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DESIGN OF STEEL PLATE SHEAR WALLS CONSIDERING BOUNDARY FRAME MOMENT RESISTING ACTION

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ABSTRACT

Conventional design of steel plate shear walls (SPSWs) assumes that 100% of the story shear is resisted by each infill panel. Following this approach, strength provided by the boundary frame moment resisting action, which provides the SPSW with an overstrength, is neglected. While this design assumption has a positive impact on seismic performance of SPSWs, no analytical work has been done to quantify the magnitude of this overstrength in general terms. Such preliminary work is conducted in this paper. Based on plastic analysis of SPSWs, this paper investigates the relative and respective contributions of boundary frame moment resisting action and infill panel tension field action to the overall plastic strength of SPSWs, followed by a procedure to make use of the strength provided by the boundary frame moment resisting action. Procedures for design of SPSWs having weak infill panels are also presented in this paper. Then, results from a series of time history analyses using validated models are presented to compare the seismic performances of SPSWs designed using different design assumptions.

Introduction

A typical steel plate shear wall (SPSW) consists of infill steel panels surrounded by columns, called Vertical Boundary Elements (VBEs), on each side, and beams, called Horizontal Boundary Elements (HBEs), above and below. Previous tests and analytical studies on single-story and multistory SPSWs (Berman and Bruneau 2003, Berman and Bruneau 2005 to name a few) recognized that a SPSW's ultimate strength combines the contributions of both the moment resisting boundary frame and the infill panels. However, strength of the wall provided by the moment resisting action of boundary frame is not explicitly taken into account in the design of SPSWs by codes (AISC 2005 and CSA 2000), typically resulting in a conservative but possibly more expensive SPSW design.

To investigate the relative contribution of boundary frames to the overall strength of SPSWs and possibly achieve an optimum design of SPSWs accounting for that contribution, this

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paper reviews knowledge on the plastic strength of SPSWs, and summarizes some design assumptions in current codes. Then, design procedures considering boundary frame moment resisting actions are presented followed by a case study to compare the performances of SPSWs designed using various assumptions on the relative strength and design of boundary frames.

Plastic Strength of Steel Plate Shear Walls

Plastic collapse mechanisms for SPSWs subjected to lateral loads have been investigated by Berman and Bruneau (2003). From that work, equations for the ultimate strength of SPSWs have been shown to agree well with the results obtained from tests. For the desired SPSW plastic mechanism (i.e. the uniform collapse mechanism), by equating the internal and external work, Berman and Bruneau (2003) derived the following general equation for the overall plastic strength of a SPSW with moment resisting HBE-to-VBE connections:

$$\sum_{i=1}^{n_s} F_i h_i = \underbrace{\sum_{i=1}^{n_s}\left(M_{pl_i} + M_{pr_i}\right)}_{\text{Contribution of boundary frame}} + \underbrace{\sum_{i=1}^{n_s}\frac{1}{2}R_{yp} f_{yp} L h_i \left(t_{wi} - t_{wi+1}\right)\sin(2\alpha_i)}_{\text{Contribution of infill panels}} \qquad (1)$$

where F_i is the lateral force applied on the wall to develop the desired mechanism; h_i is the i^{th} story elevation; M_{pl_i} and M_{pr_i} are the expected plastic moments at the left and right ends of the i^{th} HBE respectively; t_{wi} is the thickness of the infill panel at the i^{th} story; R_{yp} is the ratio of expected to nominal yield strength of infill panels; f_{yp} is the nominal yield strength of infill panels; L is the SPSW bay width; n_s is the total number of stories; and α_i is the tension field inclination angle at the i^{th} story. As shown in Eq. 1, lateral strength of the wall combines the contribution of boundary frame and that of infill panels.

Current Design Requirements

Based on Eq. 1, one can obtain the following equation for calculating the shear strength of a single infill panel:

$$V_i = \frac{1}{2} R_{yp} f_{yp} L t_{wi} \sin(2\alpha_i) \qquad (2)$$

where V_i is the expected strength of the considered infill panel.

Dividing the infill panel strength determined from Eq. 2 by an overstrength factor, as defined by FEMA 369 (FEMA, 2001), and taken as 1.2 in this case (Berman and Bruneau, 2003), and also excluding the R_y factor used for calculating the expected plate strength, one can obtain the following infill panel nominal shear strength:

$$V_{ni} = 0.42 f_{yp} L t_{wi} \sin(2\alpha_i) \qquad (3)$$

where V_{ni} is the nominal strength of the considered infill panel.

Eq. 3 is implemented in the AISC Seismic Provisions and the CSA S16 Standard, and is used for sizing the thickness of infill panels of SPSWs. Thus, by neglecting the contribution from the boundary frame moment resisting action on the SPSW strength, and solving for t_{wi} from Eq. 3, one can obtain the following design equation to select thickness of the infill panel.

$$t_{wi} = \frac{V_{ni}}{0.42 f_{yp} L \sin(2\alpha_i)} \tag{4}$$

Then, the AISC Seismic Provisions and the CSA S16 Standard require that the boundary frame members be designed using capacity design procedures to ensure that the boundary frame members can anchor the infill panel yield forces developed by the infill panel thickness determined from Eq. 4. As such, following this approach, strength provided by the boundary frame moment resisting action provides the SPSW with an overstrength (which has a positive impact on seismic performance). At the time of this writing, no analytical work has been done to quantify the magnitude of this overstrength in general terms and to investigate how to make use of it to achieve an optimum design of SPSWs. Such work is presented in the following section.

Steel Plate Shear Wall Overstrength and Balanced Design

In order to best understand the SPSW overstrength resulting from the design approach inferred by current design codes, which assumes that all the lateral forces applied on the wall are resisted by the infill panel tension field actions alone, this section investigates the overstrength of SPSWs designed considering that various percentages of the lateral design forces are resisted by the infill panels. Both single-story and multistory SPSWs are considered.

Single-Story SPSWs

A single-story SPSW is first studied here because this simple case provides some of the building blocks necessary to understand the more complex scenario (i.e. multistory SPSWs) presented later. Consider the single-story SPSW shown in Fig. 1 and assume that its VBEs are pinned to the ground.

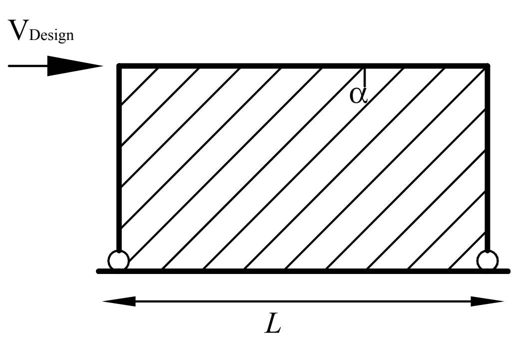

Fig. 1 Single-Story SPSW Example

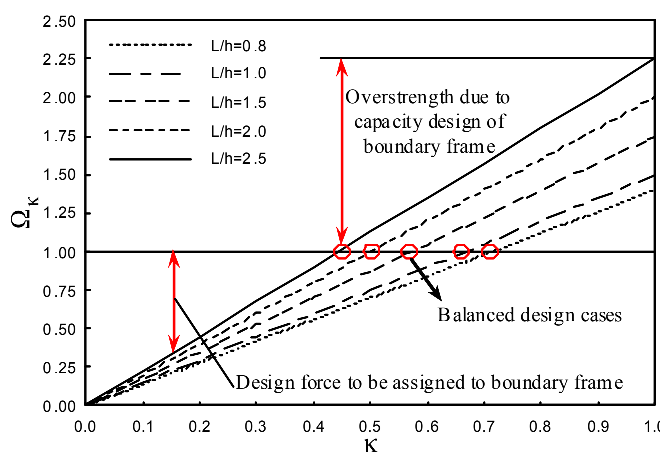

Fig. 2 The Relationship between Ω_K and κ (assuming α=45° and η=1.0)

Assuming that the percentage of the total lateral design force assigned to the infill panel is κ, the required infill panel thickness is determined by solving for t_w from the following equation:

$$\kappa V_{design} = \frac{1}{2} R_{yp} f_{yp} L t_w \sin(2\alpha) \quad (5)$$

where V_{design} is the lateral design force applied on the wall.

Considering the plastic strength of the wall, V_p, based on the procedure developed by Berman and Bruneau (2003), and also assuming the top beam is sized using the capacity design procedure given by Vian and Bruneau (2005), the ratio of V_p to V_{design}, which is denoted as Ω_κ and is used to describe the overstrength of the SPSWs designed using different values of κ, can be determined as

$$\Omega_\kappa = \kappa \left[1 + \frac{1}{2} \tan^{-1}(\alpha) \left(\frac{L}{h} \right) \cdot \frac{\eta}{1 + \sqrt{1 - \eta^2}} \right] \quad (6)$$

where η is the plastic section modulus reduction ratio accounting for the presence of reduced beam section (RBS) connections and other terms have been defined previously. Please note that detailed derivations of Eq. 6 are presented in Qu and Bruneau (2009)

Explicitly shown in Eq. 6, the factor, Ω_κ, depends on a series of variables including, κ, α, L/h, and η. The effects of those terms can be investigated based on Eq. 6. Here, a parametric study is conducted to discuss the impact of κ on Ω_κ for the given values of other terms. Note that, for simplicity, the inclination angle of the tension field action is assumed to be 45^o and η is assumed to be unity (i.e. no RBS connections are used in the HBEs). Note that results are not expected to vary substantially for other values of α. In addition, in the parametric study, the infill panel aspect ratio (i.e. L/h) was chosen to vary between 0.8 and 2.5, which are the limits allowed by the AISC Seismic Provisions (AISC 2005).

The corresponding results are illustrated in Fig. 2. As shown, a higher percentage of the lateral design forces assigned to the infill panel (i.e. greater value of κ) results in a greater overstrength of the wall (i.e. greater value of Ω_κ). Under the design assumption presented in the AISC Seismic Provisions and the CSA S16 Standard (i.e. when $\kappa = 1.0$), the wall has a significant overstrength varying from 1.4 to 2.25 over the code-compliant range of infill panel aspect ratios of $0.8 \le L/h \le 2.5$. Note that the example wall assumes that the VBEs are pinned to the ground. It is recognized that, for the SPSWs that with VBEs fixed to the ground, the overstrength would be even greater.

Also observed from Fig. 2, when κ is reduced to certain level, showed by the circles on that figure, the lateral force resisted by the boundary frame of the SPSW is exactly equal to that which will be required if that frame is designed to resist the infill panel yield forces per capacity design principles. Therefore, at that particular point, the boundary frame does not provide any overstrength for the system as the division of the lateral load resistance, and the overstrength (Ω_κ) is therefore equal to unity. Such a design case is termed "balanced" design case in this paper. For this case, the value of κ can be determined by setting the constraint $\Omega_\kappa = 1.0$ into Eq. 6 and solving for κ. The resulting value of κ for the balanced case, designated as $\kappa_{balanced}$, is therefore:

$$\kappa_{balanced} = \left[1 + \frac{1}{2}\tan^{-1}(\alpha)\left(\frac{L}{h}\right)\cdot\frac{\eta}{1+\sqrt{1-\eta^2}}\right]^{-1} \tag{7}$$

As shown in Fig. 2, when reducing κ to a value below $\kappa_{balanced}$, the SPSW plastic strength is not sufficient to resist the lateral design force. In other words, the boundary frame designed only to resist the infill panel yield forces, per capacity design principles, has to be strengthened to fill the gap between the available strength of the wall and the expected lateral design demand. Incidentally, detailed information about the design of such SPSWs will be presented later.

Figs. 3 and 4 respectively plot, based on (7), the relationships between $\kappa_{balanced}$ and η for various values of L/h, and $\kappa_{balanced}$ and L/h for various values of α. As shown in Fig. 3, the value of $\kappa_{balanced}$ increases when η reduces. This observation is reasonable because the reserved strength of the wall due to the moment resisting action of the boundary frame decreases when RBS connections are introduced in the HBE (i.e. when $\eta \le 1.0$), which means that, in this case, a higher percentage of lateral design force should be resisted by the infill panel tension field action. Note that, when η reduces to zero, which physically corresponds to simple HBE-to-VBE connections, $\kappa_{balanced}$ becomes unity, indicating that 100% of the lateral force is resisted by the infill panel.

Fig. 4 illustrates the trends in $\kappa_{balanced}$ for the code-compliant range of infill panel aspect ratios and the typical range of tension field inclination angles. As shown, the value of $\kappa_{balanced}$ decreases when the aspect ratio increases. This is also reasonable since a bigger HBE member has to be used to anchor the tension field action in a "squat" wall (which has a greater aspect ratio) in comparison with a slender wall (which has a smaller aspect ratio), resulting in a higher strength of the wall provided by the moment resisting action of the boundary frame.

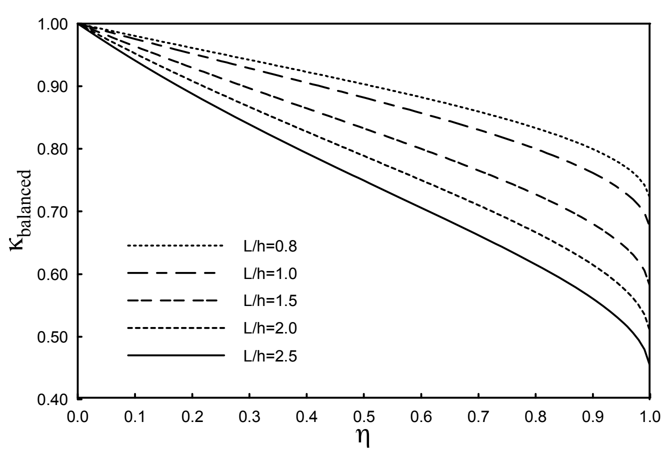

Fig. 3 The Relationship between $\kappa_{balanced}$ and η (assuming α=45º)

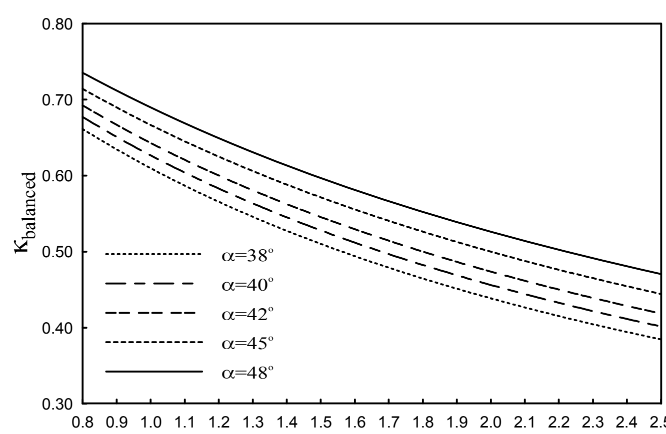

Fig. 4 The Relationship between Aspect Ratio and $\kappa_{balanced}$ (assuming η=1.0)

Multi-Story SPSWs

Following the same logic used for single-story SPSWs, one can obtain the following equation for $\kappa_{balanced}$ of multi-story SPSWs:

$$\kappa_{balanced\,i} = \left[1 + \frac{1}{2}\tan^{-1}(\alpha_i)\left(\frac{L}{h_i}\right)\cdot\frac{\eta_i}{1+\sqrt{1-\eta_i^2}}\right]^{-1} \quad (8)$$

To have a better understanding of the lateral forces respectively resisted by the infill panels and the boundary frame in the balanced design case for a multistory SPSW, consider a four-story SPSW without RBS connections as an example. Assume that the lateral design forces linearly distribute along the height of the wall as shown in Fig. 5 and the story heights and infill tension field inclination angles (45º) are constant in all stories. Fig. 6 illustrates the percentage of the story shear resisted by the infill panel at each level (i.e. to be considered to size the infill panels at each story). For comparison purpose, the results for the case when 100% of the story shear is resisted by each infill panel (i.e. the design case implied by the AISC Seismic Provisions) are also provided. As shown, the story shears assigned to the panels are reduced in the balanced design case. For example, when the panel has an aspect ratio of 1.5, 78% of the base shear is resisted by the first-story panel when the wall develops the desired plastic mechanism.

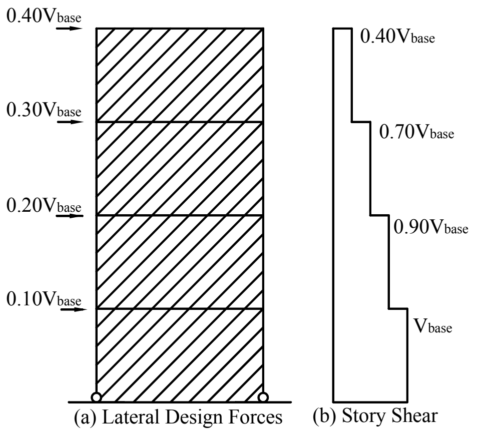

Fig. 5 Description of Example Four-Story SPSW

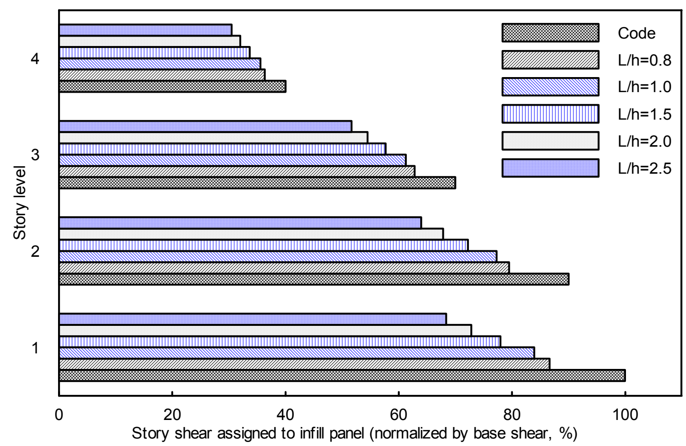

Fig. 6 Modified Design Forces of an Example Four-Story SPSW

Boundary Frame Design of SPSWs Having Weak Infill Panels

When the infill panel thickness is smaller than that corresponding to the balanced design case, the SPSW will not have a sufficient strength to resist the lateral design force if the boundary frame is only proportioned using capacity design procedures (i.e. designed only to resist the infill panel yield forces). Here, such walls will be termed SPSWs having weak infill panels.

Three HBE design equations, which lead to different designs but same global plastic strength of the boundary frame, are presented in Table 1. What conceptually differs in the three methods considered below is that each case assumes a different distribution of HBE strength along the height of the SPSW for which the global plastic strength is satisfied but the local story-by-story strength is not necessarily satisfied. Detailed derivations of the equations for Methods I to III are provided in Qu and Bruneau (2009). Note that these three methods are based on plastic analysis and the yielding sequence of members and infill panels of the SPSW under the lateral forces is out of the scope of the work presented here.

Table 1 Design Equations of HBEs in SPSWs with Weak Infill Panels

Design Methods	**Plastic section moduli of HBE cross-sections**
I	$Z_{bi} = \sum_{i=1}^{n_s}(1-\kappa_i)F_{Di}h_i \Big/ 2f_y\sum_{i=1}^{n_s}\eta_i$
II	$Z_{bi} = (1-\kappa_i)F_{Di}h_i / 2f_y\eta_i$
III	$Z_{bi} = \sum_{j=i}^{n_s}(1-\kappa_j)F_{Dj}h_{si} \Big/ 2f_y\eta_i$

Case Study

Prior sections presented different approaches for SPSW design. To be able to determine the relative merits of any of those designs which will provide the same overall lateral load resistance, it is important to conduct nonlinear time history analyses to compare the seismic performances of the walls respectively designed assuming various distributions of the lateral loads between the boundary frame and infill panels as well as various distributions of HBE strength that satisfy the total plastic strength of the structure. Such a study is conducted in this section. The following briefly describes assumptions, design results, and the performances of those differently designed SPSWs in the nonlinear time history analyses.

Assumption and Design Summary

An eight-story single-bay SPSW was used as the prototype structure in this study. The VBEs were assumed to be pinned to the ground and the first-story infill panel was assumed to be anchored to the ground rather than to an anchor HBE at that level. The bay width and constant story height were assumed to be 18 ft and 10 ft, respectively, resulting in an infill panel aspect ratio of 1.8, which is within the range allowed in the 2005 AISC Seismic Provisions. The structure was assumed to be located on class B soil in Northridge, CA. Its weight was assumed to be 5092.5 kips distributed as 652.5 kips at all levels except at the roof where it was 525 kips. Seismic design loads were calculated using FEMA 450 (FEMA, 2003) and the associated spectral acceleration maps. Design short and 1-second spectral ordinates, S_{DS} and S_{D1}, were respectively calculated to be 1.43 g and 0.50 g. The period of the structure was estimated (using the FEMA procedures) to be 0.54 second, and using a response modification factor, *R*, of 7, and an importance factor, *I*, of 1, the base shear was found to be 674.5 kips. The corresponding lateral forces up the height of the structure along with the infill panel thicknesses determined from different design procedures are presented in Table 2

The AISC design procedure, the balanced design procedure, and the design procedure assuming that 40% of the story shear is resisted by the infill panel at each story (i.e. the procedure for the SPSWs having weak infill panels) are considered to select the SPSW infill panel thicknesses. Note that it is assumed that the calculated infill panel thicknesses are available in all cases.

Table 2 Summary of Design Story Shears and Infill Panel Thicknesses

Story Level	Elevation (ft)	Lateral Force (kip)	Modified Story Shear (kip)			Infill panel thickness (in)		
			AISC	Balanced design	Weak infills	AISC	Balanced design	Weak infills
8	80	127.1	127.1	113.0	50.8	0.033	0.029	0.013
7	70	137.9	264.9	233.6	106.0	0.069	0.060	0.027
6	60	117.9	382.8	335.5	153.1	0.099	0.087	0.040
5	50	97.9	480.7	422.9	192.3	0.124	0.109	0.050
4	40	78.0	558.7	492.0	223.5	0.144	0.127	0.058
3	30	58.2	616.9	542.8	246.8	0.160	0.140	0.064
2	20	38.5	655.5	575.8	262.2	0.170	0.149	0.068
1	10	19.0	674.5	590.1	269.8	0.174	0.153	0.070

For the design using weak infill panels, all three HBE design methods were considered for design of the boundary frame. As a result, a total of five SPSW designs were obtained, i.e. one from the AISC design procedure, one from the balanced design procedure, and three from the procedure for SPSWs having weak infill panels (respectively using Methods I, II and III). Please note that detailed information of the boundary frame members from those five designs are listed in Qu and Bruneau (2009).

Analytical Model and Artificial Ground Motions

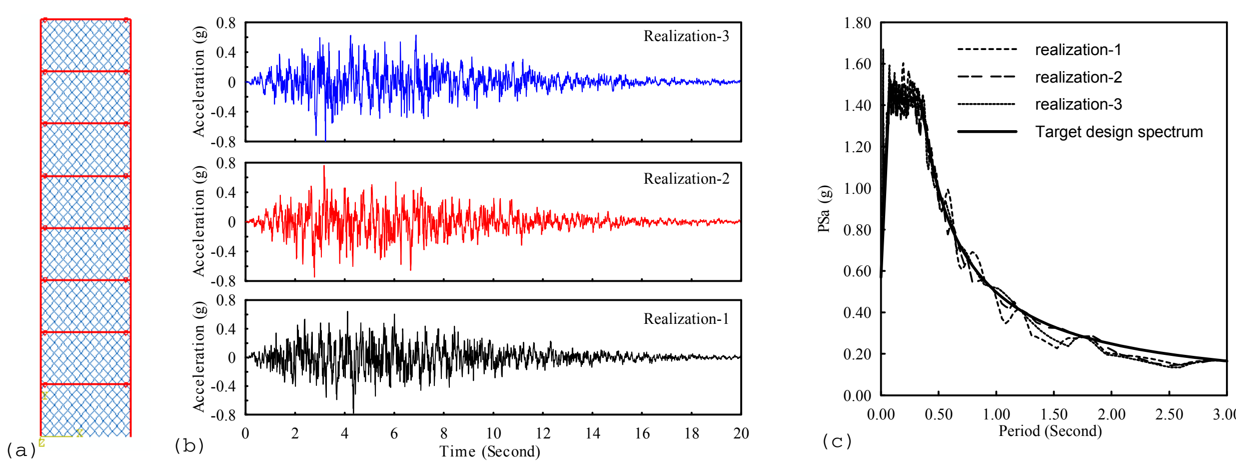

Fig. 7 Dual Strip Model and Ground Motion Information (a) Dual Strip Model (b) Ground Motion Histories (c) Pseudoacceleration Spectra

To quantify the seismic performance of those SPSWs designed using different procedures, nonlinear time history analyses were conducted on models constructed following the dual strip procedure described and validated against cyclic test results (Qu et al. 2008). Note that in these models the infill steel plates were represented by two series of inclined tension-only members (i.e. strips) to replicate the behavior of SPSWs under cyclic loads. Three realizations of the target design spectra compatible ground motions were obtained using the computer program, TARSCTHS, by Papageorgiou et al. (1999) and were used as excitations for the nonlinear time history analyses. The dual strip model, ground motion realizations and acceleration spectra are shown in Fig. 7.

Result Comparison

The maximum drift of each story was obtained from the nonlinear time history analyses to compare the seismic performances of the 5 different SPSWs. Fig. 8 presents the corresponding results along the height of the walls for each considered earthquake.

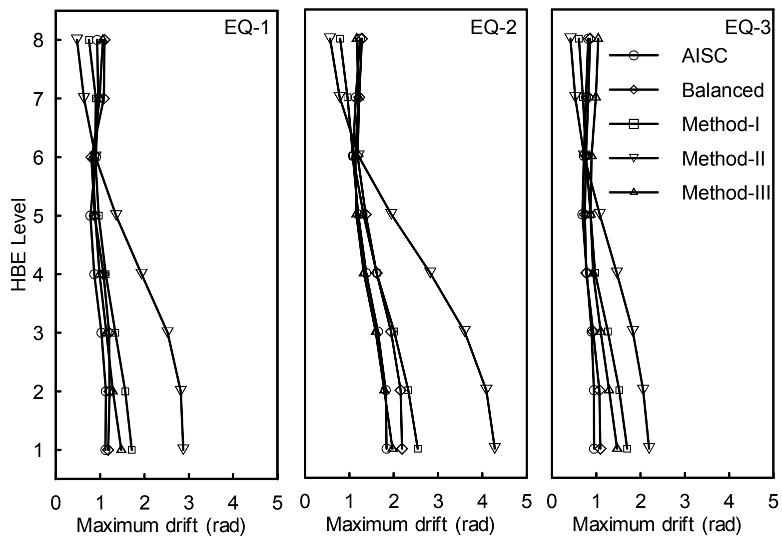

Fig. 8 Results from Time History Analyses

As shown, the wall from the balanced design exhibits similar performance to that of the wall designed using the AISC Seismic Provisions. For example, the average of the maximum first-story drifts of the wall designed using the balanced design procedure for the three earthquake realizations is 1.48%, which is close to the corresponding result of 1.30% from the wall designed using the AISC Seismic Provisions. For comparison purpose, the corresponding averages of the maximum first-story drifts the SPSWs having weak infill panels and designed using methods I, II and III are calculated to be 1.98%, 3.12% and 1.65%, respectively. It should also be mentioned that although the balanced design procedure could lead to a SPSW that exhibits satisfactory responses according to the limited analytical data provided here, such a procedure should not be used without further confirmation from experimental studies on the system performance of SPSWs.

Another observation from Fig. 8 is that the maximum story drifts are distributed in a uniform pattern along the heights of all SPSWs except for the wall having weak infill panels and designed using method II. As shown, significant deformations concentrated in the lower stories of that SPSW although smaller responses are observed in its upper stories.

Conclusions

This paper investigated the lateral load resistance of SPSWs respectively provided by the boundary frame moment resisting action and the infill panel tension field action. A design procedure considering the contributions of these two actions to the overall SPSW strength (i.e. a procedure to achieve a balanced design) was developed. Design of SPSWs having weak infill panels was also studied and three different methods that assume different HBE strength distributions along the SPSW height were presented.

A series of nonlinear time history analyses were conducted to evaluate the seismic performance of SPSWs designed using these different procedures (i.e. respectively designed using the AISC Seismic Provisions, using the developed balanced design procedure, and using the three methods for the SPSWs having weak infill panels). It was shown that the SPSW designed using the balanced design procedure exhibits similar performance to that designed using the AISC Seismic Provisions.

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