Utilizing diaphragm action for wind load design of timber frame and structural insulated panel buildings

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Abstract

Current design methodologies for timber frame structures do not formally incorporate the structural benefits of structural insulated panels (SIPs) as diaphragm elements, which contribute significantly to the ability of these buildings to resist lateral loads. Both timber frame and post-frame buildings that have not necessarily been formally designed have long histories of good performance when subjected to wind and seismic forces. Strength and stiffness tests were conducted on three 2.44-m- (8-ft.-) deep and 7.32-m- (24-ft.-) wide roof diaphragm assemblies, and two 6.10-m- (20-ft.-) deep and 7.32-m- (24-ft.-) wide roof diaphragm assemblies. Data from these tests were collected, tabulated, and analyzed according to existing methods typically utilized for post-frame diaphragm testing, which exemplified the effectiveness of SIPs as diaphragm elements and reduced the forces in timber frame members to within code-conforming levels. A sample timber frame diaphragm design was performed to provide building designers with methods for including diaphragm action when designing timber frame buildings to resist lateral loads and to develop an engineering rationale for the excellent performance of laterally loaded timber frame buildings.

For centuries, buildings all over the world have been constructed utilizing large timbers joined together using various types of wood joints and secured with wood pegs and wedges to create structural skeletons that are enclosed to provide shelter and interior spaces as shown in **Figure 1**. Contemporary timber frame structures, especially in the United States, typically utilize structural insulated panels (SIPs) attached to the timber frame skeleton to enclose the frame and create a functional structure. The SIP panels consist of a layer of rigid insulation that is typically covered on one side by oriented strandboard and on the other side by oriented strandboard, gypsum drywall, or some other interior finish, such as tongue and groove paneling. Current design methodologies for timber frame structures do not include, in a formalized way, the potential structural benefits of SIPs considered as diaphragm elements.

Lateral forces resulting from wind and earthquakes can induce considerable shear and bending stresses into building components. While timber frame structures are typically well within safety limits with regard to gravity loads, lateral loads can potentially exceed the stress limits of timbers and joints. One area prone to overstress in timber frame structures is the tenon portion of a beam because it is much smaller than the rest of the member and is further limited in strength by holes for the wood dowels and short edge distance from the dowel holes to the end of the tenon. When timber frame structures are analyzed when subjected to typical wind speeds or seismic ground accelerations, it is very likely that the tenons on members where tension forces are induced will be overstressed if the contribution

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Figure 1. — Timber framing shown for a "great room" near Blacksburg, VA. Nominal timber sizes typically range from 140 mm (6 in.) by 140 mm (6 in.) to 292 mm (12 in.) by 292 mm (12 in.) and common species include southern pine, Douglas-fir, and mixed oak.

of the building diaphragms and shear walls are not included in the structural analysis. Timber frame structures have an excellent performance record, and it is therefore assumed that even though the frame alone does not have the strength to resist lateral loads, the contribution of SIPs, or other cladding, contributes to the ability of these structures to effectively resist lateral loading.

For over 20 years, post-frame structures have been the subject of considerable research aimed at quantifying "the tremendous contribution of the ceiling and/or roof diaphragm and endwalls to the stiffness and strength of the building assembly" (Gebremedhin et al. 1992). Results have been used to develop a methodology that allows engineers and designers to quickly and easily calculate the forces present within post-frame structural elements and account for the diaphragm action of the roof and endwalls. These advances in diaphragm design have resulted in more cost-effective design and a better understanding of the structural responses of post-frame buildings, which have grown in use from simple agricultural buildings to commercial, residential, institutional, and industrial structures (Walker et al. 1992).

An initial investigation (Carradine et al. 2000) indicated that timber frame building design could benefit from incorporating SIPs as part of the lateral force resisting system of the building. Testing conducted on several timber frame and SIP roof diaphragm assemblies, and subsequent data analyses, provided critical information on the strength and stiffness of timber frame and SIP roof assemblies and served to quantify the contributions made by SIPs and the timber frames in resisting lateral loads applied to them. This knowledge was incorporated with existing procedures for diaphragm design of post-frame structures in order to develop a similar methodology for the design of timber frame buildings that included the SIPs as diaphragm elements for lateral load resistance and began to develop an engineering rationale for the excellent performance history of timber frame buildings subjected to lateral forces.

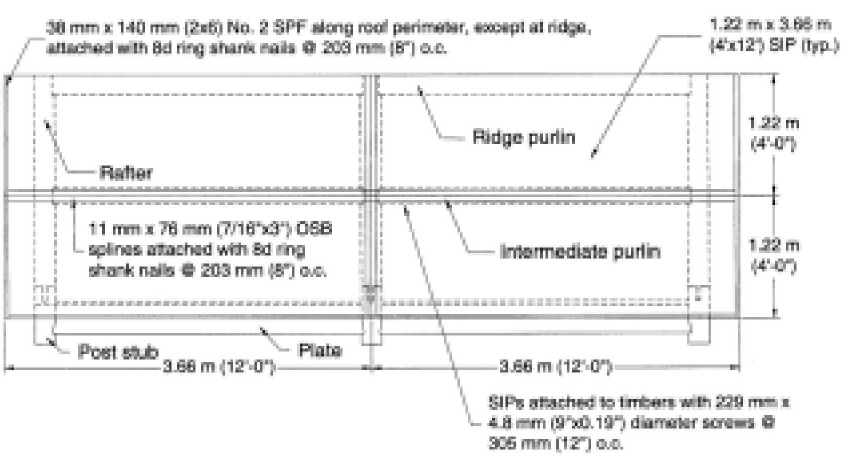

Figure 2. — Plan view of 2.44- by 7.32-m (8- by 24-ft.) basic test panel assembly.

While research included diaphragm testing protocols for both seismic and wind design, the scope of this paper is limited to analysis procedures and design methods for resisting wind loads only. Testing and analysis for resisting seismic loads will be presented in another paper.

Objective

The objective of this paper was to present methods for designing timber frame and SIP buildings to resist wind loads, which include the diaphragm contribution of the SIPs in order to bring frame member forces to within acceptable design limits, and provide timber frame building designers with means of designing code-conforming, efficient, and economic structures.

Timber frame and SIP roof assembly testing

Based on recommendations made by Carradine et al. (2000), the most important elements lacking in order to apply a diaphragm-frame-interaction design to timber frame buildings were test data on the strength and stiffness of timber frame and SIP roof assemblies. Typical timber frame construction lent itself most easily to the use of a simple beam test utilizing a single load applied to the center rafter of an assembly having three rafters and two bays. Roof assembly strength and stiffness were determined by monotonically applying a load to the center rafter, in a direction parallel to the rafters, until failure occurred. Tests were conducted on three basic test panel assemblies (2.44-m [8-ft.] by 7.32-m [24-ft.]) and two full-scale sized roof panel assemblies (6.10-m [20-ft.] by 7.32-m [24-ft.]) in order to determine stiffness and strength values, as well as how predictions should be made from test panel assemblies to full roof diaphragms. A plan view of the basic test panel assembly is shown in **Figure 2**.

Full-scale roof panel assemblies utilized 10 SIPs and 8 intermediate purlins, but were otherwise the same as the basic test panel assemblies. Construction of all tested assemblies was based on typical timber frame joinery and SIP installation methods utilizing hardened steel screws, 229 mm (9 in.) long with a shank diameter of 4.8 mm (0.19 in.) and a root diameter of 4.3 mm (0.17 in.) to attach 165- mm- (6.5-in.-) thick SIPs to southern pine timbers, as illustrated in **Figure 3**. In an effort to avoid a soft

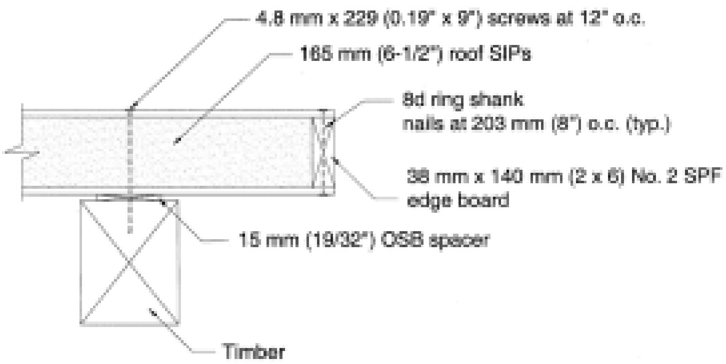

Figure 3. — Example of SIP to timber connection. OSB spacer was attached using 380-mm- (1-1/2-in.-) long deck screws (3.5 mm [0.138 in.] shank diameter) at 305 mm (12 in.) on-center.

Table 1. — Summary of monotonic failure test data for timber frame and SIP roof diaphragm assemblies. Assemblies 1, 2, and 3 were 2.44 by 7.32 m (8 by 24 ft.) and Assemblies 4 and 5 were 6.10 by 7.32 m (20 by 24 ft.).

Assembly	Maximum applied load (P_{ult})	Ultimate shear strength (V_{ult})	Allowable design shear strength (V_a)	Effective shear stiffness (c)
1[a]	10,600 lb.	663 lb./ft.	265 lb./ft.	22,100 lb./in.
	(47.2 kN)	(9.68 kN/m)	(3.87 kN/m)	(3.86 kN/mm)
2	11,600 lb.	725 lb./ft.	290 lb./ft.	44,600 lb./in.
	(51.6 kN)	(10.6 kN/m)	(4.24 kN/m)	(7.92 kN/mm)
3	12,600 lb.	788 lb./ft.	315 lb./ft.	41,700 lb./in.
	(56.1 kN)	(11.5 kN/m)	(4.60 kN/m)	(7.23 kN/mm)
4	38,300 lb.	958 lb./ft.	383 lb./ft.	107,000 lb./in.
	(170 kN)	(13.9 kN/m)	(5.56 kN/m)	(18.9 kN/mm)
5	37,600 lb.	940 lb./ft.	376 lb./ft.	133,000 lb./in.
	(167 kN)	(13.7 kN/m)	(5.48 kN/m)	(23.0 kN/mm)

[a]Weathered assembly; not used for determination of shear strength and stiffness for 2.44- by 7.32-m (8- by 24-ft.) assemblies.

layer between the SIPs and timbers, all SIPs tested utilized 11-mm (7/16- in.) OSB for inner and outer skins rather than using a SIP with gypsum or drywall as the inner skin. While alternate methods for connecting SIPs to each other and for connecting SIPs to timbers do exist, utilizing 76-mm (3-in.) OSB splines between SIPs and 229-mm (9-in.) by 4.8-mm (0.19-in.) screws to attach SIPs to timbers is common practice throughout the world for roof systems. Specific details on fabrication of test specimens were provided by Carradine (2002).

Results and analysis of roof assembly tests

Monotonic testing of five timber frame and SIP roof assemblies provided diaphragm strength and stiffness data. Failure of all assemblies was a result of breaking of the 229-mm (9-in.) by 4.8-mm (0.19-in.) screws attaching SIPs to the timber frames. Specifically, once enough screws were broken, the load required to deflect the assemblies declined until the test was terminated. Properties of SIP screws were critical in determining strength and stiffness of assemblies and should be taken into consideration for design purposes. Screws utilized were manufactured from hardened steel, which resulted in a non-ductile connection between the SIPs and timbers, observed as shearing rather than yielding of screws during failure tests.

Strength and stiffness values were calculated from load and displacement data obtained during testing. A summary of strength and stiffness values for each assembly is provided in **Table 1**. Ultimate shear strength (V_{ult}) was determined utilizing the following equation:

$$V_{ult} = P_{ult}/(2b) \qquad [1]$$

where:

V_{ult} = assembly shear strength at maximum load, N/m (lb./ft.)

P_{ult} = maximum load at assembly failure, N (lb.)

b = diaphragm length measured from ridge to eave, m (ft.)

Allowable design shear strength values were calculated by dividing ultimate shear strength values by a factor of safety of 2.5. Allowable shear stiffness was calculated as the allowable design shear strength divided by the corresponding corrected displacement of the center rafter. Corrected displacements were obtained by subtracting the average displacement of end supports from the displacement of the center rafter and eliminated rigid body movement due to sinking of reaction supports.

In accordance with ASAE EP558 (ANSI/ASAE 1999b), determination of allowable design shear strength was calculated as the lowest calculated shear strength value and allowable design shear stiffness was calculated as the average of calculated stiffness values from conducted tests. Results from testing provided allowable design shear strength of 4.24 kN/m (290 lb./ft.) for 2.44- by 7.32-m (8- by 24-ft.) roof assemblies and 5.48 kN/m (376 lb./ft.) for 6.10- by 7.32-m (20- by 24-ft.) roof assemblies. Effective shear stiffness was calculated as 7.58 kN/mm (43,200 lb./in.) for 2.44- by 7.32-m (8- by 24-ft.) roof assemblies and 21.0 kN/mm (120,000 lb./in.) for 6.10- by 7.32-m (20- by 24-ft.) roof assemblies. Therefore, for timber frame and SIP roof systems, allowable shear strength per unit length and effective shear stiffness were not constant with increasing diaphragm slope length, but increased as diaphragm slope length increased.

It is worth noting that increases in diaphragm slope length resulted in decreases in the aspect ratios of tested roof diaphragm assemblies. Decreasing the aspect ratio likely contributed to the increased strength and stiffness of the assemblies.

Differences between typical post-frame diaphragm action and SIP diaphragm action contributed to nonlinear increases in shear strength and stiffness observed in SIP diaphragm tests. The

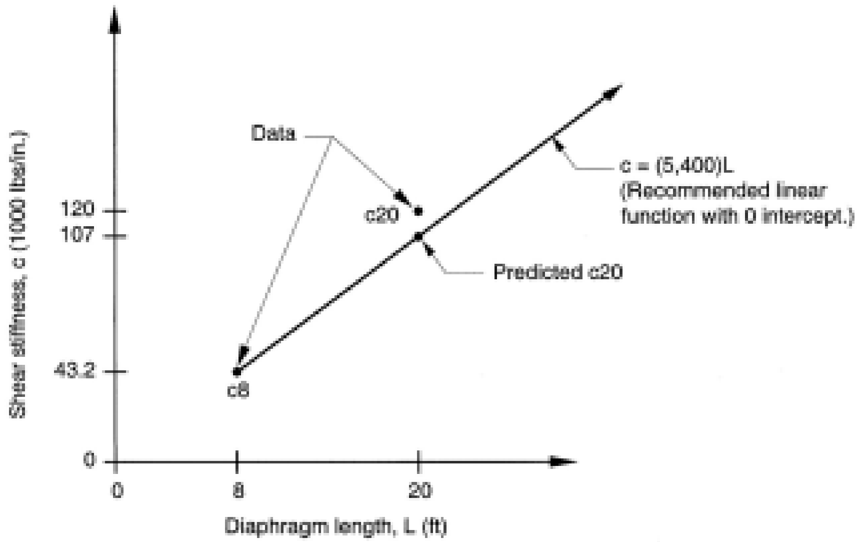

Figure 4. — Plot of effective shear stiffness (c) as a function of diaphragm length (L). Actual shear stiffness at slope length, L = 6.10 m (20 ft.), obtained from testing is greater than the predicted shear stiffness at L = 6.10 m (20 ft.) utilizing the linear function based on shear stiffness of tested 2.44- by 7.32-m (8- by 24-ft.) diaphragms.

slip of all fasteners and the relative slip between metal sheathing panels in a typical post-frame diaphragm result in a linear relationship between assembly stiffness and diaphragm (roof slope) length and a constant shear strength on a per foot basis. The rigid, monolithic plate action associated with the SIPs as assembled and tested in this project likely stemmed from the two component slow-rise polyurethane insulation foam (Handi-Foam SR, manufactured by Fomo Products, Inc.) that was installed between the panels. Because the insulating foam has shear strength, like the expanded polystyrene foam in the SIPs, and all panels were joined at the seams, all fasteners more effectively shared the shear forces, causing the relationship between shear strength and stiffness and diaphragm length to be a function of overall fastener number rather than simply the number of fasteners in a line at the edge of the diaphragm.

Sound engineering practice dictates that design methods provide conservative estimates of forces within structural members for safe construction. Although allowable shear strength increased with panel length in our tests, a reasonable and conservative design approach would be to use the allowable shear strength obtained from testing 2.44-m- (8-ft.-) long roof diaphragm assemblies for roof diaphragm constructions of the type we tested having slope lengths greater than 2.44 m (8 ft.).

As with allowable shear strength, effective shear stiffness also increased with diaphragm slope length. ASAE EP484.2 (ANSI/ASAE 1999a) diaphragm design methodology utilizes effective shear stiffness within the roof and endwalls to reduce member forces in building frames. Greater effective shear stiffness in the roof results in lower member forces when designing for lateral load resistance. It would therefore be conservative to establish methods for extrapolating effective shear stiffness from 2.44-m- (8-ft.-) long test diaphragms that would provide lower shear stiffness than that which was observed during testing of 6.10-m- (20-ft.-) long test diaphragms.

Figure 4 illustrates possible means of determining shear stiffness for diaphragms with slope lengths greater than 2.44 m (8 ft.) and data points from tests of 2.44-m- (8-ft.-) and 6.10-m- (20-ft.-) long diaphragms. Since it is unlikely that effective shear stiffness would decrease with increases in panel length for the SIP roof system tested, a reasonable design assumption for determining building diaphragm effective shear stiffness would be that effective shear stiffness obtained from testing 2.44-m- (8-ft.-) long roof diaphragm assemblies should be assumed to increase linearly for all roof diaphragms greater than 2.44 m (8 ft.) in length. Based on effective shear stiffness of 7,580 N/mm (43,200 lb./in.) for 2.44-m- (8-ft.-) long roof assemblies, predicted effective shear stiffness for 6.10-m- (20-ft.-) long roof assemblies would be 19,600 N/mm (107,000 lb./in.) rather than an experimentally obtained 21,000 N/mm (120,000 lb./in.) shear stiffness value. This results in conservative frame member forces when used with ASAE EP484.2 diaphragm design procedures and is consistent with sound engineering practice.

It is recommended that for future assessment of timber frame and SIP roof constructions similar to those tested for this paper, designers assume that effective shear stiffness of building diaphragms do increase linearly with diaphragm length, with a zero intercept. Caution should be exercised when designing diaphragms with aspect ratios greater than 3:1. At this time it is uncertain as to whether the linear extrapolation of diaphragm stiffness would be as conservative for diaphragms more slender than those tested for this paper.

Tested roof assemblies were fabricated to include all potential slip conditions in an effort to simulate a worst case scenario with regard to slip. If future 2.44- by 6.10-m (8- by 24-ft.) timber frame and SIP roof assembly tests are not designed to include all connection slip possibilities as outlined above, it is not recommended that extrapolations to longer roof slope lengths utilize a linear increase in shear stiffness without testing larger test assemblies to verify this assumption. Roof systems that utilize methods of connecting SIPs to one another and of attaching SIPs to timbers different from those discussed herein should be subjected to similar testing regimes before conclusions are drawn about the applicability of design methods presented in this paper.

Example diaphragm design for a timber frame structure

Previously discussed test results and analyses provided ample quantification of timber frame and SIP roof system behavior to allow designers to include diaphragm action as a means of reducing forces in timber frame members, and

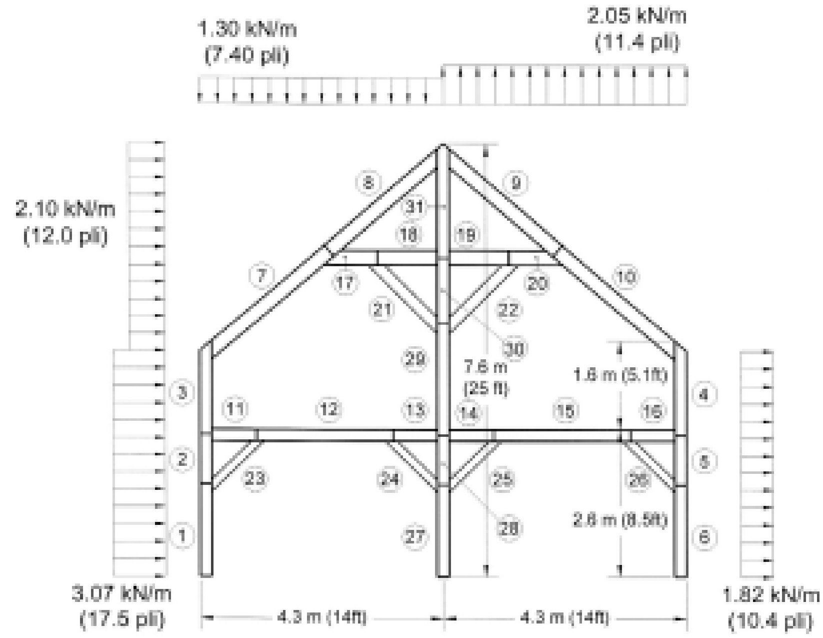

Figure 5. — The center frame of the Moran timber frame including wind loads when the 40.3 m/sec. (90 mph) wind is assumed to act perpendicular to ridge, based on the ICC 2000.

providing sound designs for timber frame and SIP structures subject to wind loads, as presented in the following examples.

The Moran timber frame by Benson Woodworking (Christian 1997) was considered a typical three frame, two bay residential timber frame, and was used to assess the applicability of post-frame diaphragm design methodology for use with timber frame and SIP building systems. Due to the complexity of typical timber frames, it was necessary to utilize a two-dimensional frame analysis program to calculate forces in frame members. In order to exemplify this type of analysis, the central frame of the Moran timber frame was entered into PPSA4 (Triche and Suddarth 1993). Dead loads on the frame were based on weight of timbers and panels, and live loads applied to the building were distributed wind pressures calculated utilizing the International Building Code 2000 (ICC 2003), using Exposure Category C and 40.3 m/sec. (90 mph) wind speeds. Applied wind loads and structural analog for use with PPSA4 are illustrated in **Figure 5**.

Inherent flexibility within the timber frame was modeled utilizing fictitious members with a specific modulus of elasticity (Triche and Suddarth 1993) that were added between knee braces and the posts they were connected to. Modulus of elasticity of these fictitious members was derived using information from research on timber frame joint behavior (Chappell 1995; Bulleit and Sandberg 1996; Schmidt et al. 1996, 1999). It should be noted that this step in the analysis required specific load-slip data on the assumed joint detail, and that in general, designers who utilize a diaphragm analysis will need to obtain data for their selected joint details. As shown in **Table 2**, without diaphragm contributions, members of the frame were overstressed by as much as 160 percent according to NDS-2001, Section 3.9, combined stress criteria (AF&PA 2001), which require the combined stress index, provided in **Table 2**, to be less than or equal to 1.0 for members to fulfill allowable stress design criteria for combinations of bending and axial loads. Values shown in **Table 2** were obtained from PPSA4, which utilizes NDS-91 interaction criteria, which are identical to the interaction criteria in NDS-01. Clearly, the timber frame was not capable of withstanding applied wind loads, therefore SIPs must be considered as a lateral load resisting element. Timber frame member sizes could be increased in order to meet code requirements for wind loading, but this was not regarded as an efficient solution considering timber frame and SIP roof assembly test results previously discussed.

ASAE EP484.2 diaphragm design procedures base the design of sidewall posts on selection of a roof diaphragm system. Timber frame procedures must include design of timber posts, beams (or bent girts), rafters, and braces for the frame under consideration. Test data for SIPs provided in-plane shear diaphragm stiffness (c_p) of 7.58 kN/mm (43,200 lb./in.) for 2.44- by 7.32-m (8- by 24-ft.) diaphragm test panels with 3.35-m (11-ft.) frame spacing. Total horizontal diaphragm shear stiffness (c_h), also known as building diaphragm shear stiffness, was calculated using the following equation:

$$c_h = 2c_p\,(\cos^2\theta)(b'/b)(a/s_f) \quad [\mathbf{2}]$$

where:

c_h = building diaphragm shear stiffness, kN/mm (lb./in.)

c_p = in-plane test panel shear stiffness, kN/mm (lb./in.)

θ = angle of roof slope, degrees

b' = *roof span length from ridge to eave, m (ft.)*

b = test panel length, m (ft.)

a = test assembly frame spacing, m (ft.)

s_f = frame spacing of building, m (ft.)

Utilizing dimensions and stiffness values from testing, a roof slope angle of 39.81 degrees, a roof span length of 5.55 m (18.22 ft.), and a building frame spacing of 4.27 m (14 ft.), the resulting building diaphragm shear stiffness (c_h) was 16.0 kN/mm (91,200 lb./in.). Use of Equation [**2**] to calculate building diaphragm shear stiffness from in-plane shear diaphragm stiffness obtained from roof assembly testing was governed by the linear extrapolation model discussed previously, which assumes that the allowable in-plane shear diaphragm stiffness increases linearly as the slope length increases beyond 2.44 m (8 ft.). Additionally, Equation [**2**] incorporates the slope of the roof and an assumed additive stiffness contribution from both sides of the roof for calculating building diaphragm shear stiffness.

Frame stiffness (k) was approximated by applying a concentrated load to the frame in PPSA4 and dividing this load by the deflection it produced at the point of application. Calculated stiffness for the Moran frame was 303 N/mm (1,730 lb./in.). Maximum potential lateral re-

Table 2. — Combined stress values for members of the example timber frame utilizing no diaphragm action (Model 1), assuming an effectively rigid endwall (Model 2), and utilizing preliminary test data from clad and unclad walls at the University of Wyoming.

	Combined stress values		
	Model 1	Model 2	Model 3
Member	(no diaphragm action included)	(with diaphragm action included)	(with diaphragm action and experimental wall stiffness included)
1	1.75	0.81	0.83
2	1.78	0.81	0.83
3	0.66	0.49	0.47
4	0.70	0.44	0.42
5	1.81	0.82	0.84
6	1.81	0.83	0.85
7	--[a]	0.25	0.25
8	0.22	0.19	0.19
9	0.10	0.33	0.33
10	--[a]	--[a]	--[a]
11	1.02	0.07	0.09
12	1.01	0.19	0.21
13	0.97	0.20	0.22
14	0.86	0.16	0.17
15	1.06	0.16	0.17
16	1.19	0.13	0.16
17	0.24	0.08	0.08
18	0.24	0.08	0.08
19	0.11	0.05	0.05
20	0.11	0.05	0.05
21	0.02	0.04	0.04
22	0.05	0.03	0.03
23	0.31	0.01	0.02
24	0.60	0.17	0.18
25	0.28	0.05	0.05
26	0.66	0.10	0.12
27	2.60	0.68	0.72
28	2.57	0.67	0.71
29	1.00	0.18	0.18
30	1.00	0.15	0.17
31	0.05	0.07	0.07

[a]If both tension and compression existed along the member length no interaction value was calculated by PPSA4.

straining force of the roof diaphragm (R), was calculated by placing a roller reaction at the leeward eave edge of the frame and applying wind and gravity loads to the frame in PPSA4. Resulting maximum lateral restraining force (R) was equal to 14.6 kN (3,289 lb.), which was the magnitude of the reaction at the installed roller.

The sidesway restraining force factor (mD) indicates the amount of lateral load required to be resisted by the center building frame and how much is being resisted by the SIP diaphragm. This factor was selected from Table 2 of ASAE Standard EP484.2 (ASAE 1999a) where the vertical axis is c_h/k and the horizontal axis is number of frames, including both end walls. The vertical axis also contains a column for k_e/k, which represents the ratio of endwall stiffness (k_e) to frame stiffness. It was assumed for this initial analysis that endwalls were rigid or effectively reinforced against lateral forces to be effectively rigid, resulting in a value for ke/k of 10,000; the highest value in the table. This was a theoretical assumption made to demonstrate the overall effectiveness of the diaphragm method. Actual building designs with endwalls containing windows and doors need to be investigated, as data are still needed on specific constructions in order to estimate endwall stiffness. Table 2 of ASAE Standard EP484.2 (ASAE 1999a) provides an mD value of 0.99, utilizing three frames in the building and a c_h/k value of 53. Multiplying roller reaction (R) by mD, the roof diaphragm provided a resisting lateral force equal to 14.5 kN (3,256 lb.). The percentage of lateral load carried by the roof diaphragm was 99 percent (or $mD \times 100\%$), thus the SIPs are resisting nearly all lateral loads for this case. Considering the relative stiffness of the SIPs, this conclusion is not surprising.

Frame member forces and stresses within the center bent of the Moran frame were determined by performing another analysis using PPSA4, where calculated diaphragm resisting capacity of 14.5 kN (3,256 lb.) was applied to the frame as horizontal distributed loads of 2.0 kN/m (140 lb./ft.) along each roof slope in the direction opposite of the wind. Reductions in member combined stress values as a result of including diaphragm action of SIPs are shown in **Table 2** as Model 2. By estimating the contribution of SIPs in resisting lateral loads due to wind, the analysis exemplified that none of the members or joints in the frame were overstressed, and all members were within NDS-01 (AF&PA 2001) combined stress design requirements. **Table 3** provides comparisons of axial force values for frame members as determined utilizing PPSA4 for conditions with and without inclusion of SIPs as diaphragm elements.

The final step of ASAE EP484.2 diaphragm design procedures requires calculating maximum roof shear and comparing this to allowable design shear strength determined through diaphragm assembly tests. Maximum roof shear was calculated by multiplying potential lateral restraining force (R) by the shear force modifier (mS) and dividing by building width, 8.53 m (28 ft.). Table 1 of ASAE Standard EP484.2 (ASAE 1999a) utilizes the same input parameters as the mD table, and provides an mS value for this design example of 1.00. Resulting maximum roof and endwall shear was 1.71 kN/m (118 lb./ft.), well below the experimentally determined allowable roof shear strength of 4.24 kN/m (290 lb./ft.). Designers utilizing ASAE EP484.2 diaphragm design procedures should check the shear strength and stiffness of the roof system and the shear strength and stiffness of endwalls to ensure adequate resistance to lateral loads.

Table 3. — Comparisons of member axial forces of the example timber frame utilizing no diaphragm action (Model 1), assuming an effectively rigid endwall (Model 2), and utilizing preliminary test data from clad and unclad walls at the University of Wyoming.

	Axial force		
	Model 1	Model 2	Model 3
Member	No diaphragm action included	With diaphragm action included	With diaphragm action and experimental wall stiffness included
	(kN[lb.])		
1	-2.34 [-527]	-11.9 [-2,670]	-11.7 [-2,620]
2	-17.0 [-3,830]	-12.5 [-2,800]	-12.5 [-2,820]
3	-5.78 [-1,300]	-8.90 [-2,000]	-8.85 [-1,990]
4	-2.46 [-553]	0.63 [141]	0.56 [126]
5	2.80 [629]	-1.85 [-416]	-1.75 [-393]
6	-14.0 [-3,140]	-4.49 [-1,010]	-4.67 [-1,050]
7	1.38 [310]	-0.93 [-208]	-0.88 [-197]
8	6.81 [1,530]	6.67 [1,500]	6.67 [1,500]
9	5.83 [1,310]	8.05 [1,810]	8.01 [1,800]
10	-1.24 [-278]	4.11 [923]	3.99 [898]
11	-18.5 [-4,150]	-3.34 [-750]	-3.66 [-823]
12	-3.78 [-850]	-2.77 [-623]	-2.79 [-628]
13	11.5 [2,850]	1.45 [327]	1.67 [376]
14	-12.5 [-2,820]	-2.49 [-559]	-2.70 [-607]
15	0.82 [185]	-0.17 [-38.1]	-0.15 [-33.3]
16	17.6 [3,950]	2.46 [554]	2.79 [627]
17	-11.0 [-2,470]	-8.10 [-1,820]	-8.18 [-1,840]
18	-3.88 [-827]	-5.29 [-1,330]	-5.87 [-1,320]
19	-9.07 [-2,040]	-6.72 [-1,510]	-6.76 [-1,520]
20	-5.47 [-1,230]	-8.14 [-1,830]	-8.10 [-1,820]
21	-10.2 [-2,290]	-3.14 [-705]	-3.29 [-739]
22	5.12 [1,150]	-2.05 [-460]	-1.89 [-425]
23	20.8 [4,670]	0.80 [180]	1.23 [277]
24	-21.6 [-4,850]	-5.96 [-1,340]	-6.32 [-1,420]
25	18.9 [4,240]	3.27 [736]	3.61 [812]
26	-23.7 [-5,330]	-3.72 [-837]	-4.15 [-934]
27	-23.9 [-5,830]	-23.8 [-5,360]	-23.9 [-5,370]
28	-22.0 [-4,950]	-22.0 [-4,940]	-22.0 [-4,940]
29	-13.6 [-3,050]	-13.5 [-3,040]	-13.5 [-3,040]
30	-9.96 [-2,240]	-9.83 [-2,210]	-9.83 [-2,210]
31	-11.2 [-2,520]	-11.0 [-2,480]	-11.0 [-2,480]

A final analysis was conducted utilizing preliminary data from testing conducted on timber frame walls with and without SIPs attached at the University of Wyoming (Erikson and Schmidt 2002a, 2002b). Test results provided values for endwall stiffness (k_e) and frame stiffness (k) of 6.58 kN/mm (37,600 lb./in.) and 226 N/mm (1,290 lb./in.), respectively. These data changed values for ke/k to 29.1, c_h/k to 71, mD to 0.954, and mS to 0.976. Diaphragm resisting capacity of 14.0 kN (3,140 lb.) was applied to the frame as horizontal distributed loads of 1.97 kN/m (135 lb./ft.) along each roof slope in the direction opposite of the wind. Member combined stress values as a result of including diaphragm action of SIPs utilizing data from testing at the University of Wyoming are shown in **Table 2** as Model 3. Alterations in frame and endwall stiffness had little effect on resulting combined stress values for frame members, all of which remained within allowable NDS-01 (AF&PA 2001) design limits. Calculated maximum roof and endwall shear was 1.68 kN/m (115 lb./ft.), well below the experimentally determined allowable roof shear strength of 4.24 kN/m (290 lb./ft.).

Analyses of a typical residential building indicated that without the diaphragm contribution of the SIPs, the timber frame could not resist lateral loads induced by 40.3 m/sec. (90 mph) wind speeds. ASAE EP484.2 procedures for diaphragm design allowed for the incorporation of diaphragm action of SIPs within the roof system of a timber frame building to effectively reduce member forces within timber frame components to acceptable design levels. Test procedures and data analyses described previously provided necessary data for timber frame and SIP building designers to use ASAE EP484.2 diaphragm design procedures and should be adhered to by designers who wish to assess the applicability of different timber frame and SIP roof system configurations for use with the described design methods. For dimensions, SIPs, and timbers specific to the building analyzed, the design example proved the effectiveness of diaphragm design for bringing timber frame and SIP structures to within codeconforming standards.

Summary and conclusions

The fundamental objective of research on strength and stiffness of timber frame and SIP roof systems was to establish procedures for incorporating the significant in-plane strength and stiffness of SIPs within lateral load design of timber frame and SIP buildings in an effort to create efficient and code-conforming designs for contemporary timber frame buildings. Initial investigations indicated that without including diaphragm action, timber frame buildings did not have the structural integrity to resist lateral loads for code-compliant designs. Monotonic testing conducted on 2.44- by 7.32-m (8- by 24-ft.) and 6.10- by 7.32-m (20- by 24-ft.) roof assemblies, and subsequent data analyses utilizing a typical residential timber frame building provided ample quantification of roof system behavior to allow designers to include diaphragm action as a means of reducing forces in timber frame members and providing code-conforming designs for timber frame and SIP structures subject to wind loads.

Member forces in frames were reduced to allowable NDS-01 (AF&PA 2001) design limits without increasing timber sizes, whereas without including diaphragm action, members and joints were overstressed and thus not code compliant. Since timber frames are indeterminate systems, a two-dimensional frame analysis program should be utilized to determine forces within frame members. In calculating frame forces, it is non-conservative to assume the roof diaphragm to be perfectly rigid, thus it is recommended that designers perform a complete frame-diaphragm interaction analysis for calculating frame forces. Diaphragm tests of two 2.44-m- (8-ft.-) long roof assembly specimens may be required in order to determine diaphragm strength and roof stiffness for roof constructions significantly different from those tested for this research. For typical timber frame and SIP construction, this research demonstrated that there are no inherent features of these structures that would be a barrier to structural code conformance according to applicable building codes with respect to lateral wind load resistance.

Data obtained from the diaphragm testing program previously described provided sufficient information on the behavior of timber frame and SIP roof assemblies subject to lateral loads to reach the following conclusions:

- A simple beam diaphragm test utilizing three rafters with load applied to the center rafter, and the outer rafters secured, is an appropriate method for assessing the strength and stiffness of timber frame and SIP roof systems.

- Structural steel properties of the 229-mm (9-in.) screws used to attach the SIPs to the timber frame limited the ultimate shear capacity of the test assembly.

- Increasing screw diameter or decreasing the spacing of the screws would most likely increase the ultimate shear capacity and shear stiffness of the test assembly.

- A "chord force" failure did not occur in any diaphragm tests conducted; however, chord force transfer must be addressed by the building designer. A method for calculating chord forces for timber frame and SIP buildings is needed.

- A diaphragm-frame interaction analysis utilizing test data from a roof diaphragm assembly can be used to demonstrate the code conformance of timber frame members and joints when subjected to wind or seismic loads.

- A diaphragm-frame interaction design cannot be executed without an estimate of the stiffness and strength of the building endwalls, thus the greatest research needs at this time are design models for stiffness and strength of SIP clad endwalls with doors and windows.

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