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DEVELOPMENT OF A DETERIORATION MODEL FOR ANALYSIS OF UNSTIFFENED STEEL SHEAR WALLS

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Abstract: Collapse assessment of a structural system under seismic loading requires a reliable analytical model that can accurately capture deterioration in strength and stiffness of the main seismic fuse. Steel infill plates are considered as main ductile fuses of steel plate shear walls (SPSWs). A strength deterioration model is developed for infill plates of unstiffened SPSWs. The model is calibrated against several experimental results. Ductility capacity, post capping ratio, and energy absorbance coefficient are considered as variables in calibration process. Seismic response sensitivity of SPSWs to the variation of these parameters are assessed using two code-designed SPSWs. Eighteen (18) variant models are chosen for each SPSW considering different combination of varying parameters and are subjected to incremental dynamic analysis using a suite of ground motions. Sensitivity of median collapse capacity of SPSW, which is expressed in terms of Sa (T1, 5%), to the selected three parameters (ductility capacity, post capping ratio, and energy absorbance coefficient) are assessed. The results indicate that the capacity of SPSW is more sensitive to ductility capacity change, while the variation of energy absorbance coefficient has a minor effect on total performance of the system.

1 INTRODUCTION

Steel plate shear walls (SPSWs) generally are consist of columns and beams which are known as vertical and horizontal boundary elements and an infill plate that is the primary member in resisting lateral loads especially earthquake loads. SPSW systems can be constructed using either stiffened and thick or unstiffened infill plates. With the aim of reducing construction costs, unstiffened infill plates have been studied by different researchers (Timler and Kulak 1983, Bhowmick et al. 2010, Choi and Park 2010, Driver et al. 1997). Unstiffened steel plate shear wall buckles in the early stages of loading and tension field action develops in steel plate shear wall to resist lateral loads. Inelastic behavior in unstiffened SPSW system is thus mainly due to tension field action in infill plate. Different analytical models were developed to predict the nonlinear behavior of SPSW systems. One modelling technique, which is widely used to model steel plate shear walls, is strip model where the tension field action developed in the infill plate is represented by series of parallel tensile strips. This method is included in the Canadian design provision for SPSW (CSA 2001). In addition, this method is outlined in the Commentary to AISC 341. The CSA provision requires a minimum of 10 strips to model the web plate in order to approximate the effects of distributed loads on boundary elements. Second proposed method is an orthotropic membrane model, which is widely used for micro modelling. In this case, the material properties are assigned to be in the same direction with tension field angle. In essence, the membrane model is identical to strip model since both are based on developed tension field action in infill plate. Use of equivalent tension brace for infill plate is the third way to model SPSWs. This method is convenient for analysis and design, but it is less accurate method for the modelling of SPSWs since this method does not account the forces that are developed from tension field action and applied to the boundary elements.

In the current study, a stress-strain based relationship for strip model is developed to model infill plate. This model is capable of considering deterioration in infill plate as well as pinching effect due to web tearing. Capability of the developed stress-strain relationship to accurately model SPSWs is validated against available experimental results. Three parameters (ductility capacity, post cap stiffness, and energy absorbance coefficient) were varied during calibration process to best fit with experimental results. The sensitivity of SPSW systems to variation of these three variables is further assessed.

2 STRESS-STRAIN RELATIONSHIP ADOPTED FOR INFILL STEEL 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 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. Figure 1-b represents the general stress-strain relationship assigned to each truss element in numerical models. During calibration process, the value of three parameters, post cap stiffness, ductility capacity, and energy absorbance coefficient, in the adopted stress-strain relationship for strip model are varied to best fit with experimental results. The calibration process is explained in detail in the subsequent section. As depicted in Fig. 1, $\varepsilon_c/\varepsilon_y$ and α represents the ductility and post cap stiffness respectively. The last parameter is energy absorbance coefficient, which takes into account the pinching effect associated with SPSW system during the experiment. In Figure 1-b, τ_{cr} is the elastic buckling stress of an infill plate subjected to pure shear and can be calculated using Eq. 1 and Eq. 2.

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

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

where k_s is the buckling coefficient and is a function of infill plate aspect ratio.

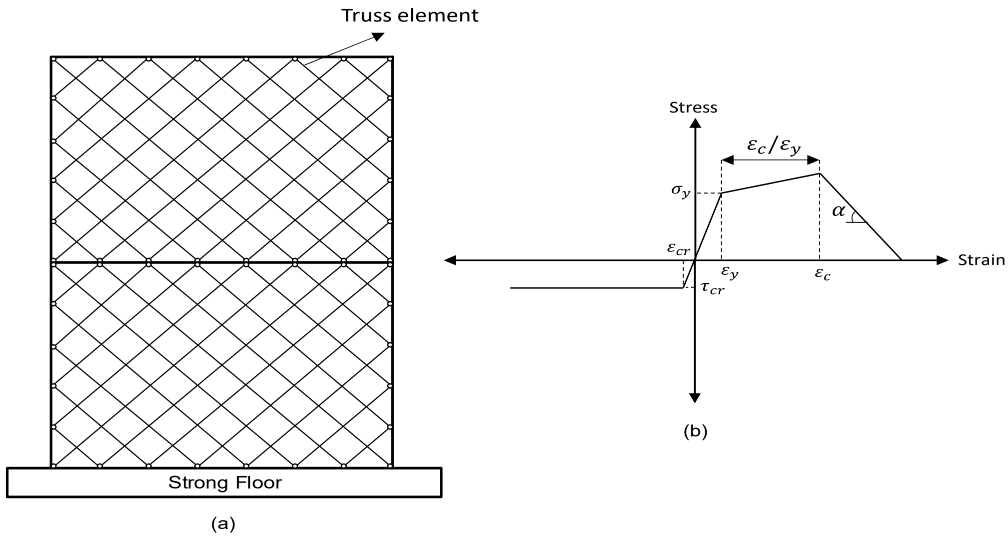

Figure 1: (a) Analytical cyclic strip model for SPSW system (b) Stress-strain relationship for strips

3 PROPOSED STRESS-STAIN RELATION FOR STELL INFILL PLATE IN SPSW

In order to estimate the three selected parameters for the proposed stress-strain relationship for infill plate, three experiments were chosen. In all three experiments, plate tearing was the main source of cyclic deterioration. All numerical simulations are performed using the framework of the Opensees software. Boundary members were modelled using displacement beam-column element in Opensees. Steel02 material was used to simulate cyclic strain hardening and Bauschinger effect in boundary elements. Cyclic parameters were set equal to the values obtained by Choi and Park (2008) for a moment resisting frame.

3.1 Single storey SPSW tested by Vian and Bruneau (2005)

The single storey specimen tested by Vian and Bruneau (2005) is depicted in Figure 2 and is chosen as the first specimen for the calibration purpose. The infill plate thickness was 2.6 mm and the panel aspect ratio (ratio of bay width to storey height) was 2.38, which is quite close to the specified limit (2.5) permitted by AISC 341-10. Details of this experiment can be found in the literature (Vian and Bruneau 2005).

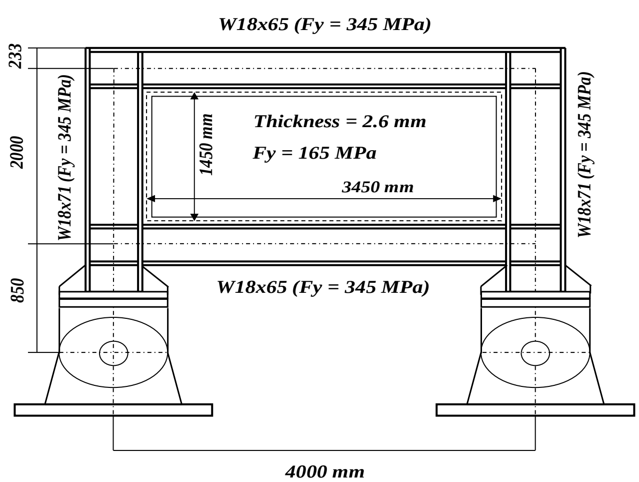

Figure 2: Model configuration tested by Vian and Bruneau (2005)

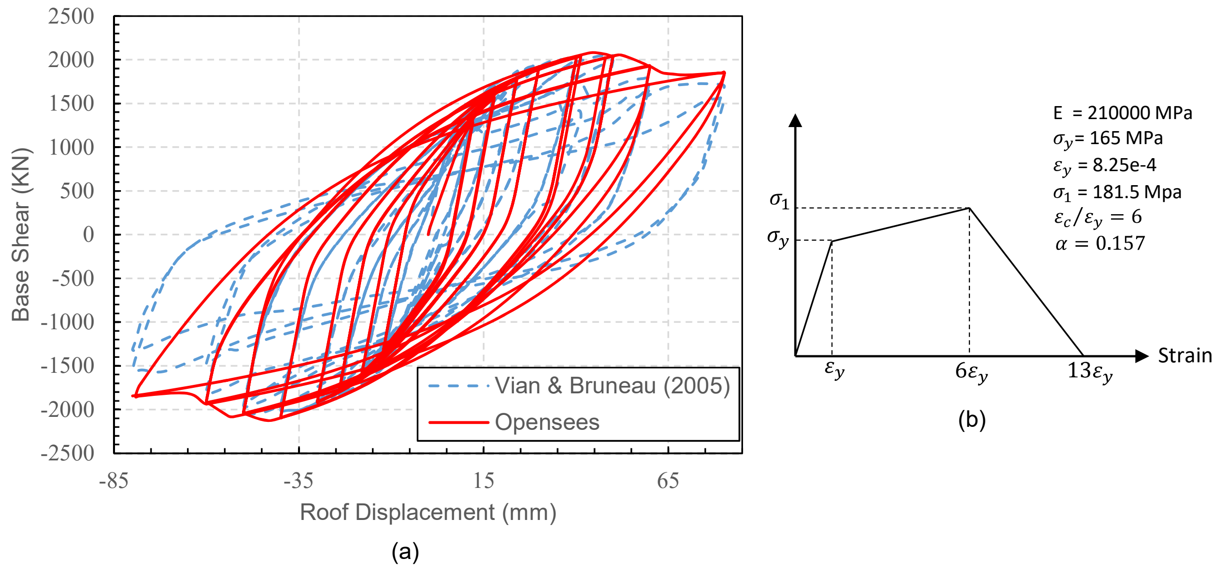

Figure 3: (a) Model validation against Vian and Bruneau (2005) (b) stress-strain relationship

Figure 3(b) represents the stress-strain relationship, which is assigned to all strips in the model. Nonlinear pushover analysis using the selected stress-stain relation indicates that there is a good agreement between analytical and experimental results (Figure 3-a). However, severe pinching in the experimental results as opposed to negligible pinching in the strip model is observed, which can overestimate the energy dissipation by the system during the test. Onset of yielding, capping point, and degradation backbone curve up to the end of the test are precisely estimated by the selected stress-strain parameters for the strip model.

3.2 Single storey SPSW tested by Sabouri and Sajjadi (2012)

Two single storey SPSWs with and without stiffeners were tested by Sabouri and Sajjadi (2012). The SPSW without stiffeners was used in this study. The specimen had infill plate thickness of 2 mm and the panel aspect ratio was 1.47, which can be considered as an average panel aspect ratio (Figure 4). The stress-strain relationship represents a slight degradation in comparison to the one developed for Vian and Bruneau (Figure 5). In addition, the length of strain hardening region is much higher than the test by Vian and Bruneau (2005). The main reason associated with significant strength degradation in Vian and Bruneau's experiment was the fracture of HBE to VBE connection, which caused a strength degradation of 24% from the ultimate strength of the specimen at 4% interstorey drift. For the other experiment, all the connections and boundary elements behaved in a stable manner and deterioration occurred due to web tearing. Since the fracture occurred in the connection of HBE to VBE prior to full yielding of tension field in the first experiment, the infill plate was not able to use its whole capacity. As shown in Figure 5-a, prediction provided by developed model agrees well with the experimental result.

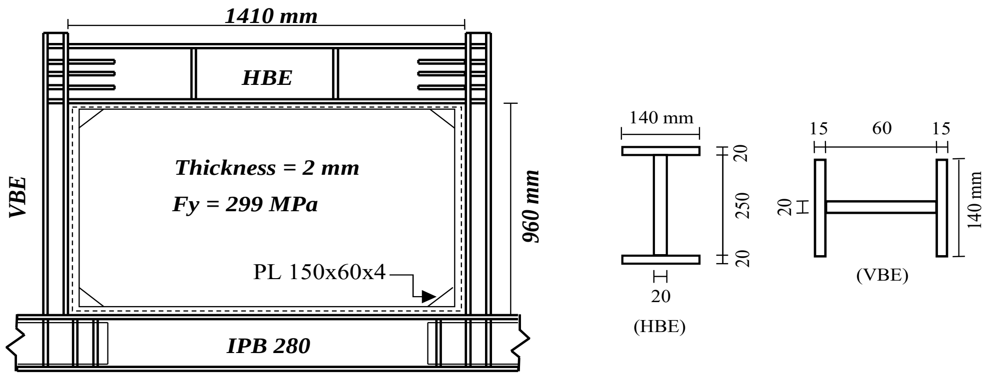

Figure 4: Model configuration tested by Sabouri and Sajjadi

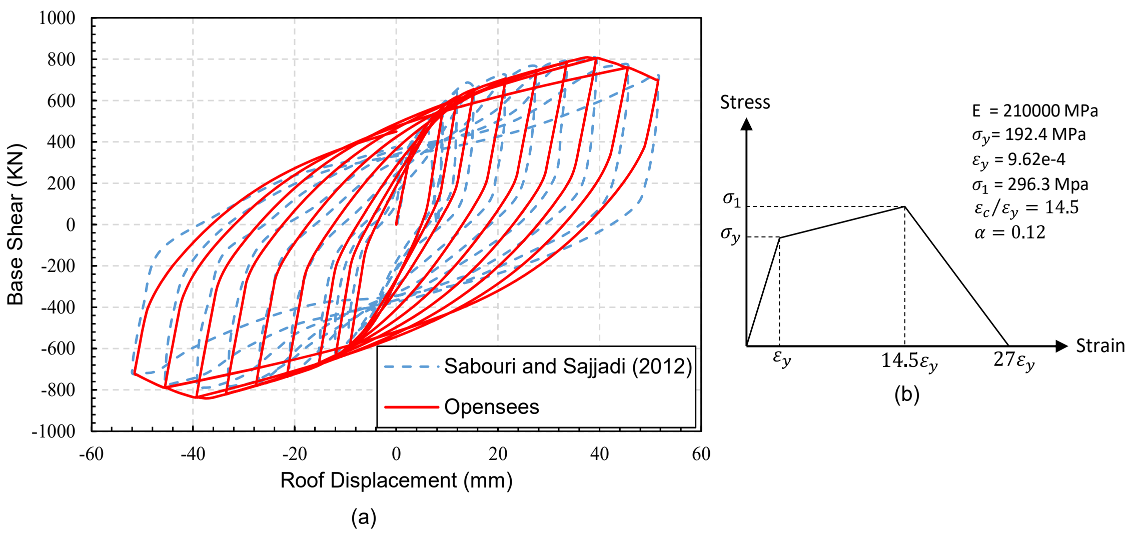

Figure 5: (a) Model validation against Sabouri and Sajjadi (2012) (b) Stress-strain relationship

3.3 Three storey SPSW tested by Choi and Park (2008)

The third specimen used in this study was a three storey steel plate shear wall tested by Choi and Park (Choi and Park 2008). Infill plate thickness was 4mm in all the stories and panel aspect ratio was 2.2, which can be categorized as large aspect ratio. Figure 6 provides details of the tested SPSW system. Based on the report given by Choi and Park (2008), extensive web tearing was observed in the second floor at the end of the experiment.

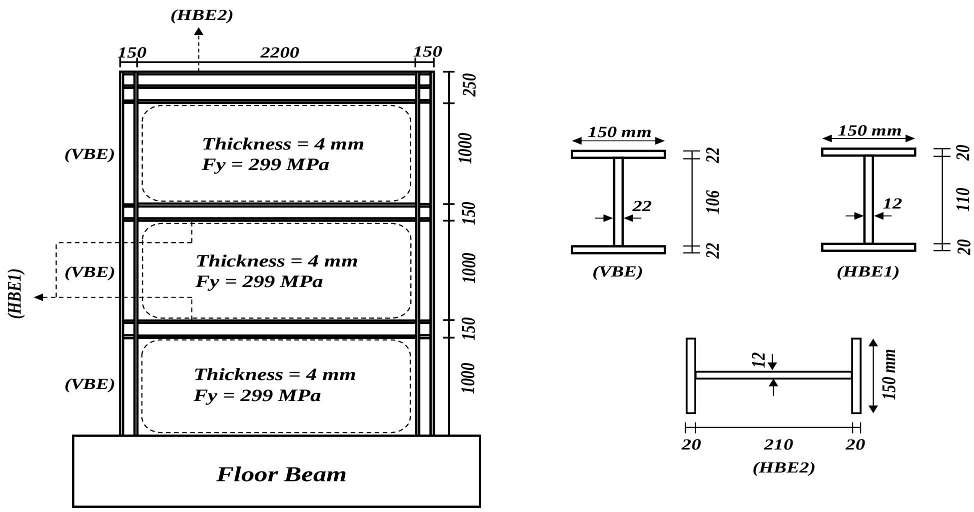

Figure 6: Model configuration tested by Choi and Park (2008)

Adopted stress-strain relationship for strips shows a good agreement between analytical and experimental results. As shown in Figure 7, onset of yielding as well as the maximum strength is captured by the strip model precisely.

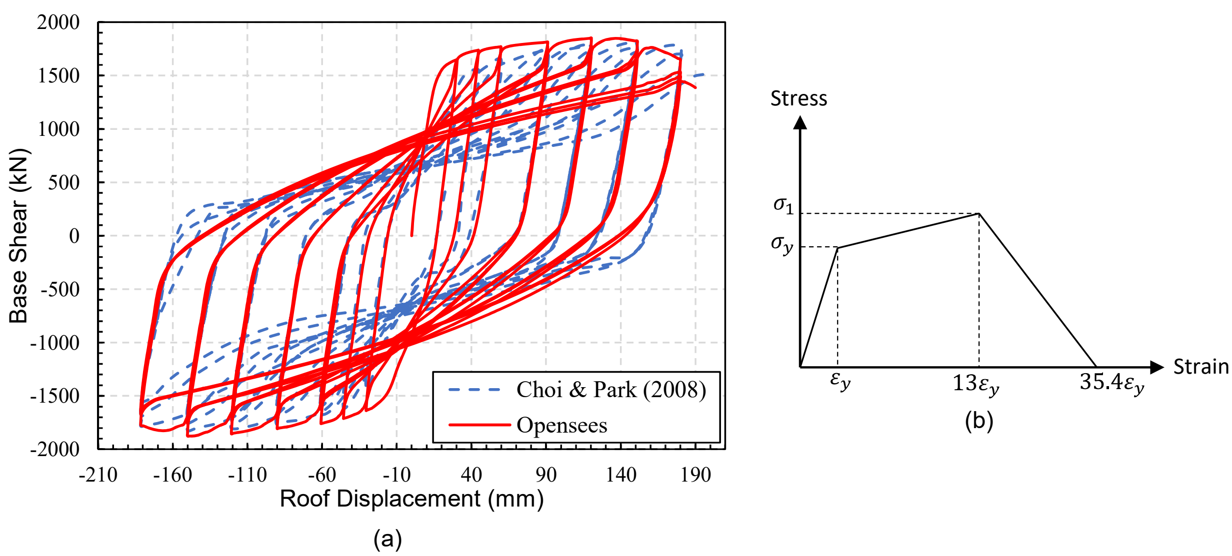

Figure 7: (a) Model validation against Choi and Park (b) Stress-strain relationship

4 SENSITIVITY ASSESSMENT STUDY

As discussed in section 2, three parameters (i.e. ductility capacity, post cap stiffness, energy absorbance coefficient) were varied in stress-strain relationship adopted for strips to obtain the best agreement with the test results. Thus, a study is required to assess the sensitivity of SPSWs' capacities when these parameters vary.

4.1 Seismic design of steel plate shear walls

In order to investigate how changing in each parameter affect the seismic performance of SPSW system, seismic analysis of two multi-storey SPSWs were conducted. Two hypothetical residential buildings (3- and 5- storey), which incorporate SPSW system as primary lateral load resisting system, were designed according to the design guidelines given in CSA S16-14 and National Building Code of Canada (NBC 2015). Table 1 provides designed boundary members with the assumption of 2mm constant thickness of infill plate in all stories. The buildings were considered to be situated in Vancouver, BC. Each building has a 30x30 symmetric square in plan. Two unstiffened steel plate shear wall systems were considered in each direction (Figure 8-a). A finite element model was developed to analyse the SPSWs. The FE model includes one SPSW and a leaning column (Figure 8-b).

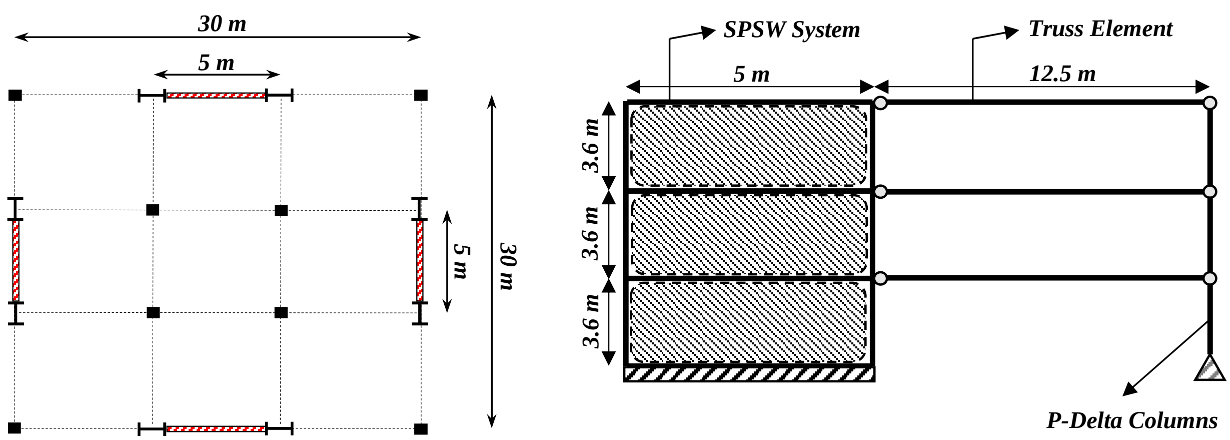

Figure 8: (a) Plan view of SPSW system (b) Analytical model for 3-storey SPSW along with leaning column concept

Table 1: Summary of steel plate shear wall frame member properties

	5-storey SPSW			3-storey SPSW	
Storey	Column sections	Beam sections	Storey	Column sections	Beam sections
1	W14 x 370	W12 x 50	1	W14 x 342	W12 x 50
2	W14 x 398	W12 x 50	2	W14 x 342	W12 x 50
3	W14 x 426	W12 x 50	3	W14 x 283	W14 x 176
4	W14 x 398	W12 x 50			
5	W14 x 342	W14 x 176			

4.2 Variant model properties

Eighteen (18) variant models were developed for each building that incorporate different values of three aforementioned parameters (ductility capacity, post cap stiffness, and energy absorbance coefficient) for the stress-strain relationship developed for strip model. For simplicity, these three parameters will be called as “post yielding parameters” in the rest of the paper. Values for post yielding parameters in each variant model are shown in Table 2. The lower and upper bound considered herein for post yielding parameters are chosen in such a way that any convergence issue is avoided (sudden degradation in stress-strain relationship adopted for strip model can create significant convergence issues). In addition, these values are selected based on engineering judgement and available data extracted during calibration process.

Table 2: Associated post yielding parameters to each variant model

Variant number	Nomenclature	Post yielding parameters		
		Ductility Capacity	Post cap stiffness ratio	Energy absorbance coef.
1	5-0.05-1,1	5	-0.05	1
2	5-0.15-0.2	5	-0.15	0.2
3	5-0.15-0.5	5	-0.15	0.5
4	5-0.15-1	5	-0.15	1
5	5-0.2-0.5	5	-0.2	0.5
6	5-0.2-1	5	-0.2	1
7	12-0.05-1	12	0.05	1
8	12-0.15-0.2	12	-0.15	0.2
9	12-0.15-0.5	12	-0.15	0.5
10	12-0.15-1	12	-0.15	1
11	12-0.2-0.5	12	-0.2	0.5
12	12-0.2-1	12	-0.2	1
13	20-0.05-1	20	-0.05	1
14	20-0.15-0.2	20	-0.15	0.2
15	20-0.15-0.5	20	-0.15	0.5
16	20-0.15-1	20	-0.15	1
17	20-0.2-0.5	20	-0.2	0.5
18	20-0.2-1	20	-0.2	1

4.3 Incremental Dynamic Analysis (IDA) procedure

To accurately assess the seismic performance of the SPSWs, a suite of ground motions were chosen for IDA analyses. Selecting the appropriate number of ground motions for IDA analysis is always a challenge. Sufficient number of ground motions that should be considered for an appropriate seismic assessment depends on various parameters like the accuracy of the adopted analysis, type of the response chosen for comparison, and considered limit states. It is clear that the dispersion of the responses taken from an Immediate Occupancy limit state for a specific structure is much less than that extracted for the same structure in a collapse condition, so the extent of nonlinearity in the structure can also affect the total number of needed ground motions for seismic assessment of the structure. Vamvatsikos and Cornell (2005) suggested that 10 to 20 ground motions should be considered to assess the seismic performance of mid-rise buildings with sufficient accuracy. In the current study, total number of 30 ground motions was chosen for seismic assessment of the SPSWs. Each variant model (18 variant models for each building) was subjected to IDA procedure using a suite of 30 ground motions. The IDA analyses have been conducted using maximum interstorey drift as the engineering demand parameter (EDP) and 5% damped spectral acceleration of the buildings first-mode period was used as intensity measure (IM).

4.4 Fragility curves and probabilistic seismic 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 of the structure. Fragility function parameters can be estimated by taking the logarithms of IM value of each ground motions associated with onset of collapse and calculate the mean and standard deviation.

[3] $ln\theta = \frac{1}{n}\sum_{i=1}^{n} lnIM_i$

[4] $\beta = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(\ln(IM_i/\theta))^2}$

where n is the total number of ground motions; IM_i is the IM value associated with onset of collapse in the ith ground motion; $ln\theta$ and β are mean and standard deviation respectively.

A lognormal distribution function is used to define fragility function (Eq. 7).

$$[5]\quad P(C|IM = IM_i) = \Phi\left(\frac{\ln\left(\frac{x}{\theta}\right)}{\beta}\right)$$

where $P(C|IM = IM_i)$ is the probability of structure's collapse that is subjected to a ground motion with IM value equal to IM_i; Φ is standard cumulative distribution function (CDF). The derived fragility curves for variant models in each building (3- and 5- storey SPSW systems) are depicted in Figure 9.

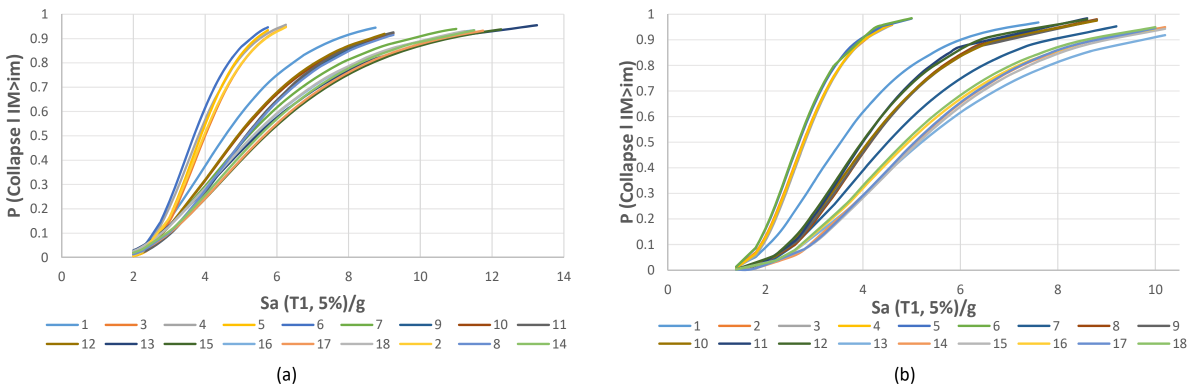

Figure 9: Derived fragility curves for all variant models (a) 3-storey SPSW (b) 5-storey SPSW

To draw a quantitative comparison between the seismic performance of all variant models in each building, the Sa value correspond to 50% probability of collapse which is known as median collapse capacity (MCC) is taken as a benchmark for comparison. The following section will concentrate on the sensitivity of each variant model to the variation of each post yielding parameters.

4.4.1 Sensitivity to ductility capacity changes

As shown in Figure 10-a, six groups were defined to assess the sensitivity of variant models to ductility capacity changes. In order to compare the extracted values for MCC obtained for each variant model, MCC values were normalized based on the largest value obtained for MCC in each building. Variant number 15 (20-0.15-0.5) and 13 (20-0.05-1) produce largest MCC values for 3- and 5- storey SPSW respectively. As depicted in Figure 10-a, MCC increases in both buildings as ductility capacity increases. This is a clear trend in all six groups of variant models selected for this study. It should be noted here that the buildings selected in this study are categorized as shear dominated systems. In all six groups, ductility capacity changes from 5 to 20 while other two post yielding parameters are kept constant in each group. Group five of the variant models represent the most sensitive group to ductility capacity changes in both buildings. About 31.2% and 46.9% increase in the MCC of 3-storey and 5-storey SPSW system respectively were observed when ductility capacity was changed from 5 to 20. The results indicate that the capacity of the SPSW building is quite dependent on ductility capacity and this dependency increases as the height of the building increases.

4.4.2 Sensitivity to post cap stiffness changes

Similar to ductility capacity, six groups of variant models were considered to assess the sensitivity of MCC to post cap stiffness variation (Figure 10-b). The first three groups consist of three variant models for each building while the remaining three groups consist of two variant models per building. In the first three groups, post cap stiffness varies from 0.05 to 0.2 while in the remaining models it varies in a smaller range from 0.15 to 0.2. Smaller post cap stiffness value corresponds to slight degradation in stress-strain relationship adopted for strip model. First group in both buildings exhibit the most sensitivity to post cap stiffness changes by 18.9% and 24.7% reduction in the collapse capacity of 3-storey and 5-storey SPSW system respectively. Results indicate that the building capacity is less sensitive to post cap stiffness variation when compared with the first post yielding parameter (ductility capacity).

4.4.3 Sensitivity to energy absorbance coefficient changes

Similar to the second post yielding parameter, three group with three variant models per building and three group with two variant models were assumed to evaluate the effect of changes in energy absorbance coefficient on the capacity of the building (Figure 10-c). This parameter is changed from 0.2 to 1 while the other two parameters are kept fixed. As opposed to ductility capacity and post cap stiffness, energy absorbance coefficient has a minor effect on the capacity of the buildings. For 3-storey and 5-storey SPSW systems, increase in energy absorbance coefficient value results in a slight reduction in MCC. Maximum reduction of 5.9% and 4% in MCC values due to energy absorbance coefficient variation was found for 3- and 5-storey SPSW system, respectively.

5 CONCLUSION

In the current study, a deterioration model is developed for analysis of steel plate shear walls. The deterioration model is implemented in the Opensees software as stress-strain relationship for strip model. This new deterioration model is able to capture the degradation due to plate tearing. The effectiveness of this new model is assessed through three experimental results. In the second phase of the study, the sensitivity of probabilistic seismic collapse capacity of SPSW systems was assessed by changing the values of post yielding parameters. For this study, two SPSWs (3-storey and 5-storey) were designed and 18 variant models were developed for each shear wall by selecting different values for post yielding parameters. These variant models were analysed using incremental dynamic analysis procedure. The results obtained from 36 variant models were compared. The maximum variation in median collapse capacity of SPSWs for different combination of post yielding parameters (i.e. ductility capacity, post cap stiffness, energy absorbance coefficient) were observed as 46.9% (5-storey), 24.7% (5-storey), and 5.9% (3-storey), respectively. In general, the behaviour of 3-storey SPSW was less sensitive to variation of post yielding parameters than that for 5-storey SPSW. For the selected SPSWs, study shows SPSW capacity is more sensitive to ductility capacity changes. On the other hand, energy absorbance coefficient changes, which mainly reflects the effects of pinching due to plate tearing in the system, does not have a significant impact on the capacity of the SPSW. In order to generalize these results, the sensitivity of more shear-dominated SPSWs should be assessed following the procedure described in this paper.

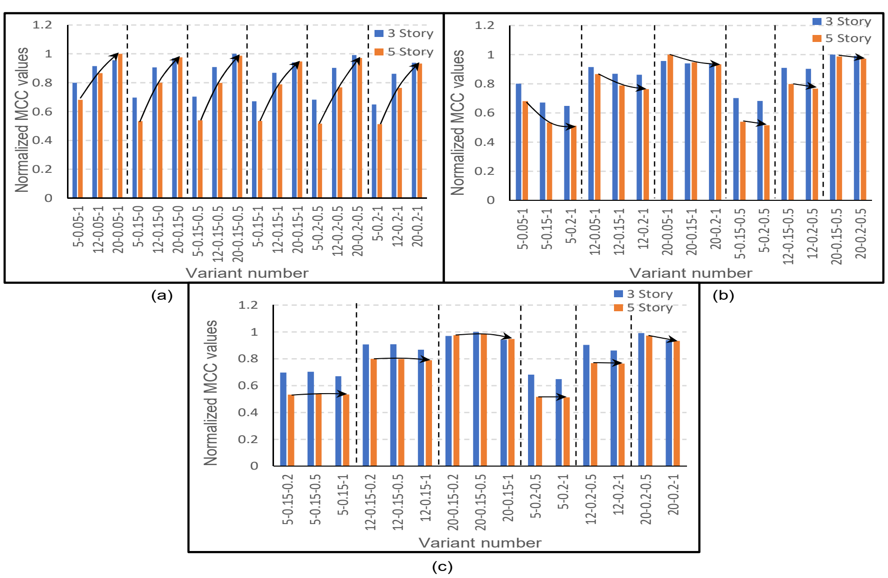

Figure 10: Sensitivity to post yielding parameters (a) Ductility capacity changes (b) Post cap stiffness changes (c) Energy absorbance coefficient changes

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