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**MODELLING FORCE TRANSFER AROUND OPENINGS  
OF FULL-SCALE SHEAR WALLS**

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# Modelling Force Transfer Around Openings of Full-Scale Shear Walls

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## Abstract

Wood structural panel (WSP) sheathed shear walls and diaphragms are the primary lateral-load-resisting elements in wood-frame construction. The historical performance of light-frame structures in North America has been very good due, in part, to model building codes that are designed to preserve life safety. These model building codes have spawned continual improvement and refinement of engineering solutions. There is also an inherent redundancy of wood-frame construction using WSP shear walls and diaphragms. As wood-frame construction is continuously evolving, designers in many parts of North America are optimizing design solutions that require the understanding of force transfer between lateral load-resisting elements.

The design method for force transfer around openings (FTAO) has been a subject of interest by some engineering groups in the U.S., such as the Structural Engineers Association of California (SEAOC). Excellent examples of FTAO targeted to practitioners have been developed by a number of sources. However, very little test data are available to confirm design assumptions. The building code requirements for FTAO are vaguely written with the requirement that the methods must meet “rational analysis”. Consequentially, countless techniques have been developed based on this performance-based notion. This paper discusses three methods which are generally accepted as meeting the rational analysis criterion. The drag strut, cantilever beam and Diekmann technique were examined in this study, which resulted in a wide range of predicted FTAO forces. This variation in predicted forces results in some structures being either over-built or less reliable than the intended performance objective.

This paper covers the two distinct portions of this research, the experimental study and the modelling analysis. Although the experimental study was partially reported by Skaggs et al. (2010), this paper is strictly focusing on the walls that were designed for force transfer around openings. Additional replications of the walls, and corrected strap force measurements are included in this paper. The modelling analysis includes simplified modelling using traditional engineering techniques, and more advanced modelling utilizing nonlinear finite element analysis. The various models were supported by 19 full-scale, 2.4 m x 3.6 m (8 ft x 12 ft) walls with various types and sizes of openings. Eight additional wall tests were conducted as part of this research program (Yeh, et al., 2011), however, these walls were not detailed for force transfer around openings, thus were not included in this paper.

This study was undertaken by a joint effort between APA – The Engineered Wood Association and the USDA Forest Products Laboratory (FPL), Madison, WI under a joint venture agreement funded by both organizations. The University of British Columbia, Vancouver, BC, provided the computer shear wall model simulation and analysis.

# 1. Introduction

Wood structural panel sheathed shear walls are important lateral force resisting components in wood-frame construction. These assemblies are effective in resisting seismic or wind loads. Wall openings for windows and doors, however, can greatly reduce the lateral resistance due to the discontinuity of load transfers as well as high force concentration around openings. The North American building codes provide three design alternatives for walls with openings. The first solution is to ignore the contribution of the wall segments above and below openings and only consider the full height segments in resisting lateral forces, often referred to as segmented shear wall method. This method could be considered the traditional shear wall method. The second approach, which is to account for the effects of openings in the walls using an empirical reduction factor, is known as the “perforated shear wall method”. This method has tabulated empirical reduction factors and a number of limitations on the method. In addition, there are a number of special detailing requirements that are not required by the other two methods. The final method is codified and accepted as simply following “rational analysis”. Much engineering consideration has been given to this topic (SEAOSC Seismology Committee, 2007) and excellent examples targeted to practitioners have been developed by a number of sources (SEAOC, 2007, Breyer et al. 2007, Diekmann, 1998). However, unlike the perforated shear wall method, very little test data has been collected to verify various rational analyses. The purpose of this study was to collect data on actual forces that are transferred around openings, and to compare, both simplified rational analysis as well as more rigorous finite element analysis.

## 2. Test Plan

In an effort to collect internal forces around openings of loaded walls, a series of twelve wall configurations were tested (Yeh et al, 2011). For this paper, a subset of eight assemblies will be discussed, as shown in Figure 1. The schematics in Figure 1 include the framing and sheathing plan, and the location of anchor bolts, hold downs and straps. This test series is based on the North American code permitted walls nailed with 10d common nails (3.75 mm diameter by 76 mm long or 0.131 in. diameter by 3 in. long) at a nail spacing of 51 mm (2 in.). The sheathing used in all cases was nominal 12 mm (15/32 in.) thick oriented strand board (OSB) APA STR I Rated Sheathing. All walls were 3.66 m (12 ft) long and 2.44 m (8 ft) tall. The lumber used for all of these tests was kiln-dried 38 x 89 mm (1-1/2 x 3-1/2 in.) Douglas-fir, purchased from the open market, and tested after conditioning to indoor laboratory environments (i.e. dry conditions). Additional framing information and boundary condition attachments are discussed in Yeh et al. (2011).

Walls 4, 5 and 6 have pier widths consistent with the narrowest segmented walls permitted by the code (height-to-width ratio of 3.5:1) when overturning restraint (hold-downs) is used on each end of the full height segments. The height of the window opening for Walls 4, 6 and 8 was 0.91 m (3 ft). Walls 5 and 9 had larger window heights of 1.52 m (5 ft). Wall 6 was common to Wall 4 with the exception that the typical 1.22 x 2.44 m (4 x 8 ft) sheathing was “wrapped around” the wall opening in “C” shaped pieces. This framing technique is commonly used in North America. It can be more time efficient to sheath over openings at first and then remove the sheathing in the openings area via a hand power saw or router.

Wall 8 has a pier height-to-width ratio of the full height segments of 2:1. Walls 10 and 11 contain very narrow wall segments for use in large openings such as garage fronts. The two walls are designed with openings on either side of pier and only on wall boundary, respectively. Finally, Wall 12 contains a wall with two asymmetric openings.

Most walls were tested with a cyclic loading protocol following ASTM E 2126, Method C, CUREE Basic Loading Protocol. The reference deformation,  $\Delta$ , was set as 61 mm (2.4 in.). The term  $\alpha$  was 0.5, resulting in maximum displacements applied to the wall of +/- 121 mm (4.8 in.). The displacement-based protocol was applied to the wall at 0.5 Hz with the exception of Wall 8b, which was loaded at 0.05 Hz. Two walls (Wall 4c and 5c) were tested following a monotonic test in accordance with ASTM E 564.

Finally, monotonic racking tests were conducted with the load being transferred directly into the top plate; thus no load head was utilized. The wall remained planar via structural tubes and low friction rub blocks directly bearing on face and back side of wall. For walls detailed as force transfer around openings, two Simpson Strong-Tie HTT22 hold-downs in line (facing seat-to-seat) were fastened through the sheathing and into the flat blocking. The hold-downs were intended to provide similar force transfer as the typically detailed flat strapping around openings. The hold-downs were connected via a 15.9 mm (5/8 in.) diameter calibrated tension bolt for measuring tension forces.

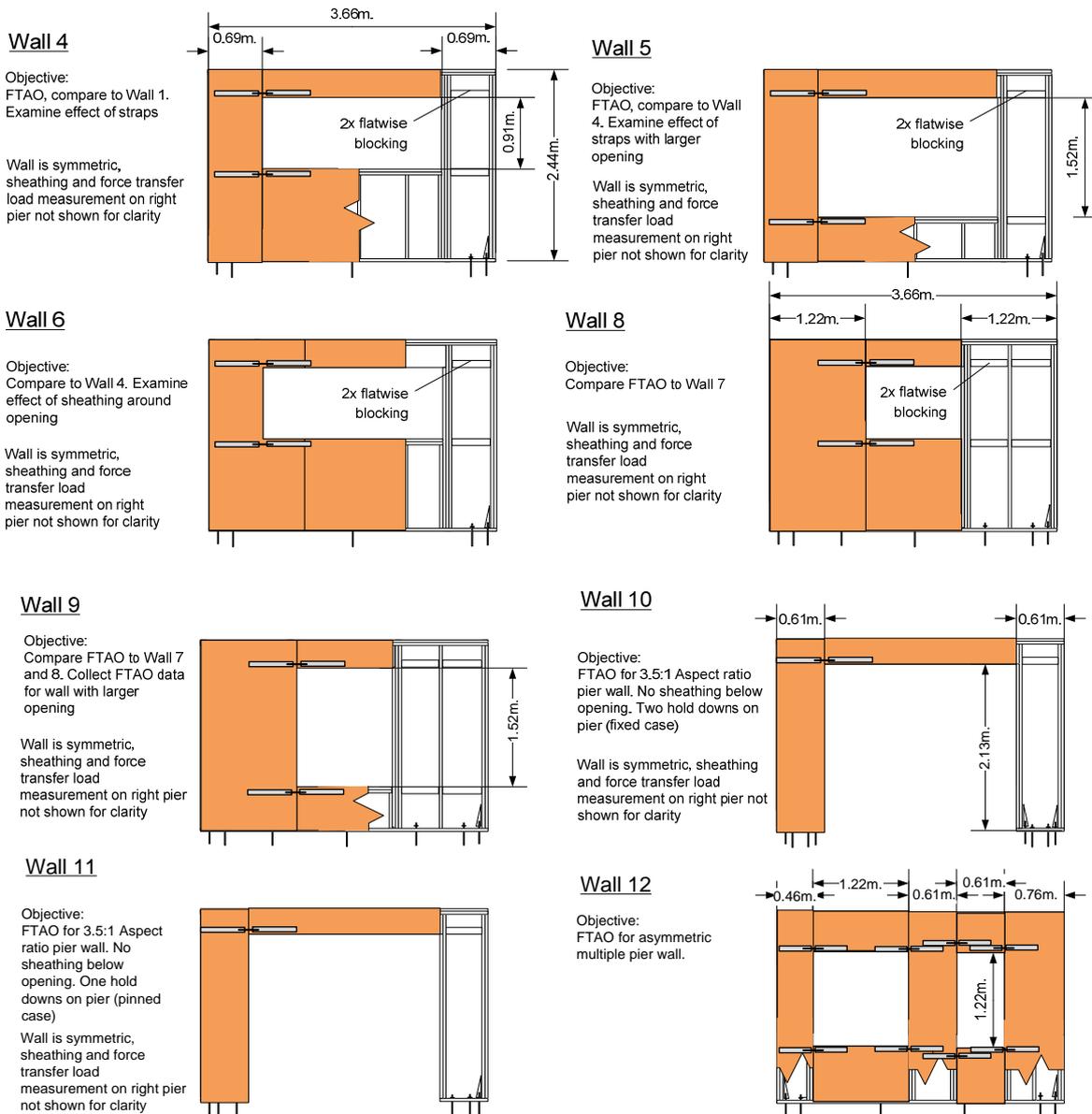


Figure 1. Test schematics for various force transfer around openings assemblies

### 3. Test Results

Table 1 presents the results of the global wall and calculated load factors. The allowable stress wall capacity is based on the code listed allowable unit shear multiplied by the effective length of the wall, as determined by the sum of the lengths of the full height piers. Table 1 also provides measured strap forces when the wall was subjected to the allowable stress wall capacity. Yeh et al. (2011) provides a comprehensive analysis of these wall tests.

Table 1. Global response of tested walls and strap forces.

Wall ID	Effective Wall Length <sup>(1)</sup>		Allowable Wall Capacity <sup>(2)</sup>		Average Maximum Load <sup>(3)</sup>		ASD Load Factor <sup>(4)</sup>	Measured Strap Forces <sup>(5)</sup>			
	(m)	(ft)	(kN)	(lbf)	(kN)	(lbf)		Top		Bottom	
								(kN)	(lbf)	(kN)	(lbf)
Wall 4a	1.37	4.5	17.4	3,915	66.4	14,930	3.81	3.05	687	6.61	1,490
Wall 4b	1.37	4.5	17.4	3,915	76.7	17,240	4.40	2.49	560	6.57	1,480
Wall 4c <sup>(6)</sup>	1.37	4.5	17.4	3,915	77.3	17,370	4.44	2.97	668	5.85	1,320
Wall 4d	1.37	4.5	17.4	3,915	68.2	15,330	3.92	4.47	1,010	7.41	1,670
Wall 5b	1.37	4.5	17.4	3,915	60.0	13,490	3.44	8.37	1,880	8.04	1,810
Wall 5c <sup>(6)</sup>	1.37	4.5	17.4	3,915	52.9	11,890	3.04	7.17	1,610	7.76	1,740
Wall 5d	1.37	4.5	17.4	3,915	52.0	11,680	2.98	7.26	1,630	10.26	2,310
Wall 6a	1.37	4.5	17.4	3,915	53.1	11,950	3.05	1.87	421	2.12	477
Wall 6b	1.37	4.5	17.4	3,915	60.4	13,580	3.47	2.71	609	2.73	614
Wall 8a	2.44	8.0	31.0	6,960	68.5	15,390	2.21	4.38	985	5.99	1,350
Wall 8b <sup>(7)</sup>	2.44	8.0	31.0	6,960	69.0	15,520	2.23	6.64	1,490	4.80	1,080
Wall 9a	2.44	8.0	31.0	6,960	67.8	15,250	2.19	7.45	1,670	7.35	1,650
Wall 9b	2.44	8.0	31.0	6,960	74.1	16,650	2.39	7.43	1,670	7.09	1,590
Wall 10a	1.22	4.0	15.5	3,480	33.2	7,470	2.15	7.03	1,580	n.a.	
Wall 10b	1.22	4.0	15.5	3,480	31.0	6,980	2.00	8.90	2,000	n.a.	
Wall 11a	1.22	4.0	15.5	3,480	28.8	6,480	1.86	10.97	2,470	n.a.	
Wall 11b	1.22	4.0	15.5	3,480	25.2	5,670	1.63	13.62	3,060	n.a.	
Wall 12a	1.83	6.0	23.5	5,220	71.3	16,030	3.07	3.59	807	5.17	1,160
Wall 12a	1.83	6.0	23.5	5,220	66.4	15,010	2.88	4.82	1,080	4.45	1,000

<sup>(1)</sup> Based on sum of the lengths of the full height segments of the wall.

<sup>(2)</sup> The shear capacity of the wall is the effective wall length times the allowable unit shear capacity, 12.70 kN/m (870 plf).

<sup>(3)</sup> The average of the absolute minimum negative and maximum positive applied forces.

<sup>(4)</sup> Average load applied to the wall divided by the wall capacity.

<sup>(5)</sup> Reported strap forces evaluated at the allowable wall capacity.

<sup>(6)</sup> Monotonic test.

<sup>(7)</sup> Loading duration increased by 10x.

### 4. Model Development

Typically walls that are designed for force transfer around openings attempt to reinforce the wall with openings such that the wall performs as if there was no opening. Generally increased nailing in the vertical and the horizontal directions as well as blocking and strapping are common methods being utilized for this reinforcement around openings. The authors are aware of at least three practical techniques which are generally accepted as rational analysis. The “drag strut” technique is a relatively simple rational analysis which treats the segments above and below the openings as “drag struts” (Martin, 2005). This analogy assumes that the shear loads in the full height segments are collected and concentrated into the sheathed segments above and below the openings. The second simple technique is referred to as “cantilever beam”. This technique treats the forces above and below the openings as moment couples, which are sensitive to the height of the sheathed area above and below the openings. The mathematical development of these two

techniques is presented by Martin (2005). Finally, the more rigorous mathematical technique is typically credited to a California structural engineer, Edward Diekmann, and well documented in the wood design textbook by Breyer et al. (2007). This technique assumes that the wall behaves as a monolith and internal forces are resolved by creating a series of free body diagrams. This is a common technique used by many west coast engineers in North America. Although the technique can be tedious for realistic walls with multiple openings, many design offices have developed spreadsheets based on either the Diekmann method or SEAOC (2007). The three aforementioned techniques could be considered practical rational analysis techniques.

For more advanced modelling, WALL2D was developed by the University of British Columbia, Vancouver, BC, Canada to model the behaviour of wood shear walls subjected to monotonic or cyclic loads (Li, et al. 2011a). This model consists of linear elastic beam elements for framing members, orthotropic plate elements for sheathing panels, linear springs for framing connections, and nonlinear oriented springs for panel-frame nailed connections. The model does not consider the rotational stiffness of framing connections. WALL2D accounts for the nonlinear behaviour of nailed connections as well as addresses the strength/stiffness degradation and pinching effects due to cyclic loading (Li, et al. 2011b). In this study, the nonlinearity in the tension-only strap connections around openings and hold-down connections was considered by nonlinear tension springs. Additionally, a type of asymmetric linear springs with higher compression stiffness but lower tension stiffness has also been introduced to consider the relatively high contact stiffness between header and blocking and wall studs when they are pushing against each other. Figure 2 illustrates the modified WALL2D model used in this study.

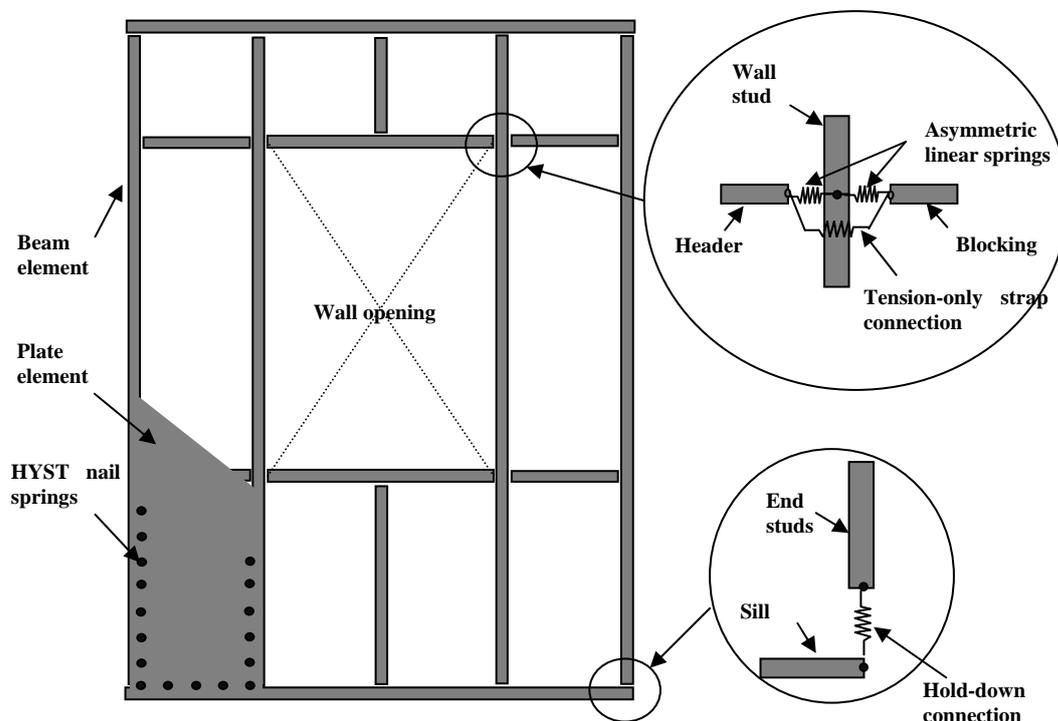


Figure 2. Schematics of WALL2D model for perforated wood-frame walls

Since nailed connections typically govern the shear behaviour of wood structural panel sheathed walls, nailed connection tests were conducted to calibrate the model, as shown in Figure 3. Additional explanation on these tests and calibration procedures and modelling

parameters can be found in Li, et al. (2011a). Further discussion of the modelling assumptions of WALL2D is found in Li, et al. (2011b).

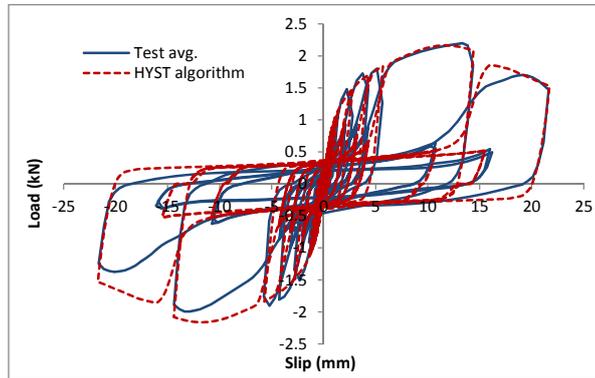


Figure 3. Calibrated nail model vs average test data.

## 5. Model Results

In the test program, the wall specimens were loaded so that maximum amplitudes of cycles in the CUREE basic protocol exceeded 100 mm (4 in.). The test results showed that, at a wall drift ratio of 2.5% (61 mm or 2.4 in.), these walls reached or approached their peak loads. In design practice, engineers are interested in evaluating the strap forces under the wall design load level which is normally significantly lower than the peak load. Therefore, in this study, the wall models were loaded until the maximum magnitudes of cycles reached 2.5% drift ratio. In general, the model predictions of eight wall configurations agreed well with the test results in terms of global load-drift responses and strap force responses, as illustrated in Figure 4 and 5, respectively.

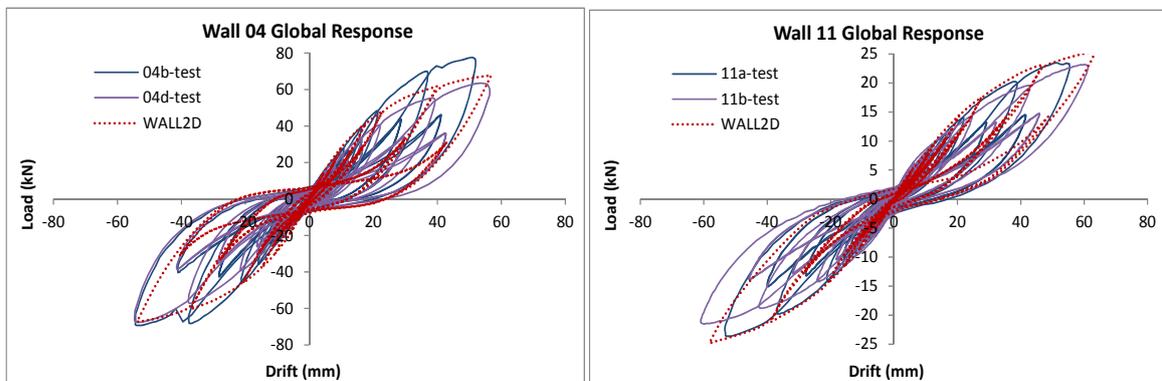


Figure 4. Global response of WALL2D model as compared to cyclic test data

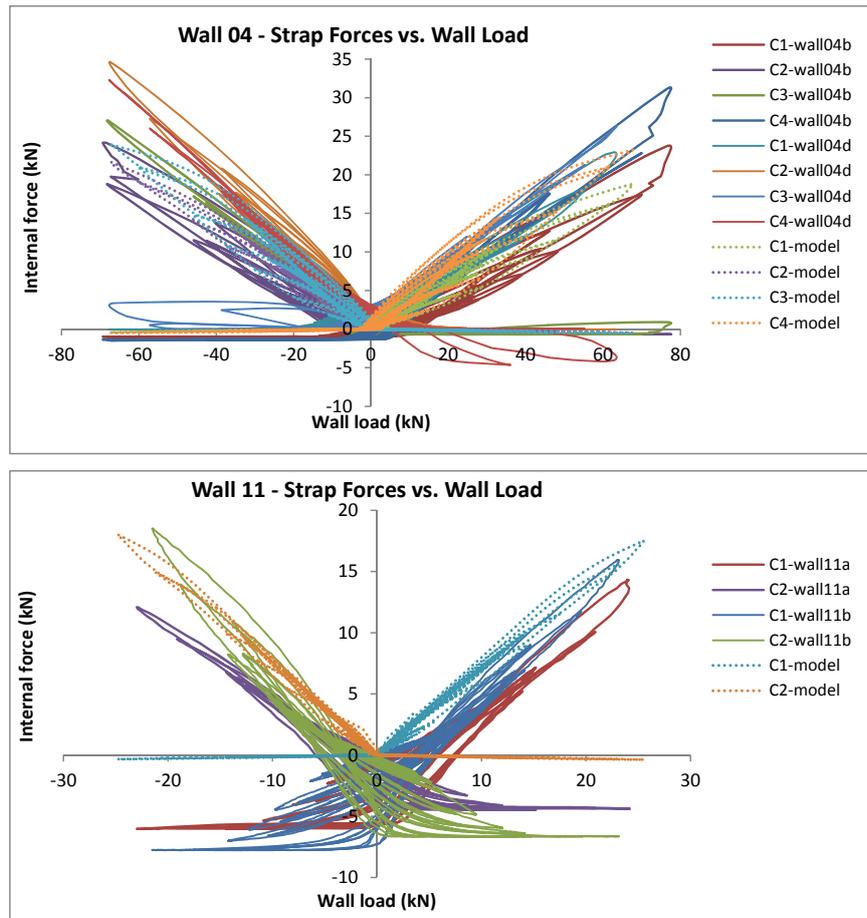


Figure 5. WALL2D predicted FTAO strap forces vs. test results

In order to design the strap connectors, it is important to evaluate the maximum forces transferred around openings under the design loads. In the U.S., 12.7 kN/m (870 plf) is a typical tabulated design load for wood-frame shear walls. Accordingly, the allowable wall capacity for a shear wall is calculated by multiplying the unit capacity with the total effective wall length (i.e., considering full-height wall segments). At the wall design load level, the predicted strap forces on the top corners (C1 and C2) and bottom corners (C3 and C4) of the opening were retrieved and compared with the test results. As expected, when the size of openings increased while the length of full-height piers remained the same, the strap forces increased. For all the wall configurations, the maximum prediction error from WALL2D was for Wall 6. Wall 6 was a special case in which “C”-shape sheathings were wrapped around the opening, resulting in an average prediction error of -15.2%. Note that additional discussion on individual walls, as well as modelled strap forces is provided by Li et al. (2011b).

Table 4 gives the maximum strap forces of four corners around the opening from the test data, WALL2D model, and three simplified design methods under the design loads. The prediction errors are given in parentheses. It can be seen that the WALL2D prediction error ranged from -15.4% to +4.3%. Drag strut method consistently underestimated strap forces except for Wall 6 with the “C”-shape sheathing panels. Cantilevered beam, and Diekmann's method, however, seemed to be very conservative. The Diekmann's method, the most sophisticated calculation method among the three practical design methods, seemed to provide reasonable predictions for the walls with window-type openings. One

might consider including a correction factor of the Diemann method on order of 1.2 to 1.3 if more accurate FTAO predictions are desired. However, many structural engineers are conservative by nature, and that decision would vary from office to office. It should be noted that the strap forces in Wall 6 with “C”-shape sheathing could not be reasonably predicted by any of three simplified methods even with the correction factor. Obviously the force load path around the openings is being transferred by the “C”-shaped sheathing. Perhaps a mechanics of material model considering either sheathing tensile strength or sheathing shear strength could be utilized to model the amount of the force transfer through the sheathing. The behaviour of walls with “C”-shaped sheathing is an interesting phenomenon which needs further studies.

Table 2. Maximum strap forces<sup>(1)</sup> (kN) predicted by WALL2D & simplified design methods

Wall No.	MEASURED	PREDICTED			
	Average force from wall tests <sup>(2)</sup>	WALL2D	Drag strut Technique	Cantilever Beam Technique	Diemann Technique
4	6.61	6.23 (-5.7%)	5.44 (-17.7%)	19.90 (201.1%)	8.71 (31.8%)
5	8.69	9.07 (4.4%)	5.44 (-37.4%)	27.36 (214.8%)	14.51 (67.0%)
6	2.43	2.05 (-15.4%)	5.44 (124.1%)	19.90 (719.5%)	14.51 (497.5%)
8	5.51	5.75 (4.3%)	5.16 (-6.4%)	35.38 (542.0%)	8.26 (49.8%)
9	7.44	7.24 (-2.7%)	5.16 (-30.7%)	35.38 (375.5%)	13.76 (84.9%)
10	7.97	7.95 (-0.2%)	5.16 (-35.2%)	34.83 (337.0%)	n.a.
11	12.29	12.01 (-2.3%)	5.16 (-58.0%)	34.83 (183.4%)	n.a.
12	4.81	4.30 (-10.70%)	4.84 (0.6%)	21.28 (342.4%)	6.64 (38.0%).

<sup>(1)</sup> 1 lbf = 4.448 N

<sup>(2)</sup> Based on the maximum of the measured average top and average bottom strap forces.

## 6. Summary and Conclusion

This paper presents test data on a subset of twelve different wall assemblies more fully described in Yeh et al. (2011). The purpose of the analysis on this subset of 8 walls was to study the behaviour, both global and internal forces, of walls that were detailed to resist force transfer around opening. In general, the forces were transferred around openings utilizing straps except that Wall 6 also utilized sheathing for this load transfer mechanism. Several of these assemblies were tested with multiple replications, including variations in test method and loading duration. The replications showed good agreement between each other, even when walls were tested monotonically or cyclically, and when test duration was extended to ten times greater the original duration.

This paper also presented a study on force transfer around openings in perforated wood-frame shear walls using a finite element model called WALL2D, developed by the University of British Columbia. A total of eight wall configurations detailed for FTAO with different opening sizes and different lengths of full-height piers were modelled and analyzed. The model predicted wall load-drift hysteresis agreed well with the test results when the walls were loaded cyclically up to a drift ratio of 2.5%. At the wall design load level, the model predicted maximum strap forces around openings were also compared with the test results to check the model validity. It was also found that the model predictions agreed well with the test results compared with the three “rational” design methods commonly used by design engineers.

The current WALL2D model considers only the nonlinearities of panel-frame nail connections, hold-down connections, and strap connections around openings. It does not consider the nonlinearity or failure mechanism in sheathing panels and framing members. Therefore, it might over predict the wall response if those wall elements, in some situations, would also contribute significantly to wall nonlinearities. In fact, tearing failure of OSB sheathing panels was observed in some wall specimens when these walls had large deformations in the post-peak softening range. Furthermore, since framing members also play an important role in transferring loads among wall components in a perforated wall system, the model simulations would be more accurate if the properties of framing members, such as modulus of elasticity, were collected non-destructively before the walls were tested. Nevertheless, this model provides a useful tool to the study FTAO problem in perforated wood-frame walls. In future research, parametric studies can be further conducted to study the walls with different geometries, different opening sizes and different metal hardware for reinforcing corners of openings, providing more information for rational designs of perforated wood-frame walls.

Of the different models considered, one can conclude that the drag strut technique consistently underestimated the strap forces, and the cantilever beam technique consistently overestimated the strap forces. The Diekmann technique, the most computationally intensive of the practical methods, provided reasonable strap force predictions for the walls with window type openings. The more advanced nonlinear finite element model, WALL2D, provided a very accurate prediction for modelling the global wall results as well as the strap forces from FTAO. In the current form, WALL2D is likely too complicated for most engineering design offices; it is possible that this model could be used in the future for either developing simplified methods, or using the concepts in WALL2D to create a user friendly design tool.

## 7. Acknowledgements

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