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Design and Validation of Steel Plate Shear Wall Building Systems

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ABSTRACT

Engineers from consulting practice, the steel design industry and researchers from Canadian universities, have collaborated on the development of a design methodology for a unique lateral load resisting system. This system, steel plate shear walls (SPSW's), has undergone numerous experimental investigations which have led to non-mandatory guidelines (Appendix M) in CAN/CSA-S16.1-94 (S16.1). Together with a professional partnership student, the collaborators carried out comprehensive designs of SPSW's of varying ductility rating for hypothetical building locations across the nation. Alternate designs for each location, predominantly of reinforced concrete form, were prepared for economic evaluation against the SPSW system. Some anomalies, with respect to the guidelines of S16.1, were identified and recommendations have been forwarded to the CSA Technical Committee for consideration. Apart from minor technical issues, the design rationale suggested, were simple to implement using basic analytical tools. Optimized SPSW structures subjected to wind and earthquake loading resulted in super- and sub-structure works less expensive than traditional reinforced concrete shear core forms. When effects of overall capital financing are considered, further savings are attainable on efficiently executed SPSW projects as construction schedules can be reduced by a reasonable margin. Notwithstanding the desirable behaviour exhibited by the SPSW system from numerous experimental investigations, in particular for earthquake prone application, the results indicate that reasonable economic benefits result from its consideration as a viable alternate to traditional construction methods.

INTRODUCTION

Numerous analytical and experimental investigations have been conducted on SPSW's primarily in Canada, the United States and Japan, following encouragement from the consulting and steel fabrication industries. Tangible benefits can be gained from a SPSW structure. The two most predominant ones being, reduced overall cost and improved seismic response.

A steel plate shear element is comprised of a column and beam frame augmented by a steel in-fill plate between the boundary wide flange members. When these in-fill plates occupy each level within a frame bay of a structure, they constitute a SPSW. Its behaviour is analogous to a vertical plate girder that is cantilevered from its base. The SPSW system exceeds the latter's limitations, however, as it optimizes performance by realizing the post-buckling behaviour of the steel in-fill panels.

Previously, pioneering designers of this building system neglected the benefits of the post-buckling behaviour which include extensions to drift control and shear resistance, increased redundancy and controlled energy dissipation characteristics. These progressive consultants did not, however, have appropriate design rules for this post-buckling behaviour. Kulak recognized this in the early 1980's and together with other researchers, initiated analytical (Thorburn et.al. 1983) and experimental (Timler et.al. 1983, Tromposch et.al. 1987, Driver et.al. 1997) studies focussed on developing design procedures suitable for national building standard acceptance. Their efforts have been successful through the embodiment of draft design procedures for SPSW's in S16.1.

The University of British Columbia, University of Alberta, a consortium of structural consultants and the Canadian Institute of Steel Construction (CISC) undertook further experimental testing and analytical evaluation of the system. Scaled tests were conducted under quasi-static and dynamic loading conditions on four storey, single bay specimens. Coincident with the experimental program, design verification on a multi-storey structure was documented for varying levels of ductility. This paper is a summary of the practicing professionals' efforts (Timler 1998) on the SPSW design verification.

DESIGN PROTOTYPE AND CRITERIA

A prototype building example with potential merit of capturing a construction market traditionally left to other capable building materials needed establishment. As the proposed analytical method of S16.1 currently identifies ductility values of

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R=2, 3 and 4 for SPSW's, it was decided to analyze and design the all steel prototype building for location specific National Building Code of Canada (NBC) minimum climatic requirements for each ductility rating prescribed. Therefore, ductility ratings and locations were assigned for the SPSW study as follows: 1) R=2, ordinary SPSW - Toronto; 2) R=3, nominally ductile SPSW - Montreal; and, 3) R=4, ductile SPSW - Vancouver.

It was also appropriate to examine the economic validity of the SPSW system through comparison to reinforced concrete and hybrid alternatives subjected to the same design criteria. The reinforced concrete ductility assignments followed as; 1) R=2, wall with nominal ductility - Toronto, Montreal (Case 1); and, 2) R=3.5, other ductile wall systems in combination with R=4, ductile coupled walls - Montreal (Case 2), Vancouver (Case 1). The hybrid example consisted of a steel gravity load system combined with a reinforced concrete shear core. Its ductility assignment for Vancouver (Case 2) was the same as 2) immediately above.

The preliminary phase of prototype development focussed on establishing building categorizations currently dominated in reinforced concrete form, which potentially would be receptive to SPSW's. These were; 1) mid- to high-rise residential towers; 2) mid- to high-rise office towers; and, 3) low-rise institutional facilities. Several building floor plate layouts were examined from an inventory of designs, either constructed or underway by the collaborative group's participants. It became clear that one functional form lent itself to more direct study than others. This led to a more detailed phase of selection that allowed other layouts of proposed or constructed structures from only the favourable building functional category to be studied. An eight storey office tower SPSW alternate was selected (Figure 1) for the design exercise.

SPSW DESIGN METHODOLOGY

Design instructions were developed for the R=2 and the R=3, and 4 cases from the guidelines presented in S16.1, the referenced research and design experience. In essence, the SPSW in-fill panel is idealized as a series of tension diagonals. For preliminary boundary frame sizing, a single brace idealization can also be made. Rationalization for the preliminary and detailed design sequence development of each follow below.

R=2, SPSW Design

For the preliminary design, the equivalent storey stiffness analogy was used to calculate a single effective pin-ended storey brace in a plane frame model. Pinned connections were assumed between the beam and column within the shear core. Both infinitely stiff and flexible column limitations were examined with infinitely stiff beams for determination of the bounding conditions of the equivalent single brace. Judgement was exercised in the selection of the cross-sectional area of the brace as the actual condition would lie between these limitations. The remaining steel framework of the structure contained pin-ended connections. Once drift limitations were satisfied for wind and seismic cases, through adjustment of the column members, detailed analysis of the shear panels commenced with the development of the multi-strut, pin-ended model. Infinitely stiff beams were used with the actual stiffness of the columns for the plane frame model. Web panel thickness, perimeter member refinement and connection details were addressed to an appropriate level from analysis on a floor by floor basis.

R=3 and 4, SPSW Design

For these designs, the two-phase process above was proposed, however, with some adjustments. These followed as; 1) moment connections were modelled between beam and columns within the shear core element; 2) actual member stiffness were used for the shear core beams and columns; and, 3) appropriate detailing requirements of S16.1's Clauses 27.2 and 27.3 for seismic moment resistances would be employed.

STEEL DESIGNS

One optimized floor system was selected to suit the NBC loading criteria for all locations. As snow loads differed per hypothetical site, however, the sizing of roof members varied. Three floor/roof systems were examined to determine the least cost option. These consisted of; 1) composite stub-girders; 2) composite trusses; and, 3) composite beam/girders. Based on minimized mass, the composite truss system was selected for the prototype. Figure 2 indicates the optimized floor layout for Vancouver.

Although this floor plate and elevator shear core arrangement was best suited for the study, due to the intersection of orthogonal oriented SPSW frames, a common column situation occurs (two locations) which introduced design complexity. Since this condition is likely to occur in practice, it was judged worthy of inclusion. To meet the requirement for built-up sections at these common column locations, a castellated fabrication was selected for the R=2 case, whereas a built-up section comprised of a wide flange with WT's welded to form a cruciform was selected for the R=3 and 4 cases. The reason for the full cross-section requirement of the latter condition was that moment continuity was required along both axes for each more ductile case.

Very stiff top beams were selected to represent the upper boundary elements of the shear cores. A stiff anchorage of the lower in-fill panel was also assumed to exist at the foundations. Floor diaphragms were modelled as infinitely stiff.

The iterative design process for final member selection was un-complex, albeit, noteworthy observations are presented. Within the single brace idealization, for the infinitely stiff column case, top of building drifts were approximately 25% of the actual stiffness case. For the multi-strut model, variability of the strut angle (while maintaining a uniform number of struts (ten) equally spaced throughout the shear panel) would stagger the node points on the beams, thus presenting non-beneficial work for the designer. A small sensitivity study was initiated to examine the feasibility of using an averaged angle of inclination throughout the shear core thereby allowing common node points on beams for struts modelled above and below. It was concluded that, for the R=2 case, moments in the columns were significantly affected by noticeable variations in the angle whereas no appreciable frame behavior occurred. This moment effect was less predominant for the R=3 and 4 cases because of the inherent fixity at the beam/column joints. As the angle differences between levels were small for the case studies, averaged values were used for simplification. When the seismic design provisions of Clause 27.2 were examined to assess the impact for the R=4 final design, it was concluded that clarification was required. The clause prescribes restriction of column axial loads to 0.3AgFy to avoid the occurrence of weak column/strong beam behaviour. For a plane moment structure, it is agreed that this limitation is required. However, for a dual (moment frame/SPSW) system, where the system's high level of redundancy precludes collapse or mechanism development), such a requirement was considered overly restrictive. Therefore, modifications to the final member sizes to accommodate the lower axial limit to the columns were not made. Details of the final shear core design for Vancouver are shown in Figure 3. Corresponding details for this higher ductility design are depicted in Figure 4.

CONCRETE AND HYBRID ALTERNATE DESIGNS

Several alternates to the SPSW schemes were examined, albeit, to a lesser degree of design detail. This included examination of four reinforced concrete schemes, and a hybrid alternate consisting of a steel gravity load system with a concrete shear core. Preliminary analysis to obtain structural form, floor plate, shear core, column size and foundations for each alternate was conducted by the same professional partnership student.

A common concrete floor plate layout was used for all locations. The floor system selected consisted of post-tensioned slab bands due to the large spans. For shorter spans at building ends, spandrel beams were more suitable. Columns were sized based only on gravity load and minimum code required eccentricity. Reduction of column size dependent on height was not exercised due to potentially large moments that would be transferred to the upper level columns from the slab system.

As with the steel options, the lowest ductility rating was selected for the Toronto location. This was an R=2 condition consisting of concrete shear walls of nominal ductility in both principal directions within the core. For the Montreal location, where the governing lateral loading condition was due to earthquake, it was decided to examine both nominally ductile and ductile reinforced concrete arrangements. The nominally ductile structural form was the same as selected for the Toronto case. For the ductile lateral framing system, an R=3.5 was selected for the transverse orientation, whereas an R=4 was assigned to the longitudinal direction. With respect to the Vancouver location, two ductile systems were examined. Each replicated the ductile lateral framing system as assigned to the second Montreal case studied for the shear core. The first Vancouver location examined, (Case 1), comprised of a concrete framed gravity system, whereas the second (Case 2), utilized the steel floor plate and non-core gravity framing system optimized for the steel example for the same location. Figure 5 depicts the reinforced concrete frame layout for Vancouver.

COST ANALYSIS

Cost estimates for each design were conducted to evaluate the SPSW system against the Canadian industry standard being the concrete shear core method. Apart from comparisons of super- and sub-structure costs, other factors were considered. These included fire-proofing for the steel and hybrid options, re-shoring of the floors for the concrete alternatives as well as design costs and comparison of construction completion schedules to estimate gains/penalties for leased floor space offsetting bridge financing costs. Costing was done in-house, however, was verified independently by industry representatives. A range of +/-15% is considered appropriate for the exercise.

Schedules from project inception through to ready-for-occupancy for the steel, concrete and hybrid options were prepared. While a slower superstructure construction start is anticipated for the steel building form, significant gains are made in its completion that subsequently would allow an earlier start on building finishes. The building finishes duration, i.e., exterior curtain wall, mechanical, plumbing, electrical and architectural, were assumed equal for each building material form. The net result was that the SPSW office tower could be ready for leasing approximately nine weeks before the concrete shear core structure. A six-week advantage results with the hybrid alternate over the full concrete scheme's construction completion.

Table 1 gives the costs for each of the locations considered for all alternatives. Normalized ratios of average steel costs relative to average concrete costs are summarized for various components of the total in Table 2.

Appreciable savings were noted if a hybrid system is selected over a reinforced concrete shear core when both systems are designed for greatest ductility for the Vancouver location. Measurable savings, prior to bridge financing considerations, are indicated for each of the steel options when compared to its location counterpart in concrete. The SPSW alternate is also less expensive than the hybrid system for the Vancouver location. Considerable savings are attainable when the bridge financing benefits are measured due to the shortened completion schedule for both the steel and hybrid options, however, the SPSW design indicates a significant savings over the hybrid design. Moderate savings are achieved in the superstructure whereas significant savings are obtained in the sub-structure costs for the SPSW alternates.

CONCLUSIONS

The design rules for SPSW's, as proposed by the guidelines of Appendix M, CAN/CSA-S16.1-94, in general, are useful for the professional, however, some anomalies were identified.

Cost estimation confirmed by industry input has been conducted for eight design layouts examined. From the exercise conducted, based on the ductility ratings assigned, SPSW schemes exhibit reasonable cost advantages based on quantities and labour rates over parallel reinforced concrete designs. When bridge financing considerations are taken into account which recognize scheduling advantages of properly executed projects per traditional approaches, substantial cost advantages are achievable.

RECOMMENDATIONS

Consideration needs to be given to analytical modelling simplifications for SPSW systems. These include averaging of the angle of the tension field, as calculated per the prescribed guidelines, for shear core stacks for buildings of moderate height. In addition, re-examination of the present axial load limitation for columns within shear core stacks being designed for R=4 ductility rating is warranted. Arguments have been presented suggesting the 0.3AgFy limitation may have room for moderate relaxation since the SPSW system has been shown experimentally and analytically, to be highly redundant, stable and confidently predictable in terms of stiffness degradation. Further research in this area is suggested to set an appropriate upper bound for axial loads of SPSW core columns.

This design study has focussed on a single building arrangement that possessed a limited number of aspect ratios for consideration. The effect of increased building height may shift the postulated governing shear deformation behaviour of SPSW's to one more influenced by bending. Studies in this area may produce further guidelines for future designers of the system.

Based on this study, which demonstrated appreciable savings potential of capital expenditures for structural systems which incorporate SPSW's with optimized steel gravity framing, it is recommended the system be promoted vigorously.

ACKNOWLEDGEMENTS

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REFERENCES

Thorburn, L.J., Kulak, G.L. and Montgomery, C.J., 1983, Analysis of Steel Plate Shear Walls, Structural Engineering Report No. 107, Department of Civil Engineering, University of Alberta.

Timler, P.A., and Kulak, G.L., 1983, Experimental Study of Steel Plate Shear Walls, Structural Engineering Report No. 114, Department of Civil Engineering, University of Alberta.

Tromposch, E.W. and Kulak, G.L., 1987, Cyclic and Static Behavior of Thin Panel Steel Plate Shear Walls, Structural Engineering Report No. 145, Department of Civil Engineering, University of Alberta.

Driver, R.G., Kulak, G.L., Kennedy, D.J.L. and Elwi, A.E., 1997, Seismic Behaviour of Steel Plate Shear Walls, Structural Engineering Report No. 215, Civil & Environmental Engineering Department, University of Alberta.

Timler, P.A., 1998, Design Procedures Development, Analytical Verification, and Cost Evaluation of Steel Plate Shear Wall Structures, Earthquake Engineering Research Facility Technical Report No. 98-01, Department of Civil Engineering, University of British Columbia.

TABLE 1: COSTS FOR STEEL, CONCRETE AND HYBRID OPTIONS AT SELECTED LOCATIONS

Material	Location	Ductility Rating (R)	Cost Estimates +/-15% (in thousands of Canadian Dollars)								
			Super-Structure		Sub-Structure	Incidental		Struc. Design Costs	Sub-Total Struc. Costs	Bridge Finance Credits	Total Struc. Costs
			Frame	Floor	Foundations	Fire Proof.	Shoring				
Steel	Toronto	2	539	2118	260	359	N/A	45	3321	-660	2661
	Montreal	3	544	2050	275	345	N/A	48	3262	-477	2785
	Vancouver	4	727	2196	315	370	N/A	50	3658	-958	2700
	Average	*N/A*	*603*	*2121*	*283*	*358*	*N/A*	*48*	*3413*	*-698*	*2715*
Concrete	Toronto	2	346	2410	386	N/A	154	62	3558	-	3358
	Mont. (1)	2	456	2529	456	N/A	147	69	3657	-	3657
	Mont. (2)	3.5, 4	461	2529	409	N/A	146	69	3614	-	3614
	Mont. (avg.)	N/A	459	2529	433	N/A	147	69	3637	-	3637
	Vancouver (1)	3.5, 4	542	2713	521	N/A	158	76	4010	-	4010
	Average	*N/A*	*449*	*2551*	*447*	*N/A*	*153*	*69*	*3669*	-	*3669*
Hybrid	Vancouver (2)	3.5, 4	488	2162	469	370	N/A	63	3551	-636	2915

TABLE 2: NORMALIZED RATIOS OF AVERAGE STEEL COSTS TO CONCRETE COSTS

Ratio	Normalized Cost Ratios						
	Super-Structure	Sub-Structure	Incidental	Design	Sub-Total	Bridge Financing	Total
Steel (avg.) / Concrete (avg.)	0.91	0.63	2.34	0.70	0.93	N/A	0.74

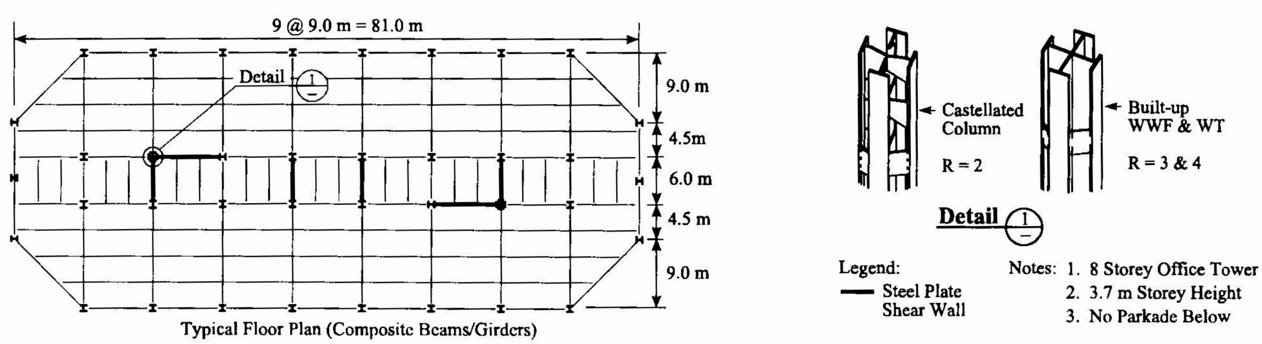


FIGURE 1: GENERIC FRAMING FOR SPSW PROTOTYPE

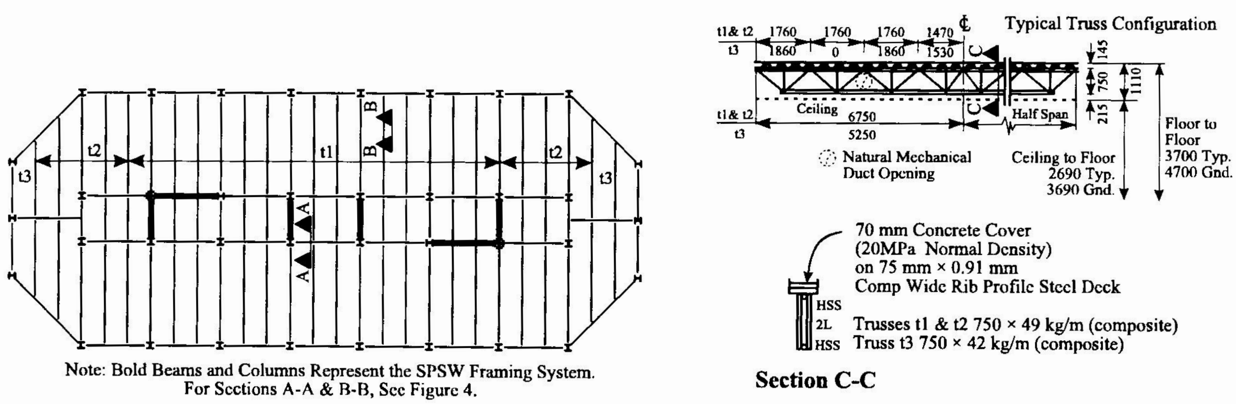
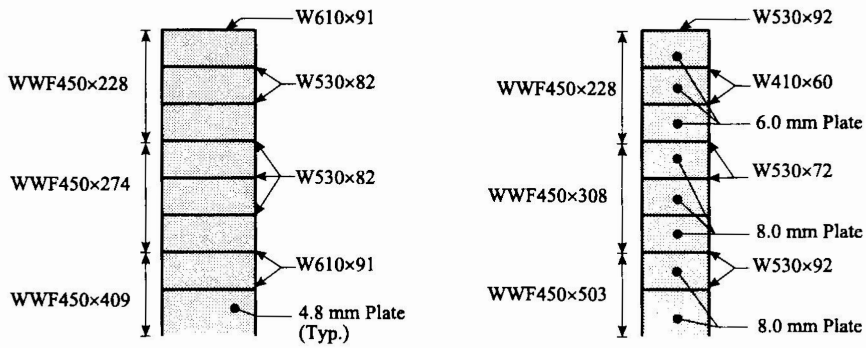


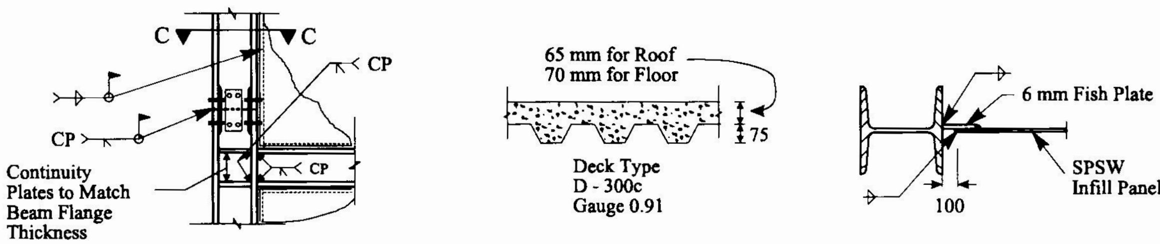
FIGURE 2: OPTIMIZED FLOOR FRAMING LAYOUT FOR VANCOUVER



Longitudinal Frames (2 Required)

Transverse Frames (4 Required)

FIGURE 3: FINAL SHEAR WALL SIZING FOR VANCOUVER

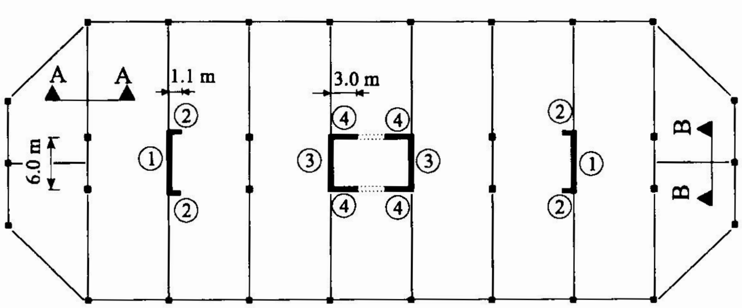


Section A-A

Section B-B

Section C-C

FIGURE 4: DETAILS FOR VANCOUVER SPSW LAYOUT



Typ. Interior Column:
750 mm × 750 mm Floors 1 to 3
600 mm × 600 mm Floors 4 to 5
500 mm × 500 mm Floors 6 to 8

Typ. Exterior Column:
600 mm × 600 mm All Floors

Floors \ Walls	1	2	3	4
1-3	600	600	600	650
4-5	400	400	400	650
6-8	300	300	300	400

Thickness of Walls (mm)

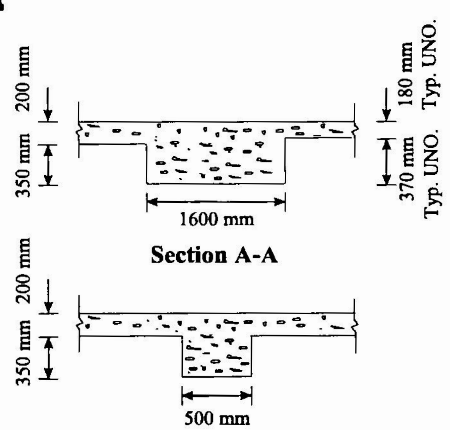


FIGURE 5: GRAVITY AND LATERAL LOAD RESISTING SYSTEM FOR VANCOUVER: CASE 1 (R = 3.5 & 4) CONCRETE OPTION