ANALYTICAL PREDICTIONS OF STRENGTH AND DEFLECTION OF LIGHT GAUGE STEEL FRAME / WOOD PANEL SHEAR WALLS

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Abstract

It is anticipated that the construction of buildings that incorporate light gauge steel frame/wood panel shear walls as primary lateral load resisting elements will increase across Canada in coming years. At present, a codified method for the prediction of shear wall strength and stiffness is not available in Canada. For this reason an investigation of various analytical prediction methods was completed. The racking strength and stiffness of steel frame/wood panel shear walls have been shown to be highly dependent on the behaviour of the sheathing connections. An experimental program involving over 200 small-scale tests was first carried out to establish the performance of steel stud to wood sheathing connections. This information was then utilized in a comparison of five existing analytical/mechanics based methods to predict the strength and deflection of wood framed shear walls. These existing analytical methods were adapted for use with the steel framed walls. A comparison of the predicted strength and deflection values was then made with the results of full-scale shear wall tests. Based on the comparison between test and predicted shear wall response the elastic models presented by Källsner & Lam were recommended for use to predict the lateral resistance and deflection of light gauge steel frame/wood panel shear walls under monotonic and cyclic loading. At the same time, the shear capacity and initial stiffness as measured from tests of single sheathing connections with an edge distance of 25 mm, and which were evaluated using an equivalent energy approach, were recommended as the input connection parameters for both the strength and deflection models.

Introduction

A typical light gauge steel frame / wood panel shear wall is composed of cold-formed steel studs and tracks that are connected with self-drilling/tapping screws to either plywood or OSB sheathing. Guidelines for the design of these shear walls do not exist in current Canadian codes. A design method for the calculation of in-plane shear stiffness and strength has been proposed for use with the 2005 NBCC *(NRCC, 2005) (Branston, 2004)*, however this method is reliant on the results of full-scale testing. Eventually, with the proven applicability of an analytical model, researchers could extend the results of small-scale connection tests to aid in the design of full size shear walls. For this reason a study was carried out to evaluate the possibility of using existing analytical methods, which were originally developed for the design of wood framed shear walls, to predict the in-plane shear stiffness and deflection of steel frame / wood panel shear walls. Methods by Källsner & Lam *(1995)*, Easley *et al.*, *(1982)* and McCutcheon *(1985)* were adapted for use with steel frame shear walls. Since all of these methods are based, to a large extent, on the stiffness and strength properties of the individual sheathing connections in a wall, tests were carried out by Okasha & Rogers *(2004)* to identify the connection properties. With the results of the sheathing connection tests Chen *(2004)* then compared the analytical predictions with the results of shear wall tests.

Objective and Scope

The objective of this research was to recommend an analytical method, which is based on a mechanics approach, to predict steel frame / wood panel shear wall strength and deflection. This involved the evaluation of five existing methods, developed for the analysis of wood framed shear walls, in

M. Pandey et al. (eds), Advances in Engineering Structures, Mechanics & Construction, 381–391.

comparison with the results of shear wall experiments. The details of a simplified strength and deflection model are provided in this paper. Test programs on single-storey shear walls and individual sheathing connections are summarized. In addition, a comparison between the test results and the predictions of an analytical approach is presented for the monotonic and reversed cyclic shear wall tests carried out by Boudreault *(2005)*, Branston *(2004)* and Chen *(2004)*.

Shear Wall Test Program

The results of a series of 109 shear walls (16 configurations) subjected to lateral in-plane loading were used in this study (Table 1). The testing consisted of three different size single-storey wall specimens: 610×2440 mm, 1220×2440 mm and 2440×2440 mm, which were composed of light gauge steel C-studs (92.1×41.3×12.7 mm) screw connected to steel tracks (92.1×31.8 mm) (Fig. 1). Both the studs and tracks were fabricated of ASTM A653 *(2002)* steel with a nominal grade and thickness of 230 MPa and 1.12 mm, respectively. Three types of wood sheathing (12.7 mm DFP *(CSA O121, 1978)*, 12.7 mm CSP *(CSA O151, 1978)*, 11 mm OSB *(CSA O325, 1992)*) were connected to one side of each test wall with screws placed at a spacing of 76, 102 or 152 mm. Simpson Strong-Tie S/HD10 hold-downs were used to connect the chord studs to the test frame. A specially constructed shear wall test frame was utilized to allow for the application of a lateral in-plane load to the top of the wall, and to provide lateral support to limit out-of-plane movement *(Boudreault, 2005; Branston, 2004; Branston et al., 2004; Chen, 2004)*.

Figure 1. Typical 1220×2440 mm shear wall test specimens.

Monotonic and reversed cyclic tests were carried out using the CUREE protocol for ordinary ground motions *(Krawinkler et al., 2000; ASTM E2126, 2005)*. In most cases, 6 specimens (3 monotonic and 3 reversed cyclic) were tested per wall configuration. Subsequently, the results of each shear wall test were evaluated using a codified version of the equivalent energy elastic – plastic (EEEP) approach for calculating the design parameters of light framed shear walls *(ASTM E2126, 2005)*. It was decided that the EEEP model best represented the behaviour of light gauge steel frame / wood panel shear walls subjected to both monotonic and reversed cyclic loading (Fig. 2) *(Branston, 2004)*. The model results in an idealized load-deflection curve, of a simple bilinear shape, that can be easily defined and constructed, yet still provides a realistic depiction of the data obtained from laboratory testing. Moreover, the EEEP model recognizes the post-peak deformation capacity by taking into account the energy dissipated by the test specimen up to failure. In the case of each reversed cyclic test a backbone curve was first constructed for the resistance *vs.* deflection hysteresis. This backbone curve and the resistance *vs.* deflection curve for monotonic specimens were then used to create EEEP curves based on the equivalent energy approach.

Table 1. Matrix of shear wall tests.

Configuration	Wall Dimensions (mm)	Sheathing Type (mm)	Fastener Schedule (mm)	Researcher
1, 2, 3, 4	1220 × 2440	CSP 12.5	102 / 305	Boudreault *(2005)*
5,6	1220 × 2440	DFP 12.5	102 / 305	"
7, 8	1220 × 2440	CSP 12.5	152 / 305	Branston *(2004)*
9, 10	1220 × 2440	CSP 12.5	76 / 305	"
11, 12	1220 ×2440	DFP 12.5	152 / 305	"
13, 14	1220 × 2440	DFP 12.5	76 / 305	"
15, 16	610 × 2440	CSP 12.5	152 / 305	Chen *(2004)*
17, 18	610 × 2440	CSP 12.5	102 / 305	"
19, 20	610 × 2440	OSB 11	152 / 305	"
21, 22	1220 × 2440	OSB 11	152 / 305	Branston *(2004)*
23, 24	1220 × 2440	OSB 11	102 / 305	"
25, 26	1220 × 2440	OSB 11	76 / 305	"
27, 28	610 × 2440	OSB 11	102 / 305	Chen *(2004)*
29, 30	2440 × 2440	CSP 12.5	152 / 305	"
31, 32	2440 × 2440	CSP 12.5	102 / 305	"
33, 34	2440 × 2440	CSP 12.5	76 / 305	"

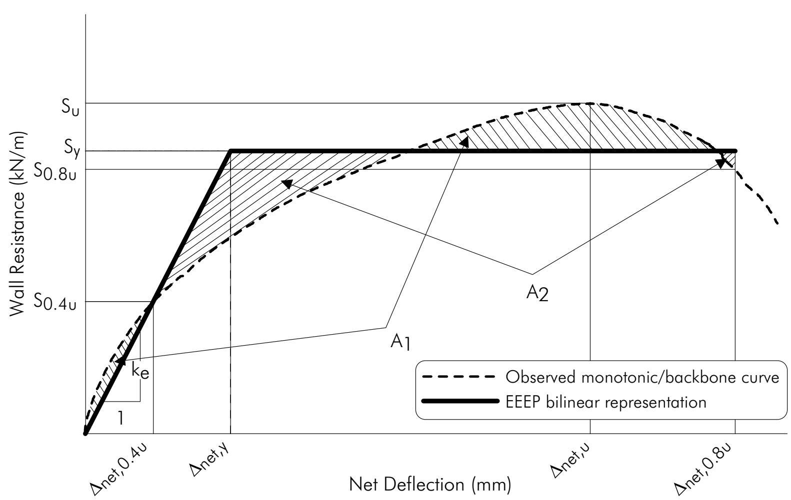

Figure 2. Equivalent energy elastic-plastic (EEEP) model *(Branston, 2004)*.

Sheathing Connection Test Program

The experimental program included 216 connection specimens that were subjected to tension monotonic and reversed cyclic loading. The monotonic tests were run at a rate of 2 mm/min, and the corresponding cyclic tests were performed by using the same CUREE test protocol as incorporated into the full-scale wall test program. The tests were conducted in accordance with the ASTM Standard D1761 *(1995)*. The test specimen matrix was selected in such a way to include the effect of the variation of wood sheathing type (DFP, CSP and OSB), thickness (9.5, 11, 12.5 & 15.5 mm) as well as orientation with respect to the grain and fastener edge distance (6, 9.5, 12.5, 16 & 25 mm). Moreover the effect of different steel stud thickness (0.84, 1.11, 1.37 & 1.73 mm) and strength (Grade 230 & 345) were included *(Okasha & Rogers, 2004)*. The primary aim of this research project was to provide information on connection performance to facilitate the predictions of the behaviour of steel frame / wood panel shear walls. Tests were carried out on individual screw sheathing to stud connections, as shown in Figure 3. The screw type, as well as sheathing and steel matched that used in the construction of the full-scale shear wall tests, the results of which were used to compare with the analytical predictions.

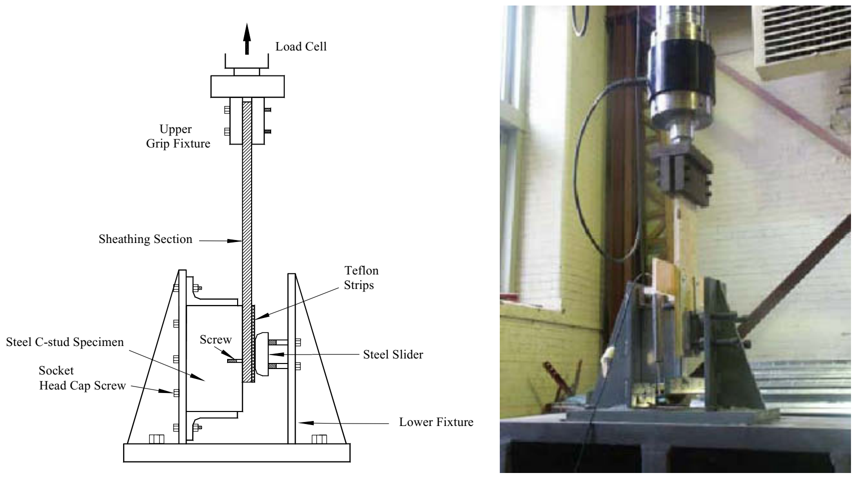

Figure 3. Sheathing connection test set-up.

Table 2. Parameters for prediction of monotonic tests *(Okasha & Rogers, 2004)*.

Specimen	Max. Load (kN)	EEEP Yield Load (kN)	k_e (0.4 max load) (kN/mm)	k_s (max load) (kN/mm)
CSP12.5-PR M	1.376	1.192	2.383	0.210
CSP25-PR M	1.740	1.487	1.513	0.197
OSB12.5-PR M	1.754	1.487	2.683	0.376
OSB25-PR M	1.955	1.643	1.168	0.194
DFP25-PR M	2.860	2.367	1.513	0.242

Table 3. Measured parameters of cyclic tests *(Okasha & Rogers, 2004)*.

Specimen	Max. Load (kN)	EEEP Yield Load (kN)	k_e (0.4 max load) (kN/mm)	k_s (max load) (kN/mm)
CSP25-PR C	2.228	2.024	0.691	0.209
OSB25-PR C	2.152	1.956	0.927	0.242
DFP25-PR C	3.186	2.785	0.735	0.311

The parameters listed in Tables 2 were adopted for the prediction of lateral resistance and deflection of full-scale shear walls under monotonic loading. Two failure modes were observed during the connection tests; screw pulled through the sheathing, and bearing / plug shear failure of the sheathing. These failure modes were in accordance with what was observed in the full-scale tests. The monotonic load capacity in connections loaded perpendicular-to-grain was higher (average 15%) than that in connections loaded parallel-to-grain. The results of the lower capacity parallel-to-grain specimens were therefore selected in order to establish the connection properties required for the monotonic wall analyses. The connection shear capacities listed in Table 3 for the cyclic tests were much higher than those obtained for the monotonic tests (Table 2). This was not in accordance with the full-scale test results, for which the shear strength of the cyclic tests was lower or close to that measured for the corresponding monotonic tests *(Chen, 2004)*. There are two possible explanations for this phenomenon; the loading speed for the cyclic connection tests was much faster than that used for the monotonic tests which resulted in a strain rate effect, and the large variation in the material composition of the wood panels, which may have a more noticeable effect when only single connections are tested compared with full size walls. In order to obtain a reasonable prediction of the wall behaviour, the load capacities for the monotonic tests were utilized for the shear walls under cyclic loading (Table 4). For the cyclic tests, the stiffness was taken as the average of the absolute values of the positive and negative results parallel-to-grain connection specimens. This average value takes into account the possible change in connection stiffness as the load in the wall changes direction.

Table 4. Parameters for prediction of cyclic tests *(Okasha & Rogers, 2004)*.

Specimen	Max. Load (kN)	EEEP Yield Load (kN)	k_e (0.4 max load) (kN/mm)	k_s (max load) (kN/mm)
CSP25-PR C	1.740	1.487	0.702	0.197
OSB25-PR C	1.955	1.643	0.926	0.230
DFP25-PR C	2.860	2.367	0.793	0.301

Analytical Method

A review of various analytical approaches that can be relied on to determine the strength and deflection of wood framed shear walls has been presented by Chen *(2004)*. These models were used to predict the lateral load capacity, which was defined as the yield shear strength, S_y, and the corresponding deflection of the wall $\Delta_{net,y}$ (Fig. 2). In this paper a summary of a strength and deflection model is presented. Figure 4 shows the assumed deformations and force distribution of a typical light gauge steel frame / wood panel shear wall. The lateral load at the top of the wall produces a moment and a horizontal force on the wall bottom. If the hold-downs are designed to fully transfer the tension force into the support through the end studs, the vertical forces acting on the end studs are balanced by the shear flow along the screw lines on the end studs, which is produced by sheathing rotation relative to the steel frame. The shear flow causes the axial forces in the end studs to distribute triangularly, with the maximum forces at the bottom of the end studs *(Stewart, 1987)*. With respect to the top track, if the screw spacing along the top edge of the sheathing and the spacing for anchors to the load beam are both small enough to assume the applied force is uniform, then no axial force exists. Similar for the bottom track, the applied force can be considered uniform if the screw and shear anchor spacing is small. The interior studs at the centreline of a panel or at the joint of two panels with the same width are assumed to carry no axial forces due to lateral loads causing in-plane shearing of the wall. The interior studs also provide out-of-plane support to stiffen the sheathing panel against shear buckling. The studs at the panel joints act as splices between adjacent wood panels; hence the design of the back-to-back studs needs to incorporate the shear force due to the opposite rotation of the two adjacent panels. Triangularly distributed forces also act perpendicular to the axes of the studs and tracks attached to the edges of panels, due to the relative displacements between studs and panels. In a capacity based design approach the size of the steel frame members is selected such that the frame itself does not fail. Given this information, and for simplification purposes, the frame members can be assumed to be rigid in the analytical models.

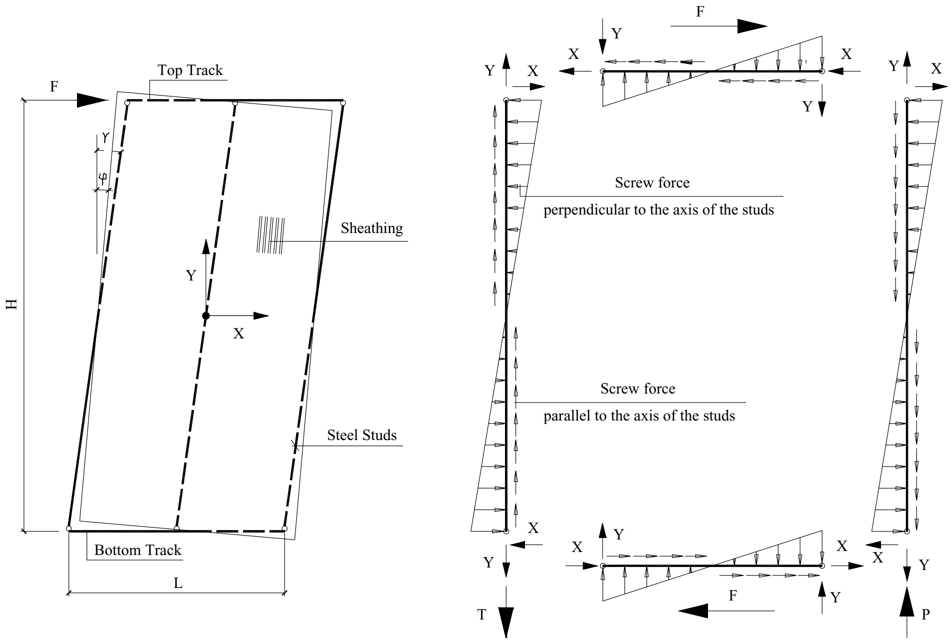

Figure 4. Deformations and force distribution in rigid framing members.

Simplified Strength Model

The racking performance of light gauge steel frame / wood panel shear walls is similar to that of wood framed shear walls. It is assumed that when a shear wall is subjected to lateral loading, the steel frame distorts as a parallelogram in which the top and bottom tracks maintain a horizontal position. The screws along the perimeter of a panel rotate about the flange of the studs; however, no obvious rotation of the screws connected to the interior studs occurs. The steel frame member connections act as hinges, which means that no lateral resistance develops in the frame itself. Rather, the lateral load is resisted by the composite action of the wood panels and steel framing through their relative rotation. The external work applied to the shear wall was assumed to be absorbed by the rotation of the screws.

In order to develop a model which can be used to predict the shear capacity of a light gauge steel frame / wood panel shear wall, some secondary behavioural characteristics need to be neglected or simplified. The following assumptions, which are similar to what was proposed by Källsner & Lam *(1995)* are applied in the model:

i) Deformation of the studs and tracks does not occur. These steel members are hinged to each other.
ii) The panels are rigid in their own plane and adjacent panels have no contact or overlap with each other.
iii) The relative displacements between the sheathing and framing are small compared with the panel size. The wood and steel also do not separate from each other during loading.
iv) No relative displacement exists between the centre of the sheathing panel and the corresponding centroid of the steel frame.
v) No horizontal panel joints exist in the same storey. Although in engineering practice, such joints are allowed, no tests with such configuration were included in this research.
vi) The shear wall is fully anchored onto the support or lower storey.
vii) The external work done by the racking loads is completely absorbed by the distortion of the sheathing-to-frame connections.
viii) The sheathing-to-frame connections have the same capacity in all directions.

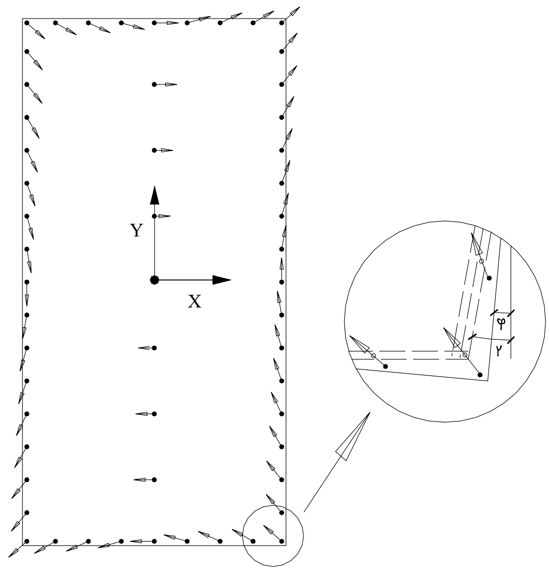

Figure 5. Assumed force distribution in sheathing connections.

The displacement of the sheathing relative to the steel frame can be viewed in Figure 4. All of the studs have rotated about their bottom ends through the angle γ, while the sheathing panel has rotated as a rigid body to an angle φ. In the simplified model these two rotations, which are taken as independent variables, result in the force distribution of the sheathing-to-frame connections as shown in Figure 5. Based on the assumed force distribution the shear capacity of the wall segment, $S_{y,wall}$, can be expressed as shown in Equation 1. The shear capacity is dependent on two factors, the first being the wall configuration including the connection pattern and the second the shear capacity per connection. This equation was originally presented by Källsner & Lam *(1995)* in their elastic model for wood framed shear walls.

$$S_{y,wall} = \frac{S_{y,conn}}{H \bullet \sqrt{\left(\frac{x_{\max}}{\sum_{i=1}^{N} x_i^2}\right)^2 + \left(\frac{y_{\max}}{\sum_{i=1}^{N} y_i^2}\right)^2}} \qquad \text{(Eq. 1)}$$

Where $S_{y,conn}$ is the shear strength of the individual sheathing connector (Tables 2&4), H is the height of the wall, x and y are the position of the fasteners (Fig. 5) and N is the number of sheathing connections.

Comparison of Predicted and Tested Shear Wall Capacity

Comparisons between the shear wall capacity measured during the laboratory testing and that predicted using the analytical model were performed. The intent was for the model to predict the shear wall capacity $S_{y,wall}$ at the level of the yield shear strength, S_y. At the same time, in order to verify that the elastic model by Källsner & Lam provided the most reasonable solution to predict the shear wall capacity, other models that were based on different assumptions and which have been applied in the prediction of wood frame shear walls, were also contained in the comparisons. These models, which were presented by Chen *(2004)*, include Källsner's & Lam's lower and upper plastic models, as well as models by Easley and McCutcheon. In this paper, only the test-to-predicted shear capacity results for the elastic Källsner & Lam model have been listed (Table 5). Each ratio and the associated statistical information represent the 16 wall configurations and a total of 103 individual shear wall test specimens.

Table 5. Full-scale shear wall test-to-predicted shear capacity *(Chen, 2004)*.

Monotonic Loading Cases	Källsner & Lam Elastic Model		
	Ratio	SD	COV
EEEP 12.5	1.270	0.181	0.143
EEEP 25	1.050	0.122	0.116
Max. Load 12.5	1.093	0.162	0.148
Max. Load 25	0.918	0.117	0.128
Cyclic Loading Cases	**Källsner & Lam Elastic Model**		
	Ratio	SD	COV
EEEP 25	1.012	0.129	0.127
Max. Load 25	0.885	0.122	0.137

In order to specify the connection test data that would best predict the shear wall capacity accurately, four cases were considered for the model for monotonic loading, *i.e.* EEEP 12.5, EEEP 25, Max. Load 12.5 and Max. Load 25; and two for cyclic loading, *i.e.* EEEP 25 and Max. Load 25. The shear capacity per connection, $S_{y,conn}$, was represented by the EEEP yield capacity or the maximum shear load; at the same time, two edge distances were considered, namely 12.5 mm and 25 mm. These cases correspond to the connection strength data listed in Tables 2 & 4. Based on the monotonic test predictions, the cases with an edge distance of 12.5 mm were not included in the predictions of cyclic tests. All of these cases were considered because it was not known which one would best represent the behaviour of the connection in the prediction of the performance of a full-size shear wall. The prediction using each model under each case was then compared with the average shear capacity, $S_{y,wall}$, of the tested full-scale walls with the matching sheathing configuration. An average shear capacity was obtained from the three or more tests that were performed for each wall configuration. Due to the large amount of the data, only the combined test-to-predicted ratios are listed in Table 5. As can be seen, the predicted shear capacity obtained using the test data from connections with a 25 mm edge distance and where the EEEP method was relied on to obtain the connection strength are the most accurate. A more detailed discussion of the comparison is provided by Chen *(2004)*.

Simplified Deflection Model

In the simplified deflection model, the same assumptions adopted for the strength model were made, except that the sheathing panels were not considered to be rigid. In contrast, the sheathing panels were defined as isotropic and deformable in terms of material properties; hence, the shear strain was assumed to be uniform over a whole panel. The purpose of the deflection model was to predict the deflection of the wall, $\Delta_{net,y}$, corresponding to the yield strength, S_y (or yield load capacity $S_{y,wall}$). The total displacement of the steel frame can be determined by considering the rotation of the frame and the bottom slippage on the support (Eq. 2).

$$\Delta_{walltop} = \frac{1}{k} FH^2 \bullet \left(\frac{1}{\sum_{i=1}^{N} x_i^2} + \frac{1}{\sum_{i=1}^{N} y_i^2} \right) + \frac{F}{GLt} + \left[\left(\frac{\Delta_{baseslip1} + \Delta_{baseslip2}}{2} \right) \right] + \left[\left(\Delta_{uplift1} - \Delta_{uplift2} \right) \times \frac{H}{L} \right] \quad \text{(Eq. 2)}$$

Where k is the stiffness of the individual sheathing connector (Tables 2&4), F is the applied shear force obtained from Eq. 1, H and L are the dimensions of the wall, G is the shear modulus of the wood sheathing, t is the thickness of the wood sheathing, Δ represents the base slip and uplift displacements, x and y are the position of the fasteners and N is the number of sheathing connections..

The final two components in Eq. 2 can be affected by many factors, such as the type of frame-to-support connections/anchorage, the shear modulus of these connections, the friction between a wall and its support, extension and slippage of the hold-down connections and the deformation of the steel frame. Although these factors have an impact on the behaviour of a tested wall, their inclusion would overly complicate the model, and hence they were not considered. Therefore, the deflection model used herein to predict the net lateral deflection was as shown in Eq. 3; which is similar to that presented by Källsner & Lam *(1995)* for predicting the deflection of wood framed shear walls.

$$\Delta_{net} = \frac{1}{k} FH^2 \bullet \left(\frac{1}{\sum_{i=1}^{N} x_i^2} + \frac{1}{\sum_{i=1}^{N} y_i^2} \right) + \frac{F}{GLt} \quad \text{(Eq. 3)}$$

Comparison of Predicted and Tested Shear Wall Deflection

Comparisons between the deflections measured during testing and the predictions made with Eq. 3 were performed to verify the accuracy of the model introduced above. Meanwhile, in order to verify that Källsner's & Lam's elastic model is more appropriate for the prediction of shear wall deflection than other models, Easley's model and McCutcheon's model were included in the comparison. The results of the Källsner & Lam model are presented in Table 6; the remaining test-to-predicted deflection ratios were tabulated by Chen *(2004)*. The same connection property cases, as described in the comparison of strength models, were incorporated in the deflection models. However, additional combinations were necessary because the deflected position of the wall depends on the estimated force, which was determined with Eq. 1 and the different connection strength values (Tables 2&4). Each combination of the listed deflection models and loading cases included all 16 wall configurations; a total of 103 individual shear wall test specimens. Only the combined ratio of the full-scale test-to-predicted deflection in each combination is listed in Table 6. More detailed information is provided by Chen *(2004)*.

For the most part, the prediction of lateral deflections was not as accurate as that of the lateral shear wall resistance. This can likely be attributed to the strong nonlinear behaviour of the sheathing-to-frame connections, as well as the overall nonlinear load *vs.* resistance performance of the shear walls (Fig. 2). The predictions based on the initial stiffness, k_e, of a connection tend to underestimate the lateral deflection under monotonic loading. In contrast, the predictions that incorporate the stiffness based on the ultimate load, k_s, can significantly overestimate the wall deflection. Källsner's & Lam's elastic model can provide a reasonable estimate of the shear wall deflection if the EEEP 25 load level and initial stiffness, k_e, connection properties are used. It should be noted, however, that the base slip and uplift of the wall have been ignored in the calculation of deflection. A variation in the type of holddowns and anchor bolts used for the full-scale shear wall tests could result in a change to the actual deflection of a wall.

Table 6. Full-scale shear wall test-to-predicted shear deflection *(Chen, 2004)*.

Monotonic Loading Cases	Källsner & Lam Elastic Model		
	Ratio	SD	COV
EEEP 12.5 & k_e	2.236	0.380	0.170
EEEP 25 & k_e	1.441	0.303	0.210
Max. Load 12.5 & k_e	1.923	0.339	0.176
Max. Load 12.5 & k_s	0.431	0.075	0.174
Max. Load 25 & k_e	1.263	0.301	0.238
Max. Load 25 & k_s	0.307	0.096	0.313
Cyclic Loading Cases	Källsner & Lam Elastic Model		
	Ratio	SD	COV
EEEP 25 & k_e	0.886	0.124	0.139
Max. Load 25 & k_e	0.775	0.133	0.171
Max. Load 25 & k_s	0.293	0.092	0.313

Conclusions and Recommendations

A number of analytical models, which had originally been developed for wood framed shear walls, were evaluated with respect to their ability to predict the strength and deflection of a series of single-storey light gauge steel frame / wood panel shear wall test specimens *(Boudreault, 2005; Branston, 2004; Chen, 2004)*. A simple analytical model based on the work of Källsner & Lam *(1995)* was presented and a comparison with the test results was completed. This was made possible by the sheathing connection tests that were carried out by Okasha & Rogers *(2004)*. The lateral shear yield resistance and the deflection of full-scale walls can be effectively predicted with the model if appropriate connection test data is available. Good agreement was obtained between the predicted and test strength values in both monotonic and cyclic cases using the Källsner & Lam approach. The deflection model only showed satisfactory agreement with the test results. The yield strength and initial stiffness per connection with 25 mm edge distance can be relied on to predict the yield lateral resistance and deflection of full-scale shear walls, if both the connection and full-scale test results are analyzed using the EEEP methods. In order to better predict the strength and deflection of the full-scale shear walls, the conditions for connection tests need to be kept consistent with those in full-scale tests, such as the loading speed and edge distance. In future connection tests, it is suggested that the edge distance parallel to the loading direction be the same as the perimeter screw spacing in the full-scale shear wall tests. As well, in order to obtain a better prediction of the full-scale tests, each connection test specimen should contain at least three screws to account for the variation of sheathing material.

Acknowledgments

The authors would like to acknowledge the support provided by the Natural Sciences and Engineering Research Council, the Canada Foundation for Innovation, the Canadian Sheet Steel Building

Institute, Bailey Metal Products Ltd., the Structural Board Association, Grant Forest Products Inc., Simpson Strong-Tie Co. Inc., Grabber Construction Products, ITW Buildex and the Canam Group Inc..

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