Modeling Force Transfer around Openings in Wood-Frame Shear Walls

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Abstract: This paper presented a modeling study on force transfer around openings (FTAO) in wood-frame shear walls detailed for FTAO. To understand the load transfer in the walls, this study used a finite-element model WALL2D, which is able to model individual wall components, including framing members, sheathing panels, oriented panel-frame nailed connections, framing connections, hold-down connections, and strap connections for reinforcing the corners of openings. The various wall models were validated through laboratory testing of 12 full-scale 2.44 × 3.66 m (8′ × 12′) shear wall configurations, which included a variety of opening types and sizes. At the wall design load level, the predicted strap forces around openings also agreed well with the test results, in contrast with four simplified analytical methods commonly used in designing shear walls with openings detailed for FTAO. This wall model thus presents a useful tool to check the accuracy of the simplified methods and develop a better understanding of the behavior of wood-frame shear walls with openings. **DOI:** 10.1061/(ASCE)ST.1943-541X.0000592.

CE Database subject headings: Frames; Shear walls; Wood; Finite element method.

Author keywords: Wood frame; Shear walls; openings; Force transfer; Finite-element model.

Introduction

Wood structural panel sheathed shear walls are important lateral force resisting components in wood-frame construction to resist seismic or wind loads. Wall openings for windows and doors, however, can greatly reduce shear wall resistance because of the discontinuity of load transfers, as well as highforce concentration around openings. Currently, the building code in the U.S. [American National Standards Institute (ANSI) / American Forest & Paper Association (AF&PA) 2005] provides three solutions to design wood-frame shear walls with openings. The first solution, also used in the Canadian code [Canadian Standard Association (CSA) 2005], is so-called segmented shear wall design method, which considers only full-height wall segments and ignores contributions from wall segments above and below openings. The second solution takes into account wall segments above and below openings by introducing a shear capacity adjustment factor C_o, considering maximum opening height and the percentage of full-height sheathing (Sugiyama 1981). These walls with openings are not designed for force transfer around the openings (FTAO). The last solution, which has been used in engineering practice for a long time, is called the FTAO method, in which shear walls with openings are designed and detailed for FTAO so that nails, metal straps, and blocking members may be required to transfer loads and reinforce corners of openings. In the FTAO method, a rational analysis is required to estimate the force transfer and choose proper metal connectors. In recent years, the FTAO method has been a subject of interest by the engineering community, such as the Structural Engineers Association of California (SEAOC). Among various calculation methods generally accepted as rational analyses in practice, drag strut, cantilever beam, and Diekmann's method are most commonly used. However, variations in internal forces calculated by the different methods result in some walls being either overbuilt or less reliable than the intended performance objective. Martin (2005) presented a detailed review of commonly used design methods of shear walls: the segmented shear wall method, the drag strut method, and the cantilevered beam analog. Depending on wall geometries, the drag strut method and the cantilevered beam method can yield very different estimates. Diekmann (2005) discussed Martin's article and presented a method he had introduced previously based on Vierendeel truss analog (Diekmann 1997). Kolba (2000) performed a detailed experimental study on shear walls with openings detailed for FTAO focusing on the applicability of Diekmann's method. Although the results were inconclusive, detailed explanations of the assumptions used by Diekmann's method were provided. Robertson (2004) discussed different methods available to engineers for analyzing and designing FTAO in plywood sheathed shear walls. He also discussed building code requirements and analyzed several examples of wall configurations using the drag strut method, the cantilevered beam method, and the coupled beam analogy (a variant of Diekmann's method that seems to lack some equilibrium rigor). Large differences in estimated FTAO values were also found. Lam (2010) also

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Note. This manuscript was submitted on August 25, 2011; approved on February 17, 2012; published online on February 22, 2012. Discussion period open until May 1, 2013; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Structural Engineering***, Vol. 138, No. 12, December 1, 2012. ©ASCE, ISSN 0733-9445/2012/12-1419–1426/$25.00.

reviewed these four methods and compared the estimates of maximum FTAO in five wall configurations. It was also found that these methods may give significantly different results.

Previous studies on wood-frame shear walls with openings mainly focused on experimental studies and modeling of perforated walls that are not designed for FTAO. The main objective was to investigate the influence of the openings on shear wall capacity and to derive empirical reduction factors that can be directly used for design purposes. Andreasson et al. (2002) studied perforated wood-frame walls using finite element modeling to check the validity of assumptions on pinned or rigid framing connections in a shear wall model. The effect of partially and fully hold-down anchoring on full-height wall piers was also studied. Doudak et al. (2006) studied the monotonic behavior of five perforated wall configurations using experimental studies and computer modeling in *SAP2000* software (*SAP2000*). The effect of hold-down anchoring was also studied. No computer modeling work has been reported to study the behavior of wood-frame shear walls with openings particularly detailed for FTAO.

To develop a better understanding of the FTAO problem in wood-frame shear walls, in this study, a shear wall model called WALL2D was used to investigate eight configurations of shear walls detailed for FTAO with different opening sizes and different lengths of full-height wall piers. To check the model validity, its predictions were then compared with test results in terms of the global wall load-drift responses and the internal forces transferred around the openings. The FTAO values estimated by the four commonly used design methods were also presented. Some suggestions on estimating the FTAO were also provided according to the shear wall test program and the modeling work.

Shear Wall Test Program

A total of 12 types of walls with openings have been tested in a shear wall test program. These walls were 3.66 m (12 ft) long and 2.44 m (8 ft) high, and the frames were constructed with 38×89 mm (2 in. $\times$ 4 in.) Douglas fir lumber. APA STR-1–rated 12 mm (15/32 in.) thick OSB panels were sheathed on framing members by 10d common nails (3.68×76 mm or 0.145 in. $\times$ 3 in.). The nails were staggered on panel edges and spaced at 51 mm (2 in.). The nail field spacing was 305 mm (12 in.). Eight of the 12 wall configurations were designed with metal straps for FTAO. Strain gauges were installed to measure the strap forces and the hold-down forces. Fig. 1 shows one wall specimen with a window-type opening, and four straps were installed on its corners. C1 and C2 denote the straps installed on top corners, and C3 and C4 denote the straps on bottom corners. Fig. 2 shows the schematics of the eight wall configurations detailed for FTAO. These walls were symmetrically built, except that Wall 12 had two window openings and three full-height piers with different lengths. It should be noted that, in Wall 06, the OSB panels covering the full-height piers were cut into C-shape pieces and wrapped around the opening. Such a framing technique is also commonly used in North America. Walls 10 and 11 had very narrow wall segments for use in large openings such as garage openings. They had the same wall configurations except that Wall 11 had one hold-down on each pier, whereas Wall 10 had two hold-downs on each pier, representing stronger boundary fixity. In all wall specimens, Simpson Strong-Tie HDQ8 hold-downs were used to provide overturning restraints, and HTT22 tension ties were used as the straps for FTAO.

The test protocol followed ASTM E2126 Method C-CUREE basic protocol (ASTM 2010). The reference wall drift was set as 2.5% of the wall height of 2.44 m (8 ft.) [i.e., 61 mm (2.4 in.)]. The

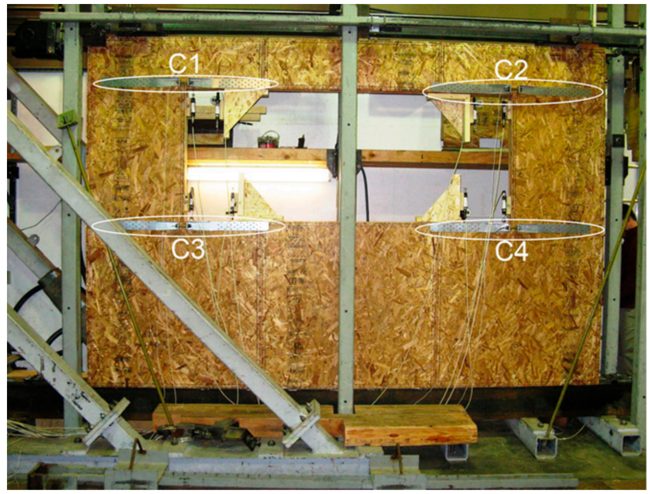

Fig. 1. Wall specimen and notations of FTAO straps

wall drift level was set on the basis of past experience of cyclic tests on wood-frame shear walls conducted at APA—The Engineered Wood Association. The displacement controlled loading rate was 0.5 Hz for all wall specimens except Wall 08b, which had a loading rate of 0.05 Hz. Detailed information about the shear wall test program can be found in a technical report (Yeh et al. 2011).

Shear Wall Model Simulations

Model Input

The WALL2D program, compiled in *Intel Visual Fortran 10.1* (*Intel (R) Fortran Compiler Integration 10.1*), was developed to model the behavior of wood shear walls subjected to monotonic or cyclic loads (Li et al. 2012). Its original version 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. A special feature about this model is the implementation of a mechanics-based protocol-independent algorithm called HYST, originally developed by Foschi (2000), to account for the nonlinear behavior of nailed connections. The HYST algorithm is able to fully address strength/stiffness degradation and the pinching effect in the nailed connections. 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 spring with higher compression stiffness but lower tension stiffness has also been introduced to consider the relatively high contact stiffness between header/blocking and wall studs when they are pushing against each other. Fig. 3 illustrates the modified WALL2D model used in this study.

Because nailed connections usually govern the shear behavior of structural panel-sheathed walls, nailed connection tests were also conducted to calibrate the HYST parameters. In the nail connection test, a 10d common nail was used to connect a piece of 38×89 mm Douglas fir lumber and a piece of OSB panel used in the wall specimens. A total of 15 specimens were tested under monotonic loading and cyclic loading, respectively. Fig. 4 shows the nail load-slip hystereses tested under the CUREE basic protocol (ISO 2003). The major failure mode was the nail pull-through. The average envelope curve from five replicates was used to calibrate the HYST

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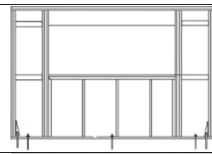
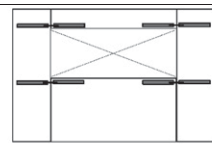
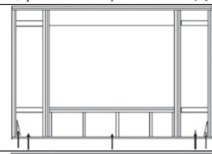
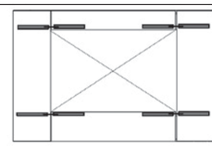
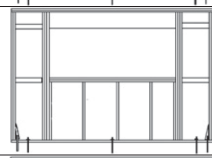
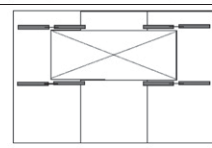
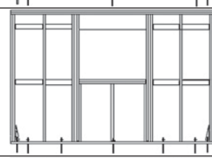
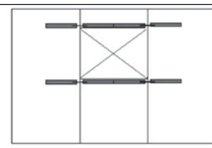
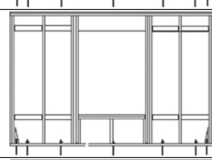
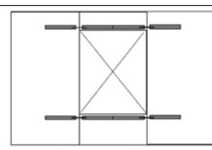
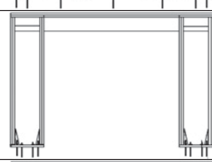
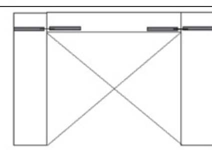
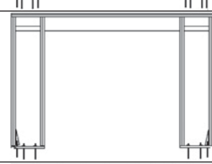
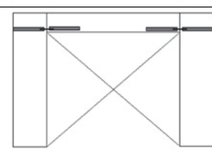
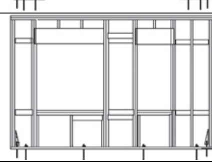
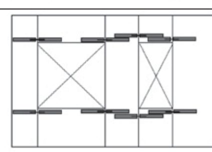
Wall No.	Configuration		Dimension	
	Frame, Hold-downs & Anchor bolts	Sheathing & FTAO Straps	Opening size (width x height)	Full-height pier width
4b 4d			2.29m x 0.91m (7.5' x 3')	1.37m (4'-6")
5b 5d			2.29m x 1.52m (7.5' x 5')	1.37m (4'-6")
6a 6b			2.29m x 0.91m (7.5' x 3')	1.37m (4'-6")
8a 8b			1.22m x 0.91m (4' x 3')	2.44m (8')
9a 9b			1.22m x 1.52m (4' x 5')	2.44m (8')
10a 10b			2.44m x 2.13m (8' x 7')	1.22m (4')
11a 11b			2.44m x 2.13m (8' x 7')	1.22m (4')
12a 12b			1.22m x 1.22m (4' x 4') 0.61m x 1.22m (2' x 4')	0.46m (1'-6") 0.61m (2') 0.76m (2'-6")

Fig. 2. Configurations of shear walls detailed for FTAO

parameters. Fig. 4 also shows that the calibrated HYST hysteretic loops agreed very well with the test results. The calibrated HYST was then implemented in the WALL2D model to represent the nailed connections. The calibration procedure, as well as the calibrated HYST parameters, can be found in work by Li et al. (2012).

According to the Canadian standard (CSA 2005), modulus of elasticity (MOE) of the Douglas-fir framing members was assumed to be 10 GPa (1,450 ksi); the Young's moduli E_x and E_y for the OSB sheathing were assumed to be 3.5 (508 ksi) and 2.0 GPa (290 ksi) along the major axis and the perpendicular axis, respectively; and the shear-through-thickness rigidity G_{xy} was assumed to be 0.5 GPa (73 ksi). Poisson ratios γ_{xy} and γ_{yx} of the OSB panels were assumed to be 0.13 and 0.23 (Thomas 2003).

In the wall specimens, HDQ8 hold-downs were connected with double 38×89 mm $(2 \times 4$ in.) end studs via 20-SDS screws [6.35×76 mm $(1/4 \times 3$ in.)], and HTT22 tension ties were fastened on the top of the OSB panels connecting the underlying headers and blocking members via 32 16d sinker nails [3.76×82.6 mm $(0.148 \times 31/4$ in.). Table 1 shows the allowable tension/compression loads and strength of the HDQ8 hold-downs and HTT22 tension ties (Simpson Strong-Tie 2010). In the wall model, the envelope curve of tension force-displacement relationship for hold-down connections was defined as a trilinear curve, as shown in Fig. 5. Under reversed cyclic loading, the load-displacement hysteresis was assumed to follow elastoplastic rules, and the unloading stiffness is the initial stiffness K_{t0}, defined as the ratio of the allowable tension load P_{ta} to the corresponding displacement Δ_{ta}. The same rule of hysteresis is also used for the tension-only HTT22 strap connections around the openings. In the shear wall test program, the headers above openings were built-up

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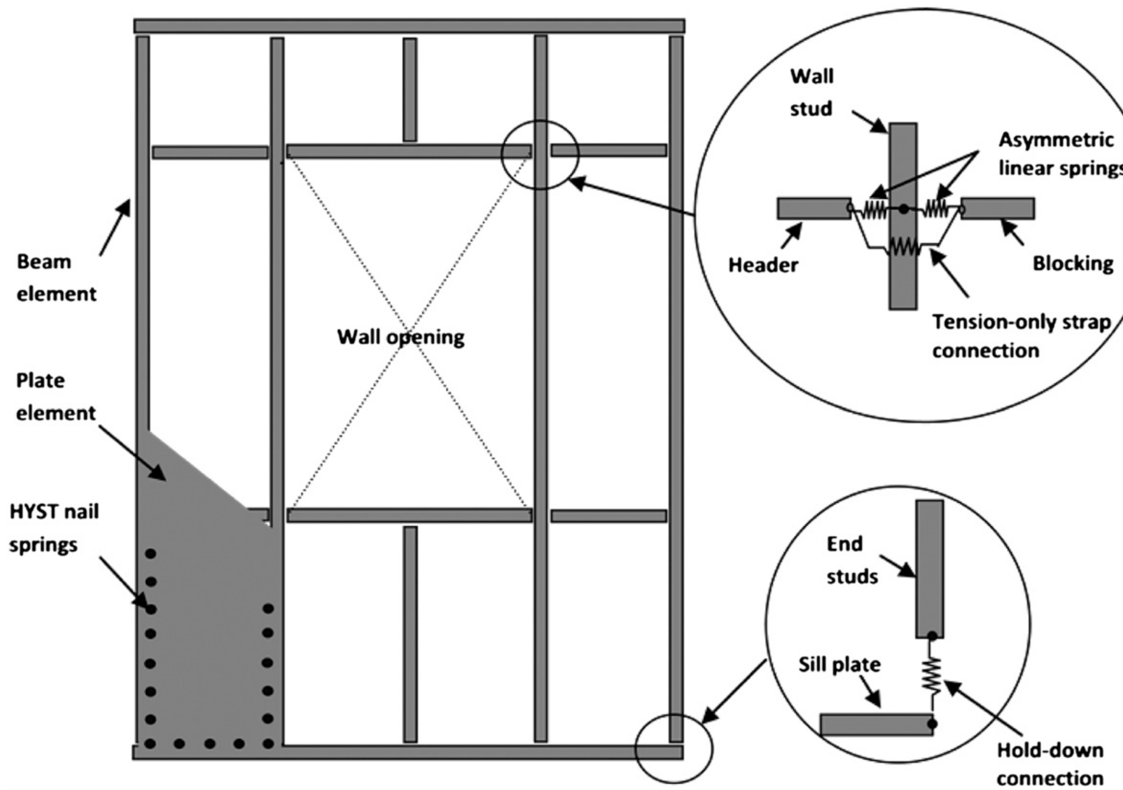

Fig. 3. Schematics of WALL2D for walls detailed for FTAO

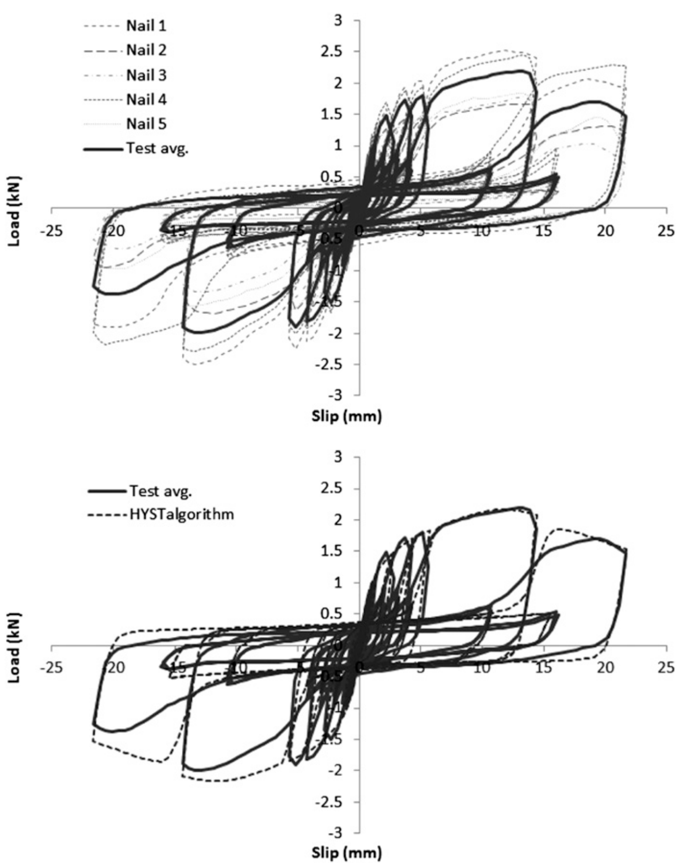

Fig. 4. Calibrated HYST hystereses versus test hystereses

double 38×286 mm $(2 \times 12$ in.$)$ lumber, with a 12.7 mm (0.5 in.) thick structural panel spacer between the two pieces of lumber. The headers were then sheathed by OSB panels. The strap connections to this assembly were identical to those at the window sill-plates. The failure modes for these strap connections would be the same. It should be noted that the assemblies for the headers have relatively high in-plane stiffness compared with normally sheathed wall segments. In the computer model, the headers were modeled by beam elements with 38×286 mm $(2 \times 12$ in.$)$ cross sections. Plate elements with strong nailing on the edges were attached onto the beam elements to maintain high in-plane stiffness and to transfer lateral forces.

Modeling 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 (2.4 in.) for these walls], the 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 a drift ratio of 2.5%. The displacement-controlled loading protocol is shown in Fig. 6.

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. Three wall configurations (Wall 04, Wall 11, and Wall 12) are illustrated here as examples. Fig. 7 shows the wall deformations at a 2.5% drift ratio plotted in the WALL2D program. It can be observed that the wall frames deformed in a parallelogram and the sheathing panels rotated

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Table 1. Capacity of Hold-Downs and FTAO Straps

	Allowable tensile load P_{ta}	Displacement at allowable tensile load Δ_{ta}	Tensile strength P_{ts}	Displacement at tensile strength Δ_{ts}	Allowable compressive load P_{ca}	Displacement at allowable compressive load Δ_{ca}
HDQ8	25.4 kN (5715 lb_f)	1.63 mm (0.064 in.)	35.5 kN (8,001 lb_f)	3.30 mm (0.130 in.)	34.3 kN (7,725 lb_f)	1.32 mm (0.052 in.)
HTT22	23.3 kN (5250 lb_f)	3.86 mm (0.152 in.)	32.6 kN (7,350 lb_f)	6.35 mm (0.25 in.)	Not available	Not available

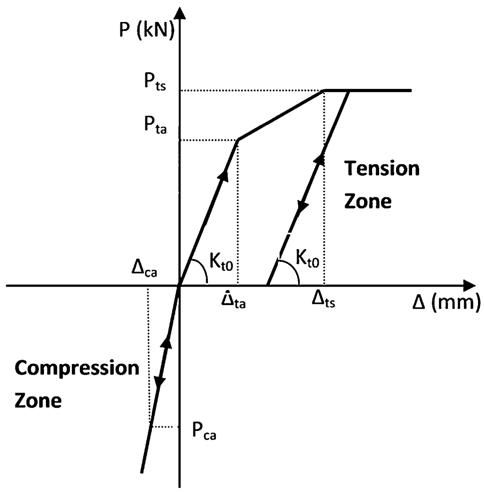

Fig. 5. Load-displacement hysteresis of hold-down connections

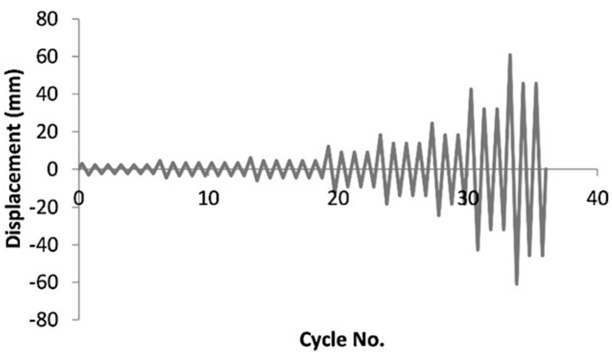

Fig. 6. Loading protocol in WALL2D simulations

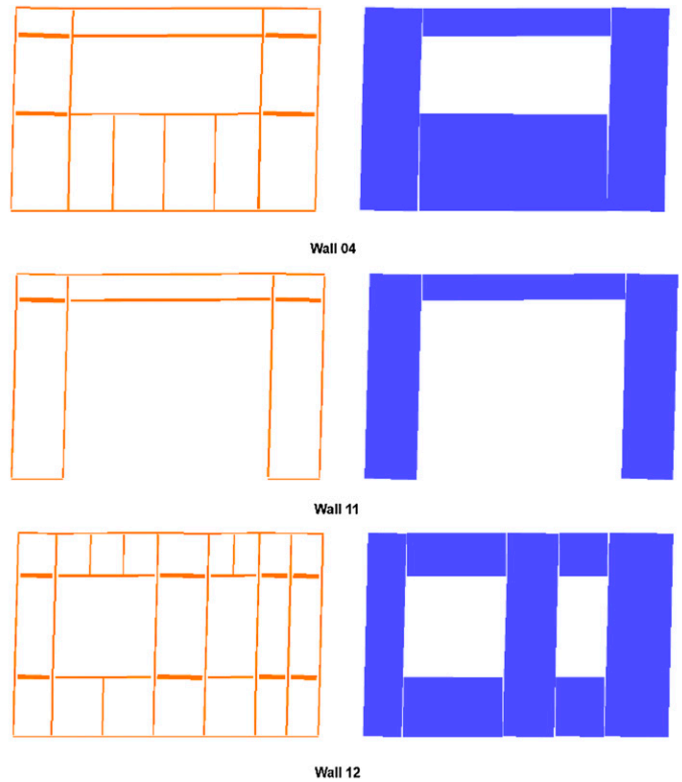

Fig. 7. Wall deformations plotted in WALL2D program (at a wall drift ratio of 2.5%)

a little. These were consistent with the observations from the shear wall tests. Fig. 8 shows the predicted global hystereses compared with the test loops. Fig. 9 shows the predicted responses of internal strap forces around the openings compared with the test measurements. In Fig. 9, the vertical y-axis denotes the magnitude of the strap forces and the horizontal x-axis denotes the corresponding racking loads applied on the wall. The notations of the straps have been illustrated in Fig. 1.

Wall 04 with a Window Opening

The opening size was 2.29×0.91 m (7.5×3 ft). Although the total length of the full-height piers was only 1.37 m (4.5 ft), this wall indicated very high strength and stiffness because of the strong nailing, the hold-down restraints, and the strap reinforcement around the opening. The difference between the peak loads for two wall specimens, however, seemed to be significant, especially in the positive excursions of the cycles, as shown in Fig. 8. The reason for this might be the variability in the wood material selected for these two walls. Test results of other wall configurations, did not show as much variation between any two wall specimens with the same configuration.

Wall 11 with a Door Opening

The opening size was 2.44×2.13 m (8×7 ft). Although the total length of full-height piers was about 89% of Wall 04, Wall 11 had a much lower load-carrying capacity than Wall 04 because of the large opening size, as indicated in Fig. 8. The peak load of Wall 11 was only 37% of the peak load of Wall 04. The contribution from the lower wall segment below the window opening in Wall 04 appeared to be very high and greatly enhanced the shear capacity. Also, both Wall 11 and Wall 10 contained very narrow piers and large openings. Under the design load level, both wall configurations experienced significantly larger strap forces than the other wall configurations. Wall 11 experienced the largest strap forces among all the wall configurations. It should be noted that Wall 11 had only one hold-down installed on each pier, whereas Wall 10 had two hold-downs on each pier, representing stronger boundary fixity. The results indicated that stronger boundary fixity may reduce FTAO values, because Wall 10 experienced an approximately one-third reduction in strap forces compared with Wall 11.

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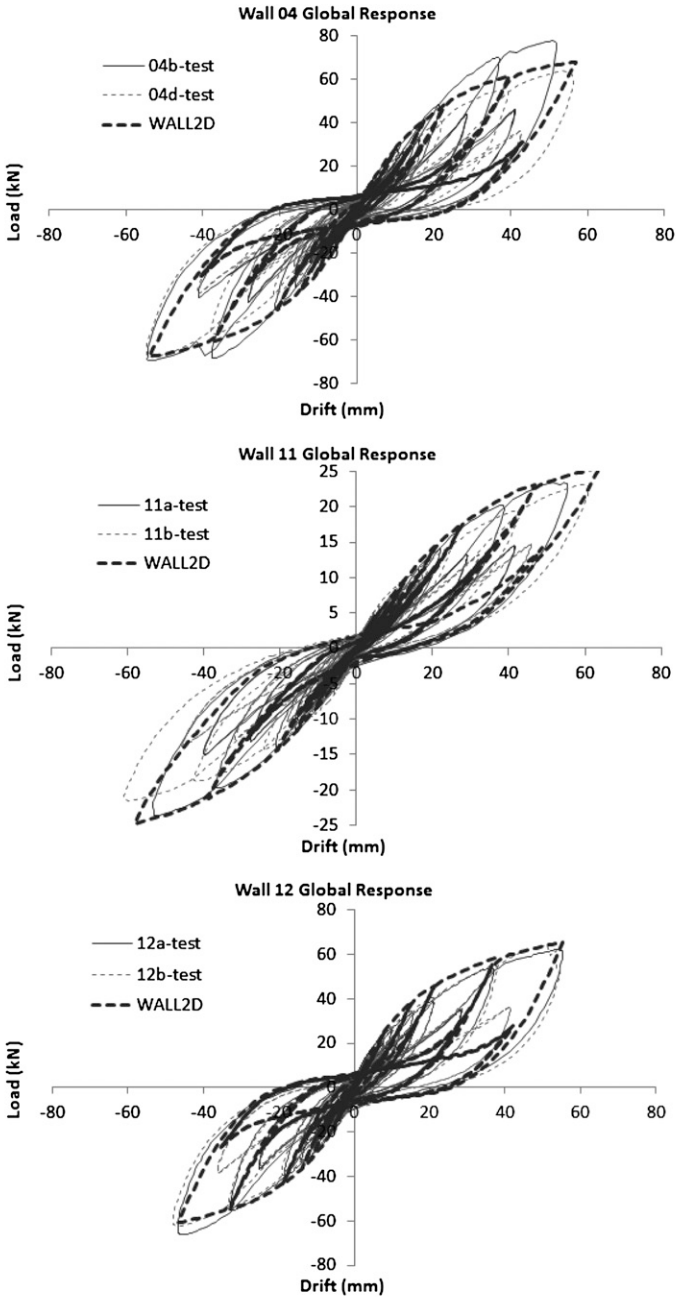

Fig. 8. WALL2D predicted hysteretic loops versus test loops

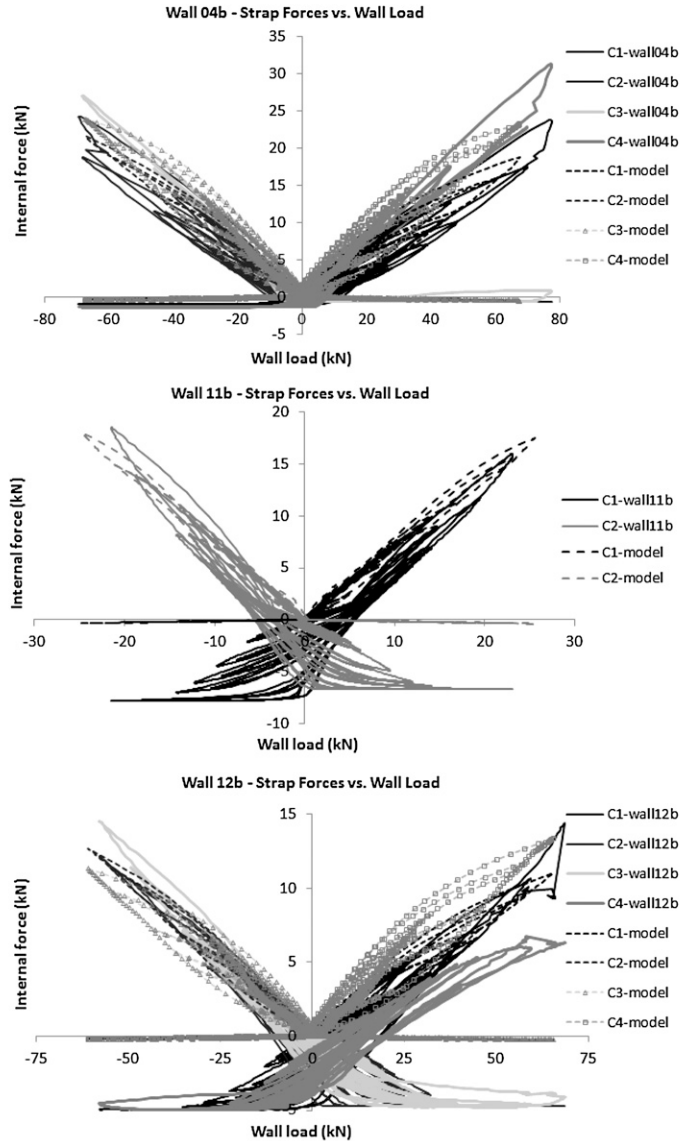

Fig. 9. WALL2D predicted FTAO strap forces versus test results

Wall 12 with Two Window Openings

Wall 12 was the only asymmetric wall configuration in this study. It had two openings of 1.22×1.22 m (4×4 ft) and 0.61×1.22 m (2×4 ft). Both the model predictions and the test observations indicated that the larger opening of 1.22×1.22 m (4×4 ft) experienced much greater strap forces than the smaller opening. Thus, in Fig. 9, only the responses of strap forces around the larger opening are shown.

To design the strap connectors, it is important to evaluate the maximum forces transferred around openings under the design loads. In the United States, 12.7 kN/m (870 $\mathrm{lb_f}$/ft) is a typical tabulated design load for wood-frame shear walls (ANSI/AF&PA 2005). Accordingly, the design load for a shear wall is calculated by multiplying the unit capacity by 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 given in Table 2. 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 was −38.2% in Wall 06, in which the C-shape sheathings were wrapped around the opening.

Table 3 shows the maximum strap forces of four corners around the opening from the test data, WALL2D model, and four simplified design methods under the design loads. The prediction errors in percentage are also given. It can be seen that the WALL2D prediction errors ranged from −15.4 to +4.3%. Drag strut method consistently underestimated strap forces, except for Wall 06 with the C-shape sheathing panels. Cantilevered beam, coupled beam, and Diekmann's method, however, seemed very conservative. The Diekmann's method, the most sophisticated calculation method among the four design methods, seemed to provide reasonable predictions for the walls with window-type openings. From the test data, the model simulations, and using Diekmann's method as a

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Table 2. WALL2D Predicted Strap Forces at Design Load Level Versus Test Results

	Wall	Effective wall length	ASD design load (kN)	Strap force around opening (kN): Maximum of top corners (C1 and C2)	Strap force around opening (kN): Maximum of bottom corners (C3 and C4)
	Test average	1.37 m (4.5 ft)	17.4 kN (3,915 lb_f)	3.48 kN (783 lb_f)	6.99 kN (1,571 lb_f)
04	Model			4.68 kN (1,053 lb_f)	6.23 kN (1,401 lb_f)
	Predicted error			34.5%	−10.9%
	Test average	1.37 m (4.5 ft)	17.4 kN (3,915 lb_f)	7.82 kN (1,758 lb_f)	9.15 kN (2,058 lb_f)
05	Model			9.07 kN (2,038 lb_f)	8.66 kN (1,946 lb_f)
	Predicted error			16.0%	−5.3%
	Test average	1.37 m (4.5 ft)	17.4 kN (3,915 lb_f)	2.29 kN (515 lb_f)	2.43 kN (546 lb_f)
06	Model			2.05 kN (462 lb_f)	1.50 kN (337 lb_f)
	Predicted error			−10.3%	−38.2%
	Test average	2.44 m (8 ft)	31.0 kN (6,960 lb_f)	5.51 kN (1,239 lb_f)	5.40 kN (1,213 lb_f)
08	Model			5.75 kN (1,292 lb_f)	4.66 kN (1,047 lb_f)
	Predicted error			4.3%	−13.7%
	Test average	2.44 m (8 ft)	31.0 kN (6,960 lb_f)	7.44 kN (1,673 lb_f)	7.22 kN (1,623 lb_f)
09	Model			7.24 kN (1,627 lb_f)	5.46 kN (1,228 lb_f)
	Predicted error			−2.7%	−24.3%
	Test average	1.22 mm (4 ft)	15.5 kN (3,480 lb_f)	7.97 kN (1,791 lb_f)	Not available
10	Model			7.95 kN (1,787 lb_f)	Not available
	Predicted error			−0.2%	Not available
	Test average	1.22 m (4 ft)	15.5 kN (3,480 lb_f)	12.29 kN (2,764 lb_f)	Not available
11	Model			12.01 kN (2,700 lb_f)	Not available
	Predicted error			−2.3%	Not available
	Test average	1.83 mm (6 ft)	23.2 kN (5,220 lb_f)	4.20 kN (945 lb_f)	4.81 kN (1,082 lb_f)
12	Model			3.67 kN (824 lb_f)	4.30 kN (966 lb_f)
	Predicted error			−12.8%	−10.7%

Table 3. Maximum Strap Forces [kN (lb_f)] Predicted by WALL2D and Simplified Design Methods

Wall no.	Test average	WALL2D	Drag strut	Cantilever	Couple beam	Diekmann's
04	6.99 kN (1,571 lb_f)	6.23 kN (1,401 lb_f) −10.9%	5.44 kN (1,223 lb_f) −22.2%	19.90 kN (4,474 lb_f) 184.7%	12.44 kN (2,796 lb_f) 78.0%	8.71 kN (1,958 lb_f) 24.6%
05	9.15 kN (2,058 lb_f)	9.07 kN (2,038 lb_f) −0.9%	5.44 kN (1,223 lb_f) −40.5%	27.36 kN (6,152 lb_f) 199.0%	17.10 kN (3,845 lb_f) 86.9%	14.51 kN (3,263 lb_f) 58.6%
06	2.43 kN (546 lb_f)	2.05 kN (462 lb_f) −15.4%	5.44 kN (1,223 lb_f) 124.1%	19.90 kN (4,474 lb_f) 719.5%	12.44 kN (2,796 lb_f) 412.2%	14.51 kN (3,263 lb_f) 497.5%
08	5.51 kN (1,239 lb_f)	5.75 kN (1,292 lb_f) 4.3%	5.16 kN (1,160 lb_f) −6.4%	35.38 kN (7,954 lb_f) 542.0%	11.79 kN (2,651 lb_f) 114.0%	8.26 kN (1,856 lb_f) 49.8%
09	7.44 kN (1,673 lb_f)	7.24 kN (1,627 lb_f) −2.7%	5.16 kN (1,160 lb_f) −30.7%	48.65 kN (10,937 lb_f) 553.7%	16.22 kN (3,646 lb_f) 117.9%	13.76 kN (3,093 lb_f) 84.9%
10	7.97 kN (1,791 lb_f)	7.95 kN (1,787 lb_f) −0.2%	5.16 kN (1,160 lb_f) −35.2%	Not available	Not available	41.28 kN (9,280 lb_f) 418.1%
11	12.29 kN (2,764 lb_f)	12.01 kN (2,700 lb_f) −2.3%	5.16 kN (1,160 lb_f) −58.0%	Not available	Not available	41.28 kN (9,280 lb_f) 235.7%
12	4.81 kN (1,082 lb_f)	4.30 kN (966 lb_f) −10.7%	Not available	Not available	Not available	6.64 kN (1,492 lb_f) 38.0%

base, a reduction correction factor of 1.2−1.3 might be considered to properly predict FTAO and account for the contributions of framing members and nailed connections within the system. Diekmann's method, however, is not suitable to predict FTAO in cases of door-type openings, e.g., a carport opening where the wall segment below the opening is not available. It should be noted that the strap forces in Wall 06 with C-shape sheathing could not be reasonably predicted by any of four simplified methods even with the correction factor. It seemed that the C-shape sheathing technique introduced much lower strap forces around the opening compared with the results of Wall 04, which had the same wall geometry except for the C-shape sheathing. This interesting phenomenon needs further studies in future.

Summary and Discussion

This paper presented a study on force transfer around openings in wood-frame shear walls using a finite-element model called WALL2D. From a laboratory test program of 12 shear wall configurations, the subset of eight wall configurations detailed for FTAO with different opening types and sizes was modeled and

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analyzed. The model predictions of wall load-drift hystereses 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 test results to check the model validity. It was also found that the model predictions agreed much better with the test results compared with the four 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 overpredict 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 postpeak softening range. Furthermore, because framing members also play an important role in transferring loads among wall components in shear walls with openings, the model simulations would be more accurate if the properties of framing members, such as modulus of elasticity, could be evaluated a priori by nondestructive testing. Nevertheless, this model provides a useful tool to the study the FTAO problem in wood-frame walls with openings. 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 wood-frame walls detailed for FTAO.

Acknowledgments

This work is part of a joint research project of APA – The Engineered Wood Association, USDA Forest Products Laboratory and the University of British Columbia. The research fund provided under the joint venture Agreement 09-11111138-117 between APA – The Engineered Wood Association and Forest Products Laboratory is greatly acknowledged.

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