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SEISMIC PERFORMANCE EVALUATION OF STEEL PLATE SHEAR WALLS SYSTEM BY CAPACITY SPECTRUM METHOD

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ABSTRACT: Unstiffened Steel Plate Shear Walls (SPSWs) system is an effective lateral load resisting system. The prescriptive strength and deflection limitations that specified in the building current seismic design standard of Canada do not provide actual assessment of the ability and the performance of the SPSWs. Capacity Spectrum Method (CSM) is a performance based design procedure which can be used for performance evaluation of any structures. In this paper, a non-linear finite element model has been developed and validated with experimental study. A four-storey and eight-storey building with SPSWs have been designed based on equivalent static force method according to National building code of Canada (NBCC-2010). These preliminary designed building was modeled and used for performance evaluation. Standard design response spectrum of Canadian building code (NBCC-2010) for seismic design of the structure is converted into constant ductility demand spectrum. Base-shear versus roof displacement curve has been transferred into capacity spectrum of an Equivalent Single degree of freedom (ESDOF) system. Top displacement and ductility demands were considered as the performance parameters for SPSWs. These performance parameters were compared with results from extensive non-linear dynamic analyses for a number of site specific ground motion records.

1. Introduction

Steel Plate Shear Wall (SPSW) system is an effective lateral load resisting system for new and existing buildings. A SPSW consists of a steel plate, which is connected as an infill to the building structural frame composed of beams and columns. Beam-to-column connection of SPSW may either be pinned or moment connections. The steel infill plate is either bolted or welded to these boundary elements using fishplates. Storey shear of a building is primarily resisted by the diagonal tension field that forms in the unstiffened infill plate when they have buckled. Due to the significant post-buckling strength of thin infill plate (Thorburn et al. 1983), the use of thin unstiffened infill plates in SPSW have been accepted by researchers and designers. Behavior of unstiffened thin SPSW system and its design procedure have been investigated by various researchers since 1970s (Driver et al. 1997, Bhowmik et al. 2010,Berman and Bruneau 2008). Current design standard of America (AISC 2010) and Canada (CAN/CSA-S16-09) follows capacity design approach for SPSW design where steel infill plates and beams ends are designed as preliminary ductile fuses to dissipate seismic energy. In an unstiffened SPSW system, thin infill panels are the main energy dissipation element, which is allowed to buckle out-of-plane.

Performance-based seismic design (PBSD) is the new concept of design where the design procedure certifies the probable level of performance of a structure under a given level of hazard. PBSD requires a precise performance assessment of the structure at various stages in the design. Evaluation of seismic performance of SPSW systems is mainly limited in nonlinear dynamic analysis, which is one of the most accurate analysis technique to examine inelastic behavior of the structure under spectrum compatible ground motions records. However, its time-consuming and complicated manner makes it incompatible choice for design office. Moreover, structural performance in recent seismic events showed the need for new methodologies and concepts for the seismic performance evaluation of structures, which should be simple and computationally inexpensive. Traditional strength based design procedure in current building codes has very little scope to evaluate the seismic performance of the structures. Various nonlinear static analysis procedures have recently been introduced for design and seismic performance evaluation of the buildings.

The Capacity Spectrum Method (CSM) is a nonlinear static analysis procedure as well as a performance-based seismic design tool, which can be applied for performance evaluation and design verification of new and existing buildings. Freemen et al. (1975) first introduced CSM, where the graphical intersection of seismic demand and capacity curve to account structure's inelastic behavior is applied to design or performance evaluation of a structure. CSM compares the seismic demand with the capacity of a structure, which gives a visual representation of the seismic performance of the structure. The intersection of the demand and capacity curve is called performance point, which represents the probable performance of the structure to the particular seismic demands. ATC-40 and FEMA-440 utilized and provided some guidelines for CSM as an effective nonlinear static procedure. Later several researchers proposed different methods to apply CSM for performance evaluation. Fajfar (1999) developed an easy to use method for CSM using constant ductility inelastic response spectrum. Application of his proposed CSM on the framed structures has been shown very impressive prediction of displacement demand and ductility demand. No research programs have been conducted that uses CSM to investigate seismic performance of SPSW. Thus, applicability of CSM methods over nonlinear dynamic analysis needs to be investigated for SPSWs. In this study, seismic performance and seismic demand of a 4-storey and an 8-storey building with two identical SPSWs have been estimated by CSM, and the results are compared with nonlinear time-history analysis results.

2. Capacity-Spectrum Method

A non-linear base shear forces-top displacement relation (pushover curve) represents the capacity of the structure. Pushover curve is converted to equivalent spectral accelerations and spectral displacement by using effective modal mass and modal participation factors. Response spectrum is used as the seismic demands of the structure. After that, both curves are plotted in the same coordinate, demand-capacity relationship readily comes out.

Development of capacity curve of Equivalent-Single Degree of Freedom (ESDOF) system:

Step-1: Estimate the first natural frequency of vibration ω_n, and associated normalized elastic vibration mode shape (*φ)* of multi-degree of freedom system (MDOF). Thus, φ is assumed displacement shape for MDOF system. Compute the base shear-roof-displacement relation (pushover curve) for lateral force distribution of $p_i = m_i\phi_i$ where, m_i is the mass of i^{th}-storey. Physical basis of this force distribution is the inertial force of the structure that opposes the deformation due to the external forces.

Step-2: A transformation factor *Γ,* is calculated to convert from MDOF system to ESDOF system. All the properties such as base shear, top displacement, and hysteretic energy of MDOF can be transferred in to force, top displacement and hysteretic energy of ESDOF system respectively by this factor. *Γ* is also known as modal participation factor calculated by, $\Gamma = \frac{m^*}{\sum m_i\phi_i^2}$, where m^* is the mass of ESDOF system.

Step-3: Top displacement *(D_t)*-base shear *(V_r)* curve of MDOF system are transformed into force *(* $F^* = V_r/\Gamma$*)*-displacement *(* $D_t^* = D_t/\Gamma$*)* relationship of ESDOF system. Force-displacement relation of ESDOF system is idealized based on energy balance consideration of FEMA-273. Finally, bilinear idealized force-displacement curve is transferred into capacity curve by representing spectral acceleration to spectral displacement curve of ESDOF system. Spectral acceleration at the yielding point is, $S_{ay} = F_y^*/m^*$. Schematic figure of the transformation of pushover curve of MDOF to capacity curve of ESDOF system is presented in **Fig. 1**.

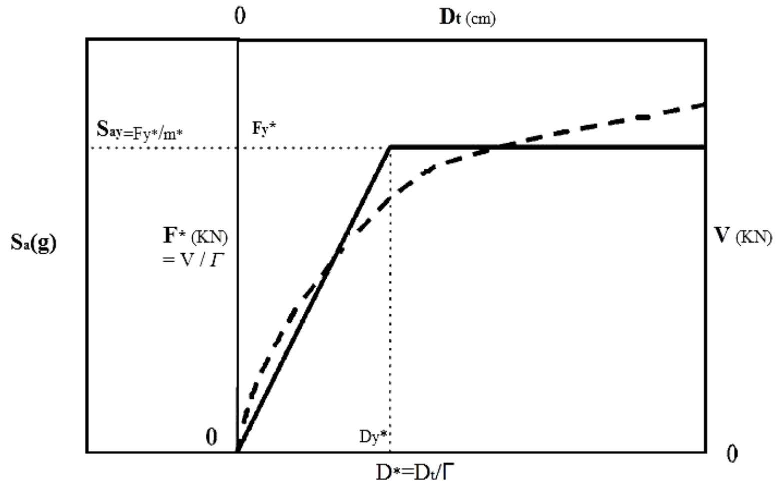


Fig. 1 - Development of the capacity spectrum of an ESDOF system Fajfar (1999)

<u>Seismic Demand in Acceleration Displacement Response Spectrum Format (ADRS):</u>
Highly damped elastic acceleration-displacement response spectrum (ADRS) is used to develop seismic demand spectra. Inelastic ADRS can be obtained directly by time-history analysis of inelastic SDOF system, or indirectly from elastic ADRS. The acceleration spectra S_a and displacement spectra S_d can be determined for an inelastic SDOF system, by using strength reduction factor R_μ. In this study, approximate bilinear expression of force reduction factor proposed by Vidic et al. (1994) has been utilized.

$$S_d = \mu \frac{T^2}{4\pi^2} S_a \tag{1}$$

$$R_\mu = (\mu - 1)\frac{T}{T_o} + 1 \quad , T \le T_o \; ; \qquad R_\mu = \mu \quad , T \ge T_o \tag{2}$$

$$T_o = 0.65\mu^{0.3} T_s \le T_s \tag{3}$$

where, S_{de} and S_{ae} are the Spectral displacement and pseudo acceleration of elastic response spectrum respectively corresponding to the period T. Force reduction factor *(R_μ)* is the ratio of elastic strength demand to inelastic strength demand of an ESDOF system for a specified ductility ratio (μ). T_s is the characteristics period, which refers the transition period where constant acceleration region intersect the constant velocity region. In this procedure, 5% damped spectrum are used.

<u>Determination of Seismic Demand and performance of ESDOF system:</u> Draw the Demand spectra and capacity spectra for ESDOF system in the same plot. Intersection point of the redial line of the capacity curve corresponding to the elastic stiffness of the ESDOF system and the elastic demand spectrum, gives the elastic strength requirement *(S_{ae})* of the structure. The yield acceleration *(S_{ay})* for the ESDOF system refers the acceleration requirements for the inelastic behavior. Ratio of the elastic acceleration demand and inelastic acceleration capacity is the reduction factor R_μ. After that, ductility can be calculated by the reverse calculation of equation 2. If the elastic period of the structures is larger than T_s, "Equal Displacement Rule" applies. It is assumed for medium-period and long-period range, inelastic displacement demand is equal to the elastic displacement demand.

3. Seismic Performance Evaluation of SPSW System

3.1. Design of SPSW System According to NBCC 2010

A 4-storey and an 8-storey building with SPSWs are designed based on current capacity design approach of CSA/CAN S16-09 with an identical floor plan, which represents a hypothetical office building located in Vancouver. Total floor area of the buildings is 2631.7m^2. The building has two identical SPSWs in each direction to resist lateral forces, so each shear wall is designed to resist one-half of the design seismic

loads. SPSWs are placed in such ways that maintain structural symmetry in horizontal and vertical direction. Therefore, only accidental torsion was considered in the equivalent static force calculation. Building was assumed to be on very dense soil and soft rock (soil class C according to NBC 2010). The aspect ratio of SPSW is 1.5 where, width of each shear wall panel is 5.7m and height is 3.8m. Dead load of 4.26kpa and live load of 2.4kpa were considered for each floor. For the roof, dead load of 1.12kpa and snow load was considered instead of live load. Design load combination is D+0.5L+E (where, D=dead load, L=live load and E= earthquake load) for floors and D+0.25Sn+E (Sn= snow load) for roof have been considered. Beam-to-column connections are considered as moment-resisting connections. In addition, the infill plates are connected with its boundary beams and columns with welded connections. Stiffness of the columns, top beam and bottom beam have satisfied the requirements that are specified in CAN/CSA S16-09 to allow uniform tension field development in the adjacent infill plates. Boundary column design was performed according to Capacity design approach of Berman and Bruneau (2008). The nominal yield strength of 350 Mpa and modulus of elasticity of 200, 000 Mpa for all the beams, columns and infill plates of SPSWs are assumed. A 3mm infill plate was selected as thinnest infill plate to meet the minimum practical available plate thickness as well as handling and welding considerations. Plan of the building and elevation of SPSW are presented in
Fig. **2**.

3.2. Finite Element Model of SPSWs System

Selected SPSWs have been modeled in ABAQUS (Hibbitt et al. 2011); where all the members have been modeled using a general-purpose four-node doubly curved shell element with reduced integration (ABAQUS element S4R). The implicit time integration method, ABAQUS/STANDARD was used for this study. This finite element modeling technique was validated with a quasi-static test result of a single-storey SPSW specimen tested by Lubell et al. (2000). A very good agreement was observed between finite element analysisl and experimental results (Fig).

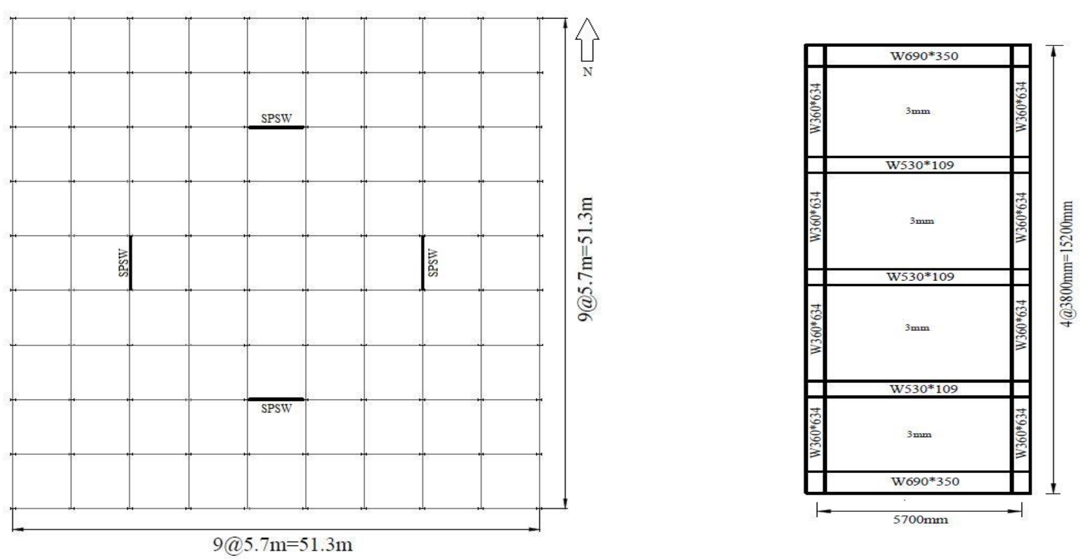


Fig. 2 - Plan view of Designed building (left) and elevation view of SPSW (right)

P-delta effect was considered during the seismic analysis by introducing a pin supported dummy gravity column in the finite element model of the SPSW system. This dummy column is connected to the plate wall at every floor level with pin ended rigid links, which maintains the constant horizontal displacement between SPSW and gravity column. This gravity column was made of 2-node linear 3-D truss (ABAQUS T2D3) and was designed to carry half of the total remaining mass at each floor level. The gravity loads of each storey were added as lumped masses on that column at corresponding floor. 5% Rayleigh proportional damping ratio was used for all the seismic analyses.

Table 1 - Details section properties of 4-storey and 8-storey SPSW systems

Storey	4-storey SPSW				8-Storey SPSW		
	Plate thickness (mm)	Column	Beam	Storey	Plate thickness (mm)	Column	Beam
0			W690*350	0			W690*350
1-2	3	W360*634	W530*109	1-2	4.8	W360*634	W530*109
3	3	W360*634	W530*109	3	4.8	W360*382	W530*109
4	3	W360*634	W690*350	4	4.8	W360*382	W690*192
				5-7	3.0	W360*216	W530*109
				8	3.0	W360*216	W690*350

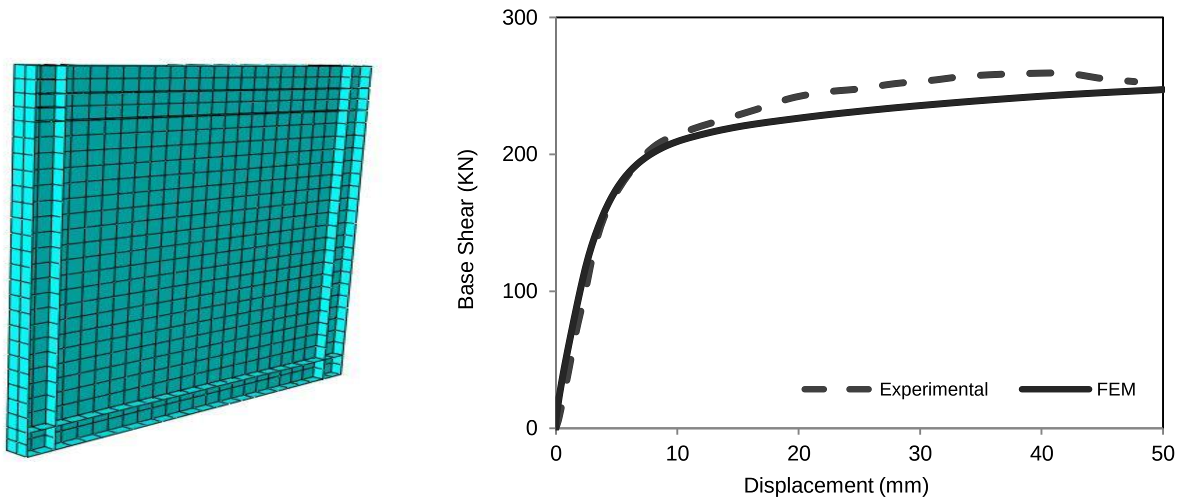


Fig. 3 - Finite Element Model (left) and validation with single-storey specimen of Lubell et al. (2000)

3.3. Nonlinear Seismic Analysis of SPSW System

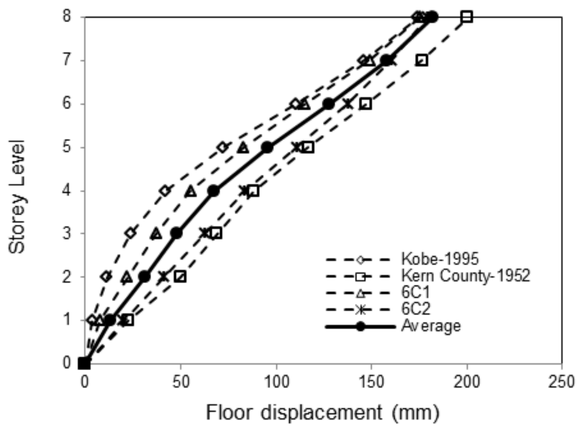
Two real and two simulated ground motion records (GMRs) have been selected and scaled for non-linear time history analysis of SPSWs. Two simulated GMRs were collected from engineering seismotoolbox (Atkinson 2009) for soil class C. One of them is near fault and another one is far fault earthquake records. Real GMRs are collected based on the ratio of their peak ground acceleration (A) to peak ground velocity (V). Real GMR were selected as such that they have A/V ratio close to 1.0, which is the recommended value for Vancouver region (Naumoski et al. 2004) and collected from strong ground motion database of Pacific Earthquake Engineering Research center, California (PEER 2010). Partial Area method of ground motion scaling has been conducted to scale the selected GMR.

Peak floor displacement and maximum inter-storey drift were estimated from the seismic analysis under selected GMRs at the instant of maximum roof displacement. Inter-storey drifts for both 4-storey and 8-storey SPSW were much lower than the NBCC 2010 drift limit. Floor displacement and inter-storey drift pattern was similar for all GMRs. The maximum floor displacements of 4-storey and 8-storey SPSWs are presented below. Average maximum dynamic base shear was 5053 KN and 6683KN for 4-storey and 8-storey SPSW respectively.

4. Application of CSM on SPSW

4.1. Capacity curve of SPSWs

Frequency analysis was performed for SPSWs to estimate fundamental frequency and associated elastic vibration mode shape of the structure. Base shear (V_r) versus top displacement (D_t) relation was estimated from pushover analysis with a monotonically increased load pattern (step-1). ABAQUS (Hibbitt, Karlsson and Sorensen 2011) was used to perform monotonic pushover analysis. Pushover analysis was performed up to the failure mechanisms are appeared. SPSW (MDOF system) properties for nonlinear pushover analysis are presented in Table 2. Nonlinear pushover curve of SPSWs are presented in Fig. **5**. SPSW systems were transferred in to ESDOF systems by introducing modal participation factor *(Γ)*. Mass of ESDOF systems are 1342 ton and 2052 ton for 4-storey and 8-storey building respectively. Modal participation factor of MDOF is 1.354 for 4-storey building and 1.566 for 8-storey building. Top displacement *(D_t)* and base shear *(V_r)* relation of MDOF systems have been transferred into force *(F^*)*-displacement *(D_t^*)* relationship of ESDOF system by using modal participation factor.



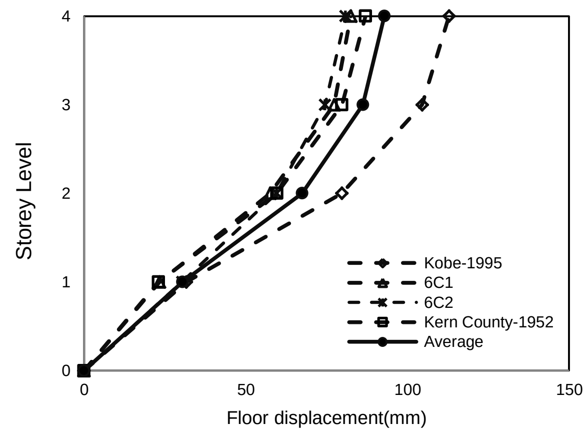


Fig. 4 - Maximum floor displacement of 8-storey SPSW (left) and 4-storey SPSW (right) under selected GMRs

After that, force *(F^*)*-displacement *(D_t^*)* relationship of ESDOF systems were idealized basing on energy balance consideration where, the post-yielding stiffness of ESDOF system is zero and area under the original pushover curve and bilinear curve are same and two curves intersect at the 60% of the yield strength. Bilinear idealized force-displacement curve of ESDOF system were now converted in to Spectral acceleration versus spectral displacement curve, which is known as capacity curve of ESDOF system. Original force *(F^*)*-displacement *(D_t^*)* curve of ESDOF systems, bilinear idealized curves and Spectral acceleration to spectral displacement curve for both SPSWs are presented in **Fig. 6**. Properties of ESDOF system for both 4-storey and 8-storey buildings are presented in Table 3.

Table 2 - SPSWs (MDOF system) properties

4 Storey				**8 Storey**			
Storey	**Mode Shape, *(ϕ)***	**Storey mass, m_i (ton)**	**Lateral force, *(P)***	**Storey**	**Mode Shape, *(ϕ)***	**Storey mass, m_i (ton)**	**Lateral force, (*P*)**
1	0.299	630	0.92	1	0.578	627	0.176
2	0.62	632	1.9	2	0.141	627	0.433
3	0.874	636	2.71	3	0.249	627	0.764
Roof	1.00	206	1.00	4	0.377	628	1.154
				5	0.536	629	1.644
				6	0.700	631	2.157
				7	0.859	636	2.665
				Roof	1.00	205	1.00

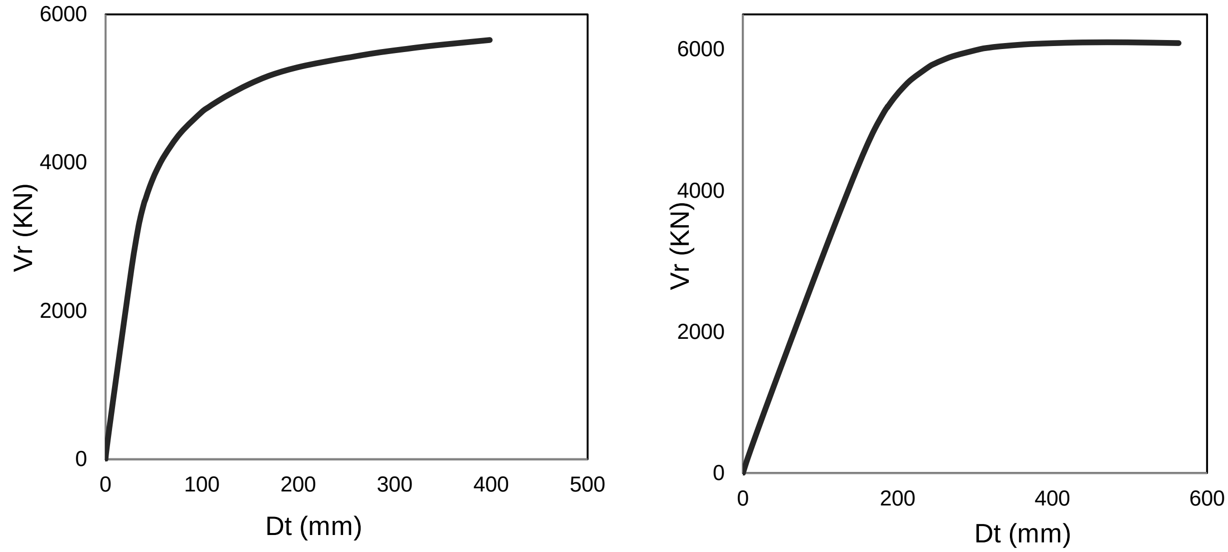


Fig. 5 - Base shear (V_r)-roof displacement (D_t) relation (Pushover curve) of 4-storey SPSW (left) and 8-storey SPSW (right)

Table 3 - Parameters of ESDOF systems

Parameters	**ESDOF system 1 (4 storey)**	**ESDOF system 2 (8 storey)**
Effective mass, *m** (ton)	1342	2052
Transformation Factor, *Γ*	1.354	1.566
Yield Strength, F_y* (KN)	3650	3800
Yield displacement, D_y* (mm)	40.5	128.9
Elastic Period, *T** (sec)	0.767	1.66

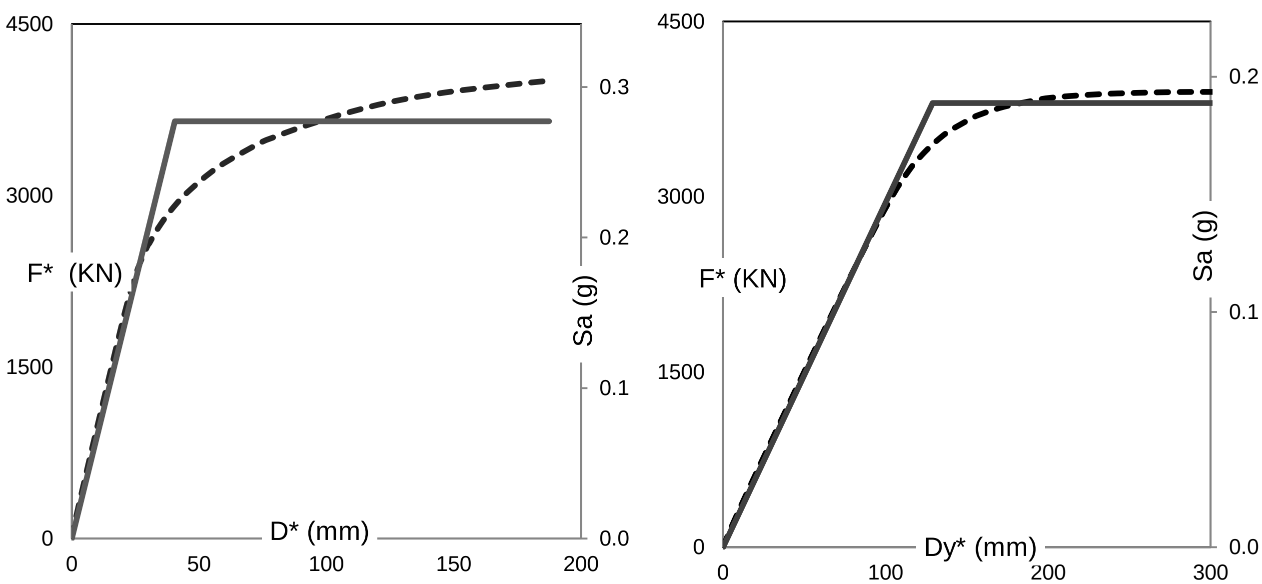


Fig. 6 - Force versus displacement curve, bi-linear idealization and spectral acceleration versus spectral displacement curve of ESDOF system of 4-storey SPSW (left) and 8-storey SPSW (right)

4.2. Demand Spectrum for SPSWs

Vancouver design spectral acceleration parameters (5% damped structure and for reference soil class C) were used to obtain seismic demand curve (Halchuk 2003). Design response spectrum of SPSW is estimated according to ASCE/SEI 7-10. Estimated design response spectrum according to ASCE/SEI 7-10 for Vancouver soil class C is presented in Fig. **7**. Displacement response spectrum is estimated from pseudo-acceleration to displacement relationship of SDOF system as given below.

$$S_d = \mu \frac{T^2}{4\pi^2} S_a \tag{4}$$

For inelastic SDOF system, acceleration spectrum S_a and displacement spectrum S_d were determined from elastic ADRS by using linear expression of reduction factor by Vidic et al. (1994). The characteristics period (T_s) is 0.35sec. In the beginning of this procedure, demand curve has been constructed for elastic response of the structure (e.g., ductility factor is equal to one).

5. Performance Evaluation of SPSW

Demand spectra and capacity spectra for ESDOF system are drawn in the same plot. Radial line of the capacity curve corresponding to the elastic period of the ESDOF system and slope of the radial line represents the elastic stiffness. Fig. 8 is showing the graphical representation of capacity curve of ESDOF system of 4-storey and 8-storey SPSW system, where elastic demand spectrum are same as **Fig. 7**. As observed, capacity curves of both systems do not intersect with elastic demand curve. However, projected radial line of capacity curve intersects with the elastic demand curve. Since the elastic periods of both of the structures were larger than T_s, "Equal Displacement Rule" was applied. Therefore, inelastic displacement demand is equal to the elastic displacement demand. Displacement demands were determined from the intersection point of the capacity curves and the demand curves corresponding to the ductility demands. Next, displacement demands of ESDOF systems were transferred in to displacement demands of MDOF systems by reverse transformation from ESDOF to MDOF systems.

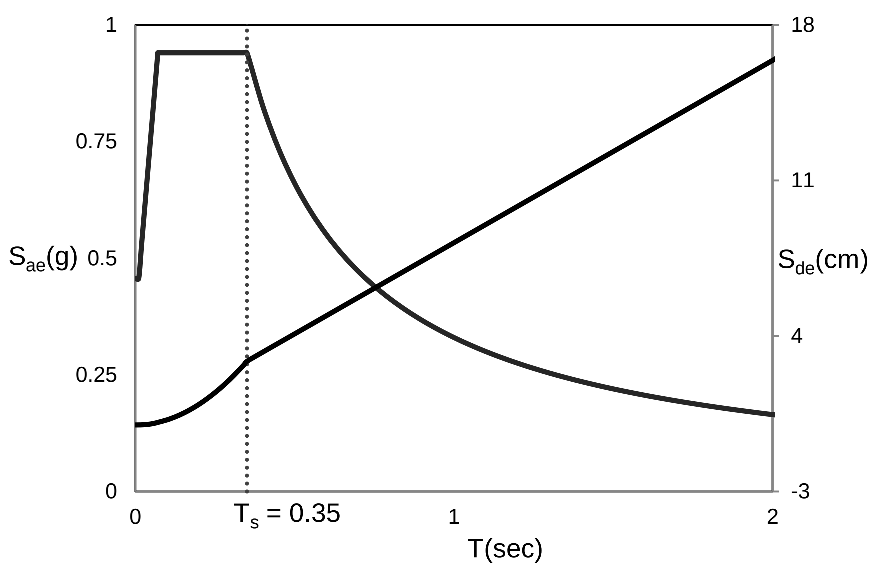
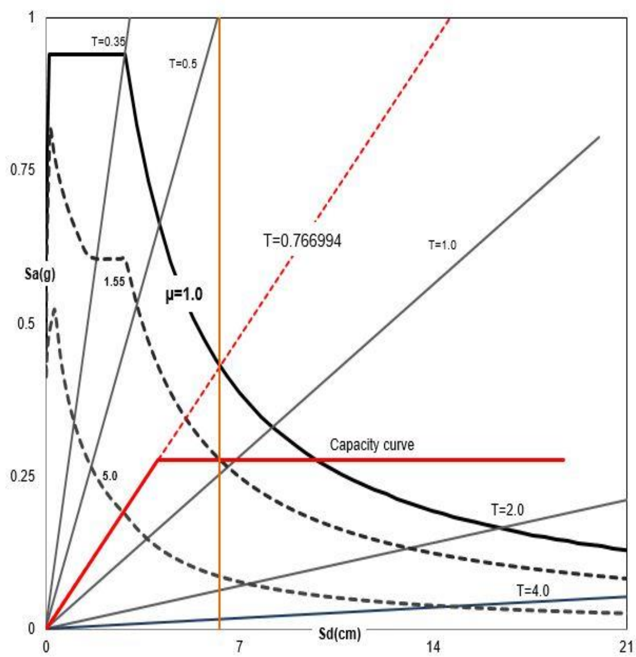


Fig. 7 - Elastic design acceleration response spectrum of Vancouver for 5% damped structure and corresponding displacement spectrum

In this study, top displacement demand for 4-storey building is 85.20mm and for 8-storey building is 212.6252mm, where both of them are very close to the non-linear time history analysis of the structure. Comparison between the Maximum top displacement of both of the buildings by CSM and non-linear time history analysis is given in Table **4**. According to current design guidelines of Canada, ductility based reduction factor for SPSW is 5.0 and over-strength related reduction factor is 1.6. In CSM, ductility demand of the structure was lower than the code suggested ductility; therefore, a seismic demand spectrum for ductility factor 5.0 has been developed in Fig. 8. To maintain practical availability and handling requirements, minimum plate thickness in the SPSW design was higher than the theoretical requirement, which increases a significant amount of overall capacity. Moreover, framing action in beams and columns has good contribution to the storey shear resistance. Therefore, capacity of the structure was very high, which is the main reason for this lower ductility demand.



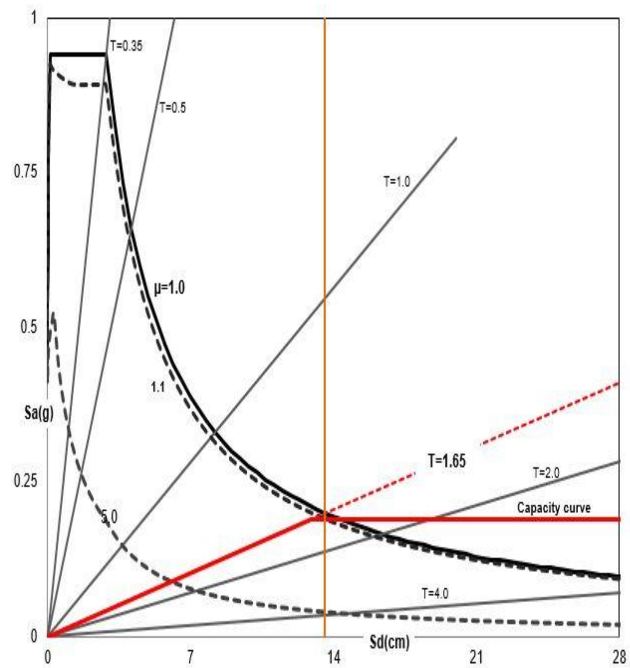


Fig. 8 - CSM graphical representation for 4-storey SPSW (left) and 8-storey SPSW (right)

Table 4 - Performance evaluation of the building by CSM and non-linear time history analysis

	4 Storey building	8 Storey building
Ductility (Pushover)	4.6	2.6
Ductility (CSM)	1.55	1.12
Maximum top displacement (mm)-CSM	85.20	212.6252
Maximum Average top displacement(mm) - time history analysis	90.81	182.224
Maximum top displacement at plastic mechanism (pushover analysis)	202.86	497.11

6. Summary and Conclusion

Seismic performance evaluation is one of the important steps in performance based seismic design procedure. In this research, capacity spectrum method was used to estimate critical performance parameters for SPSWs. Displacement demand and ductility demand were calculated from the intersection point of the capacity curve and demand curve. Displacement demands in CSM for both of the structures were close to the non-linear time history analysis results. The error is prediction for the displacement demands was only 6% for 4-storey SPSW and 16% for 8-storey SPSW. On the other hand, ductility demands were lower than the design consideration and non-linear pushover analysis results. Lower ductility demand in CSM was because of over strength of the structures. In the SPSW design, practical plate thickness availability was considered in the design consideration.

Thus, capacity-spectrum method can be used for performance evaluation and rapid design assessment for SPSW system to get a global idea of the building performance instead of non-linear time history analysis. However, Capacity spectrum method needs to assume a displacement shape and a lateral load pattern for nonlinear pushover analysis. In this method, first elastic vibration mode shape was used as an assumed mode shape. Therefore, this method cannot include higher mode contribution in the overall building performance. Therefore, this method is suitable for such structure, which is mainly dominated by its fundamental mode of vibration. This is also reflected in this study, as with the use of capacity-spectrum method, inelastic displacement demand of 4-storey SPSW predicted better than 8-storey SPSW that vibrates in higher modes.

7. Acknowledgements

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