DESIGN OF STEEL PLATE SHEAR WALLS CONSIDERING INELASTIC DRIFT DEMAND

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ABSTRACT:

The unstiffened steel plate shear wall (SPSW) system has emerged as a promising lateral load resisting system in recent years. Heavily stiffened SPSW systems that were designed earlier were not very popular because of being uneconomical. During the Northridge (1994) and the Kobe (1995) earthquakes, SPSW systems behaved very satisfactorily which led engineers and researchers to study and employ unstiffened SPSW systems in a greater extent. However, seismic code provisions for these systems are still based on elastic force-based design methodologies. With ever increasing demands of efficient and reliable design procedures, a shift towards performance-based seismic design (PBSD) is necessary for these systems as well. The PBSD philosophy explicitly considers inelasticity in the lateral load resisting system along with preferring displacement-based design criteria to force-based criteria. In this paper, a new PBSD procedure for SPSW systems based on target inelastic drift and pre-selected yield mechanism is used. This design procedure is simple, yet it aims at an advanced design criterion. A 4-story test building is designed based on the proposed procedure for different target drifts under various earthquake scenarios. The designs are checked under the selected ground motion scenarios through nonlinear response-history analyses. The actual inelastic drift demands are found to be close to the selected target drifts. This shows the effectiveness of the new design procedure. In addition, the plastic hinge locations are also compared with the selected yield mechanism. Future modifications required for this design procedure for different SPSW configurations are identified based on these test cases.

KEYWORDS: steel plate shear walls, performance-based seismic design, displacement-based design, yield mechanism, plastic design

1. INTRODUCTION

In the past two decades, interest has grown the world over on the application of thin unstiffened steel plate shear walls (SPSWs) for lateral load resistance in building structures. Steel plate shear wall system has emerged as an efficient alternative to other lateral load resisting systems, such as reinforced concrete shear walls, various types of braced frames, etc. SPSWs are preferred because of the various advantages they have over other systems, [1]: primarily, substantial ductility, high initial stiffness, fast pace of construction, and the reduction in seismic mass. The design of SPSW was implemented as early as 1970 as a primary load resisting system. Initially, only stiffened SPSWs were used in order to resist the shear forces within their elastic buckling limits, as in the Sylmar Hospital in Los Angeles, the Nippon Steel Building in Tokyo, etc. With the analytical and experimental research carried out by various researchers, in Canadian and US universities, it was observed that the post-buckling ductile behaviour of the unstiffened SPSW is much more effective than the elastic behaviour of the stiffened SPSW in resisting seismic forces, as these unstiffened plates exhibit very stable hysteretic energy dissipation behaviour. However, the design codes which incorporate seismic design using SPSW, such as the CAN/CSA-16 [2] or the AISC-LRFD [3], do not explicitly consider the large inelastic displacement capacity these systems can offer.

Earthquake resistant design of structural systems in general is moving from simplified force-based deterministic design methods towards performance-based seismic design (PBSD) techniques, with emphasis on better characterization of structural damage and on proper accounting for uncertainties involved in the design process.

Traditional force-based consideration of structural response is not suitable for estimating structural damage during earthquakes, since it does not take into account the inelastic response of the structure explicitly. PBSD techniques need to use inelastic response parameters, such as inelastic drift, ductility, hysteretic energy, or combinations of these parameters, to quantify damage. Although, various design methodologies have been proposed considering such inelastic performance parameters for other lateral load resisting systems [4-6], no similar recommendations are available as yet for SPSWs, specifically. This paper focuses on the application of a new design methodology for buildings with SPSW considering an inelastic drift/displacement criterion explicitly.

The design methodology is similar to the method proposed by Leelathaviwat et al. [5], and is a modified version of the preliminary proposal by Ghosh & Ghosh [7]. The method aims at designing a SPSW system to have a specific inelastic drift/displacement under a given earthquake scenario. The main objective of this paper is to validate the effectiveness of this method by designing a 4-story steel structure with pin-connected beams with one SPSW bay. The effectiveness is measured in terms of how close the achieved inelastic displacement is to the target.

2. BASIC FRAMEWORK OF THE DESIGN PROCEDURE

The preliminary design procedure can be found in a previous article [7]. Only the important features and further modifications in that procedure are mentioned here. The details can be found in [8]. First, the total strain energy (elastic and plastic) which is imparted to an inelastic system, is estimated as

$$E_e + E_p = \gamma\left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}C_e g\right)^2 \tag{2.1}$$

where, E_e = elastic strain energy demand, E_p = plastic strain energy demand, γ = energy modification factor, M = total seismic mass of the frame, S_v = pseudo velocity corresponding to T, T = fundamental period, C_e = elastic force coefficient, and g = gravitational acceleration. The energy modification factor is calculated based on the ductility of the system (μ) and ductility reduction factor (R):

$$\gamma = \frac{2\mu - 1}{R^2} \tag{2.2}$$

The MDOF system is idealized as an equivalent system by selecting a typical yield mechanism for the peak monotonic demand. The mechanism is composed of yielding of all the plates and plastic hinge formation at the base of the boundary columns, as shown in Figure 1b. Equating the inelastic work done in the plates and the two plastic hinges with the estimated inelastic strain energy, we get the required yield base shear (V_y) as

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2}, \text{ where } \alpha = \left(\sum_{i=1}^{n}\lambda_i h_i\right)\frac{8\theta_p \pi^2}{T^2 g} \tag{2.3}$$

where, W = total seismic weight of the frame, h_i = ith story height, θ_p = target plastic drift based on an assumed yield drift (θ_y), The factor λ_i can be obtained by studying the shear force distribution in SPSW systems. We adopt a distribution based on statistical studies on steel MRF and EBF systems [9-10]. The required plate thickness at each story is obtained considering that the plate carries the full plastic shear:

$$t_i = \frac{2P_i}{0.95F_y L} = \frac{2V_i}{0.95F_y L} \tag{2.4}$$

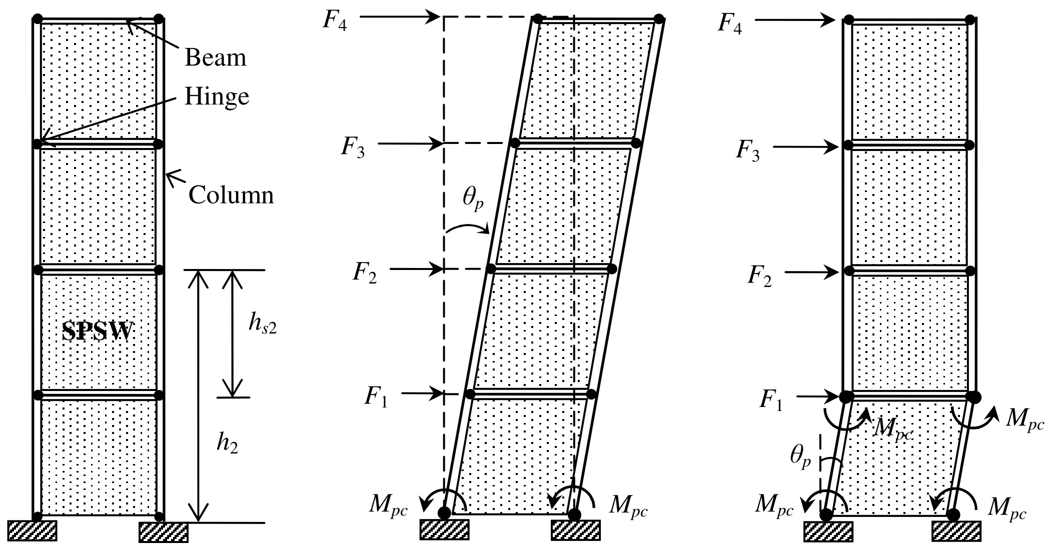

Figure 1 (a) Schematic of the SPSW system; (b) Selected yield mechanism; (c) Soft ground story [7]

where, P_i = plastic shear capacity using a multi-strip idealization, V_i = story shear demand, F_y = material yield strength and L = bay width. The derivation for Eqn. 2.4 is provided in [8]. The base column moment capacity (M_{pc}) is obtained based on Driver et al.'s recommendation [11] for ensuring plasticity in steel plate before in boundary columns:

$$M_{pc} = \frac{50 t_1 h_1^2}{16} \tag{2.5}$$

The axial force (P_c) on the columns is calculated based on the moment equilibrium about the base. The ground story column section is selected for these demands based on the code prescribed P-M interaction and the criterion for compact section [3]. In addition, it is checked that soft story does not form for the selected column section by using an equation similar to

$$V_i \le \frac{4 M_{pc}}{h_{si}} + P_i \tag{2.6}$$

where, V_i = shear demand on ith story, h_{si} = ith story height and P_i = steel plate capacity at ith story.

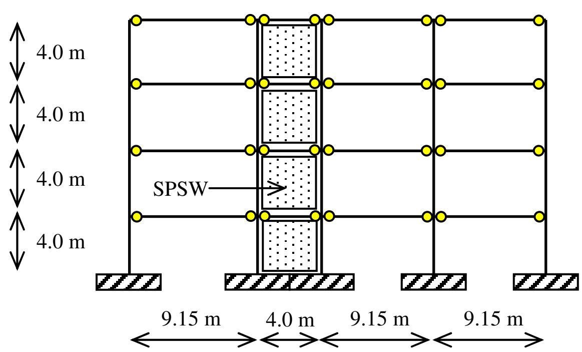

Figure 2 Study frame with SPSW

3. APPLICATION OF THE PROPOSED PROCEDURE

A 4-story steel frame building with pinned beam to column connections (Figure 2) is designed with one bay of steel plate shear walls. Initially we consider the SPSW bay to have a span equal to the story height. This span is later varied in order to consider design scenarios with various aspect ratios of the steel plate panel. The SPSW is designed against specific earthquake records for selected target ductility (μ_t) values. This ductility is defined in terms of the roof displacement. Three strong motion records from the 1994 Northridge, USA and 1995 Kobe, Japan earthquakes (Table 3.1) are used. The details regarding these and other designs are available in [8]. The designed buildings are checked against the same records to measure the effectiveness of the proposed design procedure in terms of the ductility achieved (μ_a).

Table 3.1 Details of earthquake records used for design

Earthquake	Date	Station	Component	PGA	Code Used
Northridge	Jan 17, 1994	Sylmar Converter	Horiz.-052	0.612g	SYL
Kobe	Jan 16, 1995	KJMA	Horiz.-000	0.812g	KJM
Kobe	Jan 16, 1995	Takarazuka	Horiz.-000	0.692g	TAZ

The designed structures are subjected to nonlinear response history analyses under the selected record, using a lumped mass model with 5% Rayleigh damping (in the first two modes). For this, a steel plate is modelled using the multi-strip idealization [12], in which the plate is modelled using parallel braces/truss members connecting the boundary elements. The truss members are aligned along the principal tensile direction (α) of the plate [13]:

$$\tan^4 \alpha = \frac{1 + \dfrac{tL}{2A_c}}{1 + th_s\left(\dfrac{1}{A_b} + \dfrac{h_s^{\,3}}{360 I_c L}\right)} \tag{3.1}$$

where, A_c = cross-sectional area of the bounding column, I_c = moment of inertia of the bounding column, A_b = cross-sectional area of the bounding beam, h_s = story height, L = story width, and t = plate thickness. 10 strips, the minimum number recommended in previous literatures, are used to model each plate panel. The lateral load resisting system is modelled and analyzed using the structural analysis program DRAIN-2DX [14]. The strips are modelled as nonlinear truss elements, while the boundary elements are modelled with nonlinear beam column elements. For all the elements the material is assumed to be elastic-perfectly plastic (EPP) steel. No geometric nonlinearity is considered in these analyses. The stiffness from the gravity frames is also neglected.

The yield displacement is calculated based on a nonlinear pushover analysis with the IBC 2006 [15] recommended lateral force distribution. The roof displacement vs. base shear plot is bilinearized by equating the areas under the actual pushover curve and the approximate one, and thus the yield point is obtained. The achieved ductility (μ_a) is calculated as the ratio of peak roof displacement to the roof displacement at yield. Table 3.2 presents the results for designs corresponding to plate aspect ratio (h_s:L) 1:1. Each design is identified here with a specific record and target ductility it is designed for. This table also provides a measure of the effectiveness of the proposed design procedure based on how close the achieved ductility is to the target. The absolute maximum difference measured as percentage of μ_t is found to be 40.8%, whereas the mean difference is –17.8%. In addition to the peak roof displacement, the displacement profiles are also studied in order to check for any localized concentration of plasticity in any story. For example, the displacement profiles at the instant of peak roof drift for the three Northridge designs are shown in Figure 3. This shows that the design procedure is very effective in distributing drift almost uniformly over the height of the building.

These results are based on original beam dimension (AISC section W14X145), since the design method as discussed in Section 2 does not include design of the beam section. Although these pin-connected beams do not carry any moment due lateral loads, they influence the behaviour of the SPSW by changing the inclination of the principal tensile direction (Eqn. 3.1). Four out of the eight original designs are further refined by changing the beam section, and the updated results are provided in Table 3.3. The selected beam dimensions satisfy the compactness criterion [3]. Table 3.3 very clearly shows that the effectiveness of the designs (in terms of achieving the target ductility) can be improved by tuning the beam dimension. The absolute maximum difference is reduced to 14.5% and the mean difference is only –3.04% for this set. This tuning is an iterative procedure, which involves updating the analytical model for α and the area of each strip. The general guideline based on this experience is to increase the beam dimension if μ_a is more than μ_t and, vice versa.

Table 3.2 Results for design of SPSW with aspect ratio 1:1

Design	Record	μ_t	μ_a	% difference
I	SYL	2	1.79	–10.5
II	SYL	3	2.49	–17.0
III	SYL	4	3.21	–19.8
IV	KJM	2	1.95	–2.50
V	KJM	3	2.81	–6.33
VI	KJM	4	2.37	–40.8
VII	TAZ	2	1.90	–5.00
VIII	TAZ	3	1.78	–40.7

Table 3.3 Results for redesigned SPSW systems with changed beam dimensions (aspect ratio 1:1)

Design	Record	μ_t	New beam section	μ_a	% difference
I-R	SYL	2	W14X82	2.08	+4.00
II-R	SYL	3	W14X99	2.99	–2.99
III-R	SYL	4	W14X53	3.42	–14.5
V-R	KJM	3	W14X82	3.04	+1.33

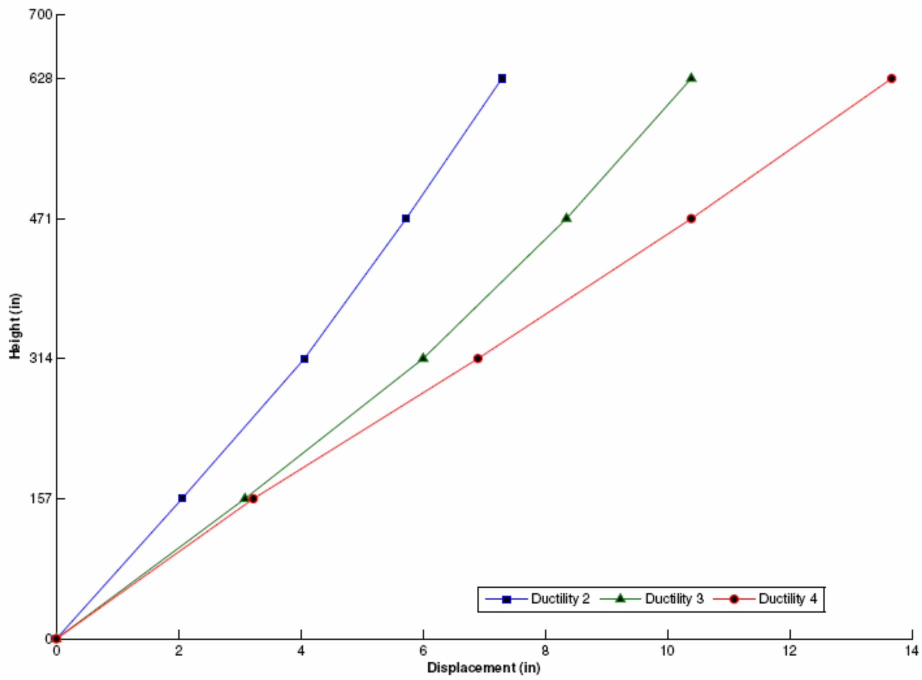

Figure 3 Displacement profiles at peak roof displacement for Designs I, II and III (1 in = 0.0254 m)

The design method is extended to SPSW configurations with panel aspect ratios other than 1:1. For this, we change the span of the SPSW bay of the original structure (Figure 1) to 1.5 times and 2 times of the original. The new designs (8 designs for each aspect ratio) are carried out following the same procedure described in Section 2, and the beam dimensions are also fine tuned in order to achieve ductility closer to the target. Tables 3.4 and 3.5 provide the details on these designs with aspect ratios (h_s:L) 1:1.5 and 1:2. The differences between the target and achieved ductility are also provided similar to Table 3.2. Tables 3.4 and 3.5 show that the proposed design procedure remains effective for aspect ratios other than 1:1 as well.

Table 3.4 Results for SPSW systems for plate aspect ratio 1:1.5

Design	Record	μ_t	μ_a	% difference
IX	SYL	2	2.09	+4.50
X	SYL	3	3.09	+3.50
XI	SYL	4	3.89	–2.75
XII	KJM	2	1.85	–7.50
XIII	KJM	3	2.80	–6.67
XIV	TAZ	2	2.06	+3.00
XV	TAZ	3	2.32	–22.7
XVI	TAZ	4	4.10	+2.50

Table 3.5 Results for SPSW systems for plate aspect ratio 1:2

Design	Record	μ_t	μ_a	% difference
XVII	SYL	2	1.97	–1.50
XVIII	SYL	3	2.99	–0.333
XIX	SYL	4	3.56	–11.0
XX	KJM	2	1.97	–1.50
XXI	KJM	3	2.81	–6.33
XXII	KJM	4	4.10	+2.50
XXIII	TAZ	2	2.00	0
XXIV	TAZ	3	2.42	–19.3

4. CONCLUDING REMARKS

An inelastic displacement-based design method for steel plate shear wall systems is presented in this paper. The method is applied to the design of 4-story steel frame structures, with different steel panel aspect ratios. The results show very clearly that this method (along with a suitable adjustment of the beam dimension) is able to achieve the target displacement ductility quite satisfactorily. The primary advantage of the proposed procedure is that (conceptually) it provides a very simplistic solution for obtaining a design of SPSW systems based on target inelastic drift and selected yield mechanism. It does not require any complicated analysis from the designer's/practicing engineer's part. The procedure remains simple while satisfying an advanced performance-based seismic design criterion, which makes it a prospective candidate for design codes.

Since the method works well for designs against specific earthquake records, it should be easily extended to designs using a code defined design spectrum. The proposed method needs to be validated for taller structures where the assumption of uniform and unidirectional story drifts during the peak response may not be realistic due to a larger participation of the higher modes. Also, for high-rise structures with large drifts, the P-Δ effects may not be negligible. The method, at its present state, is applicable to SPSW systems with pin-connected boundary beams. However, similar methods based on yield mechanism can be developed for other connection types as well.

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