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SEISMIC EVALUATION OF CODE DESIGNED STEEL PLATE SHEAR WALLS UNDER MAINSHOCK-AFTERSHOCK EARTHQUAKE SEQUENCES

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Structures are often subjected to sequences of mainshock and aftershocks during their service life. Strong aftershocks have been known to cause extensive structural damage and losses of human lives in addition to the damage and losses of the mainshock. Steel plate shear walls (SPSWs) have recently attracted considerable interest as a promising lateral-resisting system in seismic-prone regions. The SPSWs consist of steel beams and columns with a thin infill steel plate. The unstiffened thin plates are expected to buckle at low seismic loads and develop tension field action that enables favorable ductility and energy dissipation. The American Institute of Steel Construction's Seismic Provisions prescribe a capacity-based design methodology that uses SPSWs as primary seismic force resisting systems, and require the design of SPSWs to be consistent with the design requirements for other ductile steel seismic force resisting systems. This paper presents numerical analysis of a code-designed eight-story SPSW building that was subjected to consecutive mainshock-aftershock earthquake events. Ground motions developed for the SAC project were selected and scaled to simulate the main shock events at maximum considered earthquake (MCE) level, and large aftershocks events at design base earthquake (DBE) level. Nonlinear response history analysis using the finite element program OpenSees were conducted to investigate the performance of the code designed SPSWs. It was demonstrated that the designed SPSW has good performance under mainshock-aftershock sequences.

Keywords: Steel plate shear wall, Mainshock, Aftershock.

1 INTRODUCTION

Steel Plate Shear Walls (SPSW) are a lateral force resisting system which resists both wind and earthquake forces. The steel beams and columns that form the frame are referred as Horizontal Boundary Elements (HBE), and Vertical Boundary Elements (VBE). The main difference between plate girders and SPSW is the greater strength and stiffness of the SPSW columns compared to the plate girder flange (Bruneau and Berman 2004, Bruneau *et al.* 2006, Berman 2011). The SPSW system has relatively high initial stiffness when compared with a reinforced concrete shear wall, and thus is very effective in limiting the drift. The SPSW system is much lighter, which can result in less weight being carried by the columns and foundations, as well as less seismic load. The SPSW system has been adopted first in the Canadian design standard (CSA 2001), and then in the American steel design standard (AISC 2005).

Earthquake events are typically composed of foreshocks, a mainshock, and aftershocks. Mainshock events release the largest amount of energy and often cause most of the structural damage. It's very common to observe many aftershocks following the mainshock. For example, there were a total of 588 aftershocks with magnitude 5 and greater recorded after the Earthquake in Japan 2011. Aftershocks may have a large ground motion intensity, longer duration and different frequency content (Li and Ellingwood 2007). However most current studies often focus on the structural performance under mainshock earthquakes.

2 PROTOTYPE SPSW DESIGN

The building is assumed to be located on a site class D and has SPSW as main lateral system in both directions. There are a total of 4 SPSWs in each direction. The building has a height of 96 feet, and each shear wall panel is 24 feet wide with an aspect ratio of 2.0 and a story height of 12 feet. The elevation of the building is shown in Figure 1. The nominal yield strength of the beams and columns was assumed to be 50 ksi, and 36 ksi for infill plates of SPSWs. The building is assumed to be an office building, therefore has a building occupancy category I as per ASCE 7 (2010). A response modification coefficient R of 8 is used, the system over-strength factor Ω is found to be 2.5, and the deflection amplification factor C_d is 6.5. For the building location, the following are the seismic design values: S_s=1.50g, S_1=0.603g, S_{MS}=1.50g, S_{M1}=0.905g, S_{DS}=1.0g and S_{D1}=0.603g. The HBEs and VBEs are designed using capacity design principles that assume all plates yield simultaneously. Moment connections were used for all HBE-to-VBE connections. Table 1 summarizes the design of the eight-story steel plate shear wall building.

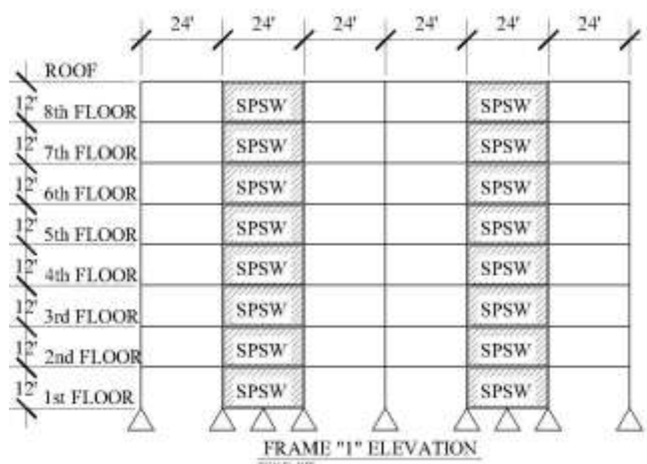

Figure 1. Elevation of the eight-story steel plate shear wall building.

Table 1. Properties of the 8-Story SPSW.

Story	HBE Size	Fy (ksi)	VBE Size	Fy (ksi)	Web Plate (in)	Fy (ksi)
1	W27x94	50	W14x550	50	0.25	36
2	W27x146	50	W14x550	50	0.25	36
3	W27x94	50	W14x550	50	0.1875	36
4	W27x129	50	W14x550	50	0.1875	36
5	W27x94	50	W14x233	50	0.134	36
6	W27x94	50	W14x233	50	0.105	36
7	W27x94	50	W14x233	50	0.075	36
8	W27x129	50	W14x233	50	0.0625	36

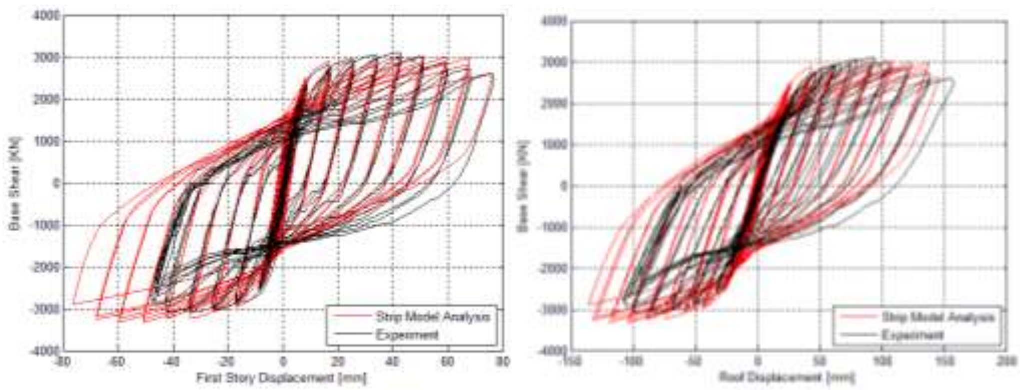

Figure 2. Hysteretic Material - Hysteresis Loop: (a) base shear vs. first story displacement; (b) base shear vs. roof displacement.

3 FINITE ELEMENT MODELING OF SPSW

The Open System for Earthquake Engineering Simulation (Mazzoni *et al*. 2006) is used to model the designed SPSW for predicting the structural response under mainshock-aftershock sequences. The force-based nonlinear elements are used to model the beams and columns. Fiber sections are used to capture the interaction between axial force and bending moment in the column response. The flanges were discretized into 8 fibers along the height and 36 fibers along the width. The webs were discretized into 4 fibers along the width and 36 fibers along the height. Each fiber contains a UniaxialMaterial element with bilinear stress-strain relationship to model the steel material. The bilinear stress-strain relationship is represented using the Giuffre-Menegotto-Pinto steel material model (Mazzoni *et al.* 2006) as Steel02 material for both kinematic and isotropic hardening. The HBE-to-VBE connections were modeled as moment resisting connections and one-eighth of the total floor mass was lumped at each of the nodes. The web plates are modeled using the strip model, where the plates are represented by two sets of parallel pin-ended, discrete tension-only strip elements. One set of strips is oriented at an angle α to the vertical, representing the angle of the tension field that develops after the web plate buckles in shear. The other set of strips is oriented at $2\pi-\alpha$ to provide resistance under reversed loading. Each strip was modeled using a truss element and the Hysteretic material model in OpenSees. A tri-linear tension stress–

strain curve was used with an expected initial yield stress of 46.8 ksi. The hysteretic material adopted in this study allows for the development of tension-only elements with “pinched” cyclic behavior, which is characteristic of web plates in tension field action undergoing cyclic yielding.

To validate the modelling approach, an analytical strip model of a four-story steel plate shear wall was also developed, and analysis results were compared with experimental results from Driver *et al*. (1998). See Figures 2(a) and 2(b), which show base shear versus first story displacement, and base shear versus roof displacement respectively. Good agreement can be observed. It should also be noted that the analyses were performed using a symmetric loading history, whereas the loading history in the experiments was not symmetric due to the limited stroke capacity of the actuators, as indicated by Driver et al. (1998). Good agreement can be observed between the numerical prediction and the experimental results.

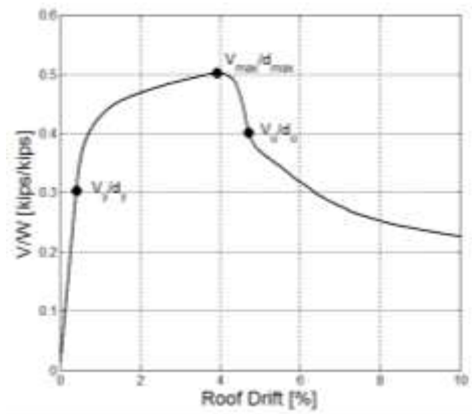

Figure 3. Nonlinear Static Analysis Pushover Curve of 8-story SPSW.

Also shown in Figure 3 is the nonlinear static push-over analysis of the 8-story SPSW. It can be observed that the analytical model yields at a roof drift of about 0.41% and base shear coefficient of 0.31. The maximum shear V_{max} is taken as the maximum base shear strength on the pushover curve, and the ultimate displacement δu is taken as the roof displacement at the point of 20% strength loss (0.8Vmax). As can be observed, the coefficient of V_{max} is 0.503, or 2,108.34 kips at the roof drift of 3.919%. The ultimate base shear coefficient is 0.402 or 1,686.66 kips at the roof drift of 4.705%. The effective elastic stiffness is 269 kips/in. These results are later used as benchmark values to evaluate the nonlinear response time history analyses.

4 PERFORMANCE EVALUATION FOR MAINSHOCK-AFTERSHOCK

Ten pairs of ground motions developed for the SAC project were used in this study and were scaled to maximum considered earthquake (MCE), with a 2% probability of occurrence in 50 years, and to design basis earthquake (DBE), with a 10% probability of occurrence in 50 years. Figure 4 presents the response spectrum of the twenty ground motions. The same scaling factor is used for the two components of each pair of ground motions, so that the average of the two spectral values match with a leastsquare error fit to the United States Geological Survey's (USGS) mapped values at 0.3, 1.0, and 2.0 seconds, and an additional predicted value at 4.0 seconds (Somerville *et al*. 1997). To select the ground motion for mainshock event, a time histories analysis of the SPSW were first conducted for all twenty ground motions scaled to DBE level. The ground motion LA27 was selected as the mainshock event since it causes the maximum story drift.

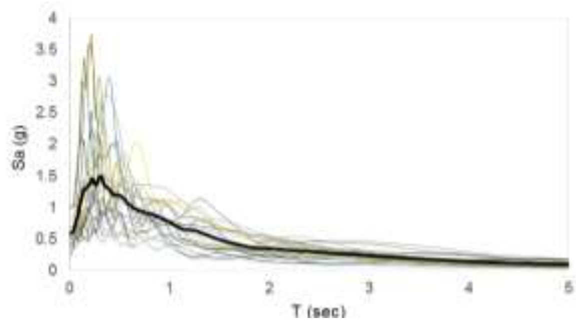

Figure 4. Response spectrum of the twenty ground motions.

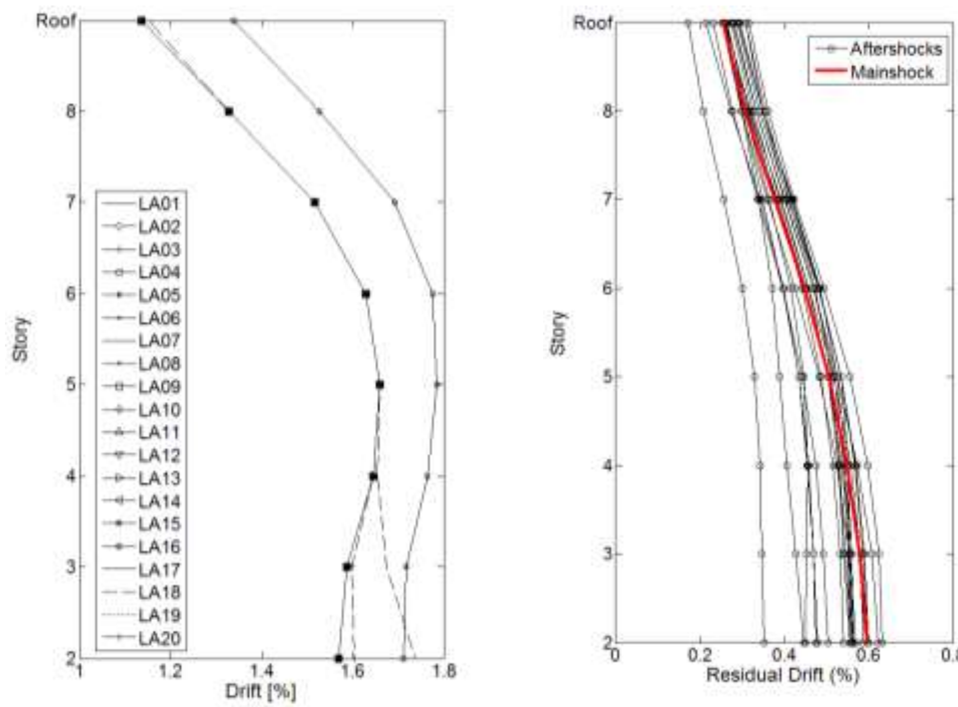

Figure 5. a) Inter-story Drifts for LA27+DBE; b) Residual Drift Results for LA27+DBE.

Figure 5(a) presents the inter-story drift for the mainshock-aftershocks of LA27+DBE. During the median level 2/50 ground motion, the structure experienced some moderate damage that contributed to the increase of inter-story drift during the

LA16 aftershock. This observation also indicates that the code-designed SPSW performed well during the mainshock-aftershock event. Figure 5(b) presents the residual drift for the same mainshock-aftershock sequence of LA27+DBE. Here the residual drift is used not as a direct method to quantify the structural damage, but as one of the indicators to identify the mainshock-aftershock sequence that produces the largest damage in the structure. The largest residual displacement is observed to occur during the LA27-LA03 sequence. This implies that the aftershock causes additional damage to the SPSW under study.

5 SUMMARY AND CONCLUSION

An eight-story SPSW was designed and its performance evaluated when subjected to a selected mainshock aftershock event. A finite-element model using OpenSees was first validated using previous experimental results, and then used to evaluate the designed SPSW. A suite of twenty ground motions were selected and scaled for the present study. It was demonstrated that the designed SPSW has good performance under mainshock-aftershock sequences.

Acknowledgments

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