International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:13, No:7, 2019

Impact of Out-of-Plane Stiffness of the Diaphragm on Deflection of Wood Light-Frame Shear Walls

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***Abstract*—**The in-plane rigidity of light frame diaphragms has been investigated by researchers due to the importance of this subsystem regarding lateral force distribution between the lateral force resisting system (LFRS). Where research has lacked is in evaluating the impact of out-of-plane raigidity of the diaphragm on the deflection of shear walls. This study aims at investigating the effect of the diaphragm on the behavior of wood light-frame shear walls, in particular its out-of-plane rigidity was simulated by modeling the floors as beam. The out of plane stiffness of the diaphragm was investigated for idealized (infinitely stiff or flexible) as well as "realistic". The results showed reductions in the shear wall deflection in the magnitude of approximately 80% considering the out of plane rigidity of the diaphragm. It was also concluded that considering conservative estimates of out-of-plane stiffness might lead to a very significant reduction in deflection and that assuming the floor diaphragm to be infinitely rigid out of plan seems to be reasonable. For diaphragms supported on multiple panels, further reduction in the deflection was observed. More work, particularly at the experimental level, is needed to verify the finding obtained in the numerical investigation related to the effect of out of plane diaphragm stiffness.

***Keywords*—**Deflection of light-frame wood shear walls, out-of-plane stiffness of the diaphragm, initial stiffness.

I. Introduction

ONE of the most important seismic design concepts is to determine the building period, which in turn is based on the structure's mass and stiffness. The mass can be estimated with relative ease but the stiffness, especially one that is representative of the "actual" behavior of the system, is much more difficult to determine. Thus, a number of mechanics-based models have been developed to predict the capacity and deflection of the light frame wood shear walls (e.g. [1]-[3]). The wood design standard [4] provides a 4-term deflection equation, which includes the contribution of stud bending, panel shear, nail slip and anchorage system elongation. In this equation, the shear wall is considered as a continuous cantilever beam; however the equation was originally limited in scope to a single-story wall. The 2009 edition of the CSA-O86 standard [4] incorporated cumulative effects due to rotation of the bottom level (bending and hold-down elongation) which is expected in multi-storey walls and an approach based on the cantilever model was adopted [4]. However, no attention has been given to the impact of out-of-plane stiffness of the diaphragm on the lateral performance of light-frame shear walls. Canadian wood design standard [4] emphasizes that there are other factors rather than cumulative effects such as out-of-plane diaphragm stiffness, that could potentially reduce building deflections but its effect is not currently well understood and therefore not addressed in the standard. A key study focusing on the deflection of multi-storey walls is that by Pei et al. [5] who developed a coupled shear-bending model to predict the dynamic response of multi-storey shear walls. The main focus of the study was on taking into account the bending deformation associated with rotation of the diaphragm due to rod elongation. The authors also validated their model by testing a 3-storey wood shear wall assembly with steel rods as continues hold-down devices on the uniaxial shake table. The setup configuration is shown in Fig 1. The diaphragms were allowed to rotate freely during the test to allow uplift accumulation in the system. Although the experimental setup physically captures the presence of the floor joists, allowing the floor diaphragm to undergo rigid body rotation does not represent the out-of-plane stiffness of the floor adequately.

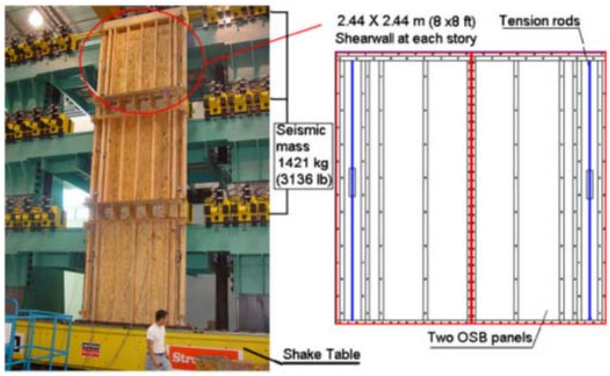

Fig. 1 3-storey shear wall test configuration [5]

II. Component Test

A. Stud Bending Test

The aim of conducting component test was to determine the structural properties of the material used in finite element modelling. In order to find the modulus of elasticity of the studs in the weak direction (consistent with their behaviour in practice) 38x89 mm (2"x4") stud lumbers were tested. The test method followed the ASTM D198-67 Standard Test

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Methods of Static Test of Lumber in Structural Size [6], where four-point loading on simply supported stud elements was chosen in this study, as illustrated in Fig 2. A total of five repeats were undertaken for the stud bending test. Fig. 3 shows the setup details.

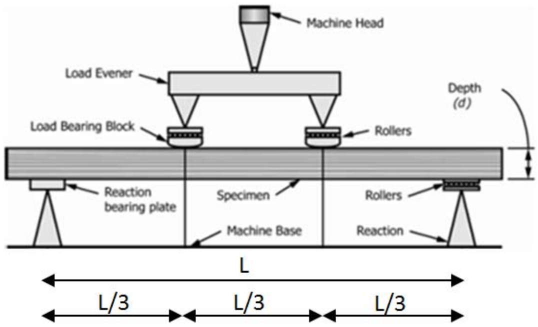


Fig. 2 4-Point loading test method of structural size of lumber [6]

The results from the stud bending tests provided estimates on the modulus of elasticity for the studs in the weak direction. ASTM D198 [6] describes the procedure to obtain the modulus of elasticity of the studs based on the chosen loading method. For the 4-point loading method used in this study, (1) estimates the modulus of elasticity of the studs as follows [6]:

$$E=\frac{23PL^3}{108bd^3\Delta} \tag{1}$$

where, E is the modulus of elasticity of the stud, P is applied load on the specimen, L is the span, b is the specimen width, d is the depth of the specimen, and Δ is the deflection at the neutral axis measured at mid-span.

Fig 4 shows a typical flexural performance of the stud and Table I summarizes the modulus of elasticity (E) values obtained using (1).

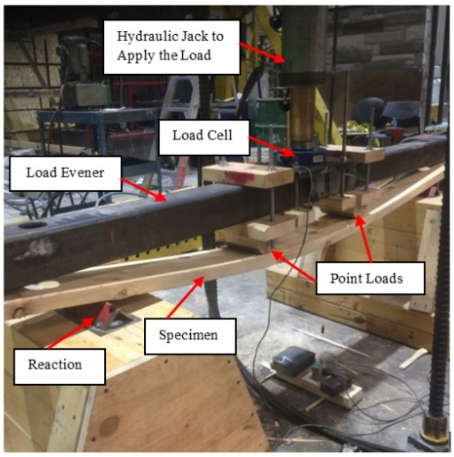


Fig. 3 Stud bending test setup

TABLE II
STUD BENDING TESTS RESULTS

Stud Size	No	$P_{0.4Peak}$ (N)	$\Delta_{0.4Peak}$ (mm)	E (N/mm^2)
	1	1780.00	24.65	12898
	2	1707.50	27.90	10931
2x4”	3	1666.40	27.20	10943
	4	1557.82	32.14	8658
	5	1560	25.80	10800

The average modulus of elasticity for the studs was 10846.18 N/mm^2 (COV= 13.84%). The modulus of elasticity obtained from the wood design standard [4] for the same species and grade (S-P-F No.2) is 9500 N/mm^2.

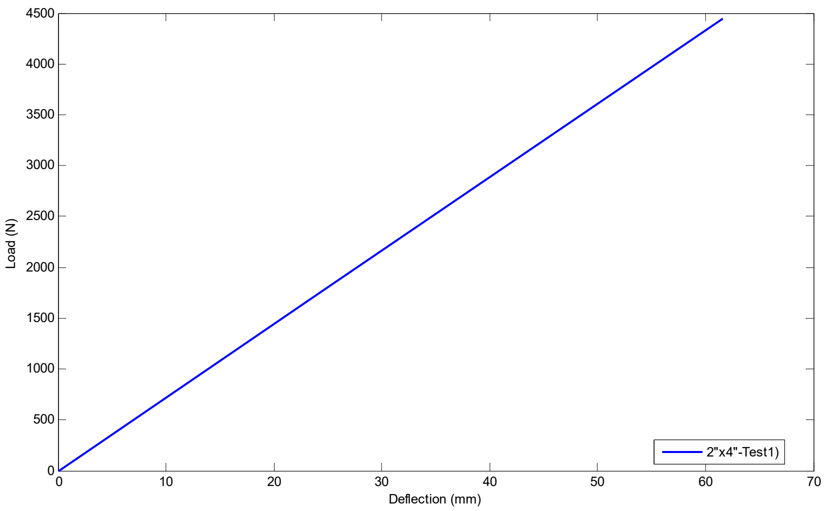
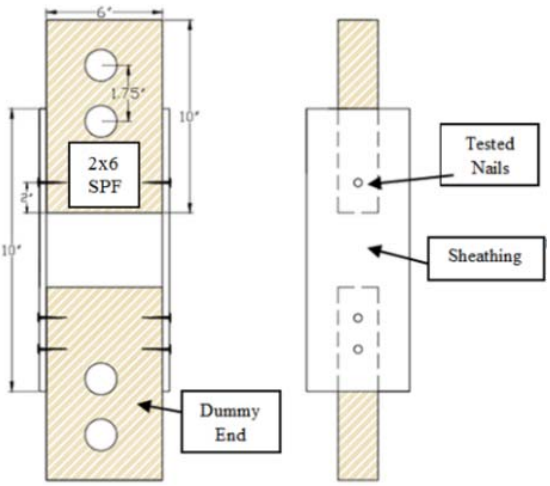


Fig. 4 2x4 Stud typical bending performance

B. Nail Joint test

The test specimens consisted of two 38 x 140 mm (2”x 6”) wood pieces with length of 254 mm (10”) and with 101.6 x 254 mm (4”x10”) OSB pieces fastened on both sides to the framing member using 2.84 mm nails (Fig. 5). Two 19.1mm (3/4") holes were pre-drilled in each framing piece. One end was considered as test joint while the other was the dummy end. The specimen had a nail placed on each side of the lumber section at the tested end. To minimize slip at the dummy end, four 12d nails with the length of 82.55 mm (3 1/4") were used. In overall, five repeats were considered. During the test, the relative displacement between the sheathing panel and lumber piece at the test end was measured which means that any possible slippage at the dummy end would not affect the test results.



(a) Nail joint test specimen sketch

(b) Typical joint test specimen

Fig. 5 Nail joint specimen fabrication

The universal testing machine (UTM) was used for the component testing using displacement controlled loading with a rate of 2.5 mm/minute. Two linear variable differential transducers (LVDT) were used on both sides of the specimen to measure the relative slip between lumber and sheathing. Fig 6 shows the nail joint test setup.

Fig. 7 shows the load-deflection plot for the tested nails to fasten the sheathing to the lumber. The average of the curves was used in modeling.

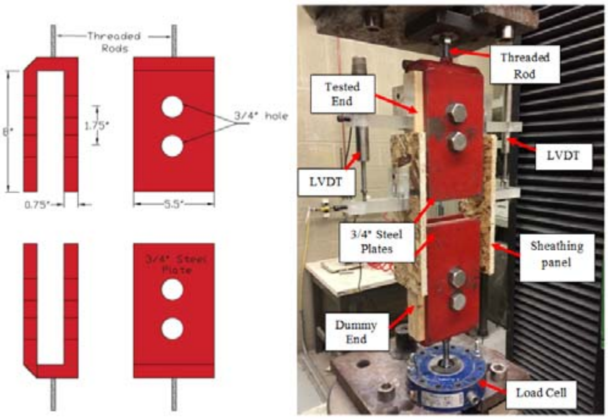


Fig. 6 Nail joint test setup

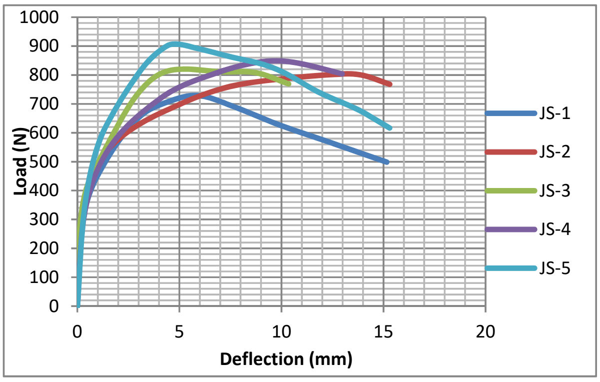


Fig. 7 Nail joint test results

III. Finite Element Modelling

A. Numerical Modelling Background

Numerical modeling is usually considered complementary to experimental testing and essential to proper interpretation of experimental findings. Once validated, models of components, substructures, and entire structural systems are used to expand the knowledge beyond what may be possible experimentally. Several attempts have been made in the past decades to model the behavior of light frame wood shear walls [7]-[13]. Existing models consist of various levels of detailing, while attempting to represent the physical behaviour of the shear wall system. The following section describes the detailed modeling procedure employed in the current study

B. Modelling Procedure

The shear wall model was developed using the commercially available software SAP 2000 [14]. All wall components including studs, sheathing and fasteners were modeled by using the tools available in the software. Framing elements such as studs and bottom and top plates, linear "frame" elements were used, while "membrane" elements were used to model the sheathing panels. Releases were provided at the end of all framing members to simulate pin-ended conditions. Elastic orthotropic material properties were assigned to all wood elements, with properties such as modulus of elasticity, shear modulus and Poisson's ratio defined in the three orthogonal directions. Poisson's ratio of 0.3 was assigned to framing and sheathing members while 10800 N/mm^2 and 11000 N/mm were assigned as modulus of elasticity of studs and sheathing-through-thickness rigidity of sheathing panels respectively. It should be noted that the modulus of elasticity of the studs and the nail slip curves were obtained from component tests conducted in this study. Other properties were obtained from published literature such as the engineering wood design standard [4] and the wood handbook [15].

Fig 8 shows a typical shear wall model, where the framing, sheathing panels, panel to framing connections, and hold-down are highlighted.

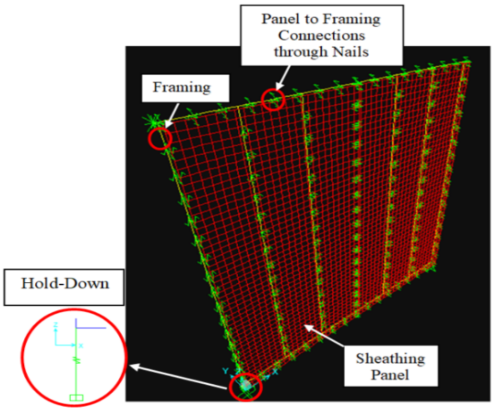


Fig. 8 Shear wall model in SAP 2000

The sheathing to framing nails were modeled using nonlinear springs (links) with properties in the horizontal and vertical directions. The nailed connections were represented through force-slip curves obtained from component tests (Fig 7). The non-linear behaviour of the nails, including the strength and stiffness degradation, was incorporated into the model using a multi-linear load-deformation function fitted to experimental results. Similarly, the hold-down devices were modeled using spring elements obtained from experimental results [16]. The effect of the diaphragm's out-of-plane rigidity on the behaviour of the shear walls is simulated by modeling the floors as beam. The stiffness of the beam is changed between zero and "infinity" and the effect on the walls is recorded. The beams are connected at each end to columns through a connection that allows full rotation (pin-ended). Two 38x140mm (2"x6") stud elements were used to simulate the boundary conditions of diaphragm joists bearing on an end wall. Fig. 9 shows the 6-storey shear wall model with simulated floor diaphragm.

The approach employed here would seem to be reasonable, as long as the beam stiffness is known. However, obtaining realistic values for the floor stiffness is not feasible due to the complexity of the system and variability of possible combinations. For this reason, idealized conditions (infinitely stiff or flexible) were considered. If the diaphragm is infinitely flexible out of plane, it can freely transfer the cumulative effects (particularly rigid body rotation) to upper floors. In contrast, infinitely rigid out of plane diaphragm represent the situation where cumulative effects cannot be transferred to the top floors.

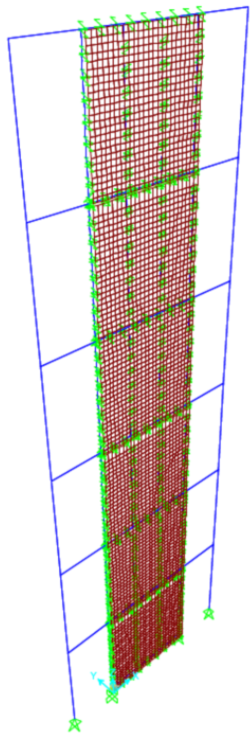


Fig. 9 3-D view of 6-storey wall with the beam at each storey level representing the diaphragm

As mentioned earlier, estimating the out-of-plane stiffness of the floor diaphragm with any level of accuracy is not possible. In an attempt to provide an estimate for this value, a situation is considered, where an exterior shear wall supports floors joists bearing perpendicular to the wall. The shear transfer from the upper storey is assumed to occur through and rim joist consisting of a multi-ply build-up LVL beam. The specifications for the LVL beam are obtained from the technical guide report [17], where SPF grade with an MOE of 15000 N/mm^2 is used. Typically, a two-ply beam is used at the end of the joisted floor; however, in this study, the effect of one, two, and three joists was investigated. It should be noted that in reality, the floor system is expected to have a significantly higher stiffness than that provided immediately above the shear walls, and as such the author believe that what is presented here could possibly be considered as a lower bound of what the floor contributes to the shear wall system.

Five cases were developed in the current study, as shown in Table III, by varying the out-of-plane stiffness of the floor diaphragm, representing the two idealized conditions (flexible and rigid), in addition to three "realistic" cases involving rim joists. In all cases, the end columns are assumed to consist of two 38x140mm (2"x6") stud lumbers.

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TABLE IV
DIFFERENT DIAPHRAGM CONDITIONS

Case No	Beam out-of-Plane Stiffness
1	0
2	Infinite
3	3 built-up 38 x 305mm LVL Joists
4	2 built-up 38 x 305mm LVL Joists
5	Single 38 x 305mm LVL

III. RESULTS AND DISCUSSION

Table V shows the normalized deflection of the 6-storey walls for the cases presented in Table VI.

The results clearly highlight the importance of the floor out-of -plane rigidity on the behaviour of the shear walls. Comparing the deflection results considering rigid diaphragm (case 2) to the flexible diaphragm (case 1) highlights that the rigid diaphragm reduces the deflection by 76.85% on average. Simply assuming three 38x305 mm built-up beam as diaphragm resulted in deflections that are very close to the case of rigid diaphragm. The table also shows that decreasing the size of the beam from three to two- or one ply 38x305 mm has no significant effect on the results. It can therefore be concluded that considering even conservative estimates of out-of-plane stiffness would lead to a very significant reduction in deflection and that assuming the floor diaphragm to be infinitely rigid out-of-plan seems reasonable. This is demonstrated graphically in Fig 10.

TABLE VII
NORMALIZED DEFLECTION OF EACH STOREY FOR DIFFERENT CASES

Storey	Case1	Case2	Case3	Case4	Case5
1	0.19	0.11	0.11	0.11	0.11
2	0.36	0.15	0.15	0.16	0.16
3	0.52	0.18	0.19	0.19	0.20
4	0.68	0.20	0.21	0.22	0.23
5	0.84	0.22	0.23	0.24	0.25
6	1.00	0.23	0.24	0.25	0.27

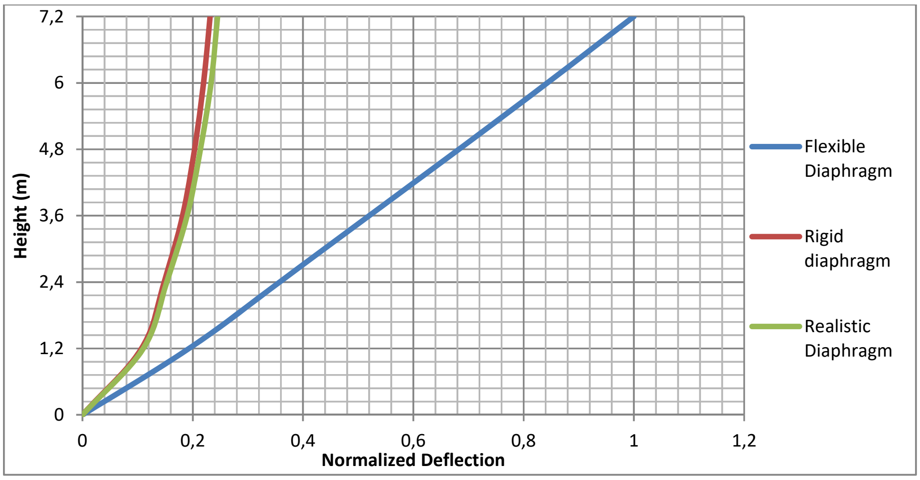

Fig. 10 Impact of diaphragm assumption on deflection of the walls

It is rare that a single shear wall panel is used in a shear line.

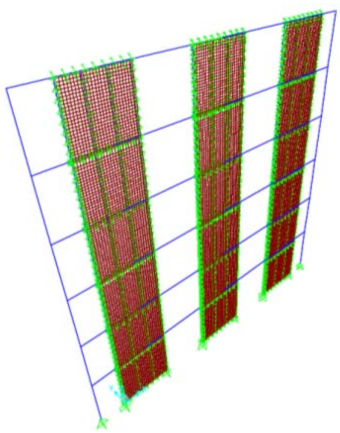

Fig. 11 3-D view of 6-storey multiple walls with the beam at each storey level representing the diaphragm

Typically a shear line would consist of multiple panels with similar configurations and construction details. The in-plane stiffness of the diaphragm usually ensures that the deflections in the panels is the same. Since it was observed in the single panel example earlier that the floor diaphragm has to bend in order to accommodate the rigid body rotation of the wall, having multiple panels would impose more demand on the floor diaphragm. This issue is investigated next for two and three shear walls in the same shear line. The wall configurations are similar to the case for single panel with 3-ply LVL beam. A unit load is applied on each wall panel to obtain comparable behaviour to that of a single wall case. Fig 11 illustrates an example of a shear line with three shear wall panels, and Table VIII presents the cases investigated in this study.

The results, normalized to the deflection obtained at the 6-storey level for a two-panel shear line, are presented in Table IX.

TABLE X
MULTIPLE SHEAR WALLS CASES IN A SHEAR LINE

Case	Number of Panels	Beam out of Plane Stiffness
A	2	3 built-up LVL joists
B	3	3 built-up LVL joists

TABLE XI
DEFLECTION OF WALLS WITH MULTIPLE PANELS

Storey	Case A	Case B Deflection	Ratio of
1	0.31	0.27	0.87
2	0.53	0.46	0.88
3	0.70	0.62	0.88
4	0.84	0.73	0.87
5	0.94	0.82	0.87
6	1.00	0.88	0.88

The results clearly show that having multiple walls would further reduce the deflection of the shear walls because the diaphragm has to accommodate the deformed shape of all wall segments in the shear line.

IV. CONCLUSION

The impact of out-of-plane stiffness of the diaphragm on deflection of multi-storey walls was investigated in this study.

The above findings are limited to numerical modeling and should as such be considered preliminary until verified by experimental testing. The finding can potentially have a very significant impact on the way a designer can approach deflection of light frame wood shear walls. The general approach has been that, assuming that light frame wood shear walls behave as cantilevers, and there is no out of plane stiffness in the floor diaphragm, then a mechanics-based approach describing the wall deflection, similar to that found in the wood design standard [4] can be used. The current study does not argue against the mathematical formulations past the adoption of the mechanical model but rather puts into question the validity of the fundamental assumptions themselves. It is proposed that assumption related to the cumulative effects is questionable and may lead to non-conservative designs. More work is required to provide better assumptions and estimates of multi-storey shear wall deflections. The results from the current study seem to point in the direction of ignoring the cumulative effects for the purpose of base shear calculations. These effects may be considered for drift calculations although it is believed that this would be too conservative. Estimates of out-of-plane floor diaphragm stiffness would be useful to investigate experimentally as well, although simple modeling approach from the current study seems to indicate that almost any diaphragm would have sufficient out-of-plane stiffness to be considered fully rigid.

ACKNOWLEDGMENTS

The authors would like to thank the University of Ottawa civil engineering laboratory staff, New Brunswick Innovation Foundation under its Innovation Research Chair Program, and the University of New Brunswick's Academic Development Fund.

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