Performance-Based Plastic Design of Type D Steel Plate Shear Walls

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Abstract: Performance-Based Plastic Design (PBPD) method is developed for the seismic design of ductile Steel Plate Shear Wall (SPSW) with moment resisting beam-to-column connections. In this method, pre-selected target drift and desirable yield mechanism of the plate walls are used as key performance criteria. The design base shear is obtained based on energy-work balance for the plate walls and an equivalent Elastic-Plastic Single Degree of Freedom (EP-SDOF) system to achieve the same target drift. Plastic design is performed to obtain the required strength of the infill panels, followed by the capacity design of boundary frame elements. To achieve an efficient design, the contribution of boundary moment frame in resisting the lateral loads is also taken into account in the proposed design procedure. To investigate the effectiveness of the proposed design procedure, a series of nonlinear time-history analyses of eight storey and fifteen storey Type D (ductile) SPSWs is performed under spectrum compatible earthquake records for Vancouver. The results of nonlinear analyses indicate that the SPSW designed according to the PBPD procedure performed well under the selected ground motions. Furthermore, comparison of the results between SPSWs designed using the PBPD method and that of designed according to Canadian code showed that an economical design can be achieved using the proposed design procedure.

1. INTRODUCTION

Steel Plate Shear Walls (SPSWs) are emerging systems used as primary lateral load resisting systems for buildings in high seismic areas. A typical steel plate shear wall (SPSW) consists of infill steel panels surrounded by columns, called vertical boundary elements VBEs and beams, called horizontal boundary elements HBEs. In current Canadian seismic design practice, the design base shear for the design of Steel Plate Shear Walls (SPSWs), similar to other structural systems, is obtained from code specified spectral response acceleration, assuming the structures to behave elastically. It is then modified using code specified ductility-related force modification factor (R_d), and overstrength-related force modification factors (R_0) to account for the inelastic behavior.

It is expected that SPSWs will experience large inelastic deformation when subjected to severe earthquake ground motions. However, the current seismic design procedure is based on the elastic analysis approach and does not account for the inelastic response of the structure in a direct and explicit manner. This emphasizes a need for developing a systematic design approach, which accounts for inelastic behavior directly, and results in a more predictable seismic performance. Goel and Chao (2008) proposed a design methodology aiming to achieve the above mentioned goals. In the proposed Performance Based Plastic Design (PBPD) procedure, the inelastic behavior of the structure is explicitly taken into account in the design process. The PBPD method, in which pre-selected target drift and yield mechanisms are used as key performance objectives, has been successfully applied to the seismic design of steel Moment Frame (MF), Reinforced Concrete (RC) moment frames, Eccentrically Braced Frame (EBF), buckling restrained braced frame (BRBF), Special Truss Moment Frame (STMF), and concentric braced frames (CBF). Ghosh *et al.* (2009) and Bayat (2010) investigated the PBPD procedure for design of SPSW structures. However, their studies were limited to the design of SPSWs with simple (pinned) HBE-to-VBE connections. In CSA S16-09 Standard (2009), SPSWs are classified as Type D (ductile) and Type LD (limited-ductility) plate walls. The design requirements for these two plate wall types are described in Clause 27 of the standard. This standard requires the HBE-to-VBE connections to be rigid for the Type D plate walls. Simple HBE-to-VBE connections are permitted for Type LD plate walls although with significantly lower ductility-related force modification factor, R_d = 2.0 versus 5.0 and less overstrength-related modification factor, R_0 = 1.5 versus 1.6 for Type D plate walls. However, SPSWs with simple beam-to-column connections are not allowed in the AISC seismic provision (American Institute of Steel Construction 2010) and they are required to be designed using moment resisting boundary frames.

In conventional design of SPSWs according to the North American design standards, the infill plate at every level is designed to resist 100% of the factored storey shear force. Hence, the lateral resistance contribution provided by boundary frame moment resisting action is neglected which typically results in a conservative and likely more expensive design. Large-scale test of a multi-storey steel plate shear wall conducted by Driver *et. al.* (1997) and subsequent experimental and analytical investigations have shown that the lateral resistance contributions provided by the boundary moment frame in overall strength of SPSW is significant. Qu and Bruneau (2009) following the plastic analysis of SPSWs investigated the relative and respective contributions of infill plates and boundary frame moment resisting action in overall strength of the system. The researchers introduced a balanced design case in which the strength provided by the boundary moment frame of the SPSW is precisely equal to the resistance required if the boundary frame is designed to anchor the fully yielded infill plates using the capacity design approach. In such design case system overstrength is equal to unity (Qu and Bruneau, 2009).

In this study, PBPD method is developed for the seismic design of ductile Steel Plate Shear Wall (SPSW) with moment resisting beam-to-column connections. In this method, pre-selected target drift and desirable yield mechanism of the plate walls are used as key performance criteria. The design base shear for a specified hazard level is obtained based on energy-work balance for the plate walls and an equivalent Elastic-Plastic Single Degree of Freedom (EP-SDOF) system to achieve the same target drift. Plastic design is then performed to obtain the required strength of the infill panels to achieve the pre-selected target drift and yield mechanism. To achieve an efficient design, the contribution of boundary moment frame in resisting the lateral loads is also taken into account in the proposed design procedure. To investigate the effectiveness of the proposed design procedure, a series of nonlinear response-history analysis of eight storey and fifteen storey SPSWs is performed under spectrum compatible earthquake records for Vancouver, BC. Then, the comparison of the seismic performance is made between the SPSW designed using the PBPD procedure and the SPSW designed according to the current Canadian standard CSA S16-09 (2009) and NBCC (2010).

2. PROPOSED DESIGN PROCEDURE

The detailed philosophy of PBPD procedure can be found elsewhere (Goel and Chao, 2008). A step-by-step proposed design procedure of a typical SPSW with moment resisting beam-column connections is summarized as follows:

1- Select a yield mechanism for desirable response and intended target drift ratio (θp) consistent with acceptable ductility and damage for expected hazard level. The desired yield mechanism for a typical SPSW involves uniform yielding of the infill plates over every story and plastic hinge formation in beam ends and columns bases, as shown in Figure 1.

2- Estimate the natural period of the structure, T_a. NBCC provides an empirical formula to estimate the fundamental period of the SPSWs. It has been shown that the code formula predicts periods that are generally shorter than the periods obtained using detailed finite element analysis of SPSWs. Bhowmick *et al.* (2011) proposed the following empirical formula for structures with SPSWs as the primary lateral load resisting system, where $h_n$is the height of the structure in meters.

[1] $T_a = 0.03h_n$

3- Estimate the yield drift ratio, θy, assuming idealized elastic-plastic (EP) force-displacement behavior of the structure. Calculate the plastic drift ratio, θp, subtracting the yield drift ratio from the target drift ratio.

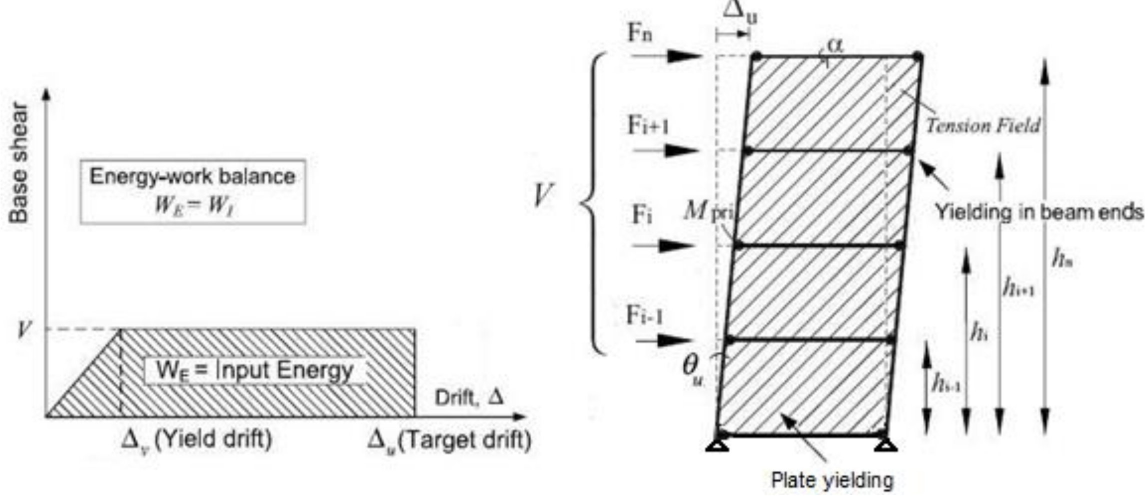


Figure 1: Energy-Work balance concept in PBPD

4- Use the lateral load distribution proposed by Chao et al. (2007), equations [2]–[4], which account for the inelastic behavior of the structure and higher modes effects. In following equation, F_i is the lateral force at each floor level; w_j and w_n are the seismic with of the j^{th} floor and roof, respectively.

[2] $F_i = C'_{vi}V$

[3] $C'_{vi} = (\beta_i - \beta_{i+1})\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}}$ where i=n, β_{n+1}=0

[4] $\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}}$

Calculate the parameter α using equation [5] considering the lateral load distribution.

[5] $\alpha = (\sum_{i=1}^{n}(\beta_i - \beta_{i+1})h_i)\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} \left(\frac{8\theta_p \pi^2}{gT^2}\right)$

5- Compute the energy modification factor, γ, using the structural ductility factor ($\mu_s = \Delta_u/\Delta_y$) and the corresponding ductility reduction factor R_μ.

[6] $\gamma = \frac{(2\mu_s - 1)}{R_\mu^2}$

It should be noted that the inelastic spectra proposed by Newmark and Hall (1982) is considered herein to build the relation between the ductility reduction factor and the structural ductility factor for the assumed EP-SDOF, as shown in Figure 2 (Goel et al. 2008).

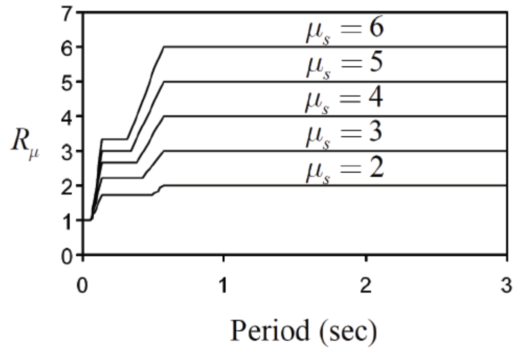
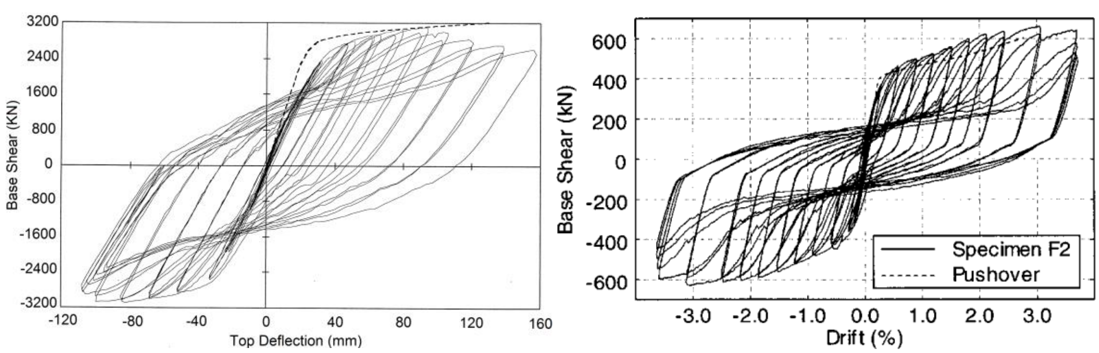


Figure 2: Relationship between R_μ, μ_s and T

6- Calculate the design base shear using equation [7], where V is the design base shear, W is the total seismic weight of the structure and S_a is the spectral response acceleration obtained from code design spectrum. Distribute the computed design base shear at various stories using the above-mentioned PBPD lateral force distribution.

$$[7] \quad \frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4(\gamma/\eta)\, S_a^2}}{2}$$

As discussed earlier, infill plates are the designated energy dissipating elements of a SPSW under seismic loading. Given the fact that the infill plate yield only in tension and has negligible compression strength, SPSWs show pinched hysteretic behavior, as shown in Figure 3, compared to energy dissipating systems such as Buckling Restrained Braced Frames (BRBFs) which develop full and stable hysteretic loops. Although the boundary moment frame of a SPSW provides complementary energy dissipation and enhances the hysteresis loops during cycle reversals, the whole system doesn't exhibit full hysteretic response under cyclic and seismic loading. Typical hysteretic behavior of SPSWs with moment resisting connections is shown in Figure 3. As the design base shear in PBPD method was originally derived assuming full hysteretic loops for the structure, reduced area of hysteretic loops can be accounted in the design by considering an energy reduction factor (η), as shown in Figure 4. Given the experimental results of the earlier research and based on a preliminary study on single storey and multiple storey SPSWs under cyclic loading, a value of η = 0.75 is suggested for the PBPD of SPSWs with moment resisting beam-column connections (see Figure 4).



(a) (b)

Figure 3: Hysteretic behavior of SPSWs with moment resisting beam-to-column connection: (a) Driver et al. (1997); (b) Berman and Bruneau (2003)

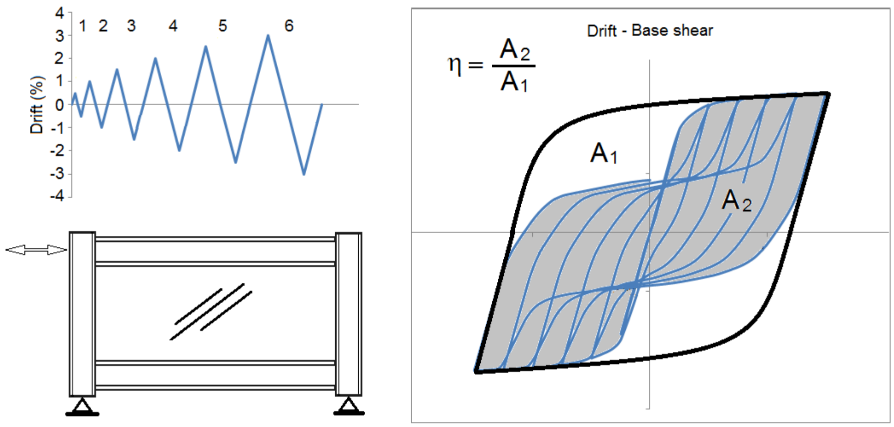


Figure 4: Dissipated energy by a typical steel plate shear wall under cyclic loading and the definition of energy reduction factor (η).

7- Estimate the contribution of the infill panel in resisting the lateral load at each storey level considering the balanced design condition (discussed earlier) using equation [8], where, κ_{bi} is the percentage of the total lateral design force assigned to the infill panel at storey level *i* in balanced condition. L and h_i are previously defined. In this equation, η is plastic section modulus reduction ratio accounting for the possible presence of reduced beam section (RBS) connections. When there is no RBS connection, the value of η is equal to unity. In case of pinned connection, which is allowed only in Type LD SPSW, the value of η reduces to zero which results in $\kappa_b = 1$ (Qu and Bruneau, 2009). It should be noted that a proper diagonal tension field angle (α) is to be assumed at this stage. Any values between 38° and 48° could be used as an initial assumption. Figure 5 shows the variation in $\kappa_{balanced}$ for different α values and infill panel aspect ratios for a single storey SPSW. It should be mentioned that the lateral loads assigned to infill panel in balanced design case (equation [8]) is calculated such that the system overstrength becomes equal to unity, and therefore boundary moment frame provides no additional lateral resistance for the system.

[8] $$\kappa_{bi} = \left[1 + \frac{1}{2}\tan^{-1}(\alpha_i)\left(\frac{L}{h_i}\right) \times \frac{\eta_i}{1+\sqrt{1+\eta_i^2}}\right]^{-1}$$

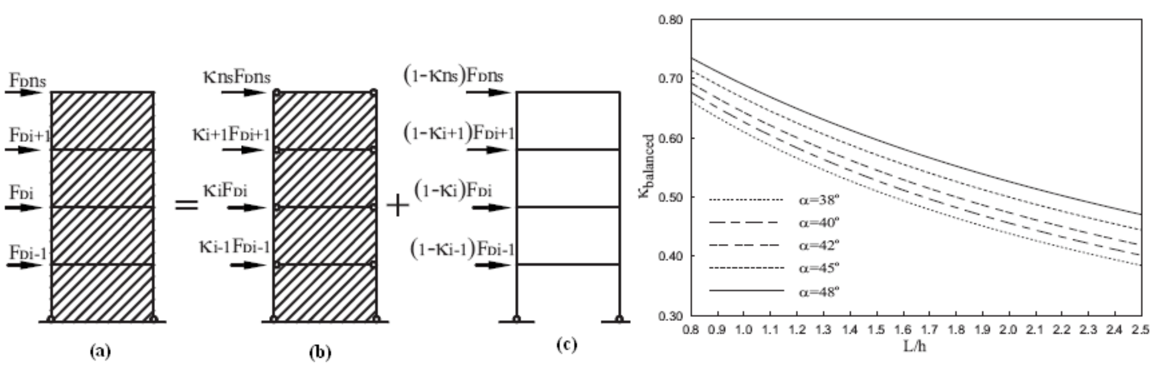


Figure 5: Decompositions of lateral forces and SPSW system (Qu and Broneau, 2009)

8- By equating external and internal work, the following equation is derived for the uniform yield mechanism (see Figure 1) of the SPSW with moment resisting beam-column connections.

[9] $$\sum_{i=1}^{n} F_i h_i \theta_p = 2\sum_{i=1}^{n} M_{pi}\,\theta_p + \frac{1}{2}\sum_{i=1}^{n} F_y L(t_i - t_{i+1})\,Sin(2\alpha_i)\,h_i\theta_p$$

It should be noted that, the first and second terms in right side of the above equation is the contribution of the boundary moment frame and infill panel, respectively. After determining the contribution of each system in resisting lateral loads using the previous step, the work equation for the infill panel (frame b) under uniform yielding over every story takes the following format:

$$[10] \quad \sum_{i=1}^{n} \kappa_{bi} F_i h_i \theta_p = \frac{1}{2} \sum_{i=1}^{n} F_y L(t_i - t_{i+1}) \, Sin(2\alpha_i) \, h_i \theta_p$$

Finally, the required plate thickness at each storey level is obtained using equation [11].

$$[11] \quad t_i = \frac{2 \sum_{j=i}^{n} \kappa_{bj} F_j}{F_y L Sin(2\alpha)}$$

9- HBEs and VBEs at each storey are designed based on the capacity design principles to resist the tensile forces expected from the fully yielded infill plates. It should be noted that HBEs are required to design such that the intended yield mechanism will be reached, i.e. plastic hinges will form in beam ends a small distance from column faces.

3. CASE STUDY

To evaluate the effectiveness of the design procedure presented in prior sections, eight storey and fifteen storey Type D steel plate shear walls are separately designed using the proposed PBPD design procedures and the Canadian codes (i.e. CSA S16-09 and NBCC 2010) design procedures. A series of nonlinear time history analyses is performed under spectrum compatible earthquake records for Vancouver, BC. The building descriptions and the design assumptions are briefly summarized in the following section.

3.1 Assumptions and Design Summary

The building under investigation is adapted from Bhowmick *et al.*, (2009). Eight storey and fifteen storey buildings are considered to evaluate the performance of the proposed design procedure. The hypothetical symmetrical office buildings are founded on rock (site class B according to NBCC) in Vancouver. The buildings have two identical SPSWs provided to resist lateral forces in each direction; thus, each shear wall will resist one half of the design seismic loads (see Figure 6). The bay width and constant story height were assumed to be 6 m and 3.8 m, respectively, resulting in an infill panel aspect ratio of 1.58. A dead load of 4.26 kPa was used for all floors and the roof. The live load and snow load were taken as 2.4 kPa and 1.66 kPa, respectively. The nominal yield strength of the beams, columns and infill plates of the SPSWs was assumed to be 345 MPa and all steel members were assumed to have a modulus of elasticity of 200000 MPa. For the comparison purposes, it was assumed that the calculated infill plate thicknesses are available in all cases for both designs. Various parameters used to compute the design base shear of SPSWs according to the two design procedures are summarized in Tables 1 and 2. A proper estimation of the yield drift of the structure is required at the beginning of the design process. As discussed by Bayat (2010), since the column axial loads in SPSWs system are substantial, it is not appropriate to consider a constant yield drift for the SPSWs with different height and infill panel aspect ratio. Lateral drift in SPSWs is comprised of two terms, namely shear drift due to the web plate yielding and flexural drift due to axial deflections of columns. The first term is constant and depends only on yield stress of the infill plates. However, the latter depends also on bay width and height of the SPSW. It should be noted that, an appropriate estimation of the average stress of the columns is required to calculate the flexural component of the yield drift (Bayat, 2010). Given the above mentioned reasons, an initial target drift corresponding to the shear deflection of the SPSW due to yielding of the infill panel is selected, which is further modified to account for the flexural deflection of the wall resulting from the axial deformations of the column. An initial target drift of 1.5% was selected for the design purpose in this study. Summary of SPSW components for two different designs are represented in Tables 3 and 4. The calculated steel weight of infill plates, HBEs and VBEs for both design cases are shown in Figure 6 for comparison purpose.

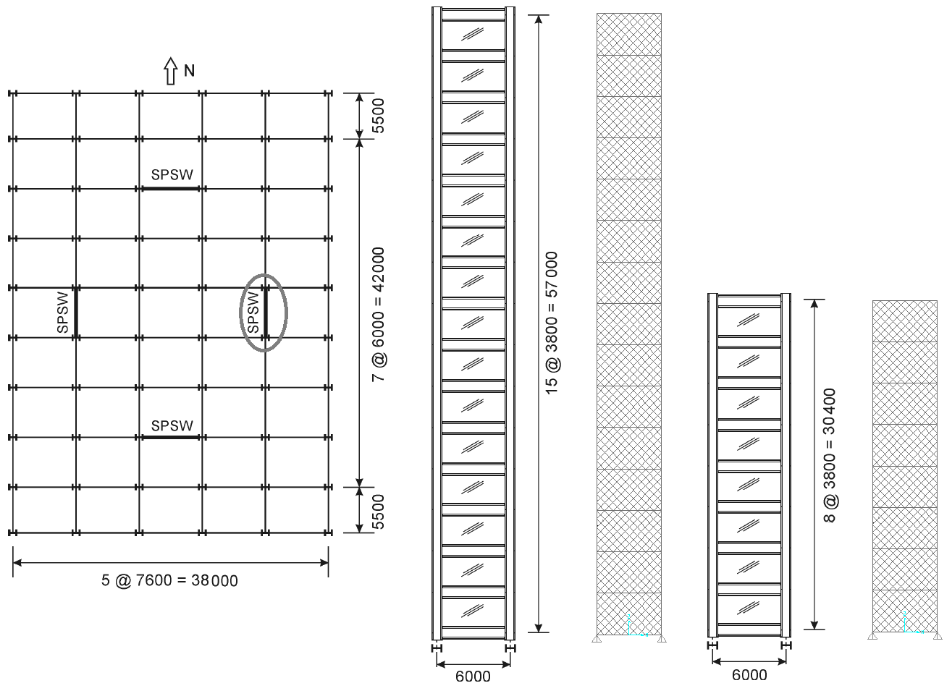


Figure 6: SPSW dimensions (mm) and the dual strip models used in this study.

Table 1: PBPD parameters

No.	PBPD parameters	8-storey	15-storey	Note
1	Fundamental period (s)	0.912	1.71	Eq. [1]
2	Basic target drift ratio, $\theta_u(\%)$	1.5	1.5	Pre-selected
3	Yield drift ratio, $\theta_y(\%)$	0.87	1.19	$\theta_y = \theta_{yf} + \theta_{ys}$
4	Modified target drift ratio, $\theta'_u(\%)$	2	2.3	$\theta'_u = \theta_u + \theta_{yf}$
5	Energy reduction factor (η)	0.75	0.75	
6	Structural ductility factor, μ_s	2.27	1.93	$\mu_s = \theta'_u/\theta_y$
7	Ductility reduction factor, R_μ	2.27	1.93	Fig. 2
8	Energy modification factor, γ	0.68	0.77	Eq. [6]
9	Design spectral acceleration, S(T_a)	0.280	0.158	
11	Base shear coefficient, V/W	0.0283	0.0187	Eq. [7]
12	Design base shear (KN)	977	1205	

Table 2: Code based design parameters

No.	Code design parameters	8-storey	15-storey
1	Fundamental period (s)	0.912	1.71
2	Force modification factor, R_0	1.6	1.6
3	Force modification factor, R_d	5	5
4	Seismic importance factor, I_e	1	1
5	Higher mode factor, M_v	1	1.14
6	Additional top floor load, $F_t\ (KN)$	77	174
7	Design spectral acceleration, S(T_a)	0.28	0.158
8	Base shear coefficient, V/W	0.035	0.0225
9	Design base shear (KN)	1210	1450

Table 3: Fifteen storey SPSW design summary

Storey	Lateral force (KN)		Lateral force assigned to infill panel (KN)		Plate thickness (mm)*		HBE		VBE	
	Code	PBPD	Code	PBPD	Code	PBPD	Code	PBPD	Code	PBPD
1	10.5	6.7	10.5	3.7	2.122	1.294	W460×106	W460×60	W360×744	W360×551
2	21.0	13.4	21.0	9.6	2.107	1.290	W460×106	W460×60	W360×744	W360×551
3	31.5	20.3	31.5	16.1	2.076	1.279	W460×106	W460×68	W360×744	W360×551
4	42.0	27.3	42.0	22.9	2.021	1.253	W460×106	W460×68	W360×634	W360×421
5	52.6	34.6	52.6	29.9	1.960	1.227	W460×106	W460×68	W360×634	W360×421
6	63.1	42.3	63.1	37.4	1.884	1.192	W460×106	W460×68	W360×634	W360×421
7	73.6	50.4	73.6	45.3	1.779	1.140	W460×106	W460×74	W360×463	W360×347
8	84.1	59.1	84.1	53.8	1.673	1.091	W460×106	W460×74	W360×463	W360×347
9	94.6	68.7	94.6	63.2	1.551	1.092	W460×106	W460×74	W360×463	W360×347
10	105.1	79.5	105.1	73.7	1.405	1.030	W460×106	W460×74	W360×314	W360×237
11	115.6	92.1	115.6	86.0	1.254	0.954	W460×106	W460×82	W360×314	W360×237
12	126.1	107.6	126.1	101.0	1.088	0.869	W460×106	W460×82	W360×314	W360×237
13	136.6	128.4	136.6	121.1	0.902	0.771	W460×106	W460×89	W360×196	W360×147
14	147.1	161.1	147.1	152.5	0.707	0.653	W460×106	W460×89	W360×196	W360×147
15	346.8	313.8	346.8	298.1	0.496	0.340	W530×138	W530×60	W360×196	W360×147

Table 4: Eight storey SPSWs design summary

Storey	Lateral force (KN)		Lateral force assigned to infill panel (KN)		Plate thickness (mm)*		HBE		VBE	
	Code	PBPD	Code	PBPD	Code	PBPD	Code	PBPD	Code	PBPD
1	30.8	20.4	30.8	11.38	1.746	0.976	W460×97	W460×68	W360×382	W360×287
2	61.6	41.1	61.6	29.5	1.701	0.963	W460×97	W460×74	W360×382	W360×287
3	92.4	62.8	92.4	49.7	1.607	0.927	W460×106	W460×74	W360×314	W360×237
4	123.2	86.0	123.2	71.9	1.475	0.870	W460×106	W460×82	W360×314	W360×237
5	154.0	112.0	154.0	96.7	1.293	0.786	W460×106	W460×82	W360×237	W360×179
6	184.8	142.9	184.8	126.3	1.072	0.675	W460×113	W460×82	W360×237	W360×179
7	215.6	184.8	215.6	166.1	0.805	0.53	W460×113	W460×89	W360×147	W360×134
8	348.0	327.3	348.0	297.9	0.497	0.34	W530×123	W530×101	W360×147	W360×134

3.2 Analytical Model and Ground Motions

Nonlinear time history analyses were performed to evaluate the seismic performance of SPSWs designed using PBPD procedure. The buildings under investigation were modeled using SAP2000 V14.1.0. The commonly accepted strip model (Thorburn et al., 1983) was used to analyze the SPSWs under seismic loads. Validated dual strip model was used to adequately capture the nonlinear dynamic and hysteretic behavior of the infill plates. In this method the steel plates were represented by two series of inclined pin ended tension only members. To simplify the geometry of the dual strip model, common beam nodes were considered to create the strip elements of adjacent storeys of the models. An angle of $45°$ was used for the strips throughout the structure. The boundary frames were constructed using rigid beam-column connections. Axial hinges were lumped at the midpoint of tension strips and P-M2-M3 Fiber hinges were defined throughout the length of the HBEs and VBEs to capture any yielding over the length of these members. A total of 16 fibers were used for each section and the hinge lengths were considered as 90% of the corresponding member depth (Purba and Bruneau, 2010). P-Delta effect due to gravity loads was considered in the analyses. Spectrum compatible seismic records of four earthquakes were used as the excitations for the time history analysis of the SPSWs located in Vancouver, BC. The four selected ground motions which have been used in earlier research (Bhowmick et al., 2009) are the followings: North-south component of the El-Centro earthquake of 1940, Petrolia station record from the 1992 Cape Mendocino earthquake, Parkfield 1966 earthquake record, and Nahanni, Canada 1985 earthquake record. Dual strip models of the considered SPSWs are shown in figures 6.

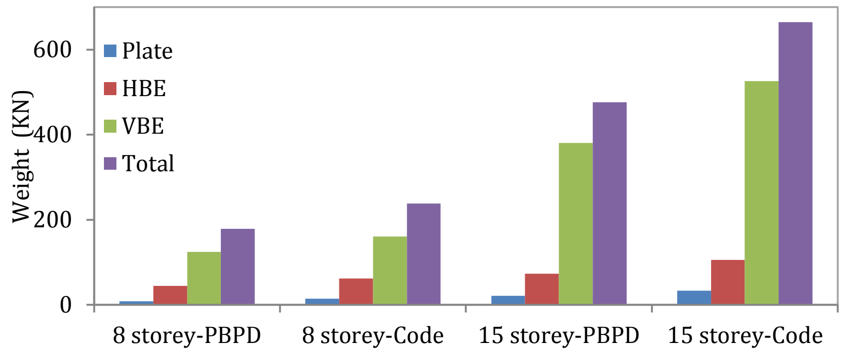
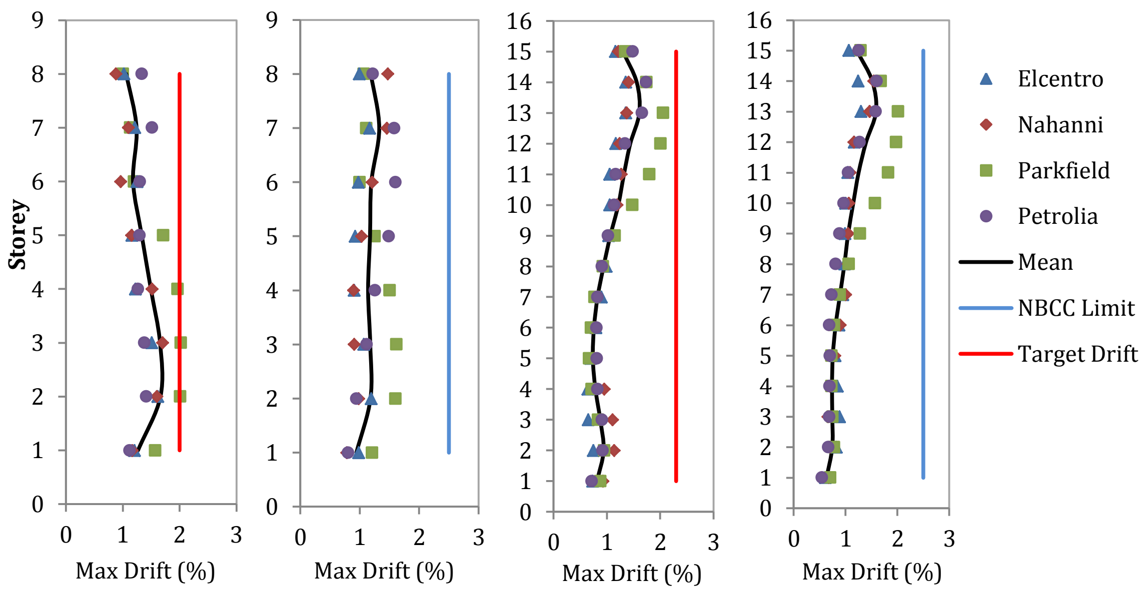


Figure 7: Comparison of steel weight of SPSW components for two different designs (i.e. PBPD and the Canadian Code design)

3.3 Seismic Performance of SPSWs

Nonlinear time history analyses were performed to evaluate and compare the seismic performance of SPSWs designed using to two different design methods. Figure 8 shows the maximum interstorey drift ratios for the selected earthquakes and the corresponding mean values for each wall. For all designs the nonlinear responses of the SPSWs were well controlled under the selected spectrum compatible ground motions, and the maximum interstorey drifts for all cases were within the NBCC limit of 2.5% of the corresponding storey height. It can be concluded from figures 8 (a) and 8 (c) that for both SPSWs designed using the proposed PBPD design procedure, the maximum drift values were well within the corresponding target values of 2% and 2.3% for 8-storey and 15-storey SPSWs, respectively. SPSWs designed using the PBPD method exhibited somewhat larger story drifts (close to the values that were targeted at the beginning of the design process) compared to the Code design. It can be noted from the comparison of steel weight, that significantly smaller member sizes were selected for the PBPD design compared to the code design, which consequently resulted in larger plastic deformations in the walls to reach the pre-selected target drift. Although the PBPD method resulted in thinner plate thicknesses and consequently smaller boundary members due to the capacity design principles, the responses for both the 8-storey and15-storey SPSWs were still well-controlled and the desired seismic performance was maintained.



(a) (b) (c) (d)

Figure 8: Peak storey drift distribution along the height of SPSWs resulted from time history analyses. (a) 8 storey-PBPD; (b) 8 storey-Code; (c) 15 storey-PBPD; (d) 15 storey-Code.

In this research, it was assumed that the calculated plate thicknesses are available in all cases, however, for any SPSW design, welding and handling requirements as well as availability of the plate thickness limit the use of very thin infill panel. To overcome this problem, solutions such as using perforated steel plates and light gauge steel have been practiced that show good promise.

4. SUMMARY AND CONCLUSION

In this paper, performance-based plastic design method is applied to the seismic design of eight storey and fifteen storey SPSWs with moment resisting beam-to-column connections. In this method, pre-selected target drift and desirable yield mechanism of the plate walls are used as key performance criteria. The design base shear is obtained based on energy-work balance for the plate walls and an equivalent Elastic-Plastic Single Degree of Freedom (EP-SDOF) system to achieve the same target drift. Plastic design is performed to obtain the required strength of the infill panels, followed by the capacity design of boundary frame elements. To achieve an efficient design of the system, the contribution of boundary moment frame in resisting the lateral loads is also taken into account in the proposed design procedure. To investigate the effectiveness of the proposed design procedure, a series of nonlinear time-history analysis of the SPSWs designed using two different design procedures (i.e. PBPD and the Canadian Code) were performed under spectrum compatible earthquake records for Vancouver, BC. Dual strip models were used to represent the infill panel in non-linear time history analysis of the SPSWs. Appropriate plastic hinges were inserted in the model to capture any nonlinearity within the elements in dynamic analyses. The results of inelastic time history analyses indicate that the SPSW designed according to PBPD procedure perform well under the selected ground motions. The maximum storey drifts were within the corresponding target values (i.e. 2% for 8-storey and 2.3% for 15-storey) as well as NBCC limit of 2.5%. The calculated PBPD design base shear for the 8-storey and 15-storey SPSWs are 81% and 83% of that obtained from the NBCC procedure. The reduced design base shear together with considering the contribution of the boundary moment frame in resisting lateral loads, resulted in thinner infill panels, and consequently smaller boundary member sizes compared to the Code designed walls. Moreover, for an initially designed SPSW using the code procedure, further iterations may be required to achieve the most economical design with the desired performance, however; the pre-selected target and seismic performance can be achieved using PBPD with less design iterations and efforts. Given the above-mentioned reasons, it is concluded that a more economical design of SPSWs can be achieved using the PBPD procedure, while maintaining the desirable performance under earthquake excitations.

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