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# 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. However, seismic code provisions for these systems are still based on elastic force-based design methodologies. Considering the ever-increasing demands of efficient and reliable design procedures, a shift towards performance-based seismic design (PBSD) procedure is proposed in this work. The proposed PBSD procedure for SPSW systems is based on a target inelastic drift and pre-selected yield mechanism. This design procedure is simple, yet it aims at an advanced design criterion. The proposed procedure is tested on a four-story test building with different steel panel aspect ratios for different target drifts under selected strong motion 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. In addition, the displacement profiles at peak responses 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.



## 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. The 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 SPSWs was implemented as early as 1970 as a primary 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, as in the Sylmar Hospital in Los Angeles, the Nippon Steel Building in Tokyo, etc. These systems were not suitable for implementing in the earthquake resistant design of structures. With the analytical and experimental research carried out by various researchers, in Canadian, US and UK universities (a list of important works is available in [2]), it was observed that the post-buckling ductile behaviour of an unstiffened SPSW is much more effective against seismic shaking than the elastic behaviour of an stiffened SPSW, since these unstiffened plates exhibit very stable hysteretic energy dissipation behaviour. However, the design codes which incorporate seismic design using SPSWs, such as the CAN/CSA-16 [3], the AISC Seismic Provisions [4] or FEMA 450 [5], so far, only implicitly (through a force reduction factor, $R$) 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 directly such inelastic performance parameters in defining design criteria for other lateral load resisting systems, for example [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 SPSWs explicitly considering an inelastic drift/displacement criterion.

The displacement-based technique which is most commonly proposed by researchers for the inelastic seismic design of structures is known as the direct displacement-based design (DDBD) [7]. The primary postulate in a DDBD is the idealization of the inelastic structure as an elastic single degree oscillator with equivalent stiffness and equivalent damping. Various flavours of DDBD are popular among researchers working in the development of advanced seismic design techniques. The design methodology

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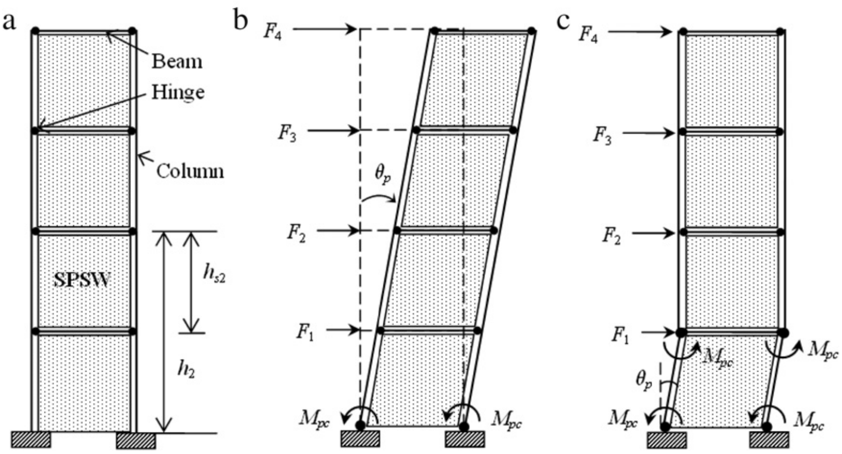


**Fig. 1.** (a) Schematic of the SPSW system; (b) Selected yield mechanism; (c) Soft ground story.

adopted in this article, however, is a different one, which is based on the energy balance during inelastic deformation and an assumed yield mechanism. The method proposed in this article aims at designing a SPSW system to have a specific inelastic drift/displacement ductility under a given earthquake scenario. The basic design framework is described in the next section. The main objective of this paper is to validate the effectiveness of this method by designing a four-story steel structure with pin-connected beams with one SPSW bay, which is discussed in Section 3. The effectiveness is measured in terms of how close the achieved inelastic displacement is to the target.

## 2. Proposed design framework

The proposed design formulation considers the inelastic energy demand on a structural system, and this energy is equated with the inelastic work done through the plastic deformations for a monotonic loading up to the target drift. This formulation, with various modifications, was used earlier for the design of steel moment frames [8,9], and steel eccentric braced frames [10]. This article presents the primary formulation of a similar inelastic displacement-based design procedure for SPSWs. The detailed design methodology for the inelastic design of SPSW systems is available in [11]. We consider a simple SPSW system 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 Fig. 1(a).

Following the method described by Lee and Goel [9], we can estimate the total strain energy (elastic and plastic) which is imparted to an inelastic system, by modifying the original proposal by Akiyama [12], 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 $E_e$ = elastic strain energy demand, $E_p$ = plastic strain energy demand, $\gamma$ = energy modification factor, $M$ = total mass of the structure, $S_v$ = pseudo velocity corresponding to $T$, $T$ = fundamental period, $C_e$ = elastic force coefficient, and $g$ = gravitational acceleration. The energy modification factor can be calculated based on the target ductility ratio of the system ($\mu_t$) and ductility reduction factor ($R$), as

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

The elastic force coefficient ($C_e$) is defined only 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. Eq. (3) is considered to be valid for all the oscillator period ranges. The multi-degree of freedom (MDOF) system is idealized as an inelastic equivalent system by selecting a typical yield mechanism for the peak monotonic demand during the ground vibration. The mechanism is composed of yielding of all the plates and plastic hinge formation at the base of the boundary columns, as shown in Fig. 1(b). The inelastic equivalent single degree of freedom (SDOF) system is assumed to undergo an elastic–perfectly plastic lateral force–deformation behaviour under a monotonic thrust during the vibration. The elastic strain energy demand during this monotonic push is calculated based on the yield base shear, $V_y$:

$$E_e = \frac{1}{2}M\left(\frac{TV_y}{2\pi W}g\right)^2. \tag{4}$$

Substituting $E_e$ in Eq. (1), the plastic energy demand on the structure is obtained as

$$E_p = \frac{WT^2 g}{8\pi^2}\left[\gamma C_e^2 - \left(\frac{V_y}{W}\right)^2\right]. \tag{5}$$

This $E_p$ should be equal to the inelastic work done through all the plastic deformations in the SPSW system.

In order to estimate the plastic energy dissipation in a SPSW system during the peak monotonic displacement, we consider that all steel plates reach their plastic shear capacity and that plastic hinges form at both the column bases. It is assumed that the plates and the column bases become fully plastic at the same instant. We also assume all the plastic deformations in the plane of the system to be unidirectional and story drift ratios to be uniform along the height of the building. The inelastic rotation up to the maximum drift is $\theta_p$, as shown in Fig. 1(b). For this yield mechanism, without considering the gravity load or P–Δ effects, we can find the total inelastic work [13]:

$$W_p = \sum_{i=1}^{n} P_i h_{si}\theta_p + 2M_{pc}\theta_p \tag{6}$$

where $n$ = number of stories, $P_i$ = plastic shear capacity of the $i$th story steel plate, $h_{si}$ = $i$th inter-story height, and $M_{pc}$ = plastic moment capacity at each column base. Equating $W_p$ with the estimated inelastic strain energy, we get the required yield base shear ($V_y$) as

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2}, \quad \text{where } \alpha = \left(\sum_{i=1}^{n} \lambda_i h_i\right) \frac{8\theta_p \pi^2}{T^2 g} \tag{7}$$

where $h_i$ = $i$th floor height and $\theta_p$ = target plastic drift based on an assumed yield drift ($\theta_y$). The factor $\lambda_i$ ($= F_i/V_y$) represents the shear force distribution in the SPSW system. This factor should be obtained based on a statistical study of peak inelastic story shear distributions in standard SPSW systems under various earthquake scenarios. Because of the lack of available standard designs for SPSW systems, we adopt a distribution based on statistical studies on steel MRF systems [9]. However, any other commonly used shear distribution, such as the one proposed for steel EBF systems [10], or the one in IBC 2006 [14], can also be adopted. Further details on the effect of the assumed shear distribution on the designs are provided in Section 3.2.

The required plate thickness at each story is obtained by considering that the plate carries the full plastic shear:

$$t_i = \frac{2P_i}{0.95 F_y L} = \frac{2V_i}{0.95 F_y L} \tag{8}$$

where $V_i$ = $i$th story shear demand, $F_y$ = material yield strength and $L$ = bay width. The plate plastic shear capacity ($P_i$) is calculated based on a multi-strip idealization [15]. The factor 0.95 in Eq. (8) represents the mean bias for the angle of inclination of the principal tensile stress in the steel plate (see Eq. (12) later), while considering a 45° nominal value for this inclination. The detailed derivation for this factor is provided in [16]. The base column moment capacity ($M_{pc}$) is obtained based on Driver et al.'s recommendation [17] for ensuring full plasticity in steel plates before any inelasticity in the boundary columns:

$$M_{pc} = \frac{50 t_1 h_1^2}{16}. \tag{9}$$

The design 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 [18]. It needs to be checked that a soft story does not form for the selected column section by using

$$V_i \le \frac{4M_{pc}}{h_{si}} + P_i \tag{10}$$

for each story, where $V_i$ = shear demand on the $i$th story. It should be noted that, for the top story, Eq. (10) changes to

$$V_n \le \frac{2M_{pc}}{h_{sn}} + P_n. \tag{11}$$

However, based on the consideration that the steel plates carry full story shear ($P_i = V_i$), the checks against soft story formation are automatically satisfied.

It should be noted here that this design procedure does not involve the design of pin-connected beam members, since these beams do not carry any moment as per the assumed yield mechanism given in Fig. 1(b). However, it will be discussed later in Section 3.1 that the beam dimension affects the overall behaviour of the system and the design can be tuned further beyond the procedure given in this section. A design flowchart is provided in Fig. 2, giving the individual design steps.

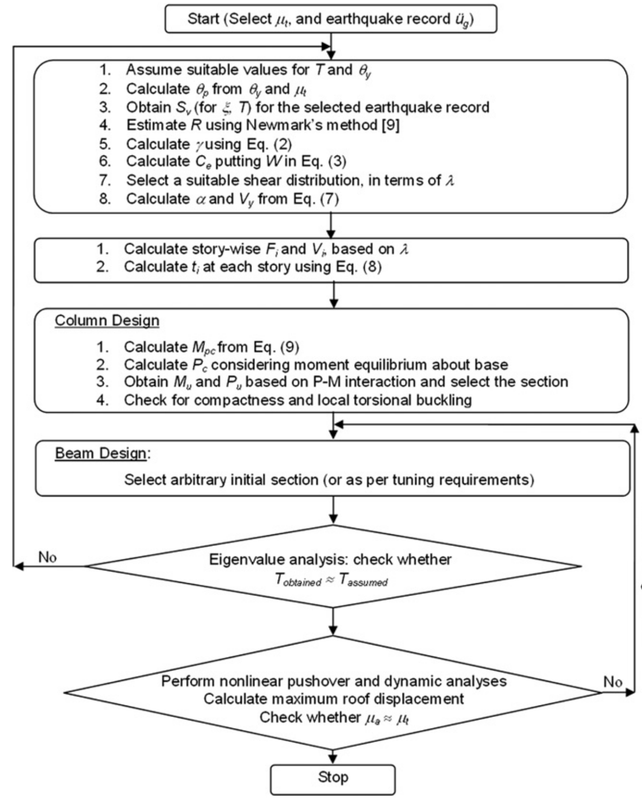


Fig. 2. Flowchart for the proposed design method.

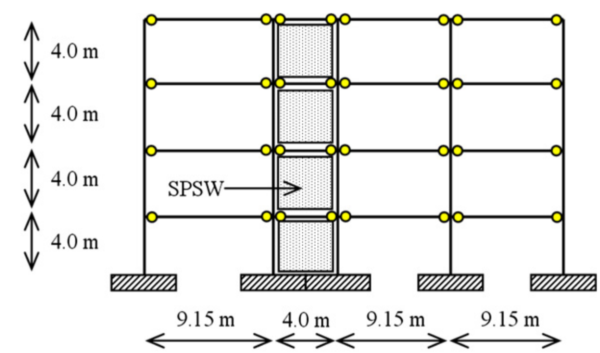


Fig. 3. Configuration of the study frame with an SPSW.

## 3. Application of the proposed design procedure

A four-story steel frame building with pinned beam to column connections (Fig. 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 ($\mu_t$) 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 1) are used for this case study. The details regarding these and other designs are available in [16]. The designed buildings are checked against the same records through nonlinear response-history analysis to measure the effectiveness of the proposed design procedure in terms of the achieved ductility ratio ($\mu_a$).

The actual design procedure based on $\mu_t$ involves the assumption of yield drift ($\theta_y$). $\theta_y$ is defined based on a nonlinear

**Table 1**
Details of earthquake records used for design.

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

static pushover analysis of the SPSW system with the IBC 2006 [14] 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 assumption of a suitable yield drift is based on the observed behaviour (under static incremental loads) of SPSW systems. The design process may need to be iterated a few times in order to achieve a convergence for this parameter. Similarly, like most other design procedures, the proposed procedure also needs an initial assumption of the fundamental time period ($T$), and this may involve iteration as well. The number of iterations needed to reach convergence depends on the experience of the designer. However, it is not difficult to reach convergence in terms of $\theta_y$, since its value does not change significantly for a wide range of target ductility ratios and ground motion scenarios. The actual required thicknesses of the SPSW panels as per the design calculation are provided here, without any due consideration to the availability of such precise thicknesses for steel sheets. Similarly, the column sections provided (with moment capacity $M_u$ and axial force capacity $P_u$) are based on design requirements ($M_{pc}$ and $P_c$). These hypothetical sections follow a P–M interaction as per the AISC-LRFD code [18], although the sections do not belong to any standard section table. The hypothetical dimensions are used so that the real effectiveness of the proposed procedure (as reflected by the calculated $t_i$, $M_u$ and $P_u$) can be measured. It is observed that redesigning for a few of the design cases reported in this paper with real column dimensions does not have any significant effect on the effectiveness of the proposed procedure [16]. A sample design case with standard column sections is discussed in Section 3.3. The required column section for the bottom story is provided at all the stories.

For the nonlinear static and response-history analyses of the structure, the steel plate is modelled using the multi-strip idealization [15], 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 ($\alpha_t$) of the plate [19]:

$$\tan^4 \alpha_t = \frac{1 + \frac{tL}{2A_c}}{1 + th_s\left(\frac{1}{A_b} + \frac{h_s^3}{360 I_c L}\right)} \tag{12}$$

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. Ten 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 [20]. 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 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. No geometric nonlinearity is considered in these analyses. The stiffness from the gravity frames is also neglected.

The details of one design calculation (Design III) are provided here for example:

**Table 2**
Result summary for designs of the SPSW with aspect ratio 1:1.

| Design | Record | $\mu_t$ | $\mu_a$ | % difference |
|---|---|---|---|---|
| I | SYL | 2 | 1.83 | −8.50 |
| II | SYL | 3 | 2.87 | −4.33 |
| III | SYL | 4 | 3.20 | −20.0 |
| IV | KJM | 2 | 2.04 | +2.00 |
| V | KJM | 3 | 2.96 | −1.33 |
| VI | KJM | 4 | 2.45 | −38.8 |
| VII | TAZ | 2 | 2.03 | −1.50 |
| VIII | TAZ | 3 | 1.82 | −39.3 |
| Average | | | | −13.6 |

- 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 $\mu_t$ and $\theta_y$, $\theta_p = 0.03$
- Fundamental period of the structure, $T = 0.90$ s
- Pseudo velocity for $T$ from the 5% SYL spectrum, $S_v = 2.26$ m/s
- From Eq. (2), $\gamma = 0.44$
- Seismic weight of the system, $W = 19.17 \times 10^3$ kN
- From Eq. (3), $C_e = 1.606$
- From Eq. (7), $\alpha = 4.09$, and $V_y = 4.968 \times 10^3$ 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: $V_4 = 3.188 \times 10^3$ kN, $V_3 = 4.169 \times 10^3$ kN, $V_2 = 4.716 \times 10^3$ kN, and $V_1 = 4.968 \times 10^3$ kN
- Plate thicknesses provided based on Eq. (8), 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 Eq. (9), $M_{pc} = 2.483 \times 10^3$ kN m, and $P_c = 15.80 \times 10^3$ kN
- Using the P–M interaction, the required capacities of the column are calculated as $M_u = 7.852 \times 10^3$ kN m and $P_u = 21.60 \times 10^3$ kN.

The nonlinear pushover analysis gives a yield displacement of 0.109 m. The nonlinear response-history analysis subjected to the SYL record gives a peak roof displacement of 0.349 m. The achieved ductility ($\mu_a$) is calculated as the ratio of peak roof displacement to the roof displacement at yield, and turns out to be 3.202 for this design case. 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 a percentage of $\mu_t$ is found to be 39.33%, whereas the mean difference is −13.6%. 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 Fig. 4. This shows that the design procedure is very effective in distributing drift almost uniformly over the height of the building for these three design cases. However, the profile may not always be as uniform as these ones; for example, see the profile for Design VIII (Fig. 5). The results presented in Table 2 are based on an assumed beam dimension: AISC section W14 × 145 [18]. The next part discusses how the effectiveness of a design is affected by a proper selection of the beam dimension.

**Table 3**
Result summary for redesigned SPSW systems with changed beam dimensions (aspect ratio 1:1).

| Design | Record | $\mu_t$ | Original beam section | New beam section | $\mu_a$ | % difference | Old $T$ (s) | New $T$ (s) |
|---|---|---|---|---|---|---|---|---|
| I-R | SYL | 2 | W14 × 145 | W14 × 109 | 2.05 | +2.50 | 0.69 | 0.70 |
| II-R | SYL | 3 | | W14 × 132 | 3.05 | −1.67 | 0.77 | 0.79 |
| III-R | SYL | 4 | | W14 × 53 | 3.55 | −11.3 | 0.92 | 0.96 |
| V-R | KJM | 3 | | W14 × 132 | 3.02 | +0.670 | 0.73 | 0.74 |
| Average | | | | | | −1.60 | | |

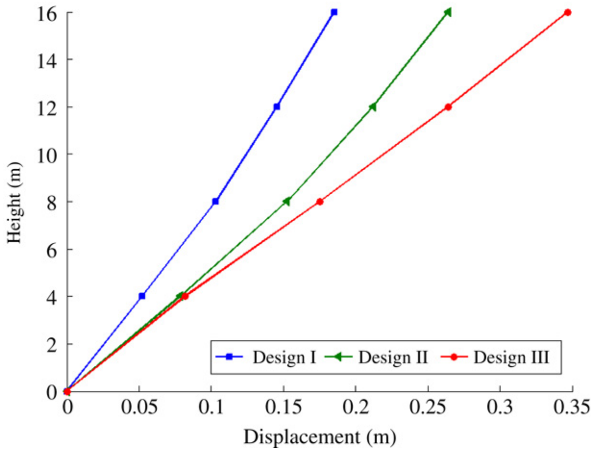


**Fig. 4.** Displacement profiles at peak roof displacement for Designs I, II and III.

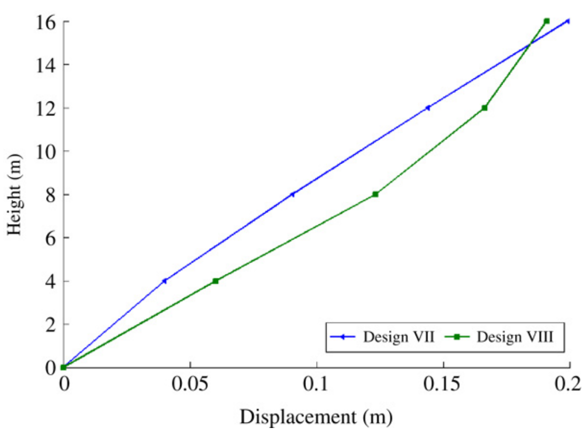


**Fig. 5.** Displacement profiles at peak roof displacement for Designs VII and VIII.

**Table 4**
Comparison of results for three different shear distributions for Design II (without beam tuning).

| | Steel MRF based [9] | Steel EBF based [10] | IBC 2006 based [14] |
|---|---|---|---|
| Design $V_y$ (kN) | 7695 | 8144 | 8358 |
| $t_4$ (mm) | 10.9 | 11.5 | 11.7 |
| $t_3$ (mm) | 10.3 | 10.6 | 10.7 |
| $t_2$ (mm) | 9.09 | 8.74 | 8.59 |
| $t_1$ (mm) | 6.88 | 5.74 | 5.16 |
| $M_u$ (kN m) | $12.30 \times 10^3$ | $12.33 \times 10^3$ | $12.35 \times 10^3$ |
| $P_u$ (kN) | $33.15 \times 10^3$ | $33.23 \times 10^3$ | $33.28 \times 10^3$ |
| $T$ (s) | 0.79 | 0.74 | 0.74 |
| Yield roof displ. (m) | 0.103 | 0.107 | 0.105 |
| Max. roof displ. (m) | 0.297 | 0.296 | 0.296 |
| $\mu_a$ | 2.87 | 2.78 | 2.83 |
| % difference | −4.29 | −7.36 | −5.72 |

### 3.1. Effect of the beam dimension

The design procedure as discussed in Section 2 does not include a design of the beam section. Although these pin-connected beams do not carry any significant moment due lateral loads, they influence the behaviour of the SPSW by changing the inclination of the principal tensile direction, as evident from Eq. (12). In reality, the beams may carry some moment due to the lateral forces applied by the strips. The dynamic analyses show that these moments are negligible except for the roof beam, where the strips are connected to only one side of the beam. Bruneau and Bhagwagar [21] discussed the importance of having a minimum stiffness for the beam member in order to develop an effective tension field distribution. The beams selected for the designs considered in this paper do not show any significant flexural deformation in both the static and dynamic analyses.

Four out of the eight original designs (with beam dimension W14 × 145) are further refined by changing the beam section, and the updated results are provided in Table 3. The selected beam dimensions satisfy the compactness criterion [18]. Table 3 very clearly shows that the effectiveness of the designs (in terms of achieving the target ductility ratio) can be improved by tuning the beam dimension. The absolute maximum difference is reduced to 11.3% and the mean difference is only −1.60% for this set. This tuning is an iterative procedure, which involves updating the analytical model for $\alpha_t$ and the area of each strip. The general guideline based on this experience is to increase the beam dimension if $\mu_a$ is more than $\mu_t$ and vice versa. The tuning of beams also changes the fundamental period ($T$) of the SPSW system, as shown in Table 3. Since the beams of the redesigned systems are lighter than the original ones, the fundamental period is slightly increased in the new designs.

### 3.2. Effect of the story-wise shear distribution adopted

Because of the lack of statistical data on inelastic shear distributions for standard SPSW designs under varying earthquake scenarios, it is recommended earlier in this work that distributions recommended for steel MRF [9], EBF [10], or any other standard distribution can be adopted in the design procedure. A detailed study on this aspect using the two distributions mentioned above and the distribution recommended by IBC 2006 [14] is performed for various design cases considered. This study shows that the EBF-based and IBC distributions do not alter the results significantly from those obtained using the steel MRF-based distribution. For example, the case of Design II (selected record: SYL, selected target ductility ratio $\mu_t = 3$) is illustrated here for these three distributions in Table 4. The ductility ratios achieved ($\mu_a$) before tuning of the beams are 2.87, 2.78, and 2.83 for the MRF-based, EBF-based and IBC 2006 distributions, respectively.

### 3.3. Effect of using real column sections

The design case studies presented in this paper use hypothetical column sections in order to check the effectiveness of the proposed procedure. These hypothetical sections follow the P–M interaction as per the AISC-LRFD code, as mentioned earlier. It is observed that using standard column sections in place of these hypothetical ones has a very minor effect on the results. Depending on the nearest available column dimension, the results may improve or

**Table 5**
Comparison of results for using hypothetical and real column sections for Design III.

| | Hypothetical column section | AISC W33 × 354 |
|---|---|---|
| $M_u$ (kN m) | $7.851 \times 10^3$ | $8.032 \times 10^3$ |
| $P_u$ (kN) | $21.60 \times 10^3$ | $23.10 \times 10^3$ |
| Beam section | W14 × 53 | W14 × 53 |
| $T$ (s) | 0.96 | 0.95 |
| Yield roof displ. (m) | 0.099 | 0.101 |
| Max. roof displ. (m) | 0.352 | 0.358 |
| $\mu_a$ | 3.55 | 3.54 |
| % difference | −11.1 | −11.4 |

worsen slightly. For example, Table 5 shows the change in various parameters while replacing the hypothetical columns with real ones for Design III (selected record: SYL, selected target ductility ratio $\mu_t = 4$). For this case, the achieved ductility ratio changes from 3.55 to 3.54. Both designs use a tuned beam section of W14 × 53.

## 4. Comparison with the SPSW design as per AISC recommendations

The AISC Seismic Provisions [4] is one of the few standards which provide design specifications for SPSW structures. The specifications provided by AISC for SPSW structures follow the AISC-LRFD design philosophy in general. In order to compare the displacement-based design method proposed herein with an existing standard method, a sample design case (Design III) is repeated following the AISC specifications. However, it should be noted that AISC recommends design of SPSW systems with rigid beam-to-column connections only. So, the comparison is between a SPSW system with pin-connected beams (proposed method) and a SPSW system with rigid beam-to-column connections under the same ground hazard scenario. Similar to Design III presented earlier in this paper, the AISC design considers the response spectrum (with 5% damping) for SYL as the design spectrum and a response reduction factor $R = 4$.

The seismic force calculations for the AISC design are based on ASCE/SEI 7 guidelines [22]. A multi-strip idealization of steel plates is used for both static and dynamic analyses. Highlights of the design calculation are provided here:

- Fundamental time period ($T$) assumed $= 0.54$ s (ASCE/SEI recommendation: $T = 0.59$ s)
- From the SYL spectrum, $A/g = 1.18$
- As per 12.8-2 of [4], $C_s = 0.295$
- Design base shear, $V = 5653$ kN
- Design equivalent lateral forces, from top to bottom: $F_4 = 2389$ kN, $F_3 = 1643$ kN, $F_2 = 1086$ kN, and $F_1 = 535.7$ kN
- Column section provided: W44 × 503
- Beam section provided: W14 × 730
- Plate thicknesses provided, from top to bottom: $t_4 = 1.25$ mm, $t_3 = 4.75$ mm, $t_2 = 7.00$ mm, and $t_1 = 7.00$ mm
- Based on static analysis: (a) for columns, maximum bending moment = 7183 kN m, maximum shear force = 2839 kN, and maximum axial force = 13 010 kN; (b) for beams, maximum bending moment = 4028 kN m, maximum shear force = 3100 kN, and maximum axial force = 0 kN; (c) maximum plate shear for fourth story = 408.5 kN, third story = 1774 kN, second story = 2665 kN, and first story = 2648 kN
- The column and beam sections satisfy compactness, minimum moment of inertia and lateral torsional buckling criteria as per AISC-LRFD specifications [18]
- The column section satisfies design criterion for combined compression and flexure as per Chapter H of AISC-LRFD specifications

**Table 6**
Result summary for designs of the SPSW system with plate aspect ratio 1:1.5.

| Design | Record | $\mu_t$ | $\mu_a$ | % difference |
|---|---|---|---|---|
| IX | SYL | 2 | 2.01 | +0.500 |
| X | SYL | 3 | 2.99 | −0.333 |
| XI | SYL | 4 | 3.75 | −6.25 |
| XII | KJM | 2 | 1.98 | −1.00 |
| XIII | KJM | 3 | 2.77 | −7.67 |
| XIV | TAZ | 2 | 2.07 | +3.50 |
| XV | TAZ | 3 | 2.39 | −20.3 |
| XVI | TAZ | 4 | 3.84 | −4.00 |
| Average | | | | −4.45 |

- The beam section satisfies the design criterion for flexure as per Chapter F of AISC-LRFD specifications (it also satisfies the strong column–weak beam requirement)
- Design shear strength ($\phi_v V_n$, for $\phi_v = 0.9$) of the plates as per Eq. (17-1) of AISC Seismic Provisions [4]: for fourth story = 477.3 kN, third story = 1814 kN, second story = 2673 kN, and first story = 2673 kN
- Fundamental period $T$ from eigenvalue analysis = 0.59 s
- From pushover analysis, yield roof displacement = 0.098 m (based on a bilinear approximation)
- From nonlinear response-history analysis using the SYL acceleration record, the maximum roof displacement = 0.123 m
- Achieved ductility ratio for this design, $\mu_a = 1.25$.

The ductility ratio achieved in the AISC-recommended design procedure is very different from the target ductility ratio of 4. Based on this case study, the proposed design procedure is much more effective (with $\mu_a = 3.55$) than the AISC procedure. Note that the AISC specifications do not explicitly include a target displacement in their formulation. The inclusion of a target displacement in the inelastic work-based formulation considered in the proposed design method makes this method more suitable in achieving a certain inelastic displacement for a given earthquake scenario. A similar design method can also be developed for a SPSW with rigid beam–column connections, if the work done in the beam plastic hinges is also included in the formulation. The proposed method also provides the advantage of selecting a failure mechanism of preference and thus structural damage is limited only to specific locations as per the designer's choice. The AISC or similar existing methods do not have this advantage and these methods rely on strong column–weak beam type checks to avoid undesirable failure mechanisms.

## 5. Design for SPSW systems with various steel panel aspect ratios

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 (Fig. 1) to 1.5 times and 2 times the original. The four-story structure remains the same otherwise. The new designs (eight 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 6 and 7 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 similarly to Table 2. Tables 6 and 7 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, the absolute maximum difference between the achieved and target ductility ratios, measured as a percentage of $\mu_t$, is found to be 20.3%, whereas the mean difference is −4.45%. The maximum and the mean difference values for the designs with aspect ratio 1:2 are 17.5% and −4.52%, respectively. These results altogether illustrate clearly that we can achieve designs sufficiently close to the target for the range of aspect ratios from 1:1 to 1:2.

**Table 7**
Result summary for designs of the SPSW system with plate aspect ratio 1:2.

| Design | Record | $\mu_t$ | $\mu_a$ | % difference |
|---|---|---|---|---|
| XVII | SYL | 2 | 1.91 | −4.50 |
| XVIII | SYL | 3 | 3.02 | +0.667 |
| XIX | SYL | 4 | 3.76 | −6.00 |
| XX | KJM | 2 | 1.97 | −1.50 |
| XXI | KJM | 3 | 3.13 | +4.33 |
| XXII | KJM | 4 | 3.30 | −17.5 |
| XXIII | TAZ | 2 | 2.08 | +4.00 |
| XXIV | TAZ | 3 | 2.53 | −15.7 |
| Average | | | | −4.52 |

## 6. 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 four-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 section) 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 a target inelastic drift and a 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.

The proposed displacement-based design procedure is validated against specific earthquake records. However, since the method is found to work well for designs against specific earthquake records, it should be easily extended to designs using a code-defined design spectrum. The method needs to be validated for a larger set of strong motion records with different characteristics. The proposed procedure also 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 and increased gravity loads at lower floors, the P–$\Delta$ 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 a selected yield mechanism can be developed for SPSWs with rigid beam-to-column connections as well. An advanced modeling technique for analyzing the designed SPSW systems, such as a three-dimensional finite element model where the out-of-plane buckling is modelled explicitly, would be a better option to check the effectiveness of the design procedure.

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