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Evaluation of behavior of the secondary columns in semi-supported steel shear walls

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ABSTRACT

Semi-supported steel shear walls (SSSW), whose steel plate is connected to secondary columns rather than main columns of the frame, have been considered as an alternative steel shear walls to the traditional type. Many investigations have been made for proportionate designing of components of SSSW system. One of the important issues in this regard is the out of plane buckling of the secondary columns. In this paper, the plastic theory of structures is utilized to find out the axial force distribution, along the compressive column. Then, using energy method, for an assumed shear wall with specific geometry and material and a given shear force, the maximum overturning moment that makes the compressive secondary column buckles, can be determined. Repeating this method, for various shear forces, makes it possible to draw some interaction curves between overturning moments and shear forces. These curves can be used to analyze and design of semi-supported steel shear walls.

1. Introduction

Since early 1970s, steel plate shear walls (SSW) have been used in building industry around the world and many experimental and numerical investigations have been made on its seismic behavior [1–10]. This system is a very good lateral resisting system for energy dissipation in cyclic loadings. By use of this system, tension field action applies large stresses on surrounding members. To prevent developing of plastic hinges in the primary columns and thus collapse of the structure, strong columns must be used. This leads to abnormal and non-economical column design for structures [7].

In recent decade, to reduce the size of columns of the structure, the idea of separation of steel plate of shear wall from the surrounding frame, has been suggested [11–15]. Moharrami et al. [16,17] proposed the semi-supported type of steel shear wall (SSSW), whose steel plate, as shown in Fig. 1, is connected to secondary columns rather than main columns of the frame. In this system, the main columns are primarily used to carry gravity loads, while the secondary columns are means of developing tension field in the steel shear panel. By use of this system, abnormal and non-economical column design can be avoided. To investigate the behavior of SSSW system, they performed some experimental and numerical studies. Their research showed that despite of using small size columns for SSSW system, the tension field phenomenon can be developed similar to the traditional type of SSW system, and considerable energy can be dissipated.

To evaluate the ultimate capacity of this type of steel shear wall, Jahanpour et al. [18]proposed a step by step method assuming fully rigid beams, and utilizing lower and upper bound solution method. This method is able to give the internal forces in the secondary columns on the plastic hinge sections as well as ultimate shear capacity of SSSW especially when it undergoes some overturning moment. Using this method, it is also possible to predict the direction of tension field in the shear wall panel and the distance between pairs of plastic hinges on compressive and tensile columns. The authors showed that the results of their method are quite close to the results of FEM, but with much less analysis time. This method can be used to proportionate the design of components of the system (i.e. wall plate thickness and secondary columns). In another investigation, the interaction between the wall plate and the surrounding frame was investigated experimentally for typical SSSW systems in which the wall-frame had a dominant bending behavior [19].

In this paper, it is shown that tension field band extends beyond inner plastic hinges of columns and by using force (flexibility) method in structural analysis, the internal force distribution on the compressive and tensile columns can be calculated and accordingly the axial force, shear force and bending moment diagrams of the columns can be obtained. The varying internal axial forces on different sections of compressive secondary column may cause undesirable buckling of the column in out of plane direction and reduce the ultimate shear capacity of the wall. Accordingly, utilizing the Rayleigh–Ritz method, the formulation of critical load in compressive columns will also be derived.

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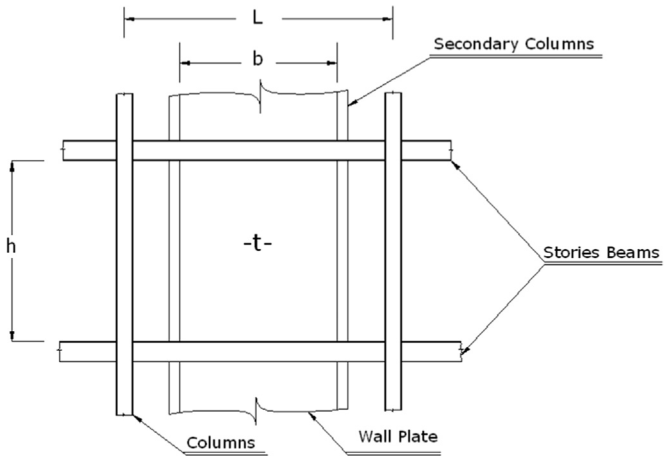

Fig. 1. General shape of semi-supported steel shear wall.

Finally, an algorithm will be proposed by which, the required specifications of compressive column can be obtained and the corresponding ultimate shear capacity of SSSW system can be determined.

2. Internal forces in the secondary columns

Fig. 2 shows a multi-storey SSSW with fully rigid beams under lateral loads, in which one of its middle stories has yielded. The mechanism state for this storey can be shown by four plastic hinges in the columns and a yielded zone on the plate. Using the Jahanpour's step by step method [18], all internal forces at plastic hinge sections as well as geometric parameters can be calculated.

Fig. 3 shows the free body diagram of the storey components shown in Fig. 2. As shown in Fig. 3, there are two distributed loads, $t\tau_{crm}$ (shear buckling force) and $t\sigma_{crm}$ (tension field force) on wall edges. These loads cause reaction on surrounding members (beams and columns) and internal forces (axial, shear force and bending moment) on different sections of columns and beams.

In Fig. 3, it is evident that for pin connections between beams and columns, the moments on the columns ends, can be removed (i.e. $M_{pc}=M_{pt}=M_A=M_D=0$). To evaluate the ultimate capacity of the wall by step by step method [18], the outer plastic moments for compressive and tensile columns, M_{pc} and M_{pt} (Fig. 3) as well as inner plastic moments (M_{mc} and M_{mt}) and their positions i.e. c_c and c_t and other corresponding internal forces should be determined. There are also two set of unknown internal forces on the A and D ends of the compressive and tensile columns. Furthermore, since in the wall plate (Fig. 3), the yield band (tension field region) extends to points W' and Y', beyond the positions of inner plastic hinges, (W and Y points), accordingly, the distance from point W' to X in compressive and Y' to Z in tensile columns that are named b_c and b_t are also unknown parameter. Thus, each column has four parameters that have not been calculated: M_A, V_A, F_A and b_c for compressive and M_D, V_D, F_D and b_t for tensile column. These parameters can be calculated using the static equilibrium equations and flexibility relations for the two columns. After these calculations, all of internal force in the columns can be determined. Then, according to the variation of internal forces, the out of plane buckling of compressive columns can be investigated.

Fig. 4 shows qualitative diagram of bending moment in a compressive column. As mentioned in Section 2, there are three unknown forces (M_A, V_A and F_A) and one unknown geometric parameter (b_c). Using shear and moment equilibrium equations for Fig. 4c, following equations can be derived:

$$V_A = t\sin^2\theta \, . \int_{c_c}^{b_c} \sigma_{tym} dx \tag{1}$$

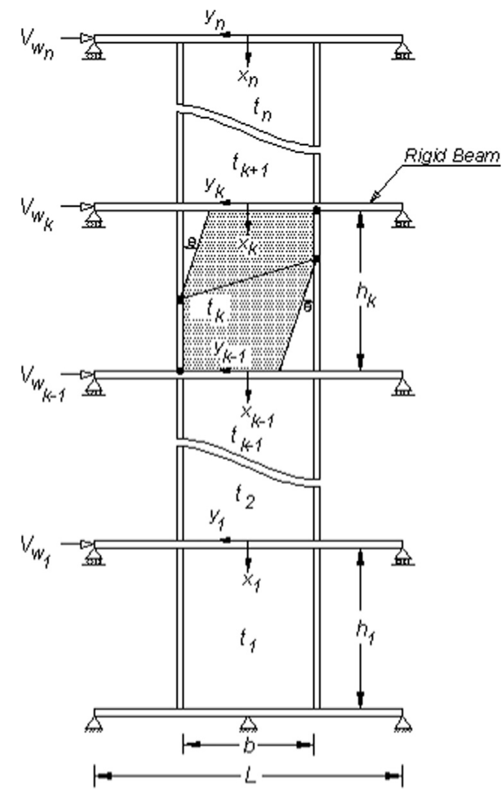

Fig. 2. A semi-supported steel shear wall with rigid beams under lateral forces.

$$M_A = M_{mc} - t\sin^2\theta \, . \int_{c_c}^{b_c} (h-x)\sigma_{tym} dx \tag{2}$$

where t=plate thickness; θ=the angle of the tensile membrane stress field with vertical axis; σ_{tym}=the modified tensile membrane stress that causes yielding. All of the above parameters have been calculated in reference [18].

Knowing that $\theta_A=0$, following equation can be extracted from flexibility relation.

$$\theta_A = \theta_{A0} + M_A\theta_{11} = 0 \quad \Rightarrow \quad M_A = -\frac{\theta_{A0}}{\theta_{11}} \tag{3}$$

where θ_{A0} and θ_{11} are slopes of point ‘A’ in primary structure under the applied load and unit load respectively and can be calculated from following equations using virtual work method.

$$\theta_{A0} = \int_{c_c}^{h} \frac{M(x)m^*(x)}{EI_{yc}} dx \tag{4}$$

$$\theta_{11} = \int_{c_c}^{h} \frac{m(x)m^*(x)}{EI_{yc}} dx \tag{5}$$

In the above equations, $m(x)$ and $m^*(x)$ are the bending moment functions that are obtained by unit load and virtual unit load respectively. Here, both of them in W-A segment of the column are unit (i.e. $m(x)=m^*(x)=1$). Also, E and I_{yc} are modulus of elasticity and moment of inertia of the column section about y-axis (perpendicular to the wall plane) respectively. It is assumed that $EI_{yc}=cte$. Finally, $M(x)$ is the bending moment function for W-A segment and can be expressed as below.

$$M(x) = \begin{cases} M_{mc} - t\sin^2\theta . \left(x\int_{c_c}^{x} \sigma_{tym} dx_1 - \int_{c_c}^{x} x_1 \sigma_{tym} dx_1 \right) & ; \quad c_c \le x \le b_c \\ t\sin^2\theta . (h-x)\int_{c_c}^{b_c} \sigma_{tym} dx_1 & ; \quad b_c \le x \le h \end{cases} \tag{6}$$