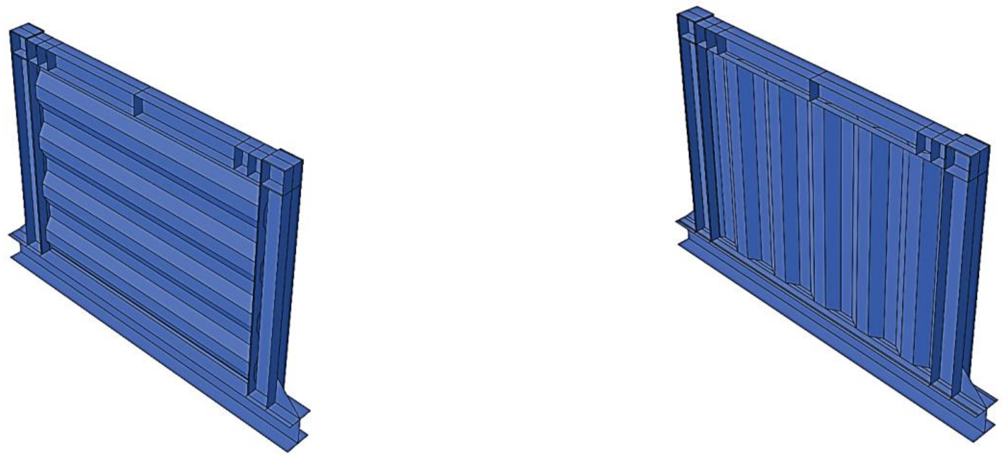
b (mm)	d (mm)	c (mm)	s (mm)	θ (°)	h (mm)	h_w (mm)	t_w (mm)
100	86.6	373.2	400	30	50	2000	1.25

Table 2. Mechanical properties of used steels.

Element	Type	Modulus of elasticity E (GPa)	Poisson's ratio ν	Steel type	Yield stress F_y (MPa)	Ultimate stress F_u (MPa)
Beam	HE-B140	210	0.3	St44	288	456
Column	HE-B160	210	0.3	St44	300	443
Web plate	Corrugated	210	0.3	St12	207	290

Finite element modeling and validation

The finite element modeling and nonlinear cyclic analyses were performed by means of ABAQUS software (2011). The members were modeled via four-node S4R shell elements with 6 degrees of freedom per node that are capable of estimating the behavior of thin-walled members reasonably and providing time-saving analyses. In order to avoid analysis errors and acquire preferable convergence mesh refinement process and partitioning techniques were conducted alongside merely use of structured quad elements. With the help of merge technique in the finite element software, whole connections were rigidly modeled. The lower part of the CSPSW models was restrained against displacements and rotations as well as upper beams in each panel were laterally braced to prevent out plane movements. The constructed numerical models are depicted in Figure 2.



(a) H-CSPSW **(b) V-CSPSW**

Figure 2. Finite element models of the corrugated steel plate shear walls.

Figure 3. shows the stress-strain relationships of the steels applied in the finite element simulation that are in accordance with Emami and Mofid (2014) tensile tests. It is noted that material and geometrical nonlinearities have been incorporated into the numerical simulation as well.

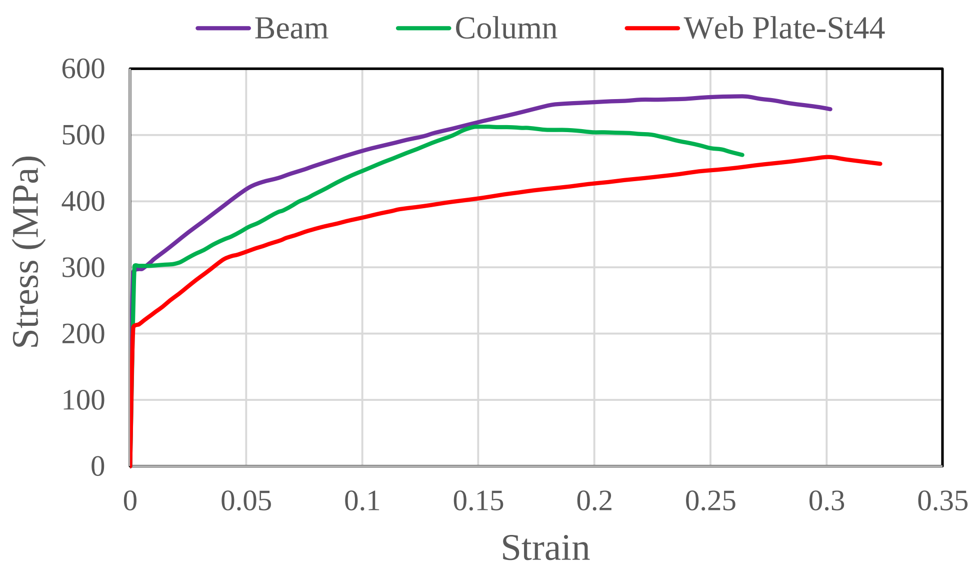
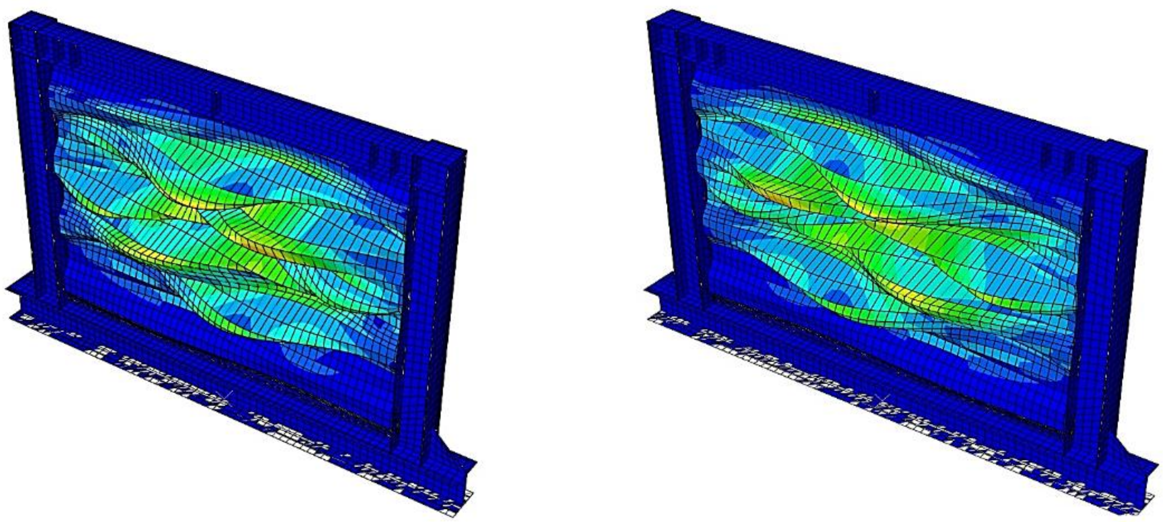


Figure 3. Uniaxial tensile behavior of used steels in members.

With regard to the fact that steel plate shear walls are mainly sensitive to the initial imperfections resulting from manufacturing or shop-working, geometrical imperfections should be regarded most in line with reality in both magnitude and distribution during numerical analyses. In this research, the initial imperfection was considered equal to 1/200 of the smallest each panel dimension (suggested by EN-1993-1-5, 2006), and according to the panels' critical buckling mode shape. Given performed sensitivity analyses, it was found that first and second buckling mode shapes of the models have the greatest impacts on their behavior. Therefore, following conducting Eigen buckling analyses to achieve maximum deformation in each buckling mode shape of the panels (as represented in Figure 4), the initial imperfection value was divided by these displacements and entered the software.



(a) (b)

Figure 4. Eigen buckling analyses results of the H-CSPSW: a) First buckling mode shape; b) Second buckling mode shape.

AC 154 (2008) quasi-static loading protocol, which is commonly implemented for simulating cyclic loading in metal sheathed shear walls tests, has been applied to the CSPSW models (Figure 5). As illustrated in Figure 5, one of the distinct features of this loading protocol is the decaying cycles subsequent to each peak displacement.

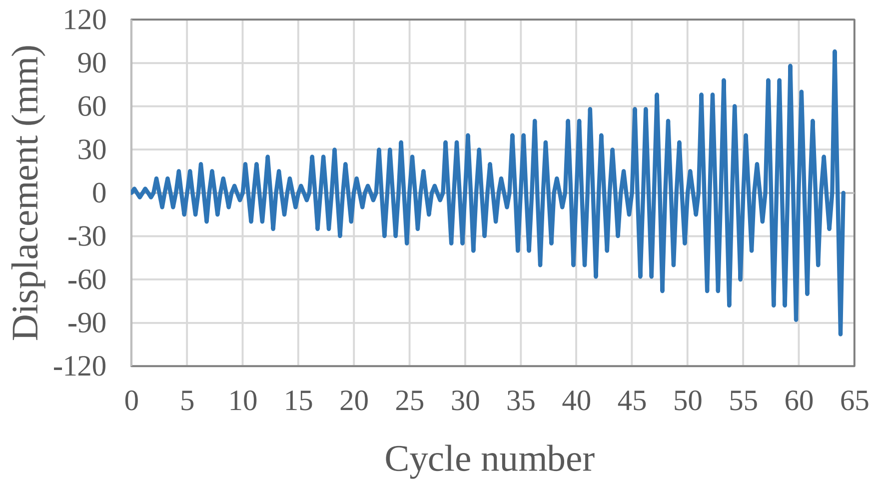
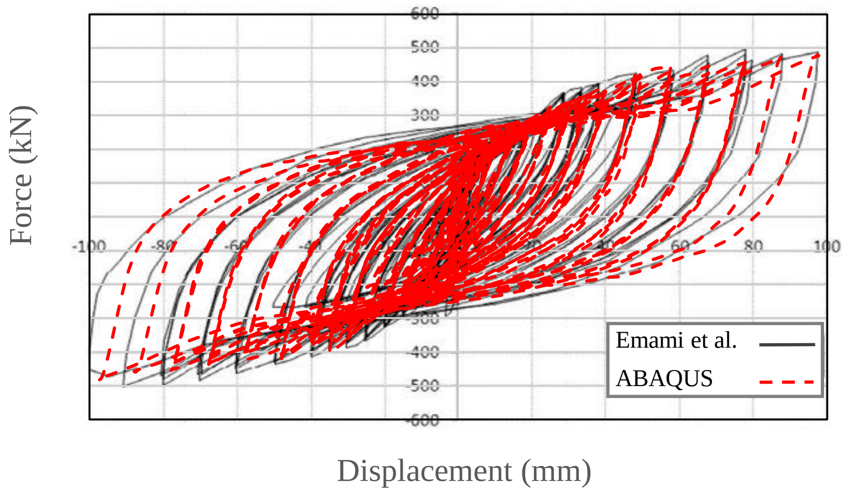
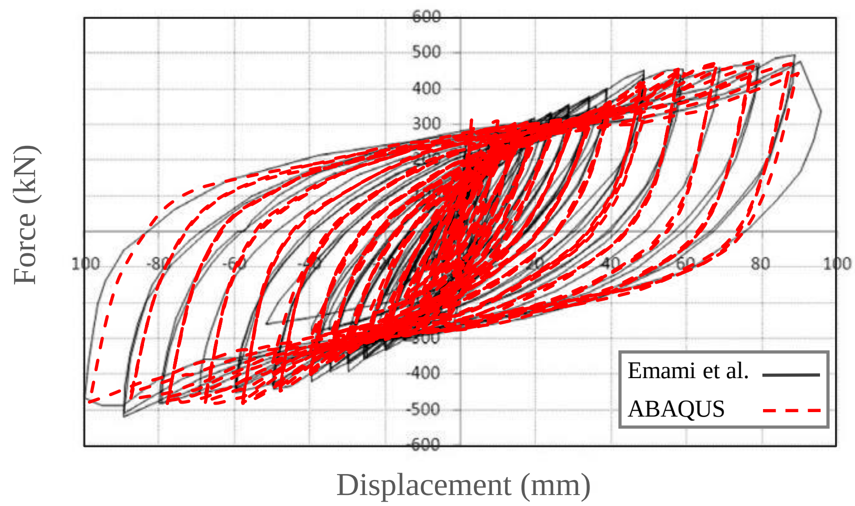


Figure 5. Cyclic displacement protocol.

The negligible influence of inertia in this kind of loading makes the static-general analysis method the best option for the finite element simulation. The combined half-cycle hardening corresponding to used steels' plastic properties was adopted for nonlinear analyses in the software so that an appropriate convergence considering a large number of applied cycles would be more likely to happen. Hysteresis behaviors of the finite element models have agreed well with those of Emami et al. (2013) tests in Figure 6.



(a)



(b)

Figure 6. Numerical models validation using Emami et al. (2013) laboratory test results: a) Hysteresis diagram of the H-CSPSW; b) Hysteresis diagram of the V-CSPSW.

Comparison among shear strength formulas of corrugated steel plates when used in different structural members

Generally, the shear buckling capacity of unstiffened flat steel web plates used in shear walls can be overlooked in comparison with corrugated steel web plates. However, both of these plates are vulnerable to buckling before reaching the preferable shear yielding. In this section, first, the buckling mode classification of corrugated steel plates are introduced, then formulas which have been gradually proposed by different researchers so far to determine the shear strength capacity of corrugated steel web plates in girders are presented. Eventually, a comparison among the shear buckling strength of the H-CSPSW obtained from experimental test and numerical simulation results with those produced by previously derived formulas for corrugated girders, will be drawn to ensure the precision of prediction by these formulas when used for steel shear walls.

As reported by the various experiments already conducted upon the corrugated web plate girders, three distinct buckling modes based on the geometry of these plates, namely, local buckling, global buckling and interactive buckling have been observed. The local buckling mode usually appears in the form of a subpanel buckling and corresponding local elastic shear buckling stress, $\tau_{L,el}$, is calculated through the plate buckling theory (Timoshenko and Gere, 1961) as follows:

$$\tau_{L,el} = k_L \frac{\pi^2 E}{12(1-\nu^2)(\omega/t_w)^2} \quad (1)$$

Where, k_L is the local shear buckling coefficient depending on the boundary conditions and aspect ratio of sub-panels, E is the modulus of elasticity, ν is the Poisson's ratio of the steel plate, t_w is equal to the plate thickness, and ω is the largest width between the longitudinal fold, b, and inclined fold, $d\,sec\,\theta$ (as shown in Figure 1). The value of k_L is calculated by the Equations (2) and (3) for simply supported edges, $k_{L,s}$, and fixed supported edges, $k_{L,f}$, respectively.

$$k_{L,s} = 5.34 + 4(\frac{b}{h_w})^2 \quad (2)$$

$$k_{L,f} = 8.98 + 5.6(\frac{b}{h_w})^2 \quad (3)$$

Where, h_w is the height of the web plate along the direction parallel to the corrugations. The parameters θ (corrugation angle), b, d, and h_w are as illustrated in Figure 1. Unlike the local shear buckling mode, the global shear buckling mode diagonally involves a large part of the plate. As a consequence of research on the corrugated steel plates utilized in diaphragms, Easley and Mcfarland (1969) developed a formula for calculation of the global elastic shear buckling stress using orthotropic plate theory, as follows:

$$\tau_{G,el} = k_G \frac{(D_x)^{1/4}.(D_y)^{3/4}}{t_w.{h_w}^2} \quad (4)$$

Where, k_G is the global shear buckling coefficient with the recommended values as 36 when the web plate is simply supported by flanges, and 68.4 when the web plate is fixed supported by flanges (Easley and Mcfarland, 1969). D_x and D_y are the transverse and longitudinal bending stiffnesses per unit length of the corrugated web plate, respectively, and are defined for the trapezoidally corrugated plates through following equations:

$$D_y = \frac{EI_y}{c} = \frac{E}{b+d}\left(\frac{b.t_w.(d.\tan\theta)^2}{4} + \frac{t_w.(d.\tan\theta)^3}{12.\sin\theta}\right) \tag{5}$$

$$D_x = \left(\frac{c}{s}\right)\left(\frac{Et_w{}^3}{12}\right) = \frac{b+d}{b+d.\sec\theta}\left(\frac{E.t_w{}^3}{12}\right) \tag{6}$$

Abbas et al. (2002) also proposed the following formulas for determination of the global elastic shear buckling stress considering the corrugated web as an orthotropic flat web:

$$\tau_{G,el} = k_G F(\theta,\beta)\frac{Et_w^{1/2}.b^{3/2}}{12h_w^2} \tag{7}$$

$$F(\theta,\beta) = \sqrt{\frac{(1+\beta).\sin^3\theta}{\beta+\cos\theta}}\ .\left\{\frac{3\beta+1}{\beta^2.(\beta+1)}\right\}^{3/4} \tag{8}$$

Where, β is the ratio of b to $d\sec\theta$. A small amount of β may end up with an uneconomical design and deeper corrugations than usual, however; a large amount of β leads to a small global shear buckling stress. Typically, the applicable values of β lie between 1 and 2. The values of θ should also be adopted in a way that each of the subpanels can efficiently support another one. Lindner and Huang (1995) recommended values greater than 30° for this parameter.

Yi et al. (2008) reported that if certain geometric limitations such as $h/t_w > 10$ and $b/h_w < 0.2$ are not satisfied in design of trapezoidally corrugated plates, these plates will be abruptly faced an instability due to the global shear buckling, even though the value of the local shear buckling stress would be smaller than the value of the global shear buckling stress in these plates. They presented the following formula for computing the global elastic shear buckling stress of the trapezoidally corrugated plates:

$$\tau_{G,el} = 36\beta E\frac{1}{[12(1-\nu^2)]^{1/4}}\left[\frac{(\frac{h}{t_w})^2+1}{6\eta}\right]^{3/4}\left(\frac{t_w}{h_w}\right)^2 \tag{9}$$

$$\eta = \frac{b+d}{b+d.\sec\theta} \tag{10}$$

In the following parts of this paper, Equation. (7) is used to compute $\tau_{G,el}$. The interactive shear buckling mode has considered the interaction between global and local modes as if it involves a couple of subpanels. Many researchers have proposed their own models to explore interactive shear buckling behavior in the corrugated web plate of girders (El-Metwally and Loov, 2003; Driver et al., 2006; Yi et al., 2008; Sause and Braxtan, 2011; EN-1993-1-5 code, 2006; Leblouba, 2017). There are two mainly dominated viewpoints for calculation of shear strength in the corrugated web plates. In the first point of view, the shear strength of the corrugated plate is directly considered equal to the interactive shear buckling stress. In the second point of view, the shear strength formula of the corrugated web plate is a derivative of the interactive elastic shear buckling stress. The interactive elastic shear buckling basic formula in most of the previously conducted researches has been expressed as follows:

$$\frac{1}{(\tau_{I,n,el})^n} = \frac{1}{(\tau_{L,el})^n} + \frac{1}{(\tau_{G,el})^n} \tag{11}$$

Where, $\tau_{I,n,el}$ denotes the interactive elastic shear buckling stress, and the exponent *n* is normally an integer. When the local and global elastic shear buckling stresses exceed 80% of the corrugated web plate's shear yield stress, τ_y, Elgaaly et al. (1996) defined Equation (12) to obtain the inelastic shear buckling stress, τ_{ine}. Based on Equation (12), Driver et al. (2006) presented the shear strength formula of the trapezoidally corrugated web plates considering the interactive shear buckling formula with $n = 2$:

$$\tau_{ine} = \sqrt{0.8\tau_y\tau_{el}} \tag{12}$$

$$\tau_A = \sqrt{\frac{(\tau_L.\tau_G)^2}{\tau_L^2+\tau_G^2}} \tag{13}$$

Where, τ_{el} is equal to elastic local or global buckling stress, τ_L and τ_G are the local and global shear buckling stresses, respectively, either in the elastic or inelastic state. According to the von Mises yield criterion, the shear yield stress of the corrugated plate is calculated as follows:

$$\tau_y = \frac{F_y}{\sqrt{3}} \tag{14}$$

Where, F_y is the uniaxial yield stress of the corrugated web plate. El-Metwally and loov (2003) provided the following shear strength formula for corrugated webs by taking into account the shear yield stress of the corrugated web plate as the upper bound:

$$\tau_M = \left(\frac{1}{(\tau_{L,el})^2} + \frac{1}{(\tau_{G,el})^2} + \frac{1}{(\tau_y)^2}\right)^{-1/2} \tag{15}$$

Sayed-Ahmad (2001) also applied Equation (15) to determine the shear strength of trapezoidally corrugated web plates and exerted a strength reduction factor of 0.6-0.7 on the design shear strength values. Yi et al. (2008) by defining $\lambda_s = \sqrt{\frac{\tau_y}{\tau_{i,1,el}}}$ as the interactive slenderness ratio, used the Design Manual (1998) to account inelasticity effects, residual stresses and initial deformations in calculating the shear strength of corrugated web plates as follows:

$$\frac{\tau_Y}{\tau_y} = \begin{cases} 1 & \lambda_s < 0.6 \\ 1 - 0.614(\lambda_s - 0.6) & 0.6 < \lambda_s \le \sqrt{2} \\ 1/\lambda_s^2 & \sqrt{2} < \lambda_s \end{cases} \tag{16}$$

Sause and Braxtan (2011) compiled and filtered a large number of published test results of various studies based on the three limitations: $\theta \geq 22^{\circ}$, $a/h_w > 1$ (a is the distance between the loading point and the reaction point) and $0.87 \leq \beta \leq 1.13$, related to the geometric and theoretical conditions of the corrugated girders. Then, they defined the following formula to calculate the shear strength of the corrugated steel web plates:

$$\tau_B = \left(\frac{1}{(\tau_{L,el})^3} + \frac{1}{(\tau_{G,el})^3} + \frac{2}{(\tau_y)^3}\right)^{-1/3} \tag{17}$$

Leblouba et al. (2017) with the help of statistical procedures and experimental tests results, derived parameters of a hyperbolic model to develop an optimal formula for predicting the shear strength of trapezoidally corrugated web plates as follows:

$$\tau_{M-1} = \frac{\tau_y}{\left(1+\left(\frac{\sqrt{\frac{\tau_y}{\tau_{I,4,el}}}}{1.58}\right)^{1.6}\right)^{1.15}} \tag{18}$$

Annex D of EN-1993-1-5 (2006) has provided the following formula to estimate the shear strength of a girder with either trapezoidally or sinusoidally corrugated steel web plate, regardless of interactive shear buckling mode:

$$\tau_{EC3} = X_c \tau_y \tag{19}$$

Where X_c is the least value between reduction factor for local shear buckling, $X_{c,L}$, and global shear buckling, $X_{c,G}$. The reduction factor of the local shear buckling, $X_{c,L}$, for the trapezoidally corrugated plates should be calculated as:

$$X_{c,L} = \frac{1.15}{0.9+\sqrt{\frac{\tau_y}{\tau_{L,el}}}} \leq 1.0 \tag{20}$$

The reduction factor of the global shear buckling, $X_{c,G}$, for the trapezoidally corrugated plates should be taken as:

$$X_{c,G} = \frac{1.5}{0.5+\frac{\tau_y}{\tau_{G,el}}} \leq 1.0 \tag{21}$$

From Figure 7, it is clear that the H-CSPSW specimen has suffered premature shear buckling in the primary cycles of loading. The values of local and global shear buckling stresses of the H-CSPSW in both elastic and inelastic states are listed in Table 3. In Table 4, the amount of shear buckling strength resulting from the test of H-CSPSW is compared with those obtained from the numerical modeling and the above-mentioned formulas.

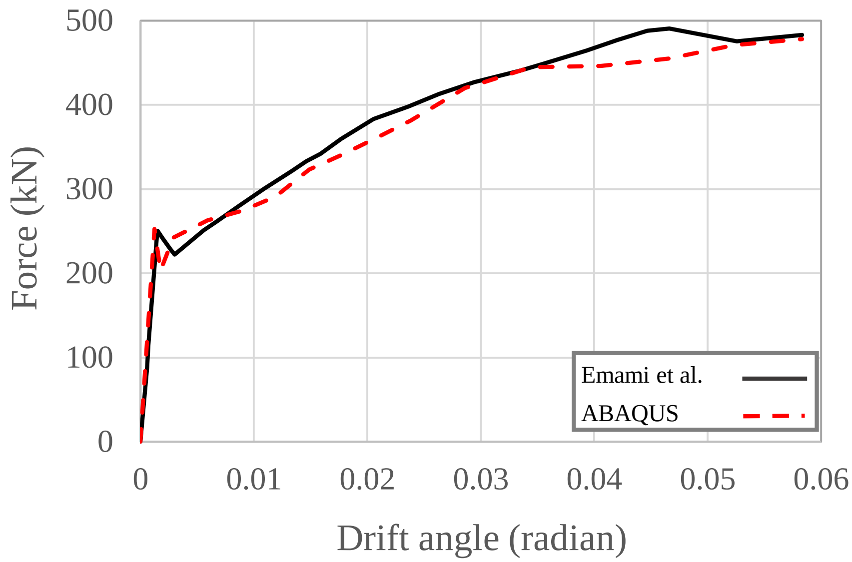


Figure 7. Backbone curve of the horizontally corrugated-web shear wall.

Table 3. Shear buckling stresses of the H-CSPSW specimen.

$\tau_{L,el}$ (MPa)	$\tau_{G,el}$ (MPa)	$\tau_{inel,L}$ (MPa)	$\tau_{ine,G}$ (MPa)
158.66	108.4	123.16	101.8

Table 4. Comparison between experimental shear buckling strength of the H-CSPSW with those obtained from the numerical modeling and previously developed formulas.

$\tau_{Experimental}$ (MPa)	$\tau_{analytical}$ (MPa)	τ_{A} (MPa)	τ_{M} (MPa)	τ_{Y} (MPa)	τ_{B} (MPa)	τ_{M-1} (MPa)	τ_{EC3} (MPa)
100.44	102	78.46	71.64	63.57	76.8	72.68	77.74

From these results, it is concluded that developed formulas, which have been applied to corrugated girders, can conservatively estimate the shear buckling stress of the H-CSPSW. The closest prediction is attributed to the τ_A, that is almost equal to 78% of the experimental shear buckling stress value. This condition would become worse if a strength reduction factor was imposed on the predicted shear strength amounts. Note that the H-CSPSW does not satisfy one of the theoretical limitations assumed for the corrugated girder formulas ($a/h_w > 1$). Thus, the shear buckling stress of the corrugated web plate has to be underestimated according to the claim of Sause and Braxtan (2011). As a result, due to fundamental differences in the aspect ratio, length of waves, boundary conditions, size and strength of boundary members, and size of the tension field between the corrugated steel plate shear walls and the corrugated web plate girders, there is a vital need to introduce new independent formula for calculation of the shear strength in CSPSW systems.

Conclusion

This paper presented the precise finite element simulation of two corrugated steel plate shear walls as well as the applicability of previously established shear strength formulas of corrugated-web steel girders for predicting shear strength of CSPSWs. The previously developed formulas to estimate the shear strength of corrugated steel web plates in girders are fundamentally based on theoretical and dimensional conditions related to the girders. Obtained results have demonstrated that employment of these formulas to determine the shear strength of corrugated steel plate shear walls may come up with underestimation of capacity. Further experimental and numerical investigations are still needed to derive proper equations that would be able to accurately predict the shear strength of CSPSWs regarding the boundary conditions and dimensions of steel shear walls. All in all, available equations can be used for the conservative estimation of CSPSWs' shear buckling strength in design.

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