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SEISMIC BEHAVIOR OF INTERMEDIATE BEAMS IN STEEL PLATE SHEAR WALLS

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ABSTRACT

This paper presents some preliminary results of an ongoing research whose objective is to investigate the seismic behavior of intermediate beams (i.e., the beams other than those at the roof and foundation levels) in multi-story steel plate shear walls (SPSWs). Of primary interest is the determination of the strength level needed to avoid the formation of in-span plastic hinges, a relevant practical issue that has not been considered in past investigations. To attain this objective, the seismic response of different SPSW models was analyzed by performing linear and nonlinear analysis. The intermediate beams of the SPSW models were designed to resist: (I) forces imposed by gravity loads only; (II) forces determined by the ASCE 7 load combinations; and (III) forces imposed by fully yielded plates. For comparison purposes, SPSW models designed according to the Canadian Standard CAN/CSA S16-01 were considered as well. It is concluded that intermediate beams designed according to criteria I and II are prone to in-span plastic hinges and to excessive plastic deformations. It was also found that, while in-span plastic hinges do not appear in the intermediate beams of the CAN/CSA S16-01 models, they do appear in the roof and foundation beams of these models when a collapse mechanism develops.

Introduction

A steel plate shear wall (SPSW) is a lateral load resisting system consisting of vertical steel plate infills connected to the surrounding beams and columns and installed in one or more bays along the full height of the structure to form a cantilever wall (Fig. 1). As determined by several experimental and analytical investigations (a literature review can be seen in Berman and Bruneau 2003), SPSWs subjected to cyclic inelastic deformations exhibit high initial stiffness, behave in a very ductile manner, and dissipate significant amounts of energy. These characteristics make them suitable to adequately resist and dissipate seismic loading. SPSWs can be used for the design of new buildings and, as indicated by recent research efforts (Vian and Bruneau 2004, Berman and Bruneau 2005), for the retrofit of existing constructions as well.

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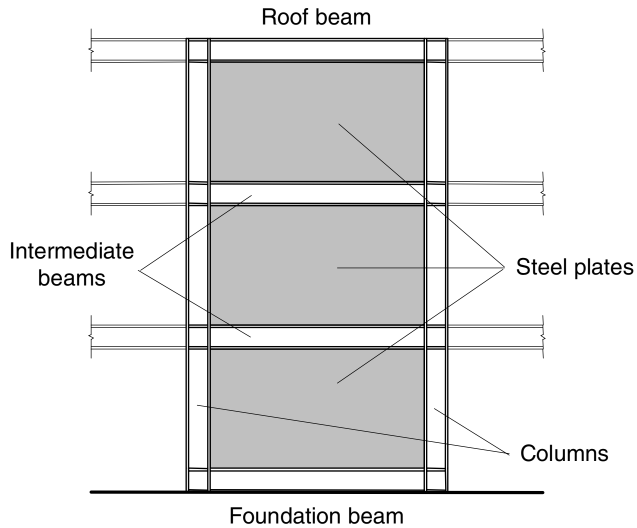


Figure 1. Schematics of a typical Steel Plate Shear Wall (SPSW)

SPSWs are a relatively novel type of structural system. Experimental and analytical studies on the application of SPSWs in building structures started in the early 1980s, and code regulations did not appeared in North America until 1994, when some minimum requirements and limitations were specified in that year's edition of the Canadian standard *Limit States Design of Steel Structures* (CSA 1994). In the USA, design guidelines appeared for the first time in the 2003 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (Building Seismic Safety Council 2004), and essentially the same guidelines have been included in the *2005 Seismic Provisions for Structural Steel Buildings* (AISC, 2005). In this paper, these documents will subsequently be referred to simply as "FEMA 450" and "AISC Seismic Provisions", respectively. In these documents, SPSWs components are designated as follows: columns are referred to as *Vertical Boundary Elements* (VBEs), beams are labeled *Horizontal Boundary Elements* (HBEs), steel panels are denoted simply as *webs*, and a web and its surrounding HBEs and VBEs constitutes a *panel*.

The FEMA/AISC regulations specify that "HBEs and VBEs adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted". In other words, inelastic deformations are allowed only in the webs and at the ends of the HBEs. In addition, the FEMA/AISC regulations indicate that "the required strength of HBEs shall be the greater of the forces corresponding to the expected yield strength (in tension) of the web calculated at an angle α or that determined from the load combinations in ASCE 7 (FEMA 450) or in the applicable building code (AISC Seismic Provisions) assuming the web provides no support for gravity loads". While the forces corresponding to the expected yield strength of the web always dominate the design of the roof and foundation beams (the so-called "anchor" beams), the forces determined from the load combinations often dictate the strength of intermediate beams, especially when the adjacent webs are of similar thickness. For clarity, these observations are illustrated in Fig. 2, where w_i is the vertical component of the forces imposed by yielding webs on the i-th story beam, f_{yp} is the yield strength of the webs, t_{wi} is the thickness of the i-th story web, n is the number of stories and α_i is the angle of inclination, measured with respect to the vertical direction, at the i-th story web (this angle is calculated using a well-known expression included in the FEMA/AISC regulations). It can be seen that the forces imposed by yielding webs on both sides of intermediate beams have the opposite orientation. As a result, the net forces imposed by yielding webs on intermediate beams are of lesser magnitude than those acting on the anchor beams. Further, if the webs adjacent to a given intermediate beam are of equal thickness (which is often the case in practice due to limited availability of plates), the magnitude of the net forces acting on the beam reduces considerably (it actually reduces to zero when $\alpha_i = \alpha_{i+1}$), and the resulting bending moments in the beam are then usually smaller than those determined from the load combinations. Since the seismic behavior of intermediate beams of SPSWs has not been investigated in previous research efforts, there is still a question about the validity of the FEMA/AISC minimum strength requirement for intermediate HBEs, especially when the webs are of equal thickness.

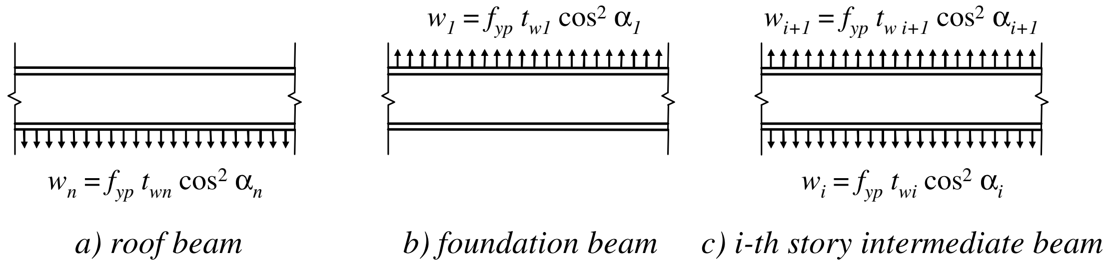


Figure 2. Vertical component of the forces imposed by yielding webs on beams of SPSWs

The objectives of the study reported in this paper are: (a) to obtain more insight into the behavior of intermediate beams of SPSWs subjected to seismic loading; and (b) to examine the adequacy of the FEMA/AISC minimum strength requirement for HBEs of SPSWs. These objectives were pursued by investigating the seismic response of various SPSW models designed according to the FEMA/AISC regulations and having intermediate beams of different strength levels. The preliminary results shown in this paper are those given by FEMA 450's Equivalent Lateral Force and Nonlinear Static ("pushover") procedures only. For comparison purposes, models designed according to the Canadian standard *Limit States Design of Steel Structures* (CSA 2001) where considered as well. In this study, this Canadian code will be subsequently referred to as "CAN/CSA S16-01".

Description of the models and modeling assumptions

The SPSW models considered in this research were designed as an alternative to provide the lateral load resisting system to the 4-story MCEER Demonstration Hospital, a reference building model used as part of a broader MCEER research project. A complete description of this benchmark building model can be seen in Astrella (2005). It was assumed that a total of 4 SPSWs provide lateral force resistance in the direction under consideration, and that the SPSWs are located at the central bay of the corresponding 3-bay frames. Seismic loads were calculated assuming that the structure is located in Northridge, California (S_S = 1.75 g and S_1 = 0.75 g). The natural period was estimated by:

$$T = 0.02\, h_n^{0.75} = 0.38 \text{ sec} \tag{1}$$

where h_n = 51 ft is the total height of the SPSW systems. Eq. 1 is indicated as appropriate for SPSWs in Appendix R of the AISC Seismic Provisions, and is the one to be used for "all other structural systems" in FEMA 450. Assuming Soil Type D, Occupancy Factor I = 1.5 (hospitals belong to Seismic Group III) and R = 7 (the value indicated by the FEMA/AISC guidelines), the seismic response coefficient is then equal to C_s = 0.25. Loads are summarized in Fig. 3.

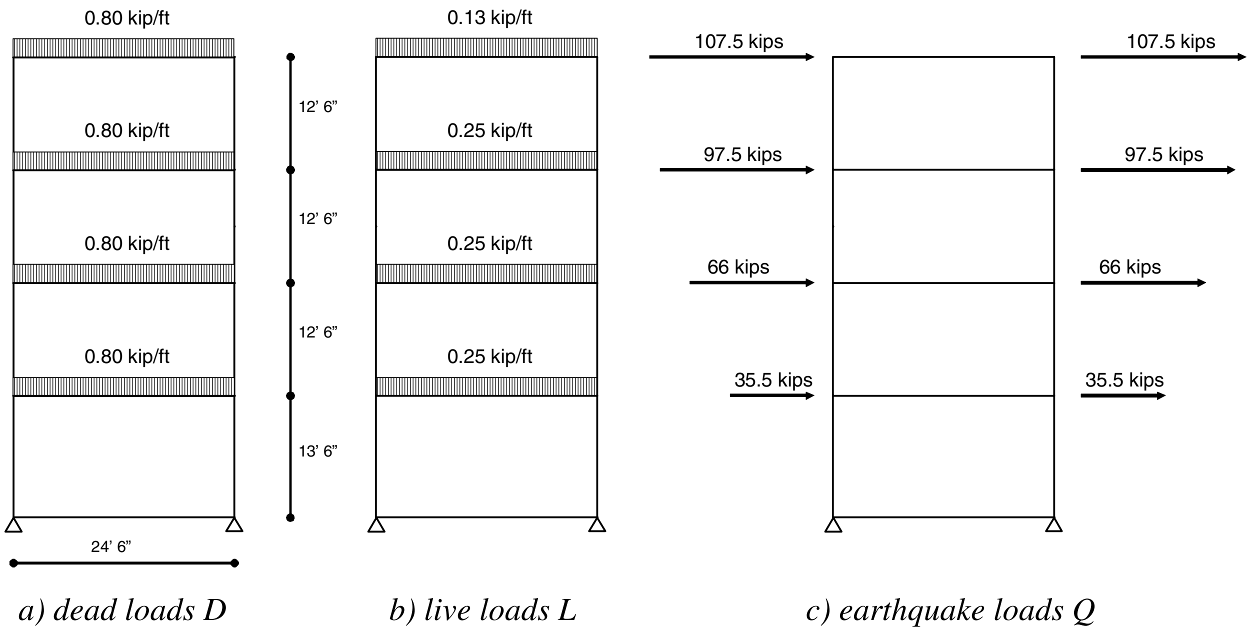


Figure 3. Geometry and loads of the SPSW models considered in this study

The SPSWs were analyzed using the commercial computer program SAP-2000 version 8.3.3 (CSI 2004). Each web was modeled by a set of 10 parallel, uniformly spaced tension-only strips pinned at both ends (i.e., elements capable of resisting tensile axial forces only), while the beams and columns were modeled by conventional frame elements. The strips' Young modulus was set equal to that of steel, the area of each strip was set equal to its tributary width times the web thickness, and the strength of each strip was set equal to its area times the yield stress of the web. This modeling approach is known as the strip model, and its accuracy has been verified through comparisons with experimental results (see, for instance, Driver et al. 1998).

Forces imposed by the ASCE 7 load combinations were assessed through FEMA 450's Equivalent Lateral Force Procedure (linear static analysis). The inelastic behavior of the SPSW models under seismic loading was analyzed by performing FEMA 450's Nonlinear Static Procedure (pushover analysis). For this, P-M plastic hinges were modeled at the ends of the beams and columns and at each intersection of a strip and a frame member. The strain-hardening ratio was set equal to 0.5%. Axial plastic hinges having an elastic, perfectly plastic force-deformation relationship were modeled at the middle of each strip. As indicated by FEMA 450, a set of gravity loads equal to $D + 0.25\,L$ was applied to the models prior to the incremental application of earthquake loads. The pattern of seismic forces was set equal to that indicated by the Equivalent Lateral Force Procedure.

Design of the SPSW models

For all models, A36 steel was assumed for the plates (f_{yp} = 36 ksi) and A992 steel was assumed for HBEs and VBEs (f_{ym} = 50 ksi). The lightest of all W-shapes satisfying the strength requirements were selected (minimum weight design), and were checked to comply with the AISC Seismic Provisions' compact section requirements. All the intermediate beams of the models considered in this paper have the same size. All models were checked to comply with FEMA 450's story drift limitations (≤ 1.50 %).

Models designed according to the FEMA/AISC regulations

The anchor beams were sized as indicated by Vian (2005), who showed that these beams can resist fully yielding plates without developing mid-span hinges if their plastic modulae comply with:

$$Z_{b1} \geq \frac{f_{yp}\ t_{w1}\ \cos^2(\alpha_1)\ L^2}{4\ f_{ym}} \text{ (foundation beam)}, \qquad Z_{bn} \geq \frac{f_{yp}\ t_{wn}\ \cos^2(\alpha_n)\ L^2}{4\ f_{ym}} \text{ (roof beam)} \qquad (3)$$

where L is the length of the beam. As indicated in the AISC Seismic Provisions (but not in FEMA 450), columns were then seized based on the beam-column moment ratio specified in Section 9.6 of the AISC Seismic Provisions, for which axial loads in the columns were set equal to those generated by fully yielded webs. Finally, intermediate beams were designed to resist: (I) forces generated by gravity loads only; (II) forces determined by the ASCE 7 load combinations; and (III) forces generated by fully yielded webs. For the latter criterion, a suitable modification of Eq. 3 for intermediate beams was adopted, i.e.:

$$Z_{bi} \geq \frac{f_{yp}\ \left[t_{wi}\ \cos^2(\alpha_i) - t_{wi+1}\ \cos^2(\alpha_{i+1})\right] L^2}{4\ f_{ym}} \qquad (i = 2, 3, \ldots, n\text{-}1) \qquad (4)$$

In all cases, the thickness of the thinnest hot-rolled plate available (= 3/16") turned out to be larger that the minimum thickness required by the FEMA/AISC guidelines, which resulted in SPSW models having the same plate thickness at all stories (constant plate thickness). In order to obtain insight into the behavior of intermediate beams of SPSWs having different plate thickness at different stories, models whose plates have the minimum thickness required were also considered for each of the abovementioned design criteria (variable plate thickness). Resulting models are summarized in Fig. 4.

Models designed according to the CAN/CSA S16-01 regulations

As recommended in the CAN/CSA S16-01 standard, preliminary sizes of webs and frame members were obtained by modeling the webs as diagonal pinned braces and by subjecting the resulting vertical truss (the so-called truss model) to the earthquake loads Q. The strip model was then used to verify the design (some adjustments were necessary). It must be noted that, according to this Canadian standard, the thickness of the webs is determined by strength considerations only (there is no web slenderness requirement), and frame members must be able to resist the forces generated by the load combinations multiplied by the ratio of the probable strength of the 1st-story plate (calculated by an equation provided by this code) to the design base shear. For the same reasons mentioned above, both constant and variable plate thickness SPSW models were developed. Resulting models are also shown in Fig. 4. These models were also checked to comply with the beam-column moment ratio specified in the AISC Seismic Provisions.

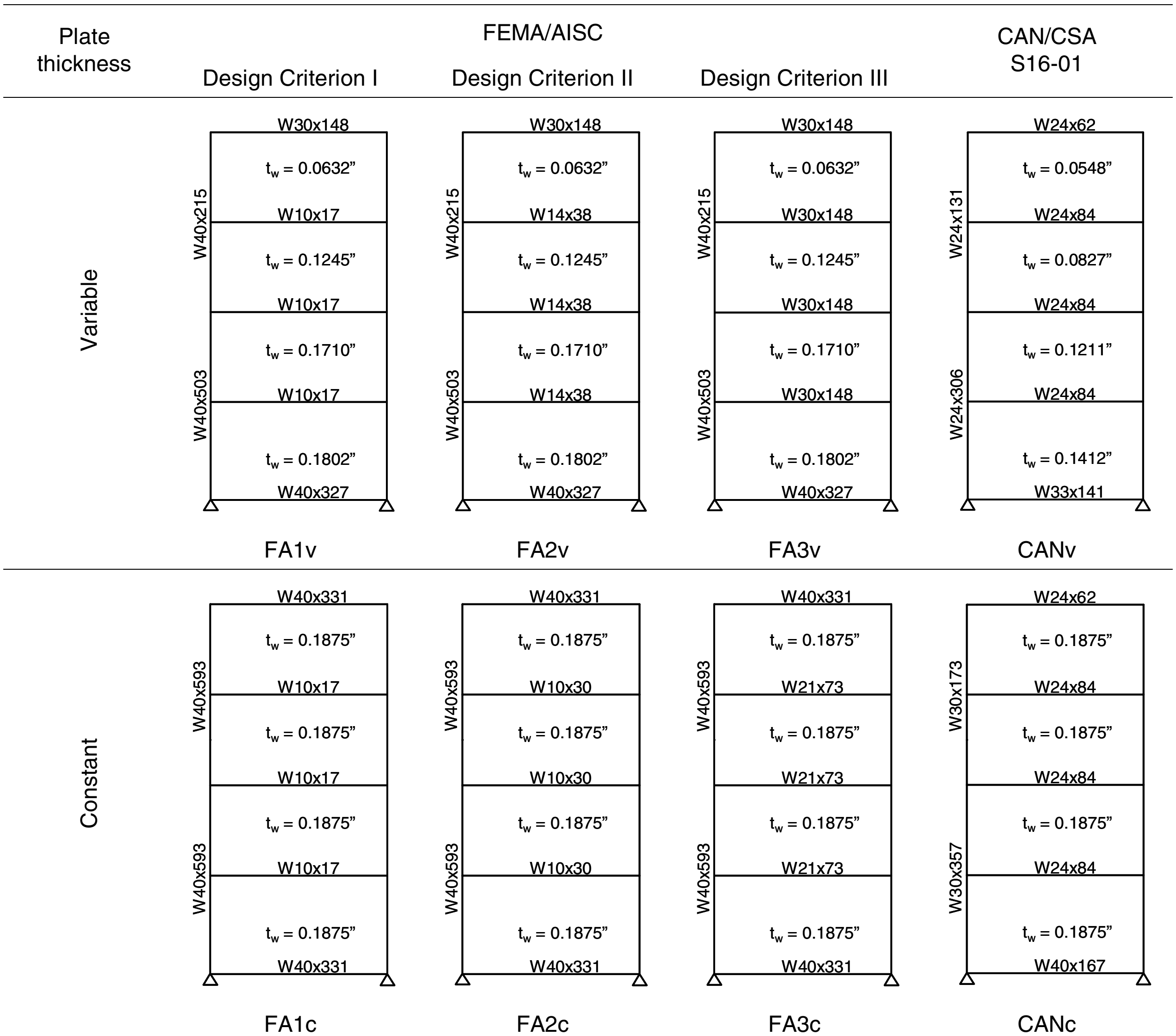
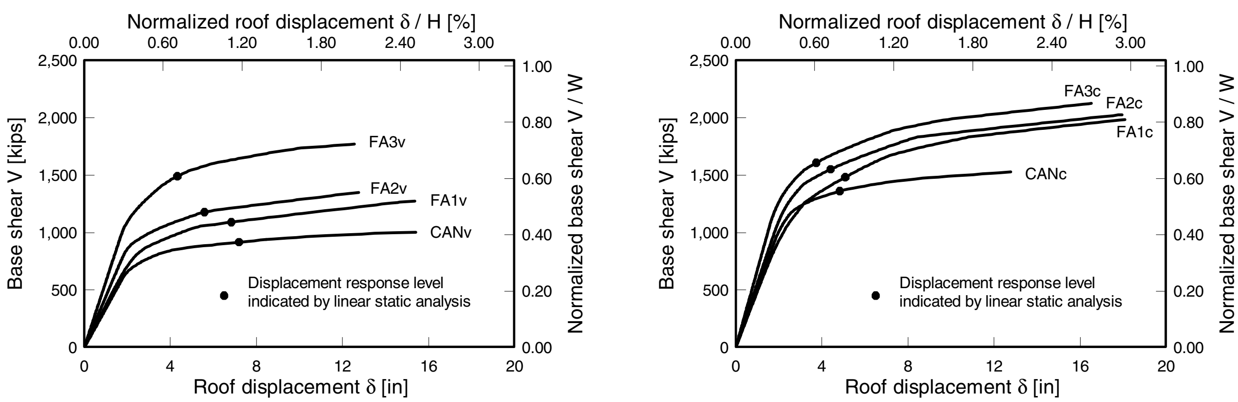


Figure 4. SPSW models considered in this study

Analysis of results

Response to the design basis earthquake

The displacement response of the SPSW systems to the design basis earthquake, (DBE) calculated through FEMA 450's Equivalent Lateral Force Procedure, was found to be very similar to that obtained through preliminary nonlinear time-history analysis using response-spectrum-compatible ground excitations. Based on this observation, inelastic deformations in the SPSW models imposed by the DBE were then assumed equal to those obtained through pushover analysis at the displacement response level indicated by FEMA 450's Equivalent Lateral Force Procedure. This approach is illustrated in Fig. 5, where it must be noted that the ultimate strength of the SPSW models is in all cases well above the design base shear (= 0.25 *W*), especially in the case of the constant-plate-thickness models. Inelastic deformations are summarized in Fig. 6, where it can be seen that the intermediate beams of the models designed according to Design Criteria I and II exhibit in-span plastic hinges. Besides, these models also exhibit an uneven distribution of web yielding. It must be noted that the 2nd-story beam of model FA2c does comply with the current FEMA/AISC regulations, yet several in-span hinges are observed in this member. The models designed according to the Design Criterion III and the CAN/CSA S16-01 standard, on the other hand, exhibit a satisfactory behavior in the sense that in-span plastic hinges do not develop in intermediate beams and almost full yielding is observed at all webs (the only exception is the top story of the constant-plate-thickness models).



a) Variable plate thickness *b) Constant plate thickness*

Figure 5. Base shear *V* vs. roof displacement δ curves obtained through nonlinear static analysis (*H* and *W* are total height and total reactive weight of the SPSWs)

Global collapse mechanism

When performing pushover analysis, seismic loads were applied incrementally until a global collapse mechanism developed (local mechanisms at intermediate beams were not considered). The corresponding inelastic deformations in the SPSW models are summarized in Fig. 7. In the case of the models designed according to Design Criteria I and II, in addition to the already observed in-span plastic hinges in intermediate beams, limited yielding is observed at many of their webs. In-span plastic hinges, however, did not develop in the anchor beams. The Design Criterion III models exhibit the desired behavior: inelastic deformations occur only at the ends of the beams and in the webs, which are yielding in full. The plastic hinges observed next to the left-end of the 1st and 2nd story beams and next to the right-end of the foundation beam in model FA3c should be interpreted as extensions of the plastic hinges at the ends of these beams rather than in-span hinges. Finally, in-span plastic hinges did develop in the CAN/CSA S16-01 models, but in the anchor beams rather than in the intermediate beams. This observation provides further support to the validity of Eq. 3, which was not considered when designing the CAN/CSA S16-01 models.

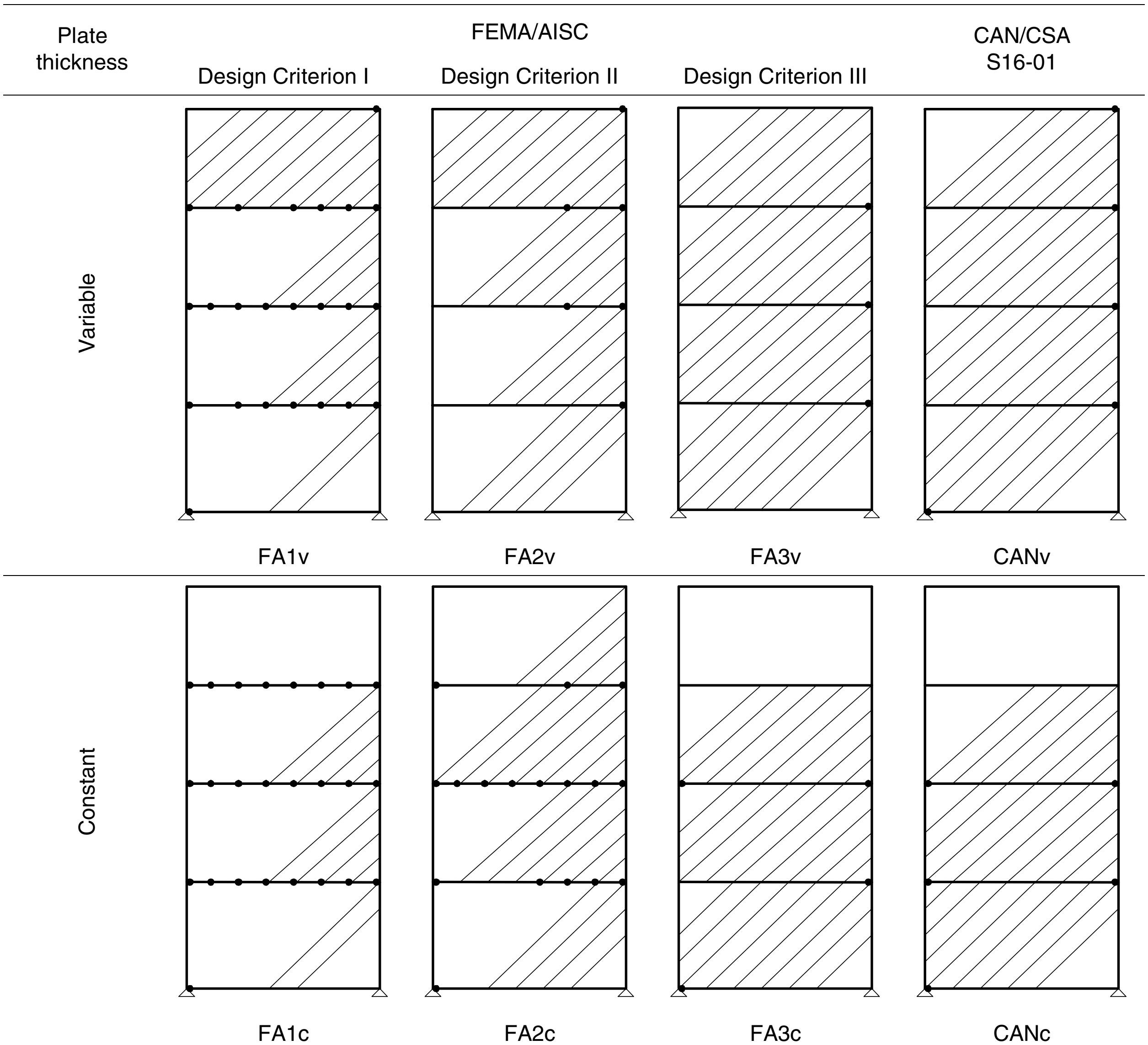


Figure 6. Inelastic deformations under the DBE (strips not yielding are not shown)

It was also observed that plastic hinges developed in the left-side, 3rd-story column of model FA1c and in the right-side, 3rd-story column of model CANc, even though these models satisfy the beam-column moment ratio specified in the AISC Seismic Provisions at all beam-columns joints. This observation raises questions about the applicability of Section 9.6 of the AISC Seismic Provisions to SPSW systems, and is the focus of an on-going investigation.

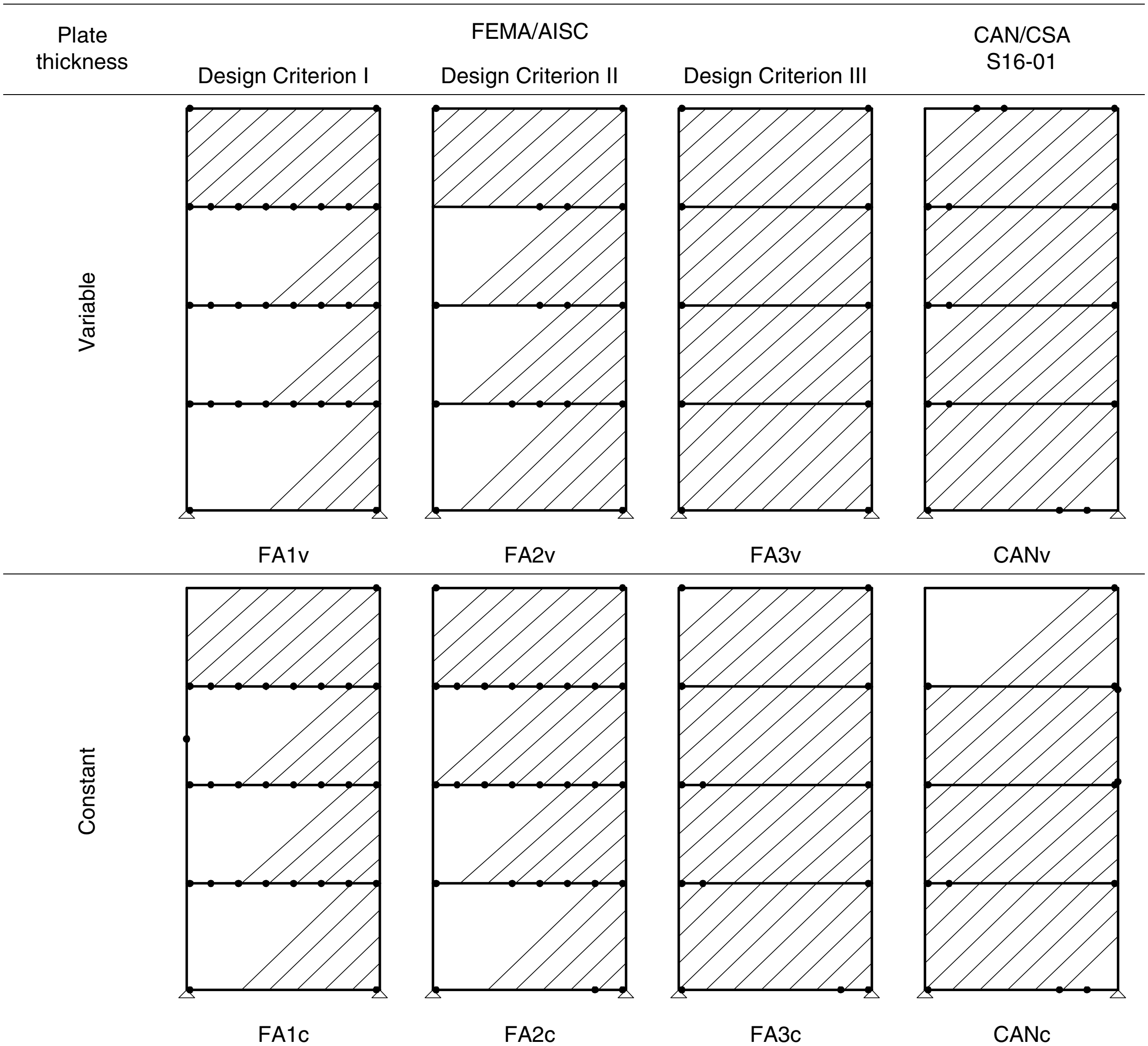


Figure 7. Global collapse mechanisms of the SPSW models (strips not yielding are not shown)

Conclusions

In this study, the seismic response of several SPSW models designed according to the FEMA/AISC regulations and the Canadian standard CAN/CSA S16-01 was analyzed through FEMA 450's Equivalent Lateral Force and Nonlinear Static procedures. The intermediate beams of the FEMA/AISC models were designed according to three different criteria: (I) for gravity loads only; (II) for the ASCE 7 load combinations; and (III) for the forces generated by fully yielded webs. It was found that the models designed according to criteria I and II developed in-span plastic hinges in intermediate beams, which is not allowed by the FEMA/AISC regulations. Models designed according to criteria III exhibited the desired behavior (i.e., inelastic deformations occur only at the ends of the beams and in the webs) even when a global collapse mechanism developed. The behavior of the CAN/CSA S16-01 models was found to be satisfactory at the response level corresponding to the DBE, but the global mechanism of these models included in-span plastic hinges in the anchor beams. All these observations were found to apply regardless of whether the SPSW systems have constant or variable plate thickness.

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