SMAR 2015 – Third Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2015

Appropriate infill plate yielding distribution in Steel Plate Shear Walls (SPSWs) under seismic effects

Ismail Gurkan Arici[1], Oguz C. Celik[2]

[1] Institute of Science and Technology, Istanbul Technical University, Istanbul, Turkey
[2] Structural&Earthquake Engineering Working Group, Faculty of Architecture, Istanbul Technical University, Istanbul, Turkey

Structural effectiveness of steel plate shear walls (SPSWs) is highly dependent on the development of tension field action under lateral seismic loading. This action is expected to be uniformly distributed through the entire system. This study presents a parametric investigation into planar SPSWs having different number of stories and thin plates with the same thicknesses. A capacity design procedure called the 'Proposed Method (PM)' was used on the strip model of SPSWs. Numerical results show that almost 80% of the strips of the intermediate stories yields for the selected buildings. The ratio of yielding strips at the top floors of the buildings is found to be between 30~50 %, revealing that using constant thickness web plates along the building height may not be preferred. This difference is large when the story number of the building is high. Furthermore, a new empirical formula for the fundamental period of SPSWs is proposed.

1. INTRODUCTION

Steel plate shear walls (SPSWs) are preferable lateral seismic load carrying systems due to their high lateral stiffness, high ductility, sufficient strength, and stable hysteretic behavior under seismic loading. Intense research (both theoretical and experimental) has been done on the seismic behavior of SPSW systems especially within the last several decades (Sabelli and Bruneau (2007), Timler and Kulak (1983), Thorburn et al. (1983)). SPSWs have an increasing use in the USA, Canada, and Japan for both in new buildings and in retrofit design of existing buildings. SPSWs have been used as an alternative for conventional steel braces (i.e. buckling braces) in seismic design of structures since 1970s. An experimental comparison between conventional braces (both tubular and solid bar) and SPSWs is made by Berman et al. (2005). The steel plate is connected to the steel boundary frame by closely spaced bolts, continuous welding (Rezai et al. (2004)), and epoxy bonding (Berman et al. (2005) or screws (Vatansever C. (2007)). A typical SPSW has three main elements (Figure 1): web (or infill) plate surrounded by vertical boundary elements (VBE, i.e. columns), and horizontal boundary elements (HBE, i.e. beams).

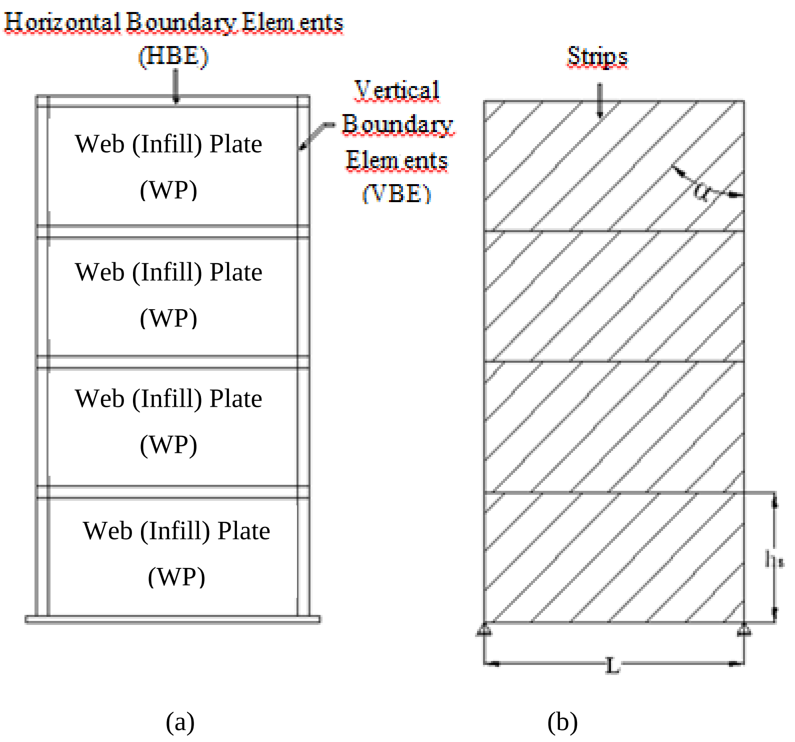


Figure1. Typical SPSW models (Berman and Bruneau, (2008)): (a) Orthotropic membrane model, (b) Strip model.

In the orthotropic membrane model, the web plates are represented by a single steel plate in each story while a series of inclined strips, minimum of ten strips per story according to the ANSI/AISC 341-05 (2005) and ANSI/AISC 341-10 (2010) is assumed in the strip model (Sabelli and Bruneau (2007)).

This study presents insights on the appropriate infill plate yielding distribution in SPSWs. A successive iteration capacity design procedure called 'the Proposed Method (PM)', which gives almost the same results compared to the nonlinear pushover analysis, is used. Web plate and the boundary elements of SPSWs are modeled with strips in which infill plates are represented by a series of pin-ended, tension-only members. The main objective is to determine and maximize the yielded portion of the web plate for every story. Parameters affecting this distribution are discussed.

2. SEISMIC BEHAVIOR OF SPSWs

The behavior of SPSWs under earthquake effects can be defined as a combination of lateral shear resistance and flexural capacity. During an earthquake, SPSW is expected to provide a lateral shear resistance through yielding of web plates, while the VBE having sufficient flexural capacity should resist these yielding forces preferably in the elastic region. Therefore, VBE design becomes an important part of SPSW design. For the design of boundary elements, especially for VBE, a design method defined as the Proposed Method (PM), can be utilized (Berman and Bruneau (2008)). This method depends on a capacity concept in which the web plates reach their lateral load carrying capacity and yield under seismic load effects. According to the tension field action mechanism, yielding forces due to yielding of web plates are inclined at an angle of α, assumed to be close to 45º from the vertical direction per ANSI/AISC 341-05 (2005). Note that, α can be taken as 40º from the vertical direction per ANSI/AISC 341-10 (2010) and Canadian Standards Association (CSA, 2009).These forces are directly resisted by the VBE and HBE members. Under these inclined yielding forces, free body diagrams of VBE and HBE can be drawn and the appropriate steel sections can be assigned by taking into consideration that VBE should remain totally elastic and plastic hinges should occur only at the ends of beams. It is noted that the Proposed Method (or capacity design) almost agrees with the pushover analyses of SPSWs. Further details about PM can be found in (Berman and Bruneau (2008)).

Effectiveness of SPSWs is highly dependent on the development of tension field action under lateral seismic loading. This action is expected to be uniformly distributed through the entire SPSWs. To obtain a uniform yielding on web plates during an earthquake, VBE should have a sufficient stiffness to allow the web plates reach their capacity and yield. Therefore, ANSI/AISC 341-05 and Canadian Standards Association (CSA, 2009) require a minimum moment of inertia for the boundary columns to prevent them from excessive in-plane flexibility and buckling failures observed from prior experimental research (Qu et al. (2012)). This requirement is given below:

$$I \geq \frac{0.00307 t h^4}{L} \tag{1}$$

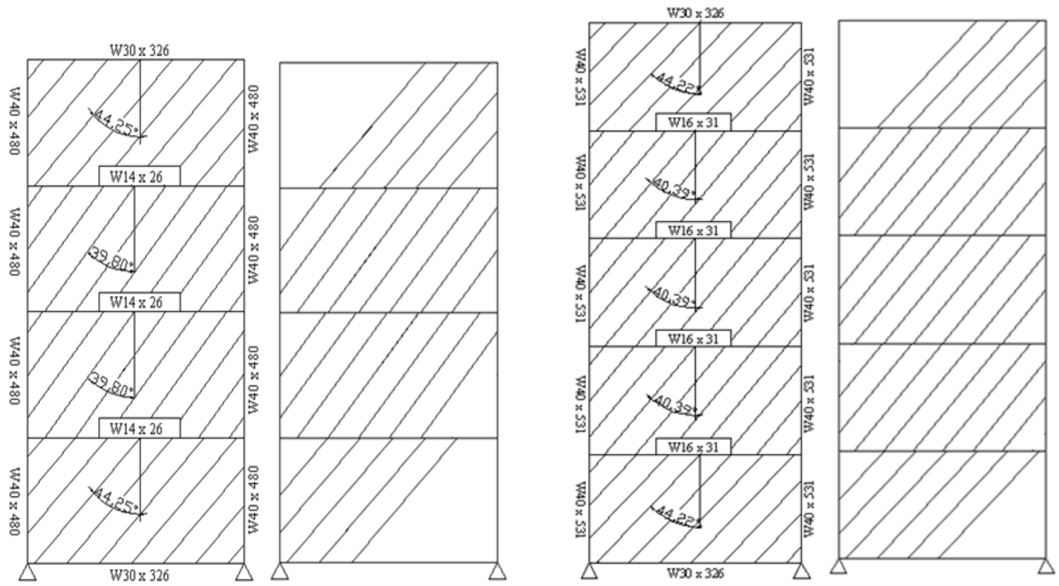
where the parameters are defined as t = thickness of the web plate, h= height of the story, L = span length. By using Eq (1), SPSWs boundary elements are designed according to the Proposed Method. It is easier to achieve full (100%) tension field action at larger story drifts when thinner infill plates are used. Stiff column sections may result in a relatively high uniform yielding on web plates and reduce the drifts along the height of SPSW systems. In addition, if the SPSW columns become relatively flexible, infill plate yielding may first occur at a certain story and then progressively spread into the other stories (Qu et al. (2012)). In this case, initially yielded web plate may have excessive plastic deformations which lead to premature failure before all the other stories yield.

Infill plate yielding sequence and distribution along the height of the SPSW depends on the key parameters such as the relative stiffness of the columns, the lateral seismic force distribution and the infill plate strength distribution (Qu et al. (2012)). In order to obtain a uniform yielding behavior for the web plates of SPSWs, each SPSW story should have sufficient ductility capacity to allow all the web plates yield and make contribution to the lateral force strength while preventing the initially yielded web plate to failure under excessive plastic deformations.

3. PLANAR SPSW DESIGN ACCORDING TO THE PROPOSED METHOD (PM)

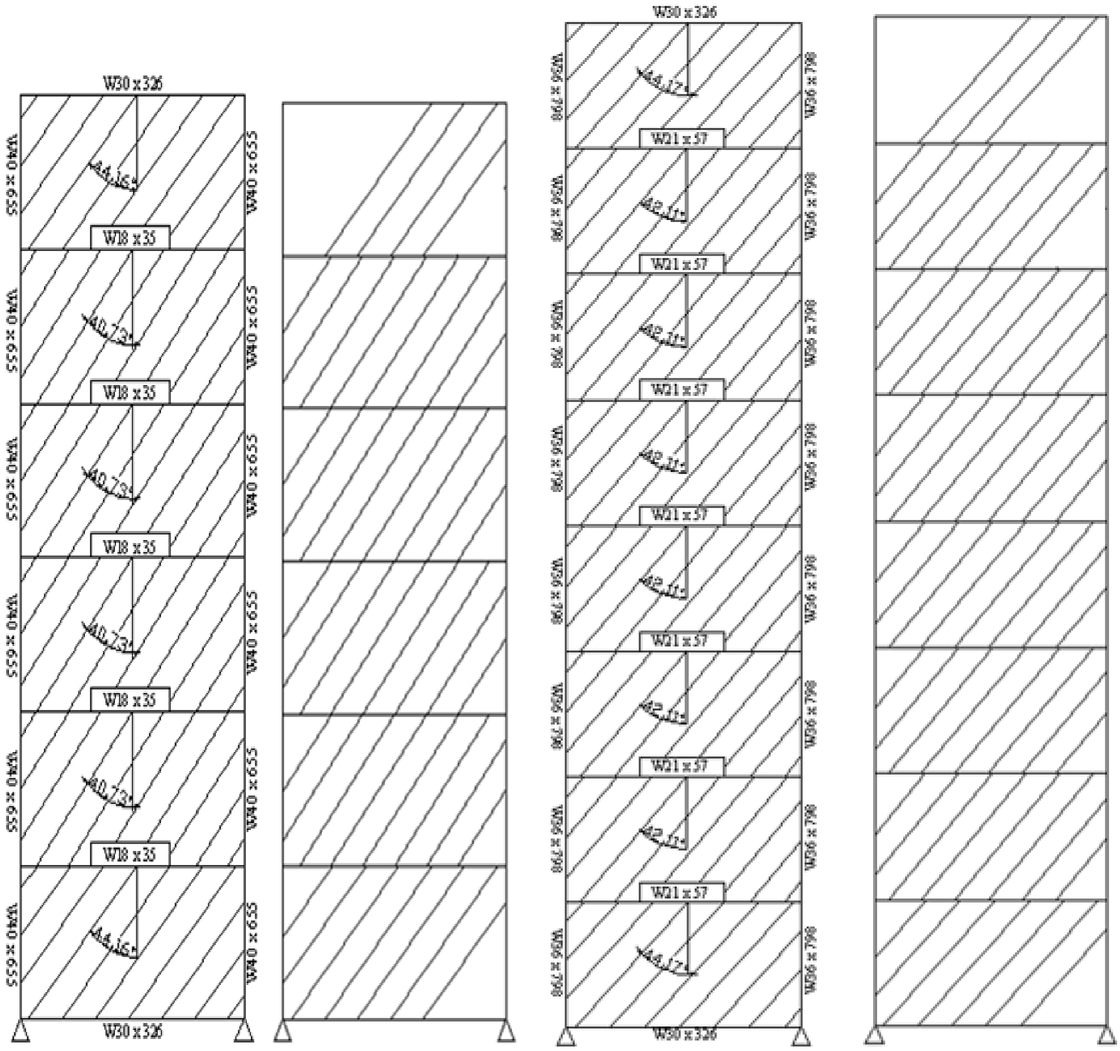
As described above, the Proposed Method can be explained as a capacity design of boundary elements of SPSWs under inclined yielding web forces due to lateral seismic effects. A successive iteration may be needed to dimension the boundary elements under a selected web plate thickness according to the formula given in ANSI/AISC 341-05 (2005) and ANSI/AISC 341-10 (2010).

As a case study, a hospital building, with a symmetrical plan (18 m x 18 m) in both (x) and (y) directions and regular story heights (h=3 meters) is examined for four different story levels. Buildings having 4, 5, 6, and 8 story are assumed to have two SPSWs in each direction by which the earthquake loads will be resisted only by these walls. Beam-to-column connections of these SPSWs are assumed as rigid while the rest of the other connections are hinged. Therefore, because of the flexural moments in the rigid connections, larger sections were chosen for the boundary elements having rigid connections in SPSWs. Axes having SPSWs are chosen and dead and live loads are transformed from a 3D model into 2D model. Then, planar SPSWs models are designed for the same span length, story height, and material characteristics. A dead load of 4,5 kN/m^2 was assumed in all stories, while live loads of 2,75 kN/m^2 and 3,5 kN/m^2 were adopted for the roof and normal stories, respectively. The weight of the roof level was 1725.3 kN while other typical stories had a weight of 1798.2 kN. Elasticity modulus of the steel was 210000 MPa and the Poisson's ratio was 0.3. Steel with yield strength of 275 MPa (Fe 44) was used for the boundary elements while a steel grade of 235 MPa (Fe 37) was used for the web plates. Strips used in the models have a rectangular area of thickness times width of the strip according to (ANSI/AISC 341-05, (2005)). Initial inclination angle of strips were assumed as α=30° and α=45°. It was seen that, the number of successive iteration is less when α=30° (5 iterations for α=45°, 3 iterations for α=30°), and the boundary frame element sections were identical regardless of the initial value of α. LRFD (1999) load combinations were used during the design. The fundamental period was calculated as 0.32 seconds according to the empirical formula given in (ANSI/AISC 341-05, (2005)). The earthquake design forces were obtained by using the method given in the SBBSZ, (2007). Further details can be found in (Arici and Celik (2011)). The final strip models are illustrated in Figure 2 (Arici and Celik (2011)).



(a) (b)

Figure 2. Planar SPSWs strip models (left) and yielded strips (right): (a) 4-story strip model, (b) 5-story strip model



(c) (d)

Figure 2 (cont'd). Planar SPSWs strip models (left) and yielded strips (right): (c) 6-story strip model, (d) 8-story strip model

All of four SPSWs models use 2 mm web plate thickness in each story. Beam span and story height are L=6 m and h= 3 m, revealing that an aspect ratio defined as the ratio of span to the story height is equal to 2. This value is acceptable since (ANSI/AISC 341-05, (2005)) requires an aspect ratio between 0.8 and 2.5.(0.6 is used for lower limit per Canadian Standards Association (CSA, 2009)) Successive iteration method was used with the PM to obtain the final strip model (Arici and Celik (2011)). As shown in Figure 2, the top and the bottom HBE sections are bigger than the intermediate HBE sections, due to having only one web plate connected to them which results in nonzero web inclined yield forces in vertical and horizontal directions. VBE sections are identical in all of the stories for each planar model. In addition, it should be noted that, the lateral load distribution according to the Equivalent Earthquake Load

Procedure given in (SBBSZ, 2007) was revised by considering the modal period (T_{modal}) instead of emprical fundamental period (T_a) used at the beginning of the design. The details of SPSW strip design models are given in Table 1.

Table 1. SPSWs strip design details

story no	VBE	HBE	α degrees	# of total strips	# of yielded strips	yielding %	t mm	F_i kN	ΦV_n kN	capacity used %
4	W40X480	W30X326	44.25	10	5	50	2	192	1066	18
3	W40X480	W14X26	39.8	10	8	80	2	331	1066	31
2	W40X480	W14X26	39.8	10	7	70	2	424	1066	40
1	W40X480	W30X326	44.25	10	5	50	2	470	1066	44

story no	VBE	HBE	α degrees	# of total strips	# of yielded strips	yielding %	t mm	F_i kN	ΦV_n kN	capacity used %
5	W40X531	W30X326	44.22	10	5	50	2	166	1066	16
4	W40X531	W16X31	40.39	10	8	80	2	289	1066	27
3	W40X531	W16X31	40.39	10	8	80	2	382	1066	36
2	W40X531	W16X31	40.39	10	8	80	2	444	1066	42
1	W40X531	W30X326	44.22	10	6	60	2	475	1066	45

story no	VBE	HBE	α degrees	# of total strips	# of yielded strips	yielding %	t mm	F_i kN	ΦV_n kN	capacity used %
6	W40X655	W30X326	44.16	10	4	40	2	147	1066	14
5	W40X655	W18X35	40.73	10	8	80	2	256	1066	24
4	W40X655	W18X35	40.73	10	8	80	2	343	1066	32
3	W40X655	W18X35	40.73	10	8	80	2	408	1066	38
2	W40X655	W18X35	40.73	10	8	80	2	451	1066	42
1	W40X655	W30X326	44.16	10	7	70	2	473	1066	44

story no	VBE	HBE	α degrees	# of total strips	# of yielded strips	yielding %	t mm	F_i kN	ΦV_n kN	capacity used %
8	W36X798	W30X326	44.17	10	3	30	2	133	1066	12
7	W36X798	W21X57	42.11	10	7	70	2	227	1066	21
6	W36X798	W21X57	42.11	10	8	80	2	307	1066	29
5	W36X798	W21X57	42.11	10	8	80	2	374	1066	35
4	W36X798	W21X57	42.11	10	8	80	2	427	1066	40
3	W36X798	W21X57	42.11	10	8	80	2	467	1066	44
2	W36X798	W21X57	42.11	10	8	80	2	494	1066	46
1	W36X798	W30X326	44.17	10	7	70	2	507	1066	48

The yielding strips under the earthquake effects are shown as in Figure 2. According to Figure 2, several results for all types of SPSWs used in this study can be obtained as:

a) A uniform yielding distribution can be seen at the web plates in intermediate stories (with a yielding percentage of approximately 80%).
b) A nonuniform yielding distribution can be seen at the top stories, due to the use of thicker web plates than the required thicknesses proposed by the codes. The shear capacity of the 2 mm web plate is calculated as 1066 kN. Lateral earthquake loads at the top stories are quite less than this capacity, which prevents the plate from a uniform yielding distribution at these stories. Therefore, a variable thickness web plate design from the bottom to the top stories of SPSWs would be much appropriate. According to Table 1, as the lateral force distribution is decreasing along the upper stories, the thickness of the web plates should also be reduced simultaneously to avoid overdesign of them. This will let the web plate use its capacity much, and make much contribution to the total shear capacity of that story.
c) Using the same VBE sections in all stories may prevent uniform yielding of the web plates. Yield forces can be decreased by using smaller web plate thickness. Therefore, smaller VBE sections could be chosen for decreasing lateral load and thickness of web plates along the upper stories.

4. FUNDAMENTAL PERIODS OF SPSWs

Results obtained from the 4, 5, 6, and 8 story planar SPSWs models are used in a regression analysis which gives a new empirical period formula in the form of $T=aH^b$ for the fundamental period of SPSWs:

$$T_a = 0.035\, H_n^{1.12} \tag{2}$$

In Eq (2), H_n is the structure height in meter and a = 0.035 and b=1.12 are the calculated values from the systems discussed in this work. Detailed explanations regarding this can be found in (Arici and Celik (2011)).

5. CONCLUSIONS

The following conclusions are possible from this parametric study:

- It is observed that full yielding of web plates in SPSWs during lateral load effects could be achieved by using appropriate web plates which are to be thinner along the upper stories due to smaller shear forces developing at the top of the building.
- Numerical results show that almost 80% of the strips representing the infill plates of the intermediate stories yields for all of the selected buildings. The ratio of yielding strips at the top floors of the buildings is found to be between 30~50 %, revealing that using constant thickness web plates along the building height may not be preferred. This difference is large when the story number of the building is higher.
- VBE (boundary columns) stiffness has an important role on the behavior of SPSWs during earthquake loading.
- Based on the systems investigated here, a new empirical formula for the fundamental period of SPSWs is proposed.

REFERENCES

ANSI/AISC 341-05, 2005, "*Seismic Provisions for Structural Steel Buildings*," Chicago, Illionis 60601-1802.

ANSI/AISC 341-10, 2010, "*Seismic Provisions for Structural Steel Buildings*," Chicago, Illionis 60601-1802.

Arici IG, 2011, "*Fundamental Periods Of Steel Plate Shear Walls (Spsw) And a Successive Iteration Method For Their Capacity Design Under Earthquake Loads*", MS Thesis, Istanbul Technical University, Turkey.

Arici, IG, Celik, O.C., 2011, "*A Successive Iteration Method for the Capacity Design of Steel Plate Shear Walls (SPSW) under Earthquake Loads*", Seventh National Conference on Earthquake Engineering, Istanbul, Turkey.

Berman, J.W., Bruneau, M., 2008, "*Capacity Design of Vertical Boundary Elements in Steel Plate Shear Walls*," Engineering Journal – First Quarter, pp. 57-71.

Berman, J.W., Celik, O.C., Bruneau, M., 2005, "*Comparing Hysteretic Behavior of Light-Gauge Steel Plate Shear Walls and Braced Frames*", Engineering Structures Journal, Vol. 27, No.3, pp.475-485.

Canadian Standards Association, September 2009, "*CSA Standard S16-09-Design of Steel* Structures*".*

LRFD, December 27, 1999, *Load and Resistance Factor Design Specification for Structural Buildings*, American Institute of Steel Construction, INC, Chicago, Illinois 60601-2001.

Rezai, M., Ventura, C. E., Prion, H., 2004, "*Simplified and Detailed Finite Element Models of Steel Plate Shear Walls*", 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, Paper no: 2804.

Sabelli, R., Bruneau, M., 2007, "Steel Design Guide-*Steel Plate Shear Walls"*American Institute of Steel Construction, Inc.

SBBSZ, 2007, "*Specification for Buildings to be Built in Seismic Zones",* Turkish Seismic Code, Istanbul, Turkey.

Thorburn, L.J., Kulak, G.L., and Montgomery, C.J., 1983, "*Analysis of Steel Plate Shear Walls*," *Structural Engineering Report No. 107*, Department of Civil Engineering, University of Alberta, Edmonton,Alberta, Canada.

Timler P.A. and Kulak G.L., 1983, "*Experimental Study of Steel Plate Shear Walls*,"*Structural Engineering Report No. 114*, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Qu, B., Guo, X., Pollino, M., Chi, H., 2012, "*Effect of Column Stiffness on Infill Plate Yielding Distribution in Steel Plate Shear Walls*", 15 WCEE, Lisboa.

Vatansever C., November 2007, "*Cyclic Behavior of Thin Steel Plate Shear Walls with Semi-Rigid Beam-to-Column Connections*" Ph.D. thesis, Istanbul Technical University-Institute of Science and Technology.