DEMONSTRATION OF WIND LOAD DESIGN FOR TIMBER FRAME STRUCTURES USING DIAPHRAGM ACTION

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ABSTRACT. *Current design procedures for timber frame structures that are enclosed by structural-insulated panels (SIPs) do not typically accommodate the contribution made by the SIPs in resisting lateral loads. However, post-frame building design procedures do include the diaphragm action of the metal cladding that encloses them, leading to more efficient structures. This investigation draws parallels between timber frame and post-frame structural systems and elucidates the appropriateness of the post-frame diaphragm design methods for contemporary timber frame buildings. A sample timber frame diaphragm design is performed and the parameters needed to effectively complete the design are identified.*
Keywords. *Timber frame structures, Wind loading, Lateral loads, Structural insulated panels, Design methods, Post-frame structures, Diaphragm design.*

Timber frame buildings consist of large timbers connected with mortise and tenon joinery, and secured with wooden pegs to create a structural skeleton that is enclosed to provide shelter and interior spaces, as depicted in figure 1. Timber frame construction has been a dependable and enduring building system worldwide for centuries. However, with the advent of industrialization and the resulting ease of producing dimensional lumber early in the 19th century, light-frame construction became much more common and replaced timber framing as the mainstay of low-rise wooden buildings. Nevertheless, a revival of interest in timber frame building in the United States occurred during the 1970s, and since that time, the timber framing industry has grown to keep up with demand for timber frame houses, barns, churches, and other structures (O'Connell and Smith, 1999). This type of construction also remains popular in Europe and Japan.

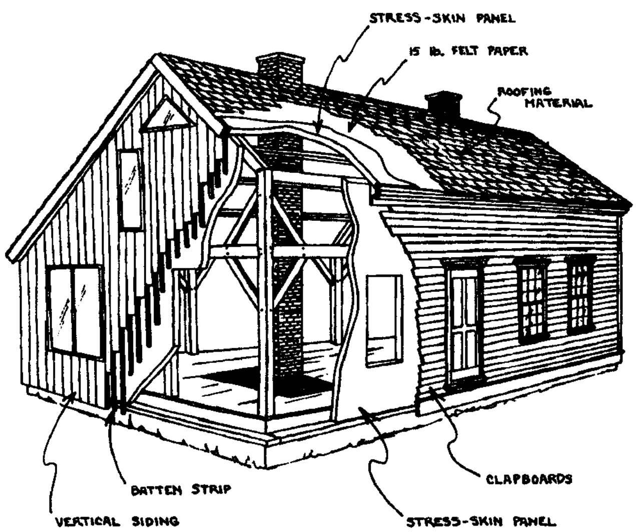

Figure 1– Typical timber frame enclosure as depicted by Benson and Gruber (1980, used with permission from Tedd Benson). Note: The "Stress-Skin Panels" in this figure should be considered to be SIPs.

Lateral forces resulting from wind and earthquakes can induce considerable stress 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 the timbers or joints. One area prone to overstress in timber frame structures is the tenon portion of a beam (see fig. 2) because it is much smaller than the rest of the member and is further limited in strength by holes for the wooden dowels and short edge

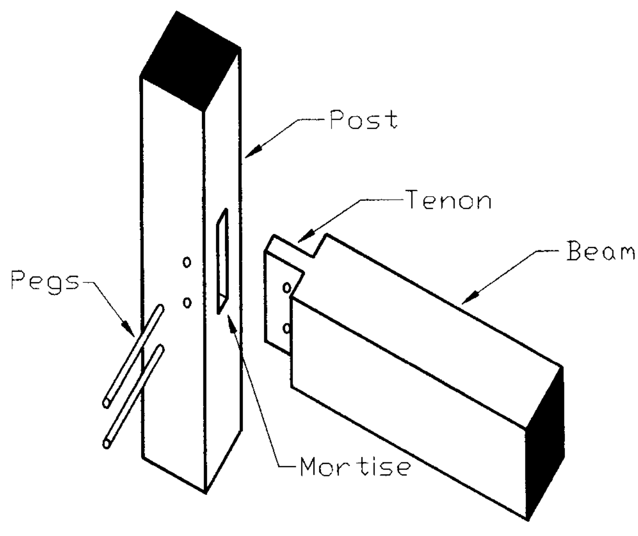

Figure 2–Elements of a mortise and tenon joint.

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distance from the dowel holes to the end of tenon. In order to accurately predict maximum stress these design parameters must be taken into account. For the sake of simplicity of this initial investigation, only wind loading will be considered.

Contemporary timber frame structures typically utilize structural-insulated panels (SIPs) attached to the timber frame skeleton to enclose the frame and create a functional structure. These panels consist of a layer of rigid insulation that is typically covered on one side by oriented strand board and on the other side by oriented strand board, gypsum drywall or some other interior finish, such as tongue and groove paneling. When designing timber frame structures, the SIPs are not assumed to assist the timber frame structurally.

Post-frame structures (see fig. 3), on the other hand, 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 the post-frame structural elements and account for the diaphragm action of the roof and endwalls.

Objective

The objective of this article was to assess whether the methods used to design for wind loads for post-frame building elements with metal cladding would be appropriate when designing to resist wind loads on timber frames using SIPs as cladding.

Literature

Recent literature on timber frame engineering has focused mainly on the analysis and design of members and joints within the structure. Levin (1993) performed a finite element analysis on a timber frame structure using two combinations of dead, live, snow, wind, and earthquake loads. This analysis provided data on the stresses in the members of the modeled timber frame. Levin (1995, 1996) then used the calculated stresses to exemplify a design procedure for the joints needed in the analyzed timber frame. A similar analysis was performed by Bulleit and Sandberg (1996). Other articles have addressed various aspects of joint design. Brungraber (1992) and Chappell (1995) discussed the engineering of tension joinery. Schmidt et al. (1996, 1999) wrote on the behavior and design of traditional timber connections and performed experiments to identify failure mechanisms of these types of wooden connections.

Although there are numerous publications on joint design and timber frame building analysis, none of the articles describe methods for including the structural contribution of materials used to clad the timber frames. Engineers and timber frame designers need to account for the diaphragm elements when determining the percentage of the total lateral load resisted by individual timber frame elements.

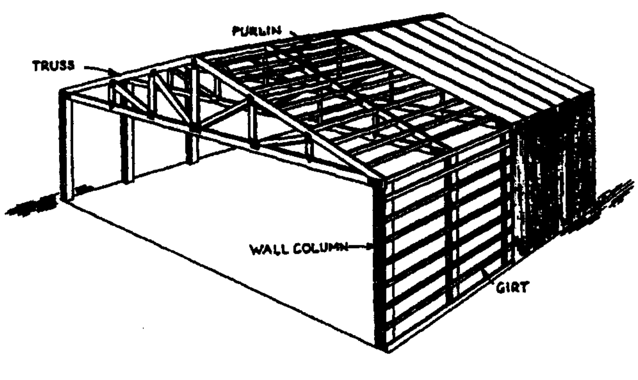

Figure 3–Post-frame structural elements from Hoyle and Woeste (1989).

Summary of Diaphragm Design Procedure for Post-Frame Structures

A post-frame building consists of a series of frames made up of trusses that are commonly assumed to be pinned to the tops of columns. The columns are embedded in the ground or attached to a concrete slab as depicted in figure 3. Wall girts span between the posts, and purlins span between the upper chords of the roof trusses. Metal cladding typically is attached to girts and purlins to create the wall and roof diaphragms, respectively. These diaphragms not only serve to enclose the structure, but contribute significantly to the stiffness and strength of the building.

Design loads for a post-frame building typically consist of wind, roof snow, and dead loads. In this article, design procedures for wind and dead loads will be considered, and it will be assumed that the controlling wind direction is perpendicular to the ridge of the roof. The following design recommendations for post-frame buildings were taken from the short course notes, *Design of Commercial Post-Frame Buildings* by Bender and Woeste (1999). Girts and purlins are designed as beams, with dimensions controlled by bending and shear stresses, and bending deflection. Connections for these members are based on the most severe loading conditions throughout the building. The building designer usually relegates the truss designs to the truss designer, who is provided with the design loads and building dimensions.

The remaining components (the frame posts and the wall and roof diaphragms) are designed using several steps. From the design wind loads, a maximum roof shear is calculated and a roof construction is chosen, based on the shear loading. Then purlins are designed to carry the roof diaphragm chord forces. The sidewall post sizes are selected based on partitioning the lateral load carried by the post-frame and the selected roof construction. The end wall construction is selected based on the maximum roof shear. Embedment of the posts can then be determined based on the ground moments that are calculated when designing the posts. These design procedures for post-frame structures have become simplified enough, that by making some appropriate assumptions, even the posts can be designed by hand (Skaggs et al., 1993), that is, without the use of computer software. Post-frame buildings, by their nature of two posts connected to a truss, lend themselves to straightforward analyses when calculating member forces and stresses.

How to Use the Post-frame Methods for Timber Frame Structures

Frames that make up timber frame buildings are usually more complex than those used in post-frame buildings, but the overall structural system is similar. A timber frame building generally consists of a series of frames, or bents, that contain several posts, beams, rafters, and knee braces. These frames are connected to each other with beams and roof purlins, and the frame posts are often placed into pockets in the first floor decking, or attached to concrete bases or a slab. SIPs are usually attached to the timbers on the outer edges of the building with long nails that penetrate through all panel layers and into the timbers several inches. If the panels are connected soundly to the timber frame and to one another, it is reasonable to assume that the panels are resisting and distributing laterally applied building loads, similar to the diaphragm action used in post-frame structures.

The Moran timber frame by Benson Woodworking (Christian, 1997) is a good example of a typical residential timber frame, and will be used to assess the applicability of the post-frame diaphragm methods for use with timber frame building systems. Due to the complexity of a typical timber frame bent, it is necessary to use a two-dimensional frame analysis program to calculate the forces in the members based on the calculated design loads and tributary load areas of the building. In order to exemplify this type of analysis, the central bent of the Moran frame was entered into PPSA4 (Triche and Suddarth, 1993). The dead loads on the frame were based on the weight of the wood and panels, and the only live load applied to the building was a distributed wind pressure based on the 1997 Uniform Building Code (ICBO, 1997), using Exposure Category C and 35.8 m/s (80 mph) wind speeds. The applied wind loads and the structural analog can be seen in figure 4.

In order to simulate some of the flexibility inherent within timber frame construction, fictitious members with a specific modulus of elasticity (Triche and Suddarth, 1993) were added between the braces and the posts that they are connected to. The modulus of elasticity of these fictitious members was derived using information from research on

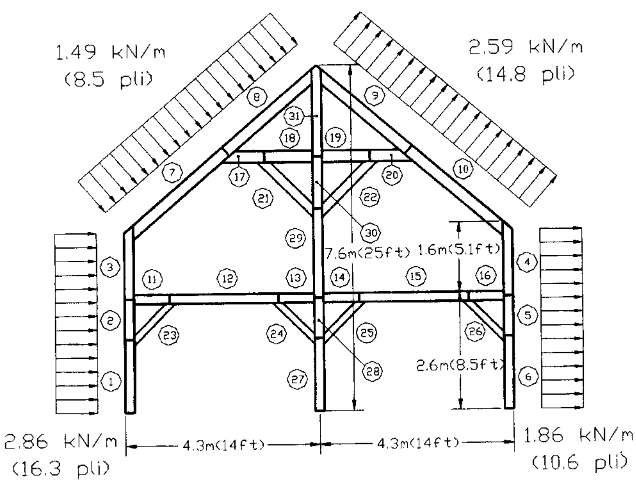

Figure 4–The center bent of the Moran timber showing the frame loads as a result of combining dead and wind loads when the wind load is assumed to act perpendicular to ridge.

timber frame joint behavior (Schmidt, 1996, 1999; Bulleit, 1996; Chappell, 1995). Two analyses were run using PPSA4, one with and one without the fictitious members. Both of these analyses assumed no contribution from the SIPs. As shown in columns 1 and 2 of table 2, both analyses resulted in members of the frame being overstressed according to NDS-1997 combined stress criteria (AF&PA, 1997), given by equations 1 and 2:

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0 \qquad (1)$$

where

f_t = actual tension stress parallel to grain (psi)
F_t' = allowable tension design value parallel to grain (psi)
f_b = actual bending stress (psi)
F_b^* = tabulated bending design value multiplied by all applicable adjustment factors except C_L (psi)

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{bl}}{F_{bl}'\left[1-\left(\frac{f_c}{F_{cEl}}\right)\right]} \leq 1.0 \qquad (2)$$

where

f_c = actual compression stress parallel to grain (psi)
F_c' = allowable compression design value parallel to grain (psi)
f_{b1} = actual edgewise bending stress (psi)
f_c $< F_{cE1} = K_{cE}E'/(l_{e1}/d_1)^2$ for uniaxial bending
K_{cE} = Euler buckling coefficient for columns
E' = allowable modulus of elasticity (psi)
l_{e1} = effective column length (in.)
d_1 = wide face dimension (in.)

These results lead to the conclusion that either the panels must be considered as a structural element that contributes to support of the timber frames, or the timber frame member sizes need to be increased in order to be effectively designed within code requirements for wind loads.

If the steps to design the post-frame building diaphragm system and posts that were previously discussed can be used to estimate the stresses in the timber frame and the amount of shear that the SIPs need to resist, then the contribution of the SIPs to the lateral load resisting capacity of the timber frame system can be quantified. Each of the post-frame diaphragm design steps is assessed for applicability and missing information is noted for future investigations.

The maximum roof shear, V_{max}, for post frame can be conservatively estimated using a simplified formula (Bender et al., 1991) that incorporates the wind pressures on the building and the building dimensions. However, for a timber frame, the braced frame is much stiffer and more complex than a typical post frame, and, therefore, the procedures of ANSI/ASAE EP484.2 (ASAE, 1999) are proposed to calculate the maximum shear in the roof and endwalls. A roof construction must be assumed to start the diaphragm design procedure.

Selecting a roof construction that can carry the maximum roof shear is difficult because of the lack of quantitative data regarding SIPs. Post-frame designers commonly use diaphragm test assembly data supplied by manufacturers to select a diaphragm system that will be strong enough. Currently, allowable shear ratings for SIPs installed in a building are not available, although for the sake of this example, it will be assumed that a suitable assembly is available.

The assumption that the entire roof is acting as a beam, results in axial chord forces being induced into the outermost roof members when the roof is subject to wind loading, as in this example. The top plates of the Moran frame are made of 152 mm × 203 mm (6 in. × 8 in.) members on some edges and 152 mm × 254 mm (6 in. × 10 in.) members on the remaining edges. The chord forces induced due to wind loading are assumed to be easily handled by these oak plates. Additionally, Pollock (1996) developed a method where the roof purlins are assumed to carry a portion of the wind load also, therefore the 203 mm × 254 mm (8 in. × 10 in.) eastern white pine interior purlins would also carry some chord forces, further reinforcing the assumption that chord forces are not an issue for the timber frame roof system in this example. All of these members are full sized, not nominal.

The next step in the post-frame procedure is to design the sidewall posts based on the selection of the roof construction. For a timber frame, it is necessary to design the posts, beams (or bent girts), rafters and braces for the frame in question. The first step is to calculate the roof diaphragm stiffness, c_h. Post-frame analysis requires a load-deflection test on a laboratory size panel assembly representative of the roof construction for determining the stiffness (and strength) of the panel (Bender and Woeste, 1999). From this test data, the stiffness of the actual roof in question can be calculated using an adjustment that relates test panel stiffness to roof stiffness. No test data of this type is currently available for SIPs. It will be assumed that the roof stiffness, c_h, is 1528 kN/m (8,727 lb/in.) for this design demonstration. The assumed stiffness value was from metal roof diaphragm testing, where the test panels were 2.44 m (8 ft) long and 3.05 m (10 ft) wide and had a stiffness, C_1, of 1243 kN/m (7,100 lb/in.) (Bender and Woeste, 1999).

Using the metal roof diaphragm values mentioned above, one assumes that the timber frame is clad as though it were a post-frame building and that the connections between the SIPs and the frame have the same stiffness as in a post-frame building. The stiffness of the SIP and timber frame assemblies will be largely dependent upon the shear connection system used to connect the SIPs to each other and to the timber frame. For example, if the SIPs were connected to each other using 12d nails on 24 in. centers, the stiffness would be much lower than if they were attached using the same nails on 6 in. centers. Testing is needed to determine the stiffness of various connection systems that are utilized by the timber frame industry. Since this data is not currently available, typical post-frame data will be used to demonstrate the potential value of diaphragm design for timber frame and SIP construction. The frame stiffness, k, is approximated by applying a concentrated load to the frame in PPSA4 and dividing this load by the deflection it causes at the point of application. The calculated stiffness, including joint slippage, for the Moran frame was 303 kN/m (1730 lb/in.). It should be noted that this frame stiffness is much higher than for a typical post-frame building. The maximum potential lateral restraining force of the roof diaphragm, R, is calculated by placing a roller reaction at the eave edge of the frame and applying the loads to the frame in PPSA4 to obtain the reaction, R. A value of R equal to 20.4 kN (4578 lb) resulted.

The sideways restraining force factor, mD, is a ratio that establishes how much of the lateral loads need to be resisted by the frames and how much is being resisted by the roof diaphragm. This factor can be found in a chart where the vertical axis is c_h/k and the horizontal axis is the number of frames, including both end walls (ASAE, 1999). Along the vertical axis as well is a column for k_e/k, which is the ratio of the endwall stiffness to the frame stiffness. Since there is a lack of data on the stiffness of timber frame endwalls and in order to simplify this example, it was assumed for this demonstration that the endwalls were rigid, resulting in a value for k_e/k of 10,000. If an endwall contained several openings, such as windows and doors, it may be possible to supply additional bracing or reinforcement in order to meet this assumption of the rigid endwall. If reinforcing the endwall to meet the "rigid" assumption is not possible, ANSI/ASAE 484.2 (ASAE, 1999) provides a method for taking endwall deflection into consideration for the calculation of the appropriate endwall stiffness. Laboratory tests performed on timber frame and SIP structures will reveal if the potentially non-conservative assumption of a "rigid endwall" is suitable.

The mD table provided a value of 0.91, where there are three frames in the building and c_h/k equals approximately 5.05. Multiplying the roller reaction, R, by mD, the roof diaphragm resists a lateral force, R, equal to 18.5 kN (4,166 lb). The calculated percentage of the lateral load carried by the roof diaphragm is 91%.

Forces and stresses within the members of the center bent of the Moran frame can be determined by performing another analysis using PPSA4 where the calculated diaphragm resisting capacity of 18.5 kN (4,166 lb) is applied to the frame as a horizontal distributed load 5.21 kN/m (357 lb/ft) along both roof slopes in the direction opposite of the wind. The reductions in member tension forces from the no diaphragm action cases (Models 1 and 2) are shown in table 1, where the axial force in each member is shown for an analysis without joint slip, with joint slip included, and for the final analysis that accounts for roof and end wall diaphragms (diaphragm action). By estimating the contribution of the SIPs in resisting lateral loads due to wind, the analysis shows that none of the members in the frame are overstressed, and all are within NDS-1997 (AF&PA, 1997) combined stress design requirements.

The final step of the procedure is to calculate the maximum roof and endwall shear and compare this to the allowable shear that is determined through laboratory diaphragm assembly tests. The maximum shear is calculated by multiplying the lateral force R by the shear force modifier mS and then dividing by the building width. The mS table uses the same input parameters as the mD table, and the value for this building is 0.96. The resulting maximum roof and endwall shear is 2.29 kN/m (157 lb/ft). The allowable shear for SIPs can be determined using ASTM E455-98 (ASTM, 2000). A factor of safety must be

Table 1. The comparison of member forces as affected by the models used to analyze the timber frame

Member	Axial Force					
	Model 1 (no joint slip) kN(lb)		Model 2 (with joint slip included) kN(lb)		Model 3 (with joint slip and diaphragm action included) kN(lb)	
1	2.59	(583)	2.59	(583)	−11.3	(−2530)
2	−18.5	(−4170)	−18.6	(−4170)	−14.4	(−3230)
3	−3.77	(−847)	−3.77	(−847)	−9.47	(−2130)
4	−1.88	(−423)	−1.89	(−424)	0.380	(85.0)
5	6.85	(1540)	6.84	(1540)	−0.820	(−185)
6	−16.3	(−3660)	−16.3	(−3660)	−5.83	(−1310)
7	5.39	(1210)	5.38	(1210)	−7.03	(−1580)
8	8.55	(1920)	8.84	(1990)	5.00	(1120)
9	5.46	(1230)	5.72	(1290)	7.00	(1570)
10	2.33	(524)	2.33	(524)	3.46	(779)
11	−24.7	(−5550)	−24.7	(−5550)	−5.45	(−1230)
12	−0.355	(−799)	−3.54	(−796)	−2.33	(−524)
13	17.0	(3830)	16.7	(3750)	3.77	(847)
14	−17.2	(−3860)	−17.0	(−3810)	−4.19	(−941)
15	1.29	(289)	1.28	(287)	−0.070	(−15.0)
16	24.4	(5490)	24.4	(5480)	4.94	(1110)
17	−8.38	(−1890)	−8.62	(−1940)	−10.6	(−2380)
18	1.76	(395)	1.14	(256)	−7.09	(−1600)
19	−6.17	(−1390)	−6.61	(−1490)	−9.26	(−2080)
20	0.220	(50.0)	0.030	(6.00)	−9.10	(−2050)
21	−14.5	(−3260)	−13.9	(−3140)	−5.00	(−1120)
22	9.13	(2050)	9.48	(2130)	0.230	(52.0)
23	29.9	(6720)	29.9	(6720)	4.41	(992)
24	−29.1	(−6540)	−28.6	(−6420)	−8.62	(−1940)
25	26.1	(5870)	25.8	(5800)	5.83	(1310)
26	−32.7	(−7350)	−32.7	(−7350)	−7.09	(−1590)
27	−24.9	(−5590)	−24.9	(−5590)	−21.5	(−4820)
28	−22.8	(−5120)	−22.9	(−5140)	−19.5	(−4380)
29	−14.5	(−3250)	−14.5	(−3250)	−11.0	(−2480)
30	−10.6	(−2390)	−11.3	(−2530)	−7.62	(−1710)
31	−12.1	(−2710)	−12.4	(−2790)	−8.78	(−1980)

Table 2. The choice of structural analog greatly affected the calculated combined stress values for the frame

Member	Combined Stress Values Using Equations 1 and 2		
	Model 1 (no joint slip)	Model 2 (with joint slip included)	Model 3 (with joint slip and diaphragm action included)†
1	2.21	2.21	0.97
2	2.22	2.22	0.98
3	1.13	1.13	0.34
4	1.2	1.2	0.27
5	2.29	2.29	1
6	2.27	2.27	1.01
7	0.23	0.23	0.3
8	0.25	0.25	0.2
9	0.19	0.19	0.34
10	*	*	*
11	1.46	1.46	0.23
12	1.43	1.43	0.32
13	1.33	1.33	0.35
14	1.18	1.18	0.29
15	1.48	1.48	0.28
16	1.66	1.66	0.31
17	0.34	0.33	0.12
18	0.35	0.33	0.12
19	0.2	0.21	0.02
20	0.2	0.21	0.01
21	0.2	0.04	0.07
22	0.09	0.09	0
23	0.44	0.45	0.07
24	0.85	0.8	0.24
25	0.39	0.38	0.09
26	0.96	0.91	0.2
27	3.44	3.44	0.98
28	3.4	3.4	0.96
29	1.49	1.5	0.41
30	1.48	1.48	0.41
31	0.07	0..08	0.01

† When diaphragm action (model 3) was included, the largest interaction value was 1.01 (slightly above the NDS-1997 limit of 1.0).
* If both tension and compression existed along the member length no interaction value was calculated by PPSA4.

applied to the test results in order to determine the maximum allowable shear for the SIP assemblies and will also be recommended as a part of the planned research. If the maximum shear that the roof and endwalls resist is less than the allowable for the diaphragm systems, that include the effect of the connections between the SIPs, the timber frame building with SIPs as cladding can be designed and built safely.

Summary and Conclusions

For typical timber frame buildings with endwalls having enough diaphragm stiffness and strength to resist the maximum roof shear, the lateral design method for post-frame buildings with metal cladding should be appropriate when applied to calculating the wind load induced stress on timber frame buildings with SIP cladding. A two-dimensional frame analysis program illustrated that the timber frame elements are not over-stressed (i.e., NDS-1997 requirements) (AF&PA, 1997) if the roof and wall diaphragms are considered to be resisting a portion of the lateral wind load on the building. Without diaphragm action, several members were overstressed as indicated by compression and bending interaction values, which were greater than 1.0 (table 2). While assessing the appropriateness of using the post-frame diaphragm methods for timber frame building construction, several gaps in existing data have been identified.

Based on the analysis of the example Moran timber frame (Christian, 1997), it was found that additional information is needed to accurately account for the contribution of SIPs within the modern timber frame structural system. Information about the behavior of structural-insulated panel assemblies for roofs and walls under lateral loading is needed in order to obtain the data that are now available for some metal on wood-frame diaphragm assemblies for use with post-frame structures. Full-scale tests of structural panel assemblies, including the connection between the panels and the timber frame, are required in order to provide this information.

A straightforward method of accounting for the diaphragm contribution of SIPs to timber frame construction potentially has great value to timber frame designers and manufacturers. Although it may never be as simplified as for post-frame building design, the structural design of timber frame buildings will be manageable and greatly simplified for building designers and for code officials who approve these types of structures.

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