Seismic Performance of C-PSW

Alireza Yadegari
Architecture Associate's student, Islamic Azad University of Varamin Branch-Pishva,Varamin,Iran
alirezayadegari@yahoo.com

Ghasem Pachideh
Phd Candidate, civil Department, Semnan University, Semnan,Iran
Ghasem_civil@yahoo.com

Majid Gholhaki
Assisstant Professor, civil Department, Semnan University, Semnan,Iran
mgholhaki@semnan.ac.ir

Morteza Shiri
Islamic Azad University of Varamin Branch-Pishva,Varamin,Iran
m.shiri1400@gmail.com

Abstract
The study also shows that design axial forces and moments in the boundary columns designed according to capacity design concepts are in good agreement with those of the nonlinear seismic analyses. A series of C-PSWs with different geometry are designed and analysed to evaluate the current period formula in building codes. It is observed that the current code predicts periods that are generally shorter than the periods obtained from finite element analysis. An improved simple formula for estimating the fundamental period of C-PSW is developed by regression analysis of the period data obtained from analysis of the selected C-PSWs. Finally, two equations for determining shear stud spacing and thickness of reinforced concrete panel for the C-PSWs are proposed.The study also shows that design axial forces and moments in the boundary columns designed according to capacity design concepts are in good agreement with those of the nonlinear seismic analyses. A series of C-PSWs with different geometry are designed and analysed to evaluate the current period formula in building codes. It is observed that the current code predicts periods that are generally shorter than the periods obtained from finite element analysis. An improved simple formula for estimating the fundamental period of C-PSW is developed by regression analysis of the period data obtained from analysis of the selected C-PSWs.
Keywords: Composite plate shear wall, Seismic analysis, Fundamental period, Shear stud spacing

Introduction

Reinforced concrete shear walls have been widely used as lateral load resisting system in the past in high-rise buildings, but there were always concerns on the local strength, ductility and construction efficiency of these systems in steel high-rise buildings, especially in high seismic zones. In recent years, more and more steel plate shear walls have been used with satisfactory results on construction efficiency and economy. Yet there were still concerns on overall buckling of the steel plates that will result in reduction of the overall shear strength, stiffness and energy dissipation capacity [1], as well as large inelastic deformation of the steel plates that will result in large cyclic rotations of the moment connections and large inter-story drifts [2]. On the other hand, composite shear walls might compensate for the disadvantages of reinforced concrete shear walls and steel shear walls and combine the advantages together. The composite shear walls have been used recently in a few modern buildings including a major hospital in San Francisco [3], but not as common as the other lateral load resisting systems. Therefore, seismic behavior of these systems and corresponding design guidelines are of high interest to design engineers. As a result, a project was conducted at the University of California, Berkeley to investigate the seismic behavior of two composite shear wall systems through large scale cyclic tests and advanced finite element analyses. The objective of this research is to investigate the inelastic dynamic response of C PSWs when subjected to severe ground motions, and thereby evaluate the degree to which the design procedures achieve the desired behaviour. This paper presents the results, such as shear distribution between steel plate, columns, and the concrete panel, design forces of boundary columns and interstorey drifts, of nonlinear dynamic analyses of a typical 4-storey and a 6-storey C-PSW designed according to capacity design provisions, when subjected to compatible earthquake ground motions of Vancouver, Canada.

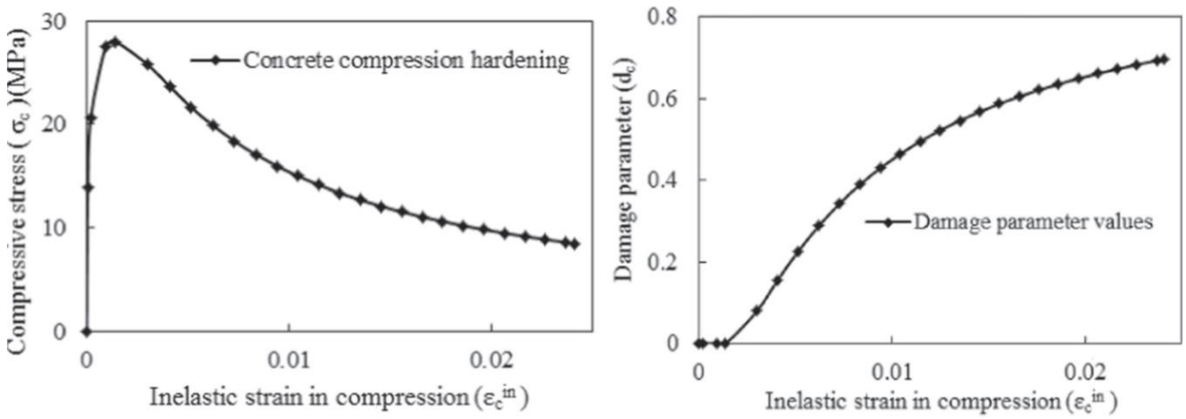
Validation of finite element model

The finite element model (FEM) has been validated by comparing the results from available test. Very few experimental works have been reported using composite shear walls. In this study, the finite element model has been validated against the composite plate shear wall test conducted by Zhao and Astaneh-Asl [4]. Between their two test specimens, traditional and innovative C-PSWs, Zhao and Astaneh-Asl [4] reported that the innovative specimen behaved in a more ductile manner and also for the innovative system, damage to the concrete panel under relatively large cycles was much less in comparison to the traditional system. Thus, only the innovative test specimen, which had 32 mm gap between the edges of the concrete wall and the surrounding boundary steel frame, was considered in this research. The test specimen was a single bay structure with a steel moment resisting frame as the boundary members and composite shear walls embedded inside the moment resisting frame. The composite shear wall consisted of a steel plate shear wall and a reinforced concrete shear panel bolted to each other. The specimen considered was of three storeys with the top and bottom panels of the specimen represented two half storeys while the middle two panels represented two whole stories. Details of the test specimen can be obtained elsewhere [4].

The innovative C-PSW specimen was modelled in ABAQUS and a pushover analysis was carried out. The material properties were chosen as the one reported by the authors' work like yield strength of boundary steel members as 350 MPa and that of infill steel plate as 248 MPa. The concrete had a minimum fc′ of 28 MPa. A reinforcement ratio of 0.92% was maintained and 13 mm diameter A325 bolts were used to connect the reinforced concrete (RC) panels with the steel infill plate in accordance with the test specimen. As in the test, displacement loading has been applied through the centre line of the top beam level. The displacement was increased to a maximum value as obtained from the envelope of hysteresis curve of physical test.

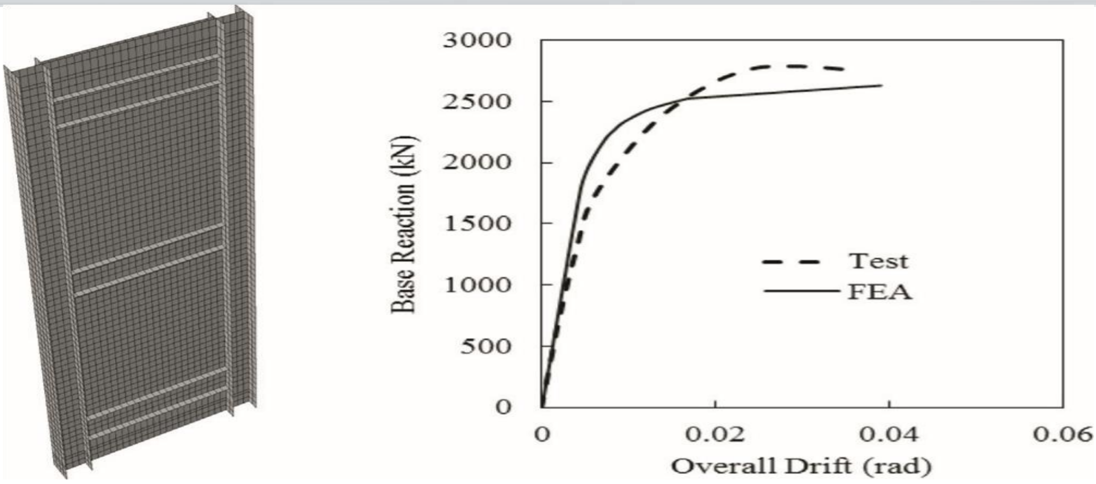
The element mesh of the composite plate shear wall is shown in Fig. 2(a). The measured (obtained from physical experimentation) and predicted (from FEA) base shear values are plotted against the overall storey drifts in Fig. 3(b). The figure indicates that the finite element model predicts the initial stiffness and post-yield response of the shear wall very well. The specimen behaved elastically up to overall drift levels of approximately 0.4%. At overall drift value of 0.6%, the experimental specimen showed yielding of all three horizontal beams and some yielding at column base. The finite element model (FEM) exhibited similar behaviour at this drift level. At overall drift level of 1.2%, the experimental specimen developed local buckling and yielding in the infill steel plates. At drift level of 2.4%, the experimental middle and bottom beams started to form web and flange local buckling. Similar behaviour was captured by the FEM at these drift levels. The ultimate capacity of the specimen is underestimated by about 6%. The finite element model was also validated by comparing cyclic analysis results with the test results of the quasi-static cyclic test conducted by Zhao and Astaneh-Asl [4]. Hysteresis curves obtained from the finite element analysis were compared with the test results in Fig. 3. The hierarchical modes of failure and yielding of different components of the test specimen were compared with that of the finite element model and close correlation was observed. The slight differences between the results from the test and the FE model might be due to the small differences in the actual experimental set up and that of the FE model. Also, detailed stress–strain curves for the steel sections used in the test were not reported and only bilinear behaviour of the steel materials was assumed.

Further validation of the finite element model was carried out by comparing cyclic analysis results with the test results of the quasi-



(a) Concrete compression hardening curve **(b) Concrete compression damage curve**

Fig. 1. Concrete damage plasticity model: (a) concrete compression hardening curve; (b) concrete compression damage curve.



(a) FE Mesh **(b) Push-over curves**

Fig. 2. Validation of Zhao and Astaneh-Asl (2004) innovative specimen: (a) FE mesh; (b) pushover curves.

static cyclic test conducted by Driver et al. [5]. Driver et al. [5] tested a 4storey steel plate shear wall (similar to C-PSW, but without the concrete panel) under quasi-static cyclic loading. Details of the test specimen are available in the literature [5]. Hysteresis curves obtained from the finite element analysis were compared with the test results in Fig. 4. In general, there is good agreement between the test results and the finite element analysis. Both the predicted capacity and stiffness of the SPSW are in excellent agreement with the test results. The hysteresis curves generated from FE analysis show slightly less pinching than that observed during the test.

One of the important factors for any seismic lateral load-resisting system is the correct estimation of seismic response factor, R. In Canada, two different factors, Rd : ductility-related force modification factor and R0 : over-strength-related force modification factor, are used in seismic design of structures (NBCC 2010). Researchers have so far proposed different methodologies for derivation of ductility-related force modification factor. Newmark and Hall [31] derived a relationship between the ductility-related force modification factor, Rd, and the ductility ratio, μ, according to the period of a structure.

Rd ¼ μ for T N 0:5s ð1Þ (1)

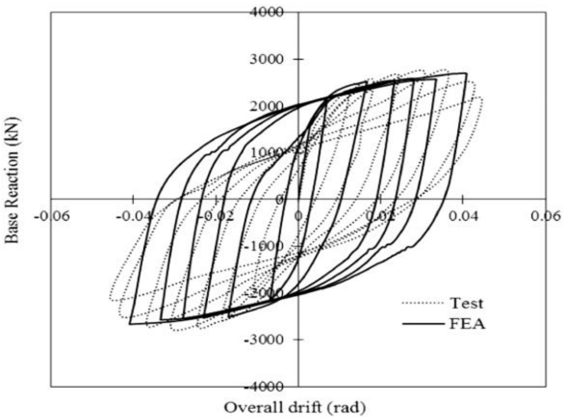


Fig. 3. Validation of cyclic curves for Zhao and Astaneh-Asl (2004) innovative specimen.

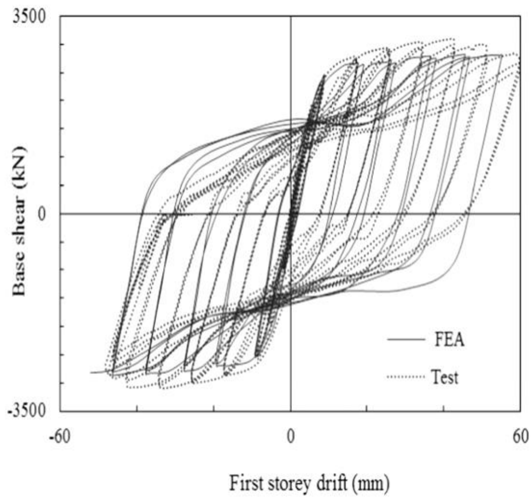


Fig. 4. Validation of cyclic curves for Driver et al. (1998) SPSW test.

$$R_d = \sqrt{2\mu - 1} \text{ for } 0.1 < T < 0.5 \text{ s} \quad (2)$$

$$R_d = 1 \text{ for } T < 0.03 \text{ s} \quad (3)$$

Ductility ratio of a structure, μ, is defined as ratio of maximum lateral displacement (Δmax) or displacement at failure to lateral displacement at yield (Δy).

$$\mu = \frac{\Delta_{max}}{\Delta_y}$$

Thus, in order to obtain ductility of a structural system, it is important to identify yield and maximum displacements of the structure from a force deformation relationship. Park (1988) proposed that displacement corresponding to first significant yielding could be considered as the yield displacement of the structure. It was also suggested that displacement corresponding to the post-peak displacement when the load-carrying capacity undergoes a small reduction (often taken as 10%–20%) might be considered as the maximum displacement of the structure [6]. The suggestions made by Park (1988) are considered in this study. Test-based ductility-related force modification factor was estimated from the force deformation relations of both traditional and innovative C-PSWs tested by Zhao and Astaneh-Asl [4]. Fig. 5 presents the cyclic envelopes of the two specimens tested by Zhao and AstanehAsl [4]. For both specimens, the overall drift value of 0.006 rad was established as the 'Significant Yield Point' as at this drift level, some yield lines appeared on the beams as well as in the column bases. Shear strength of the innovative specimen dropped to about 80% of the maximum shear strength of the specimen at an overall drift level of 0.044 rad, and the specimen was considered failed. In case of traditional C-PSW, test showed that the strength dropped to about 80% of the ultimate shear strength at a drift level of 0.042 rad. These values (0.044 rad and 0.042 rad) of overall drift levels, as indicated in Fig. 5, were considered the maximum overall drifts to reach 'Points of Maximum Ductility.' Using the relation between maximum drift to yield drift as presented in Eq. (4), the overall ductility values for Innovative and Traditional C-PSW specimens were calculated as 7.33 and 7.0, respectively. Assuming that the natural periods of vibration of the 4-storey and 6-storey C-PSWs studied in this research greater than 0.50 s, which is verified later from frequency analysis, the Rd values could be selected for the selected C-PSWs with Eq. (1). In the current edition of National Building Code of Canada (NBCC 2010), Rd factor ranges from 1.0 for brittle systems such as unreinforced masonry to 5.0 for the most ductile systems. It is believed that this range is realistic for building structures [7,8]. NBCC 2010 [9] and CSA S16-09 [10] assign the highest ductility-related force modification factor, Rd, of 5.0,

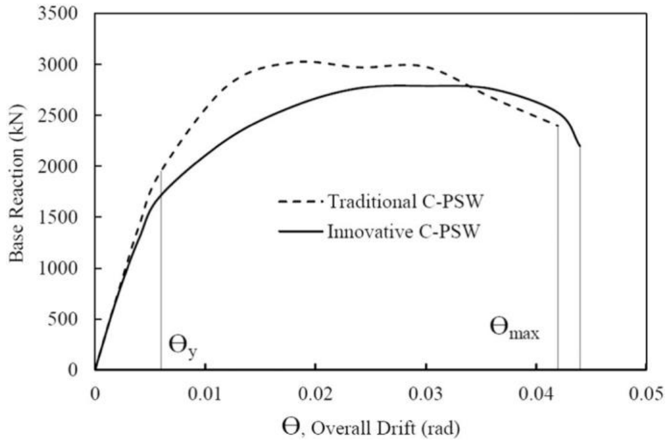


Fig. 5. Load deformation relations of C-PSW tests by Zhao and Astaneh-Asl (2004).

to ductile SPSW. In both SPSW and C-PSW, the hierarchical modes of failure and yielding are same: steel infill plate yielding is considered as the main ductile fuse, followed by yielding at the end of steel beams and finally plastic hinging at the base of columns. Thus, based on the results of the test program by Zhao and Astaneh-Asl [4] and in the absence of any provision for C-PSW in Canada, similar to the provision for ductile SPSW, a ductility-related force modification factor, R_d, of 5.0 is used for design of C-PSWs.

Seismic design of composite plate shear walls

Selection of composite plate shear walls

The buildings considered here for seismic analysis are one 4-storey and one 6-storey hypothetical office building located in Vancouver having a plan area of 2014 m^2. Fig. 6(a) shows typical floor plan of the hypothetical buildings considered for seismic analysis. As shown in the plan, each of the buildings has two identical C-PSWs to resist lateral forces in each direction; thus, each composite shear wall will resist half of the design seismic loads. Only innovative C-PSW system was considered in this study. The C-PSW under consideration for seismic analysis is designated as C-PSW1. For simplicity, torsion was neglected. Each C-PSW was 3.8 m wide, measured from centre to centre of columns, and had an aspect ratio of 1.0 (storey height of 3.8 m). Thus, the 4storey building had a total height of 15.2 m and the 6-storey building had a total height of 22.8 m. The buildings were assumed to be founded on very dense soil or soft rock (site class C according to NBCC 2010). A dead load of 4.26 kPa for each floor and 1.12 kPa for the roof were used. The live load on all floors was taken as 2.4 kPa and no live load was considered at the roof level. NBC 2010 [30] recommends use of load combination ‘1.0 D + 1.0 E + 0.5 L or 0.25 S’ (where, D = dead load, L = live load, S = snow load, and E = earthquake load) when earthquake load is present. Thus, load combination ‘D + 0.5 L + E’ was considered for floors, and for the roof, the load combination ‘D + 0.25 S + E’ was considered. A steel plate thickness of 4.8 mm was used as the minimum practical thickness based on requirements to be bolted with the reinforced concrete panels and handling issues. 13 mm diameter A325 bolts were selected for connecting the steel infill plate with the RC panel.

Design of composite plate shear walls

In order to design the C-PSWs, the equivalent static force method was employed to find out the storey shear forces at each storey according to NBCC 2010 [11]. The design seismic base shear (V) calculated according to NBCC 2010 is as follows:

$$V = \frac{S(T_a) M_V I_E W}{R_d R_0} \geq \frac{S(2.0) M_V I_E W}{R_d R_0} \quad (4)$$

where $S(T_a)$ is the spectral acceleration; M_V is an amplification factor accounting for higher mode effects on base shear; I_E is the importance factor for the structure; W denotes the total dead load in addition to 25% of the snow load; similar to ductile SPSW, an over-strength force modification factor R_0 of 1.6 was used in the design of C-PSW. According to the NBCC 2010, for structures having R_d greater than 1.5, the design base shear should assume a maximum value as:

The final base shear calculated was distributed at each storey of the structure as

$$F_X = (V - F_t)\frac{W_X h_X}{\sum_{i=1}^{i=n} W_i h_i} \quad (5)$$

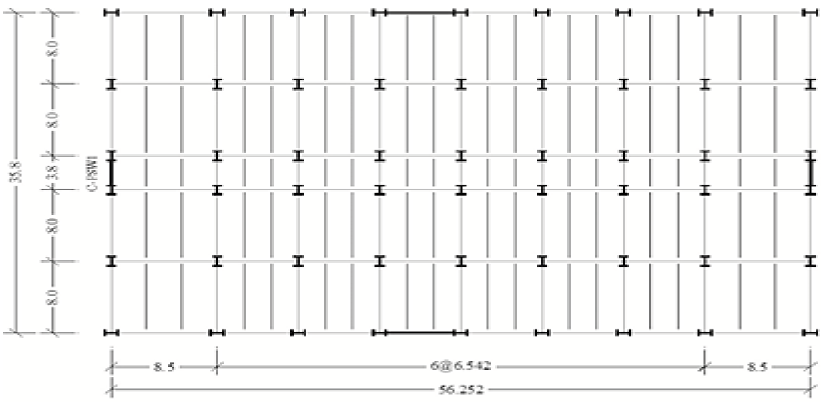
where F_t is an extra lateral force component applicable to the top floor; W_i or W_x denotes the dead load in addition to 25% snow load applicable to the storey ior x and h_x or h_i denotes the height from the base to the storey level i or x, respectively. The equivalent static lateral forces determined based on the NBCC 2010 for the 4-storey C-PSW were 152.5 kN, 305.1 kN, 457.7 kN, and 206.3 kN for the first storey, second storey, third storey, and roof, respectively. The lateral forces determined for the 6storey C-PSW were 104.2 kN, 208.4 kN, 312.6 kN, 416.8 kN, 521.1 kN, and 211.3 kN for the first storey, second storey, third storey, fourth storey, fifth storey, and roof, respectively. AISC 341-10 [12] requires that the steel infill plates of C-PSWs be designed as the main energy dissipating elements. The design shear strength of the plate is based on the shear yielding of the stiffened steel plate and is given by

$$V_r = \phi 0.6 A_{sp} F_y \quad (6)$$

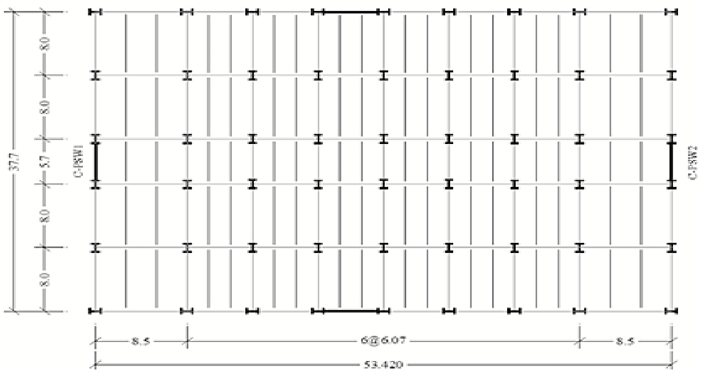
where ϕ=0.9; A_{sp} is the horizontal area of the stiffened steel plate; F_y is the specified yield stress of the steel plate.

Thus, the steel infill plates can be selected to resist the total seismic load calculated using equivalent lateral force method in NBCC 2010. As per the capacity design method in AISC 341-10 [12], the beams and columns of the C-PSW shall be designed for the expected strength of the steel infill plates in shear, $0.6A_{sp}R_yF_y$, where R_y=1.1 and the beams and columns adjacent to the composite webs shall be designed to remain essentially elastic under the maximum forces that are developed by the fully yielded steel infill plates, except that plastic hinging at the ends of beams is allowed. Also, plastic hinges are allowed at base of the boundary columns.

Boundary members for the C-PSW were designed according to the capacity design approach similar to what was proposed by Berman and Bruneau [13] for ductile SPSW. AISC 341-10 recommends adequate stiffening of the steel infill plate by encasement of the steel plate or attachment with an RC panel. The concrete panel was selected as per provisions of AISC 341-10, which was selected to be of 200 mm thickness and reinforcement ratio of 0.0025 was maintained with the bar spacing not exceeding 450 mm to comply with the minimum requirements. A shear stud spacing of 300 mm was selected for all the C-PSWs. The shear stud spacing and the thickness of reinforced concrete panel used for the C-PSWs were also checked based on the equations, developed later using the concepts of classical buckling theory of stiffened steel plate. The selected C-PSWs are shown in Fig. 6.



(a) For C-PSWs with aspect ratio 1.0



(b) For C-PSWs with aspect ratio 1.5

Fig. 6. Floor plans of sample buildings: (a) for C-PSWs with aspect ratio 1.0; (b) for C-PSWs with aspect ratio 1.5.

Nonlinear dynamic analyses of composite plate shear walls 5.1. FE model and initial conditions

FE model and initial conditions

The selected C-PSWs were modelled in ABAQUS. A mesh sensitivity study was conducted in order to help determine the effect of mesh size on the performance and behaviour of the C-PSWs. Element dimensions were varied from 80 to 300 mm at the steel plate region and suitably at the boundary elements based on the dimension available. It was observed that the mesh size in the above range did not affect the local or global performance of the C-PSWs. Hence, a mesh of approximately 300 mm in element dimension at the steel infill plate region was used for the nonlinear dynamic analysis.

The nominal yield strength of steel infill plates, boundary columns, and beams were selected as 350 MPa and all steel members were assumed to have a modulus of elasticity of 200,000 MPa. The concrete was selected to have compressive as well as tensile damage and had a compressive strength of 28 MPa. Frequency analyses for the C-PSWs were carried out prior to seismic analyses to find out the fundamental mode shapes and frequencies for the C-PSWs. A dummy gravity column was incorporated into the finite element model to take account of P-Δ effects. Fig. 7 presents analytical model for 4-storey C-PSW. In this model, the gravity column was made of 2-node linear 3-D truss (ABAQUS element T2D3) elements and was connected with the CPSW at every floor with pin-ended rigid link connections. Thus, at each floor, the horizontal degree of freedom of the gravity column was constrained to be the same as that of the C-PSW to maintain displacement compatibility of structural members interacting through rigid floor diaphragms. The gravity column was designed so as not to provide any lateral stiffness and it carried half of the total remaining mass of the building since there are two C-PSWs in each mutually perpendicular directions of the building plan. From frequency analyses, the first two mode periods (in-plane) of the 4-storey C-PSW (aspect ratio 1.0) were obtained as 0.63 s and 0.20 s, respectively. For 6-storey C-PSW.

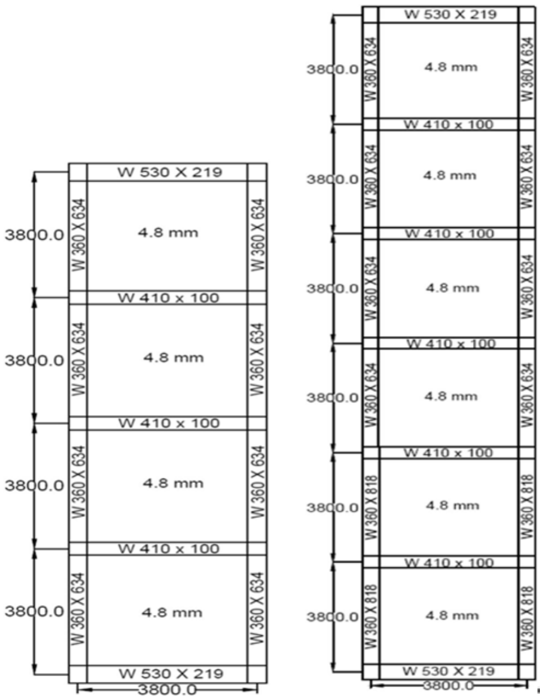


Fig. 6. 4-storey and 6-storey C-PSWs (aspect ratio 1.0)

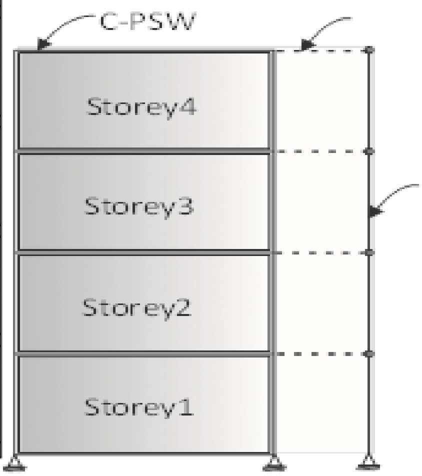


Fig. 7. Analytical model for 4-storey C-PSW

the first two in-plane periods were 1.16 s and 0.31 s, respectively. These periods were used to determine Rayleigh proportional damping constants for 4-storey and 6-storey C-PSWs. A 5% Rayleigh proportional damping was assumed in the first two modes of vibration, which included a cumulative modal mass equal to more than 90% of the total mass applied on the C-PSW.

Seismic response of C-PSWs

Nonlinear time history analyses were performed in ABAQUS. Under all earthquake records, the 4-storey C-PSW behaved in a stable and ductile manner. The RC panels were capable of successfully restraining out-of-plane motion of the steel infills and were undamaged under all ground motions except for one record (San Fernando earthquake), where minor damage was identified at the first storey. Fig. 9 presents. the average peak storey shears for 4-storey and 6-storey C-PSWs.

Table 1. Ground motion parameters of selected real ground motions.

Event name	Magnitude	Site	Maximum acceleration A (g)	A/V	Scaling factors	
					4-storey	6-storey
Kobe, Japan, 1995	6.6	HIK	0.143	0.968	1.84	1.60
San Fernando, California, 1971	6.61	La-Hollywood Stor. LOT	0.188	1.04	1.65	1.53
Imperial Valley, California, 1979	6.53	Aeropuerto Maxicali	0.3118	1.03	1.25	0.93
Imperial Valley, California, 1979	6.53	El-Centro array	0.525	1.04	0.99	1.0

under the selected artificial and real ground motions and the contributions by the various components of the C-PSWs, namely, the steel infill, boundary columns, and the RC panel. For 4-storey C-PSW under simulated earthquake records, the maximum base shear was found as 5390 kN, obtained for 6C2 earthquake record. The peak storey shear contributions by the boundary columns and the RC panel at the base, for 6C2 record, were 27% and 10%, respectively. As observed from Fig. 10, for 4-storey C-PSW under simulated earthquake records, the average shear contributions by the columns and the RC panel at the base, are 23.5% and 10%, respectively. Storey shear percentage contributions by the RC panels for higher stories were observed to be practically insignificant. For the 4-storey C-PSW, under real earthquake records, the maximum base shear was found as 5170 kN for Imperial Valley 2 record. For this earthquake record, the storey shear contributions at the base by the boundary columns and the RC panel were observed as 25% and 10%, respectively. Fig. 9 also shows that, for 4-storey C-PSW under real earthquake records, the average shear contributions from the columns and the RC panel at the base, are 22% and 10.8%, respectively. For 4-storey C-PSW, for all ground motions, steel infill plates for the first and second storey fully yielded. This is also observed from Fig. 9 as the average dynamic shears for the bottom two storeys of 4-storey C-PSW are very close to the nominal shear strength of the plate web, 3353 kN, as calculated by Eq. (8).

The 6-storey C-PSW also behaved in a ductile and stable manner. For all the earthquake records except for 7C2 earthquake record, steel infill plates of the bottom three floors were yielded. Yielding in infill plates occurred when the dynamic shears reached or exceeded the nominal shear strength of the plate web of 6-storey C-PSW, 3312 kN, as calculated by Eq.

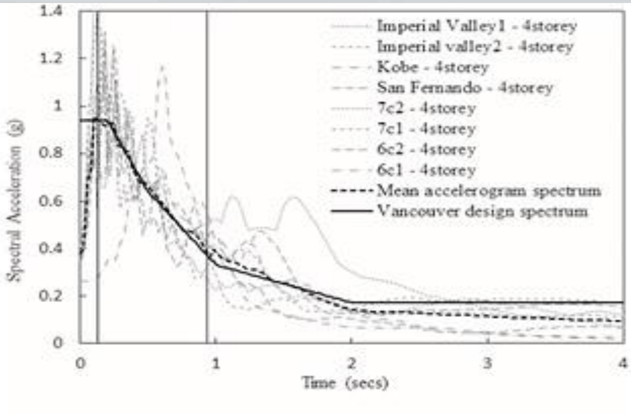
(8). For 7C2 earthquake record, infill plate at the fourth floor also yielded. Fig. 9 presents the average peak storey shears for 6-storey C-PSW under the selected ground motions. The maximum dynamic base shear was found as 5313 kN, obtained for 7C1 earthquake record. The peak storey shear contributions by the boundary columns (observed for 7C1 record) and the RC panel (observed for 7C2 record) at the base were 29% and 8.5%, respectively. As observed from Fig. 9, for 6-storey C-PSW under simulated earthquake records, the average shear contributions by the columns and the RC panel at the base are 26% and 6%, respectively. Similar to 4-storey C-PSW, storey shears taken by the RC panels in higher stories were very small. For the 6storey C-PSW, under real earthquake records, the maximum base shear was found as 5285 kN for Imperial Valley 2 earthquake record. For this earthquake record, the storey shear contributions at the base by the boundary columns and the RC panel were observed as 26% and 9.5%, respectively. Fig. 9 also shows that, for 6-storey C-PSW under real earthquake records, the average shear contributions from the columns and the RC panel at the base are 21% and 8.5%, respectively.

It can be clearly observed from Fig. 9 that NBCC 2010 static base and storey shear forces calculated are much lower than those from seismic analysis. This is mainly due to the over-strength in the C-PSWs caused by the use of thicker steel plates than required due to handling and practical requirements. Also, a significant portion of shear is taken by boundary columns and reinforced concrete panels, which is not considered in the current design approach of C-PSW since total shear is assumed to be resisted by the steel infill plates only.

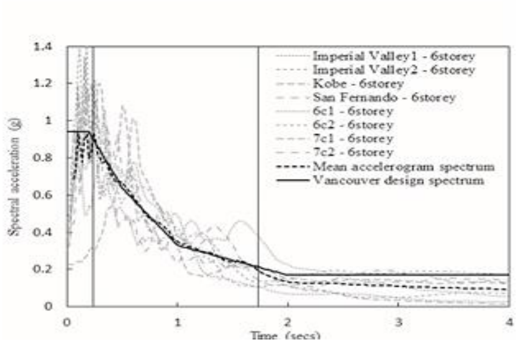
For some cases, very small partial yielding was observed in the outer flanges of steel boundary columns at the base, thereby achieving design objective of C-PSW to sustain the full yield force from the steel infill plates. For the 6-storey C-PSW, RC panels were essentially undamaged except for two earthquake records (Imperial Valley 2 and San Fernando earthquakes) where small amount of micro-cracking was observed. Microcracks were concluded based on plastic strain in tension(ABAQUSPEEQT) output values corresponding to concrete strain in tension beyond the point of maximum tensile strength based on the Belarbi and Hsu [14] concrete constitutive model in tension.

Table 2. Parameters of selected simulated earthquake record.

Earthquake event name	Magnitude	Maximum acceleration A (g)	Maximum velocity V (m/s)	A/V	Scaling Factors	
					4-storey	6-storey
6C1	6.5	0.345	0.26	1.33	0.71	0.76
6C2	6.5	0.35	0.266	1.32	1.31	1.44
7C1	7.5	0.426	0.406	1.05	0.79	0.89
7C2	7.5	0.409	0.445	0.92	1.72	1.81



(a) for 4-storey C-PSW



(b) for 6-storey C-PSW

(c) **Fig. 8. Acceleration spectra for selected accelerograms and design spectra for Vancouver: (a) for 4-storey C-PSW; (b) for 6-storey C-PSW.**

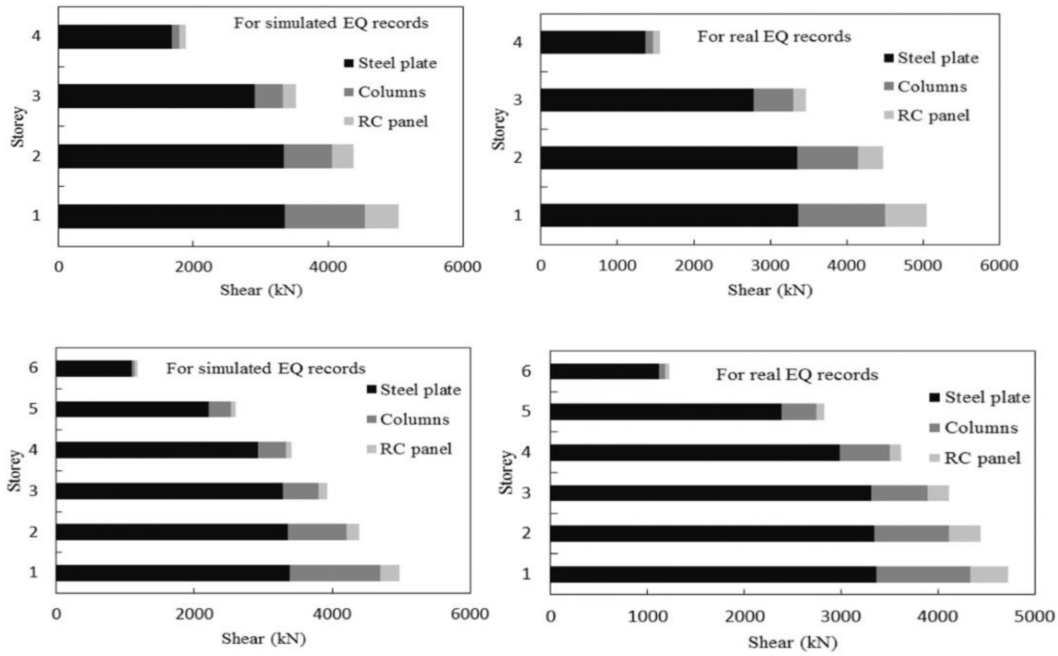


Fig. 9. Average peak storey shear contributions of 4-storey and 6-storey C-PSWs.

Summary and conclusions

Nonlinear seismic analyses under earthquake ground motions typical of Western Canada were performed to evaluate the performance a typical 4-storey and 6-storey composite plate shear wall. The analyses provided information on the shear and flexural demand on the lateral load-resisting system. The key findings from this study are as follows:

(1) The finite element model developedwas found to provide excellent correlation with the experimental specimen in quasi-static pushover and cyclic analysis. The model captured all essential behavioural features of the test specimen analysed.

(2) The 4-storey and 6-storey C-PSW finite element specimens analysed under a set of eight strong earthquake records were found to provide excellent structural performance in terms of stiffness, ductility, and high shear strength accompanied by minimal damage in terms of concrete cracking and crushing. It was observed from the seismic analyses that the boundary columns and RC-panel together can contribute towards a significant amount of shear strength, as much as 30% (more than 20% of total shear strength is resisted by columns), which is ignored in the current AISC 341-10. This shall be acknowledged in the current code and as such, beams at every storey of C-PSW must have sufficient flexural resistance such that at least 20% of the applied factored storey shear force can be resisted by the boundary moment resisting frame.

(3) No plastic hinges were formed at the boundary columns, which were capacity designed. Design column moments and axial forces were shown to agree well with the results from the nonlinear seismic analyses of the selected C-PSWs, while providing slightly conservative results.

(4) The interstorey drifts obtained from the nonlinear time history analyses were well within the NBC 2010 limit of 2.5% of the interstorey height.

(5) It can be observed fromthe frequency analyses of the selected CPSWs that the current code formula predicts periods that are generally shorter than those obtained from detailed finite element analysis.

References

[1] Zhao Q., Astaneh-Asl A, (2004). “Cyclic Behavior of an Innovative Steel Shear Wall System”. Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada.

[2]Allen H.G., Bulson P.S. (1980). Background to Buckling. McGraw Hill Book Company, U.K.

[3]Dean R.G., Canon T.J., Poland C.D. (1977). “Unusual Structural Aspects of H.C. Moffit Hospital.” Proceedings of the 46th Annual Convention, SEAOC, Coronado, CA.

[4] Zhao Q, Astaneh-Asl A. Cyclic behavior of traditional and innovative composite shear walls. ASCE J Struct Eng 2004;130(2):271–84.

[5] Driver RG, Kulak GL, Kennedy DJL, Elwi AE. Seismic behaviour of steel plate shear walls.” Structural engineering report no. 215. Edmonton, Canada: Department of Civil and Environmental Engineering, University of Alberta; 1997.

[6] Park R, Paulay T. Reinforced concrete structures. New York: John Wiley & Sons; 1975.

Paulay T, Priestley MJN. Seismic design of reinforced concrete and masonry buildings. New York: John Wiley & Sons; 1992.

[7] NBCC. National Building Code of Canada. National Research Council of Canada (NRCC), Ottawa, ON, Canada, 2010.

[8] Canadian Standards Association. CAN/CSA-S16-09. Limit states design of steel structures. Toronto, Ontario, Canada; 2009.

[9] Belarbi H, Hsu CTT. Constitutive laws of concrete in tension and reinforcing bars stiffened by concrete. ACI Struct J 1994;91(4):465–74.

[10] Berman JW, Capacity Bruneau M. Design of vertical boundary elements in steel plate shear walls. Engineering Journal 2008:57–71 First Quarter.

[11] Allen HG, Bulson PS. Background to buckling. U.K.: McGraw Hill Book Company; 1980

[12] Park R. Ductility evaluation from laboratory and analytical testing. Proceedings of the 9th world conference on earthquake engineering. Japan: Tokyo; 1988. p. 605–16.

[13] Paulay T, Priestley MJN. Seismic design of reinforced concrete and masonry buildings. New York: John Wiley & Sons; 1992.

[14] AISC. 341–10. American Institute of Steel Construction, Chicago, IL: Seismic provisions for structural steel buildings; 2010.