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SEISMIC COLLAPSE ANALYSIS OF STIFFENED STEEL PLATE SHEAR WALLS

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Abstract: Steel plate shear walls (SPSWs) are lateral load resisting system consisting of vertical thin steel infill plates connected to the surrounding boundary elements. They are constructed with and without stiffeners. Unlike unstiffened SPSWs, limited research is available on stiffened SPSWs. This research work focuses on seismic collapse analysis of stiffened SPSWs. A new constitutive material model to simulate the behavior of stiffened steel plates is developed in this research. The newly developed model takes into account cyclic degradation in strength and stiffness under repeated reversed loading. The accuracy of the developed model is verified against the available experimental results. This paper also compares performances of stiffened and unstiffened steel plate shear walls. A total of four multi-storey SPSWs (3-, 5-, 7-, and 10-storey) are considered with panel aspect ratio of 1.39. Pushover and cyclic analyses are performed on all the SPSW models with and without stiffeners. The results indicate that stiffened SPSWs can provide higher initial stiffness and higher strength compare with unstiffened SPSWs. Finally, to evaluate the effects of the stiffeners in the overall performance of the SPSWs, incremental dynamic analyses were performed on all models. Median collapse capacity, which is expressed as S_a (T1, 5%) in this study, is extracted and considered as a benchmark for comparison between stiffened and unstiffened SPSWs.

1 INTRODUCTION

In the past three decades, unstiffened steel plate shear walls (SPSWs) have been used as a primary lateral load carrying system in a number of buildings in Japan and United states. The SPSW resembles to a vertical plate girder and is placed along the height of the building. In SPSW, thin infill plate is connected to surrounding boundary members (beams and columns) using fillet welds or bolts and acts similar to the web of the plate girder. The columns, which are known as vertical boundary elements, perform like the flanges of the plate girder, while the beams act like the stiffeners installed on the web of the plate girder. The main difference between a SPSW and a plate girder is that columns in SPSW carry a significant amount of axial loads. This issue makes the SPSW more complicated compare with plate girder. An unstiffened SPSW buckles elastically in the early stages of loading and tension field action forms in infill plate to resist applied loads. Buckling of infill plate in elastic region can decrease initial elastic stiffness as well as energy absorbance in the system. To solve this issue, thin infill plate can be stiffened or thick infill plate can be implemented instead of thin one to prevent elastic buckling in order to increase the system's initial stiffness as well as energy absorbance capacity (Sabouri and Sajjadi 2012). Thick infill plates are not practical and have many disadvantages such as more costs, higher demands on boundary elements, and higher weight, which results in larger seismic forces applied on the system. In order to avoid these disadvantages, stiffened plate can be used instead of thick plate to enhance the overall performance of the system. By stiffening the infill plate sufficiently, local buckling of infill plate occurs in subpanels instead of overall buckling in thin infill plate which postpone the buckling of infill plate and increase the tolerable compressive stresses acting on infill plate. By increasing the maximum tolerable compressive stresses in infill plate, demand on boundary elements (beam and columns) due to the tension field action decreases. This reduction can result in smaller cross section required for boundary elements supporting stiffened infill plate.

In the current study, seismic performance of stiffened SPSWs are investigated and compared with unstiffened SPSWs. To do this, a new constitutive material model is developed to model stiffened SPSW. The accuracy of the newly developed material model is verified against available experimental results. Several SPSWs ranging from 3- to 10- storey with and without stiffeners were designed according to the design guidelines of National Building Code of Canada (NBC) and CSA S16-14. Nonlinear static and dynamic analyses of the designed stiffened and unstiffened SPSWs were conducted.

2 TYPES OF BUCKLING IN SPSW SYSTEMS

Infill plate under pure shear can buckle in two modes: (1) global buckling and (2) local buckling. Stiffeners are designed in such a way that local buckling occurs prior to global buckling. In order to force infill panel to buckle in subpanels (local buckling mode), a minimum moment of inertia is required for horizontal and vertical stiffeners. Global buckling occurs when the stiffeners are very slender (low moment of inertia). The following two sections will address on finding shear stress in infill plate correspond to onset of global and local buckling in steel plate shear wall using classical buckling theory.

2.1 Global buckling

Critical shear stress corresponds to onset of global buckling can be derived by the assumption of orthotropic stiffness. This means that the stiffened SPSW can be considered with two different stiffness in two global directions. The critical shear stress for global buckling, τ_{crg}, can be calculated using following equations. (Eq. 1 to Eq. 3) (Timoshenko and Gere 1961, Sayed-Ahmed 2001):

[1] $\tau_{crg} = \frac{K_g\,\pi^2}{d^2\,t}\left(D_x^{0.75}\,D_y^{0.25}\right)$

[2] $D_x = \frac{E\,I_x}{s_x} + \frac{E\,t^3}{12\,(1-\vartheta^2)}$

[3] $D_y = \frac{E\,I_y}{s_y} + \frac{E\,t^3}{12\,(1-\vartheta^2)}$

where d is the panel's height; I_x and I_y are the stiffener's moment of inertia about x-axis and y-axis, respectively; t is the plate thickness; ϑ is the Poisson's ratio of steel plate; E is the modulus of elasticity; s_x and s_y are the stiffeners spacing in x and y direction respectively; K_g is the global buckling factor that is the function of D_x, D_y, s_x, and s_y as well as plate boundary condition. The minimum value of K_g for plate to frame connection of pinned and rigid is 3.64 and 6.9, respectively. In order to have a conservative design, the value of τ_{crg} is calculated by the assumption of pinned connection of plate to frame since this assumption result in smaller value for onset of global buckling in the infill plate.

2.2 Local buckling

The critical shear stress corresponds to onset of local buckling in sub panel can be derived using Eq. 4 (Galambos 1998):

[4] $\tau_{crl} = \frac{k_s\,\pi^2\,E_s}{12\,(1-\vartheta^2)}\left(\frac{t}{s_x}\right)^2$

k_s is local buckling coefficient which is a function of panel's aspect ratio and is calculated using Eq. 5:

[5] $k_s = \begin{pmatrix} 5.6 + 8.98\left(\frac{s_x}{s_y}\right)^2 & if\ \left(\frac{s_x}{s_y}\right) \geq 1 \\ 8.98 + 5.6\left(\frac{s_x}{s_y}\right)^2 & if\ \left(\frac{s_x}{s_y}\right) \geq 1 \end{pmatrix}$

The values obtained for τ_{crg} and τ_{crl} must be less that shear yield stress of infill plate (τ_{sy}). Shear yield stress can be calculated using Von Mises yield criterion.

2.3 Minimum required moment of inertia for stiffeners

To make sure that local buckling in sub panels occurs prior to global buckling of infill plate, following equation must be satisfied:

[6] $\tau_{crg} > \tau_{crl}$

By substituting Equations 1 to 4 in Equation 6, the following Equation can be derived:

[7] $\frac{K_g\,\pi^2}{d^2\,t}\left(\frac{E\,I_x}{s_x} + \frac{E\,t^3}{12\,(1-\vartheta^2)}\right)^{0.75}\left(\frac{E\,I_y}{s_y} + \frac{E\,t^3}{12\,(1-\vartheta^2)}\right)^{0.25} > \frac{k_s\,\pi^2\,E_s}{12\,(1-\vartheta^2)}\left(\frac{t}{s_x}\right)^2$

Equation 7 provides the minimum required moment of inertia for stiffeners to ensure that local buckling occurs prior to global buckling. To simplify Eq. 7, vertical and horizontal stiffeners are assumed to be equally spaced and have an equal moment of inertia. Eq. 8 represents the simplified form of the Eq. 7.

[8] $I > 0.916\left(\frac{k_s\,d^2}{s\,K_g} - s\right)t^3$

Other requirement for this study is that the spacing between horizontal and vertical stiffeners should be determined in such a way that shear yielding of sub panels happens prior to local buckling. This is done since this study concentrates on a fully stiffened SPSW condition. To ensure shear yielding, the following equation must be satisfied:

[9] $\tau_{crl} = \tau_{sy}$

By assuming $s_x = s_y = s$, minimum spacing required for stiffeners to guarantee shear yielding occurrence in sub panels can be derived using Eq. 10:

[10] $s_{min} = 4.775\,t\,\sqrt{\frac{E_s}{\sigma_y}}$

where σ_y is the yield strength of the steel infill plate; E_s is the modulus of elasticity for steel infill plate.

A strip model was developed for each specimen in Opensees software. In Opensees model, strips are defined as series of parallel truss elements (pin ended) with equal width connected to surrounding boundary elements oriented in the direction of tension field action (*Figure 1*-a). Tension field angle is calculated as per Eq. (F5-2) of AISC 2010 and is a function of panel's height and width, geometric properties of boundary elements, and infill plate thickness. In strip model, instead of modelling horizontal and vertical stiffeners directly, the effect of fully stiffened SPSW system is considered in the stress-strain relationship adopted for each strip. In fully stiffened condition, compressive stresses acting on stiffened infill plate can reach to shear yield stress as shown in *Figure 1*-b where $\frac{\sigma_y}{\sqrt{3}}$ is shear yield stress; $\frac{\varepsilon_c}{\varepsilon_y}$ is the length of strain hardening region in infill plate; α determines the slope of degradation that is caused by plate tearing. On the other hand, compressive stresses acting on unstiffened infill plate are negligible and are calculated based on Eq. 1.

3 MODEL VALIDATION

Two one storey similar steel plate shear walls with and without stiffeners which were experimented by Sabouri and Sajjadi (2012) are chosen for model validation. All numerical simulations are conducted using Opensees framework (Mazzoni 2004). Displacement based fiber element is used to model boundary elements in which Steel02 material with Opensees default parameters has been assigned. Each strip is modeled using truss element. The associated material for truss element in both stiffened and unstiffened SPSW system will be discussed in the following two sub sections.

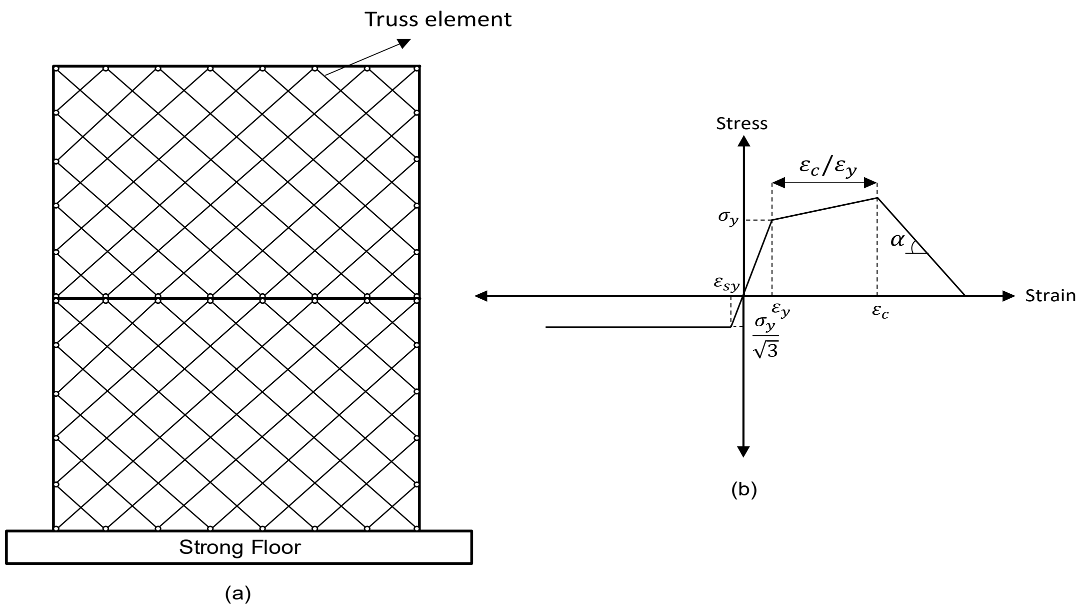


Figure 1: (a) Analytical cyclic strip model for SPSW system (b) Stress-strain relationship for strips

3.1 Unstiffened SPSW tested by Sabouri and Sajjadi (2012)

For the unstiffened SPSW, during the test, the first significant yielding occurred at the storey shear displacement of 1.7 mm (0.18% drift). The maximum load carrying capacity of the system was 789.6 KN, which happened at storey shear displacement of 39 mm (4.06% drift). In order to prevent zipping effects, four small stiffeners were installed at the corners of the infill plate. Web tearing in plate was developed initially in the triangular region surrounded by stiffener and boundary elements and then start to propagate out of the triangular region. Also, plastic hinges were developed at the top and bottom of the columns during the experiment. Infill thickness is 2mm and panel aspect ratio is 1.47, which can be considered as an average panel aspect ratio based on the lower and upper limits of 0.8 and 2.5, respectively in AISC (AISC 341-10). The specimen is shown in Figure 2-a.

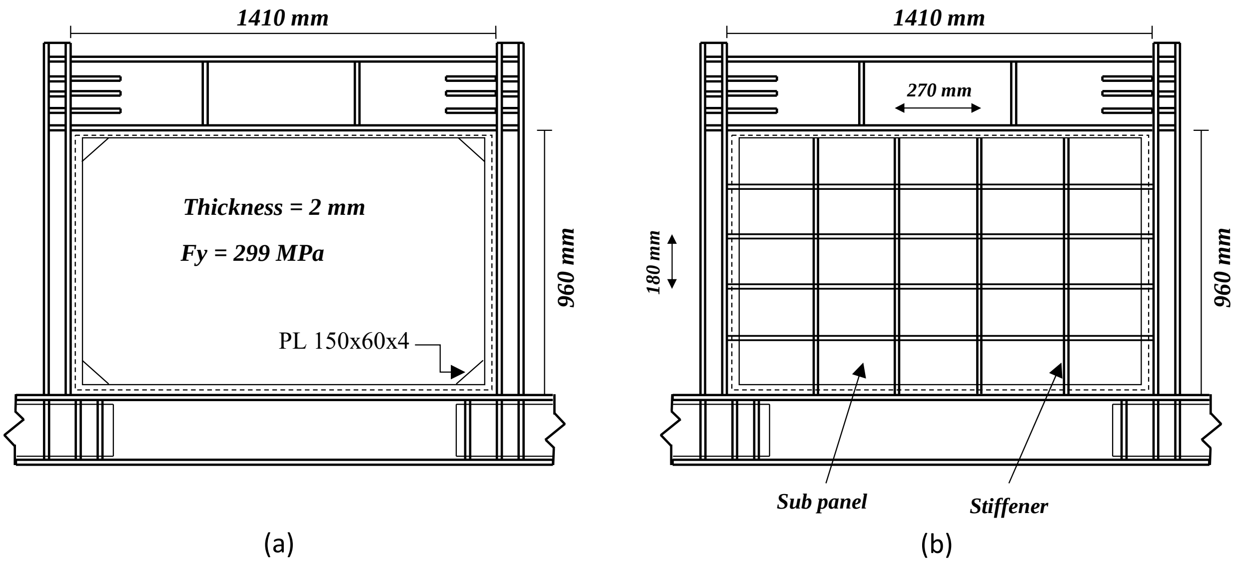
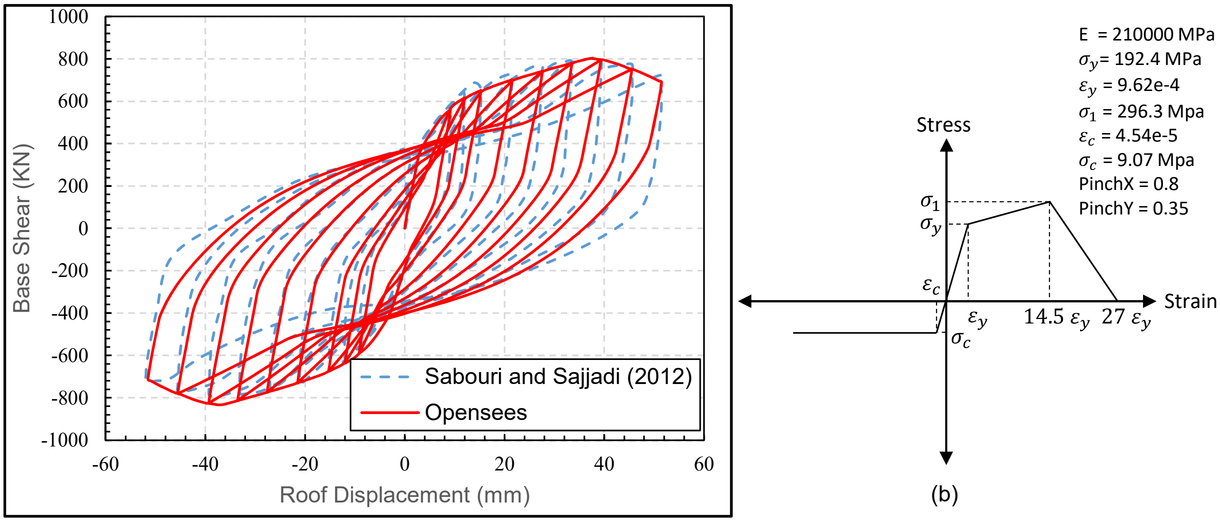


Figure 2: SPSW configurations tested by Sabouri and Sajjadi (2012) (a) Unstiffened (b) Stiffened

Figure 3 (a) presents the performance of the analytical model under cyclic loading, which agrees well with the experimental results. Thus, proposed material model can accurately capture post yielding behavior of the panel as well as the point associated with significant yielding in the system. Average pinching effect observed during experiment due to plate tearing is also captured by the new material model.

3.2 Stiffened SPSW tested by Sabouri and Sajjadi (2012)

During the test of stiffened SPSW, in the sixth cycle of loading, first significant yielding was observed at the storey shear displacement of 1.58 mm (0.16% drift). Maximum shear displacement of 808 KN was recorded at shear displacement of 34.05 mm (3.55% drift). The first minor tear was observed in one of the middle subpanels in 2.25% drift and by continuing cyclic loading procedure, more tearing was observed in other subpanels. By increasing storey shear displacement, tear in the middle sub panel is developed and propagated. At the end of the test, the steel plate shear wall strength decreased when the sub steel plates lost their continuity. The vertical boundary elements remained essentially elastic. It means that no local and global buckling observed in the columns during the test. In stiffened SPSW, significant compression forces are developed in addition to tension forces that are existed in unstiffened SPSW as well. Thus, presence of compression forces decrease the demand on boundary elements especially in columns. As reported by Sabouri and Sajjadi (2012), columns in stiffened SPSW remained essentially elastic while plastic hinges were developed at both ends of columns in the unstiffened SPSW. This is one of the main advantages of installing stiffeners on infill plate since it reduces flexural forces (demand) that are applied to boundary elements. Spacing between stiffeners and other geometric specifications for stiffened SPSW are depicted in Figure 2-b.



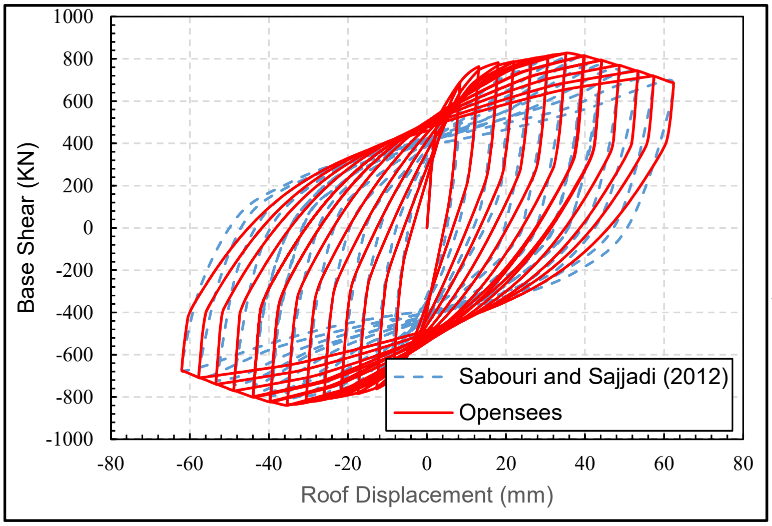
(a)

Figure 3: (a) Model verification against Sabouri and Sajjadi (2012) (b) Material model proposed for unstiffened SPSW

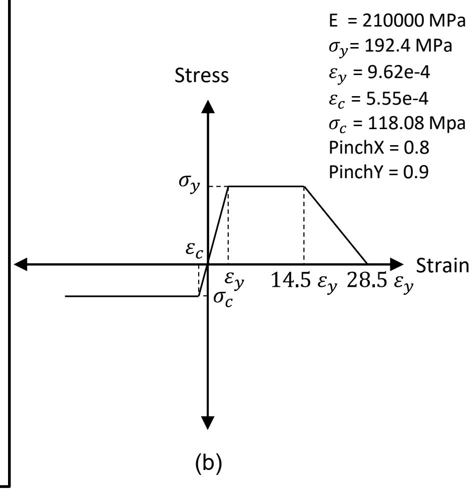
Figure 4 exhibits the cyclic behavior of analytical model developed for stiffened SPSW in which the proposed stress-strain relationship was assigned to strips. However, the analytical model overestimates the yield strength of the system. Slight pinching, capping point, and degradation backbone curve until the end of experiment is captured precisely.

4 DESIGN AND MODELLING SPECIFICATIONS

Four multi storey (3-, 5-, 7-, 10- storey) SPSW systems with and without stiffeners were designed according to the design guidelines given in CSA S16-14 and National Building Code of Canada (NBCC 2015). Boundary elements in SPSWs were designed according to capacity design approach that is proposed by Berman and Bruneau (2008). All buildings had a square plan of 30 mx30 m in which two SPSWs were resisting seismic loads in each direction. The buildings were assumed to be located in Vancouver (high seismic zone) (Figure 5-a). Equivalent static force procedure that is presented in National Building Code of Canada was implemented to calculate design base shear and seismic forces in each storey level (NBCC 2015). Minimum plate thickness of 2 mm was considered during the design of 3- and 5- storey SPSW systems. 3 mm plate thickness was assumed in first 2 and 5 stories of 7- and 10- storey SPSW systems, respectively. Remaining stories were designed with minimum plate thickness. A finite element model was developed to analyse the SPSWs. The FE model includes one SPSW and a leaning column (Figure 5-b).



(a)



(b)

Figure 4: (a) Model verification against Sabouri and Sajjadi (2012) (b) Material model proposed for stiffened SPSW

In Opensees software, elastic beam column element (EBC) was used to model leaning column. The cross sectional area of the leaning column was increased by 100 times of the largest cross sectional area available in the model for columns. In reality, the cross sectional area of the EBC should be multiplied by the total number of the gravity columns present in the model. However, this approach was not considered because the total number of gravity columns and their sizes will change from one model to another. Since the axial deformation of the gravity columns are not the concern of this study, multiplying cross sectional area of EBC by 100 will not affect the results (Purba and Bruneau 2015). Two rotational springs were located at top and bottom of each EBC and their rotational stiffness were set as a small value to apply the pin ended connections for each EBC. Tributary area and associated gravity loads for all the gravity columns present in the model were calculated and applied on leaning columns. NBCC 2015 recommends use of load combination '1.0D + 1.0E + 0.5L or 0.25S' (where D = dead load, L = live load, E = earthquake load, and S = snow load). Thus, load combination '1.0D + 1.0E + 0.5L' was used for the floors and load combination '1.0D + 1.0E + 0.25S' was used for roof. Truss elements were used in each storey level to connect the SPSW system to the leaning columns. Similar to EBC, cross sectional areas of truss elements were increased by 100 times of the largest HBE available in the model. This was done to simulate the rigid link condition in connecting steel plate shear wall to leaning column and avoid axial deflection of truss elements.

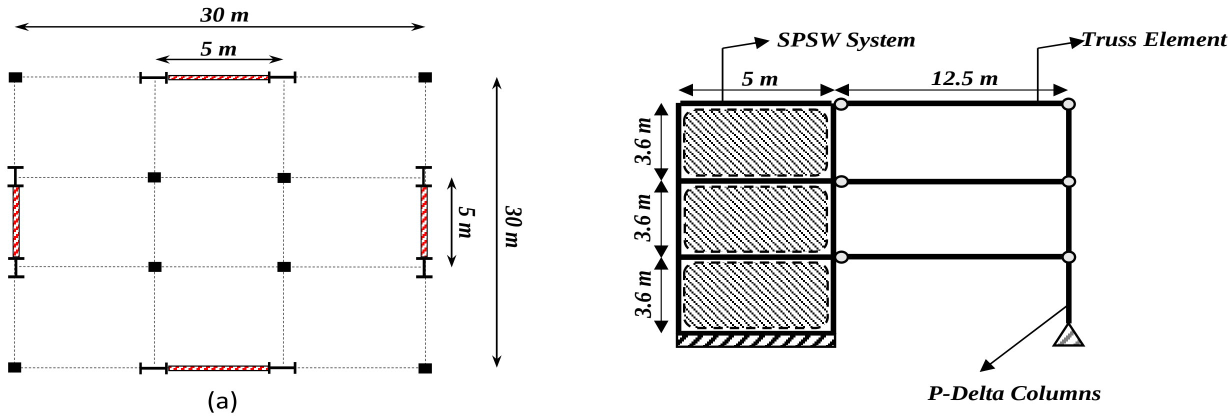


Figure 5: (a) Plan view of SPSW system (b) FE model for 3- storey SPSW

5 DISCUSSION ON ANALYSIS RESULTS

5.1 Static pushover analysis

Static pushover analyses are performed on all SPSWs and the results are extracted. For pushover analysis, normalized base shear and roof displacement are presented instead of absolute one. Base shear is normalized based on design base shear and roof displacement is normalized in regards to building's height. The most noteworthy result taken from pushover analyses is the overstrength provided in each building. Overstrength is defined as the maximum strength of the building taken from pushover analysis divided by design base shear. The overstrength calculated for 3-, 5-, 7-, and 10-storey SPSW systems are 4.671, 2.132, 1.761, and 1.505 for buildings without stiffeners, respectively and 5.16, 2.4, 2.031, and 1.774 for buildings with stiffeners, respectively (Figure 6). The large overstrength value obtained for 3 storey SPSW with and without stiffeners is attributed to the minimum available infill plate thickness of 2 mm that has been considered in this study. In practice, thinner infill plate thickness can be replaced with the minimum thickness considered in this study to decrease the overstrength in the system. Large value of overstrength obtained for stiffened and unstiffened 3-storey SPSW system will reduce the extent of nonlinearity experienced by the system during incremental dynamic analysis, which will be discussed in the subsequent section. To compare the effects of stiffeners in each building, a limit state should be defined to interrupt the pushover analysis when the corresponding limit state is reached. Based on FEMA P695, the displacement in which the shear strength of the system drops by 20% with regard to maximum shear strength can be considered as a benchmark for interrupting the analysis. Three values (i.e. initial stiffness, ultimate displacement, maximum base shear) are chosen as comparative parameters to investigate the effects of stiffeners on SPSW systems. Table 1 compares the three aforementioned parameters between stiffened and unstiffened SPSWs. By adding stiffeners to the system, shear strength of the system does not increase significantly (less than 20%), but the shear strength increases as the number of stories increase. Also, by adding stiffeners ultimate shear displacement increases significantly in 3- and 5-storey SPSW. It is also observed that by adding stiffeners, initial stiffness of the system will experience a sudden increase, which is close to 50% in 3-storey SPSW.

5.2 Incremental dynamic analysis

To more accurately compare the differences between stiffened and unstiffened SPSWs, incremental dynamic analyses were conducted on all the buildings using a suite of artificial ground motions (30 ground motions) that are developed for western Canada (Assatourians and Atkinson 2010). The analyses have been conducted using 5% damped spectral acceleration at structure's first-mode period (S_a (T1,5%)) as intensity measure and maximum interstorey drift as engineering demand parameter (EDP). The results are depicted for 3-, and 10-storey stiffened and unstiffened SPSW systems in Figure 7.

Low level of nonlinearity observed in 3-storey SPSWs especially in stiffened one are associated with minimum available thickness of 2 mm that is considered in this study for infill plate. This minimum thickness produces higher overstrength in the 3-storey SPSW.

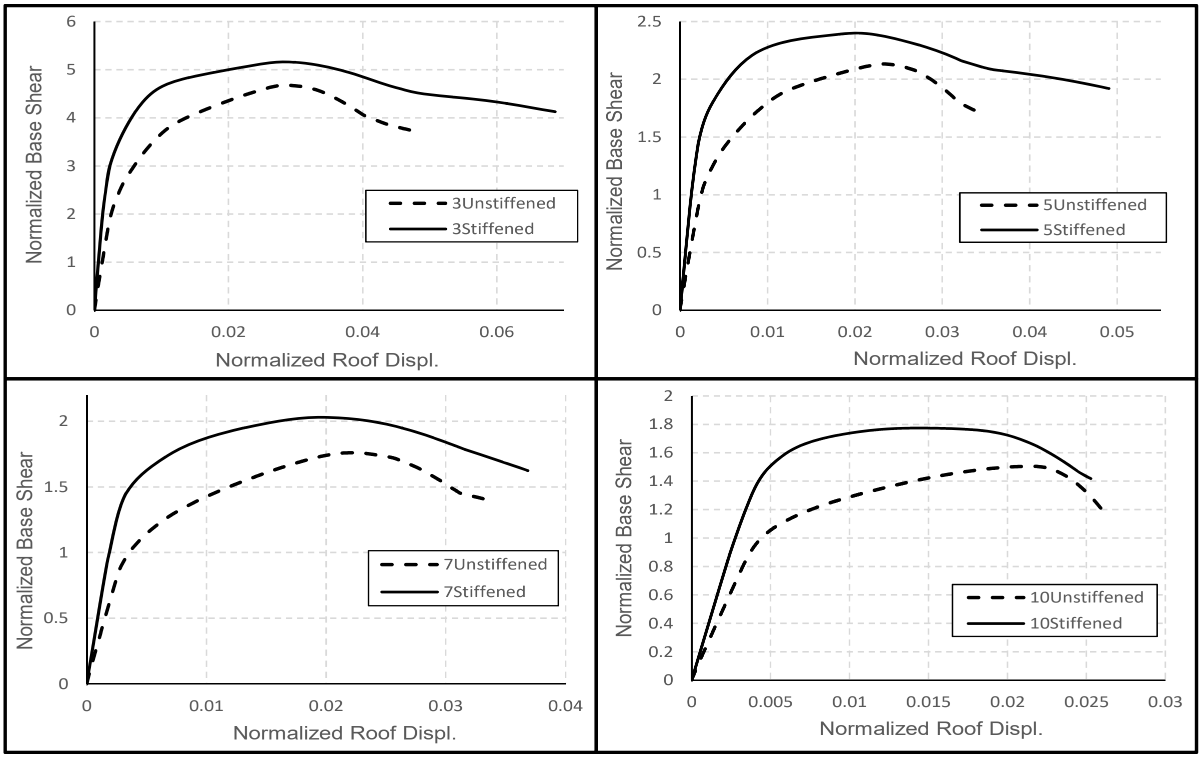


Figure 6: Pushover curves of SPSW systems

Table 1: Comparison the shear strength, ultimate displacement, and initial stiffness between stiffened and unstiffened SPSW with respect to stiffened SPSW

	3-storey	5-storey	7-storey	10-storey
$\frac{F_{max-S} - F_{max-U}}{F_{max-S}}$	9.5%	11.2%	13.3%	15.2%
$\frac{\Delta_{U-S} - \Delta_{U-U}}{\Delta_{U-S}}$	31.2%	29.3%	9.8%	-2.8%
$\frac{k_{initial-S} - k_{initial-U}}{k_{initial-S}}$	48.8%	47%	39.7%	31.7%

5.3 Probabilistic seismic collapse assessment

The probability of collapse can be derived by subjecting the structure to a suite of ground motions whose IM values are scaled to be equal or greater than specific values. Thus, for each ground motion, there is a specific value for IM that corresponds to onset of collapse in the structure. Fragility function parameters can be estimated by taking the logarithms of each ground motion's IM value associated with onset of collapse and calculate the mean and standard deviation. Fragility curves obtained for the selected SPSWs are depicted in Figure 8.

To compare the results, Sa corresponding to 50% probability of exceedance, which is known as median collapse capacity (MCC), is chosen as an indicator of structure's capacity. As shown in Figure 8 by red arrow, when stiffeners are added on infill plate, the capacity of SPSW building is significantly increased. The MCC increase in stiffened SPSWs is calculated as 38.3%, 26.4%, 32.95%, and 21.88% for 3-, 5-, 7-, and 10-storey, respectively.

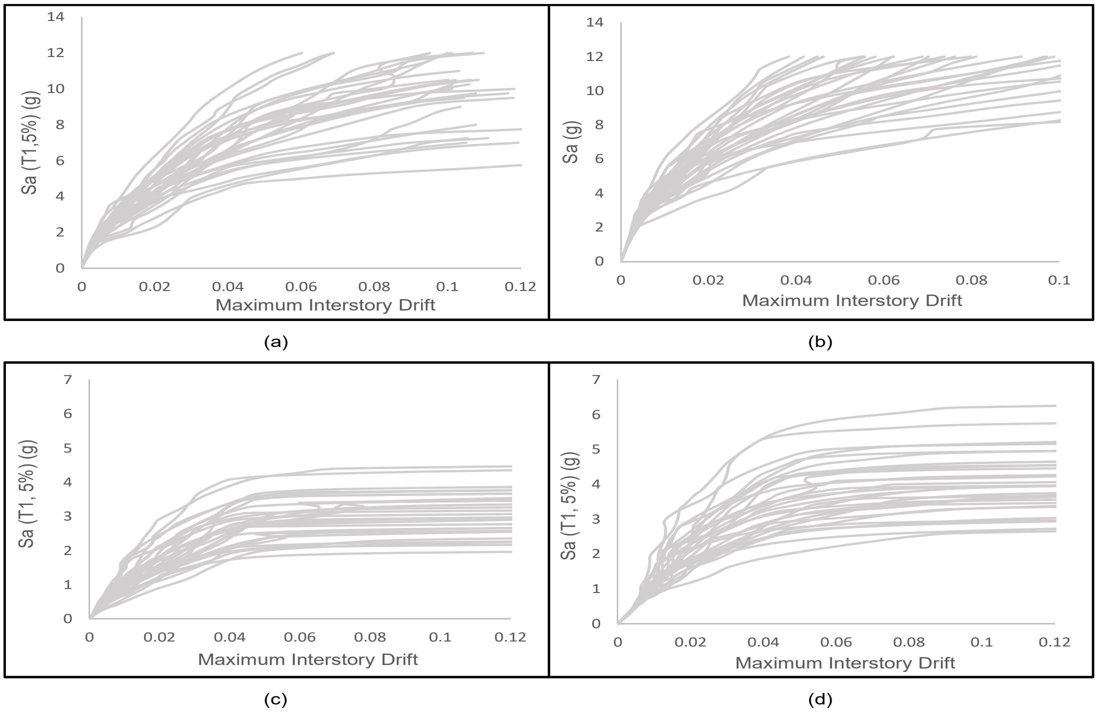


Figure 7: (a) 3-storey unstiffened (b) 3-storey stiffened (c) 10-storey unstiffened (d) 10-storey stiffened

6 CONCLUSION

In this study, the effects of stiffeners on overall performance of SPSW systems were investigated. Two types of common buckling modes (i.e. Global and local buckling modes) were discussed and the related equations were presented. To avoid global buckling prior to local buckling in stiffened SPSW systems, minimum moment of inertia for horizontal and vertical stiffeners were calculated. In addition, minimum spacing between stiffeners were calculated to ensure that shear yielding of fully stiffened SPSW system is occurred prior to global or local buckling of the system. Then, a material model was developed for stiffened SPSW, which is assigned to strips in the numerical model. The accuracy of the developed material model was investigated against available experimental results. A total of four multi-storey SPSWs (3-, 5-, 7-, and 10-storey) are considered with panel aspect ratio of 1.39 to evaluate the performance of stiffened and unstiffened SPSWs. Pushover analyses are performed on all the SPSW models with and without stiffeners. The results indicate that installing stiffeners improve the initial stiffness of the whole system as well as maximum strength. The maximum increase in initial stiffness was as much as 50% for three-storey SPSW system. The aforementioned four multi-storey SPSWs were subjected to IDA procedure. The results of performed IDA were used to derive collapse capacity of each system by choosing the median collapse capacity (MCC) as the criterion. The results indicate that adding stiffeners on infill plate shifts the fragility curve to right and cause a significant increase in MCC’s value.

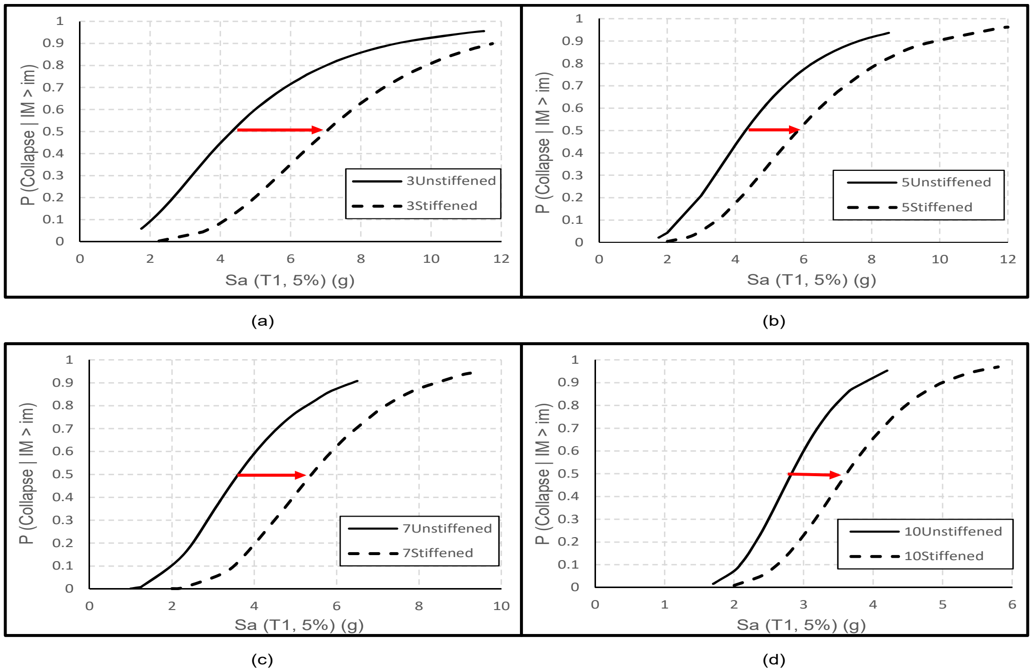


Figure 8: Collapse fragility curves for different buildings

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