



## **DEVELOPMENT OF A RECENTERING STEEL PLATE SHEAR WALL AND ADDRESSING CRITICAL STEEL PLATE SHEAR WALL RESEARCH NEEDS**

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### **ABSTRACT**

Innovative advances in steel plate shear wall technology could produce systems that meet stringent performance requirements, including zero residual drifts. Here, an overview of a recently awarded NEESR-SG research project is presented that seeks to develop a resilient steel plate shear wall system. The system strategically combines the benefits of self-centering and steel plate shear wall technologies to create a robust, ductile, and easily repairable system that has the potential to reduce life-cycle costs for buildings. Analytical models of a proposed resilient steel plate shear system are presented and preliminary design requirements for achieving specific performance objectives are discussed. Nonlinear response history analysis results show that the resilient steel plate shear wall concept is capable of recentering and meeting other critical performance objectives. The project also seeks to fill critical knowledge gaps in the understanding of steel plate shear wall behavior and design approaches. These knowledge gaps include: (i) tools for performance-based design, (ii) expedient and accurate modeling techniques, especially for high-rise steel plate shear walls, and (iii) the behavior of, and design approach for, coupled steel plate shear walls. The experimental and analytical research program to address the knowledge gaps, their progress, and the preliminary results are briefly discussed.

### **Introduction**

Steel plate shear walls (SPSWs) feature thin plates (denoted web plates) that infill a frame of beams (denoted horizontal boundary elements, HBEs) and columns (denoted vertical boundary elements, VBEs). The web plates are typically very slender and buckle in shear at low load levels, developing a tension field that then resists lateral loads. Design requirements for SPSWs now appear in both the AISC Seismic Provisions for Steel Buildings (AISC 2005) and the Canadian Standard CAN/CSA S16-09 (CSA 2009). The research described here aims to push SPSW technology forward by developing a resilient steel plate shear wall (R-SPSW) that utilizes recentering technologies to create a high performance lateral load resisting system that minimizes post-event repair costs and downtime. To develop the R-SPSW a combination of

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analytical and experimental research tasks are underway at the University of Washington and the University at Buffalo. This paper will describe the initial analytical work, a preliminary approach for design that focuses on achieving specific performance objectives, and the future experimental tasks that will use the NEES facility at the University at Buffalo and Structural Research Laboratory at the University of Washington.

In parallel with the development of the R-SPSW, research is also being conducted to fill critical knowledge gaps in the behavior and design of more conventional SPSWs. This paper will highlight the development of performance-based design tools for SPSWs for implementation in the ATC-58 performance-based seismic design procedures. Also briefly discussed are the planned analytical and experimental research tasks, the latter to be conducted at the University of Illinois using the NEES MUST-SIM facility, to investigate the behavior of coupled SPSWs, which are necessary for implementation in core systems in high-rise buildings but for which little design guidance is available. Further, the development of new expedient models for representing web plate tension field is briefly discussed.

The NEESR SPSW research project is a collaboration that includes researchers at the University of Washington, University at Buffalo, University of Illinois, and the National Center for Research in Earthquake Engineering in Taiwan.

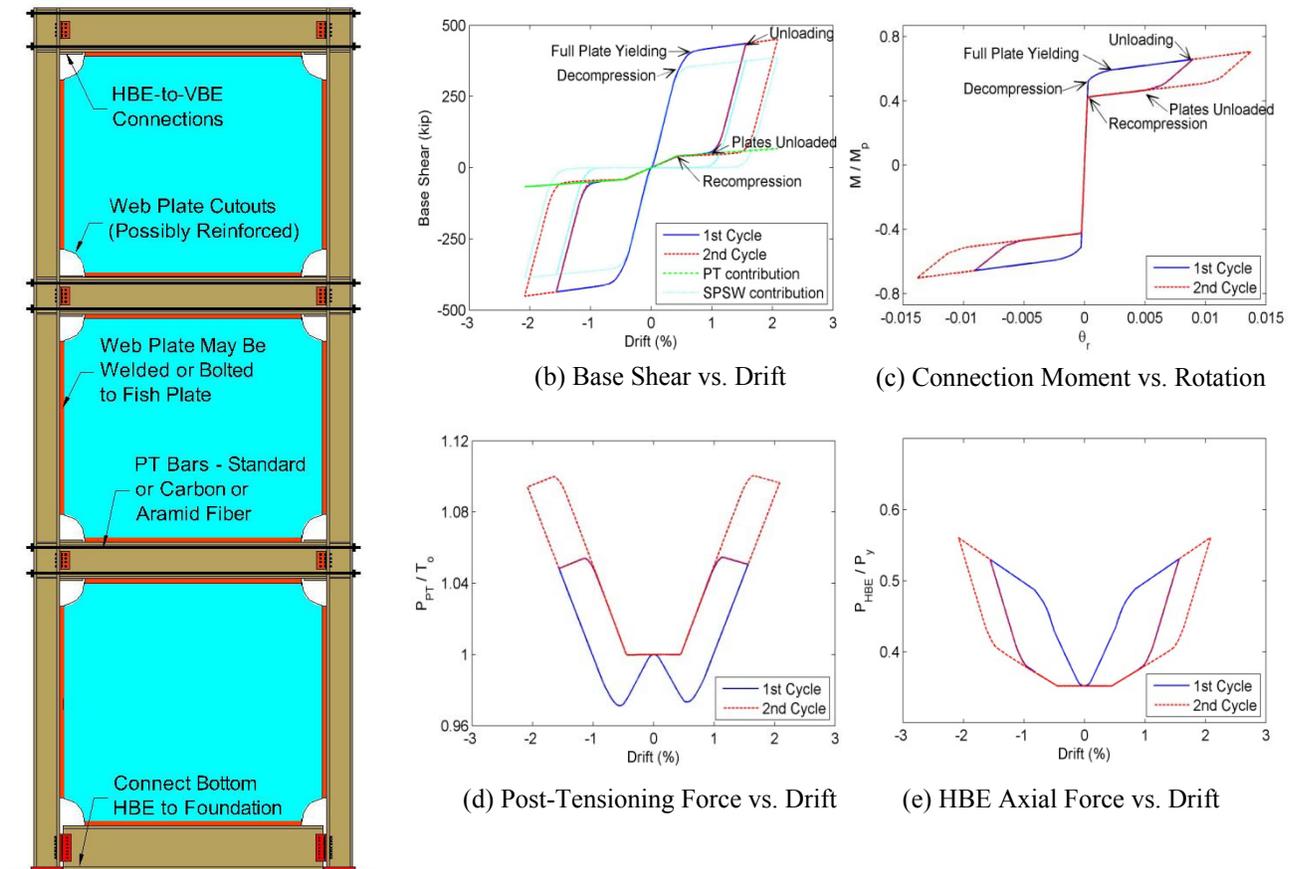
### **The Resilient Steel Plate Shear Wall System**

The SPSW system utilizes thin steel infill plates to act as the ductile fuses of the lateral force resisting system to distribute yielding over the entire height of the structure. The relatively large energy dissipation capabilities of the steel plates, the high initial stiffness of the system, and the “replaceability” of the ductile fuses after a seismic event make the system especially attractive in high seismic regions. The benefits of the SPSW system can be improved further with the integration of self-centering technologies. The self-centering systems considered here rely on the elasticity of post-tensioned strands, made of steel or other materials, to bring the building back to an operational state after a seismic event. Although by themselves, self-centering systems do not offer any energy dissipative qualities, they can be paired with systems that do, in this case the SPSW. The combination of these two systems results in a more efficient and robust design that allows for easier post-event repair and quicker return to occupancy.

Fig. 1a shows a schematic of the proposed R-SPSW. The system is a SPSW that also features post-tensioned moment resisting HBE-to-VBE connections (denoted PT connections) that are similar to those developed in recent research by others (Ricles et al. 2001, Christopoulos et al. 2002, and Garlock et al. 2005, among others). Web plate cutouts at the intersection of the HBEs and VBEs are necessary to prevent damage to the web plate as the post-tensioned connections rock about the beam flanges. The primary advantage that this system delivers relative to post-tensioned moment resisting frames is that it has a high initial stiffness provided by the web plates that will help to reduce drifts and reduce the number of lateral load resisting systems necessary in a building.

The behavior of the system and its key components are also illustrated in Fig. 1, where the values are for a preliminary system design and are presented for illustrative purposes only.

Since the web plates have low buckling resistance and rely on tension field action, their hysteretic behavior resembles that of tension only brace, albeit a brace that is spread out over the entire bay. Thus the response of the system is rather pinched; however, the PT connections do provide some stiffness around zero story drift and the large volume of steel web plate that yields provides significant energy dissipation. The response of the PT connections is illustrated in Fig. 1c as connection moment,  $M$ , versus connection gap angle,  $\theta_r$ , where  $M_p$  is the plastic moment capacity of the HBE. The figure demonstrates the impact of web plate tension field yielding on the PT connection strength: as the web plates develop tension field action, axial load develops in the HBE that works to increase the effective post-tensioning in the connection and creating a flag shaped moment-rotation response. The post-tensioning force versus drift is shown in Fig. 1d, where  $P_{PT}$  is the axial load in the post-tensioning strands and  $T_o$  is the initial post-tensioning force, and demonstrates the impact of web plate tension field development on the post-tensioning force. A key component of this system is the HBE. This element is subjected to large axial loads from post-tensioning of the PT connections and from web plate tension field yielding, and large flexural demands from the PT connections. Fig. 1e shows the HBE axial load,  $P_{HBE}$ , normalized by the squash load for the HBE,  $P_y$ , versus drift and demonstrates increasing axial load from both the post-tensioning and web plate with drift.



(a) Schematic of the R-SPSW

Figure 1. A Schematic of the Resilient Steel Plate Shear Wall and the Behavior of the System and Key Components

## Design Objectives

For the R-SPSW system, preliminary design objectives have been developed to generate R-SPSWs that meet specific performance requirements beyond those considered for conventional design. Those objectives are:

1. No connection decompression under wind or gravity loading;
2. Web plates remain elastic under frequent earthquake demands (i.e., the demand representing the 50% probability of exceedance in 50 year earthquake [50/50]);
3. System recentering for the design earthquake and maximum drift limited to 2% (approximated here as the demand representing the 10% probability of exceedance in 50 year earthquake [10/50]);
4. Collapse prevention for the maximum credible earthquake (i.e., the demand representing the 10% probability of exceedance in 50 year earthquake [2/50]);

Table 1 demonstrates which system parameters are controlled by which performance objective. Performance Objective 1 governs the strength design of the PT connections as it requires a certain magnitude decompression moment, which is generally given by the equation:

$$M_d \leq d_{HBE} T_o / 2 \quad (1)$$

where  $d_{HBE}$  is the depth of the HBE and  $T_o$  is the initial post-tensioning. Performance Objective 2 relates to strength and stiffness of the R-SPSW. To achieve this objective it is recommended that the maximum story drift for the 50/50 earthquake be less than 0.5% and that a strength design of the web plates with a response modification factor,  $R$ , equal to 1.0 be performed using the 50/50 ground motion parameters. Therefore, Performance Objective 2 will impact the web plate thickness and the HBE and VBE sizes as all of these contribute to the system stiffness. Performance Objective 3 requires that the recentering system has sufficient stiffness to overcome P-Delta effects once the web plate has yielded such that recentering is ensured. The stiffness of the PT connections is directly affected by the cross-sectional area of the post-tensioning. Further, the maximum drift of 2% in the design level earthquake again requires the system stiffness to be considered. Performance Objective 4 requires capacity design of the HBES and VBEs to support web plate yielding and the moment generated from the PT connections at the large drifts expected in the 2/50 earthquake. Further, PT strains should be less than the yield strain.

Table 1. R-SPSW Performance Objectives and Design Parameters

R-SPSW Design Parameter	Performance Objective			
	1	2	3	4
Web Plate Thickness		X		
PT Area			X	X
Initial PT Force	X			
HBE Depth	X		X	
HBE Strength and Stiffness		X	X	X
VBE Strength and Stiffness		X	X	X

## Preliminary R-SPSW System Design

A preliminary R-SPSW system has been designed based on the 3 story SAC building (Gupta and Krawinkler 1999). The PT connections used in this prototype design feature steel strands spanning the entire length of the HBES within their depths and anchored at the exterior

faces of the VBEs. The HBEs are connected to the VBEs with a simple shear connection that allow the HBEs to rotate freely about their flanges (See Fig. 1).

The 3 story building has story heights of 13 feet with plan dimensions, loading, and seismic masses equivalent to those of the 3 story SAC building. The building was assumed to be on soil of Site Class D with spectral response parameters compatible with the ground motions developed for the SAC project (Gupta and Krawinkler 1999). Initial web plate sizes were selected to satisfy the seismic design loads per the equivalent lateral force method in ASCE 7-05 (ASCE 2005) with a response modification factor,  $R$ , of 7 and an importance factor,  $I$ , of 1. The building was designed to have 6 bays of SC-SPSW in each of the orthogonal directions, each with a bay width of 15 feet. Performance Objective 1, elastic web plates for the 50/50 hazard level, governed the web plate thickness and was ultimately used to design them. Again the equivalent lateral force procedure was used, except  $R$  was taken as 1.0 and the spectral parameters were those for the 50/50 hazard. Plate thicknesses were chosen from available ASTM A36 plate thicknesses per AISC Design Guide 20: Steel Plate Shear Walls (Sabelli and Bruneau 2007).

The PT connections were designed to meet the performance objectives outlined above such that: (1) they provided sufficient stiffness to ensure recentering under the influence of P-Delta effect, and (2) the decompression moment at each connection was large enough such that the connection would not rock open during wind or gravity loading. The connection stiffness calculation determines the minimum number of strands for a given beam depth required to ensure that the connection stiffness, when reduced by P-Delta effects, remains positive. The decompression moment calculation then provides an initial post tensioning force,  $T_o$ , required for that given beam depth.

To ensure recentering at the 10/50 hazard level and collapse prevention at the 2/50 hazard level, the HBEs were designed to remain mostly elastic during a 2/50 seismic event. Since the ultimate axial load in the HBE is dependent upon the elongation of the PT tendons caused by the connection gap opening, a maximum gap angle,  $\theta_r$ , must be assumed. In the conservative case of rigid HBEs and VBEs with no initial post-tensioning,  $\theta_r$  is equal to column drift,  $\theta_c$ ; therefore, the maximum  $\theta_r$  was assumed to be 4%, corresponding to the maximum drift assumed at the 2/50 hazard level. The VBEs were then design using capacity design methods (Berman and Bruneau 2008).

To further ensure collapse prevention at the 2/50 hazard level, the PT tendons were also designed to remain elastic up to the predetermined 4% drift level. The PT tendons used in this design were 1/2" diameter seven-wire strands made with ASTM A416 Grade 270 steel, with a yield stress,  $F_y$ , of 245 ksi. It is important to note that for the case of HBE rocking about its flanges, the layout of the PT tendons within the beam depth does not affect the connection behavior, as long as the tendons are placed symmetrically about the centroid of the beam. However, when checking for PT yielding it is important to that the maximum stresses occur in the extreme tendons and must be determined from the tendon layout and the gap angle.

For the purposes of this prototype design, the gravity loads carried by the HBE were assumed to be negligible compared to the seismic loads and were not considered. Table 2 shows

the resulting R-SPSW design.

Table 2. Prototype R-SPSW

Story	Web Plate Thickness (in)	HBE Size	VBE Size	# of PT Strands	Initial PT Tension (kip)
3	0.1345	W18x158	W24x370	10	34
2	0.25	W24x176	W24x370	12	48
1	0.25	W24x192	W24x370	16	80

### Analytical Model

The preliminary design was modeled in OpenSees. The steel infill plates were modeled using a series of tension-only truss elements with a hysteretic uniaxial material that characterized the pinched cyclic behavior of web plates in tension field action. The plate material had a yield stress of  $R_y F_y$ , where  $F_y$  was 36 ksi and  $R_y$  was 1.3 to account for the ratio of expected yield strength to specified yield strength of A36 plate steel (AISC 2005). After yielding, the material was assigned 2% isotropic strain hardening until an ultimate strain of 20 times the yield strain, after which the steel acted effectively perfectly plastic with a small strain hardening ratio of 0.2% in order to avoid zero-stiffness convergence problems. The angle of inclination, cross-sectional area, and spacing of the strips were calculated per AISC Design Guide 20 (Sabelli and Bruneau 2007) and were based off of the center-line dimensions of the frame. The strips were laid out symmetrically to allow for tension field action to develop in both directions of loading (See Fig. 2).

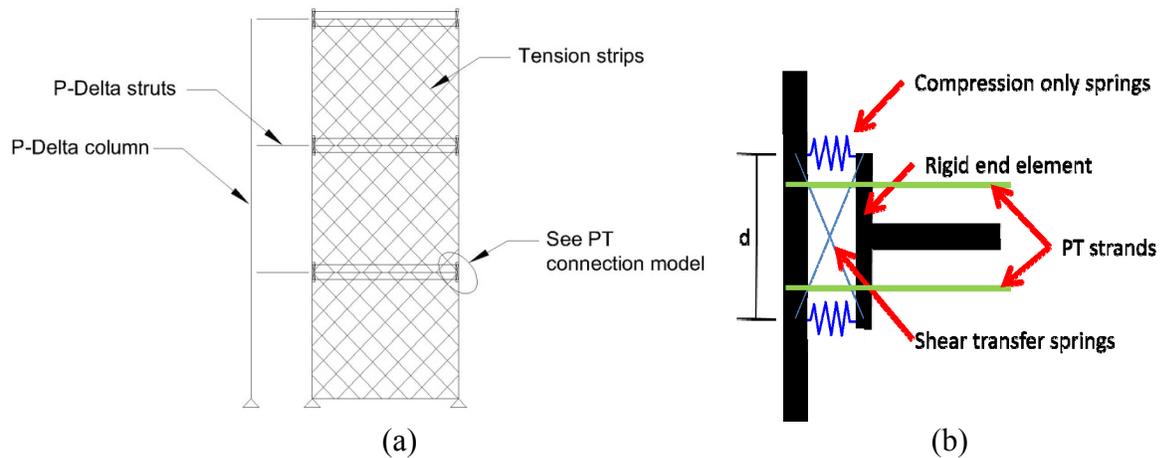


Figure 2. (a) Schematic of the R-SPSW model (b) Post-tensioned connection model

A PT connection model was developed to allow the HBE to rock about its flanges and cause PT elongation and to allow for shear load transfer without inhibiting the rocking action. The PT connection model consisted of (1) very stiff horizontal compression only springs located at the HBE flanges to allow for decompression of the flanges at gap opening, (2) stiff compression only diagonal springs to transfer shear forces while allowing for gap opening and rotation, and (3) a stiff elastic beam element connecting the end of the HBE to the connection springs to ensure that the end of the HBE in contact with the VBE remains planar and is orthogonal to the HBE's longitudinal axis at that point. The tension only truss elements used for the PT tendons were connected to the VBEs and were placed within the depth of the HBE (See

Fig. 2). The initial post-tensioning stress was obtained with the Steel02 material in OpenSees (Mazzoni et al. 2006), which allows for an initial stress value. The PT connection model was developed previously without the presence of steel infill plates and compared to existing research on post-tensioned beam-to-column connections to validate the modeling techniques (Garlock et al. 2005).

The boundary frame elements were modeled using nonlinear beam-column elements with fiber cross-sections. The fibers were modeled using the Giuffre-Menegotto-Pinto material model, with an effective yield stress of 50 ksi, a rounded yield surface to help with numerical convergence, and 2% strain hardening. Shear deformation in the panel zones was ignored since it was assumed that the VBE webs would be adequately reinforced in these areas. A P-delta column was also included in the model to simulate the gravity loads (1.0 DL +0.5 LL) from the leaning columns of the building that would contribute to P-delta effects on the wall. The seismic mass attributed to the wall was modeled as lumped masses at the beam-to-column joints at each story.

### Dynamic Analysis and Preliminary Results

The nonlinear model was subjected to three suites of 20 ground motions representative of ground motions at different hazard levels for the selected site; one suite approximated design basis earthquakes (DBE) with a 10% probability of exceedance in 50 years, the second suite approximated maximum credible earthquakes (MCE) with a 2% probability of exceedance in 50 years, and the third suite approximated frequent earthquakes with a 50% probability of exceedance in 50 years. Additional zeros were appended to all ground motions to allow for free vibration decay and to capture any residual deformations of the structure.

The response parameters that were evaluated were the peak story drift,  $\theta_{s,max}$ , the residual roof drift,  $\theta_{resid}$ , and a measure of maximum frame element damage, referred to herein as the damage value,  $D_{VBE}$  and  $D_{HBE}$  for VBEs and HBEs respectively. The frame element damage was measured by evaluation of the interaction equation per AISC Specifications Eqn. H1-1a (AISC 2005b). A damage value greater than or equal to one indicates full plastic hinges forming in the member, and values approaching one may also have some yielding occurring in the section.

Table 3. Median Response Parameter Values for the Prototype R-SPSW

Response Parameters	Hazard Level		
	50/50	10/50	2/50
$\theta_{s,max}$	0.41%	0.84%	1.45%
$\theta_{resid}$	0.0005%	0.0006%	0.019%
$D_{VBE}$	0.20	0.40	0.60
$D_{HBE}$	0.54	0.81	0.97

Median results for residual and peak story drift and HBE and VBE damage from the response history analyses of the preliminary prototype R-SPSW are shown in Table 3. As shown, the median residual drift for both the 50/50 and 10/50 ground motions are essentially zero, demonstrating that the system is indeed recentering as intended. Further the median value of peak story drift is below 0.5% for the 50/50 earthquakes, which should ensure that the web plates

remain elastic for such frequent earthquakes. The median peak story drift for the 10/50 earthquake was less than 1% which achieves the performance objective of limiting drift to 2% for that hazard level. Median VBE and HBE damage values are also less 1.0 for all hazard levels; however, some HBE yielding is expected for ground motions consistent with the 2/50 hazard level as the median damage value is 0.97. In summary, the prototype R-SPSW achieved the performance objectives.

### Future Development of the R-SPSW

The continuing development of the R-SPSW system as part of this project includes the development of PT connection details and compatible web plate details, such as corner cutouts, that will be tested at the University of Washington. The test setup is capable of producing appropriate boundary conditions for one HBE with PT connections on each end and web plates either both above and below or just below the HBE. The primary goal of the connection testing is to study the interaction between the tension field action in the web plates and the PT connections, and their impact on HBE demands. The results of the component tests will then be used to select details for a three-story, 1/3 scale, specimen that will be used for quasi-static and shake table tests at the NEES facility at the University at Buffalo, as shown schematically in Fig. 3a. A full-scale 3-story specimen will then be tested at the National Center for Earthquake Engineering Research in Taiwan. Finally, the analytical work described above will be expanded to include a parametric study to investigate the impact of the many design parameters on system performance.

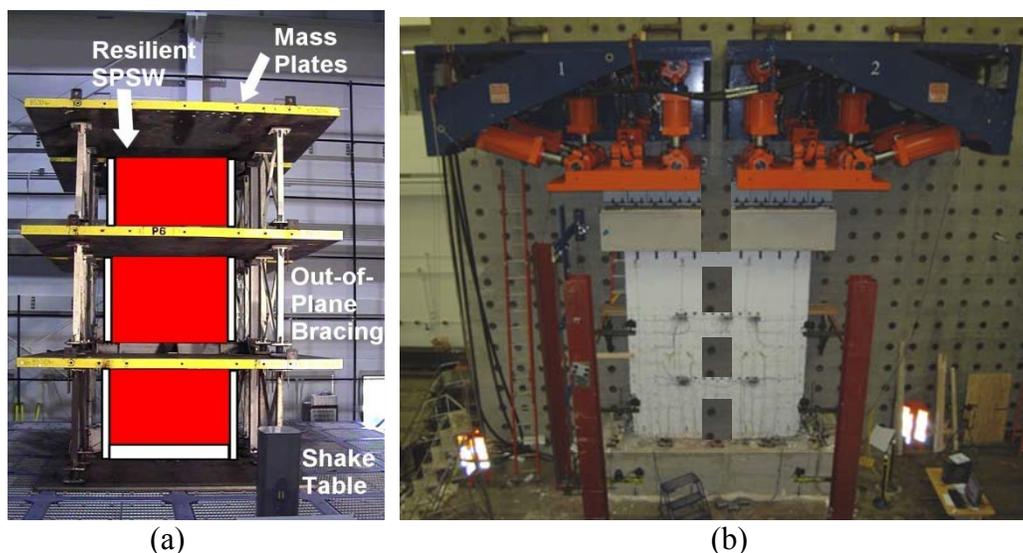


Figure 3. (a) Schematic of the Shake Table Tests of the Resilient SPSW, (b) Coupled Concrete Shear Wall at MUST-SIM, a Similar Configuration will be Used for Coupled SPSW Tests.

### Addressing Knowledge Gaps in SPSW Behavior

This project is also addressing critical knowledge gaps in the seismic behavior of SPSW. There are three primary areas of focus: (i) tools for performance-based design, (ii) expedient and accurate modeling techniques, especially for high-rise steel plate shear walls, and (iii) the behavior and design approach for coupled steel plate shear walls.

A database of all previous thin SPSW experiments that have been performed has been assembled and is being used to develop rational damage states and fragility functions for use in performance-based design. The methods and procedures are consistent with those being used in the ATC-58 project and the results will be useful to engineers seeking to design structures for specific performance requirements. The resulting damage states, fragility data, and descriptions of the research programs used to generate the information will be published for the engineering community in archival journals and submitted to the ATC-58 project team. See the project website at <http://depts.washington.edu/spsw/index.html> for additional details.

The results of previous research as well as experimental data generated from this study will be used to develop an infill panel model that is more efficient than the strip model, but preserves the strip model's accuracy. The four primary design objectives for this new model are 1) accurate simulation of the global response of SPSW infill panels under the range of cyclic load histories that develop in typical SPSWs, 2) efficient model-building, including objective procedures for calibration that are based on panel geometric, material and design properties, 3) a simple model formulation, and 4) portability to commercial software for nonlinear analysis of structural systems such as SAP2000, PERFORM3D ([www.csiberkeley.com](http://www.csiberkeley.com)), or ABAQUS ([www.abaqus.com](http://www.abaqus.com)). The initial model development activities will employ MATLAB ([www.mathworks.com](http://www.mathworks.com)); ultimately a new element will be developed for use with OpenSees, the simulation component for NEESit, and made available to the community. This part of the project is currently underway although no preliminary results are available at the time of this writing.

Architectural demands often result in core-wall systems in the interior of buildings with doorways to accommodate access to elevators and stairways. These openings require coupled walls in which slender walls are coupled by coupling beams. To date, the only research addressing coupled SPSW systems was a project-specific experimental investigation (Asteneh-Asl and Zhou 2001). Despite the good performance of the specimen the results of this study do not provide complete understanding of the seismic response of coupled SPSW systems. It should be noted that for coupled concrete walls the desired yield mechanism consists of flexural yielding in coupling beams and at the base of the walls, while in coupled SPSW systems, yielding of coupling beams and web plates at levels other than the base is desired. This difference in the inelastic mechanisms for coupled SPSW versus concrete walls prevents extension of the body of research results for concrete systems to these steel systems. This project will also be investigating the impact on system behavior of the: 1) relative strength and stiffness of coupling beams, 2) sequence of yielding (i.e., coupling beam yielding then web plate yielding, or vice-versa), and 3) distribution of that yielding up the entire height of the building. In support of this, experiments on coupled steel plate shear walls will be performed at the NEES MUST-SIM facility at the University of Illinois, which is shown in Fig. 3b with a coupled concrete wall being tested.

### **Summary and Conclusions**

An overview of an ongoing NEESR project that is developing a self-centering SPSW and also filling critical knowledge gaps in SPSW behavior has been provided. A preliminary design procedure for the self-centering SPSW that uses explicit performance targets was presented and

used to design a prototype three-story resilient SPSW with post-tensioned beam-to-column connections. Analytical modeling of the prototype structure in OpenSees was discussed. Nonlinear response history analysis demonstrated that the prototype structure achieved all performance objectives, including elastic behavior for the 50% probability of exceedance in 50 year earthquake hazard, recentering in for the 10% probability of exceedance in 50 year hazard, and collapse prevention for the 2% probability of exceedance in 50 year hazard. Additional ongoing research activities involving testing of resilient SPSW components and systems were described. Finally, ongoing research activities aimed at addressing the critical knowledge gaps in SPSW behavior of: (i) tools for performance-based design, (ii) expedient and accurate modeling techniques, especially for high-rise steel plate shear walls, and (iii) the behavior of and design approach for. coupled steel plate shear walls, were also described.

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