International Journal of Advanced Structural Engineering, Vol. 1, No. 2, Pages 93-110, December 2009

Published online December 2009 at (http://journals.azad.ac.ir/IJASE)

DUCTILITY-BASED SEISMIC DESIGN OF STEEL PLATE SHEAR WALLS: PRACTICAL APPLICATION USING STANDARD SECTIONS

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Received 21 November 2009
Revised 4 December 2009
Accepted 7 December 2009

Over the past decade of extensive research works, the thin un-stiffened steel plate shear wall (SPSW) has now emerged as a promising lateral load resisting system. Considering the demand of performance-based seismic design (PBSD) philosophy in current and future seismic design codes, a ductility-based design was recently proposed for SPSW systems with pin-connected boundary beams. However, the effectiveness of that method was not tested using standard steel sections. The focus of this paper is to check the applicability of that PBSD procedure for practical designs of SPSW systems in the US and Indian context, using standard rolled steel sections available commercially in these countries. Based on sample design case studies on 4-story test buildings, the method is found to be a practicable solution for PBSD of SPSW systems. In addition, the distribution of inter-story drift over the height of the structure is also found to be suitable for adopting in design guidelines. The need for widening the range of available Indian Standard sections for realistic PBSD applications is recommended based on this study.

Keywords: steel plate shear walls, performance-based seismic design, ductility-based design, AISC steel sections, Indian Standard sections

1. Introduction

The thin unstiffened steel plate shear wall (SPSW) is now accepted as an efficient lateral load resisting system in building structures. Standard design guidelines or code specifications for

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SPSW are available in many countries, including Canada (CSA 2001) and USA (AISC 2005a, Sabelli and Bruneau 2007). SPSWs are sometimes preferred over other lateral load resisting systems because of the various advantages they provide (Astaneh-Asl 2001); primarily, substantial ductility, high initial stiffness, fast pace of construction, and the reduction in seismic mass. However, it should be noted that SPSWs are yet to gain wide acceptance in structural engineering practice similar to what steel moment frames or reinforced concrete shear walls have gained. The design of SPSW was implemented as early as 1970 as a primary (lateral) load resisting system. Initially, only heavily stiffened SPSWs, with closely spaced horizontal and vertical stiffeners, were used in order to resist the shear forces within their elastic buckling limits. Examples of such constructions are the Sylmar Hospital in Los Angeles and the Nippon Steel Building in Tokyo. These systems were not suitable for implementing in earthquake resistant design of structures effectively. Analytical and experimental research works on the response of SPSW against lateral loads carried out primarily in Canadian, US and UK universities, showed that the post-buckling ductile behaviour of the thin unstiffened SPSW is much more effective against seismic shaking than the elastic behaviour of the heavily stiffened SPSW. A list of important works is available in (Berman et al. 2005). The unstiffened plates, under cyclic load reversals, exhibit very stable hysteretic energy dissipation behaviour along with significant ductility, which make them very good lateral load resisting systems. However, the design codes that incorporate seismic design using SPSW, such as the CAN/CSA-16 (CSA 2001), the AISC Seismic Provisions (AISC 2005a) or the AISC design guide for steel plate shear walls (Sabelli and Bruneau 2007), so far, have not been able to fully exploit the ductility capacity since they incorporate this capacity implicitly (only through a force reduction factor, R).

Considering the general gradual shift of earthquake resistant design of structural systems from simplified force-based deterministic design methods towards performance-based seismic design (PBSD) techniques, Ghosh et al. (2009) recently proposed a displacement/ductility-based design methodology of steel plate shear wall systems with pin-connected boundary beams. The philosophy of PBSD emphasizes on better characterization of structural damage and on proper accounting for uncertainties involved in the design process. Considering this, the inelastic displacement- or ductility-based design approach was proposed as one of the prescriptive approaches of PBSD (SEAOC Vision 2000 Committee 1995). The method proposed by Ghosh et al. (2009) is a deterministic design technique which considers the target displacement ductility ratio (μ_t) as the design criterion. Thus it can utilize the ductility capacity of SPSW systems efficiently. It should be noted that the existing design guidelines (CSA 2001, Sabelli and Bruneau 2007) suggest the use of SPSW with rigid beam-to-column connections. A similar ductility-based design method for such SPSW systems is currently being developed by the authors of this article.

The design methodology proposed by Ghosh et al. (2009) is based on the energy balance during inelastic deformation and an assumed yield mechanism. Their method aims at designing a SPSW system to have a specific inelastic drift/displacement ductility under a given earthquake scenario. They applied this method for designing a 4-story steel structure with pin-connected beams with one SPSW bay, subjected to various ground motion scenarios and for different target ductility ratios. The method was found to be effective for those scenarios and for various steel plate panel aspect ratios. However, one major limitation of their work was that they considered hypothetical sections for designing the column sections (that is, vertical boundary elements or VBEs). These hypothetical sections conformed to a P-M interaction similar to what AISC-LRFD recommends (AISC 2005b), but the cross-sectional dimensions did not specifically belong to any section available in the steel tables provided by AISC, or anywhere else. The required design values (for example, required plastic moment capacity, M_u) were considered to be the section properties provided, in their design examples. Although, these hypothetical sections were suitable for checking how good the proposed theoretical design framework is, the design case studies presented by Ghosh et al. (2009) were not useful from a practical design perspective. A structural designer would like to see if the design method proposed in their work is also useful while using standard sections available in the market. The present paper aims at addressing this issue by applying Ghosh et al.'s (2009) ductility-based design methodology to a similar 4-story SPSW system with pin-connected beams under various ground motion scenarios and for different target ductility ratios, but using standard sections for the VBEs. Standard sections available in the US as per AISC (2005b) are used for a large set of design cases presented in this article. In addition, a few design cases are presented using standard sections available in India, as per the corresponding Indian Standard (BIS 1964). The effectiveness of using these sections for the proposed design method is checked based on how closely the real designs get to the target ductility ratios. These results are also compared with the outcome of the same designs using hypothetical sections created by Ghosh et al. (2009). The present paper also addresses the issue of using actual values of the angle of inclination of principal tensile stress (α_t) in individual steel plates in the multi-strip analysis (Thorburn et al. 1983) of SPSW systems. Ghosh et al. (2009) considered only an average value (over all 4 stories) of α_t in their analyses instead of the actual α_t for a particular story. Overall, the primary focus of the present article is to check the viability of practical application of a proposed performance-based seismic design method for SPSW systems in the US and Indian context.

The next section briefly reviews the design methodology proposed by Ghosh et al. (2009) and provides the step-wise design method. Case study 1, in Section 3, deals with the application of this method using standard AISC sections. The use of standard Indian sections is discussed in Section 4 (Case study 2). Section 5 provides a summary and the significant conclusions based on the work presented in this article.

2. Design Method Proposed by Ghosh et al. (2009)

The ductility-based design method used here is based on equating the inelastic energy demand on a structural system with the inelastic work done through the plastic deformations for a monotonic loading up to the target drift. This section presents a brief overview of the primary design formulation presented in (Ghosh et al. 2009). A simple SPSW system is considered for this where the beams are pin-connected at their ends to the columns, while the columns are fixed at their bases and are continuous along the height of the system, as shown in Figure 1(a).

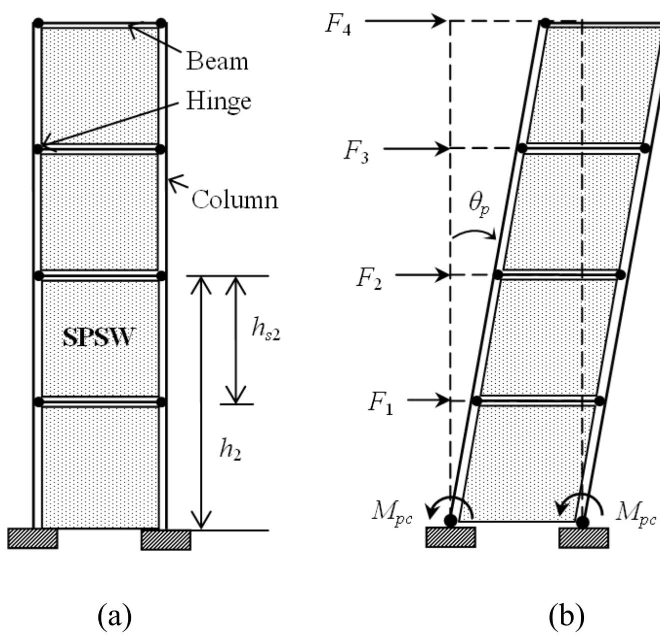

Figure 1. a) Schematic of the SPSW system; b) Selected yield mechanism

The total strain energy (elastic and plastic) which is imparted to an inelastic system, is estimated as:

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

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

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

The elastic force coefficient (C_e) is defined in terms of the design pseudo acceleration (A) or the design (elastic) base shear (V_e):

$$C_e = \frac{A}{g} = \frac{V_e}{W} \tag{3}$$

where, W is the seismic weight of the structure. The structure is idealized as an inelastic equivalent single degree system by selecting a typical yield mechanism for the peak monotonic demand, where the mechanism is composed of yielding of all the plates and plastic hinge formation at the base of the boundary columns (Figure 1b). The elastic strain energy demand (E_e) during this monotonic push is calculated based on the yield base shear, V_y, and substituting this in Equation (1), we get the plastic energy demand (E_p) as:

$$E_p = \frac{WT^2 g}{8\pi^2}\left[\gamma C_e^{\,2} - \left(\frac{V_y}{W}\right)^2\right] \tag{4}$$

This E_p is equated with the inelastic work done (W_p) through all the plastic deformations in the SPSW system:

$$W_p = \sum_{i=1}^{n} P_i h_{si}\theta_p + 2M_{pc}\theta_p \tag{5}$$

where n is number of stories, P_i is plastic shear capacity of the ith story steel plate, h_{si} is ith inter-story height, and M_{pc} is plastic moment capacity at each column base, θ_p is target plastic drift based on an assumed yield drift (θ_y) as shown in Figure 1(b) (an elastic-perfectly plastic behaviour is assumed here), and we get the required yield base shear (V_y) as:

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2}, \text{ where } \alpha = \left(\sum_{i=1}^{n}\lambda_i h_i\right)\frac{8\theta_p \pi^2}{T^2 g} \tag{6}$$

where, h_i is ith floor height, θ_p is target plastic drift based on an assumed yield drift (θ_y). The factor λ_i (= F_i/V_y) represents the shear force distribution in the SPSW system as discussed by Ghosh et al. (2009) Similar to their work, we adopt a distribution based on statistical studies on steel MRF systems (Lee and Goel 2001). However, as shown by Ghosh et al. (2009), other commonly used shear distributions, such as the one proposed for steel EBF systems (Chao and Goel 2005), or the one in IBC 2006 (ICC 2006), can also be adopted.

The required plate thickness at each story is obtained using the following equation:

$$t_i = \frac{2P_i}{0.95F_y L} = \frac{2V_i}{0.95F_y L} \tag{7}$$

where, V_i is ith story shear demand, F_y is material yield strength and L is bay width. The base column moment capacity (M_{pc}) is obtained as per recommendations by Roberts (1995).

$$M_{pc} = \frac{F_y t_1 h_1^2}{16} \tag{8}$$

The design axial force (P_c) on the columns is calculated based on the moment equilibrium about the base. The column section is selected from available steel tables as per AISC (2005b) or Indian Standard (BIS 1964) for these demands based on the code prescribed P-M interaction and the criterion for compact section. This design is further modified by tuning the pin-connected beam member so as to achieve actual ductility ratio closer to the target ductility ratio. A design flowchart is provided in Figure 2 giving the individual design steps.

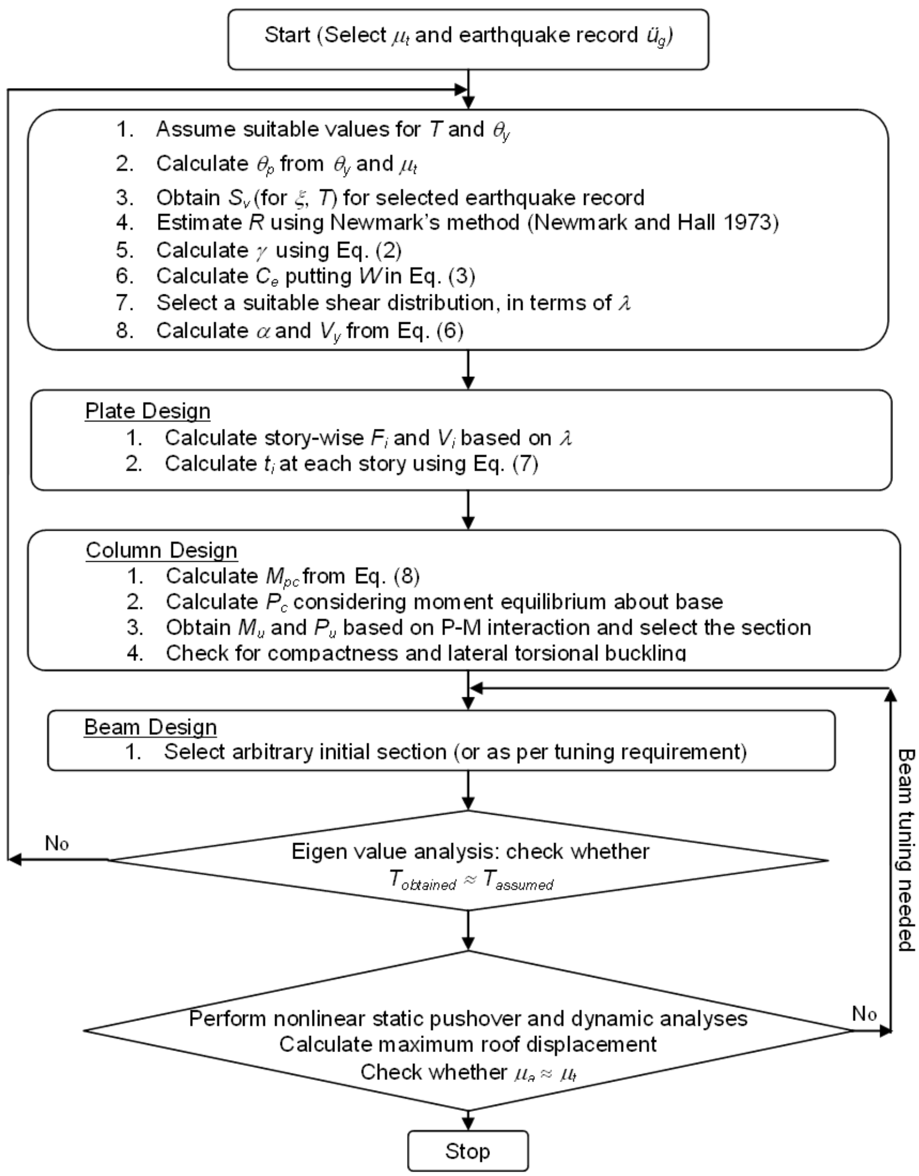

Figure 2. Flowchart for the ductility-based design method (Ghosh et al. 2009)

3. Case Study 1: Application Using Standard AISC Sections

A 4-story steel frame building with pinned beam to column connections (Figure 3) 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 building is assumed to have seismic weights of 4693 kN per floor, except for the roof where it is 5088 kN. The SPSW is designed against specific earthquake records for selected target ductility ratio (μ_t) values. This ductility ratio is defined in terms of the roof displacement. Three strong motion records from the 1994 Northridge, USA and 1995 Kobe, Japan earthquakes (Table 1) are used for this case study. Each design is identified by a ground motion and a target ductility ratio. The same structure, design cases and ground motion records were in (Ghosh et al. 2009) for designing with hypothetical column sections. Similar to the approach used by them for measuring the effectiveness of the designs, the new designs with standard sections available from AISC tables (AISC 2005b) are checked against the same records through nonlinear response-history analysis in order to obtain the achieved ductility ratio (μ_a). Details for all the designs and analyses presented here (including the results shown in the next section) are available in a detailed report (Gupta 2009).

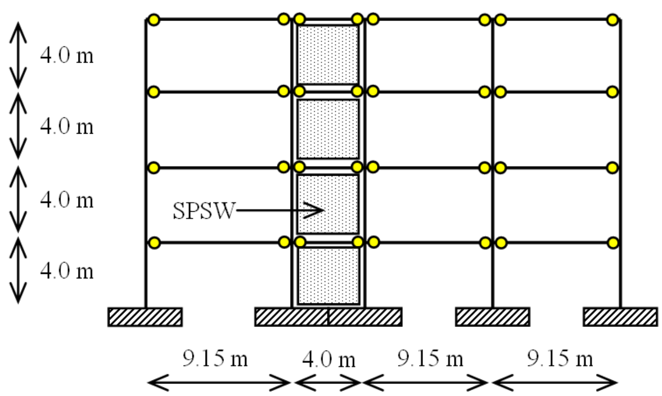

Figure 3. Configuration of the 4-story study frame with SPSW

The assumption of a suitable yield drift (θ_y) is based on observed behaviour (under static incremental loads) of SPSW systems. Like most other design procedures, the proposed procedure also needs an initial assumption of the fundamental time period (T), which involves iteration. The number of iterations needed to reach convergence depends on the experience of a designer. The actual required thicknesses of the SPSW panels as per the design calculation are provided in each design, without any due consideration to the availability of such precise thicknesses for steel sheets. However, the column sections provided (with moment capacity M_u and axial force capacity P_u) are based on available sections in the market (or at least in the design codes) design requirements (M_{pc} and P_c). This is the primary difference with the sample design cases studied by Ghosh et al. (2009).

Table 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

As mentioned earlier, the steel plate is modelled using the multi-strip modelling technique (Thorburn et al. 1983) for nonlinear static and response-history analyses, in which the diagonal strip/truss members are aligned along the principal tensile direction (α_t) of the plate (Timler and Kulak 1983):

$$\tan^4 \alpha_t = \frac{1 + \dfrac{tL}{2A_c}}{1 + th_s\left(\dfrac{1}{A_b} + \dfrac{h_s^3}{360 I_c L}\right)} \tag{9}$$

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, and t = plate thickness. 10 strips, the minimum number recommended in previous literatures, are used to model each plate panel. The actual α_t for each story are used for all the analyses, whereas Ghosh et al. (2009) have used an average value for all 4 stories. The SPSW system is modelled and analyzed using the structural analysis program DRAIN-2DX (Prakash et al., 1993) using nonlinear truss and beam-column elements. For all the elements the material is assumed to be elastic-perfectly plastic steel with yield stress, F_y = 344.74 MPa (= 50 ksi), and without any overstrength factor. The system is modelled using a lumped mass model with 5% Rayleigh damping (in the first two modes) for the response-history analysis. Geometric nonlinearity and the nominal lateral stiffness from the gravity frames are neglected in these analyses. The detail design calculations for a sample design case (Design III) are provided in Appendix.

Table 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 the target ductility ratio 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 37.0%, whereas the mean difference is –16.1%. For comparison, results for these design cases using hypothetical column section, as per Ghosh et al. (2009), are also presented in the same table. The results show that there is no significant change in the results for replacing the hypothetical section with a standard AISC section. The design with an AISC section remains as effective as the one with a hypothetical column section. Table 2 presents the results for design cases when the assumed beam (W14×145) section is not tuned. Table 3 presents the results when 4 of the designs here presented in Table 2 are revised through beam tuning. The result improves for each of the revised cases and overall (for 4 designs) the mean difference changes from –16.1% to 0.15%. Similar improvements in results were also observed for the design cases with hypothetical column section that are presented alongside in Table 3.

Table 2. Result summary for designs with AISC sections (steel panel aspect ratio 1:1)

Design	Record	μ_t	AISC section		Hypothetical	
			μ_a	% difference	μ_a	% difference
I	SYL	2	1.53	–23.5	1.83	–8.50
II	SYL	3	2.66	–11.3	2.87	–4.33
III	SYL	4	3.34	–16.5	3.20	–20.0
IV	KJM	2	2.02	1.20	2.04	2.00
V	KJM	3	2.81	–6.33	2.96	–1.33
VI	KJM	4	2.52	–37.0	2.45	–38.8
VII	TAZ	2	1.94	–3.00	2.03	1.50
VIII	TAZ	3	2.02	–32.7	1.82	–39.3
Average				–16.1		–13.6
Abs. max.				37.0		39.3

Table 3. Result summary for beam-tuned systems with AISC sections (steel panel aspect ratio 1:1)

Design	Record	μ_t	AISC section		Hypothetical	
			μ_a	% difference	μ_a	% difference
I-R	SYL	2	2.11	5.50	2.05	2.50
II-R	SYL	3	3.04	1.33	3.05	1.67
III-R	SYL	4	3.63	–9.25	3.55	–11.3
V-R	KJM	3	3.09	3.00	3.02	0.670
Average				0.15		–1.60
Abs. max.				9.25		11.3

In addition to the ductility achieved in terms of the peak roof displacement, the displacement profiles are also studied in order to check for any localized concentration of plasticity in any story. Figure 4 and Figure 5 present the displacement profiles at the instant of peak roof drift for the three Northridge and two Kobe (Takarazuka) designs. These figures show that the design procedure remains very effective, even while using standard column sections, in distributing drift almost uniformly over the height of the building for these five design cases.

In terms of using the actual angle of inclination of the tension strips (α_t), the change in analysis results for these design cases is found to be almost negligible. For all the design cases, α_t is maximum at the lowest story and it decreases as we go up. The difference between the maximum and minimum α_t values for any design is below 1.5° (Gupta 2009). Based on this set of 24 sample design cases, it can be recommended that an average α_t can be used for all stories for the analysis of a SPSW system. This will reduce much of the computation in creating an analytical model of the structure.

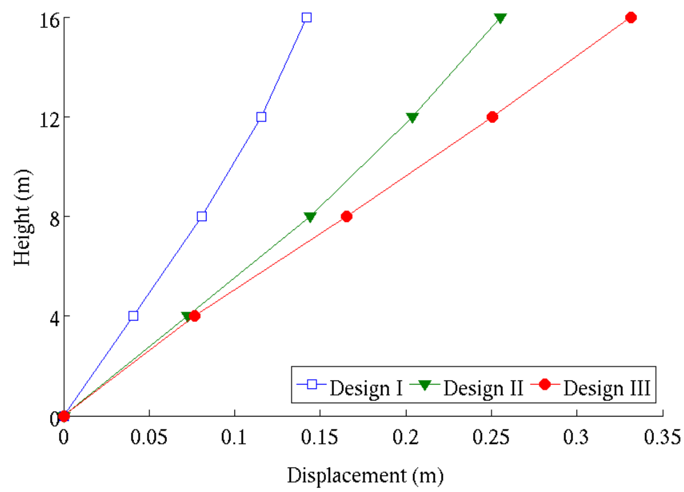

Figure 4. Displacement profiles at peak roof displacement for Designs I, II and III

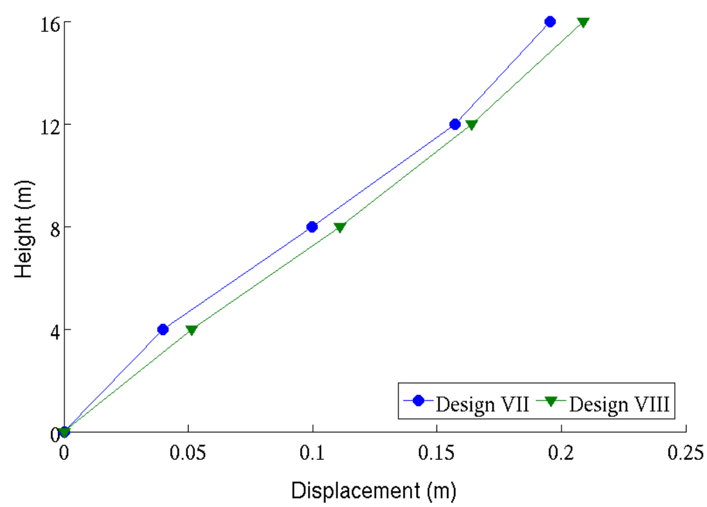

Figure 5. Displacement profiles at peak roof displacement for Designs VII and VIII

With regards to tuning of boundary beams, Ghosh et al. (2009) suggested that if the achieved ductility ratio is less than the target, a closer ductility ratio can be achieved by reducing the pin-connected beam section, and vice versa. A detailed and closer look at this issue reveals that this is not always true, although this recommendation generally holds good. Figure 6 presents the beam tuning results summary for Design III, where the achieved ductility ratio values are written next to the corresponding beam section selected. For μ_t = 4, the first trial with an assumed beam section of W14×145 gives μ_a = 3.34. As we select a lighter section (W14×43) for the beam, the achieved ductility ratio goes up to 3.63. However, for the next three trials, with reducing beam sections, the increase in ductility ratio is negligible, and for the two lightest beam sections (W12×16 and W12×14) the achieved ductility ratio again starts reducing. It can be concluded from this study that the achieved ductility ratio is not a monotonic function of the capacity of the pin-connected beam.

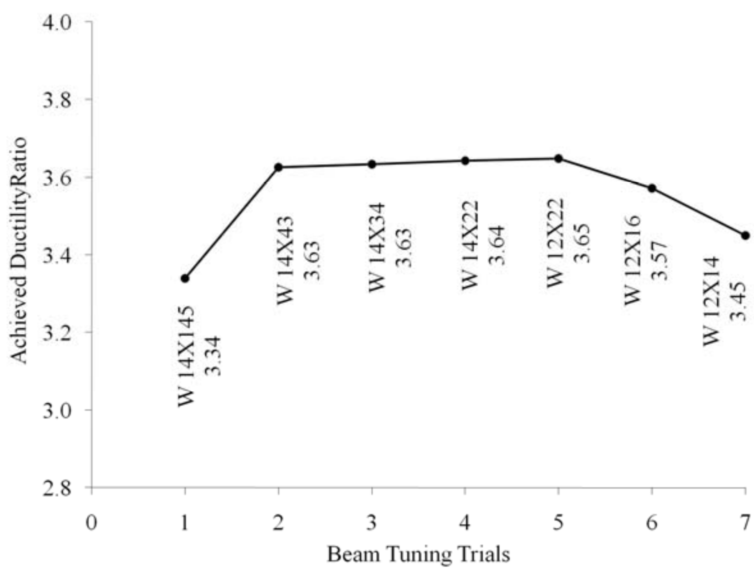

Figure 6. Change in the achieved ductility ratio due to beam tuning for Design III; (The number next to the beam section provides the corresponding μ_a)

3.1. Designs for Steel Panel Aspect Ratios of 1:1.5 and 1:2

The same ductility-based design method is applied to the designs of SPSW configurations with panel aspect ratios (h_s:L) 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 4-story structure remains the same otherwise. A similar exercise was conducted for designs with hypothetical column sections as well. The new designs (8 designs for each aspect ratio) are carried out following the procedure illustrated in Figure 2, and the beam dimensions are also fine tuned in order to achieve ductility closer to the target. Tables 4 and 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 the achieved ductility are also provided similar to Table 2. These results show that the proposed design procedure remains very effective for aspect ratios other than 1:1 as well. For designs with aspect ratio 1:1.5 (Table 4), the absolute maximum difference between the achieved and target ductility ratio, measured as percentage of μ_t, is found to be 11.3%, whereas the mean difference is –3.79%. The corresponding values for designs with hypothetical sections as presented in the same table were 20.3% and –4.45%, respectively. These data show that the designs are almost same – if not slightly better, overall – in terms of achieving the target ductility while using the standard AISC sections. For designs with aspect ratio 1:2, the designs using standard AISC sections remain similarly effective in achieving the target ductility (Table 5). The absolute maximum and the mean difference values for the designs with aspect ratio 1:2 are 13.3% and –1.17%, respectively. For designs with hypothetical column sections the absolute maximum and the mean differences between μ_t and μ_a were17.5% and –4.52%, respectively.

These results altogether illustrate very clearly that we can achieve designs sufficiently close to the target for the range of aspect ratios from 1:1 to 1:2, while using standard AISC sections for the ductility-based design of SPSW systems, and the effectiveness of these designs is very similar to the designs presented by earlier researchers using hypothetical ("capacity = demand") column sections. The primary reason for this success with the AISC sections is that the selected column sections for any design provides bending moment and axial force capacities very close to the demands for that design.

Table 4. Result summary for designs with AISC sections (steel panel aspect ratio 1:1.5)

Design	Record	μ_t	AISC section		Hypothetical	
			μ_a	% difference	μ_a	% difference
IX	SYL	2	1.90	–5.00	2.01	0.500
X	SYL	3	3.23	7.67	2.99	–0.333
XI	SYL	4	3.80	–5.00	3.75	–6.25
XII	KJM	2	1.96	–2.00	1.98	–1.00
XIII	KJM	3	2.68	–10.7	2.77	–7.67
XIV	TAZ	2	2.01	0.50	2.07	3.50
XV	TAZ	3	2.66	–11.3	2.39	–20.3
XVI	TAZ	4	3.82	–4.50	3.84	–4.00
Average				–3.79		–4.45
Abs. max.				11.3		20.3

Table 5. Result summary for designs with AISC sections (steel panel aspect ratio 1:2)

Design	Record	μ_t	AISC section		Hypothetical	
			μ_a	% difference	μ_a	% difference
XVII	SYL	2	1.92	–4.00	1.91	–4.50
XVIII	SYL	3	3.11	3.67	3.02	0.670
XIX	SYL	4	3.75	–6.25	3.76	–6.00
XX	KJM	2	2.17	8.50	1.97	–1.50
XXI	KJM	3	3.15	5.00	3.13	4.33
XXII	KJM	4	3.60	–10.0	3.30	–17.5
XXIII	TAZ	2	2.14	7.00	2.08	4.00
XXIV	TAZ	3	2.60	–13.3	2.53	–15.7
Average				–1.17		–4.52
Abs. max.				13.3		17.5

4. Case Study 2: Applications Using Indian Standard Sections

The same ductility-based design method is also applied for the design of the 4-story SPSW structure, mentioned in the previous section, using Indian Standard sections. The primary reason for checking the same design method with another set of standard sections is that the steel tables as per the Indian Standard (BIS 1964) do not cover a similar wide range (as AISC) in terms of ultimate moment and axial force capacities of the rolled sections. Besides, the number of sections available in that range is also less compared to the AISC tables (AISC 2005b). These limitations may result in: a) standard rolled sections not being available for designs subjected to strong earthquakes and low target ductility ratios, and b) capacities of the actual section provided being very different from the design requirements or demands. In addition, this may also reduce the effectiveness of tuning the pin-connected beams to achieve ductility ratios closer to the target.

It is observed that standard rolled sections are not available in SP: 6(1) (BIS 1964) or the current IS: 800 tables (BIS 2007) to meet the demands for designs subjected to the records in Table 1, even for μ_t up to 4. Therefore, for these design cases we scale down the selected records. The design scenarios for a few sample designs with Indian Standard sections with the scaled records and the selected target ductility ratios are provided in Table 6. The design procedure remains the same as in the previous section. Yield stress for these sections are considered to be F_y = 250 MPa. The results of these three sample designs (including tuning of beams), along with their counterparts considering hypothetical column sections, are also provided in Table 6. These results show that the designs with Indian Standard sections are as effective in achieving the target ductility ratios as the designs with their hypothetical counterparts. The absolute maximum difference between the target and the achieved ductility ratio using Indian Standard sections is 19.0% and the mean is –11.4%. The column sections used in these three designs are ISWB 550 for Design XXV and ISWB 600 for Designs XXVI and XXVII. ISWB 600 is the heaviest section as per SP: 6(1) (BIS 1964). The ground motion scale factors are so selected that the column bending moment and axial force demands are within the capacities this section. Although column sections were available to closely meet the design demands for these three cases, this may not be the case for all designs using Indian Standard sections because of the wide gaps (in terms of capacity) that exist between two successive sections available in SP: 6(1). The difficulty of beam tuning using Indian rolled sections, resulting from the same lack of closely-spaced sections, cannot be judged from a comparison with the designs using hypothetical column sections, because those designs also use the Indian Standard sections for beams. In addition, for these design cases subjected to weak ground motion records and large μ_t, the required thickness of the steel plate panel becomes very low. The commercial availability of such plates may be a problem for practical design applications.

Table 6. Result summary for designs with Indian Standard sections (steel panel aspect ratio 1:1)

Design	Record	Scale factor	μ_t	IS section		Hypothetical	
				μ_a	% difference	μ_a	% difference
XXV	SYL	0.35	4	4.00	0	3.52	–12.0
XXVI	KJM	0.90	4	3.39	–15.3	3.34	–16.5
XXVII	TAZ	0.78	4	3.24	–19.0	3.28	–18.0
Average					–11.4		–15.5
Abs. max.					19.0		18.0

It should be stated, however, that the limitation in section capacities as per SP: 6(1) does not make the application of ductility-based design of SPSW for earthquake resistant design of buildings in the Indian scenario somewhat impractical. The present study does not consider the use of built-up sections for boundary columns, which is expected to broaden the range of application to a great degree, specifically for strong earthquake-low target ductility design cases. These sections will also let us achieve more effective designs as the capacities of the column can be easily tuned to the design requirements. The use of built-up sections need a detailed investigation both in terms of design advantage and economic viability, and this is beyond the scope of present article.

5. Summary and Conclusions

The focus of this paper is in the practical application of an inelastic displacement-based design method, developed earlier, for steel plate shear wall systems. This method is applied to the design of 4-story steel frame structures, with different steel panel aspect ratios, using standard AISC and Indian rolled sections. The following significant conclusions can be drawn from the work presented here:

1. The applications using standard AISC sections are very satisfactory and as effective in

achieving the target ductility ratio as the hypothetical designs presented by Ghosh et al. (2009).

2. Thus, the method discussed in Section 2 becomes a practical performance-based design solution for earthquake resistant design problems. This method, as provided in the design flowchart (Figure 2), is simple but efficient enough for adopting in standard design guidelines.

3. The use of the actual value of α_t (inclination angle of the tension field) does not alter the results significantly. Therefore, the use of an average α_t is recommended for practical purpose. The ability to use average α_t for analysis/design checking makes it more appealing to a practicing engineer.

4. The use of Indian Standard sections following the same design method is also found to be satisfactory.

5. However, due to the lack of available standard rolled Indian sections with large capacities, the application gets limited to weak earthquake-large ductility designs. In order to be able to utilize this or similar advanced earthquake design methods, the range of available sections in India needs to be enhanced.

6. Also, more closely-spaced sections (in terms of section dimensions/capacities) need to be available to apply this method effectively in the Indian scenario. Built-up sections need to be explored in detail for application of the ductility-based SPSW design method in the Indian context.

Appendix: Design Example of Case Study 1

The detail design calculations for a sample design case (Design III) are provided here for example:

- Selected record: SYL
- Target ductility ratio selected for this design, $\mu_t = 4$
- Yield drift (based on roof displacement) assumed for design, $\theta_y = 0.01$
- Plastic drift for the selected μ_t and θ_y, $\theta_p = 0.03$
- Fundamental period of the structure, $T = 0.90$ sec
- Pseudo velocity for T from the 5% SYL spectrum, $S_v = 2.26$ m/sec
- From Equation (2), $\gamma = 0.44$
- Seismic weight of the system, $W = 19.17\times10^3$ kN

- From Equation (3), $C_e = 1.606$
- From Equation (6), $\alpha = 4.09$, and $V_y = 4.968\times10^3$ kN
- Based on the assumed shear distribution, the design equivalent lateral forces, from top to bottom: $F_4 = 3188$ kN, $F_3 = 981.7$ kN, $F_2 = 546.2$ kN, and $F_1 = 251.5$ kN
- Story shears from top to bottom: $V_4 = 3188$ kN, $V_3 = 4169$ kN, $V_2 = 4716$ kN, and $V_1 = 4968$ kN.
- Plate thicknesses provided based on Equation (7), from top to bottom: $t_4 = 4.51$ mm, $t_3 = 5.90$ mm, $t_2 = 6.68$ mm, and $t_1 = 7.03$ mm
- Based on Equation (8), $M_{pc} = 2.483\times10^3$ kNm, and $P_c = 15.80\times10^3$ kN
- Using P-M interaction, the demands on the boundary columns are calculated as: $M_u = 7.852\times10^3$ kNm, and $P_u = 21.60\times10^3$ kN
- Standard AISC column section selected for these demands: W36×330
- Capacities of the selected column section: $M_u = 7.965\times10^3$ kNm, and $P_u = 21.73\times10^3$ kN

The nonlinear pushover analysis gives a yield displacement of 0.103 m. The nonlinear response-history analysis subjected to the SYL record gives a peak roof displacement of 0.344 m. The achieved ductility ratio (μ_a) is calculated as the ratio of peak roof displacement to the roof displacement at yield, and comes to be 3.34 for this design case.

Acknowledgements

Support for this research on developing a displacement-based design method for steel plate shear wall structures is provided by the Department of Science and Technology (DST), India. The findings and the opinions expressed herein are those of the authors and do not necessarily represent the views of DST.

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