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ANALYSIS AND DESIGN OF STEEL PLATE SHEAR WALLS USING ORTHOTROPIC MEMBRANE MODEL

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Abstract

Present paper deals with a type of thin steel plate shear wall (SPW) called "special plate shear wall" (SPSW). Although several methods have been proposed to predict the behavior of SPW, but lack of a comprehensive method containing a complete design procedure, have always confused the designers. Absence of the mentioned method has also restricted usage of steel plate shear wall significantly. Recently a new design technique using "*orthotropic membrane model*" has been proposed in "*AISC Steel Design Guide 20*". In this method, after preliminary design of the system, in order to correctly distribute the forces between the wall members, an orthotropic membrane model is developed using ETABS program. Tension field angle in each web plate is calculated after specifying web plates and boundary elements characteristics. This angle affects the distribution of applied forces to wall members and the web plates shear strength. Finally, the effect of changes in web plate thickness on the behavior of SPSW is assessed.

Keywords: Steel plate shear wall, web plate, orthotropic membrane model, tension field action.

1. INTRODUCTION

Steel plate shear wall (SPW) has been used in steel buildings since the last 4 decades as a lateral load resisting system. This system consists of a steel plate (web) connected to its surrounding frame (HBE and VBE). The term web plate is used to refer to the steel plate that resists the lateral loads in the wall (Sabelli and Bruneau 2006). In a frame, the HBE and VBE correspond to the surrounding beams (horizontal boundary elements) and columns (vertical boundary elements), respectively. In recent years, applications of unstiffened slender-web (thin steel plate) shear walls are very common. These types of walls have

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negligible compression strength and buckle under very low lateral loads. Diagonal tension field action is the preliminary mechanism to resist lateral loads in this system. This type of walls called “SPSW” in AISC Seismic provisions (AISC 2005a) and are utilized as a basic Seismic force Resisting System.

2. SEISMEC DESIGN OF SPECIAL PLATE SHEAR WALL

In this research, SPSW system is applied to 3, 6 and 9-story buildings considered in a zone of high seismicity. The height of each story is 3.8 m. Response modification coefficient (R) for all buildings is considered to be equal to 7 (AISC 2005a; ASCE 7-05). Building site characteristics are according to the chapter 5 of the AISC Steel Design Guide 20 (Sabelli and Bruneau 2006). Web plate and boundary elements materials are ASTM A36 and ASTM A992 steel, respectively. Figure 1 represents the common plan considered for all buildings in this study. SPSWs were designed according to “the capacity design principles”.

Equivalent lateral force procedure is used to analyze the SPSW (ASCE 7-05). Based on this method, the lateral forces on each SPSW levels in different buildings are given in Table 1.

Table 1: Lateral forces in each SPSW (KN)

Level	Roof	Ninth story	Eighth story	Seventh story	Sixth story	Fifth story	Fourth story	Third story	Second story
3-story SPSW	265.8	----	----	----	----	----	----	177.2	88.6
6-story SPSW	303.7	----	----	----	253.1	202.5	151.9	101.2	50.6
9-story SPSW	332.2	292	252.3	213.1	174.5	136.7	99.8	64	30

2.1. Preliminary design of SPSW

When the SPSW is subjected to the design earthquake forces, significant inelastic deformations are expected to withstand by the web plates. VBEs (vertical boundary elements) and HBEs (horizontal boundary elements) must be designed in a way to remain elastic under the maximum forces that can be generated by the fully yielded web plates and only plastic hinging is allowed at the ends of HBE (AISC 2005a). In the preliminary design it is assumed that the web plate resists entire of the shear in each story because the sizes of HBE and VBE are not specified. The design shear strength and tension field angle (α) in the web plate are calculated according to the limit state of shear yielding using the following equations:

$$\phi V_n = 0.90(0.42)F_y t_w L_{cf} \sin 2\alpha \tag{1}$$

$$\tan^4 \alpha = \frac{1+\dfrac{t_w L}{2A_c}}{1+t_w h\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)} \tag{2}$$

Where tw is the web plate thickness; Lcf is the clear distance between VBE flanges; h is distance between HBE centerlines; Ab is cross-sectional area of a HBE; Ac is cross-sectional area of a VBE; Ic is the VBE moment of inertia; and L is distance between VBE centerlines (AISC 2005a). The value of angle α is necessary to calculate φV_n and at this stage the angle α is conservatively assumed to be 30°. The value of Lcf is assumed to be bay length minus 40cm. The value of φV_n is obtained for each web plate using Eq.1. The preliminary amount of tw in each story is determined by comparing the SPSW shear force in each level (Table 1) with φV_n calculated for different web thicknesses.

Design of the VBE must satisfy both the strength and stiffness criteria. However, at this step only the stiffness requirement, necessary to prevent the VBEs from buckling, is controlled based on AISC (AISC 2005a):

$$I_c \geq 0.00307 t_w h^4 / L \tag{3}$$

The preliminary design of HBE is based on the difference between vertical components of tensile forces resulting from web plates above and below HBE. This force is distributed along the length of HBE and reaches to its maximum value when web plates are yielded. This force can be calculated as follows:

$$w_u = R_y F_y \left(t_i - t_{i+1}\right) \cos^2 \alpha \tag{4}$$

Where Ry is the ratio of the expected yield stress to the specified minimum yield stress of the steel plate (AISC 2005a).

After preliminary selection of SPSW members, more accurate estimation of the wall properties is possible using equations 1-4. The boundary-element sections and tw are modified at this step. The aim of design modification at this step is to reduce the number of required iterations during the analysis (Sabelli and Bruneau 2006). Using this method only two or three iterations are required to improve the design of SPSW system.

2.2. Analysis

In the preliminary step of design of SPSW, as the dimensions of HBE and VBE are not available, it is assumed that the total story shear is supported by the web plate. In the next step a numerical model was proposed in order to properly distribute the forces between SPSW components and rigid beam-to-column connections. In the present study, an elastic analysis method using ETABS software has been utilized. Here, the analyses were carried out using a new method named Orthotropic Membrane Model, as mentioned in AISC Steel Design Guide 20, instead of conventional strip modeling method. In the Orthotropic Membrane Model method, the stiffness assigned to the compression diameter is less than that of the tension diameter of the web plate (Astaneh-Asl 2001). The local axes of membrane elements have been rotated considering the calculated angle α in each story. An orthotropic membrane model of SPSW is shown in Figure 2. The advantages of this method over the strip modeling method are as follows:

a) In strip model, the web plate is replaced by a series of diagonal tension-only strips. With modification of VBE sections, the angle α will change (Eq.2). This change causes modification of the strip element properties and the node location on the VBE. So, this is a tedious method (Sabelli and Bruneau 2006).

b) Orthotropic membrane model depends also on α angle. But design iterations are related to recalculation of α and reorientation of local axes of membrane elements. This practice is easily performed by most of the available structural modeling programs.

The results of orthotropic membrane and strip models are the same. Both of them are coincident to the SPSW behavior in testing (Sabelli and Bruneau 2006).

Considering structural analysis results, it was found that story drift limitation governs designing related to some stories. This prevents the reduction of the web plate's thickness. All sections in Tables 2 to 4 satisfy strength and drift requirements.

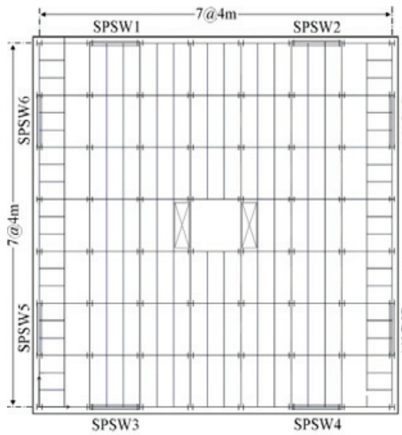

Figure 1: Plan of all buildings

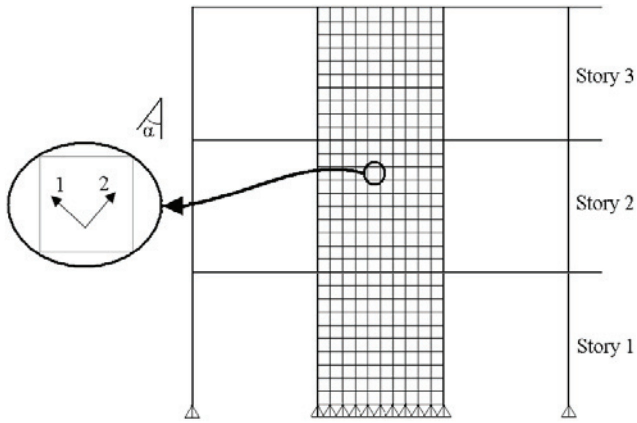

Figure 2: Orthotropic membrane model of SPSW

Table 2: Final dimensions of SPSW members for 3-story building

Level	Web plate thickness tw(mm)	VBE	HBE	Panel dimensions			
				Lcf(cm)	L(cm)	hc(cm)	h(cm)
Roof	—	—	W310×52	—	—	—	—
Third floor	0.76	W250×80	W310×52	380	348.2	400	374.4
Second floor	1.29	W250×131	W250×38.5	377.2	348.2	400	372.5
First floor	1.55	W250×149	W410×75	387.5	353.8	400	371.8

Table 3: Final dimensions of SPSW members for 6-story building

Level	Web plate thickness tw(mm)	VBE	HBE	Panel dimensions			
				Lcf(cm)	L(cm)	hc(cm)	h(cm)
Roof	—	—	W410×60	—	—	—	—
Sixth floor	0.88	W310×143	W310×67	375	339.7	400	373.6
Fifth floor	1.64	W310×158	W310×67	380	349.3	400	367.2
Fourth floor	2.28	W310×226	W310×67	380	349.3	400	367.2
Third floor	2.76	W310×283	W310×67	380	349.3	400	366.7
Second floor	3.13	W310×313	W310×67	380	349.3	400	366.7
First floor	3.2	W310×375	W610×92	394.8	349.3	400	363.5

Table 4: Final dimensions of SPSW members for 9-story building

Level	Web plate thickness tw(mm)	VBE	HBE	Panel dimensions			
				Lcf(cm)	L(cm)	hc(cm)	h(cm)
Roof	—	—	W410×60	—	—	—	—
Ninth floor	0.96	W310×143	W410×60	380	339.3	400	367.7
Eighth floor	1.83	W310×202	W360×72	377.4	339.3	400	365.9
Seventh floor	2.61	W360×237	W360×72	380	344.9	400	362
Sixth floor	3.29	W360×287	W360×72	380	344.9	400	360.7
Fifth floor	3.88	W360×347	W360×72	380	344.9	400	359.3
Fourth floor	4.35	W360×421	W310×74	378	344.9	400	357.5
Third floor	4.69	W360×509	W460×97	387.8	349	400	355.5
Second floor	4.88	W360×551	W310×74	372.2	333.3	400	354.5
First floor	5.0	W360×634	W610×153	395.6	349	400	352.6

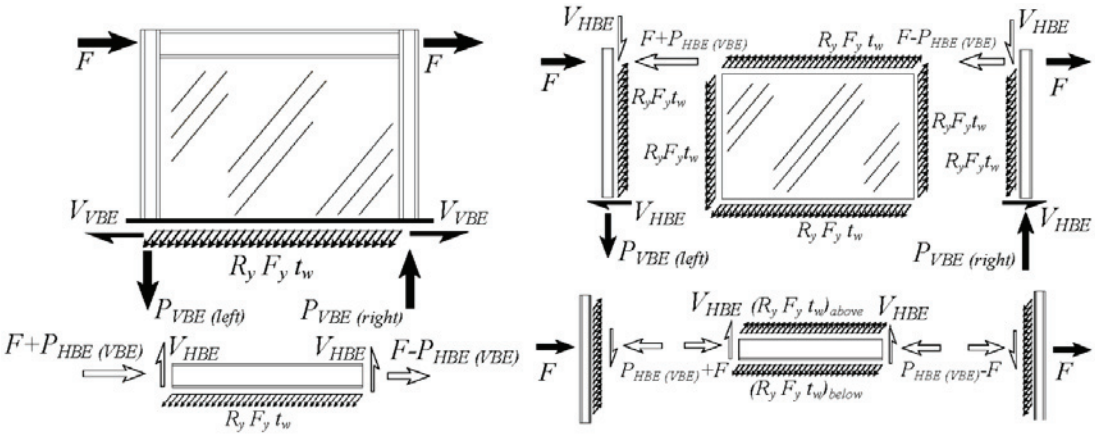

Figure 3: Applied forces on SPSW (Ericksen and Sabelli 2008)

2.3. Final design of SPSW

Typical applied forces on SPSW are shown in Figure 3. (The end moments are not shown)

2.4. HBE design

Beams in SPSW are subjected to axial and flexural forces resulting from web plate tension and gravity loads, as well as shears and moments caused by deformation of the frame. It can be assumed that the bending forces produced by deformation of the frame, cause plastic hinging in both ends of the beam. If the assumed simple span beams have sufficient strength to withstand web plate tension, we can ignore the bending forces due to frame deformations. Thus, the moment in the middle span is equal to:

$$M_u = \frac{w_u L_h^2}{8} + P_u^* \left[\frac{L}{3} - \frac{d_c}{2} - \frac{d_b}{2} \right] \tag{5}$$

Where P_u* is the secondary beams force; d_b and dc are beam and column height, respectively; and L_h is the distance between plastic hinge locations in the beam and is equal to $L_h = L - 2S_h$ and $S_h = 1/2(d_c + d_b)$.

Axial force in beam is calculated as follows:

$$P_u = P_{HBE} = P_{HBE(VBE)} \pm \frac{1}{2} P_{HBE(web)} \tag{6}$$

$$P_{HBE(VBE)} = \sum \frac{1}{2} R_y F_y \sin^2(\alpha) t_w h_c \tag{7}$$

$$P_{HBE(web)} = \frac{1}{2} R_y F_y \left[t_i \sin(2\alpha_i) - t_{i+1} \sin(2\alpha_{i+1}) \right] L_{cf} \tag{8}$$

h_c is the clear distance between HBE flanges above and below the web plate. It is assumed that the horizontal component of distributed force applied by the web plate to the columns will be transferred equally to connected beams (HBE) of upper and lower floors. The horizontal component of the force of the web plates applied to the beam is also divided equally between ends of the beam.

Then the required second-order axial and flexural strengths must be calculated. The amplified first-order elastic analysis method is considered as acceptable methods for second-order elastic analysis of braced framing systems (AISC 2005b).

Shear force in beam was calculated as follows:

$$V_u = \frac{2M_{pr}}{L_h} + P_u^* + \frac{w_g + w_u}{2} L_{cf} \tag{9}$$

$$M_{pr} = 1.1 R_y F_y Z_x \tag{10}$$

Where wg is the gravity distributed load applied on beam and Mpr is the flexural strength in plastic hinge. Mpr can be reduced considering the axial force of the beam at beam-to-column connection (Sabelli and Bruneau 2006). To calculate the reduced value of Mpr, one should refer to chapter H of AISC 2005b.

2.5. VBE design

Axial force of column is as follows:

$$E_m = \sum \frac{1}{2} R_y F_y \sin(2\alpha) t_w h + \sum V_u \tag{11}$$

The first term in the above equation, represents the effect of axial force due to web plates. The second term is the total shear forces caused by the earthquake in all the beams above the considered column. Therefore, the equation 11 can be rewritten as follows:

$$E_m = \sum \frac{1}{2} R_y F_y \sin(2\alpha) t_w h_c + \sum \left[\frac{2M_{pr\,HBE}}{L_{h\,HBE}} + P_u^* + \frac{w_u}{2} L_{cf} \right] - \sum \left[\frac{2M_{pr\,Adj}}{L_{h\,Adj}} \right] \tag{12}$$

In this equation, Mpr Adj is expected flexural strength in beams adjacent to the wall. Column bending is due to web plate tension and HBE plastic hinging. The moment at the end of column resulting from web plate tension is equal to:

$$M_{VBE(web)} = R_y F_y \sin^2(\alpha) t_w \left[\frac{h_c^2}{12} \right] \tag{13}$$

The moment resulting from HBE plastic hinging is calculated based on flexural strength of the beams at connection. We can consider that the moment in each segment of the column is equal to one-half of the flexural strengths of the beams at the connection (AISC 2005a):

$$M_{VBE(HBE)} = \frac{1}{2} \sum M_{pb} \tag{14}$$

$$M_{pb} = M_{pr} / (1.1 R_y) + V_u S_h \tag{15}$$

Finally the shear force of VBE will be calculated. This force is due to web plate tension and a portion of shear story that is not resisted by web plate. This part of VBE shear force corresponds to HBE plastic hinging:

$$V_u = V_{VBE(web)} + V_{VBE(HBE)} \tag{16}$$

$$V_{VBE(web)} = \frac{1}{2} R_y F_y \sin^2(\alpha) t_w h_c \tag{17}$$

$$V_{VBE(HBE)} = \sum \frac{1}{2} \left[\frac{M_{pc}}{h_c} \right] \tag{18}$$

M_{pc} is the VBE flexural strength.

2.5.1. Required controls for beams and columns

The required controls are as follows:

a) Compactness check (AISC 2005a)

b) Lateral bracing check (AISC 2005a). This criterion is carried out for beams.

c) Shear strength check (AISC 2005b)

d) Combined compression and flexure check (AISC 2005b)

e) Minimum beam moment of inertia. There are no certified criteria for required stiffness in HBE. But it is recommended that the HBE must have minimum moment of inertia as follows (Sabelli and Bruneau 2006):

$$I_{HBE} \geq 0.003(\Delta t_w) L^4 / h \tag{19}$$

In above equation, Δt_w is the difference between the web plate thickness above and below the HBE.

f) Minimum thickness of HBE web. This criteria is recommended to be applied as follows (Sabelli and Bruneau 2006):

$$t_{w\,HBE} \geq \frac{t_w R_y F_y}{F_{y\,HBE}}$$

$R_y F_y$ is the expected yield stress of the web plate material; and $t_{w\ HBE}$ is the thickness of the HBE web. When HBE and VBE, passed all the above controls, their design procedure will be completed.

The changes of web plate thicknesses in each story of SPSW for buildings of 3, 6 and 9 story are computed and represented in Figure 4.

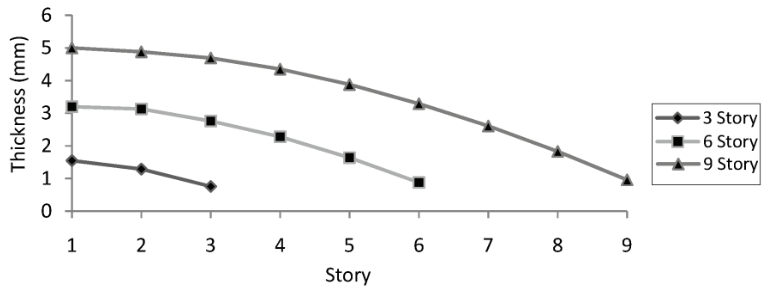

Figure 4: Changes of web plate thickness of SPSW in different stories

3. CONCLUSIONS

In this paper the complete steps of wall design was described and it was found that one can design the SPSW, only by having the base shear of wall. Finally the changes of web plate thicknesses in SPSW were evaluated. Obtained results show that as moving towards lower stories in a building, the web plate thicknesses will increase. The differences between web plate thicknesses are reduced at lower stories of a building. The trends of changes in web plate thicknesses are nearly the same in all the three buildings.

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