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The Use of Limit States (LRFD) Method in Unstiffened Steel Shear Wall Analysis and Design with Special Ductility

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ABSTRACT

Steel plate shear wall (SPSW) is a lateral force resisting system that is not only applicable in construction of buildings which are designed from the outset, but also they are efficient systems in seismic retrofit of structures. This lateral force resisting system with individual specifications and performance is used in order to increase lateral resistance and stiffness to lateral forces especially earthquake. The main role of SPSW similar to the other lateral force resisting systems is to supply the strength and stability against shear forces that are produced in stories because of the earthquake and they resist against overturning moment that is produced from the force mentioned. Canada steel structures design codes (CAN/CSA S16-01/2001and FEMA450) offered criteria in design of structures in 2004. The provisions related to the design of these shear walls were added particularly to steel structures seismic design codes as AISC341-2005in 2005. In this paper we aim to analyze, calculate, design and evaluate a 9 stories steel structure that has special steel plate shear walls, i.e. SPSW with special ductility and we use limit states specifications (LRFD method) while the design of horizontal boundary elements is specially considered. The emphasis of this paper is to design and control one of the main elements of this system including on beam in one of the stories as a sample by using from limit states method (LSD) named LRFD provisions. This paper consists of: the calculation of bending moment due to uniform load and two centralized axial forces, strength, reduced bending strength, compact control, shear control and bending moment, and axial force interaction control for HBE.

KEYWORDS: unstiffened special plate shear walls, special ductility, limit state provisions, web plate thickness, diagonal tension field action, vertical and horizontal boundary elements, Inelastic deformations.

1. INTRODUCTION

1-1- Steel Plate Shear Wall presentation

Steel plate shear wall is a system consisting of steel plates (stiffened or unstiffened), steel boundary columns and steel beams which is constructed in the level of each story. The lateral load resisting system SPSW could be considered similar to plate- girders in which its frame columns behave like plate girders flanges, its beams similar to plate girders intermediate stiffener and its steel plate is equivalent to plate girders web. The main components of SPSW system consists of steel plate called serve as web plate, columns serve as vertical boundary member or VBE and beams serve as horizontal boundary members or HBE (Fig. 1). [1], [3]

One of the simple analytical models that was in accordance with the high stiffness of vertical boundary components (VBE), proposed by Torbern et al. in 1983, in which the produced tension field behavior in the web plate was modeled by a large number of truss members with identical slope angle. In the above model tension yield strength steel shear wall web plate is considered serve as limit stress in truss members shown in Fig. 2. Canada shear walls design code (CAS2001) has recommended the strip model as suitable method in analysis. Steel shear walls can be utilized to different types. As regards the buckling limit strength in plates is much less than their post buckling strength, the use of post buckling capacity in steel plates has been taken into consideration. It should be noted that plate buckling is not meant to result the destruction and failure of a structure and if the plate has enough lateral support, its post buckling forces could be up to several times the theoretical buckling limit forces. Thin steel shear wall buckles at low loads and after which strength - load behavior from inter - plate shear changes to diagonal tensional field, that this field could withstand loads up to achieve to steel yield limit.

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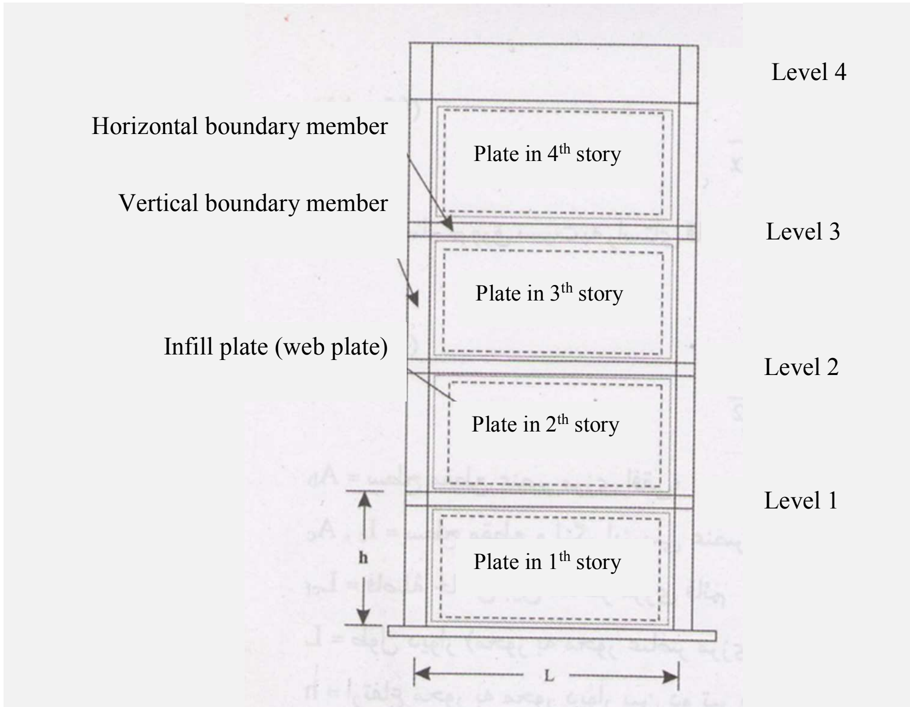

Fig. 1. The main components of SPSW system

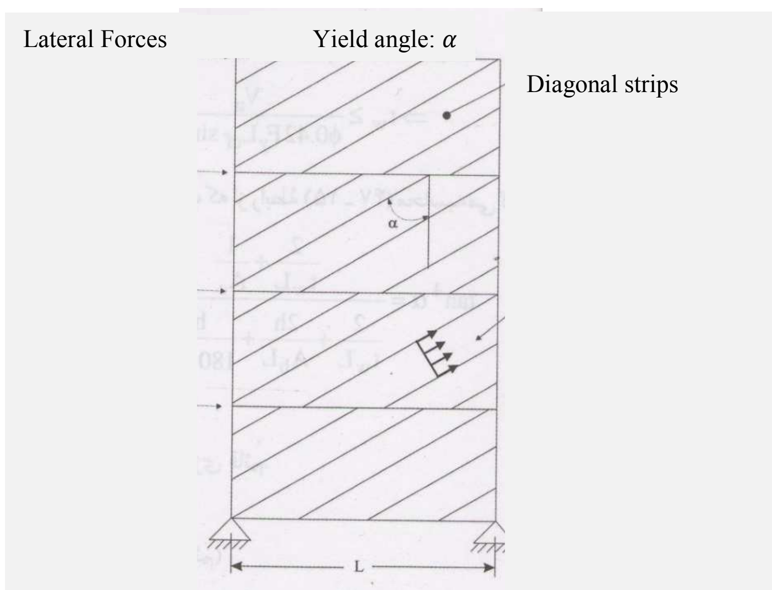

Fig. 2. Steel shear wall strip model

It should be noted that structure damage ought to happen by the formation of diagonal tension field and destruction of wall plate which in process columns remain safety. In this case, the correct philosophy of loading is that when the steel shear wall is subjected to the lateral loading, its force transmission is in a manner that tensional and compressive force will be formed in columns and it is transferred to the adjacent foundations. Existing shear is withstood by the formation of diagonal tension field in wall plate too. The pure shear in wall plate resists and loads with to be decomposed to tensional and compressive diagonal forces couple. The buckling hazard could produce for wall plate along the compressive component. Therefore, with usage of the stiffener, shear wall could be considered as stiffened in which its buckling strength will be increased [2].

1-2- Limit states method (LRFD)

Limit states are those situations in which the structure could not be able to do its duties. Limit states usually are classified to two parts consisting of strength and serviceability limit states. Strength limit state is included yield, buckling, fatigue, failure, over-turning and slip limit states. Serviceability limit state is included utilization and usage of the building like deformation, vibration, crack and corrosion. In study of limit states, random variables consisting of applied loads to the structure and structural strength should take in to consideration. According to the existing uncertainties in applied loads and structure strength, LRFD method utilizes load and resistance factors for supplying the safety of the structure. In this method nominal strength is multiplied to the resistance factor that it is less than or equal to 1.0 to obtain the design strength. On the other hand the applied loads are multiplied to the load factors that they are greater than or equal to 1.0 to obtain factored or ultimate loads. The LRFD method considers the design process to be more rational and real than the allowable stress design traditional method. The use of the LRFD method has more interest and approach in the world developed and advanced countries [5].

1-3- Ductility

Ductility is a characteristic of structure that it could accept inelastic deformations specially, in seismic loads without considerable reduction in strength and stiffness, and it could withstand the earthquake lateral forces with less relative strength. Ductility is evaluated with rotational capacity that exists in structure joints. Joint rotation is defined in the ratio of upper story relative displacement of that joint to story height. Three ductility limits in steel structures is considered that consist of high or special, intermediate and low ductility in accordance with the ductility capacity that will be expected. In high or special ductility limit, the rotational capacity that would be expected in joints is high and this rotational capacity stays in range of inelastic [5].

2.The provisions of the Limit States Method in Steel Shear Wall Analysis and Design

Steel shear walls consist of two categories, thin (unstiffened) and stiffened. Thin steel shear walls are categorized in two groups containing ordinary unstiffened SSW and special unstiffened SSW. Behavior factor (R) is considered equal to 6.0 for ordinary unstiffened SSW.

These walls should withstand against deformations and forces of the loads combinations. Special unstiffened SSW should withstand against inelastic deformations of wall plate due to earthquake. Vertical and horizontal boundary components adjacent the wall plate should remain elastic situation under the effect of forces that yield the web plate. The formation of plastic hinges in the end of the horizontal boundary elements is allowable in the special unstiffened SSW. Behavior factor (R) is considered equal to 10.0 for special unstiffened SSW [4]. The requirements that have been brought in paper text, are considered for the provisions of special unstiffened SSW design in LRFD method.

Shear strength of the web plate design is obtained by using the shear yield state from equation (1) and the plate yield angle (α), respect to vertical line is obtained from equation (2).

$$\emptyset V_n = \emptyset 0.42 F_y t_w L_{cf} sin2\alpha \rightarrow t_w \geq \frac{V_u}{\emptyset 0.42 F_y L_{cf} sin2\alpha} \quad (1)$$

$$\tan^4\alpha = \frac{\frac{2}{t_w L} + \frac{1}{A_c}}{\frac{2}{t_w L} + \frac{2h}{A_b L} + \frac{h^4}{180 I_c L^2}} \quad (2)$$

In the above equations L_{cf} is the pure distance between vertical boundary members, L is the length of wall (the length axis to axis between vertical boundary members, A_c and I_c are cross section and moment of inertia of the vertical boundary elements respectively, h is the height axis to axis of wall between two story beams (horizontal elements) andA_b is the cross section of the horizontal boundary elements. The ratio of length to height should be limited to 0.8 up to 2.5 in both ordinary and special unstiffened shear wall ($0.8 \leq \frac{L}{h} \leq 2.5$).

In connection with vertical boundary elements stiffness, the moment of inertia in them around the axis that is perpendicular on the wall plate sheet, should provide the equation $I_c \geq 0.00307 t_w h^4/L$, in both ordinary and special unstiffened shear wall.

In connection with horizontal boundary elements stiffness, the moment of inertia in them around the axis that is perpendicular on the wall plate sheet, should provide the equation $I_b \geq 0.003(\Delta t_w)h^4/L$, in both ordinary and special unstiffened shear wall, in whichΔt_w is equal to the difference of upper and bottom plates in horizontal boundary element [4].

3-steel shear wall analysis methods

There are three methods for modeling and analyzing the steel shear wall that are tensional strip model, finite element method with orthotropic membrane element and nonlinear analysis that the tensional strip model and orthotropic membrane are explained briefly.

In the tensional strip model, the wall plate is modeled with the use of some parallel diagonal tensional strips. At least ten tensional strips is utilized for modeling the wall plate to estimate the effects of distributed loads on the boundary elements. The cross section of these strips are determinable by the equation 3.

$$A_s = \frac{[Lcos\alpha + hsin\alpha]t_w}{n} \quad (3)$$

In the above equation L , h and t_w are length, height and thickness of shear wall web plate and n is the numberof these strips.

Orthotropic membrane model use inhomogeneous orthotropic members to model the difference between tensional and compressive strength for wall plate. By virtue of this subject that the tension is in the diagonal line, the local axis of orthotropic elements ought to be in conformity with α angle. The material properties of elements should be considered similar to wall materials properties. From stiffness that is perpendicular on the wall plate is relinquished [4], [6].

4- Internal forces in vertical and horizontal boundary elements

Due to external gravity loads and tensional performance of web plate under lateral loads, internal forces are created in vertical and horizontal boundary elements (Fig 3-1, 3-2), [4].

4-1- Horizontal boundary elements

Bending moment is obtained in the member according to limit states from equation (4).

$$M_u = \frac{(w_u + w_g)L_h^2}{8} + M^{'} \tag{4}$$

In the above equation w_g is uniform distributed load, w_u is distributed load due to tensional performance of the web plate (Eq. 5) and $M^{'}$ is moment due to external load.

$$(w_u = R_y F_y (t_i - t_{i+1}) cos^2\alpha) \tag{5}$$

Axial force is obtained in the horizontal boundary element from Eq. 6, 7 and 8.

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

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

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

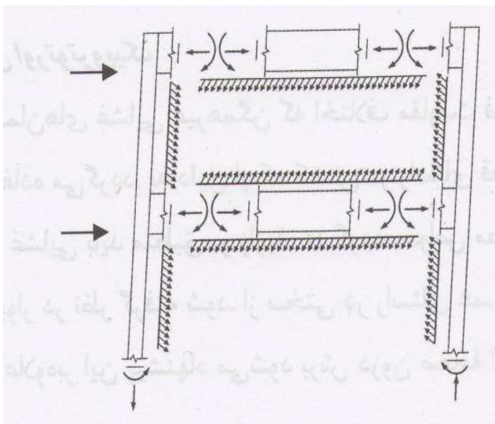

Fig. 3-1.

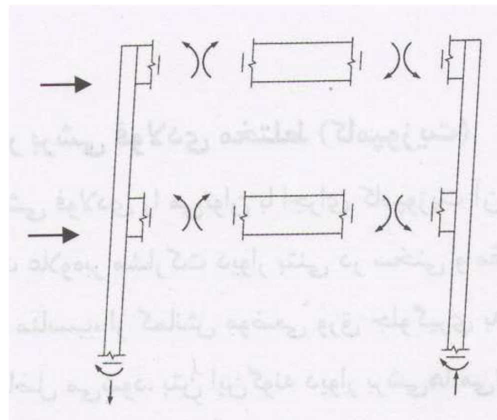

Fig. 3-2.

Fig. 3-1. Internal forces in the vertical and horizontal boundary elements due to tensional performance of the web plate
Fig. 3-2. Internal forces in the vertical and horizontal boundary elements due to bending deformation of the frame

4-2- vertical boundary elements

Bending moment in this member is created from tension in web plate and plastic hinge in the horizontal boundary member, Eq. 9, 10 and [4].

$$\mathrm{M_{VBE}(web)} = \frac{\mathrm{R_y F_y sin^2(\alpha) t_w h_c^2}}{12} \tag{9}$$

$$\mathrm{M_{VBE}(HBE)} = \frac{1}{2}\mathrm{M_{PB}} \tag{10}$$

5- case study

In this study we present a plane of a structure that has a series of steel walls with special ductility (Fig. 4) and a schematic view of one of its frames is shown in Fig. 5. By obtaining the share of each shear wall from story shear force, the analysis and design of the web plate of it and its horizontal boundary element will be done according to the limit states method that is load and resistance factors LRFD. The consumable steel yield stress of the web plate is 2400 ${kg}/_{cm^2}$ (ST37) and it for wall boundary elements (beams and columns) is considered 3500 ${kg}/_{cm^2}$ (ST52) [4]. The plane of the structure contains 45×45 m dimensions and it has four spans from typically special steel shear wall in x and y directions that its spans are 6 m length.

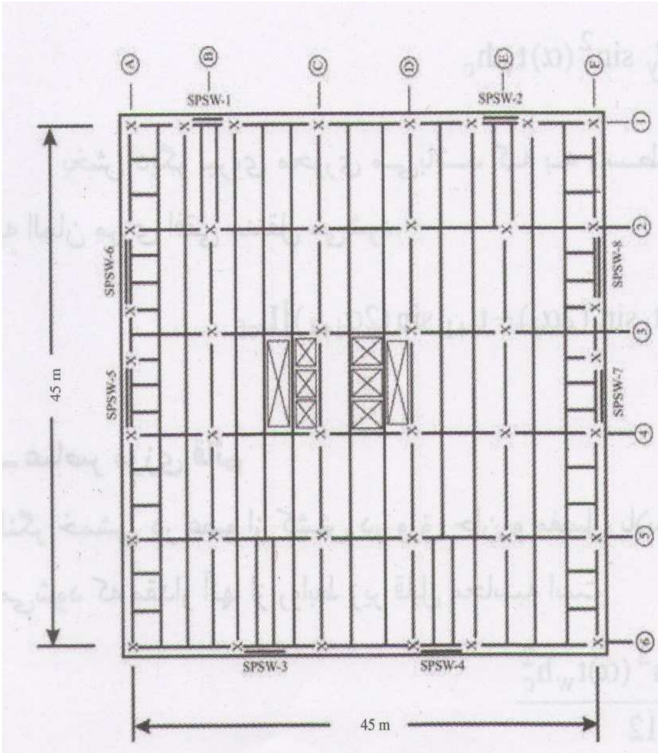

Fig. 4:The plane of the structure containing a series of steel shear walls with special ductility

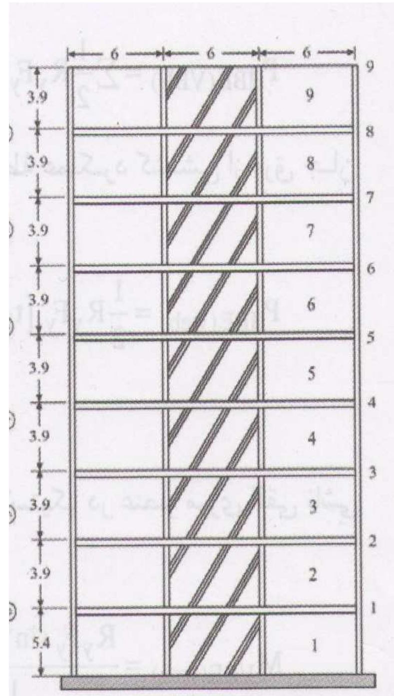

Fig.5: A view of one of the frames

The shear force associated with each of the shear walls, is observed in table1 according to the lateral force in the level of each story.

Table1. The shear force associated with each of the shear walls according to the lateral force in the level of each story

level	Lateral force (ton)	story	Shear (ton)
9	89.4	9	89.4
8	69.0	8	158.4
7	60.4	7	218.8
6	51.3	6	270.1
5	42.7	5	312.8
4	33.6	4	346.4
3	25.0	3	371.4
2	17.0	2	388.4
1	9.4	1	397.8

We will do the below steps to analyze and design the web plate of the shear wall and vertical and horizontal boundary components too with the use of the table1.

5-1- To determine the initial thickness of the web platet_w
5-2- To determine the minimum moment of inertia for vertical boundary elements
5-3- To estimate the initial cross-section of the vertical and horizontal boundary elements and final thickness of the web plate
5-4- To estimate the angle of diagonal boundary field **α**
5-5- To design the horizontal boundary elements HBE
5-6- To design the vertical boundary elements VBE

5-1- To determine the initial thickness of the web platet_w

We suppose at first that all of the story shear force is withstood by the shear wall. We use the equation (1) to determine the initial thickness of web plate. We consider the amount of **α**angle to be 30º and the section dimension of vertical boundary elements to be 45 cm in initial calculations.

Table 2. determining the initial thickness of the web plate and controlling the ratio of available shear strength to design strength

level	Plate thickness t_w (mm)	Existing shear strength $V_u(ton)$	Design *shear* strength (ton) $\emptyset V_n = 0.9 \times 0.42 F_y t_w L_{cf} \sin(2\alpha)$	Ratio of shear stress $\frac{V_u}{\emptyset V_n}$
9	2	89.4	87.21	1.03
8	4	158.4	174.42	0.91
7	5	218.8	218.02	1.00
6	6	270.1	261.62	1.03
5	6	312.8	261.62	1.02
4	8	346.4	348.83	0.99
3	8	371.4	348.83	1.06
2	8	388.4	348.83	1.11
1	10	397.8	436.04	0.91

5-2- To determine the minimum moment of inertia for vertical boundary elements

Minimum moment of inertia is obtained from Eq.11 for vertical boundary elements.

$$I_c \geq 0.00307 \frac{t_w h^4}{L} \quad (11)$$

In the above relationship h is equal to axis to axis distance in horizontal boundary elements and L is axis to axis distance in vertical boundary elements. The required moment of inertia for VBEs has been determined in table3 according to the above relationship (Eq.11).

Table 3. The required moment of inertia for VBEs

Story	*Web plate thickness* t_w (cm)	*Steel Shear Wall Dimensions* h (cm)	L (cm)	The required moment of inertia for VBEs $I_c = 0.00307 \frac{t_w h^4}{L}$
9	0.2	390	600	23674.2
8	0.4	390	600	47348.4
7	0.5	390	600	59185.5
6	0.6	390	600	71022.6
5	0.6	390	600	71022.6
4	0.8	390	600	94696.9
3	0.8	390	600	94696.9
2	0.8	390	600	94696.9
1	1.0	540	600	435073.0

5-3- To estimate the initial cross-section of the vertical and horizontal boundary elements and final thickness of the web plate

This assumption had been considered that story shear force is sustained by the web plate in determining the initial thickness for web plate. The partial of story shear is sustained by the boundary elements practically. We get the share of web plate from story shear while discussed frame modeling and its loading to determine the amount of shear that is distributed between web plate and boundary elements from story shear. The actual thickness of web plate is determined by having the actual share of web plate from the shear force.

Table 4. The share of web plate shear and boundary elements from story shear

Story	Lateral force (ton)	Shear (ton)	The share of web plate from shear (ton)	The percentage of the share of web plate from shear (%)
9	89.4	89.4	40.05	45.3
8	69.0	158.4	93.46	59.0
7	60.4	218.8	139.81	63.9
6	51.3	270.1	156.39	57.9
5	42.7	312.8	207.07	66.2
4	33.6	346.4	236.94	68.4
3	25.0	371.4	257.75	69.4
2	17.0	388.4	271.88	70.0
1	9.4	397.8	265.73	66.8

The cross-sections of the horizontal and vertical boundary elements according to the software analysis shown in Fig. 6.

5-4- To estimate the angle of diagonal boundary field α

As it said the tensional field angle in the plate of shear wall depends up on the frame geometric dimensions consisting of boundary elements and web plate thickness that its value is determined from Eq. 2. By the obtained data from table 6, the final value α has been determined in table 7.

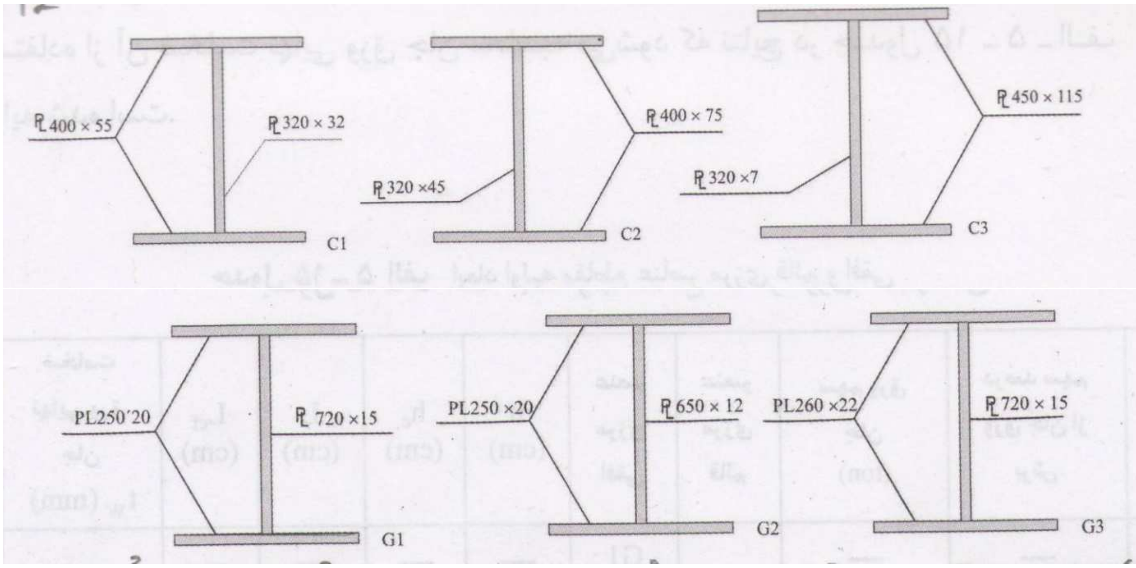

Figure 6. The cross-sections of the horizontal and vertical boundary elements according to the software analysis and cross-section geometric characteristics

Table 5. cross-section geometric characteristics

The type of Columns	Cross-section height *(cm)*	The existing moment of inertia for columns around strong axis cm^4	The type of Beams	Cross-section height *(cm)*	The existing moment of inertia for beams around strong axis cm^4	*The existing moment of inertia for beams around weak axis* cm^4	*Existing cross-section for beam* cm^2	*Radius of gyration for beam section around x axis cm*	*Radius of gyration for beam section around y axis cm*
C1	*43*	*164534.8*	*G1*	*76*	*183589*				
C2	*47*	*249138.0*	*G2*	*69*	*139720.8*	*5217.7*	*178.0*	*28.02*	*5.41*
C3	*55*	*502937.7*	*G3*	*76.4*	*204163.4*				

Table 6. to assign the cross-sections of the horizontal and vertical boundary elements and to calculate the final thickness of web plate

Level	*Web plate share from shear (ton)*	**vertical boundary element VBE**	**horizontal boundary element HBE**	***h (cm)***	h_c *(cm)*	**L (cm)**	L_{cf} *(cm)*	*Final web plate thickness* t_w *(mm)*
Roof	-	-	G1	-	-	-	-	-
9	40.05	C1	G2	390	390-76=314	600	600-43=557	2
8	93.46	C1	G2	390	390-69=321	600	600-43=557	3
7	139.81	C1	G2	390	390-69=321	600	600-43=557	4
6	156.39	C2	G1	390	390-69=321	600	600-47=553	5
5	207.07	C2	G2	390	390-76=314	600	600-47=553	5
4	236.94	C3	G3	390	390-69=321	600	600-55=545	5
3	257.75	C3	G2	390	390-76=314	600	600-55=545	6
2	271.88	C3	G2	390	390-69=321	600	600-55=545	6
1	265.73	C3	G1	540	541-69=471	600	600-55=545	6

Table 7. calculation of tensional field angle in the shear wall plate

Story	$\tan^4\alpha = \dfrac{\frac{2}{t_w L}+\frac{1}{A_c}}{\frac{2}{t_w L}+\frac{2h}{A_h L}+\frac{h^4}{180 I_c L^2}}$ **(α) In terms of Degree**
9	42.8
8	41.7
7	41.0
6	40.3
5	40.8
4	40.2
3	40.4
2	39.6
1	37.5

5-5- To design the horizontal boundary elements HBE

We design G2 beam in the ninth story for example.

5-5-1- calculation of bending moment resulting from uniform distributed load and two concentrated load

$$M_u = \frac{(w_u + w_g)L_h^2}{8} + M' = \frac{(w_u + w_g)L_h^2}{8} + P_u\left[\frac{L}{3} - \frac{d_c}{2} - \frac{d_b}{2}\right]$$

$$w_g = 0$$

$$w_u = R_yF_y(t_i - t_{i+1})\cos^2\alpha$$

$$w_u = R_yF_y(t_8\cos^2\alpha_8 - t_9\cos^2\alpha_9)$$

$$w_u = 1.15 \times 2400 \times (0.3 \times \cos^2 41.7 - 0.2 \times \cos^2 42.8) = 164.4\,\frac{kg}{cm}$$

$$w_u = 16.44 \text{ ton/m}$$

$$\mathbf{L_h} = L - 2S_h = L - 2\left[\frac{1}{2}(d_c + d_b)\right] = 600 - 2 \times \left[\frac{1}{2}(43 + 69)\right] = 488 \text{ cm}$$

$$P_u = 10.57 \text{ ton (due to slab joist)}$$

$$M_u = \frac{(16.44 + 0) \times 4.88^2}{8} + 10.57\left[\frac{6}{3} - \frac{0.46}{2} - \frac{0.69}{2}\right] = 64.0\ ton.m$$

5-5-2- calculation axial force

We obtain the transferred axial force from vertical boundary element due to web plate tension corresponding to Eq. 7.

$$P_{HBE}(VBE) = \sum \frac{1}{2}R_yF_y\sin^2(\alpha)t_wh_c =$$

$$\frac{1}{2} \times 1.15 \times 2400 \times [0.2 \times 314 \times sin^2 42.8 + 0.3 \times 321 \times sin^2 41.69] \times 10^{-3} = 98.8\ ton$$

Then we obtain the transferred axial force from web plate corresponding to Eq. 8.

$$P_{HBE}(web) = \frac{1}{2}R_yF_y[t_i sin(2\alpha_i) - t_{i+1}sin(2\alpha_{i+1})]L_{cf} =$$

$$\frac{1}{2} \times 1.15 \times 2400 \times [0.3sin(2 \times 41.7) - 0.2 \times sin(2 \times 42.8)] \times 557 \times 10^{-3}$$
$$= 76.1 \text{ ton}$$

The axial force in horizontal boundary element is obtained from Eq. 6.

$$P_{HBE} = P_{HBE}(VBE) \pm \frac{1}{2}P_{HBE}(web) = 98.8 \pm \frac{1}{2} \times 76.1 = 136.85\ , 60.75\ ton$$

As regards the two above forces are compressive force, thus the 136.85 ton force is more critical.

5-5-3-Calculation of strength

We determine the moment amplification factor $\boldsymbol{C_m}$ in this part for considering $\mathbf{P-\delta}$ effect from beam columns relationship.

$$P_e = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 \times 2.1 \times 10^6 \times 139720.8}{600^2} = 8044102.6\ kg\ \approx 8044.1\ ton$$

$$\alpha = 1\ , C_m = 1\ , B_1 = \frac{C_m}{\left(1 - \frac{\alpha P}{P_e}\right)} \geq 1 \Rightarrow B_1 = \frac{1}{\left(1 - \frac{136.85}{8044.1}\right)} = 1.02 > 1.0\ \checkmark$$

$$M_r = B_1M_{nt} + B_2M_{lt} \approx B_1M_u = 1.02 \times 64.0 = 65.28 \text{ ton.m}$$

The shear force in the horizontal boundary element is obtained from Eq. 12.

$$V_u = \frac{2M_{pr}}{L_h} + P_u + \frac{(w_g + w_u)}{2}(L_{cf}) \tag{12}$$

In the above equation M_{pr} is obtained from Eq. 13.

$$M_{pr} = 1.1R_yF_yZ_{RBS} = 1.1R_yF_y\left(\frac{2}{3}Z_x\right) \tag{13}$$

In the above relationship, $\boldsymbol{Z_{RBS}}$ is the reduced plastic modulus of section.

$$Z_x(G_2) = 25 \times 2 \times 33.5 \times 2 + 32.5 \times 1.2 \times 16.25 \times 2 = 4617.5\ cm^3$$

$$M_{pr} = 1.1 \times 1.15 \times 3500 \times \left(\frac{2}{3} \times 4617.5\right) \times 10^{-5} = 136.3\ ton.m$$

5-5-4- Calculation the reduced bending strength by using the compressive force (Eq. 14)

$$M^*_{pr} = \frac{9}{8}(1.1 \times R_yF_yZ_{RBS})\left[1 - \frac{P_u(HBE)}{P_y}\right] \tag{14}$$

$$P_y = F_yA_g = 3500 \times 178 \times 10^{-3} = 623\ ton$$

In the compression side

$$\frac{P_u}{P_y} = \frac{136.85\ ton}{623\ ton} = 0.22 > 0.2$$

$M^{*}_{pr} = \frac{9}{8} \times 1.1 \times 1.15 \times 3500 \times \frac{2}{3} \times 4617.5 \times \left(1 - \frac{136.85}{623}\right) \times 10^{-5}$
$M^{*}_{pr} = 119.65\ ton.m$
In the tension side

$$M^{*}_{pr} = \left(1.1 \times R_y F_y Z_{RBS}\right)\left[1 - \frac{1}{2}\left(\frac{P_u(HBE)}{P_y}\right)\right] \quad (15)$$

$\mathrm{M}^{*}_{\mathrm{pr}} = \left(1.1 \times 1.15 \times 3500 \times \frac{2}{3} \times 4617.5\right)\left[1 - \frac{1}{2}\left(\frac{136.85}{623}\right)\right] \times 10^{-5}$

$M^{*}_{pr} = 121.32\ \mathrm{ton.m}$
Since $w_g is\ equal\ to\ zero$, shear force in horizontal boundary element is obtained from Eq. 12.
$\mathrm{V_u} = \frac{\left(\mathrm{M}^{*}_{\mathrm{pr}}(\text{فشار}) + \mathrm{M}^{*}_{\mathrm{pr}}(\text{کشش})\right)}{\mathrm{L_h}} + \mathrm{P_u} + \frac{(\mathrm{w_g} + \mathrm{w_u})}{2}(\mathrm{L_{cf}})$
$V_u = \frac{(119.65 + 121.32)}{4.88} + 10.57 + \frac{(0 + 16.44)}{2}(5.57) = 95.16\ ton$

5-5-5- compact section control

$\frac{\mathrm{b_f}}{2\mathrm{t_f}} \leq 0.3\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \Rightarrow \frac{25}{2 \times 2} = 6.25 \leq 0.3\sqrt{\frac{2.1 \times 10^6}{3500}} = 7.35\ \checkmark$

$\mathrm{C_a} = \frac{\mathrm{P_u}}{\emptyset_c \mathrm{P_y}} > 0.125 \Rightarrow \frac{\mathrm{h}}{\mathrm{t_w}} \leq 1.12\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}}(2.33 - \mathrm{C_a}) \Rightarrow$

$\mathrm{C_a} = \frac{136.85}{0.9 \times 623} = 0.244 > 0.125 \Rightarrow$

$\frac{65}{1.2} = 54 \leq 1.12\sqrt{\frac{2.1 \times 10^6}{3500}}(2.33 - 0.244) = 57.23\ \checkmark \Rightarrow$ the section is compact.

5-5-6- shear control

$\frac{\mathrm{h}}{\mathrm{t_w}} = \frac{65}{1.2} = 54 \leq 2.24\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} = 2.24\sqrt{\frac{2.1 \times 10^6}{3500}} = 54.87$
$\emptyset_v \mathrm{V_n} = \emptyset_v \times 0.6 \mathrm{F_y} \mathrm{A_w} \Rightarrow \emptyset_v \mathrm{V_n} = 1.0 \times 0.6 \times 3500 \times (65 \times 1.2) \times 10^{-3}$
$\emptyset_v \mathrm{V_n} = 163.8 > 95.16\ \mathrm{ton}\ \checkmark$

5-5-7- interaction formula control between axial force and bending moment
$\frac{\mathrm{KL}}{\mathrm{r_x}} = \frac{1 \times 600}{28.02} = 21.41\ , \frac{\mathrm{KL}}{\mathrm{r_y}} = \frac{1 \times 200}{5.41} = 36.97$

$\lambda_{max} = 36.97 < 4.71\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} = 4.71 \times \sqrt{\frac{2.1 \times 10^6}{3500}} = 115.4 \Rightarrow \mathrm{F_{cr}} = \left[0.658^{\left(\frac{\mathrm{F_y}}{\mathrm{F_e}}\right)}\right]\mathrm{F_y}$

$\mathrm{F_e} = \frac{\pi^2 \mathrm{E}}{\lambda^2} = \frac{\pi^2 \times 2.1 \times 10^6}{36.97^2} = 15164.22$
$\mathbf{F_{cr}} = \left[\mathbf{0.658}^{\left(\frac{3500}{15164.22}\right)}\right]\mathbf{3500 = 3177.7\ ^{kg}/_{cm^2}}$
$\emptyset_c \mathrm{P_n} = \emptyset_c \mathrm{F_{cr}} \mathrm{A_g} = 0.9 \times 3177.7 \times 178 \times 10^{-3} = 509.1\ \mathrm{ton}$
$\frac{\mathrm{P_r}}{\emptyset_c \mathrm{P_n}} = \frac{136.85\ \mathrm{ton}}{509.1\ \mathrm{ton}} = 0.27 > 0.2$

$\mathrm{L_b} = 0 < \mathrm{L_p} = 1.76 \mathrm{r_y}\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} = 1.76 \times 5.41 \times \sqrt{\frac{2.1 \times 10^6}{3500}} = 233.2\ \mathrm{cm}$

Thus, it is not necessity to consider lateral-torsional buckling.
$\emptyset_b M_n = \emptyset_b Z_x F_y = 0.9 \times 4617.5 \times 3500 \times 10^{-5} = 145.45\ ton.m$
$\frac{\mathrm{P_r}}{\emptyset_c \mathrm{P_n}} + \frac{8}{9}\frac{\mathrm{M_r}}{\emptyset_b \mathrm{M_n}} \leq 1 \Rightarrow 0.27 + \frac{8}{9}\frac{65.28}{145.45} = 0.67 \leq 1\ \checkmark$

5-6- To design the vertical boundary elements VBE

5-6-1- compressive and tensile axial force in the VBE

As it is said, the compressive axial force in the vertical boundary element that is the sum of the plate strength and the shear due to horizontal boundary element is determined from Eq. 11.

$$E_m = \Sigma \frac{1}{2} R_y F_y \sin(2\alpha) t_w h_c + \Sigma V_u$$

In the above equation $\sum V_u$ is the shear force due to all of the beams in upper levels.

$$V_u = \frac{2M_{pr}}{L_h} + \frac{w_u}{2} L_{cf}$$

Also, in order to consider the effect of connected beams that are in opposite direction, the shear force that is towards the top of them and is due to frame behavior is considered according to the relation. $V_u = \frac{2M_{pr}}{L_h}$. As a result of this, the equation 18 is obtained for compressive forces in the column.

$$E_m = \Sigma \frac{1}{2} R_y F_y \sin(2\alpha) t_w h_c + \left[\frac{2M_{pr}(HBE)}{L_h} + \frac{w_u}{2} L_{cf} \right] - \Sigma \left[\frac{2M_{pr}(Adj)}{L_h} \right] \tag{18}$$

According to the shear calculation in the 9th story beams that is applied as compressive and tensile force in the boundary columns, these compressive and tensile force is obtained.

$$\left[\frac{2M_{pr}(HBE)}{L_h} + \frac{w_u}{2} L_{cf} \right] - \Sigma \left[\frac{2M_{pr}(Adj)}{L_h} \right] = 107.16 + 157.99 - 40.27 - 81.27 = 143.6 \quad ton$$

$$\left[\frac{2M_{pr}(HBE)}{L_h} + \frac{w_u}{2} L_{cf} \right] - \Sigma \left[\frac{2M_{pr}(Adj)}{L_h} \right] = -31.78 + 15.44 - 40.27 - 81.27 = -105.2 \quad ton$$

Therefore ($\Sigma V_u = 143.6 \quad ton$) and ($\Sigma V_u = -105.2 \quad ton$) are obtained compressive and tensile force respectively. As a result the compressive and tensile forces in vertical boundary elements are determined as follows:

$$E_m = \frac{1}{2} \times 1.15 \times 2400 \times \left[0.2 \times 314 \times \sin(2 \times 42.8) + 0.3 \times 321 \sin(2 \times 41.7) \right] \times 10^{-3} + 143.6 \Rightarrow$$

$$E_m = 362.0 \quad ton$$

For obtaining P_u, we add a force of 47 ton as the dead load.

$$P_u = 362 + 47 = 409 \quad ton$$

$$B_1 = \frac{1}{1 - \left(\frac{P_u}{P_e} \right)} = \frac{1}{1 - \left(\frac{409 \times 10^3}{\frac{\pi^2 \times 2.1 \times 10^6 \times 164535}{1 \times 390^2}} \right)} = 1.02 \succ 1.0$$

$$E_m = \frac{1}{2} \times 1.15 \times 2400 \times \left[0.2 \times 314 \times \sin(2 \times 42.8) + 0.3 \times 321 \sin(2 \times 41.7) \right] \times 10^{-3} - 105.2 \Rightarrow$$

$$E_m = 113.2 \quad ton$$

5-6-2- Bending moment in vertical boundary member

Bending moment in vertical boundary member is caused by shear wall plate and plastic hinge formation in the horizontal element.

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

$$M_{VBE}(web) = 1.15 \times 2400 \times (\sin^2 41.7) \times 0.2 \times \left(\frac{321^2}{12} \right) \times 10^{-5} = 21 \quad ton$$

$$M_{VBE}(HBE) = \frac{1}{2} M_{pb} \quad , \quad M_{pb} = \frac{M_{pr}}{1.1R_y} + V_u \times s_h$$

We have from the G2 beam:

$$M_{pb} = \frac{M_{pr}}{1.1R_y} + V_u \times s_h = \frac{136.3}{1.1 \times 1.15} + 107.16 \times \left(\frac{0.43 + 0.69}{2} \right) = 167.75 \quad ton.m$$

$$M_{pr}^* = 1.1 R_y F_y Z_{RBS} \left[1 - \frac{1}{2} \left(\frac{P_u}{P_y} \right) \right] = 1.1 \times 1.15 \times 3500 \times 4617.5 \times \frac{2}{3} \left[1 - \frac{1}{2} \times \frac{136.85}{623} \right] = 123.0$$

$$M_{pb} = \frac{M_{pr}}{1.1R_y} + V_u \times s_h = \frac{123.0}{1.1 \times 1.15} + 40.27 \times \left(\frac{0.43 + 0.69}{2} \right) = 118.2 \quad ton.m$$

$$M_{VBE}(HBE) = \frac{1}{2} M_{pb} = \frac{1}{2} \times (118.2 + 167.75) = 143.0 \quad to\,n.m$$

$$M_u = M_{VBE}(web) + M_{VBE}(HBE) = 21 + 143 = 164 \quad ton.m$$

5-6-3- Bending moment and axial force interaction control

$$P_u = 409 \quad ton$$

$$M_u = B_1 M_u = 1.02 \times 164 = 167.28 \quad ton.m$$

$$\frac{KL}{r} = \frac{390}{10.41} = 37.46 \Rightarrow F_{cr} = 3169.5 \quad kg/cm^2$$

$$\phi_c P_n = 0.9 \times 542.4 \times 3169.5 \times 10^{-3} = 1547.22 \quad ton \quad , \quad \frac{P_u}{\phi_c P_n} = \frac{409}{1547.22} = 0.264 \succ 0.2$$

$$L_b \prec L_p \Rightarrow \phi_b M_n = \phi_b F_y Z_x = 0.9 \times 3500 \times 9069.2 \times 10^{-5} = 285.68 \quad ton.m$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \frac{M_u}{\phi_b M_n} \le 1 \quad , \quad 0.264 + \frac{8}{9} \times \frac{167.22}{285.68} = 0.784 \le 1 \quad ✓$$

5-6-4- Shear control

Shear of the vertical boundary element is due to the tension of horizontal element in addition to the part of the base shear.

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

$$V_{VBE}(web) = \frac{1}{2} \times 1.15 \times 2400 \times \sin^2 41.7 \times 0.3 \times 321 \times 10^{-3} = 58.8 \quad ton$$

$$V_{VBE}(HBE) = \sum \frac{1}{2} \left(\frac{M_{pc}}{h_c} \right) = \frac{1}{2} \frac{(164 + 143)}{3.21} = 47.8$$

$$V_u = V_{VBE}(HBE) + V_{VBE}(web) = 58.8 + 47.8 = 106.6 \quad ton$$

$$\frac{h}{t_w} \le 2.24 \sqrt{\frac{E}{F_y}} \quad , \quad \phi_v = 1 \quad \Rightarrow \phi_v V_n = \phi_v \times 0.6 \times F_y \times A_w = 1 \times 0.6 \times 3500 \times (32 \times 3.2) \times 10^{-3} ✓$$

$$\Rightarrow \quad V_n = 215 \quad \succ \quad 106.6$$

6- RESULTS

In the recent decades and most of developed countries, it has been created an application and usage approach for a code that could use from elastic analysis method for analyzing the structure in a way and on the other hand, the structural components strength is considered in ultimate limit states. So application of LRFD method or load and resistance factors method becomes common, necessary, applicable and essential in the steel structures design. As regards the latest tenth issue of national building regulations has been provided and codified corresponding to above style, we decided to study the usage of limit states method (LRFD) to analyze and design a special steel shear wall consisting of web plate and its horizontal boundary element (a beam) in a case study in which the structure is 9 stories. Its frame and steel shear wall shown in Figs. 4 and 5. According to the calculation results it could be observed that designing in LRFD method is more logical and tangible compared with allowable stress design method (ASD). Its main reason is that in LRFD method the safety margin must be selected to obtain the design reliability of the structure, and must include both the variability in the load and the resistance. The LRFD method simplifies this process and reduces the number separate variability considerations by separating the variability of the resistance from the variability in loading by introducing separate load factors,γ, and resistance factors, $\emptyset$. The ASD method uses an alone safety factor and exerts it only in a one stage, which must account for both variation in member behavior and uncertainty in the loading. Unfortunately, these variations may be very different for different conditions, and it is difficult to define a single factor of safety, FS, which covers the wide range of these variations.

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