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Behavior of columns of steel plate shear walls with beam-connected web plates

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ABSTRACT

Steel plate shear walls with beam-connected web plates (B-SPSWs) are an alternative configuration of steel plate shear walls (SPSWs) where web plates are connected to the beams only. Detaching web plates from columns and introducing simple beam-column connections in B-SPSWs eliminate flexural demands in the columns resulting from web plate tension field action; consequently, the columns of B-SPSWs are designed primarily for axial loads. A recent study, however, showed that the columns of B-SPSWs resist significant flexural demands during earthquake shaking due to differential interstory drifts that result in significant column rotations at floor levels. Typical design methods (i.e., the Equivalent Lateral Force method and Modal Response Spectrum analysis) do not capture these rotations associated with differential drifts that might lead to column instability. A two-phase numerical study is conducted to evaluate the behavior and stability of B-SPSW columns. In the first phase, three-dimensional nonlinear response-history analyses are conducted to investigate the column stability for eighteen B-SPSWs with different geometric characteristics designed following two design approaches. The results suggest that column buckling is a possible mode of failure for one of the design approaches. In the second phase, a parametric study is undertaken to further investigate potential column buckling failure modes in B-SPSW columns and to establish an upper-bound estimate for the column buckling strength reduction due to column rotations at floor levels that are not considered in traditional design approaches.

1. Introduction and background

Steel plate shear walls (SPSWs) are a reliable lateral load resisting system known for their high lateral stiffness, stable hysteresis characteristics, and high energy dissipation capacity [1–5]. Owing to moment-resisting beam-column connections in the boundary frame, the moment frame resists some portion of the lateral load; however, the primary lateral load resisting elements of SPSWs are web plates that are connected to beams and columns on all four edges. Due to a mechanism called tension field action (TFA) [6], thin web plates have a significant lateral load capacity and lateral stiffness after web plate buckling, provided that the surrounding boundary frame (i.e., beams and columns) has sufficient flexural stiffness and strength to anchor the inclined pull-in forces resulting from TFA. The inclined pull-in forces of the adjacent stories acting on an intermediate story beam create bending demands in opposite directions (i.e., the pull-in forces of the upper-story and lower-story web plates result in hogging and sagging in the beam, respectively). Unlike the beams of SPSWs, the pull-in forces act on only one side of the columns of SPSWs; consequently, the flexural demands resulting from TFA are more critical for columns. In addition to flexural demands, the columns of SPSWs resist significant axial loads due to both TFA and frame action, which leads to very large column sections for conventional SPSWs [7,8]. The behavior of the columns of SPSWs have been investigated by several researchers [9–12].

An alternative configuration of SPSWs called SPSWs with beam-connected web plates and simple beam-column connections (B-SPSWs) is proposed in the literature, where web plates are attached to the beams only [5,13–18]. Releasing web plates from the columns eliminates both flexural and axial demands in the columns due to the pull-in forces of the TFA acting on columns. Similarly, the simple beam-column connections prevent bending moment demands in columns resulting from frame action. Consequently, the columns of B-SPSWs show a similar behavior to gravity columns (leaner columns) and are expected to primarily resist axial loads, which might lead to lighter columns compared to the columns of conventional SPSWs.

Due to the difference in the boundary conditions of web plates, B-SPSWs show a different behavior in terms of the formation of TFA compared to conventional SPSWs. Fig. 1(a) shows the formation of

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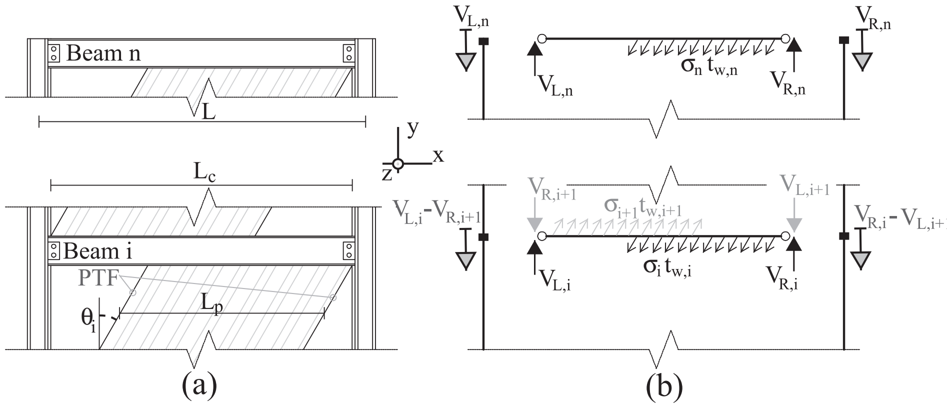

Fig. 1. PTF in B-SPSWs (right sway): (a) formation of PTF and (b) column demands due to PTF.

tension field action in the web plates of B-SPSWs when the B-SPSW undergoes a right lateral sway. Since the columns of B-SPSWs do not anchor the web plates, the formation of TFA is limited to some diagonal portion of the web plates, resulting in a partial tension field (PTF). The length of the PTF (L_p) and the inclination of the PTF measured from the vertical axis (θ) are given by Eq. (1) and Eq. (2), respectively, where L_c and H_c are the clear length and the clear height of the web plate, respectively [19]. The semi-empirical derivation of Eq. (2) for the PTF inclination is described in detail by Ozcelik and Clayton [19]:

$$L_p = L_c - H_c\tan\theta \tag{1}$$

$$\theta = max\begin{pmatrix} 0.55-0.03\frac{L_c}{H_c} \\ 0.51 \end{pmatrix}\tan^{-1}\frac{L_c}{H_c} \tag{2}$$

Fig. 1(b) shows the beam end vertical reactions and corresponding axial loads at the floor levels acting on the columns due to the PTF acting on the beams. The axial load transferred by the ith floor beam (where i is the story index) to the right column is equal to the difference between the beam right end upward vertical reaction due to the PTF forces of the ith story plate ($V_{R,i}$) and the beam right end downward vertical reaction due to the PTF forces of the (i + 1)th story plate. Owing to the equilibrium of the web plate, the beam right downward vertical reaction of the ith floor beam due to the PTF forces of the (i + 1)th story plate is equal to the beam left end upward vertical reaction of the (i + 1)th floor beam due to the PTF forces of the (i + 1)th story plate ($V_{L,i+1}$). Similarly, the axial load transferred by the ith floor beam to the left column is equal to the difference between the beam left end upward vertical reaction due to the PTF forces of the ith story plate ($V_{L,i}$) and the beam left end downward vertical reaction due to the PTF forces of the (i + 1)th story plate ($V_{R,i+1}$). Assuming the web plate stress in the PTF (σ) is uniform, the axial loads in left and right columns in the ith story due to lateral loads ($P_{R,i}$ and $P_{L,i}$, respectively), $V_{R,i}$, and $V_{L,i}$ can be calculated using Eq. (3), Eq. (4), Eq. (5), and Eq. (6), respectively, where t_w is the web plate thickness and n is the number of stories.

$$P_{R_i} = \sum_{j=i}^{n} V_{R_j} - \sum_{k=i+1}^{n} V_{L_k} \tag{3}$$

$$P_{L_i} = \sum_{j=i}^{n} V_{L_j} - \sum_{k=i+1}^{n} V_{R_k} \tag{4}$$

$$V_{R_i} = \sigma t_{w_i}\cos^2\theta_i L_{P_i}(1-0.5L_{p_i}/L_c) \tag{5}$$

$$V_{L_i} = \sigma t_{w_i}\cos^2\theta_i L_{P_i}(0.5L_{p_i}/L_c) \tag{6}$$

A proof-of-concept study was undertaken by Ozcelik and Clayton [20,21] to assess the performance of B-SPSWs designed for low-seismic regions. Three-, six-, and nine-story B-SPSWs were designed for Boston adopting the dead and live loads and the floor plan of the three-story prototype building given in the SAC steel project [22]. Three aspect ratios (L_c/H_c) were considered in the parametric study to cover a wide range of applications. Ozcelik and Clayton [20] adopted two different design approaches, namely, the non-seismically-detailed design (ND) and the seismically-detailed design (SD); consequently, eighteen B-SPSWs were studied. The response modification factors (R) were chosen as 3 for the ND to be consistent with the R = 3 assumed for non-seismically detailed steel systems, and R = 3.25 was assumed for the SD to be consistent with the comparable Ordinary Concentrically Braced Frame steel system suitable for low-seismic design [23]. The web plate stress, σ, to be used in Eqs. (5) and (6) to determine the column demands is given in Eq. (7) for both design approaches. Note that σ for the ND is equal to the web plate stress resulting from the forces that are used to proportion the web plate; whereas, the SD adopts capacity design principles.

$$\sigma_i = \begin{cases} \dfrac{2\sum_{k=i}^{n} F_k}{L_{p_i}t_{w_i}\sin2\theta_i} & \text{for } ND \\ R_yF_{yw} & \text{for } SD \end{cases} \tag{7}$$

where F_k is the portion of the base shear applied at the story level k determined from the Equivalent Lateral Force method (ELF), and R_yF_{yw} is the expected yield strength of the web plate material.

Following the ND and SD approaches, Ozcelik and Clayton [20] designed the columns of the B-SPSWs for the axial loads given in Eqs. (3) and (4) and the moments resulting from these loads acting at the beam end, which is offset from the column centerline (note that moment demands from this eccentric force accounted for less than 10% of the design axial load-moment interaction ratio). Details of these designs, including geometry, beam and column member sizes, and web plate thicknesses are provided by Ozcelik and Clayton [20]. Two-dimensional nonlinear response-history analyses of the B-SPSWs were performed in OpenSEES [24], where the columns were represented by nonlinear beam-column elements without considering initial imperfections and residual stresses in the columns. The reason not to consider initial imperfections in the columns in the OpenSEES model [20] was that the columns were oriented for weak-axis buckling out-of-plane, which was not explicitly considered in the two-dimensional model. The results of nonlinear response-history analyses revealed that the beams of SPSWs performed satisfactorily (i.e., the beams remained elastic); however, axial and moment demands in the columns suggested yielding and/or instabilities occurred in the columns of some buildings in spite of the fact that the axial load demands were less than the design axial loads. This result was attributed to the fact that unequal interstory drifts (ISD) causing rotations at floor levels resulted in significant moment demands in continuous multistory columns, having maximum demands at floor levels, which led to column failures. Note that the *AISC Seismic Provisions* (AISC 341-10) [23] states that ELF and the