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Collapse Assessment of Stiffened Steel Plate Shear Walls Using FEMA P695 Methodology

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ABSTRACT

Quantification of building system performance and response parameters such as ductility–related force modification factor (R_d) and overstrength–related force modification factor (R_0) are vital for designing of steel plate shear walls (SPSWs). Unlike stiffened SPSWs, there are several analytical models available to assess the seismic performance of unstiffened SPSWs. This paper presents a new component strength deterioration model for stiffened infill plate to evaluate the seismic performance of stiffened SPSWs using FEMA P695 procedure. The newly developed component strength deterioration model was validated against the available experimental results. A total of three multi-storey (7-, 10-, and 13-storey) stiffened SPSWs with panel aspect ratio of 1.39 are considered in this study. Static pushover and incremental dynamic analyses using a suite of 44 ground motions compatible to Western Canada are conducted for all archetypes. Obtained adjusted collapse margin ratios for designed stiffened SPSWs with similar response parameters of those for unstiffened SPSWs, are compared with allowable limits given in FEMA P695. The results indicate that proposed values for response parameters of unstiffened SPSW can be used for stiffened one. Since interstory drift is an important factor in performance-based analysis, the variation of maximum interstory drift was captured in all stories for all designed archetypes during incremental dynamic analysis.

Keywords: Stiffened SPSW, FEMA P695, Incremental dynamic analyses, Response parameters, Maximum interstory drift.

INTRODUCTION

Unstiffened SPSWs have been widely used as a primary load carrying system in many buildings in Japan and North America. Unstiffened SPSW consists of a thin infill plate which is connected to surrounding boundary members using bolts or fillet weld. Significant number of analytical as well as experimental studies have been conducted on the behavior of unstiffened SPSW systems. The results exhibit the effectiveness of the system in resisting applied lateral loads by forming the tension field action (TFA). The main drawback of the system is the buckling of thin infill plate in relatively small compressive stresses which results in significant reduction in energy dissipation capacity and initial stiffness of the whole system. One available approach to postpone the buckling of infill plate is to replace thin infill plate by a thick one. This method is not practical and have many disadvantages, such as more cost, higher demands on boundary elements, and higher weight, which result in larger seismic forces applied on the system. Other available method is to install a series of horizontal and vertical stiffeners on thin infill plate to postpone the buckling of infill plate. Sabouri-Ghomi and Asad-Sajjadi [1] tested two one story similar SPSWs with and without stiffeners. The results indicated that stiffener's installation caused 26% increase in energy dissipation capacity of the whole system. In addition, initial elastic stiffness of the system increased by 51% whereas the effect of stiffeners installation on maximum strength of the system was found to be negligible. Unlike unstiffened SPSW, few analytical as well as experimental studies are available for stiffened SPSWs. In this study, research will be carried out to investigate the seismic performance of stiffened SPSWs. At the time of this writing, there is no specific recommendation in NBCC 2015 [2] regarding ductility-related force modification factor, R_d, and overstrength-related force modification factor, R_o, for designing of stiffened SPSWs. Thus, three multi-storey (7-, 10-, and 13-storey) stiffened SPSWs were designed using same factors as of unstiffened one. Efficiency of the aforementioned modification factors were further assessed for stiffened SPSW using the methodology addressed by FEMA P695 [3]. FEMA P695 provides a rational basis to evaluate the accuracy of adopted response modification factors for a specific type of load resisting system by assessing the probability of collapse under maximum considered earthquake (MCE).

BUCKLING OF STIFFEND SPSW SYSTEM

Infill plate under pure shear can buckle in two modes: (1) global buckling and (2) local buckling. Stiffeners are required to be designed in such a way that local buckling of infill plate in sub panels occur prior to global buckling to ensure appropriate structural performance. To satisfy this requirement, a minimum moment of inertia is required for horizontal and vertical stiffeners to prevent global buckling. Eq. (1) provides the minimum required moment of inertia for stiffeners to ensure that local buckling of infill plate happens in subpanel prior to global buckling of entire infill plate.

$$I > 0.916\left(\frac{K_l\, d^2}{s\, K_g} - s\right) t^3 \tag{1}$$

where K_l and K_g are local and global buckling factors respectively; d is the height of the panel; t is infill plate thickness; s is the spacing between stiffeners (by the assumption of equal spacing in x and y direction). Other requirement for this study is that the spacing between horizontal and vertical stiffeners must be determined in such a way that shear yielding of infill plate occurs prior to local buckling to ensure utilizing the whole capacity of the infill plate. By assuming $s_x = s_y = s$, minimum spacing required to guarantee shear yielding occurrence in subpanels can be derived using Eq. (2):

$$s_{min} = 4.775\, t\sqrt{E_s/\sigma_y} \tag{2}$$

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

MODELLING APPROACH

As discussed, infill plate resist applied lateral forces by forming the tension field action. Thorburn et al [4] proposed an analytical model to predict the behavior of unstiffened SPSW systems. Appropriateness of proposed analytical model was experimentally confirmed by Timler and Kulak [5]. The proposed analytical model consist of series of parallel truss elements known as strips oriented in the direction of tension field angle, α, to resist applied lateral loads. A general configuration of strips and their connections to surrounding boundary members are depicted in Fig. 1(a). Stress-strain relationship adopted for truss members generally follows a tri-linear relationship in tension zone and a bilinear relationship in compression zone (Fig. 1(b)). In the case of fully stiffened SPSW, shear buckling stress, τ_{cr}, is replaced by shear yielding, τ_y, to represents that the infill plate is sufficiently stiffened by vertical and horizontal stiffeners. It is worth mentioning that the tri-linear stress-strain relationship considered in tension zone for strips is capable of considering the wide range of possible behaviors that infill plate can experience during excitations such as elastic, yielding, strain hardening, and degradation due to web tearing.

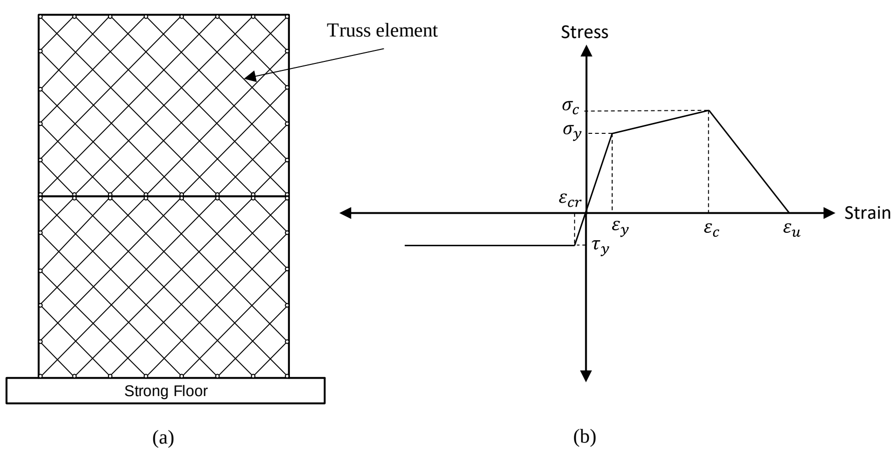

Figure 1: (a) Analytical cyclic strip model considered for stiffened SPSW; (b) stress-strain relationship assigned to strips

MODEL VALIDATION

Two one story similar SPSW systems with and without stiffeners were tested by Sabouri-Ghomi and Asad-Sajjadi (2012). During the cyclic test of stiffened SPSW, first significant yielding was observed at a story shear displacement of 1.58 mm. Maximum shear capacity of 808 KN was reached at shear displacement of 34.05 mm and maximum interstory of 6.44% was recorded at the end of the test for the stiffened specimen. At story shear displacement of 21.6 mm (2.25% drift), minor tearing occurred in one of the middle sub panels and the tearing propagated within the subpanel by increasing the shear displacement. SPSW strength decreased when the sub steel plates lost their continuity due to extensive web tearing. Unlike unstiffened SPSW, no local and global buckling were observed in columns at the end of the experiment. As reported by Sabouri-Ghomi and Asad-Sajjadi, columns in stiffened SPSW remained essentially elastic, while plastic hinges were formed at both ends of the columns in unstiffened SPSW. This is due to the fact that in stiffened SPSW, significant compressive forces are developed in infill plate which counteract a significant portion of tension forces created in boundary members due to formation of tension field action in infill plate. Thus, existence of stiffeners will reduce the demand (Flexural and shear forces) on boundary members. General configuration of stiffened SPSW system along with spacing between horizontal and vertical stiffeners is depicted in Fig. 2(a). The measured (obtained from experimental results) and predicted (obtained from *OpenSees*) base shears are plotted against roof displacement in Fig. 2(b). As depicted in the figure, there is a good agreement between numerical model and experimental test. Yield point, capping point, slight pinching, and degradation backbone curve until the end of the experiment is captured precisely by the developed numerical model.

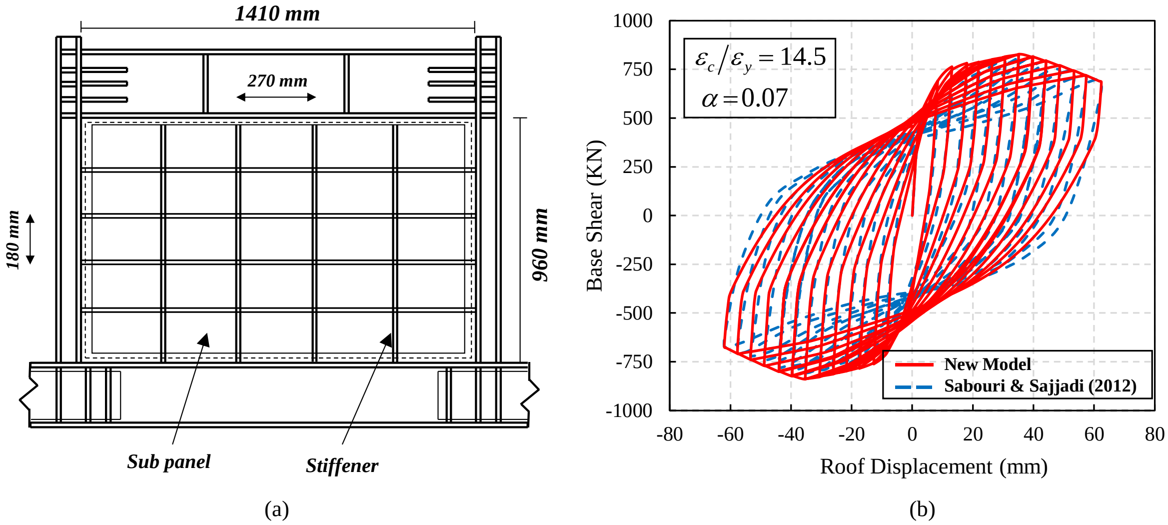

Figure 2: (a) Stiffened SPSW configuration tested by Sabouri and Sajjadi (2012); (b) model verification

DESIGN AND MODELLING SPECIFICATIONS

The buildings studied herein, are three multi-story (7-, 10-, and 13-storey) stiffened SPSW systems. These buildings are hypothetical residential buildings located in Vancouver, Canada and share a 30x30 symmetric square plan in which two stiffened SPSW systems resist applied lateral loads in each direction. All archetypes were designed according to CSA S16-14 [6] and NBCC 2015. A dead load of 4.1 kPa for each floor and 3.3 kPa for roof level was considered. Live load on all floors was taken as 2.4 kPa (other than roof level). The snow load was calculated to be 1.64 kPa at the roof level for all archetypes. To avoid unwanted effects of initial imperfection on overall performance of the whole system and satisfy the requirements regarding minimum thickness of infill plate to ease welding process, a minimum thickness of 2 mm was considered during design process of stiffened infill plates. The whole story shear is assumed to be resisted by the infill plate only. As presented in steel design guide 20 [7], the design shear strength of stiffened SPSW is calculated based on shear yielding of stiffened infill plate as follows:

$$V_r = 0.6\, \emptyset\, F_y\, t_w\, L_{cf} \tag{3}$$

where $\emptyset = 0.9$; F_y is specified yield stress of infill plate; t_w is the plate thickness; L_{cf} is the clear length of the panel between VBE flanges. The capacity design procedure proposed by Berman and Bruneau [8] was employed to design surrounding boundary members. This procedure is based on uniform collapse mechanism which includes uniform yielding of infill plates in all stories along the height of the building as well as plastic hinges formation at the ends of HBEs. Table 1 presents the designed boundary members for the selected stiffened SPSWs. Regarding the thickness of infill plate, 3 mm plate thickness was assumed in first 2, 5, and 7 stories of 7-, 10-, and 13-storey stiffened SPSW systems, respectively. Remaining stories were designed with minimum infill plate thickness (2 mm).

Table 1. Summary of stiffened SPSW frame member properties

	7-storey stiffened SPSW		10-storey stiffened SPSW		13-storey stiffened SPSW	
Storey	**HBE section**	**VBE section**	**HBE section**	**VBE section**	**HBE section**	**VBE section**
0	W360x382	NA	W360x382	NA	W360x421	NA
1	W310x79	W360x818	W310x79	W360x900	W310x79	W40x655
2	W360x179	W360x634	W310x79	W360x744	W310x79	W1000x883
3	W310x67	W360x634	W310x79	W360x744	W310x79	W1000x748
4	W310x67	W360x634	W310x79	W360x744	W310x79	W1000x748
5	W310x67	W360x634	W360x179	W360x744	W310x79	W1000x748
6	W310x67	W360x592	W310x67	W360x744	W310x79	W1000x748
7	W360x287	W360x509	W310x67	W360x744	W360x179	W1000x642
8	----------	----------	W310x67	W360x677	W310x67	W1000x591
9	----------	----------	W310x67	W360x634	W310x67	W1000x554
10	----------	----------	W360x287	W360x509	W310x67	W1000x539
11	----------	----------	----------	----------	W310x67	W1000x483
12	----------	----------	----------	----------	W310x67	W1000x412
13	----------	----------	----------	----------	W360x314	W1000x296

ANALYSIS OF STIFFENED SPSWS

Static pushover analysis

Nonlinear pushover analysis was performed on all stiffened SPSWs archetypes to extract system overstrength factor, Ω, and period-based ductility, μ. These two factors are calculated using the following equations:

$$\Omega = \frac{V_{max}}{V_d} \tag{4}$$

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{5}$$

where V_{max} is the maximum base shear obtained from nonlinear pushover analysis; V_d is the design base shear; Δ_u is the ultimate roof displacement of the building correspond to 20% strength loss with regards to maximum strength; Δ_y is the yield roof displacement. Fig. 3 exhibits the results of nonlinear pushover analysis along with idealized bi-linear pushover curves for all archetypes.

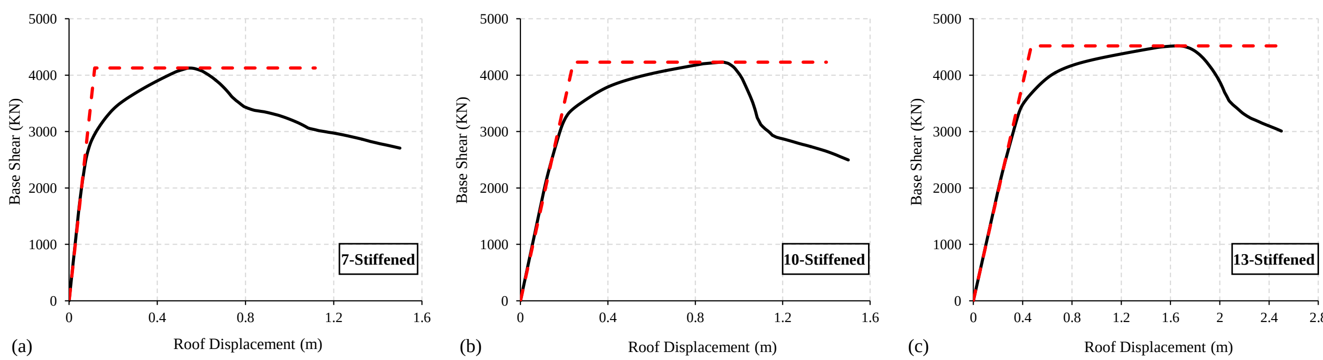

Figure 3: Monotonic pushover analysis results

Incremental dynamic analysis

Incremental dynamic analysis (IDA) was conducted on all archetypes using a set of 44 artificial ground motions developed for western Canada by Atkinson [9]. Table 2 specifies different characteristics of each ground motion including magnitude (M), closest distance to fault rupture, R_{fault}, and the ratio of maximum velocity to maximum acceleration, v/a. It is worth mentioning that incremental dynamic analyses have been performed using 5% damped spectral acceleration at fundamental period of the structure as intensity measure (IM) and maximum interstory drift as engineering demand parameter (EDP). The results of incremental dynamic analyses are depicted in Fig. 4. Collapse fragility function was developed for each archetype based on the results obtained from incremental dynamic analysis. The IM value corresponding to initiation of collapse in each ground motion is calculated using the IDA results and a cumulative distribution function is fitted to resulting IM values (Fig. 4). The term "Discrete Probability" in Fig. 4 refers to the ratio of collapse cases to the total number of analyses for each IM level and the term "Cumulative Distribution" refers to theoretical cumulative distribution function which is calculated based on a normal distribution with a specified median and standard deviation. 50% probability of collapse under a suite of ground motions which is known as median collapse capacity (MCC), S_{CT}, and collapse margin ratio, CMR, are calculated from fragility results for each archetype and are presented in Table 3. To account for the effects of spectral shape on collapse capacity of the structure, CMR is modified by multiplying a factor known as spectral shape factor (SSF) to obtain adjusted collapse margin ratio (ACMR). SSF is a function of fundamental period, T, period-based ductility, μ_T, and seismic design category which is assumed to be D_{max} in this study. For performance evaluation purposes, uncertainties related to design requirements (β_{DR}), modelling (β_{MDL}), test data (β_{TD}) chosen for verification of numerical model, and record to record variability (β_{RTR}) must be quantified based on the guideline provided by FEMA P695 methodology.

Uncertainty related to design requirements: Design requirements uncertainty refers to the completeness and robustness of the design procedure followed to design stiffened SPSW. In this study, the design guideline provided in CSA S16-14 and NBCC 2015 was followed to design stiffened SPSW. CSA S16-14 provides sufficient design requirements to ensure a safety margin against unanticipated failure modes. Thus, in the current study, design requirements uncertainty was rated as B (good) and the corresponding value of 0.2 was assigned.

Uncertainty related to Modelling: This uncertainty refers to capability of developed analytical model to cover the wide range of possible failure modes in the system. Based on the study performed by Purba and Bruneau [10], web tearing of thin infill plate and flexural failure of boundary members are main sources of degradation in SPSW systems. Developed analytical model is capable of considering both sources of degradation in the system. Hence, the nonlinear model development in this study was rated as B (good) and the corresponding value associated with modelling uncertainty was assumed to be 0.2.

Uncertainty related to test data: This uncertainty refers to the reliability of the test data which are used for verification purposes and total number of experiments that are used for verification. At the time of this writing, very limited number of experimental studies have been performed on stiffened SPSW systems. On the other hand, in unstiffened SPSW, total number of tested specimens are less than 50 which is quite less than other lateral load resisting systems (e.g., moment resisting frames). In the current study, test data rated as C (fair) and corresponding value associated with test data uncertainty was assumed to be 0.35.

Table 2: Characteristics of selected unscaled ground motions

Event	M	R_{fault} (km)	PGA (g)	v/a	**Event**	M	R_{fault} (km)	PGA (g)	v/a
west6c2.1	6.5	19.7	0.223	0.0959	west7c1.19	7.5	21.6	0.433	0.1032
west6c2.2	6.5	19.7	0.27	0.087	west7c1.20	7.5	21.6	0.309	0.0989
west6c2.4	6.5	21.6	0.222	0.0799	west7c1.22	7.5	20.3	0.341	0.1044
west6c2.5	6.5	21.6	0.244	0.0776	west7c1.23	7.5	20.3	0.325	0.1596
west6c2.10	6.5	21.6	0.174	0.0788	west7c1.25	7.5	18.1	0.58	0.0949
west6c2.11	6.5	21.6	0.184	0.0848	west7c1.26	7.5	18.1	0.516	0.11
west6c2.16	6.5	21.8	0.239	0.0753	west7c1.31	7.5	26.3	0.33	0.0811
west6c2.17	6.5	21.8	0.176	0.1013	west7c1.32	7.5	26.3	0.284	0.1289
west6c2.22	6.5	25.8	0.168	0.0676	west7c1.34	7.5	26.3	0.179	0.1224
west6c2.23	6.5	25.8	0.208	0.0968	west7c1.35	7.5	26.3	0.248	0.109
west6c2.37	6.5	27.8	0.183	0.076	west7c1.37	7.5	26.3	0.245	0.1182
west6c2.38	6.5	27.8	0.204	0.0854	west7c1.38	7.5	26.3	0.229	0.0928
west7c1.1	7.5	16.4	0.522	0.112	west7c1.40	7.5	26.3	0.262	0.0815
west7c1.2	7.5	16.4	0.588	0.0793	west7c1.41	7.5	26.3	0.22	0.1371
west7c1.4	7.5	17.1	0.327	0.0931	west7c1.43	7.5	26.3	0.185	0.1376
west7c1.5	7.5	17.1	0.284	0.108	west7c1.44	7.5	26.3	0.276	0.1103
west7c1.10	7.5	17.1	0.342	0.1067	west7c2.1	7.5	47.4	0.162	0.1321
west7c1.11	7.5	17.1	0.413	0.1106	west7c2.2	7.5	47.4	0.189	0.1293
west7c1.13	7.5	17.1	0.351	0.0704	west7c2.4	7.5	45.7	0.253	0.1108
west7c1.14	7.5	17.1	0.32	0.1297	west7c2.5	7.5	45.7	0.197	0.1319
west7c1.16	7.5	21.6	0.294	0.1208	west7c2.13	7.5	30.2	0.203	0.208
west7c1.17	7.5	21.6	0.392	0.1165	west7c2.14	7.5	30.2	0.256	0.0984

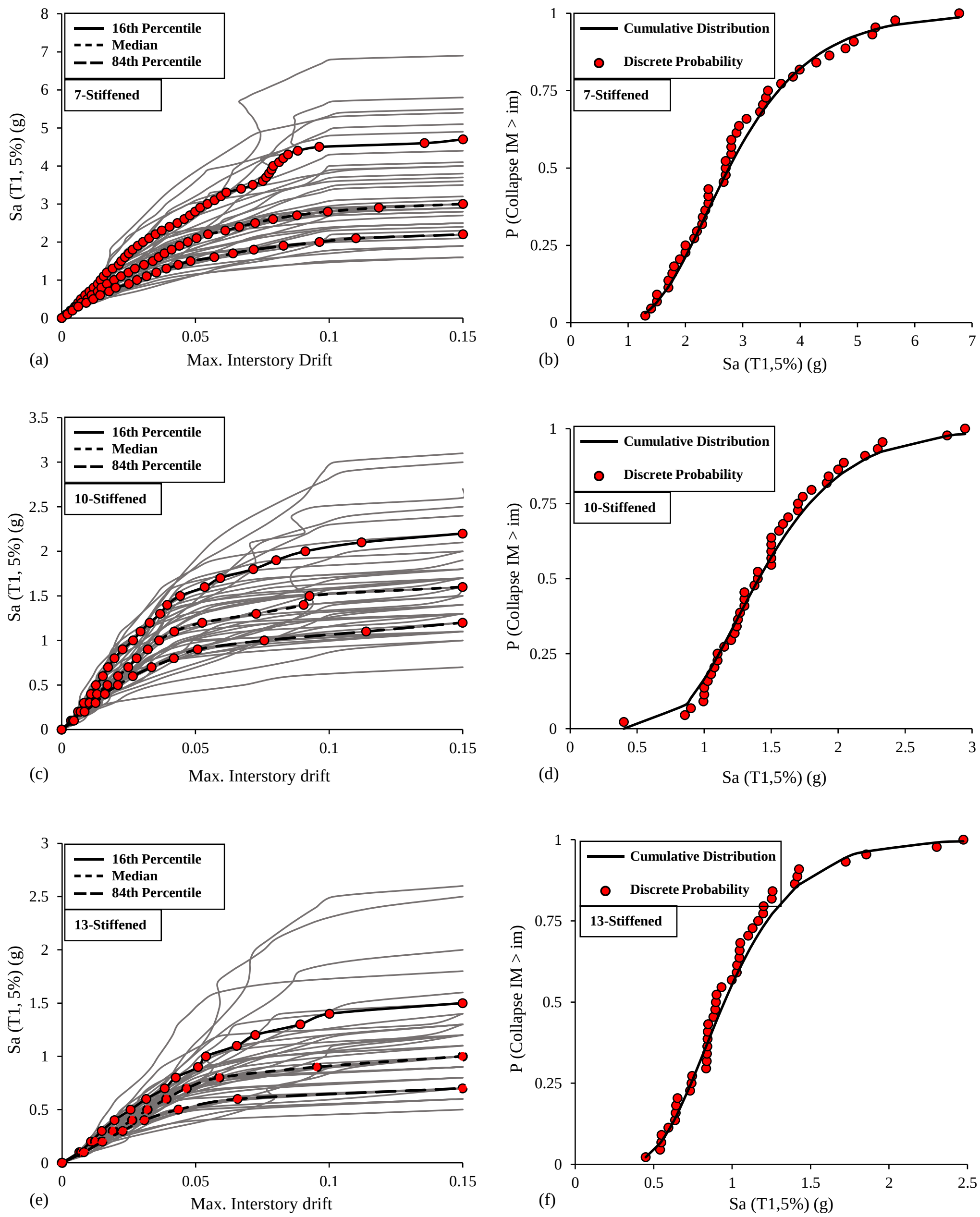

Figure 4: IDA results and corresponding derived fragility curves

Uncertainty related to record-to-record variability: Record-to-record uncertainty is due to the variation in response of a single archetype to a group of ground motions. This variability is mainly due to the variation in frequency content and dynamic characteristics of different ground motions. A fixed value of $\boldsymbol{\beta_{RTR} = 0.4}$ is considered for structures with significant period elongation (i.e., period-based ductility, $\boldsymbol{\mu_T \geq 3}$). For most of the systems with limited ductility, the same record-to-record uncertainty as of ductile systems can be used since most of the systems will experience a significant period elongation before collapse.

Total system collapse uncertainty is calculated based on combination of aforementioned four sources of uncertainties as follows:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \qquad (6)$$

Based on the given values for each uncertainty, total system uncertainty of 0.6 is obtained using Eq. 6. Acceptable value of adjusted collapse margin ratio ($ACMR_{10\%}$) is based on total system collapse uncertainty, β_{TOT}, and established values of acceptable probabilities of collapse which is normally assumed to be 10% probability of collapse under maximum considered earthquake (MCE) for assessing the performance of a group of archetypes. According to the table provided by FEMA P695, the acceptable ACMR for 10% probability of collapse under MCE ground motions for β_{TOT} of 0.6 is 2.16. As indicated in Table 3, all archetypes successfully passed the performance criterion which were prescribed by FEMA P695 procedure.

Table 3: Summary of results obtained from pushover and IDA analyses for all structural configurations

Archetype	Pushover results						IDA results			Performance evaluation		
	$\Delta_y(mm)$	$\Delta_u(mm)$	μ_T	V_d (KN)	V_{max} (KN)	Ω_0	S_{MT} (g)	S_{CT} (g)	CMR	SSF	ACMR	Pass/fail
7	116	937	8.07	1243.7	4126.5	3.3	0.452	2.76	6.11	1.3	8.24	Pass
10	242	1074	4.44	1502.7	4231.4	2.8	0.332	1.41	4.24	1.2	5.4	Pass
13	471	2059	4.37	1616.2	4520	2.8	0.237	0.95	4	1.3	5.22	Pass

Since interstory drift is an important indicator of structure's functionality in performance-based earthquake engineering, the variation of maximum interstory drift in each story is recorded under each ground motion and depicted in two different style (i.e., profile and boxplot) for each archetype. As shown, the box represents inner range (Q16 – Q84) which begins from 16th percentile and finishes at 84th percentile. The solid line inside the box represents the median of the data at the corresponding story. Two lines located outside of the box is used to indicate the upper and lower bound of the data. As can be seen in Fig. 5, maximum interstory drift in all stories for all archetypes are quite less than the allowable limit of 2.5%, as prescribed by NBCC 2015.

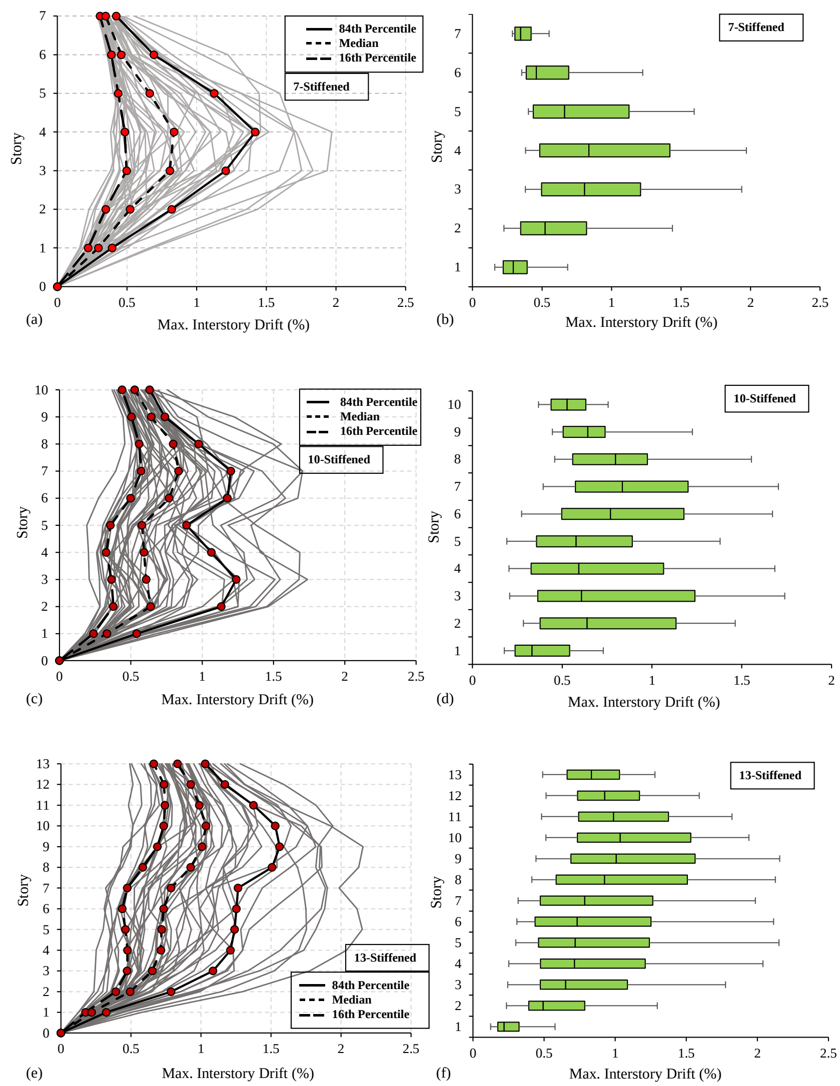

Figure 5: Interstory drifts of all archetypes under 44 ground motions at design level

CONCLUSIONS

This study presents results from nonlinear pushover analysis and IDA using a nonlinear macro-modelling approach to assess the probability of collapse for stiffened SPSWs. A set of 44 artificial ground motions compatible with uniform hazard spectrum of Vancouver region were used to investigate the seismic performance of three multi-storey (7-, 10-, and 13-storey) stiffened SPSWs. Results from the study are as follows:

- The modelling approach adopted in this study showed good agreement between FE analysis and experimental test conducted on stiffened SPSW (reported in the literature) under cyclic loading. Yield point, capping point, pinching, and degradation backbone curve until the end of the experiment were captured precisely by the developed numerical model.
- All archetypes studied herein showed ductile and stable behaviour and provided reliable safety margin against collapse. CMR and ACMR values were calculated for each archetype using the results of IDA and were compared with allowable ACMR value presented in FEMA P695. It shows a robust structural performance for all archetypes when subjected to strong ground motions.
- Satisfactory results were obtained for all designed stiffened SPSWs during performance evaluation, which was conducted based on methodology given by FEMA P695. Thus, based on acceptable results, ductility-related force modification factor (R_d) of 5 and overstrength-related force modification factor (R_0) of 1.6, which are used to design unstiffened SPSWs, can be used for stiffened one as well.
- The variation of maximum interstory drift in each story was recorded under each ground motion and indicated in two different style (i.e., profile and boxplot) for each archetype. The results indicated the effectiveness of the designed system to control the drift in all stories when subjected to strong ground motions. The upper bound value recorded for drift in each story in all archetypes was quite smaller than the allowable value of 2.5% that is given by NBCC 2015.

It is acknowledged that to recommend the applicability of aforementioned two factors (R_d and R_0) for designing of stiffened SPSWs to the engineering community, several multi-storey stiffened SPSW with wide range of building heights, bay widths, and different seismic design categories must be analysed and this task is currently in progress.

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