13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2804

SIMPLIFIED AND DETAILED FINITE ELEMENT MODELS OF STEEL PLATE SHEAR WALLS

Mahmoud REZAI[1], Carlos E. VENTURA[2], Helmut PRION[3]

SUMMARY

Recent research has demonstrated that Steel Plate Shear Wall (SPSW) structures can act as an effective and economical lateral bracing system. While the results of experimental research unanimously support the rationale of using the post-buckling strength, tension field action, and the stable energy absorption capacity of the steel panels in resisting lateral cyclic loading, the numerical modelling aspect of the SPSW research has resulted in mixed responses. In this paper, simplified and detailed analytical models of a 4-storey specimen tested at the University of British Columbia (UBC) were generated to assess the ability of current analysis techniques to reasonably describe the behaviour observed during the experiment. The results of this investigation showed that the simplified and detailed analytical models overpredicted the elastic stiffness of the test specimen. The yield and ultimate strength as well as post-buckling behaviour of the specimen were reasonably well predicted. The orthotropic model representation of a SPSW system produced stresses in the beams, columns and infill plates consistent with the results obtained from a detailed explicit finite element formulation.

INTRODUCTION

In recent years the idea of utilizing the post-buckling strength of thin infill steel plates connected to a boundary frame has gained wide attention from researchers in Canada, Japan, Europe and the United States. A number of static and quasi-static cyclic tests performed on large and small scale models have been reported since 1970. These studies have examined the behaviour of steel plates throughout the entire range of loading, from elastic to plastic and from pre-buckling to post-buckling stages. The results obtained from the studies unanimously support the rationale of using the post-buckling strength, tension field action, and the stable energy absorption capacity of the steel panels in designing the primary lateral load resisting system for buildings.

In this study the numerical modelling aspect of slender Steel Plate Shear Wall (SPSW) structures is investigated. Nonlinear finite element models of the SPSWs tested at the University of British Columbia (UBC) in Canada were developed to study the load-displacement response, effect of boundary members and structural details of the SPSW systems.

---

[1] Principal Engineer, EQTech Eng. Ltd., Vancouver, British Columbia, Canada; mrezai@eq-tech.com
[2] Professor, Dept. of Civil Engineering, Univ. of British Columbia, Canada; ventura@civil.ubc.ca
[3] Associate Professor, Dept. of Civil Engineering, Univ. of British Columbia, Canada; prion@civil.ubc.ca

OVERVIEW OF THE SPSW SYSTEM

The concept of using steel plate shear panels in structural systems was developed decades ago. The aircraft engineering and shipbuilding industry have been using the steel panels in both stiffened and unstiffened applications for many years. The application of this system in building construction is rapidly being recognized as an effective structural steel lateral load resisting system. In the past 30 years, university researchers and leading structural engineers in the United States, Canada, Japan and the United Kingdom have studied the seismic behavior of the SPSW system. They have shown that the system possesses robust cyclic behavior with little degradation, and that it can be considered as one of the most ductile earthquake resistant systems.

The main function of a SPSW system is to resist shear forces and overturning moments due to lateral loads. The system consists of steel plate panels connected at all four sides by a steel frame consisting of beams and columns. Typically, the beams are positioned at floor levels and the column location is determined by architectural requirements. The steel plate is connected to the steel frame by closely spaced bolts or continuous welding. Openings required by windows and doors can generally be accommodated within the steel plates by stiffening the shear panel around the openings, as shown in Figure 1 (BPA Group, 2003).

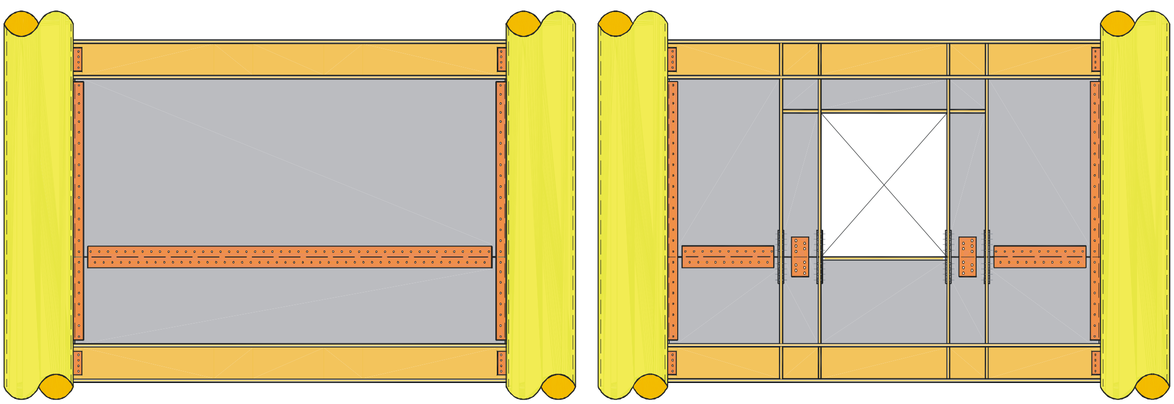


Figure 1: A typical SPSW bay with and without opening (after BPA Group)

The SPSW system uses welds or bolts to connect the plates to the boundary beams and columns. This is similar to the "fish plate" connection tabs and fillet welds used in many of the test articles to connect the infill plates to the boundary frame members, as shown in Figure 2. The infill steel plates could either be welded or bolted to the "fish plate".

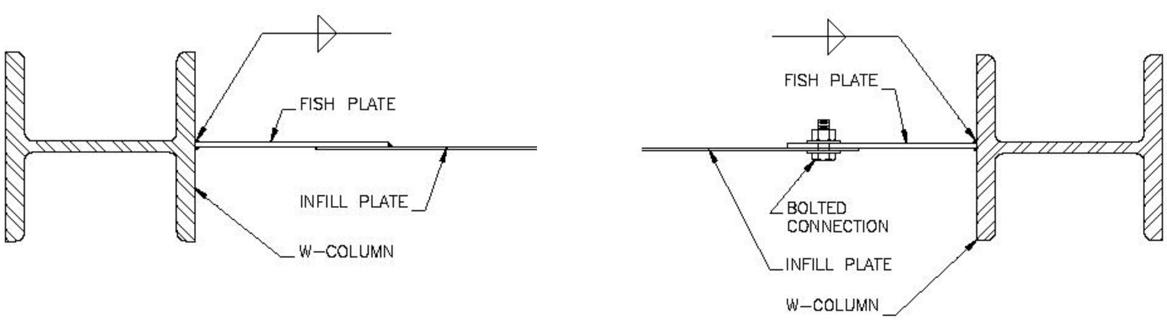


Figure 2: Typical "fish plate" to boundary frame and infill plate to "fish plate" connections

The main lateral load-resisting element in a SPSW system is the infill plates. The steel plates are expected to buckle along compressive diagonals under relatively small shear forces. After buckling, the story shear forces are resisted by the plates through formation of a tension field. This behavior is captured in the analysis by modeling the infill plates as shell elements that include both geometric and material nonlinearities.

SPSW MODELLING METHODOLOGY

The formation of tension field action within the infill plates is the primary mechanism to resist story shear forces. This behavior should be considered in the analysis by modeling shear plates using shell elements that can buckle. This complex plate buckling formulation (geometric nonlinearity) is not readily available in conventional structural analysis programs used for routine seismic design. To facilitate the analysis and design of structural elements for building applications including the gravity beams and columns using a conventional structural analysis program a simplified methodology for modeling the steel plates has been employed.

Strip Model Methodology

Thorburn et al. (1983) developed an analytical method to study the shear resistance of thin unstiffened steel plate shear walls. The model was based on the theory of pure diagonal tension field by Wagner (1931) which did not account for any shear carried by the infill plates prior to shear buckling. The so-called strip model represented the shear panels as a series of inclined strip members, capable of transmitting tension forces only, and oriented in the same direction as the principal tensile stresses in the panel. Each strip was assigned an area equal to the product of the strip width and the plate thickness. Figure 3 shows the strip model representation of a typical steel plate shear panel for the two cases of infinitely stiff and completely flexible columns proposed by Thorburn et al. (1983).

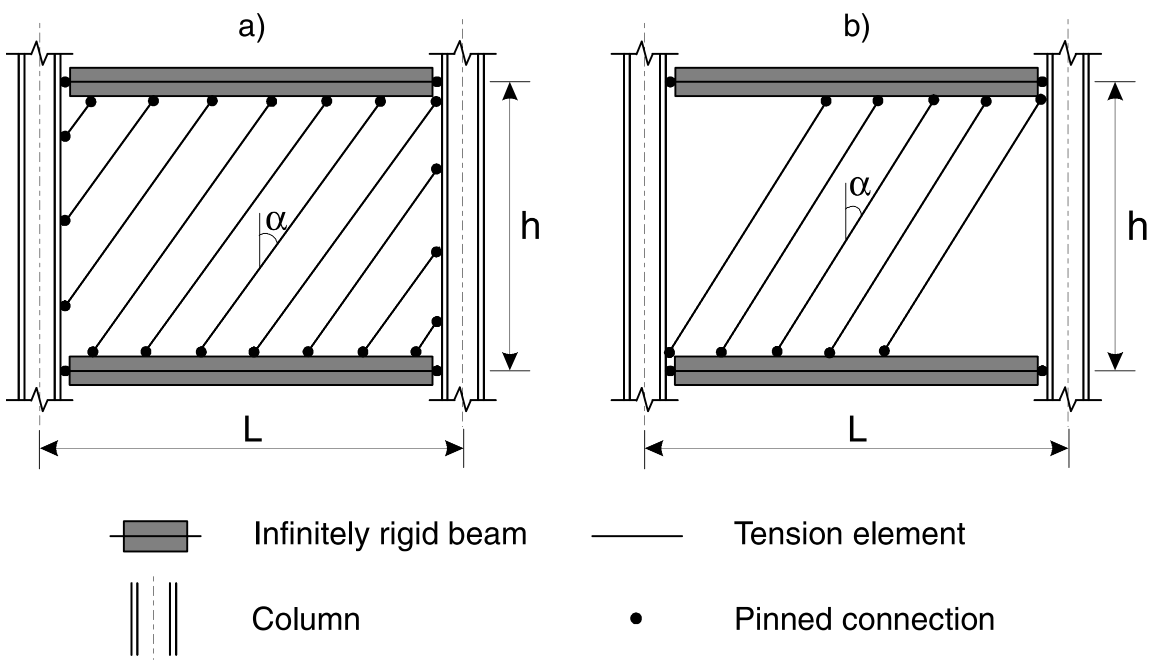


Figure 3: Strip model representation of a typical steel panel developed by Thorburn et al. (1983) a) complete tension field b) partial tension field

Timler and Kulak (1983) re-evaluated the expression for angle of inclination for diagonal tension strips developed by Thorburn et. al. in 1983. A revised formula for calculating the angle of inclination of tension strips in multi-storey buildings considering the bending effects of columns was derived. This equation has been implemented in the Canadian National Standard on Limit States Design of Steel Structures, CAN/CSA-S16-01 (2001) for determining the angle of inclination of tension strips in multi-storey buildings.

By replacing a plate panel with these tension strips, the resulting steel structure can be analyzed using currently available computer analysis software, such as SAP2000 and ETABS Nonlinear and inelastic pushover programs (CSI, 2000).

The strip model analogy assumes the distribution of the in-plane stress field across the panel width to be constant. The tension field stresses, however, are known from tests (Rezai, 1999 and Driver, 1997) to be non-uniform and that the behaviour of the panels is somewhat more complex than a series of strips at a constant angle can represent. The experimental studies at UBC indicated that the angle of the tension strips was closer to vertical at the corners and more horizontal around the mid-point of the plate. This was primarily related to the interaction of infill plates and boundary elements at the corners. UBC researchers (Rezai, 1999) have proposed a "multi-angle strip model" for steel plate shear walls. Using a non-linear analysis program and the model shown in Figure 4, they have produced analytical predictions that are reasonably close to the test results.

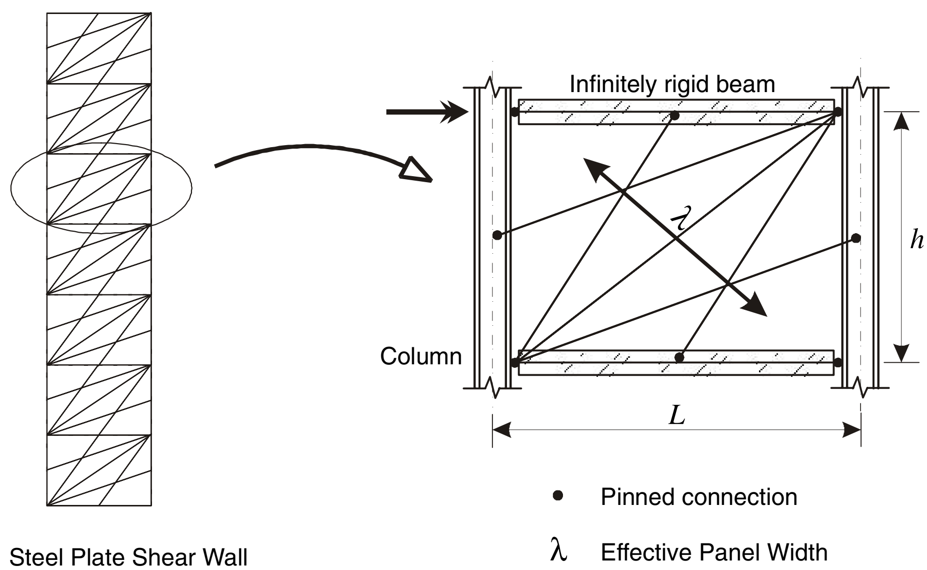


Figure 4: Proposed strip model simulating post-buckling behaviour of a thin steel plate, UBC Model (1999)

Orthotropic Modelling Methodology

In a linear or pushover finite element formulation of a structure with SPSWs the infill plate elements can be modeled as shell elements. To simulate the buckling of the compression diagonal in the plates, orthotropic material properties could be assigned to all shell elements. Orthotropic material enables the analyst to assign different moduli of elasticity and shear moduli to three principal directions of the plate. This will permit the analyst to model the compression diagonal with much less stiffness than the tension diagonal, and ensure that it will attract much less shear in proportion to its buckling capacity than the tension diagonal. The story shear force acting on the cross section of the panel can be calculated by adding up the shear in the shell elements. This is accomplished by reorienting the in-plane local axes of the infill plates in a 45 degrees fashion (or other angles if deemed appropriate) with respect to the horizontal line. Then, orthotropic material properties are assigned to the plates, taking into account the full modulus of elasticity of the plates along the axis 1 (the axis corresponding to the tension diagonal) and only a 2 to 5% elastic modulus along the 2nd axis (the axis corresponding to the compression diagonal). The shear modulus used in orthotropic material is set to zero. This causes the principal tensile stresses to form at about 45 degrees in a diagonal direction. Figure 5 shows a schematic representation of the orthotropic model with local axes of the infill plates oriented at 45 degrees.

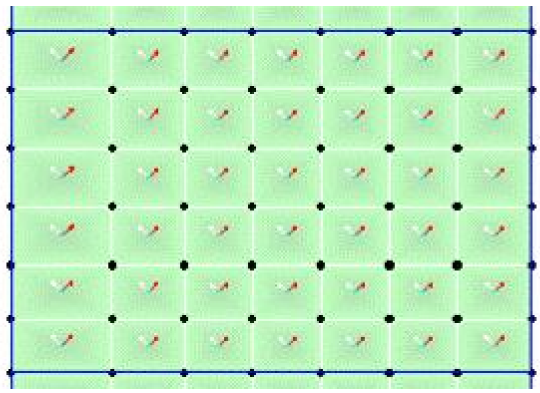


Figure 5: A schematic representation of the orthotropic model with local axes of the infill plates oriented by 45 degrees

Finite Element Modelling Methodology

Other researchers (Elgaaly et al., 1993, Driver et al., 1997 and Rezai, 1999, Behbahanifard and Grondin, 2001) have used the finite element formulation with both geometric and material nonlinearities to model the complex plate wall frame interaction. In general, the results of finite element analyses showed that the analytical models were somewhat stiffer than the experimental models. This was mainly attributed to the initial imperfection and/or residual stresses in the plates caused by the welding process.

UNIVERSITY OF BRITISH COLUMBIA SPSW TESTS (1999)

The overall strength, elastic post-buckling stiffness, interaction between frame action and shear panel behavior, effects of beam and column rigidities, the formation of diagonal tension field action combined with diagonal compression buckling of the infill plate and the stability of panel hysteresis curves were the main issues investigated during the quasi-static and dynamic shake table tests of scaled SPSW specimens at the University of British Columbia (Rezai, 1999 and Lubell et al., 2000).

The UBC test specimens represented about 25% scale models of one bay of a steel-framed office building core. Column-to-column centerline spacing of 900 mm (3 ft.) was used for each specimen, with unstiffened infill panels having a width to height aspect ratio of 1:1. Each specimen was comprised of S75×8 (S3×5.7) columns, kept continuous through the height of the frame, and S75×8 beams. To better anchor the internal panel forces, a second single story specimen (SPSW2) incorporated an additional S75×8 top beam welded along adjoining flange tips, while the four-story SPSW4 specimen utilized a deep stiff S200×34 beam at its roof level. Full moment connections were provided at all beam-column joints by continuous fillet welds of the entire beam section to the column flanges making the specimens a dual system. Full flange continuity stiffeners were used at all joints. The infill panels were constructed from 16 gauge (1.5 mm) hot rolled steel plate. Photographs of the SPSW1, SPSW2 and SPSW4 specimens are shown in Figure 6. SPSW1 and SPSW2 are equivalent to the first story of the SPSW4 specimen, with the top beam variation for SPSW2 as noted above.

The principal sequence of significant inelastic action in the single-panel specimens consisted of yielding of the infill panel followed by yielding of the boundary frame. The columns of the multi-story SPSW4 specimen yielded before significant inelastic action occurred in the infill panel, resulting in a state of global instability. Inelastic response in the specimens resulted from infill panel yielding (SPSW1, SPSW2), plate tearing (SPSW2), localized weld fractures (SPSW1, SPSW2) and the formation of plastic hinges in the boundary frame (top beam - SPSW1; bottom of columns - SPSW1, SPSW2, SPSW4; top of columns - SPSW2). Significant inelastic shear deformations were also observed in the columns of SPSW2 near the top beam-column joints. The single story specimens experienced significant inelastic deformations up to ductility of about six. The overstrength was about 1.5. It was concluded that the two one-story specimens demonstrated that the infill steel plates significantly reduced demand on the moment-resisting frame by producing redundant diagonal story braces that alleviated the rotation demand on the beam-to-column connections. For the SPSW4 each panel of the specimen, a maximum displacement ductility of 1.5 was achieved prior to a global instability failure propagated by yielding of the columns. The specimen exhibited over-strength of about 1.20. The 4-story specimen proved to be somewhat more flexible than the one-story specimens.

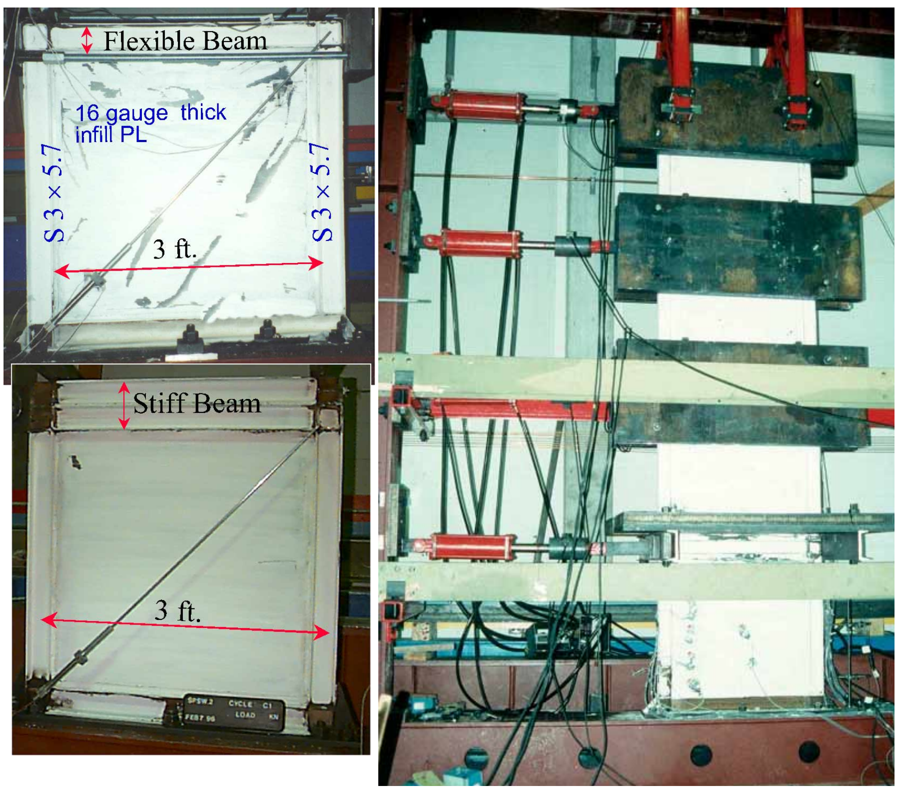


Figure 6: UBC two single and one 4-story SPSW specimens tested cyclically

4-Storey SPSW Test Results and Observation

The 4-storey specimen proved to be somewhat more flexible than the one-storey specimens tested before. This was expected because the influence of overturning moment becomes more significant as the height of a structure increases. The yield deflection in the first storey panel was determined as 9 mm at a storey shear of about 150 kN. Prior to reaching this value, three cycles each of ±25 kN, ±50 kN, ±80 kN, ±100 kN and ±125 kN were conducted to explore the elastic and initial inelastic behaviour of the specimen. During these cycles marginal stretching of the load-deformation hysteresis loops and a small permanent horizontal drift of about 1.5 mm for the first panel was observed. After three cycles of ±150 kN with a yield deflection of δy = 9 mm, the deflection in the first storey was increased in multiples of the global yield deflection. As the deflection was increased to about 1.6 times of the yield value the post-yield stiffness of the specimen decreased significantly and the load reversed at a storey deflection of about 15 mm. As the load reversed, the unloading stiffness was parallel to the initial elastic stiffness. Increasing the load to the opposite (negative) direction caused some pinching in the hysteresis curve with moderate reduction in the stiffness. This was also observed in the previous tests because the first storey infill plate was stretched and buckled inelastically during the previous loading direction. It would thus not be fully effective in the reversed loading until the load would be increased enough to compensate for the effects of Poisson's ratio. Once the tension field started to form in the web plate, the stiffness increased. The load was brought up to the negative yield strength of the specimen and increased by a manually adjusted control valve. As the first storey deflection reached -15 mm, the compression column buckled in the out-of-plane direction. The test was terminated at this point. Figure 7 shows the storey shear plotted against the interstorey drifts at all floors.

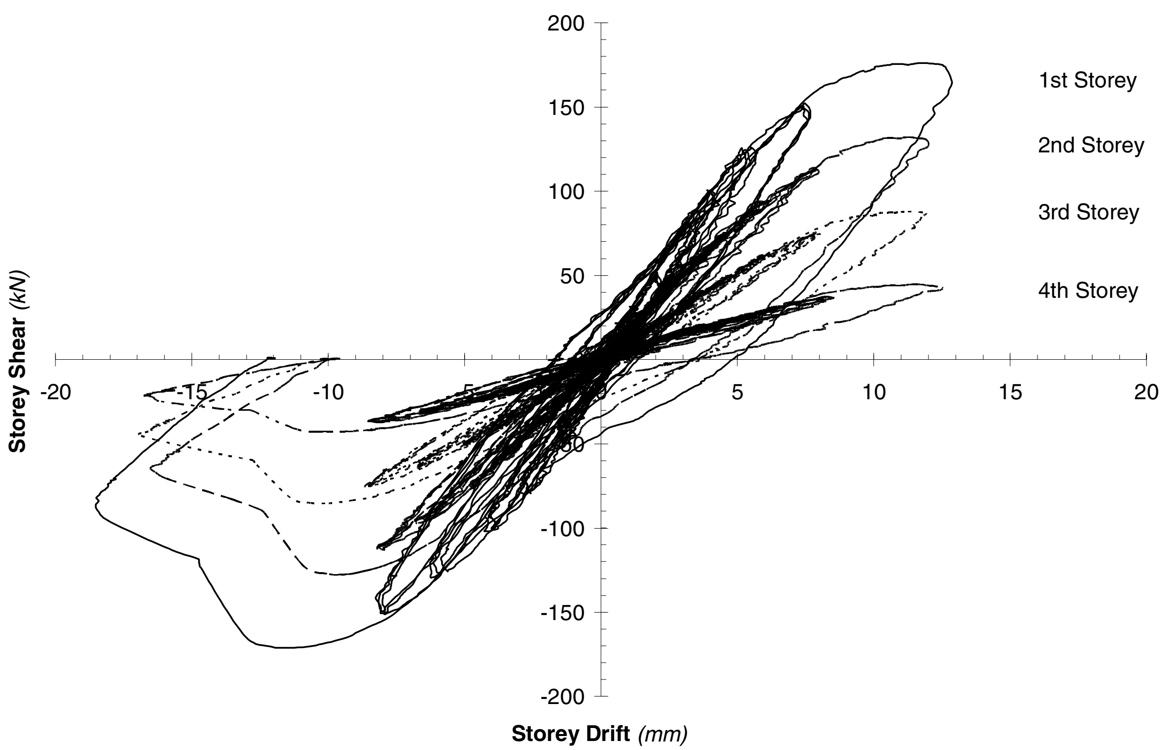


Figure 7: Storey shears versus interstorey drifts for the 4-storey specimen at different levels (after Lubell et al., 2000)

A significant portion of the input energy was dissipated by the first storey panel through column yielding and infill panel diagonal buckling and yielding. As no evidence of yielding was observed above the first storey level, the load-deformation hysteresis loops of the upper storey floors did not reflect the significant contribution of these floors to the overall energy dissipation mechanism through shear yielding or plate buckling. The load-deformation hysteresis loops of the upper stories reflected a rigid body rotation due to the column shortening or plastic hinging in the first storey. This may be verified by the similarity between the load-deformation plots of the upper storey levels with the pattern observed for the first storey panel.

The 4-storey test specimen failed as a result of global out-of-plane buckling of the first storey column. Prior to column buckling, the first storey infill panel had undergone considerable inelastic buckling and yielding. Figure 8 illustrates the state of damage to the first storey column and infill panel at the termination of the test. Even though a premature failure mechanism occurred when the specimen was experiencing inelastic deformations, useful data regarding initial elastic stiffness, post-yield stiffness, storey drifts, overall deformed column profile, variation of principal tensile stresses in the plate and the interaction of the boundary frame with the steel panels was collected from the test.

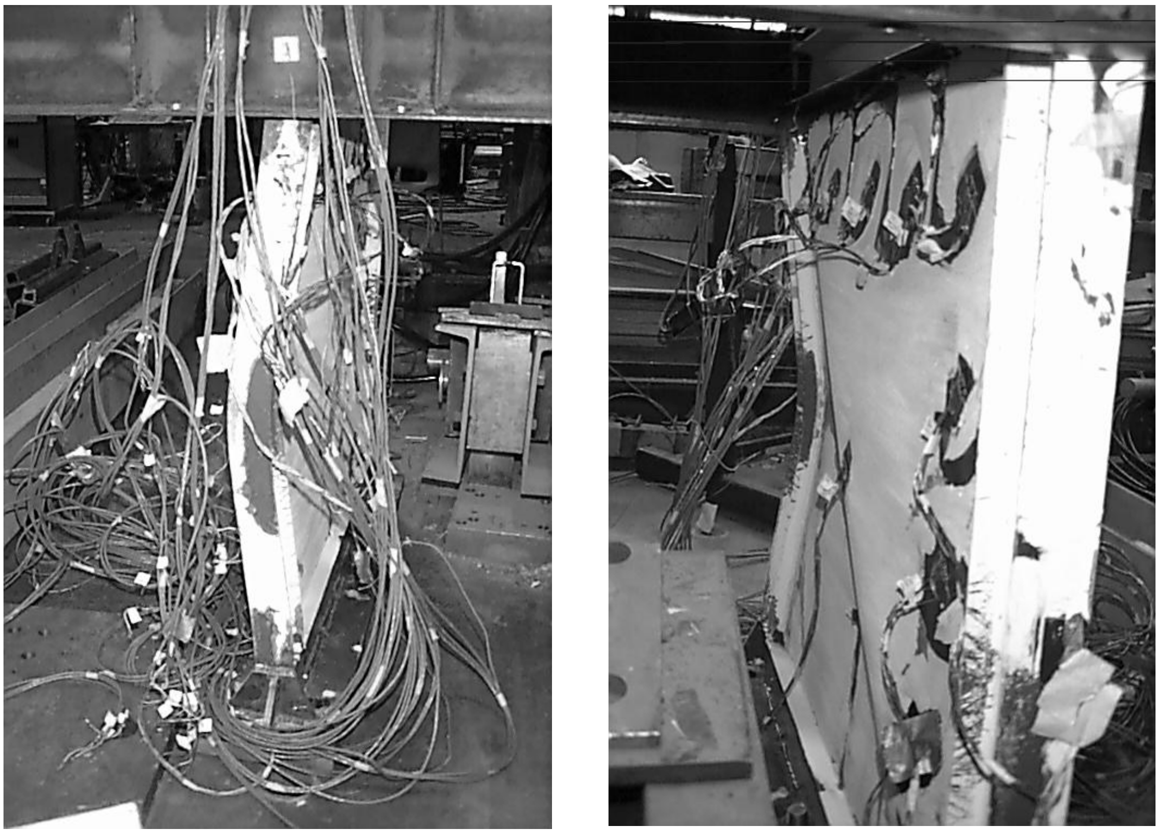


Figure 8: State of damage to the first storey panel and columns of the quasi-static 4-storey shear wall frame at the termination of test

Comparison with Single-Storey Experimental Results

To illustrate the effectiveness of the proposed methodology in modeling a SPSW system a comparison of predicted load-displacement response with the experimental results was carried out. The second single-storey specimen tested at the University of British Columbia was analyzed using the strip and orthotropic model assumptions. A comparison of the experimental and analytical load-displacement plots for the UBC single-story specimen is shown in Figure 9. Only the first quadrant of the hysteresis plots is shown for the cyclic loading.

Figure 9 indicates that a good correlation between the numerical and experimental results is achieved. The strip models predict the elastic stiffness and yield strength of the specimens reasonably well. The elastic stiffness, however, is slightly over predicted. The orthotropic modelling assumption reasonably predicted the initial elastic as well as the ultimate strength of the specimen.

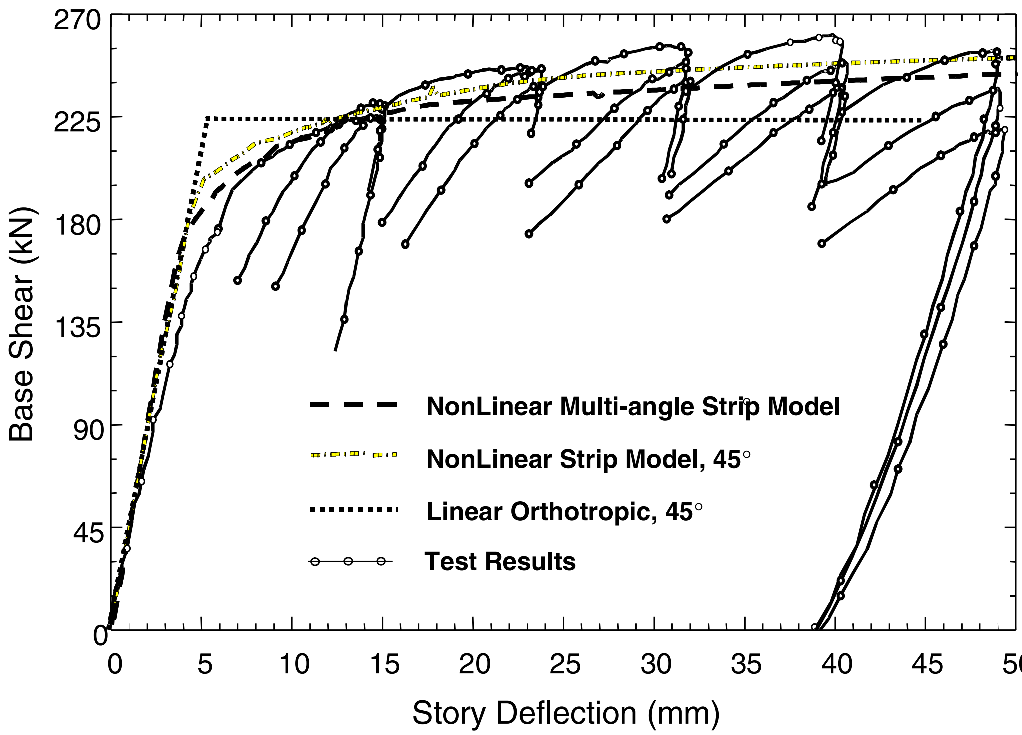


Figure 9: Comparison of analytical monotonic load-deflection curves of proposed models with test results for the UBC single-story specimen

FINITE ELEMENT MODELLING OF THE 4-STOREY SPECIMEN

The numerical model of the 4-story steel plate shear wall specimen at UBC was developed using LS-DYNA, a general purpose nonlinear finite element program (Livermore Software, 2003). Beams, columns and infill plates were all modeled as three- or four-node shell elements with both material and geometric nonlinearities turned on. Tri-linear stress-strain relationships using the results of coupon tests for the beams, columns and infill plates were defined for various elements in the model. Figure 10a shows a 3D view of the finite element model together with a close-up of the beam-to-column-to-plate connection detail. The presence of "fish plate" connection plates was ignored in the finite element model formulation of the specimen.

An orthotropic finite element model of the UBC 4-storey specimen was also prepared. The local axes of the infill plates were rotated by 45 degrees. An orthotropic material property assigned to the plates taking into account the full modulus of elasticity of the plates along the axis 1 (the axis corresponding to the tension diagonal) and only a 3% elastic modulus along the 2nd axis (the axis corresponding to the compression diagonal). The shear modulus used in orthotropic material was set to zero. Figure 10b illustrates the 3D view of the orthotropic finite element model of the UBC 4-storey specimen. The orientation of the infill plate local axes 1 and 2 is shown on the figure.

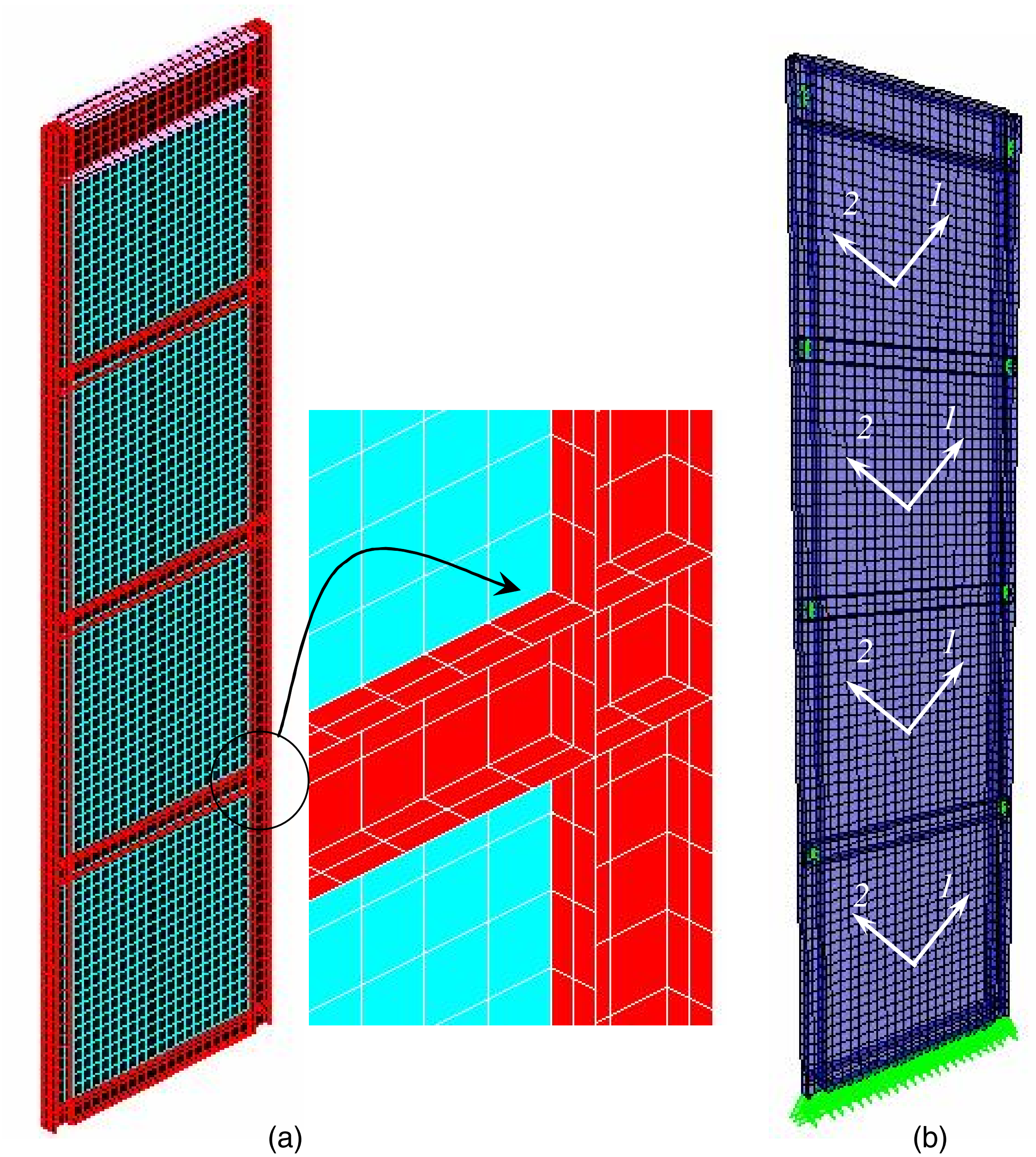


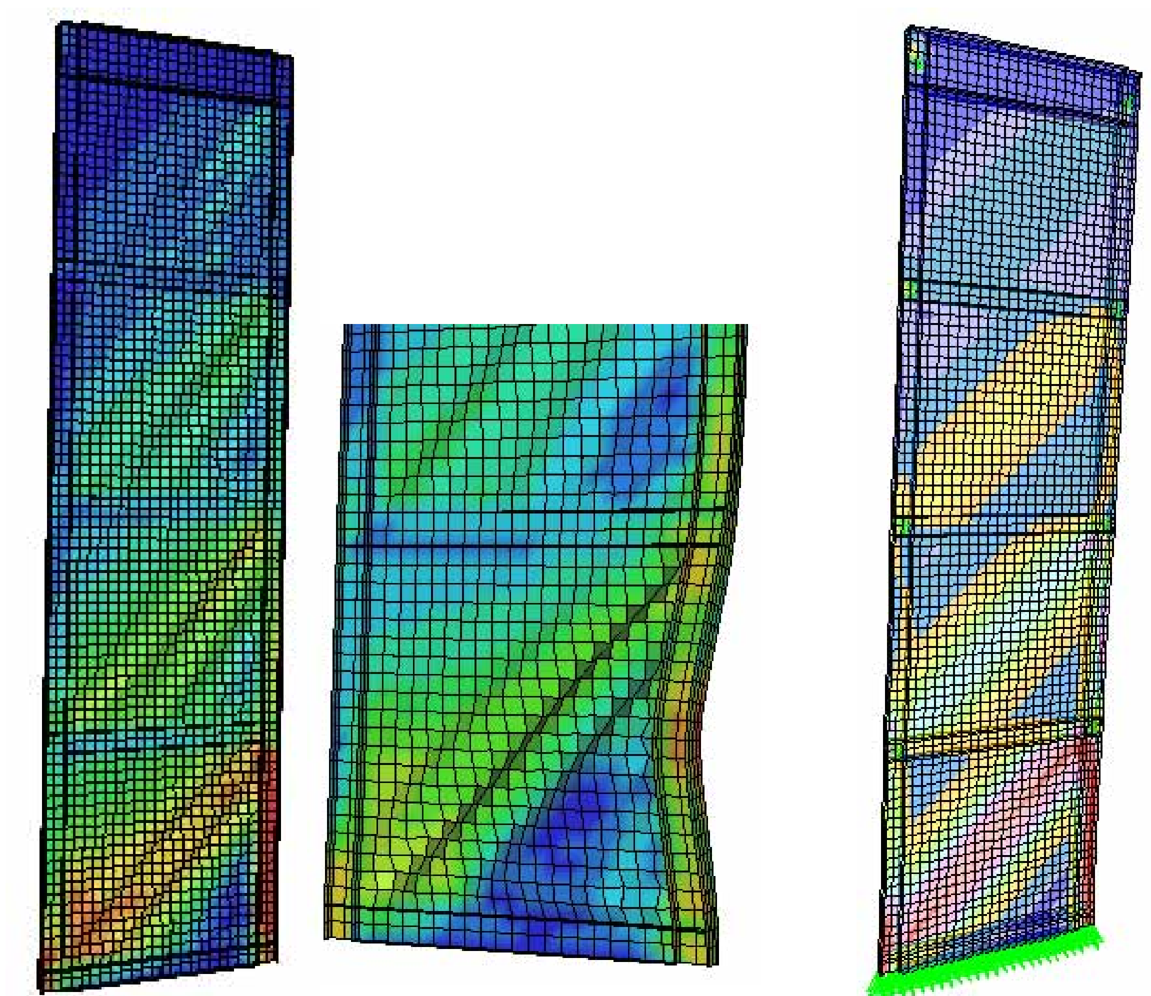
Figure 10: a) A 3D view of the finite element model of the 4-storey specimen together with a close-up of the beam-to-column-to-plate joint b) A 3D view of the orthotropic finite element model of the 4-storey specimen

The finite element models were loaded with vertical and horizontal loads at each floor level. The horizontal load was increased linearly from zero to the ultimate capacity of the frame. A small load perpendicular to the plane of the plates was applied to simulate the effect of plate imperfection.

The deformed profile of the steel plate shear wall model together with the Von-Mises stress contours for a base shear of approximately 160 kN before and after compression column buckling is shown in Figure 11a. The buckled configuration observed in the panels are consistent in number and orientation with those observed in the cyclic test. Figure 11b shows the Von-Mises stress contours for the orthotropic finite element model.

Figure 11 indicates that both models produce a consistent stress plots. The linear orthotropic model formulation is solved in a fraction of a time needed to solve the detailed finite element model. This is mainly because no material/geometric nonlinearity is considered in the orthotropic model. The overall behaviour of the specimen under prescribed loadings is adequately captured by the orthotropic model indicating its effective use in a design office for practical applications. Detailed finite element models might then be needed for back checking and comparative design purposes.

The formation of tension field in the plates for the first and second floors and to some degree for the third floor is pronounced in both plots. The regions with red color indicate hot spots where the material has reached its yield strength. It can be observed that the normal stress in compression column, just before buckling, exceeded the specified yield strength of the steel material. The plastic buckling of the compression column is adequately captured by the detailed explicit finite element model.



(a) (b)

Figure 11: a) Von-Mises stress plot just before global buckling (left) and after global buckling of the column in compression (middle) b) Von-Mises stress plot for the orthotropic finite element model

Figure 12 shows a comparison between the experimental and analytical results of the load-displacement response of the 4-storey specimen for the 1st storey level. The load-displacement curves are linear until yielding starts at a load of about 150 kN. It can be observed that the results of detailed and orthotropic finite element models overestimate the initial elastic stiffness of the frame. The yield strength and post-compression buckling capacity of the 4-storey specimen is reasonably predicted by the detailed finite element model.

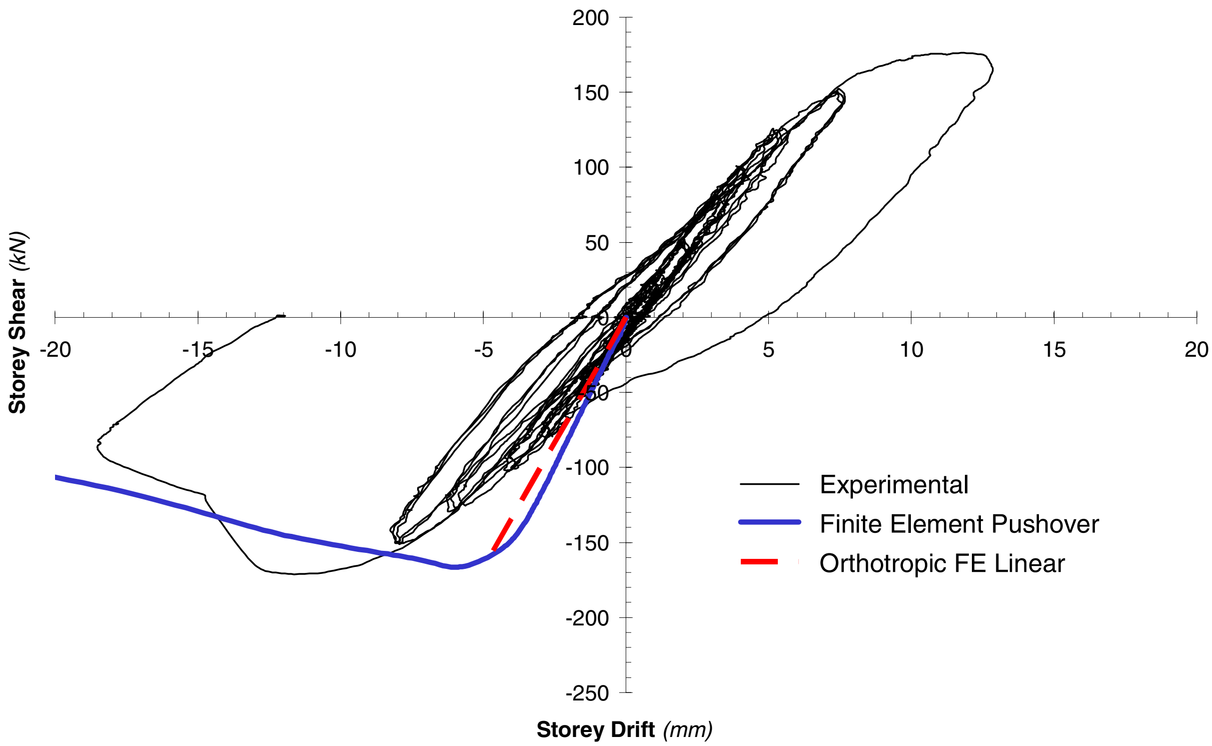


Figure 12: Comparison of experimental and analytical load-displacement response of the UBC 4-storey specimen for the first storey level

CONCLUSIONS

Results from the UBC scaled steel plate shear wall tests were used to verify the accuracy of the numerical models and to gain an understanding on how the various modelling techniques for the shear resistance of thin infill plates would affect the predicted results. In general, the strip, orthotropic and detailed finite element models overpredicted the elastic stiffness of the test specimens. The discrepancy between the analytical and experimental results was more dramatic for the 4-storey specimen compared to the single-storey specimens. This was mainly related to the small overall aspect ratio of the four-storey specimen and, thereby, increased moment to base shear ratio, which tended to dominate the flexural deformation compared to shear behaviour. It is noted that the storey floor displacement and interstorey drift profiles of the four-storey specimen illustrated the tendency of the shear behaviour domination for the bottom storey, while the top floors behaved as a rigid body rotating about the first floor.

Simplified SPSW models that can effectively predict the load-displacement response of the system and the corresponding member forces in beams, columns and plates are essential for facilitating the use of this system in a design office. The orthotropic model is an effective methodology to predict the overall response of a SPSW system to gravity and lateral loadings. In an orthotropic model formulation there is no need to replace the infill plates with a series of strips representing the tension field action in the plate. The reorientation of the infill plate in-plane local axes and assigning different modulus of elasticity of the steel material for the axes in compression and tension adequately simulates the tension field action in the plates.

Detailed finite element models help to verify the results of the simplified models and could be used for comparative design purposes. The need for further analytical studies using finite element formulation including both material and geometric nonlinearities is necessary for slender steel plate shear wall specimens. The effect of boundary element details, panel aspect ratio as well as plate imperfection on the overall response of a SPSW frame would need to be further studied.

REFERENCES

1. Behbahanifard, M.R. and Grondin, G. Y., 2001, "Cyclic Behaviour of Three-Storey Unstiffened Steel Plate Shear Wall", Porcs. of the 29th Annual Conf. of the Canadian Society for Civil Engineering, Victoria, British Columbia, Canada, 8 pages.
2. BPA Group, 2003, "Steel Plate Shear Walls Cyclic Behavior and Blast Performance Characteristics", Report #02080-2, Bellevue, Washington, US, 58 pages.
3. Canadian Institute of Steel Construction (CISC), 2001, "S16-01, Limit States Design of Steel Structures", Sixth Edition, Canadian Standards Association.
4. CSI, 2000, "SAP2000, Version, 7.44, Three Dimensional Static and Dynamic Finite Element Analyses of the Structures", Computers and Structures Inc., Berkeley, CA.
5. Driver, R.G., 1997, "Seismic Behaviour of Steel Plate Shear Walls", Ph.D. Dissertation, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, AB.
6. Elgaaly, M., Caccese, V. and Du, C., 1993, "Postbuckling Behaviour of Steel-Plate Shear Walls under Cyclic Loads", Journal of Structural Engineering, ASCE, Vol. 119, No. 2, pp. 588-605, February.
7. Livermore Software Technology, 2003, "LS-DYNA - General Purpose Transient Dynamics Finite Element Program".
8. Lubell, A.S., Ventura, C.E., Prion, H.G.L. and Rezai, M., 1999, "Unstiffened Steel Shear Wall Performance under Cyclic Loading", ASCE, Journal of Structural Engineering, Vol. 126, No.4, April 2000, pp. 453-460.
9. NBCC. 1995, National Building Code of Canada, National Research Council of Canada, Ottawa, Ontario.
10. 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, Edmonton, AB.
11. 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, Edmonton, AB.
12. Rezai, M., "Seismic Behaviour of Steel Plate Shear walls by Shake table Testing", Ph.D. Dissertation, University of British Columbia, Vancouver, Canada, 1999.