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DISPLACEMENT-BASED SEISMIC DESIGN OF STEEL PLATE SHEAR WALL SYSTEMS WITH RIGID-CONNECTED BEAMS

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ABSTRACT

The steel plate shear wall (SPSW) system is an innovative and effective lateral load resisting system consisting of thin steel panel added as an infill to the building structural frame. Various properties of thin unstiffened steel plate shear wall system (SPSW) like significant post-buckling strength, enhanced elastic stiffness, stable hysteresis characteristics, large plastic energy absorption capacity and substantial ductility make it a robust and attractive alternative to conventional lateral load resisting systems. Current seismic design codes for SPSW systems are still not based on proper inelastic considerations. As a result, its significant inelastic displacement capacity can not be utilized in design. A shift towards the performance-based seismic design (PBSD) method is needed where the inelastic behaviour of the SPSW can be incorporated in an efficient way. In this paper, a PBSD methodology is proposed for SPSW systems with rigid beam-to-column connections. The proposed method aims at designing a SPSW system for a specific target ductility ratio under a given earthquake scenario, and for a pre-selected yield mechanism. The proposed performance based methodology is analytically validated through designs of four-story test buildings for different target ductility ratios and subjected to various earthquake scenarios. Almost for all design cases the achieved ductility is found to be close to the target ductility. In addition, the actual plastic hinge locations are compared with the selected yield mechanism. These results show that for the selected earthquake scenarios, the design procedure yielded good results, although with varying levels of accuracy.

Introduction

The thin unstiffened steel plate shear walls (SPSW) are now gaining wide acceptance as primary lateral load resisting systems in earthquake resistant design of buildings. In a typical form, the steel plate shear wall assembly consists of thin steel infill plates added as an infill to the building structural frame composed of beams and columns. The steel plate panels are either stiffened or unstiffened as per the design methodology and are bolted or welded to boundary elements. The connection between beams (HBE) and columns (VBE) can range from simple to

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moment resisting. Initially, thick and heavily stiffened SPSWs were used in order to resist the shear within their elastic buckling limit. With the analytical and experimental research work carried in Canadian, US and UK universities, it was observed that the post-buckling strength of unstiffened SPSW is much more effective than heavily stiffened SPSW in resisting seismic forces. A list of important works is available in Berman et al. (2005). The main advantages of the thin unstiffened SPSW are high initial stiffness, substantial ductility, stable hysteretic characteristics and a large capacity for plastic energy absorption. Moreover, this system offers a light weight structure, increased floor area, faster speed of construction, considerable economy and better quality control when compared to a conventional reinforced concrete shear wall system.

Current seismic design codes for these systems such as the CAN/CSA-16 (CSA 2001), the AISC Seismic Provisions (2005a) or the AISC Design Guide 20 (Sabelli and Bruneau 2007) for steel plate shear walls are still not based on proper inelastic design methodology. As a result, the significant inelastic displacement capacity of these systems could not be explicitly accounted for in design. The performance-based seismic design (PBSD) methodology is a more general, reliable, and efficient method and it explicitly considers the inelastic behaviour of a lateral load resisting system. Thus a shift towards this methodology for SPSW system is very much needed. Considering this, Ghosh et al. (2009) had recently proposed a displacement/ductility-based design methodology of steel plate shear wall systems with pin-connected boundary beams. The method proposed by Ghosh et al. (2009) considers the target displacement ductility ratio (μ_t) as the design criterion. Thus it can utilize the ductility capacity of SPSW systems efficiently.

The existing design guideline of CAN/CSA-16 (CSA 2001) and AISC Design Guide 20 (Sabelli and Bruneau 2007) for steel plate shear walls suggest the use of SPSW with rigid beam-to-column connections. In this paper, a performance-based design method is proposed for SPSW with rigid beam-to-column connections based on a target displacement ductility ratio. The proposed method, similar to method proposed by Ghosh et al. (2009) for SPSW with pin-connected boundary beams, aims at designing for a specific inelastic drift/displacement ductility subjected to a specific earthquake scenario and for a pre-selected yield mechanism.

Fundamentals of the proposed design framework are described in the next section. The following section deals with an analytical validation of the proposed method through the designs of a single bay SPSW system in a four-story test building frame for different target ductility ratios and subjected to various earthquake scenarios. The effectiveness of the proposed method is measured in terms of how close the achieved inelastic displacement is to the target. Last two sections provide detail discussions on the results of these case studies and concluding remarks on the research presented in this paper.

Proposed Design Methodology

The new design method is based on concepts similar to those used by Ghosh et al., (2009) which considers the inelastic energy demand on a structural system and this energy is equated with the inelastic work done in plastic hinges for monotonic loading up to the target drift. This formulation, with various modifications, was used earlier for the design of steel moment frames by Leelathaviwat et al. (1998), Lee and Goel (2001), and for steel braced frames by Chao et al. (2007).

The proposed performance-based design formulation is for a target drift and pre-selected yield mechanism. Fig. 1(a) shows a typical SPSW with rigid beam-to-column connections. The selected or desired unidirectional yield mechanism is shown in Fig. 1(b).

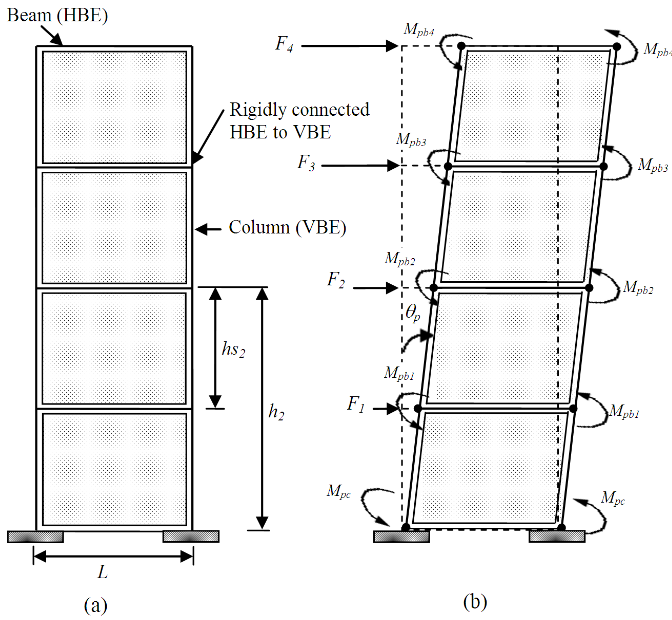

Figure 1. (a) Schematic of SPSW system with rigid beam-to-column connections; (b) Selected unidirectional yield mechanism.

The following assumptions are made for formulating the design procedure:

i. The assumed yield mechanism is composed of yielding of steel plates and formation of plastic hinges at column bases and at both ends of floor beams as shown in Fig. 1(b)
ii. The formation of plastic hinges and yielding of plates in the assumed yield mechanism occurs simultaneously
iii. The assumed yield mechanism follows a uniform, unidirectional inter-story drift along the height
iv. The energy dissipated in this assumed unidirectional yield mechanism up to the peak roof displacement is equal to the monotonic plastic energy demand on the system
v. The effect of rigid floor diaphragm action is considered, which results in zero axial force in all HBE

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

and ductility reduction factor (R):

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

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

$$C_e = \frac{A}{g} = \frac{S_v T}{2\pi g} = \frac{V_e}{W} \tag{3}$$

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 as shown in Fig. 1(b). 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 Eqn. (1), the plastic energy demand (E_p) is obtained as:

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

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

$$\left(\sum_{i=1}^{n} F_i h_i\right)\theta_p = 2M_{pc}\theta_p + 2\sum_{i=1}^{n} M_{pbi}\theta_p + \sum_{i=1}^{n} P_i h_{si}\theta_p \tag{5}$$

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, M_{pbi} = plastic moment capacity of the i^{th} story beam, θ_p = target plastic drift based on an assumed yield drift (θ_y) as shown in Fig. 1(b). 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 = height of i^{th} floor. The factor λ_i (= F_i/V_y) represents the shear force distribution in the SPSW system. Since there are no specific recommendations for inelastic shear distribution for standard SPSW designs one can use shear distribution proposed for steel MRF by Lee and Goel (2001) or for steel EBF by Chao and Goel (2005) which are based on the inelastic state of structures. The required plate thickness (t_i) at each story is obtained using the following equation proposed by Ghosh et al., (2009):

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

where V_i = i^{th} story shear demand, F_y = material yield strength and L = bay width.

The primary selection for the column section is based on the following stiffness requirement as mentioned in CAN/CSA-16 (CSA 2001) and the AISC Seismic Provisions (2005a). This requirement ensures that there will be no premature buckling of a column under the pulling action of the plate.

$$I_c \geq \frac{0.00307 t h_s^{\,4}}{L} \tag{8}$$

where I_c is the moment of inertia of column perpendicular to plane of web. 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 the available steel tables as per AISC (2005b) for these demands based on the code prescribed P-M interaction and compact section criterion.

The assumption of rigid floor diaphragm action results in zero axial force in all floor beams. The floor beams are designed to anchor properly the panel tension field. Here, the selection of the beam section is an iterative procedure which involves the assumption of the angle of tension field (α_t) of steel plate and the calculation of the beam cross sectional area (A_b) using the following formulation (Timler and Kulak 1988):

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

Analytical Validation of the Proposed Design Method

A four-story steel frame building having a single SPSW bay as the lateral load resisting system is considered here for the analytical validation of the proposed design method. Except for the SPSW bay, all beam-to-column connections are pin-connected as shown in Fig. 2. Initially, the SPSW bay with a span equal to the story height is considered. This span is later doubled in order to test the proposed method for SPSW with aspect ratio (h_s:L = 1:2). 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. Two 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 designed SPSW systems are checked against the same records through nonlinear response-history analyses (NLRHA) to measure the effectiveness of the proposed design procedure in terms of achieved ductility ratio (μ_a).

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. The column and beam sections are selected from available steel tables as per AISC (2005b).

As mentioned earlier, the selection of beam section is an iterative procedure which depends on the angle of tension field α_t of steel panel. α_t for steel panels in all stories is kept constant, which results in almost the same beam cross section for all floors. In order to achieve ductility closer to the target, α_t is varied between 35° to 45° and thus the beam sections are tuned. A design flow chart is provided in Fig. 3 giving the individual design steps.

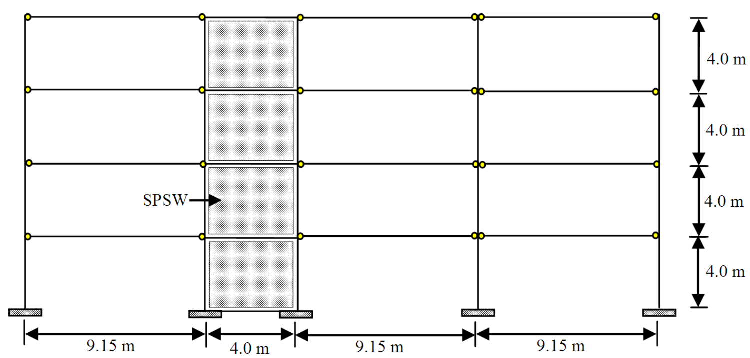

Figure 2. Configuration of four-story steel frame test building with SPSW.

Table 1. Details of earthquake records used for design.

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

The SPSW system is modeled using the multi-strip idealization (Thorburn et al. 1983) and the structural analysis program DRAIN-2DX (Prakash et al. 1993) is used for nonlinear static and response-history analyses. The nonlinear truss and beam-column elements are used for strips and boundary elements, respectively. For all the elements, the material is assumed to be elastic-perfectly plastic steel with yield stress F_y = 344.74 MPa and without any overstrength factor. The IBC 2006 (ICC 2006) recommended lateral force distribution is used for nonlinear static pushover analyses and the roof displacement versus base shear plot is bilinearized in order to obtain the yield point. The system is centreline modeled 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.

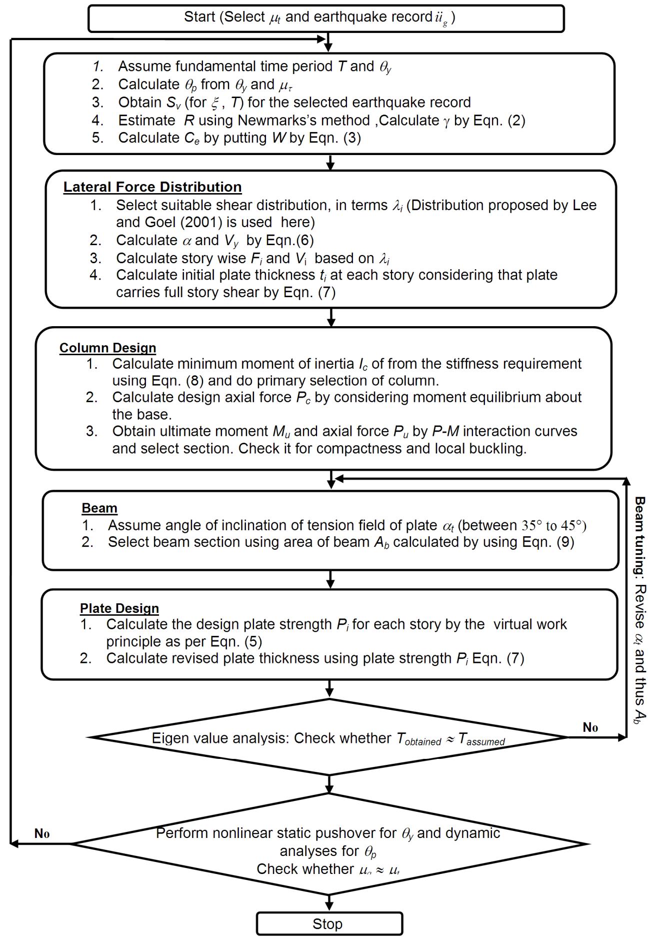

Figure 3. Flowchart for the proposed design method.

Results and Discussion

Tables 2 and 3 present the results summary for all the designs of SPSW with steel panel aspect ratios of 1:1 and 1:2, respectively. These tables present the percentage difference between μ_t and μ_a, with respect to μ_t, for each design. The mean percentage difference along with its absolute maximum value (AbsMax) is also presented for all the designs with a typical aspect ratio. This mean is primarily used as a measure of the effectiveness of the proposed design procedure. The mean values of percentage differences are –1.67% and –1.10% for steel panel aspect ratios 1:1 and 1:2, respectively, and the corresponding values of absolute maximum (AbsMax) differences are 7.00% and 4.50%. These results show that the proposed method works well for different ground records, different target ductility ratios (μ_t), and for various plate aspect ratios. 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. Fig. 4 presents displacement profiles at the instant of peak roof drift for all the Northridge designs, which show that the structure follows assumed plastic mechanism with nearly uniform inter-story drift along the height.

Table 2. Result summary of designs for steel panel aspect ratio 1:1.

Design	Record	μ_t	VBE	HBE	Panel Size (mm)				μ_a	% Diff.
					t_1	t_2	t_3	t_4		
I	SYL	2	W 36X529	W 12X152	11.42	10.80	9.46	7.09	1.92	–4.00
II	SYL	3	W 36X487	W 12X53	9.85	9.33	8.21	6.20	3.12	4.00
III	SYL	4	W 40X327	W 12X65	6.37	6.04	5.33	4.07	3.94	–1.50
IV	KJM	2	W 36X652	W 12X210	14.04	13.27	11.62	8.69	1.95	–2.50
V	KJM	3	W 36X529	W 12X210	10.30	9.75	8.56	6.44	3.03	1.00
VI	KJM	4	W 40X397	W 12X30	7.48	7.09	6.25	4.75	3.72	–7.00
Average										–1.67
AbsMax										7.00

Table3. Result summary of designs for steel panel aspect ratio 1:2.

Design	Record	μ_t	VBE	HBE	Panel Size (mm)				μ_a	% Diff.
					t_1	t_2	t_3	t_4		
VII	SYL	2	W 36X330	W 14X90	6.73	6.37	5.30	4.18	2.10	5.00
VIII	SYL	3	W 36X232	W 12X210	4.00	3.78	3.33	2.51	3.03	1.00
IX	SYL	4	W 40X183	W 12X58	3.31	3.14	2.77	2.10	3.99	–0.250
X	KJM	2	W 36X361	W 14X233	7.17	6.77	5.92	4.41	1.93	–3.50
XI	KJM	3	W 40X278	W 12X230	5.09	4.82	4.23	3.18	2.87	–4.33
XII	KJM	4	W 40X211	W 14X109	7.48	7.09	6.25	4.75	3.82	–4.50
Average										–1.10
AbsMax										4.50

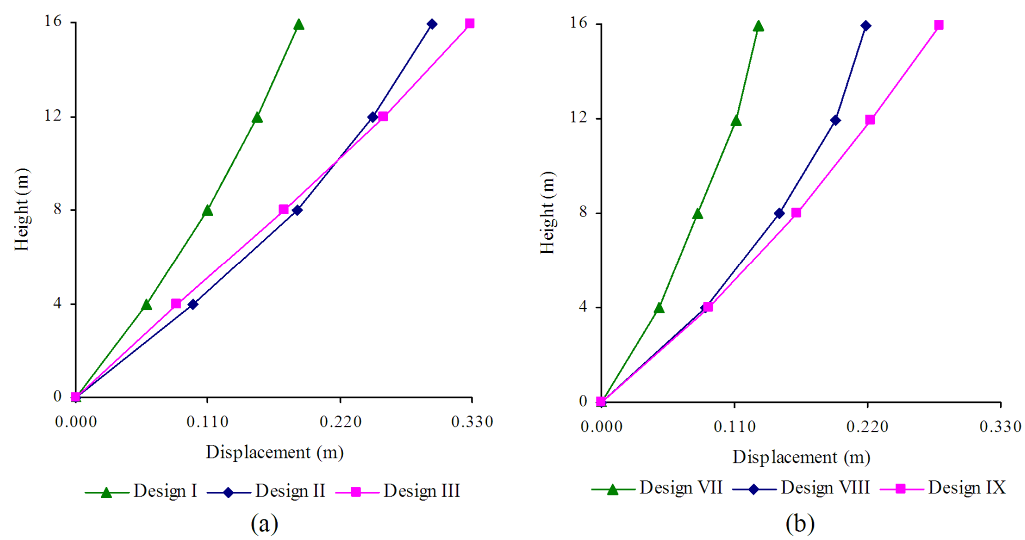

Figure 4. Displacement profiles at peak roof displacement.

The base shear versus roof displacement pushover plots with yielding hierarchy for two sample designs (Designs III and XI) are presented in Fig. 5 in order to check the locations and sequence of yielding.

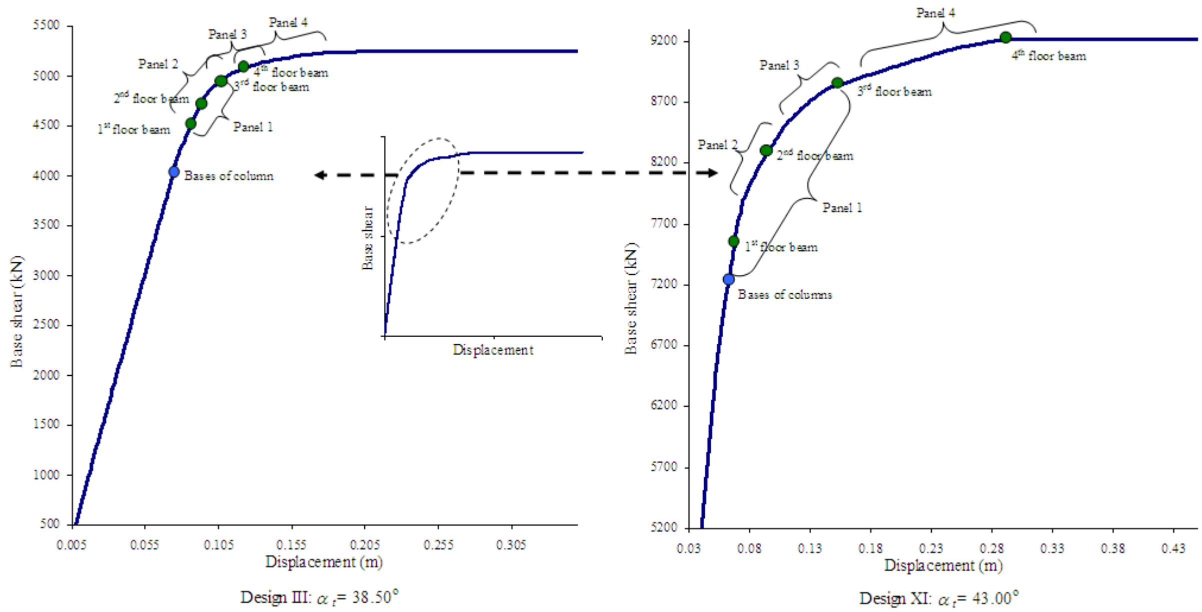

Figure 2. Pushover plots with yielding hierarchy for Designs III and XI.

For Design III, the yielding of panels is delayed much after the plastic hinge formation at the column bases when compared to Design XI. Also, the plastic hinge formation at the ends of floor beams in case of Design III is more closely spaced along the pushover curve as compared to that for Design XI where it is more spread out. The justification for this is that a large value of α_t (above 40°) results in heavier beam sections and comparatively thinner panels (for example, Design XI), when compared to smaller values of α_t (for example, Design III) which gives lighter beam sections and moderately thin panels. As a result, for designs with thinner panels and heavier beams, the yielding of ground story panel starts just after the formation of plastic hinges at column bases, which is then followed by the yielding of upper story panels and hinging in corresponding floor beams, and vice versa.

Concluding Remarks

A performance-based design method with displacement ductility ratio (μ_t) as the design criteria for SPSW with rigid beam-to-column connections is proposed in this paper. The method is applied to the design of a four-story steel frame building having a single bay of SPSW as the lateral load resisting system. The proposed method is tested for two different SPSW aspect ratios. The analytical test results show that the proposed design procedure is very effective in achieving the target ductility ratios as well as following the pre-selected yield mechanism. The main advantage of proposed procedure is that it provides a simplistic solution for obtaining a design of SPSW system based on target inelastic drift with selected yield mechanism. It dose not require any complicated analysis from designer/practising engineer's part. The design procedure remains simple while satisfying an advanced performance based design.

As the proposed performance-based design procedure is validated against specific earthquake records, it should be easily extended to designs using a code-defined spectrum in order to have a more general design format. The proposed design method also needs to be validated for high-rise SPSW system with due consideration to the P-Δ effect. Also, the analytical validation through detailed three-dimensional finite element modelling of SPSW designs, where geometric nonlinearity is accounted for explicitly, would be a better option to check the effectiveness of the proposed design method.

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