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Analysis and design recommendations for corrugated steel plate shear walls with a reduced beam section

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ABSTRACT

As the design methodology of the corrugated steel shear wall (CSSW) has not yet been included in the design standards, this paper presents a step-by-step design procedure based on the corrugated panel-frame interaction (CPFI). To ensure that the plastic hinges occur at the beam ends and not within the beam span or in the columns, a reduced beam section (RBS) connection was used in steel plate shear walls (SPSWs). In this paper, the corrugated steel shear wall with a reduced boundary beam section (RBS-CSSW) is presented as a promising type of lateral-load-resisting system. Analytical equations were proposed to estimate the strength of RBS-CSSW based on the frame plastic hinge development considering the interactive shear buckling stress in the corrugated panel. The equations of strength estimation for RBS-CSSW were also evaluated by comparing their results with the numerical results of the two models. The comparison showed that the strength based on the proposed analytical equations were accurate (with more than 95% accuracy) compared to the strength from the FE pushover analyses for the two models.

1. Introduction

Steel plate shear walls (SPSWs) are lateral-load-resisting systems typically used in high seismic zones utilizing tension field action developed by shear buckling prior to shear yielding. The system includes infill plates (stiffened or unstiffened) bounded by a steel frame with vertical and horizontal structural elements. Several studies illustrated that the SPSW is an economical and efficient system for high-rise buildings [1] as it reduces the walls' dead loads and increases the usable floor space, which can be utilized not only for new constructions but also for the retrofitting of existing structures [2].

Most of the numerical and experimental researches about SPSW that were conducted in the past decade were based on the flat plates used as infill panels [3–5]. Sabouri-Ghomi et al. [6] employed the plate-frame interaction (PFI) for the analysis and design of flat shear walls with a boundary frame. The most important feature of the PFI model is its independence from the panel section. It can be used with thin or thick plates, stiffeners, and perforated infill plates. Investigations have been conducted to develop potential approaches to avoiding out-of-plane buckling prior to shear yielding, and to increasing the strength and stiffness of buildings by employing relatively thick steel plates or reinforcing with vertical and horizontal stiffeners [7,8].

Due to their high in- and out-of-plane geometric stability, corrugated steel plates were proposed as replacements for the stiffened steel plates in girders in the 1980s. The higher stiffness of corrugated plates, even if they are thinner compared to the flat ones, has made them useful for the construction of light girders [9]. Generally, corrugated plates have low stiffness perpendicular to the corrugation direction while their strength for resisting the in-plane forces along the corrugation is significant [10]. Mo and Perng [11] installed a corrugated steel shear wall (CSSW) in the reinforced concrete (RC) frame and evaluated its seismic performance experimentally. Berman and Bruneau [12] tested three one-story specimens: two specimens with flat infill plates and one specimen with a corrugated infill plate. It was observed that the frame with a corrugated wall could significantly dissipate energy and reduce the demands on the boundary elements. Emami et al. [13] studied the cyclic behavior of trapezoidally corrugated walls through three experimental tests, and their study results showed that

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the corrugated walls had a larger initial stiffness, ductility ratio, and energy dissipation capacity than the flat walls. Emami and Mofid [14] reported that finite element (FE) analysis could not precisely determine the buckling behavior of the corrugated walls but could determine the buckling shear strength. Vigh et al. [15] employed a genetic algorithm (GA) to calibrate the cyclic behavior of a corrugated wall. Farzampour et al. [16] and Farzampour and Yekrangnia [17] conducted a numerical parametric study using FE analysis to investigate the effect of an opening in CSSWs, and proposed the optimized rectangular opening locations. Yu and Yu [18] conducted a study on ten full-scale light-gauged CSSWs with different circular opening configurations and concluded that the circular holes may increase the ductility of the wall but the existence of an opening leads to a significant reduction in the stiffness and strength of the shear walls.

Zhao et al. [19] represented the results of their numerical parametric study on CSSWs with different configurations as well as the effects of gravity load using the ABAQUS software. Recently, Shon et al. [20] studied the hysteresis behavior of trapezoidal corrugated walls through an experimental test and concluded that such walls have higher energy dissipation, ductility, and ultimate capacity compared to the bare frame. Several numerical and experimental studies associated with cold-formed steel-framed walls sheathed with corrugated steel sheets have also been proposed [21,22].

Along the same lines, an economical connection type used for steel structures in seismic zones is the reduced beam section (RBS) [23]. To postpone the brittle fractures near the beam flange-to-column welds, the RBS design has been extensively used since after the 1994 Northridge and 1995 Kobe earthquakes. Chi and Uang [24], 24] put forth RBS moment connection design recommendations based on the results of their analytical study with deep columns. RBS concepts have also been used of late in various structural systems. Over the past decade, a limited number of studies on the use of RBS in SPSWs were conducted to ensure that the frame plastic hinges occur at the ends of the beams and not in the columns [25–28].

As corrugated steel shear walls with reduced beam sections (RBS-CSSWs) have not yet been included in the design provisions, this paper suggests a clear design procedure for them. The behavior of RBS-CSSW as a new lateral-load-resisting system was investigated, and analytical solutions considering the interactive shear buckling stress in the corrugated panels were proposed to estimate the strength of the wall. Computational models were then developed to study the wall as well as to validate the proposed analytical equations. Ultimately, the behaviors of the RBS connections with SSW and CSSW were compared in terms of their strength, stiffness, and ductility.

2. Behavior and design of CSSWs

2.1. Equation development

To better understand the effect of the RBS connection used with CSSWs, the contribution of the frame including the RBS connection to the strength was considered separately. The total strength of the RBS-CSSW is represented by Eq. (1).

$$F_{su} = F_{pt} + F_{fu} \tag{1}$$

where F_{su} is the ultimate strength of the RBS-CSSW, F_{pt} is the strength of the plate, and F_{fu} is the strength of the frame with an RBS connection. Fig. 1 shows the state of stresses of a general steel plate during the post-buckled stage if θ is assumed as the inclination of tension field. The state of stress indicated in Fig. 1b in the steel wall at the yield condition in the x-y coordinates ignoring the critical stresses in the buckle state is given by the following equations:

$$\sigma_{xx} = \sigma_{ty} sin^2\theta \tag{2}$$

$$\sigma_{yy} = \sigma_{ty} cos^2\theta \tag{3}$$

$$\sigma_{xy} = \quad \sigma_{yx} = 0.5 \quad \sigma_{ty}\sin 2\theta \tag{4}$$

in which σ_{ty} is the tension field stress corresponding to the yield. The total state of stress in the steel plate at yield considering the critical buckling stresses is as follows:

$$\sigma_{xx} = \sigma_{ty} sin^2\theta \tag{5}$$

$$\sigma_{yy} = \sigma_{ty} cos^2\theta \tag{6}$$

$$\sigma_{xy} = \quad \sigma_{yx} = \tau_{cr} + 0.5 \quad \sigma_{ty}\sin 2\theta \tag{7}$$

Yielding of the steel plate occurs as the total existent stresses reaches the Von-Mises yield criterion, which is represented by Eq. (8).

$$(\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{zz} - \sigma_{xx})^2 + 6\sigma_{xy}^2 + 6\sigma_{yz}^2 + 6\sigma_{zx}^2 - 2\sigma_y^2 = 0 \tag{8}$$

where σ_y is the yield stress; σ_{xx}, σ_{yy}, and σ_{zz} are the normal stresses in the X, Y, and Z directions, respectively; and σ_{xy}, σ_{yz}, and σ_{zx} are the shear stresses. Substituting Eqs. (5)–(7) into Eq. (8) and considering that $\sigma_{zz} = \sigma_{yz} = \sigma_{zx} = 0$ will result in Eq. (9).

$$\sigma_{ty}^2 + (3\tau_{cr,\ in}^e sin2\theta)\sigma_{ty} + (3 \quad {\tau_{cr,\ in}^e}^2 - \sigma_y^2) = 0 \tag{9}$$

Therefore, the value of σ_{ty} can be calculated from Eq. (9), implementing the yielding of the steel plate (σ_y) and the total elastic interactive shear buckling stress ($\tau_{cr,in}^e$). The $\tau_{cr,in}^e$ is calculated using Eq. (10) [29].

$$\frac{1}{\tau_{cr,\ in}^e} = \frac{1}{\tau_{cr,\ L}^e} + \frac{1}{\tau_{cr,\ G}^e} + \frac{1}{\tau_y} \tag{10}$$

where the critical local and global shear buckling stresses shall be obtained from Eqs. (11) and (12), respectively.

$$\tau_{cr,\ L}^e = [5.34 + 4\left(\frac{a}{h}\right)^2]\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{a}\right)^2 \tag{11}$$

$$\tau_{cr,\ G}^e = \frac{36\varphi E}{[12(1-\nu^2)]^{0.25}}\left[\frac{\left(\frac{d}{t}\right)^2+1}{6\gamma}\right]^{0.75}\left(\frac{t}{h}\right)^2 \tag{12}$$

where τ_y is the yielding shear stress; E and υ are Young's modulus of elasticity and Poisson's ratio, respectively, as indicated in Fig. 2; a is the length of the flat panel; h is the panel height; φ is the boundary condition factor, which varies from 1.0 to 1.9 and is assumed as 1.0 in this study for the simple support condition; d is the corrugation depth; and γ is the corrugation geometric factor, as indicated in Eq. (13).

Fig. 2 shows axes 1 and 2 representing the direction perpendicular and parallel to the corrugation direction, respectively. γ is the geometric parameter calculated using Eq. (13).

$$\gamma = \frac{a+b}{a+c} \tag{13}$$

in which b is the horizontal projection of the inclined panel width and c is the inclined panel width. The shear strength of the corrugated plate indicated on the righthand side of Eq. (1) is obtained using Eq. (14).

$$F_{pt} = \sigma_{xy}.\ L.\ t \tag{14}$$

Substituting Eq. (7) into Eq. (14) will result in Eq. (15).

$$F_{pt} = (\tau_{cr,in}^e + 0.5\sigma_{ty} sin2\theta).\ L.\ t \tag{15}$$

where L and t are the length and thickness of the panel, respectively; and $\tau_{cr,in}^e$ is the interactive elastic shear buckling capacity of the plate.

The shear displacement of the steel plate at yield is the summation of the shear displacement of the steel plate in the buckling condition ($U_{w,cr}$) and of the shear displacement due to the tension field action ($U_{w,tf}$). Therefore, the total shear displacement is obtained using Eq. (16).

$$U_w = \quad U_{w,cr} + U_{w,tf}, \tag{16}$$