

SIMPLIFIED LATERAL DESIGN OF POST-FRAME BUILDINGS

Education can help industry to move forward

Post-frame construction is growing across a variety of building markets because of advantages related to cost, reliability and ease of construction. Much of the structural efficiency of post-frame buildings is attributed to diaphragm action distributing lateral loads (e.g., wind and seismic forces) to the shear walls of the buildings. When embedded posts are used in the foundation system, the frames and the roof diaphragm interact to resist lateral loads. Sophisticated design methods have evolved to account for the interaction between frames and diaphragm (e.g., ASAE, 2003; Bohnhoff, 1992a, 1992b; Anderson, Bundy, & Meador, 1989), and to design embedded post foundations with a range of detailing and soil behavior assumptions (e.g., ASAE, 2005; McGuire, 1998; Meador, 1997). Designers that specialize in post-frame construction are well acquainted with these design tools. However, these design methods are not readily available or familiar to structural engineers with limited experience in post-frame; hence limiting the expansion of post-frame construction. As post-frame construction grows into new markets, rational, simplified design methodologies that can be quickly learned and economically implemented by design and building regulatory professionals are needed. The objective of this article is to present a simplified design method that provides conservative designs for roof diaphragms, shear walls, post members and embedded post foundations. Hence a structural engineer with a limited number of post-frame building projects per year can justify the cost of learning the design method.

Design Overview

A good technical resource for diaphragm and shear wall design for light-frame wood construction is APA Publication L350A, Diaphragms and Shear Walls—Design/Construction Guide (APA Engineered Wood Association, 2007). Lateral design is the same for post-frame and light-frame wood construction, except for the following:

- If posts are embedded, the distribution of lateral loads between the foundation and the roof diaphragm is changed. In this article, we conservatively ignore the contribution of the frame because it is much less stiff than the diaphragm, and we obtain solutions that are conservative and easier to comprehend.
- Standardized diaphragm and shear wall design capaci-

ties are available for light-frame construction (e.g., ANSI/AF&PA SDPWS–2008 standard (American Forest and Paper Association [AF&PA], 2008). Less data are available in the public domain for post-frame diaphragm and shear wall constructions that use metal cladding on wood framing. The National Frame Building Association (NFBA) is currently working to develop the standardized diaphragm and shear wall data needed by designers.

- Post-frame roof diaphragms have repetitively framed purlins that can share chord forces, which has a significant impact on chord member and splice connection design.

Determining Roof Diaphragm Forces Using the Rigid Roof Design Method

The rigid roof design method (Bender, Skaggs, & Woeste, 1991) is conservative with respect to roof diaphragm design because a propped cantilever analog is assumed, as shown in Figure 1. The pin supporting the top of the post represents an infinitely stiff roof diaphragm, thus attracting load to the diaphragm. More complicated analysis procedures are available that model the diaphragm as a spring supporting the top of the post, resulting in lower diaphragm loads.

Figure 1. Propped cantilever analog for rigid roof design

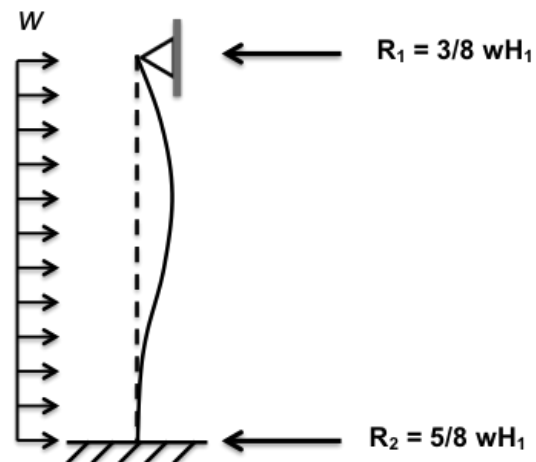
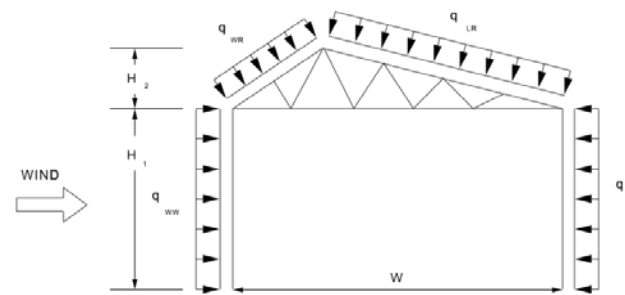


Figure 2 shows a hypothetical wind loading on a post-frame building. The resulting unit shear for the building is given by:

$$v = \frac{K(q_{ww} - q_{LW})H_1L + (q_{WR} - q_{LR})H_2L}{2W}$$

- v = unit roof shear intensity
- K = 3/8 for embedded posts
- K = 1/2 for surface-mounted posts
- q_{ww} = design windward wall pressure (+ sign for inward pressure, - sign for outward pressure)
- q_{LW} = design leeward wall pressure (+ inward, - outward or suction)
- q_{WR} = design windward roof pressure (+ inward, - outward or suction)
- q_{LR} = design leeward roof pressure (+ inward, - outward or suction)
- H_1 = side-wall height
- H_2 = roof height
- W = building width
- L = building length

Figure 2. Wind loading on post-frame building



At this point, a diaphragm construction can be selected to meet the conservative estimate of unit shear demand. Some allowable design values are available for metal-clad wood-framed diaphragms and shear walls in the Post-Frame Building Design Manual (NFBA, 1999), and NFBA is currently working to develop a standardized design database. Another option is to use wood panels on wood framing; design data can be found in the ANSI/AF&PA SDPWS–2008 standard (AF&PA, 2008).

Shear Wall Design

If the shear wall has no openings, simply use the unit shear calculated for the roof diaphragm and select a shear wall construction to carry the load.

If the shear wall has an opening such as an overhead door, the segmented shear wall approach can be used where the end of each wall segment has a hold-down (or post). The unit shear demand is given in the following equation:

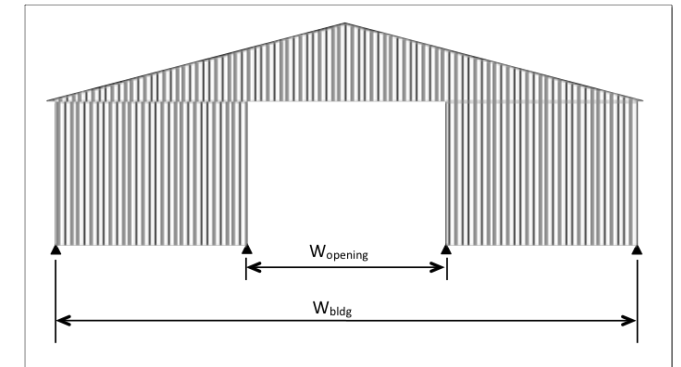
$$V_{\text{shear wall}} = V_{\text{max}} / (W_{\text{bldg}} - W_{\text{opening}})$$

where

$$V_{\text{max}} = vW \text{ (defined previously)}$$

W_{bldg} and W_{opening} as illustrated in Figure 3.

Figure 3. Shear wall with opening



At this point, a shear wall construction can be selected to meet the unit shear demand.

Roof Diaphragm Chord Forces

Usually the perimeter chords are assumed to resist all of the bending moment in light-frame wood diaphragms. In post-frame roof diaphragms, roof purlins can be assumed to share the chord forces as described by Pollock, Bender and Gebremedhin (1996) and illustrated in Figure 4.

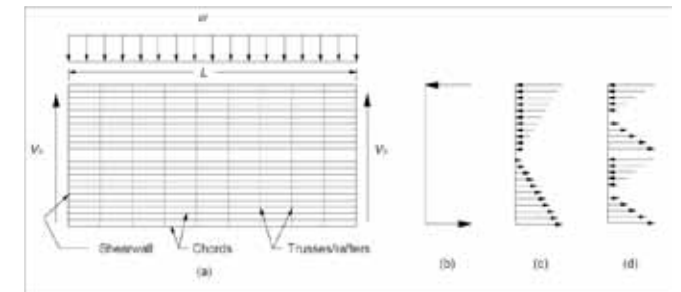


Figure 4. (a) Plan view of a diaphragm under a uniform load, w . Chord force distribution when (b) moment is resisted by edge chords only, (c) chord force distribution is linear, and (d) chord force distribution is linear, but diaphragm halves are assumed to act independently in resisting moment (NFBA, 1999).

Procedure for calculating chord forces

1. Solve for maximum shear force, V_{max} , in the roof diaphragm using the rigid roof equation.
2. Calculate the resulting uniform load, w , on the roof diaphragm, assuming the load is evenly distributed along the building length.

$$w = 2V_{\text{max}}/L$$

3. Solve for the maximum bending moment, M , in the roof diaphragm. Engineering judgment is required with regard to the end conditions of the roof diaphragm. If a simple beam with pin and roller supports is assumed, the maximum moment is at the mid-length of the building as follows:

$$M = wL^2/8$$

If a beam with fixed conditions is assumed, the maximum

moment will occur at the ends of the building as follows:

$$M = -wL^2/12$$

4. Solve for the maximum axial force on the perimeter purlin, T_n , using the following equation

$$T_n = \frac{Md}{\sum_{i=1}^n [d - 2(n-i)s(\cos\theta)]^2}$$

where

- M** = bending moment in roof diaphragm (ft-lb)
- d** = diaphragm depth (ft) (roof span)
- i** = purlin number, starting from the ridge and working to the eave
- n** = number of purlins from ridge to eave (one side of the roof and not counting the ridge purlin)
- s** = purlin spacing (ft)
- θ** = roof slope

The NFBA's Post-Frame Building Design Manual used the approach in Step 4 to create a convenient design table (see Table 1). In this case, the maximum chord force is given by:

$$T_n = M \alpha / d$$

Table 1. Reduction factor, α , for axial force in edge chords

n^*	α	n^*	α
2	1.000	22	0.249
3	1.000	23	0.239
4	0.900	24	0.230
5	0.800	25	0.222
6	0.714	26	0.214
7	0.643	27	0.206
8	0.583	28	0.200
9	0.533	29	0.193
10	0.491	30	0.187
11	0.455	31	0.181
12	0.423	32	0.176
13	0.396	33	0.171
14	0.371	34	0.166
15	0.350	35	0.162
16	0.335	36	0.158
17	0.314	37	0.154
18	0.298	38	0.150
19	0.284	39	0.146
20	0.271	40	0.143
21	0.260	41	0.139

* n is the total number of purlins in the diaphragm.

Finally, the maximum chord force T_n is used to size the chord members and splices.

Post Member Forces and Embedded Foundation Design

Post member design

Maximum eave deflection will occur at the mid-length of a symmetric building, so this is usually the critical frame with respect to post member design and required embedment. Using the propped-cantilever model, we can calculate the maximum positive and negative (ground line) moments on the post, yet this simple structural analog does not allow any eave deflection.

In a real building, the eave will deflect an amount Δ_{eave} under lateral loading as shown in **Figure 5a**. The eave deflection will cause a negative moment in the post as shown in **Figure 5b**. By superposition, we can solve for the maximum moments in the positive and negative regions of the post.

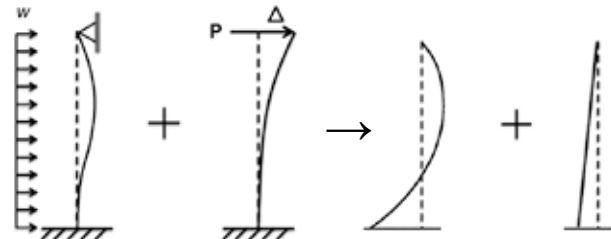


Figure 5. (a) Superposition of analogs (b) Superposition of moments

Bending moment for propped cantilever with uniform load

From beam tables, we can find the maximum positive and negative moments for a propped-cantilever member with uniform load as follows:

$$M_{max}^+ = 9wH_1^2/128 \quad (\text{occurs at } 3/8 \text{ L down from top of post})$$

$$M_{max}^- = wH_1^2/8 \quad (\text{occurs at ground line})$$

Bending moment for cantilever with point load P

If we know the eave deflection Δ_{eave} at building mid-length,¹ we can solve for the force P that would cause that deflection using the following equation:

$$\Delta_{eave} = PH_1^3/3EI$$

Solving the equation for P , and then substituting $M = PH_1$, we have the equation for bending moment caused by eave deflection Δ_{eave}

$$M_{max}^+ = PH_1 = 3\Delta_{eave}EI/H_1^2 \quad (\text{at ground line})$$

$$M^- = 3/8 PH_1 = 9\Delta_{eave}EI/8H_1^2 \quad (\text{at } 3/8H_1 \text{ down from top of post})$$

Combine moments using superposition

The maximum net moment in the positive region occurs at approximately $3/8H_1$ from the top of the post. Note the difference in signs of the two moments.

$$M_{max}^+ = 9wH_1^2/128 - 9\Delta_{eave}EI/8H_1^2$$

Similarly we solve for the maximum negative moment at the ground line:

$$M_{max}^- = wH_1^2/8 + 3\Delta_{eave}EI/H_1^2$$

The ground line moment is needed to calculate post embedment depth and often controls member design.

Note that all relevant load combinations should be checked when designing any member. For post-frame posts, a combi-

nation of gravity and lateral loads typically controls. Gravity loads are straightforward to calculate.

The post member design is accomplished using the combined bending and axial compression Equation 3.9-3 in the ANSI/AF&PA NDS-2005 (AF&PA, 2005).

An alternate form to calculate post moments is presented on page 9-7 of the *Post-Frame Building Design Manual* (NFBA, 1999). This approach gives equations to calculate shear and moments at different points in a post by summing forces based on statics. These equations yield identical results to that of the preceding equations.

Calculating eave deflection

Calculating eave deflection is required to determine the maximum post moment. Pope, Bender and Mill (2012) present a three-term equation to predict diaphragm deflection that includes deflection contribution from bending of the diaphragm framing and chord slip which is presented as

$$\delta_{dia} = \frac{15vL^3}{4EA_s n(n+1)} + \frac{.25vL}{1000G_a} + \frac{\Delta_c}{W} \sum x_i$$

where

- v** = applied unit shear (lb/ft),
- L** = diaphragm length (ft),
- E** = modulus of elasticity of the diaphragm chords (psi),
- A** = cross-sectional area of the chords (in²),
- s** = chord spacing (ft),
- n** = number of chords
- W** = diaphragm width (ft),
- G_a** = apparent shear wall stiffness (k/in),
- Δ_c** = diaphragm chord slip (in), and
- x** = distance from chord splice to nearest support (ft).

The three terms account for deflection due to diaphragm framing bending, shear, and chord slip, respectively. This equation is similar to that of the code-accepted ANSI/AF&PA SDPWS-2008 (AF&PA, 2008) equation for deflection of diaphragms with wood sheathing on wood framing, which is given as

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{.25vL}{1000G_a} + \frac{\sum(x_i \Delta_c)}{2W}$$

The difference in bending terms between the two equations stems from the fact that the SDPWS equation considers wood-sheathed, wood-framed diaphragms to act as a deep beam where only the outermost framing member acts as chords to resist the moment in the diaphragm (typically the double top plate). The equation of Pope et al. (2012) accounts for the contributions of all chords (purlins), not just the outer ones.

The third term, which accounts for chord slip, varies in how the cumulative distances from chord splices to the end walls are calculated because of how purlin splices are located in post-frame buildings. It is also assumed that the butt joints in the chords are not perfectly tight and that the slip of the tension

chord equals the slip of the compression chord. Therefore, the total splice slip would be double that of the tension or compression slip alone (Pope et al., 2012). This explains why the third term in Pope's deflection equation is twice that of the SDPWS equation. A more detailed explanation of the differences and derivation of these equations can be found in Pope et al. (2012).

The total eave deflection, Δ_{eave} , is the deflection of the shear wall added to the diaphragm deflection. The shear wall deflection can be calculated by Equation C.4.3.2 - 2 (ANSI/AF&PA SDSWS-2008, in AF&PA, 2008):

$$\delta_{sw} = \frac{8vH_1^3}{EA_b} + \frac{vH_1}{1000G_a} + \frac{H_1}{b} \Delta_a$$

where v , H_1 , G_a defined previously, and E are the modulus of elasticity of the end posts (psi), A is the cross-sectional area of the end wall posts (in²), b is the shear wall length (ft), and Δ_a is shear wall anchorage slip (in). Similar to the SDPWS equation for diaphragm deflection, the three terms of the shear wall deflection equation above account for deflection due to framing bending, shear, and wall anchorage slip, respectively. Because the posts are embedded in the ground for post-frame construction, it is assumed that no wall anchorage slip occurs, therefore eliminating the third term of the equation.

Embedded post foundation design

Once the ground line moment is determined, the post embedment depth can be calculated using Equation 18-3 in Section 1807.3.2.2 of the 2009 International Building Code (IBC). It should be noted that this equation applies to the situation where there is ground line constraint, such as provided by a concrete slab on grade. The posts on the leeward side of the building are commonly tied into the concrete slab with steel rebar.

$$d = \sqrt{\frac{4.25M_g}{S_3b}}$$

where

- d** = post embedment depth, ft
- M_g** = moment in the post at grade, ft-lb
- b** = diameter of round pole or diagonal dimension of post, ft
- S₃** = allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment, psf. Section 1806.3.3 allows lateral pressures from Table 1806.2 to be increased by the tabular value for each additional foot of depth, to a maximum of 15 times the tabular value. Note that an initial guess of embedment is needed to calculate the starting value for S_3 .

More extensive post embedment design equations to cover situations such as posts with collars and post uplift, as well as a variety of soil resistance assumptions, can be found in ANSI/ASAE EP486.1 Shallow Post Foundation Design (ASAE, 2005), McGuire (1998) and Meador (1997). For the simple constrained post case given in the IBC, the ASAE EP486.1 standard gives a more convenient form of the equation that does not require iterative calculations ($S' = S_3/d$)

$$d = 3 \sqrt{\frac{4.25M_g}{S'b}}$$

S' = allowable lateral soil-bearing pressure in psf/ft from Table 1806.2 (IBC, 2009)

Design Example

We will work the same example problem that is given in the NFBA'S *Post-Frame Building Design Manual*, Chapter 9. Table 2 gives the building specifications for the example.

Table 2. Example building specifications

Width (truss length)	36 ft
Length (along ridge)	60 ft
Height at post bearing	12 ft
Roof slope	4/12 (18.43°)
Bay spacing	10 ft
Number of frames (including end walls)	7
Post embedment depth	4 ft
Post grade & species	No. 2 S. Pine
Post size	Nom. 6- by 6-in.
Roof snow load	30 psf
Roof dead load	5 psf
Concrete slab?	Yes
Ceiling?	No
Wind speed	80 mph
Exposure category	B
Windward wall, q_{ww}	8.13 psf
Leeward wall, q_{lw}	-5.08 psf *
Windward roof, q_{wr}	3.05 psf
Leeward roof, q_{lr}	-7.12 psf *

* Negative loads act away from the surface in question. Positive loads act toward the surface in question.

Roof diaphragm

Calculate maximum unit shear in roof diaphragm

$$v = \frac{K(q_{ww} - q_{lr})H_1L + (q_{wr} - q_{lr})H_2L}{2W}$$

Select roof diaphragm construction to carry at least 100.4 lb/ft.

Shear wall

Assume a 12-ft wide overhead door in one end wall.

$$V_{max} = vW = 100.4 * 36 \text{ ft} = 3,614 \text{ lb}$$

$$v_{shear \text{ wall}} = V_{max} / (W_{bldg} - W_{opening}) = 3,614 / (36 - 12) = 151 \text{ lb/ft}$$

Select shear wall construction to carry at least 151 lb/ft.

Solve for roof diaphragm chord forces

The roof purlins need to be sized to carry all anticipated load combinations (e.g. dead, snow, live and wind). The axial chord forces can be calculated as follows.

Procedure

1. Solve for maximum shear force, V_{max} , in the roof diaphragm using the rigid roof equation.

$$V_{max} = 3,614 \text{ lb}$$

2. Calculate the resulting uniform load, w , on the roof diaphragm, assuming the load is evenly distributed along the building length.

$$w = 2V_{max}/L = 2 * 3,614/60 = 120.5 \text{ lb/ft}$$

3. Solve for the maximum bending moment, M , in the roof diaphragm, assuming a simple beam with pin and roller supports.

$$M = wL^2/8 = 120.5 (60)^2/8 = 54,212 \text{ lb/ft}$$

4. Solve for the maximum axial force on the perimeter purlin, T_n , using the following equation with $(36/2 + 1) = 19$ purlins.

From Table 1: $\alpha = 0.284$

$$T_{max} = M \alpha / d = 54,212 * 0.284 / 36 = 428 \text{ lb}$$

The maximum chord force T_n is used to size the chord members and splices.

Post moments

From the NFBA example:

$$E = 1.2 \times 10^6 \text{ psi}$$

$$I = 76.26 \text{ in}^4 \text{ (nominal 6x6 post)}$$

$$\Delta = 0.655 \text{ in}$$

$$w = 8.13 \text{ psf} * 10 \text{ ft} / 12 \text{ in/ft} = 6.78 \text{ lb/in}$$

Maximum positive moment near $H_1/3$ down from top of post is

$$M_{max} = \frac{9wH_1^2}{128} - \frac{9\Delta_{eave}EI}{8H_1^2} = \frac{9(6.78)(144)^2}{128} - \frac{9(0.655)(1.2 \times 10^6)(76.26)}{8(144)^2} = 6,633 \text{ in} \cdot \text{lb}$$

The maximum negative moment at the ground line is

$$M_{max} = \frac{wH_1^2}{8} + \frac{3\Delta_{eave}EI}{H_1^2} = \frac{6.78(144)^2}{8} + \frac{3(0.655)(1.2 \times 10^6)(76.26)}{144^2} = 26,246 \text{ in} \cdot \text{lb}$$

Post embedment

From ASAE EP486.1 Shallow Post Foundation Design:

$$d = 3 \sqrt{\frac{4.25M_g}{S'b}} = 3 \sqrt{\frac{4.25(26,246/12)}{200(0.648)}} = 4.2 \text{ ft}$$

How Conservative is the Simplified Lateral Design Approach?

Mill (2012) compared two options (fixed ground line support and pin-roller) of the simplified lateral design method with the more rigorous ANSI/ASAE EP484.2 (ASAE, 2003) method over a wide range of building aspect ratios, diaphragm/shear wall stiffnesses, and wall heights. Space limitations do not permit us to show the details here, but Table 3 summarizes the predic-

tions of unit shear, eave deflection and maximum post moment for the two methods divided by the predictions from ANSI/ASAE EP484.2. In other words, a ratio of 1.0 means perfect agreement between the simplified and EP484.2 methods, while a ratio of 1.25 means that the simplified method conservatively overpredicts the value by 25%.

Table 3. Comparison of unit shear, deflection and post moment to ANSI/ASAE EP484.2.

L:W	hw=16 ft					
	Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ_{eave} ratio	M ratio	v ratio	Δ_{eave} ratio	M ratio
1	0.97	1.05	1.17	1.02	1.10	1.00
2	1.02	1.08	1.19	1.07	1.14	1.02
3	1.09	1.18	1.25	1.15	1.24	1.08
4	1.18	1.31	1.34	1.24	1.37	1.18

As expected, the simplified method gives more conservative predictions of unit shear and post moment as the building length increases relative to its width. When the ground line is modeled as a pin (at 0.7 times the post embedment depth) and roller at the ground line, the predictions of post moment are closer to the EP484.2 method. For buildings with an aspect ratio up to 3, the conservative estimates of unit shear and post moment may still give economic designs with regard to post selection and embedment depth. Of course, a design professional can always apply a more rigorous design approach that accounts for diaphragm/frame interaction to gain some efficiency as justified.

Summary and Conclusions

There is a need to educate design and building regulatory professionals about lateral design of post-frame buildings. The objective of this article is to present a streamlined approach that is easier to learn than the more rigorous methods (that account for diaphragm/frame interaction) and will yield conservative designs. The method presented conservatively ignores the contribution of the frames in resisting lateral loads. For buildings with length-to-width ratios of 3 and less, predictions of unit shears and post moments are relatively close to those predicted with the more rigorous method, because of the fact that roof diaphragms are so much stiffer than the frames. Depending on the needs of the design professional, the added investment of time to learn the more rigorous methods may be justified.

Bender is professor of civil engineering and director of the Composite Materials and Engineering Center at Washington State University, Pullman, Wash. He taught courses and conducted research on wood engineering for 28 years. He serves on the American Wood Council's Wood Design Standards Committee responsible for the National Design Specification and Wood Frame Construction Manual, the American Lumber Standard Committee responsible for the PS 20 Softwood Lumber Standard and the project committee developing the new TPI-3 truss-bracing standard. He can be reached at bender@wsu.edu.

Mill is a former graduate student at Washington State University, Pullman, Wash. He is currently employed as a structural design engineer by KPFF Consulting Engineers in Seattle, Wash. He can be reached at drew.mill@kpff.com.

Note

1. Eave deflection can be calculated as the sum of the shear wall and roof diaphragm deflections. Equations for calculating deflections for conventional wood-frame diaphragms are given in the 2009 IBC (International Code Council, 2009) and in ANSI/AF&PA SDPWS-2008 (AF&PA, 2008). A modification of the diaphragm deflection equation for metal-clad post frame that includes load sharing of purlins and slip at purlin connections is given in this paper.

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