SIMPLIFIED LATERAL DESIGN OF POST-FRAME BUILDINGS – COMPARISON OF DESIGN METHODOLOGIES AND UNDERLYING ASSUMPTIONS

By

DREW PATRICK MILL

A thesis submitted in partial fulfillment of
the requirements for the degree of

Master of Science in Civil Engineering

WASHINGTON STATE UNIVERSITY
Department of Civil and Environmental Engineering

AUGUST 2012

To the Faculty of Washington State University:

The members of the Committee appointed to examine the thesis of DREW PATRICK MILL find it satisfactory and recommend that it be accepted.

Donald A. Bender, Ph.D., Chair

James D. Dolan, Ph.D.

Vikram Yadama, Ph.D.

ACKNOWLEDGEMENT

I would like to thank my advisor and committee chair Dr. Don Bender for his guidance and mentorship throughout the process of researching this topic and writing this thesis. I would also like to thank my committee members, Dr. Dan Dolan and Dr. Vikram Yadama for their support and agreement in being a part of my committee. Finally, a very special thanks to Mr. Brent Leatherman for his generosity in providing me with his design spreadsheet "NFBA Design Tool for Design of Post-Frame Building Systems", which played an integral role in the timely completion of this paper.

SIMPLIFIED LATERAL DESIGN OF POST-FRAME BUILDINGS – COMPARISON OF DESIGN METHODOLOGIES AND UNDERLYING ASSUMPTIONS

Abstract

by Drew Patrick Mill M.S.
Washington State University
August 2012

Chair: Donald A. Bender

As the application of post-frame buildings has increased, rigorous design methods have been developed to accurately model how these buildings perform under lateral loading. Such methods attempt to predict the force distribution interaction between the post-frames and roof diaphragm. This is a complex analysis that requires computer software that may not be necessary when designing all post-frame buildings. This paper describes a rational, simplified procedure for lateral design of post-frame buildings that conservatively ignores the contribution of frames to the lateral building stiffness, does not require costly computer software, and allows the designer to predict deflection, roof/wall shears, and maximum post bending moments. This simplified method was compared to what is considered the state of the art post-frame design methodology. Building wall heights of 12 and 16 ft, widths of 40 and 56 ft, and effective diaphragm shear moduli of 4.7 and 7.5 k/in. were examined for building aspect ratios ranging from 1:1 to 4:1. The simplified method gave conservative design values for unit shear, eave deflection, and maximum post moment compared to the more complicated

procedure that accounts for frame-diaphragm interaction, and proved it can be conservatively substituted for a simplified design of post-frame buildings.

TABLE OF CONTENTS

APPENDIX

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Post-frame construction is becoming increasingly popular due to its versatility, durability, constructability, and cost effectiveness. Once thought of as strictly agriculture buildings, post-frame buildings can be used in virtually any low-rise building application. With minimal wall/roof framing materials and footing/foundation materials, post-frame construction is generally less expensive than traditional light-frame wood construction. Figure 1 below shows a typical post-frame building.

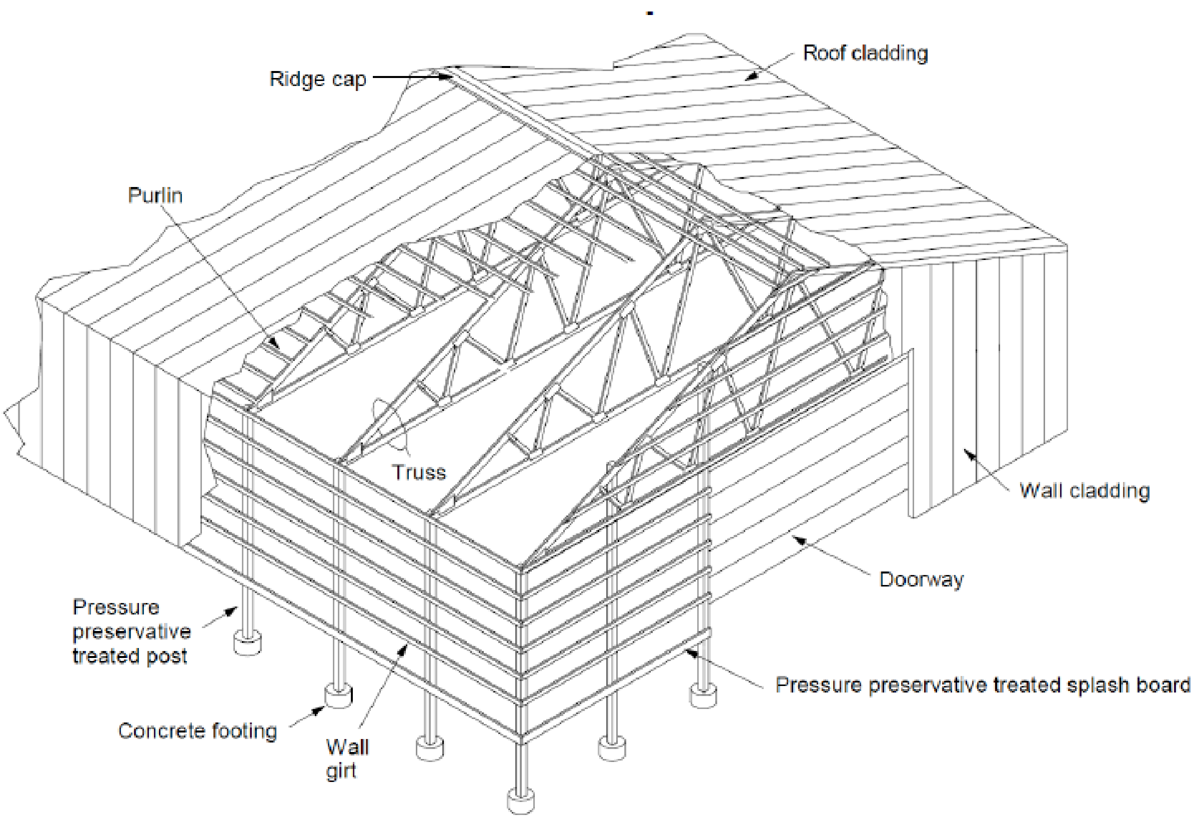


Figure 1. Typical post-frame building (Source: NFBA Design Manual)

The frames, when discussing post-frame buildings, are the assembly of two cantilever posts connected by a truss that spans the width of the building. It is assumed the truss is pin-connected at the top of the posts, and the frames are able to resist moment by their embedment in the ground. Wall girts and roof purlins are attached to the walls and roof, respectively. Upon which, metal cladding is attached to form the skin of the building. Under lateral loading, the roof assembly acts as a diaphragm that transmits load to the shear walls at the ends of the building.

Much of the structural efficiency of post-frame buildings is attributed to diaphragm action distributing lateral load to the shear walls of the buildings. Without including this effect of diaphragm action, post sizes and embedment depths would be increased significantly to effectively resist the applied lateral loads, thus making it important to consider when designing post-frame buildings. An experienced crew can erect the posts, trusses, purlins, and wall girts of a typical post-frame building in 2-3 days (NFBA.org). In addition, nearly all types of finishes/facades can be used in post-frame construction. Many structural engineers are not familiar with post-frame building design, and there is a need for a rational, simplified design methodology that can be reasonably implemented by design and building regulatory professionals.

ANSI/ASAE EP484.2 is a standard engineering practice promulgated by the American Society of Agricultural and Biological Engineers for diaphragm design in post-frame buildings. This standard is has a long learning curve due to complexity in learning and implementing the design procedure. The majority of this difficulty can be attributed to analyzing the interaction between the frames and roof diaphragm. The standard includes provisions to account for the force distribution between the building frames and the roof diaphragm. Because the roof diaphragm is generally orders of magnitude more stiff than the frames, the contribution of the interior frames when resisting lateral loading is minimal. For this reason, Bender et al. (1991) developed the "Rigid Roof" method for diaphragm design of post-frame buildings. This is a simplified approximate method for predicting the maximum roof shear in post-frame buildings. This method is based on the assumption that the roof diaphragm acts as an infinitely stiff, deep beam and transmits 100% of the lateral load to the shear walls of the building, with the interior frames not resisting any load. When this assumption is made, the maximum roof shear can be easily calculated based on wind pressures and building geometry. The rigid roof method is conservative with respect to diaphragm design because the "infinitely stiff" diaphragm assumption attracts more load to the diaphragm than if the diaphragm was considered to be flexible. With the maximum roof shear known, the maximum moment that occurs in the diaphragm can be calculated and the maximum axial chord forces can be found.

Maximum eave deflection will occur at the mid-length of a symmetric building, so this is usually the critical post with respect to member design and required embedment depth. Post moments can be approximated by superimposing the moments of propped-cantilever and cantilever beam analogs. The propped cantilever model represents the top of the post being supported by an infinitely stiff roof diaphragm. This is a reasonable assumption for the design of the diaphragm, however, this analog doesn't account for the additional bending moment that would result from the eave deflection. (Pope et al., 2012) outlined a procedure for calculating post-frame eave deflection under lateral loading using a variation of the deflection equation given in the ANSI/AF&PA-2008 Special Design Provisions for Wind and Seismic standard. Superposition of the forces and moments from the propped-cantilever and cantilever structural analogs provides the information needed to design the post and post foundation.

The ASAE EP484.2 method is a complex procedure geared toward determining the force distribution between the diaphragm and post-frames. Doing so can require computer software to determine post stiffness, applied eave loads, and to calculate the interaction of the diaphragm and frames due to the assumption that the diaphragm is flexible. This procedure is computationally intensive and requires a significant time investment to learn. The simplified method presents a rational approach to lateral design of post-frame building that eliminates the need to calculate force distribution between the frames and diaphragm. This greatly simplifies the design procedure and can be easily understood and implemented by design and building regulatory professionals not familiar with the design of post-frame buildings. A simplified design procedure would result in cost savings due to less time required to learn and implement designs. Additionally, a simplified design procedure would result in greater market acceptance and face validity for post-frame buildings. A comparison between the different methods is required to show that the simplified method can be conservatively substituted in place of ASAE EP484.2.

OBJECTIVES

The objectives of this paper are to present a simplified lateral design approach for post-frame buildings, and to compare the unit shears for the diaphragm and shear walls, deflections, and post moments with those from more complex lateral design approaches that account for diaphragm-frame interactions.

MODEL DEVELOPMENT

In this study the ASAE EP484.2, or standard method, is used as the benchmark for the comparison of lateral design of post-frame buildings. This standard requires the designer to determine the stiffness of the frames and diaphragm in order to predict the relative force distribution between the two elements. The simplified method presented eliminates the need for determining this frame/diaphragm interaction by conservatively ignoring the contribution of frame stiffness, and assuming the entire lateral load is resisted by the diaphragm. Both methods allow the designer to predict maximum unit shears in the diaphragms and shear walls, eave deflection, and post moment.

ANSI/ASAE EP484.2 Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings

The ANSI/ASAE EP484.2 standard provides a step-by-step approach for the design of metal-clad wood-frame rectangular buildings. Frame and diaphragm stiffness are calculated, along with the applied eave load, in order to predict the force distribution between the frames and diaphragm. The posts, diaphragm, and shear walls are then designed appropriately. The ANSI/ASAE EP484.2 is also referenced in Chapter 23 of the 2012 IBC.

Step 1: Determine diaphragm roof stiffness, C_h.

The total horizontal diaphragm shear stiffness, C_h, is the sum of the horizontal shear stiffness of the individual roof and ceiling diaphragms. The horizontal shear stiffness of an individual diaphragm, $C_{h,i}$, (for width, s) can be calculated from EP 484.2 Eq. 2 or Eq. 3 as follows:

$$C_{h,i} = C_{p,i} (\cos^2\Theta_i) \qquad \text{[EP 484.2 Eq. 2]}$$

$C_{h,i} = G (\cos\Theta_i) (b_{h,i} / s)$ [EP 484.2 Eq. 3]

where $b_{h,i}$ is the horizontal span of the diaphragm. In most circumstances the individual diaphragm segments have the same shear stiffness. The total horizontal diaphragm shear stiffness represents the shear stiffness that resists lateral loads from diaphragm action.

Step 2: Calculating frame/end-wall stiffness, k/k_{sw}.

Post stiffness is defined as the ratio of horizontal load to horizontal deflection at the eave. When the trusses are assumed to be pin-connected at the posts, and each post is assumed to be fixed at the base, the frame stiffness, k, is simply that of a cantilever beam and can be calculated by:

$$k = \frac{3EI}{{h_w}^3}$$

Where E = post MOE (psi), I = post moment of inertia (in^4), and h_w = post height (in). A typical post-frame consists of two posts connected by a spanning truss, so the post stiffness is multiplied by two. The overall post-frame stiffness then becomes:

$$k = \frac{6EI}{{h_w}^3}$$

Frame stiffness can also be calculated using a plane-frame structural analysis program in which the member properties and fixity are modeled in the structural analog. A point load, P, of an arbitrary magnitude is then applied at the eave of the frame. The corresponding frame stiffness can then be calculated by $k=P/\Delta$, where Δ is the resulting horizontal frame deflection.

The shear walls of the building are clad with metal sheathing and transmit load carried by the diaphragm into the post foundation. The shear wall stiffness is typically orders of magnitude more stiff that that of the post-frames due to the metal cladding. This stiffness is most accurately obtained from full-scale building tests, or from tests of equivalent assemblies and is calculated by: $k_{sw} = G_a \frac{W}{h_w}$, where G_a is the apparent shear wall shear stiffness and h_w, W are the shear wall height and width, respectively.

Step 3: Determine eave load, R.

In post-frame buildings lateral design loads (usually wind) acting on the projection of the building are replaced by concentrated point loads that act at the eave of each post-frame. Eave loads can be determined by using a plane-frame structural analysis program such as SAP or Visual Analysis. In this procedure, all building loads are applied as line loads to a single post-frame analog for the building under consideration. The post/truss properties and member fixity are input to represent the actual post-frame assembly. A roller support is placed at the eave opposite from where the eave load is applied in order to restrain all lateral movement of the frame. After the analysis is complete, the horizontal reaction acting at the roller is determined to be the applied eave load.

Another method to determine eave loads is by using a frame base fixity factor, f. This factor is dependent on how the post embedment is modeled, and represents the amount of load that is transferred to the top of the post and then resisted by the diaphragm. The remainder of the applied load is transferred into the post foundation. For a post that is considered to be perfectly fixed at the base and pinned at the top, f = 3/8, whereas a post pinned at *both* ends would have f = 1/2. In reality, however, neither of these conditions are perfect analogs of an embedded post condition. Figure 5.6 on Page 5-5 of the NFBA Post-Frame Building Design Manual show other structural analogs typically used to model post embedment, each resulting in different post fixity. For symmetrical base restraint conditions, the eave load can then be calculated by:

$$R = s[h_r (q_{wr} - q_{lr}) + h_w f (q_{ww} - q_{lw})] \qquad \text{[EP 484.2 Eq. 6]}$$

Where q_{wr},q_{lr},q_{ww},q_{lw} are windward and leeward roof/wall pressures, respectively, h_r and h_w are roof and wall heights, respectively, and s is the post spacing.

Step 4: Load distribution.

When a lateral load acts on a post-frame building, the eave load, as previously defined, is the total sidesway load resisted by the diaphragm and the post-frames. Because these elements have different stiffness, each will resist a different amount of the applied lateral load. To determine this interaction, EP484.2 tabulated shear force modifiers (mS, Table 1) and sidesway restraining force factors (mD, Table 2), which are used to predict the maximum total diaphragm shear force, V_h, and the sidesway restraining force, Q. In order to present a reasonable number of tables, mS and mD values are tabulated for: symmetric buildings with a shear wall at each end, and constant values of diaphragm, frame, shear wall stiffness, and eave loads throughout the building. The inputs for both tables are the ratios of shear wall stiffness to frame stiffness (k_{sw}/k), diaphragm stiffness to frame stiffness (C_h/k), and the total number of frames in the building, including the shear walls.

The maximum diaphragm shear, V_h, is the maximum shear force in the diaphragm that occurs in the diaphragm segments adjacent to the building shear walls. This is calculated by multiplying the appropriate mS value by the eave load, R, and is represented by: V_h = mS(R).

The sidesway restraining force, Q, represents the force from the roof diaphragm that helps to resist the applied lateral load, otherwise known as diaphragm action. Because the maximum deflection will occur at the mid-span of a symmetric building, the highest loaded frame occurs closest to the building mid-span. Q is calculated by multiplying the appropriate mD value by the eave load, R, and is represented by: Q = mD(R). If the amount of load resisted by a particular frame is known, Q can also be calculated by subtracting that load resisted by the frame from the eave load, R. In this paper a computer program, DAFI (Diaphragm and Frame Interaction) was used, which calculated the individual frame forces. The sideway restraining force was then calculated by: Q = R – (load resisted by frame).

It is not always the case that each frame, diaphragm, and shear wall will have the same stiffness. Additionally, as post spacing and building symmetry vary, the eave loads at each frame will differ. If this occurs, the program DAFI can be used to solve for the forces in each frame/diaphragm element, as well as the eave deflection of each frame. DAFI essentially solves equations of equilibrium that relate the applied eave loads to the stiffness of each frame and diaphragm element (Bohnhoff, 1992). DAFI gives the same results when comparing values obtained from using the mS and mD tables of ASAE EP484.2. In order to calculate more accurate results that don't require interpolation between given ratios of the EP484.2 tables, DAFI was used in this paper to determine the maximum shear/diaphragm and frame forces, as well as frame deflections.

DAFI is a useful tool in determining diaphragm/frame forces and deflections when variations in building geometry and member properties exist. One aspect of DAFI, however, that may be considered inaccurate is the formulation of eave deflections. The ANSI/AF&PA SDPWS – 2008, a code-referenced design specification, presents a three-term equation to calculate the deflection of wood-sheathed diaphragms, which is given in Eq. C4.2.2-2. The three terms in this equation account for deflection due to diaphragm framing bending, shear deformation, and chord splice slip, respectively. The deflection output by DAFI is merely a function the applied eave load and the stiffness of the post-frames and adjacent diaphragms. Each diaphragm and frame is assigned a single value of stiffness. When entering a single value of stiffness in DAFI, it is difficult to account for contribution of deflection from framing bending or chord splice slip separately. Therefore, the DAFI deflection values calculated should be an under-prediction of the actual post deflections that may occur because the diaphragm stiffness entered is a function of only the effective shear modulus of the diaphragm.

Step 5: Post/member design.

After all loads are properly distributed, frames should be designed for combined bending and axial compression using the design loads and sideway restraining forces applied to the post-frame. Such an analysis can be completed by any method of frame analysis and by following the ANSI/AF&PA NDS National Design Specification for Wood Construction. In this paper the frame analysis was complete using Visual Analysis software. The windward and leeward wind loads were applied to the frame, and the calculated sideway restraining force, Q, was applied at the eave opposite of the windward wall to account for the restraining force due to diaphragm action. Visual Analysis calculated the maximum post moment, which in most cases occurs at the ground line.

In this paper the more complex options of the ANSI/ASAE EP484.2 method will be used to compare results to the proposed simplified method. DAFI was used to determine the load sharing interaction between the post-frames and roof diaphragm. An example of this procedure can be seen in the Model Validation section, with a complete example problem presented in Appendix C.

Summary: "Standard Method"

The above procedures outlined in the standard method yield accurate results, however, this method of post-frame design is complex, computationally intensive, requires computer software, and has a long learning curve associated with the design procedures. As a result, this method may require a significant investment of time for someone that is new to designing post-frame buildings. A flow chart showing the required steps for the standard method is shown in Figure 2 below.

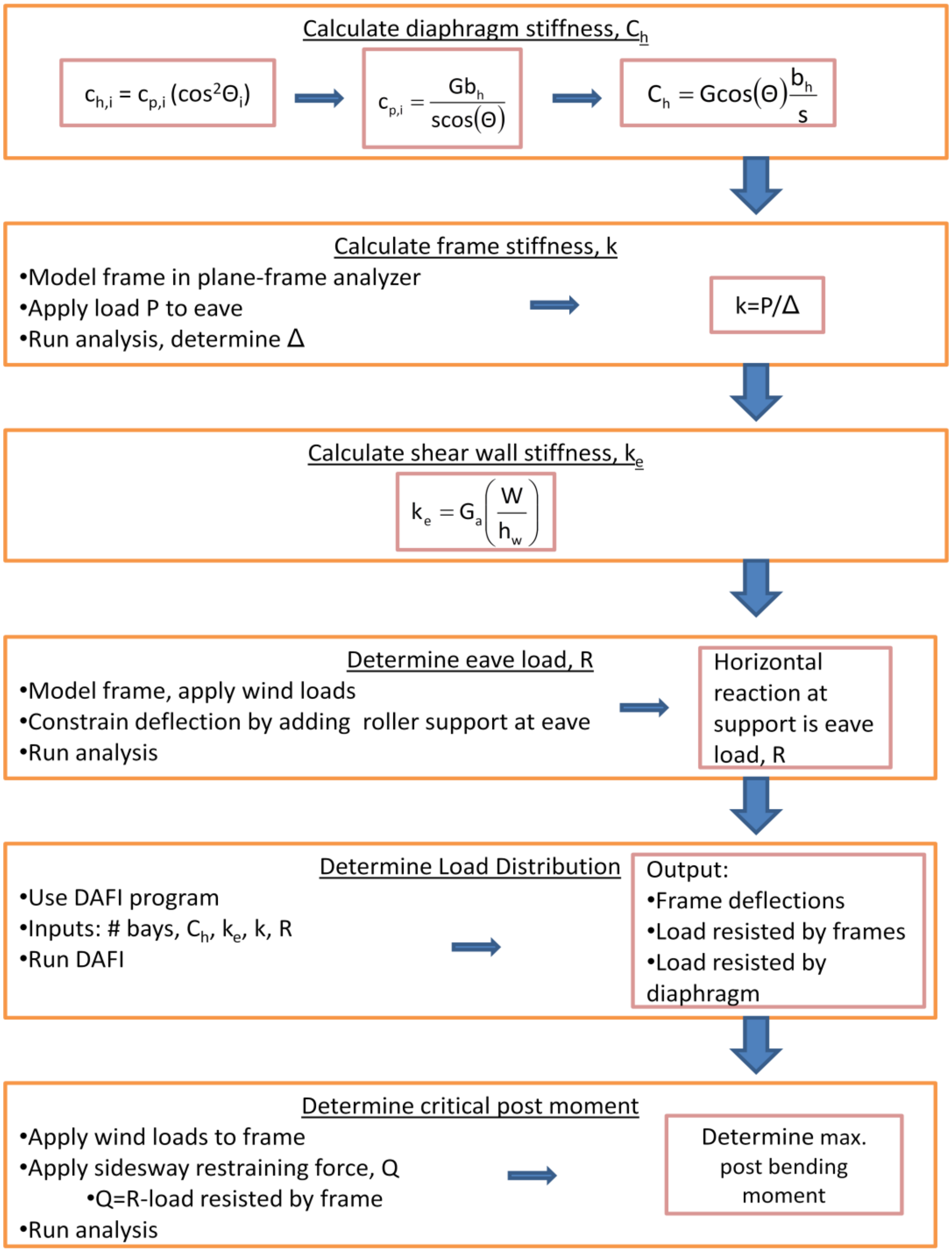


Figure 2. Flow chart for standard method.

Simplified Method

The simplified method considers the roof diaphragm to act as an infinitely stiff beam, therefore allowing the designer to bypass complex calculations of frame/diaphragm stiffness and the force distribution between them. The unit shears and post moment can then be calculated based on building geometry and post base fixity alone. The only value of stiffness utilized in this method is the effective shear modulus, G, in order to calculate mid-span eave deflection, as shown in Step 3 below. Applications of the simplified method are limited by allowable building/diaphragm aspect ratios. Additionally, the simplified method is limited to symmetric buildings with consistent values of frame and diaphragm stiffness throughout.

Step 1: Calculate unit shear.

The simplified method follows the rigid roof design assumption that treats the diaphragm as an infinitely stiff, deep beam (Bender et al., 1991). Additionally, a fixed condition at the base is assumed to model the embedded posts. Each post can then be modeled as a propped-cantilever with a uniformly distributed line load as shown in Figure 3 below. Because the diaphragm is assumed to be infinitely stiff, it isn't necessary to calculate diaphragm or frame stiffness. From this assumed analog, 3/8 of the applied load is resisted by the diaphragm, and the remaining 5/8 is transferred into the foundation.

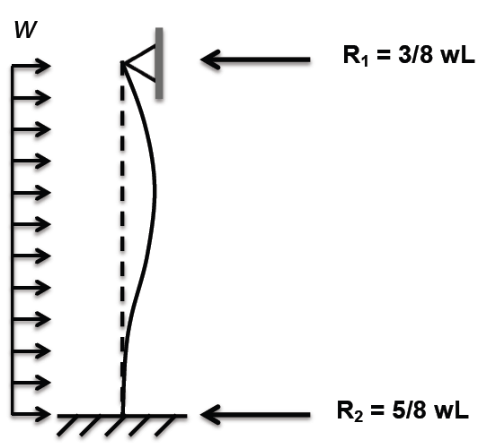


Figure 3. Propped-cantilever analog.

By essentially combining Equations 6 and 8 of EP 484.2, a simplified equation for calculating unit shear can be calculated by:

$$v = \frac{f(q_{ww} - q_{lw})h_w L + (q_{wr} - q_{lr})h_r L}{2W} \qquad \text{[Eq. 1]}$$

where f, q_{ww}, q_{lw}, q_{wr}, q_{lr}, h_r, and h_w are as defined in Step 3 of the standard method, and L, W are the building length and width, respectively. The unit shear is a result of the diaphragm resisting the entire applied eave load, which is transferred through the diaphragm to the building shear walls. This represents the maximum unit shear in the diaphragm as well as the unit shear that acts at the top of the shear walls.

Step 2. Calculation of post moments.

The propped-cantilever analog is a conservative assumption with respect to diaphragm design, however, the pin support at the top does not allow for post deflection that will occur under lateral loading. In order to accurately calculate the maximum post moment, moment that occurs in a cantilever beam with an applied load at the end is superimposed on that of the propped-cantilever. This can be seen in Figures 4 and 5 below.

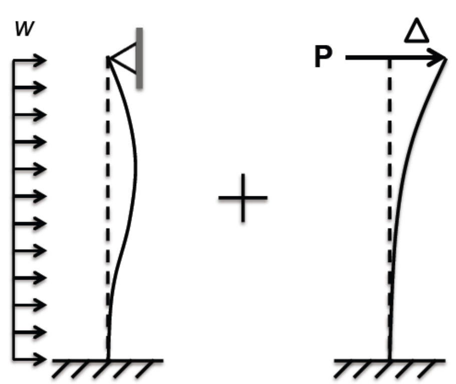


Figure 4. Superposition of Analogs

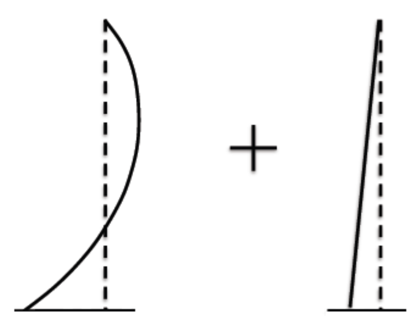


Figure 5. Superposition of moments.

The maximum moment for both analogs is typically a negative moment that occurs at the ground line. Summing these two values gives the maximum moment seen by the post, which governs embedment depth and post design. The equation for maximum moment of a propped-cantilever can easily be found in beam tables and is calculated by:

$$M_{max} = \frac{wh_w^{\ 2}}{8} \qquad \text{[Eq. 2]}$$

The moment for a cantilever beam can be calculated by M = Ph_w. However, the exact applied point load is not known, so we must derive it from a known eave deflection. The maximum deflection of a cantilever beam can be calculated by:

$$\Delta_{max} = \frac{Ph_w^{\ 3}}{3EI} \qquad \text{[Eq. 3, (Kassimali)]}$$

Solving for P in Eq. 3 results in:

$$P = \frac{3EI\Delta}{h_w^{\ 3}} \qquad \text{[Eq. 4]}$$

Since M = Ph_w, P = M/h_w. Substituting this into Eq. 4 and multiplying each side by h_w results in an equation to calculate moment in a cantilever beam based on deflection:

$$M_{max} = \frac{3EI\Delta_{eave}}{h_w^{\ 2}} \qquad \text{[Eq. 5]}$$

The ground line moment is then calculated by summing equations 2 and 5:

$$M_{max}^{(-)} = \frac{wh_w^{\ 2}}{8} + \frac{3EI\Delta_{eave}}{h_w^{\ 2}} \qquad \text{[Eq. 6]}$$

The ground line moment always governs the post embedment depth; however, in certain situations it is possible for a positive moment to govern post design. The propped-cantilever analog results in a maximum positive moment, which occurs approximately 3/8 of the post length from the top of the post. This equation for positive moment can be found in beam tables and is presented as:

$$M_{max}^{(+)} = \frac{9wh_w^2}{128} \qquad \text{[Eq. 7]}$$

The net positive moment would then be the moment determined by Eq. 7 plus the negative moment from the cantilever analog calculated at 3/8 of the post length from the top of the post. This can be calculated by multiplying Eq. 4 by 3/8, which results in:

$$M^{(-)} = \frac{9EI\Delta_{eave}}{8h_w^{\ 2}} \qquad \text{[Eq. 8]}$$

The maximum positive moment in the post is then calculated by summing Equations 7 and 8.

$$M_{max}^{(+)} = \frac{9wh_w^{\ 2}}{128} - \frac{9EI\Delta_{eave}}{8h_w^{\ 2}} \qquad \text{[Eq. 9]}$$

An alternate form to post moment calculation is presented on Page 9-7 of the NFBA Post-Frame Building Design Manual (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 Equations 6 and 9 above.

Step 3. Calculating post/diaphragm deflection.

Step 2 showed that obtaining the diaphragm/post deflection is crucial to calculating the maximum post moment. Pope et al. (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:

$$\delta_{dia} = \frac{15vL^3}{4EAs\,n(n+1)} + \frac{.25vL}{1000G} + \frac{\Delta_c}{W}\sum x_i$$ [Eq. 10]

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

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{.25vL}{1000G} + \frac{\sum(x_s\Delta_c)}{2W}$$ [Eq. 11]

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 outer-most framing member act as chords to resist the applied lateral load. In other words, the moment of inertia of the interior framing members is ignored (Pope et al., 2012). Pope's equation for the deflection of metal-sheathed diaphragms, however, does consider the effect on moment of inertia from the interior framing members (roof purlins) because load sharing occurs between the center and outer purlins.

The third term, which accounts for chord slip, varies mainly from differences in calculating the cumulative distance from chord splices to the end walls due to variations in construction patterns of purlin splices 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).

In order to make a fair comparison between the standard method and this proposed simplified method, it is important to use a consistent procedure for calculating frame/diaphragm deflection. Unfortunately, the deflection values that DAFI produces don't account for contribution of deflection from framing bending or chord splice slip. For this reason, for the purpose of accurately comparing the two methods, only the deflection due to shear deformation will be considered when calculating the diaphragm deflection for the simplified method. Additionally, DAFI considers the frame to resist some of the applied loads, whereas the diaphragm deflection equation doesn't consider any load resistance by the frame. For this reason, the deflection values given by DAFI should be less than those predicted by the diaphragm deflection equation.

The diaphragm deflection equation assumes the end of the diaphragms to be fixed, meaning no shear wall deflection will occur. In reality, however, the shear walls will deflect some small amount when loaded with the unit shear, v, from the diaphragm calculated in Step 1 above. The total eave deflection, Δ_{eave}, will then be the deflection of the shear wall added to the calculated diaphragm deflection. The shear wall deflection can be calculated by Equation C.4.3.2 – 2 (ANSI/AF&PA SDSWS – 2008):

$$\frac{8vh_w^{\ 3}}{EAb} + \frac{vh_w}{1000G_a} + \frac{h_w}{b}\Delta_a \qquad \text{[Eq. 12]}$$

where v is the applied unit shear (lb/ft), h_w is the height of the shear wall (ft), E is the modulus of elasticity of the end posts (psi), A is the cross-sectional area of the end wall posts (in^2), b is the shear wall length (ft), and G_a is the apparent shear wall stiffness (k/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.

When entering end-wall stiffness values into DAFI, the program only considers a single value of end-wall stiffness that is based on the apparent shear wall stiffness, G_a. Fortunately, the testing of metal-clad shear walls used to obtain the apparent shear wall stiffness in this paper did not separate the deflection caused by bending from deflection caused by shear formation. In other words, the value of G_a used to calculate shear wall deflection accounts for the first and second terms of SDPWS Eq. C.4.3.2 – 2, as the bending effects are represented within the G_a value, along with shear. Additionally, the contribution of shear wall deflection from bending is small when compared to that of deflection due to shear. For this reason, and for the sake of a consistent comparison between the standard method and the simplified method, shear wall deflection is calculated using the single shear term with the corresponding value of G_a, making it easy to input the shear wall stiffness in DAFI.

The total deflection then, represents the "Δ_{eave}" term in equations 6 and 9 above to calculate the maximum post moment. The maximum frame deflection given by the DAFI output can then be compared to the total deflection that is comprised of the shear term of the shear wall deflection equation, and the shear term of the diaphragm deflection equation. As previously mentioned, the deflections given by DAFI should be lower than those calculated using the deflection equations.

Summary: "Simplified Method."

The simplified method provides a quick and simple way to calculate the maximum shear wall and diaphragm forces, posts deflections, and in turn post bending moments. This method does not require computer-based plane frame analysis software, and provides the designer with an easy way to design post frame buildings without delving into the complicated interaction of load sharing between the roof diaphragm and posts.

<u>Simplified Method with Different Ground Line Support Condition</u>

The fixed ground line support condition may lead to overly conservative estimates of the ground line moment. As an alternative, a more realistic base constraint is to assume a roller support at the ground level, and a pin support located a distance of 0.7(d) from the ground line, where d is the post embedment depth. This base analog is shown in Figure 5.6 (b) of the NFBA Post-Frame Building Design Manual. The roller support at the ground line is a result of assuming a rigid concrete floor slab is poured within the building, providing a horizontal reaction force against the post under lateral loading. Such a model is given by Skaggs et al. (1993), and is shown in Figure 6 below. For the rest of this paper this will be referred to as the Simplified – Pin/Roller Method.

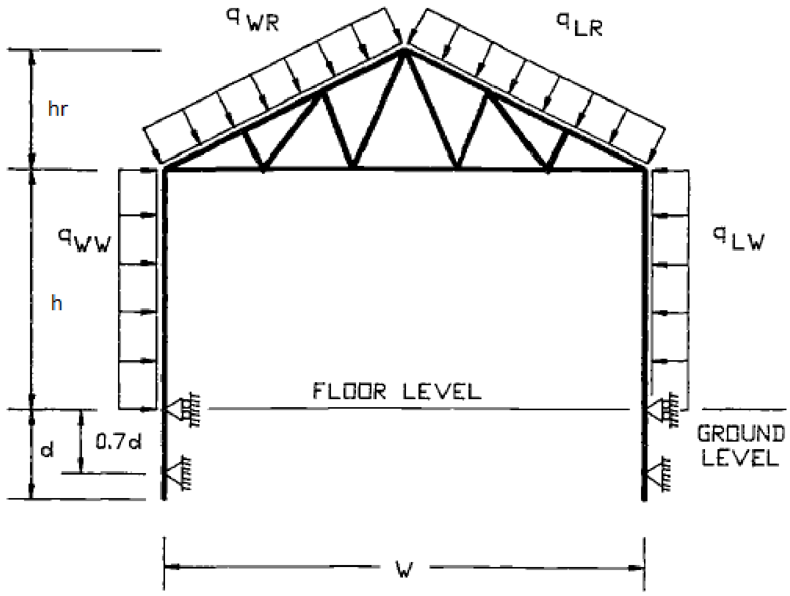


Figure 6. Analog for pin/roller embedded post-frame (Skaggs et al., 1993).

With the pin/roller ground line condition, the maximum post moment can be determined following the same procedure as Step 3 above, only the reaction forces will change due to the different analog assumed. Superposition of the post moment is done by the same procedure, just by using slightly different equations. Similar to Figures 4 and 5, this superposition is shown in Figures 7 and 8 below.

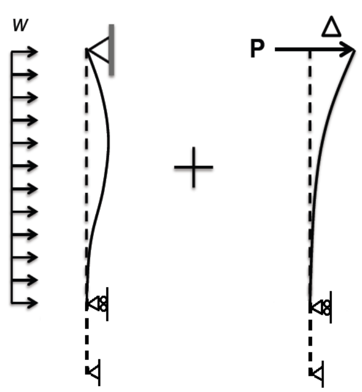


Figure 7. Superposition of Analogs

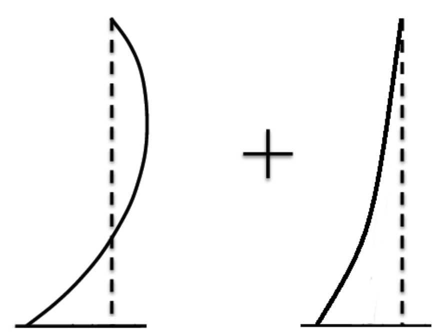


Figure 8. Superposition of moments.

For the propped cantilever, the derivation of the ground line moment was done using the slope-deflection method of structural analysis as shown in Appendix A. The ground line moment can be calculated by:

$$M_{max}^{(-)} = \frac{3EI\left[\dfrac{\dfrac{-wh_w^{\,2}}{8}}{\dfrac{3EI}{h_w} + \dfrac{3EI}{0.7(d)}}\right]}{0.7(d)} \qquad \text{[Eq. 13]}$$

The equation for deflection of a cantilever beam with a roller-pin base fixity as shown in Figure 5 above is found in beam tables and calculated by:

$$\Delta_{max} = \frac{Ph_w^{\,2}}{3EI}\left(h_w + 0.7(d)\right) \qquad \text{[Eq. 14]}$$

Again, solving Eq. 14 for P, substituting in M/h_w, and substituting like terms, the ground line moment can be found by Eq. 15 below. The complete derivation of this is also shown Appendix A.

$$M_{max}^{(-)} = \frac{3EI\Delta_{eave}}{h_w\left[\left(0.7(d)\right) + h_w\right]} \qquad \text{[Eq. 15]}$$

The Δ term is calculated in the same way as before, but the unit shear will be slightly different due to the different base condition attracting more load to the diaphragm. To account for this difference, (Skaggs, et al., 1993) gives the equation:

$$R = s\left[\frac{h_w\left(q_{ww} - q_{lw}\right)\left(2.8d + 3h_w\right)}{8\left(0.7d + h_w\right)} + h_r\left(q_{wr} - q_{lr}\right)\right] \qquad \text{[Eq. 16]}$$

This calculates the eave load, R, for the pin/roller base condition, assuming the trusses are pinned at the posts and the roof diaphragm is completely rigid. Eq. 16 can then be modified to represent the unit shear that acts on the diaphragms adjacent to the shear walls. The resulting equation is:

$$v = \frac{L\left[\frac{h_w(q_{ww} - q_{lw})(2.8d + 3h_w)}{8(0.7d + h_w)} + h_r(q_{wr} - q_{lr})\right]}{2W}$$ [Eq. 17]

This unit shear can then be used to calculate the diaphragm and shear wall deflection that will occur under this new assumption of base fixity. With the eave deflection known, the maximum post bending moment can be calculated by summing Equations 13 and 15. Under this base fixity assumption the ground line moment will be reduced compared to that of a perfectly fixed base condition. In many circumstances this analog may provide a more accurate estimate of the post moments. A flow chart showing the required steps for the simplified method is shown in Figure 9 below.

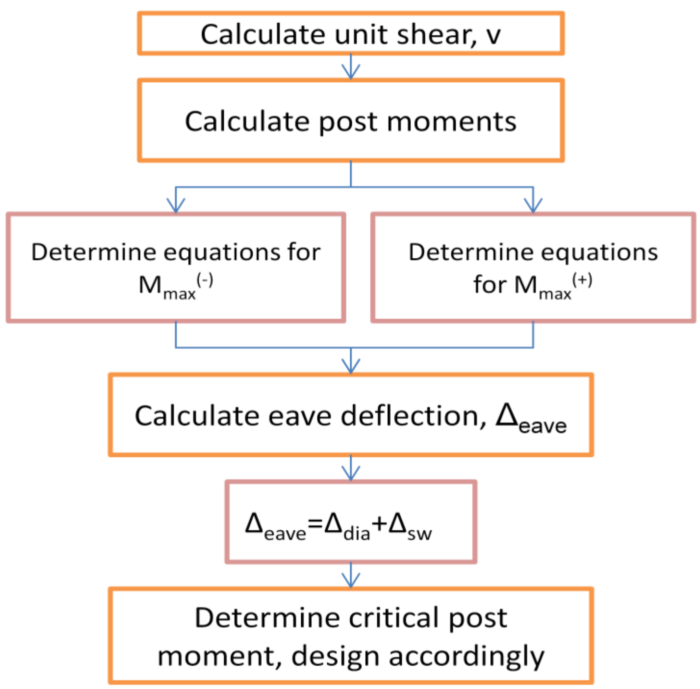


Figure 9. Flow chart for simplified method.

MODEL VALIDATION

To adequately validate the proposed model, the ANSI/ASAE EP 484.2 standard was used to compare results to the proposed simplified design procedures. The EP 484.2 represents the most accurate procedure for designing post-frame building systems. The following example problem was conducted for a building length:width ratio of 2:1, a common aspect ratio for post-frame buildings.

<u>Example Building Dimensions:</u>

Building Length:	L = 112 ft
Building Width:	W = 56 ft
Post/Wall Height:	h_w = 16 ft
Post Spacing:	s = 8 ft
Frame Base Fixity Factor	$f = \frac{3}{8}$
Roof Pitch:	3.5:12
Roof Angle:	$\Theta = \tan^{-1}\left(\frac{3.5}{12}\right) = 16.26^{o}$
Roof Overhang	2 ft
Roof Height	$h_r = \frac{W}{2}\left(\frac{3.5}{12}\right) = 8.17$ ft

<u>Post Properties:</u>

Post Width:	b = 4.31 in
Post Depth:	d = 7.19 in

Post MOE: $E = 1{,}700{,}000$ psi

Post MOI: $I = \frac{bd^3}{12} = 133.5$ in

ANSI/ASAE EP484.2 (Standard) Method

Wind Pressures:

Wind pressures were calculated based on the procedures outlined in Section 6.5 of ASCE 7-05. These wind pressures were used to evaluate the standard method as well as the proposed simplified methods. Based on the calculated wind pressures, the windward roof pressure was a negative pressure (acting away from the roof) and was higher than the negative leeward roof pressure. For this reason, higher unit shears were calculated if the roof pressures were not considered. Therefore, to achieve the highest design values, only the windward and leeward wall pressures are assumed to act on the building. These wind pressures assumed a building with no interior pressure.

$q_{ww}=5.5$ psf

$q_{wr}=0$ psf

$q_{lr}=0$ psf

$q_{lw}=-4.4$ psf

The DAFI program requires five inputs: number of bays in the building, diaphragm shear stiffness, shear wall stiffness, interior frame stiffness, and eave load on the interior frames.

The number of building bays is calculated by L/s. In this example, $\frac{L}{s} = 14$

The diaphragm shear stiffness, C_h, is calculated by: $G\left[\cos(\Theta)\frac{\left(\frac{W}{2}+\text{overhang}\right)}{s}\right]2$

The panel construction assumed for this example problem is Test Assembly Number 9 from Table 6.1, Steel-Clad Roof Diaphragm Assembly Test Data on Page 6-8 of the NFBA Design Manual. This test assembly lists an effective shear modulus, G, of 4.7 k/in. Multiplying by 2 accounts for the diaphragm stiffness of both halves of the roof assembly. The above equation results in a total diaphragm shear stiffness of 33,840 lb/in.

The shear wall shear stiffness, k_{sw}, is calculated by: $G_a(1000)\left(\frac{W}{h_w}\right)$

G_a, the apparent shear wall shear stiffness, was determined to be 19.3 k/in. This value was obtained by evaluating data obtained by Ross et al. (2009) in a study that evaluated the strength and stiffness of post-frame shear walls with wood plastic composite skirtboards. The 19.3 k/in. represents the average shear stiffness of the 12 different shear wall configurations that were tested. The resulting shear wall shear stiffness was 67,550 lb/in.

The interior frame stiffness was determined using Visual Analysis software. The post/truss properties were defined in the model and the proper fixities/supports were assigned. In this model it was assumed that the truss-to-post connection is pinned. The embedded post analog follows that of Figure b on Page 5-5 of the NFBA Design Manual. A point load of 100 lb was applied to the eave of the frame. The corresponding horizontal deflection was found to be 0.612 in. These models are shown in Figures 10 and 11 below. The total frame stiffness was calculated as P/Δ = 163.4 lb/in.

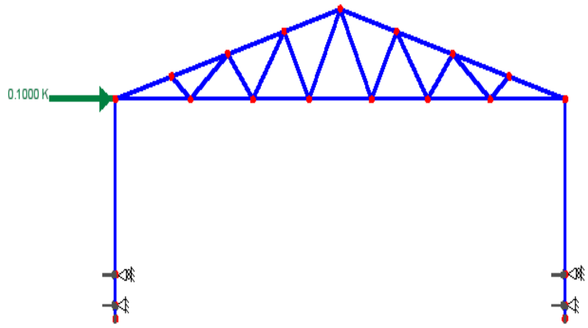


Figure 10. Load applied at eave.

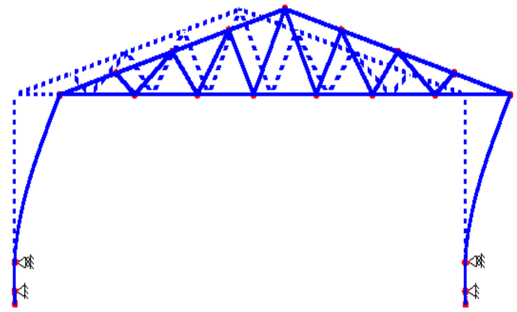


Figure 11. Corresponding eave deflection.

To calculate the eave load acting on an interior frame, the windward and leeward wind pressures were converted to line loads based on the tributary width of the interior frames, s=8 ft. w_{ww} = 5.5 psf x 8ft = 44 lb/ft. w_{lw} = -4.4 psf x 8 ft. = 35.2 lb/ft. These loads were then applied to the windward and leeward walls, respectively. The leeward eave was then constrained from horizontal movement, i.e a roller support was assigned to the eave. This is shown in Figures 12 and 13 below. The analysis is run and the resulting reaction force at the support represents the horizontal eave load, R, applied to each interior frame. From this analysis it was determined that the horizontal eave load is 498 lb.

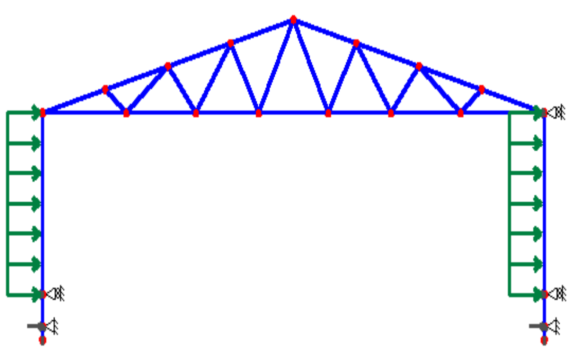


Figure 12. Applied horizontal wind loads.

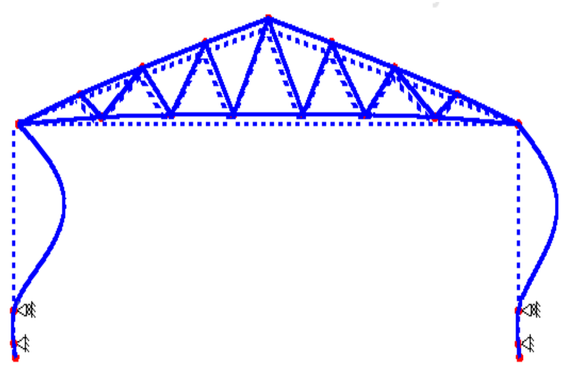


Figure 13. Deflected shape with eave constrained.

These five variables were input to DAFI, and the following data was produced:

Frame #	Frame Stiffness (lb/in)	Eave Load (lb)	Frame Deflection (in)	Load Resisted (lb)
1	67550	249	0.047	3189.92
2	163.4	498	0.134	21.92
3	163.4	498	0.207	33.82
4	163.4	498	0.266	43.48
5	163.4	498	0.312	50.94
6	163.4	498	0.344	56.25
7	163.4	498	0.364	59.43
8	163.4	498	0.370	60.48
9	163.4	498	0.364	59.43
10	163.4	498	0.344	56.25
11	163.4	498	0.312	50.94
12	163.4	498	0.266	43.48
13	163.4	498	0.207	33.82
14	163.4	498	0.134	21.92
15	67550	249	0.047	3189.92

Figure 14. Example Problem DAFI Output

From this data the maximum unit shear that occurs in the diaphragm/adjacent shear wall is calculated by taking the maximum load resisted by a single frame and dividing by the width of the building, W. The maximum force occurs in the shear walls and in this case is 3190 lb (see Figure 14 above). The resulting maximum unit shear is $v = \frac{3190}{W}$ =56.96 lb/ft.

The maximum frame deflection occurs in the center of the building, which in this case is Frame 8. This deflection can be seen in the above DAFI output and is 0.370 in.

The maximum post moment can then be found by applying the calculated wind loads to the Visual Analysis model without the leeward eave being constrained. In order to account for diaphragm action, which represents the force from the diaphragm that helps to resist lateral loads, the sidesway restraining force, Q, is applied to the model. Q is found by subtracting the load resisted by the frame from the eave load applied to the frame. To design the posts conservatively, the center frame is considered because this frame experiences the highest deflection and has the smallest contribution of diaphragm action. The resulting sideway restraining force is 498-60.48 = 437.52 lb. This load is applied as a point load on the leeward eave that acts in the opposite direction of the wind loads. This model is shown in Figure 15 below. The maximum post moment occurs in the windward post and is found after completing the analysis of the model. The maximum post moment for this example was found to be -20,364 lb-in.

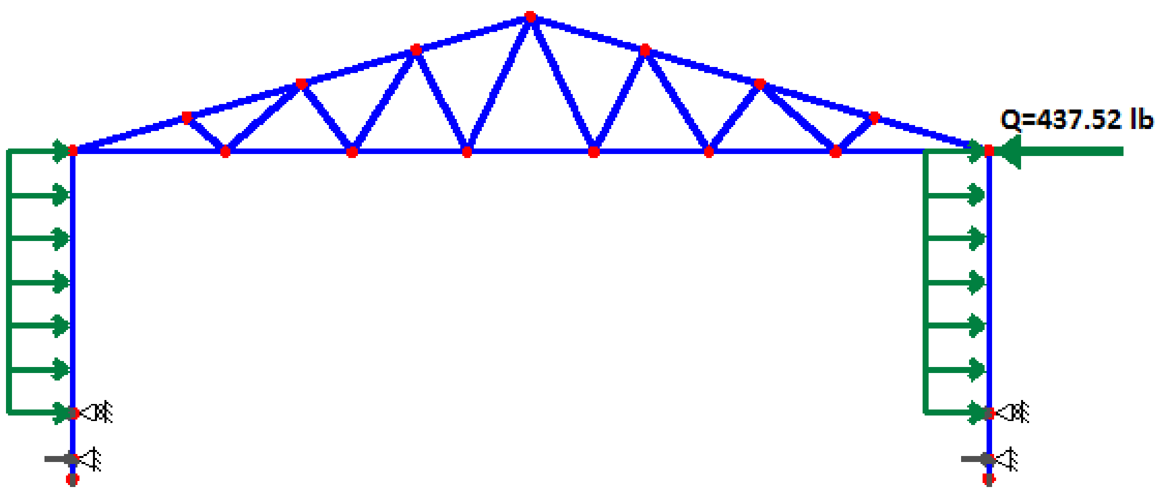


Figure 15. Visual Analysis model with applied wind loads and diaphragm force, Q.

<u>Simplified – Fixed Method:</u>

The maximum unit shear is calculated by: $v = \frac{f(q_{ww} - q_{lw})h_w L + (q_{wr} - q_{lr})h_r L}{2W}$

When f is assumed to be 3/8, the resulting unit shear is 59.4 lb/ft.

In order to calculate post forces, the shear wall and diaphragm deflections must be calculated.

Diaphragm deflection is calculated using the shear term of the diaphragm deflection equation,

[Eq. 7]: $\Delta_{dia} = \frac{.25vL}{1000G}$

The effective shear modulus, G, remains the same 4.7 kips/in. Using the new value of unit shear, the resulting diaphragm deflection is calculated to be .354 in.

The shear wall deflection is calculated using the shear term of the shear wall deflection equation, [SPDWS Eq. C.4.3.2-2]. $\Delta_{sw} = \frac{vh_w}{1000G_a}$

The apparent shear wall shear stiffness, G_a, remains the same 19.3 kips/in. The resulting shear wall deflection was found to be .049 in. Therefore, the total post/diaphragm deflection, Δ_{eave}, at the building mid-span is .354+.049 = .403 in.

Equation 6, can then be used to calculate the maximum post moment: $M_{max}^{(-)} = \frac{wh_w^2}{8} + \frac{3EI\Delta_{eave}}{h_w^2}$

w is the distributed load on the post being designed and is taken to be:

$$w = \frac{\max(q_{ww}, q_{lw})s}{12}$$

In this example, w=3.667 lb/in., Δ = .403 in, and the resulting post moment is -24340 lb-in. Comparing these results to those of the standard method, the simplified procedure proves to be a comparable, yet conservative design for the unit shear, deflection, and post moment. The ratios of results for the simplified method compared to the standard method are as follows:

Unit shear: 59.4/56.963=1.04

Deflection: .403/.3701=1.09

Post Moment: -24340/-20364=1.19

This means that the simplified approach gave values for unit shear, eave deflection, and post moment that were conservative by 4%, 9%, and 19%, respectively.

<u>Simplified Method – Pin/Roller:</u>

The maximum unit shear is calculated using [Eq. 17]:

$$v = \frac{L\left[\frac{h_w\left(q_{ww} - q_{lw}\right)\left(2.8d + 3h_w\right)}{8\left(0.7d + h_w\right)} + h_r\left(q_{wr} - q_{lr}\right)\right]}{2W}$$

Assuming an embedment depth, d, of 4 ft., the resulting unit shear is calculated to be 62.35 lb/ft. With this new value of unit shear the maximum frame/diaphragm deflection can be calculated by:

$$\Delta_{dv} = \frac{.25vL}{1000G} + \Delta_{sw} = \frac{vh_w}{1000G_a}$$

Therefore, the total frame/diaphragm deflection, $\Delta_{eave,}$ at the building mid-span is .371+.052 = .423 in.

The maximum post bending moment can be calculated by summing Equations 13 and 15:

$$M_{max}^{(-)} = \frac{3EI\left[\frac{\frac{-wh_w^{\ 2}}{8}}{\frac{3EI}{h_w} + \frac{3EI}{.7(d)}}\right]}{0.7(d)} + \frac{3EI\Delta}{h_w\left[(0.7(d)) + h_w\right]}$$

w remains the same 3.667 lb/in. as previously calculated, and Δ=.423. The resulting post moment at the ground-line is calculated to be 21,030 lb-in.

The Simplified Pin/Roller procedure proves to be a comparable, yet conservative design for the unit shear, deflection, and post moment compared to the standard method. The Simplified-Pin/Roller approach gave more conservative values than the simplified method for unit shear and eave deflection, but gave a far less conservative approximation for the maximum post moment. The ratios of results for the Simplified - Pin/Roller method compared to the standard method are as follows:

Unit shear: 62.35/56.963=1.09

Deflection:.423 /.3701=1.14

Post Moment: -21030/-20364=1.03

Again, these ratios represent that the values determined for unit shear, eave deflection, and post moment were conservative with respect to the standard procedure by values of 9%, 14%, and 3%, respectively. The values for unit shear and deflection given by the Simplified-Pin/Roller method were more conservative than those of the simplified method. However, the Simplified - Pin/Roller approximation for ground line moment was much closer to that predicted by the standard method, with only 3% conservatism compared to 19% from the Simplified – Fixed.

RESULTS

The same building used in the model validation was analyzed for L:W ratios ranging from 1:1 to 4:1. The building width and wall heights were held constant at 56 and 16 ft, respectively. The effective shear modulus, G, and the apparent shear wall shear stiffness, G_a, were also held constant at 4.7 and 19.3 k/in, respectively. The ratios presented in Table 1 below represent the values of unit shear, eave deflection, and maximum post moment calculated by the Simplified - Fixed and Simplified – Pin/Roller methods compared to those calculated by the standard ANSI/ASAE EP484.2 method.

Table 1. Comparison of unit shear, deflection, and post moment to the standard method.

L:W	hw=16					
	Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ ratio	M ratio	v ratio	Δ 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

Figure 16 shows values of unit shear as a function of building L:W ratios and diaphragm stiffness. The Simplified – Pin/Roller method gave the highest value of unit shear, followed closely by the Simplified – Fixed method. Both simplified methods had the same value of unit shears regardless of the diaphragm stiffness, as the diaphragm stiffness isn't considered during the simplified procedures. Unit shears for the standard method were quite comparable to those of the simplified methods for smaller L:W ratios, but became less comparable as the L:W ratio approached 4.0. As would be expected, the diaphragm with the higher effective shear modulus of 7.5 k/in resulted in higher unit shears because a stiffer diaphragm attracts more load.

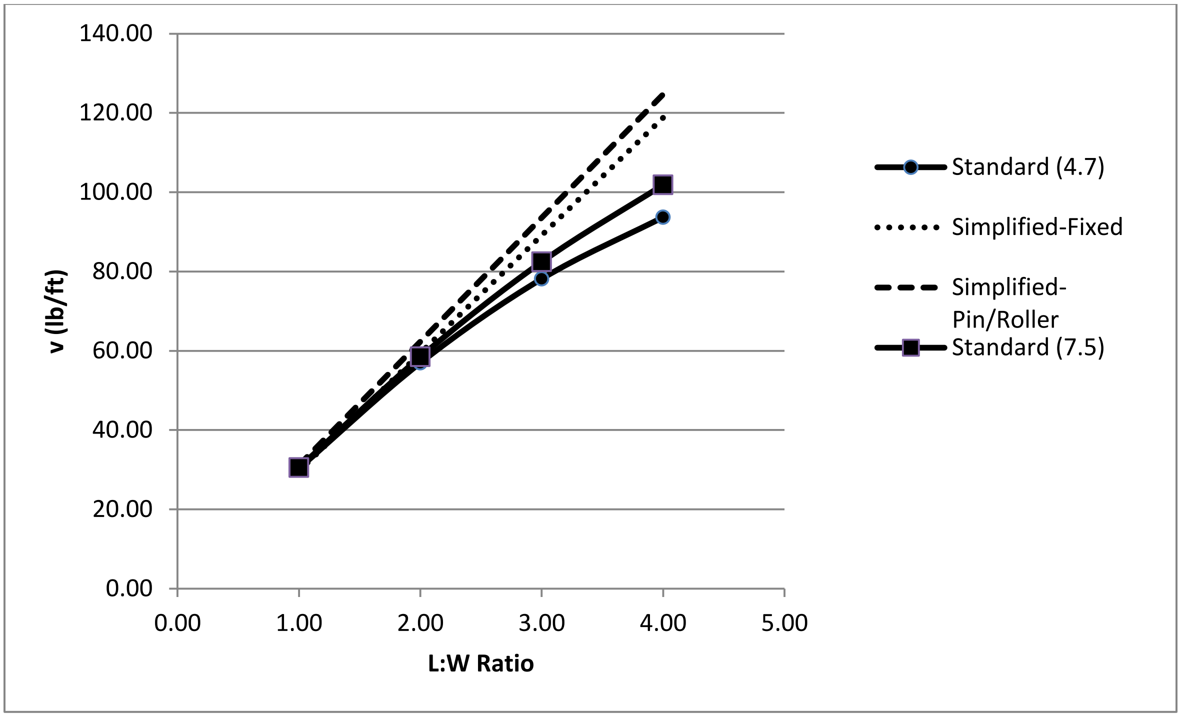


Figure 16. Comparison of shear wall unit shears for varying diaphragm stiffness.

Figure 17 shows values of mid-span eave deflection as a function of building L:W ratios and diaphragm stiffness. The Simplified – Pin/Roller method resulted in the highest values of eave deflection, followed by the Simplified – Fixed and standard methods. The diaphragm with the lower effective shear modulus of 4.7 k/in (lower stiffness) resulted in higher deflections for every L:W ratio compared to an effective shear modulus of 7.5 k/in. This plot shows that when a stiffer diaphragm is used, the differences in deflections from each method are decreased. In other words, a stiffer diaphragm makes the deflections from the different methods more comparable.

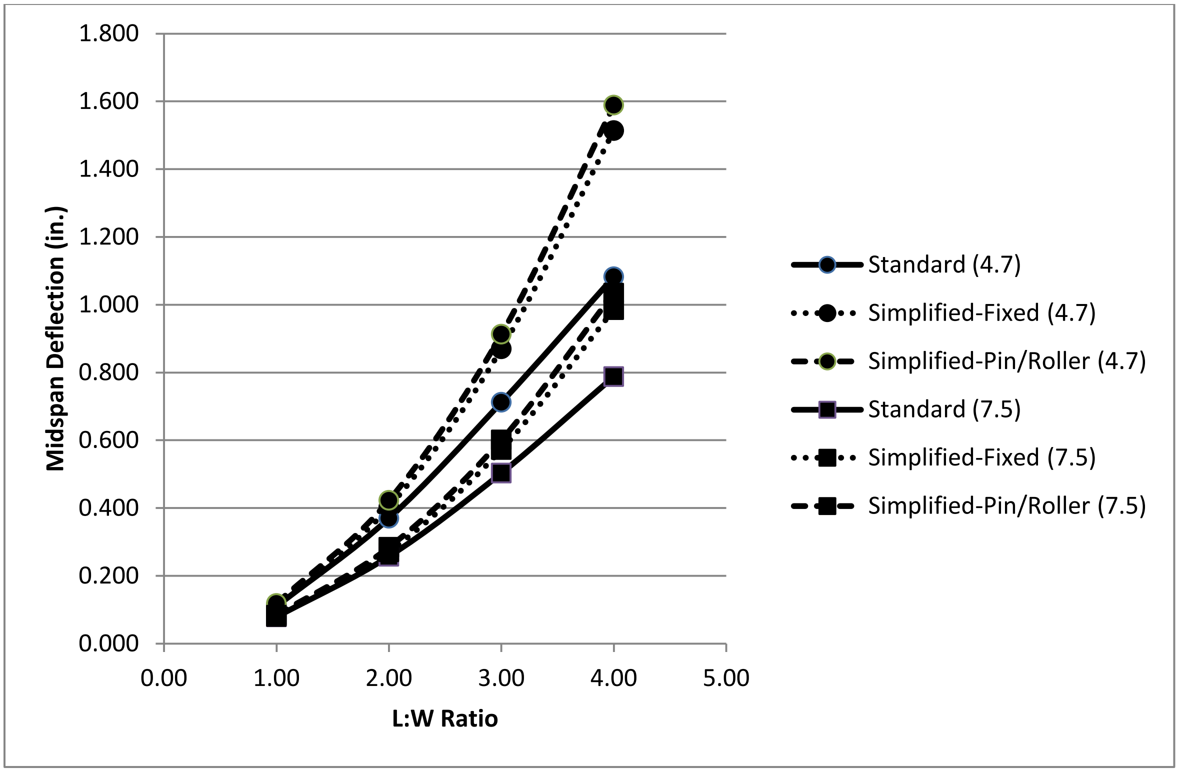


Figure 17. Comparison of mid-span eave deflections for varying diaphragm stiffness.

Figure 18 shows values of ground line moment as a function of building L:W ratios and diaphragm stiffness. The Simplified – Fixed method gave the highest values of ground line moment. This is a reasonable response because the perfectly fixed base condition attracts more load to the base than the pin/roller configuration used in the other methods. The Simplified – Pin/Roller method resulted in the second highest values of ground line moment, followed by the standard method. As diaphragm stiffness increased, ground line moment decreased. Similar to the results shown in Figure 14, a stiffer diaphragm results in less variability in ground line moment between the different methods of analysis.

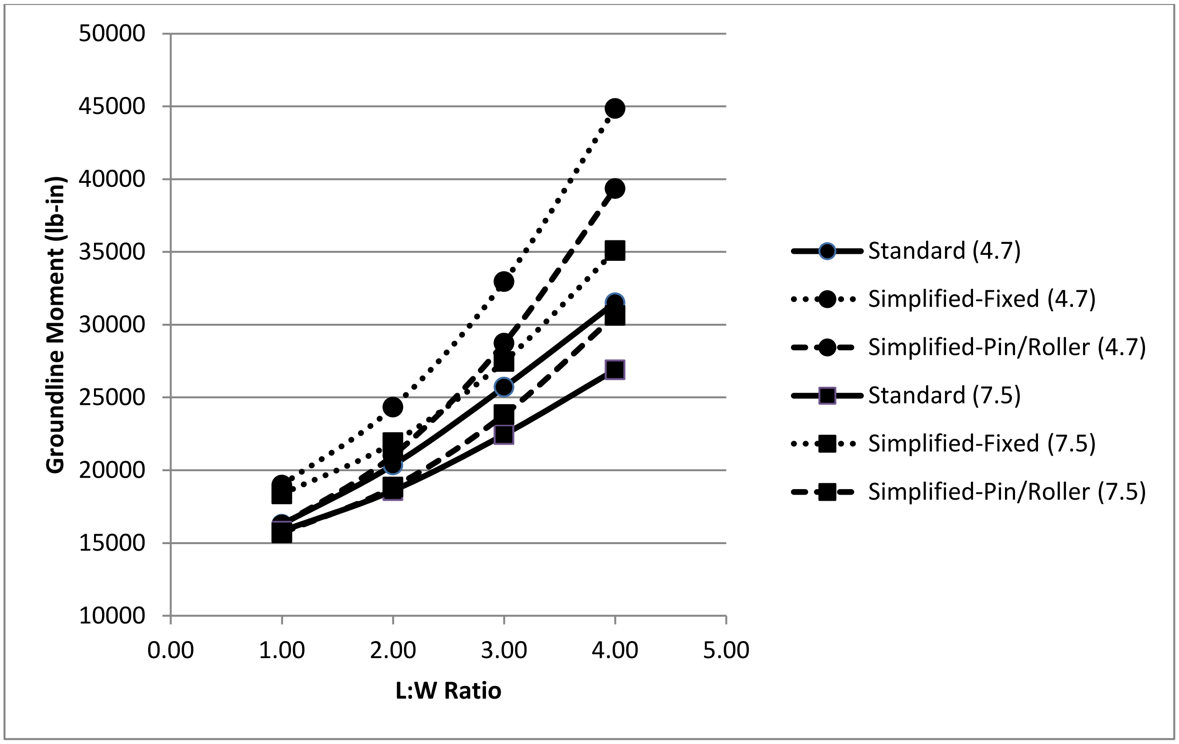


Figure 18. Comparison of ground-line post moment for varying diaphragm stiffness.

SENSITIVITY ANALYSIS

To analyze the effects that changing certain variables had on the resulting unit shears, eave deflections, and post moments, a number of analyses were run to show the relative impact of each change. Tables 2 - 5 below show the resulting ratios with wall heights of 12 and 16 ft, building widths of 40 and 56 ft, and effective diaphragm shear moduli of 4.7 and 7.5 k/in.

The two values for effective shear modulus were chosen to represent un-stitched and stitched diaphragm configurations. Stitched diaphragms are when the overlapping sheets of metal cladding have intermediate screws that "stitch" sheets together down the major ribs of the metal. The additional connectivity from the intermediate screws makes a stitched diaphragm assembly much stiffer than its un-stitched counterpart. Because lower diaphragm stiffness results in higher deflection and in turn higher post forces, the lowest values for an un-stitched and stitched diaphragm obtained from a simple beam test configuration were chosen from the published values in the NFBA Design Manual. The un-stitched assembly was G=4.7 k/in. and the stitched configuration was G=7.5 k/in (NFBA, 1999). The higher stiffness of the stitched diaphragm is a result of the intermediate screws throughout the diaphragm assembly. These values are for the case of the roof purlins resting on top of the rafters, rather than being recessed. This diaphragm assembly was tested using the following cladding and framing specifications:

<u>Cladding:</u>

Base Metal Thickness Gauge: 29

Major Rib Spacing, inches: 9

Major Rib Height, inches: 0.62

Major Rib Base Width, inches: 1.75

Major Rib Top Width, inches: 0.75

Yield Strength, ksi: 80

Wood framing properties:

Purlin Size: 2- by 6-inch

Purlin Species and Grade: No.2 DFL and 1650f DFL

Rafter Species and Grade: No. 2 DFL

Additional details on the diaphragm test assembly are given in Table 6.1 of the NFBA Manual.

Table 2. W=56 ft, G=4.7 k/in.

(W=56) L:W	**G=4.7 k/in**											
	h_w=12						h_w=16					
	Simplified - Fixed			Simplified - Pin/Roller			Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio
1	1.00	1.05	1.22	1.06	1.12	1.00	0.98	1.02	1.16	1.03	1.07	1.00
2	1.13	1.21	1.33	1.20	1.28	1.12	1.04	1.09	1.20	1.09	1.14	1.03
3	1.33	1.50	1.59	1.41	1.59	1.34	1.14	1.22	1.28	1.20	1.28	1.12
4	1.58	1.90	1.97	1.68	2.02	1.68	1.27	1.40	1.42	1.33	1.47	1.25

Table 3. W=56 ft, G=7.5 k/in.

(W=56) L:W	**G=7.5 k/in**											
	h_w=12						h_w=16					
	Simplified - Fixed			Simplified - Pin/Roller			Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio
1	0.98	1.02	1.21	1.04	1.09	0.99	0.97	1.01	1.16	1.02	1.06	0.99
2	1.07	1.13	1.27	1.14	1.20	1.06	1.02	1.05	1.18	1.07	1.10	1.01
3	1.21	1.31	1.42	1.28	1.39	1.20	1.08	1.14	1.22	1.13	1.19	1.06
4	1.38	1.56	1.65	1.47	1.66	1.40	1.17	1.25	1.30	1.22	1.31	1.14

Table 4. W=40 ft, G=4.7 k/in.

(W=40) L:W	G=4.7 k/in											
	h_w=12						h_w=16					
	Simplified - Fixed			Simplified - Pin/Roller			Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio
1	0.98	1.07	1.22	1.04	1.13	1.00	0.97	1.05	1.17	1.02	1.10	1.00
2	1.08	1.17	1.30	1.15	1.24	1.08	1.02	1.08	1.19	1.07	1.14	1.02
3	1.23	1.38	1.48	1.31	1.47	1.24	1.09	1.18	1.25	1.15	1.24	1.08
4	1.42	1.66	1.74	1.51	1.77	1.48	1.18	1.31	1.34	1.24	1.37	1.18

Table 5. W=40 ft, G=7.5 k/in.

(W=40) L:W	G=7.5 k/in											
	h_w=12						h_w=16					
	Simplified - Fixed			Simplified - Pin/Roller			Simplified - Fixed			Simplified - Pin/Roller		
	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio	v ratio	Δ ratio	M ratio
1	0.97	1.04	1.22	1.03	1.11	0.99	0.97	1.03	1.17	1.02	1.08	1.00
2	1.04	1.11	1.26	1.10	1.18	1.04	1.00	1.05	1.18	1.05	1.10	1.01
3	1.14	1.25	1.36	1.21	1.33	1.14	1.05	1.12	1.21	1.10	1.18	1.05
4	1.27	1.43	1.52	1.35	1.52	1.28	1.11	1.20	1.26	1.17	1.26	1.10

DISCUSSION

The sensitivity analysis allowed for analysis of the three input variables subject to change and how they affect the results of the three methods of analysis. These variables are the effective shear modulus, G, wall height, h_w, and building width, W.

Tables 2-5 show that higher diaphragm stiffness results in lower ratios of unit shear, deflection, and moment for both the Simplified – Fixed and Simplified – Pin/Roller methods when compared to the standard method. It can be seen that change in G has a far greater impact on the results for lower wall heights than for higher wall heights. For example, note the differences between the underlined values of Table 4 and those of Table 5. As G increases from 4.7 to 7.5, the ratios for a 12 ft wall height decrease by 0.15, 0.23, and 0.22 for shear, deflection, and moment ratios, respectively. As G increases from 4.7 to 7.5, the ratios for a 16 ft wall height decrease by 0.07, 0.11, and 0.08 for shear, deflection, and moment, respectively.

As the wall height increased, the ratios of shear, deflection, and moment decreased for both of the simplified methods when compared to the standard method. This trend held true regardless of the building width and the effective diaphragm shear modulus used. However, for shorter building widths and higher diaphragm stiffness, the 16 ft. walls gave values closer to those produced from the standard method. The trend can be seen by looking at the difference in underlined values of Table 4, which for the lower G value of 4.7 k/in., results in changes of 0.24, 0 .35, and 0.4 for unit shear, deflection, and moment ratios, respectively as the wall height changes from 12 to 16 ft. Similarly, Table 5 shows the higher G value of 7.5 k/in. results in changes of 0.16, 0.23, and 0.26, for shear, deflection, and moment ratios, respectively as the wall height changes from 12 to 16 ft.

Changes in wall height had a greater effect on results from the standard method because this method requires the designer to calculate frame stiffness, an equation where the wall height term is cubed. However, in the simplified methods frame stiffness need not be calculated.

As the building width increased, the unit shear values for the standard method decreased. This is a function of dividing the end-wall shear force by a larger W. Because the equations of unit shear for the Simplified – Fixed and Simplified – Pin/Roller methods are dependent on the L:W ratio of the building, the values of unit shear remain unchanged as building width increases because the ratios are still held constant. Because unit shears for the standard method decrease, and those for the simplified methods remain the same, the ratios of unit shear compared to the standard method increase as building width increases. As the building length increases (higher L:W ratio), building mid-span deflections increase which in turn increases the maximum post moment. In general, as shown in Tables 2-5, the ratios of unit shear, deflection, and moment increase as building width increases. Noting the differences in Tables 2-4 and 3-5, the change in building width had little effect on the ratios of unit shear, deflection, and moment for wall heights of 16 ft. Wall heights of 12 ft showed a greater impact on values of unit shear, deflection, and moment as the building width changed, however, the results were quite comparable.

Table 6 below shows the actual values of unit shear, building mid-span deflection, and ground line post moment for varying shear modulus and wall height of a 56 ft wide building. These are the values calculated for building L:W ratios of 4.0, as this is the ratio that shows the greatest difference between the values of the standard and the simplified methods.

Table 6. Values of unit shear, deflection, and post moment for W=56 ft (L:W=4.0)

W=56 ft. (L:W=4.0)	**G=4.7 k/in**					
	h_w=12			h_w=16		
	Standard	Simplified - F	Simplified - P/R	Standard	Simplified - F	Simplified - P/R
v (lb/ft.)	52.28	82.80	88.02	93.69	118.80	124.70
Δ (in.)	0.55	1.04	1.10	1.08	1.51	1.59
M (lb-in.)	21,792	42,896	36,523	31,512	44,858	39,359
W=56 ft. (L:W=4.0)	**G=7.5 k/in**					
	h_w=12			h_w=16		
	Standard	Simplified - F	Simplified - P/R	Standard	Simplified - F	Simplified - P/R
v (lb/ft.)	59.97	82.80	88.02	101.85	118.80	124.70
Δ (in.)	0.43	0.67	0.71	0.79	0.99	1.03
M (lb-in.)	18,660	30,803	26,100	26,904	35,098	30,640

The best comparison between the standard and simplified methods are shown in Table 3 above for a building width of 56 ft with 16 ft walls and a stitched diaphragm assembly with an effective shear modulus of 7.5 k/in. The building aspect ratio of 4:1, which gives the most conservative results, shows conservative, yet comparable results for the Simplified methods compared to the Standard method. The Simplified - Fixed method gave values of unit shear, mid-span eave deflection, and ground line moment that were conservative by 17%, 25%, and 30% respectively when compared to results from the standard method. Similarly, The Simplified – Pin/Roller method gave values of unit shear, mid-span eave deflection, and ground-line moment that were conservative by 22%, 31%, and 14% respectively when compared to results from the standard method. Building aspect ratios of 1:1 – 3:1 gave even more comparable results.

When comparing results of the Simplified – Fixed and Simplified – Pin/Roller methods, the Simplified – Fixed method gave lower values of unit shear and building mid-span eave deflection. This is because the analog of the base condition for the Simplified – Pin/Roller method allows for more load to be transferred into the diaphragm. Higher forces in the top of the post result in higher unit shear, and therefore higher deflections. Conversely, the Simplified – Pin/Roller analog for base fixity attracts less load to the base compared to the Simplified – Fixed analog because it is less stiff than the assumed perfectly fixed condition. With less load attracted to the base, the ground line post moment determined from the Simplified – Pin/Roller method is substantially less than that of the Simplified – Fixed method. The only non-conservative results (ratios under 1.0) came from building L:W ratios of 1.0. However, this unconservatism was, at most, less than 3% of what was calculated by the standard method.

The above tables show that in some cases the simplified methods can predict values of unit shear, deflection, and moment that are quite conservative with respect to those of the standard method. However, this conservatism may not have as much impact on the overall design of the building as one might think. To illustrate this example the building with the highest induced post moment was chosen for analysis. This building was 56 ft wide with a L:W ratio=4.0 with 16 ft walls, and an effective shear modulus for the diaphragm of 4.7 k/in. The resulting post moments were 31,512 lb-in and 44,858 lb-in from the standard and Simplified – Fixed methods, respectively. In other terms, the Simplified – Fixed method produced a moment that was conservative by 42% compared to the standard method.

The calculated maximum dead and snow loads acting on a single post were found to be 2370 lbs and 6861 lbs. The governing ASD load combination for these loads was D+.75(S+W). This load combination resulted in an axial load of 7516 lbs and maximum post moments of 23,634 lb-in and 33,644 lb-in for the standard and Simplified – Fixed methods, respectively. Using NDS equation 3.9-3 for combined bending and axial compression, it was found that the critically loaded posts in the above building used only 41% of its strength capacity when using post moment from the standard method. When using the moment from the Simplified – Fixed method, the post used only 56% if its strength capacity. In this case, the conservative value for post moment found by the Simplified – Fixed approach still allowed the design to check with much capacity to spare. Additionally, the ground line moment that was conservative by 42% only reduced the posts remaining capacity by 15%. Similarly, the calculated embedment depth when using the standard method was found to be 3.91 ft, while the embedment depth for the Simplified – Fixed method was 4.46 ft; a difference of just 6.5 in. Based on these example calculations it can be concluded that the conservatism of these methods is tolerable, as they do not appear to have a significant impact on the overall design of the main structural elements. Detailed calculations for the post design/capacity, as well as embedment depth, are shown in Appendix B.

SUMMARY AND CONCLUSIONS

Two methods of lateral design for post-frame buildings were examined. The ANSI/ASAE EP484.2, referred to throughout this paper as the standard method, presents a rigorous approach for determining maximum diaphragm/shear wall unit shears, eave deflections, and post moment. The majority of difficulty associated with this method is a result of calculating the frame/diaphragm stiffness and determining the force distribution between these elements. This method is widely accepted throughout the post-frame community; however, it can be difficult to grasp without a background in the design of post-frame buildings. A simplified method was presented in which the contribution of frame stiffness was conservatively ignored with regard to total building sidesway displacement. Two simplified methods were explored: one considering a fixed base condition (Simplified – Fixed), the other considering a pin/roller base support (Simplified – Pin/Roller). The simplified methods provide rational alternatives for the lateral design of post-frame buildings that can be easily understood by design and building regulatory professionals with limited experience in post-frame building design. After comparing results from the simplified methods to those of the standard method for varying building geometries and diaphragm configurations, it was found that the simplified methods gave conservative results with respect to unit shear, eave deflections, and post moment. These results showed that this simplified approach could be conservatively used in place of the ANSI/ASAE EP484.2 design methods. The results show that higher values of diaphragm stiffness make the simplified methods more comparable to the standard method. The simplified methods outlined in this paper could be substituted for the standard method for L:W ratios up to 4.0 without a significant loss in accuracy/building economy. The best application for these proposed simplified procedures are buildings with walls of 16 ft. or higher when stitched diaphragm configurations are used. When this is the case these simplified methods provide very close approximations for unit shear, deflection, and post moment.

The proposed simplified methods gave conservative results when shorter walls heights of 12 ft were examined, especially when paired with a lower stiffness diaphragm. Based on these results, in order to prevent building designs from being overly conservative when using shorter wall heights, the simplified analyses presented should be limited to L:W ratios of 2.0 when an unstitched diaphragm with an effective shear modulus on the order of 4.7 k/in. is used. When a stitched diaphragm is used, resulting in an effective shear modulus on the order of 7.5 k/in, the simplified analyses would be effective for L:W ratios up to 3.0.

RECCOMENDATIONS/FURTHER RESEARCH

In order to calculate maximum post moment using the simplified methods the mid-span diaphragm deflection must be first be calculated. In doing so the effective diaphragm shear modulus, G, is required. Little research has been conducted for determining the effective shear modulus of wood-framed, metal-clad diaphragms. The NFBA Design Manual is one reference for these values, but the test configurations and results are limited. There is a need to create design tables similar to that of Table 4.2 in ANSI/AF&PA SDPWS which tabulate values of effective shear modulus and unit shear capacities for a range of common post-frame diaphragm constructions.

Similar to the effective shear modulus of roof diaphragms, limited data are available with regard to the apparent shear stiffness, G_a, and unit shear capacities of metal-clad post-frame shear walls. Additional research to create a design tables similar to Table 4.3 in ANSI/AF&PA SDPWS, which would tabulate values of effective shear modulus and unit shear capacities for a range of common post-frame shear wall constructions, would be very helpful to designers utilizing these simplified methods of post-frame design.

BIBLIOGRAPHY

AISC Steel Construction Manual, 13th Ed. 2005. American Institute of Steel Construction.

American Forest & Paper Association, Inc., *National Design Specification for Wood Construction*, 2008 edition, Washington, DC.

American Forest & Paper Association, Inc., *Wind & Seismic, Special Design Provisions for Wind and Seismic*, 2008 edition, Washington, DC.

ASAE Standards, 37th Ed. 2003, EP484.2. Diaphragm Design of metal-clad, post-frame rectangular buildings. St. Joseph, MI: ASAE

Bender, D.A. 1991. Rigid roof design for post-frame buildings. Frame Building Professional 3(6):4-11,30-33.

Bohnhoff, D.R. 1992. Expanding Diaphragm Analysis for Post-Frame Buildings. ASAE Paper No. 0883-8542/92/0804-00509.

Kassimali, A. Structural Analysis, Fourth Edition. 2005. Stamford, CT: Cengage Learning.

Leatherman, B. NFBA Design Tool for Design of Post-Frame Building Systems.

NFBA: National Frame Builders Association. 1999. Post-Frame Building Design Manual. Glenview, IL.

NFBA.org (2011). National Frame Builders Association: The Post-Frame Advantage. Retrieved Jan. 20, 2012, from http://nfba.org/uploads/files/PFA_GuideNS1.pdf.

Pollock, D.G., D.A. Bender and K.G. Gebremedhin. 1996. Designing for chord forces in post-frame roof diaphragms. Frame Building News 8(5): 40-44.

Pope, E., D.A. Bender, D.P. Mill. (2012) Deflection of Steel-Clad Wood-Frame Diaphragms. MS report, Washington State University, Pullman, WA.

Ross, L.A., D.A. Bender, D.M. Carradine. 2009. Strength and stiffness of post-frame shear walls with wood plastic composite skirtboards. Frame Builders News, November 2009.

Skaggs, T.D., F.E. Woeste and D.A. Bender. 1993. A simple analysis procedure for calculating post forces. Applied Engineering in Agriculture 9(2):253-259.

NOMENCLATURE

ANSI/ASAE EP 484.2: Standard for diaphragm design of metal-clad, wood-frame rectangular buildings

$\mathbf{C_{h,i}}$: Horizontal shear stiffness of diaphragm i with width, s

$\mathbf{C_{p,i}}$: In-plane shear stiffness of diaphragm i with width, s

d: Post embedment depth

DAFI: Diaphragm and Frame Interaction computer program (Bohnhoff, 1992)

f: Frame base fixity factor (f=3/8 for fixed base condition)

Frame/Post-frame: The assembly of two cantilever posts connected by a truss. The truss is assumed to be pin-connected to the top of the posts. The frames are able to resist moment by being embedded into the ground.

E: Post modulus of elasticity

G: Effective diaphragm shear modulus

$\mathbf{G_a}$: Apparent shear wall shear stiffness

$\mathbf{h_r}$: roof height

$\mathbf{h_w}$: Building wall height=post height

I: Post moment of inertia

k: Frame stiffness

k_{sw}: shear wall stiffness

L: Building length

L:W: Building length:width ratio

mD: Sidesway restraining force modifier (From ANSI/ASAE EP 484.2, Section 7)

mS: Shear force modifier (From ANSI/ASAE EP 484.2, Section 7)

NDS: National Design Specification For Wood Construction (ANSI/AF&PA NDS)

P: Applied point load on the end of a cantilever beam analog

Q: Sidesway restraining force

qlr: Leeward roof pressure (psf)

qlw: Leeward wall pressure (psf)

qwr: Windward roof pressure (psf)

qww: Windward wall pressure (psf)

R: Eave load. This is a point load applied to the eave of a frame that represents the portion of the total applied load that is transferred into the diaphragm, based on the tributary area of the frame.

s: Post/frame spacing

SDPWS: Special Design Provisions For Wind And Seismic (ANSI/AF&PA SDPWS)

Simplified – Fixed method: Simplified method of analysis with fixed base condition

Simplified – Pin/Roller method: Simplified method of analysis with pin/roller base condition

Standard method: See ANSI/ASAE EP 484.2

v: Maximum unit shear in diaphragm/building shear walls

VA: Visual Analysis (Computerized structural design software:

$\mathbf{V_h}$: Maximum shear force in roof diaphragm

W: Building width

$\mathbf{\Delta_{dia}}$: Deflection of diaphragm at building mid-span

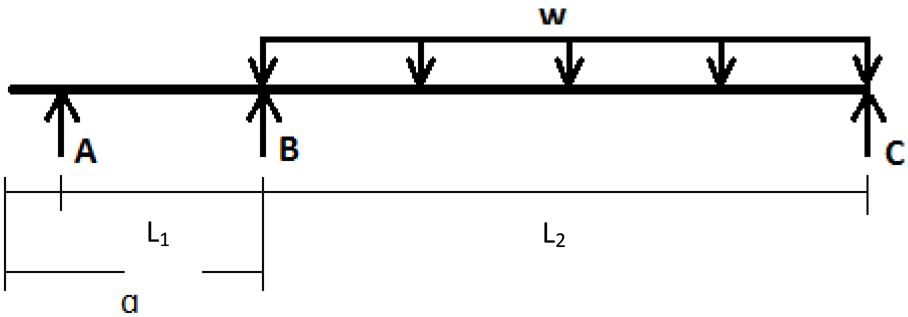
$\mathbf{\Delta_{eave} = \Delta}$: Total mid-span building deflection ($\Delta_{eave} = \Delta_{dia} + \Delta_{sw}$)

$\mathbf{\Delta_{sw}}$: Deflection of shear walls

Θ: Roof slope (degrees) from the horizontal of diaphragm

APPENDIX A: DERIVATIONS FOR PIN-ROLLER PROPPED CANTILEVER ANALOG

Derivation of Ground-line moment for pin/roller supported propped-cantilever analog:



Appendix Figure 1. . Pin/Roller supported propped-cantilever post analog.

For the following derivation it is assumed that the embedment (d-L1) of the post to the left of the pin support, A, does not have any significant affect on the ground-line moment, M_{BA}. Using the slope-deflection equations we can then derive the moment at the roller support, B, which represents the ground-line moment of an embedded post.

Slope-deflection equations:

$$M_{AB} = \frac{2EI}{L}\left(2\theta_A + \theta_B - 3\psi_{AB}\right) + FEM_{AB}$$

[Kassimali (2005), pg. 641]

$$M_{BA} = \frac{2EI}{L}\left(\theta_A + 2\theta_B - 3\psi_{AB}\right) + FEM_{BA}$$

Where $\psi_{AB} = \frac{v_B - v_A}{L}$

1. Designate unknown degrees of freedom:
   $\theta_A, \theta_B, \theta_C$

2. Write slope-deflection equations for each member:
   $FEM_{AB} = FEM_{BA} = 0$

   $v_A, v_B, v_C = 0$ therefore, $\psi_{AB} = \psi_{BC} = \frac{0-0}{L} = 0$

$$M_{AB} = \frac{2EI}{L_1}\left(2\theta_A + \theta_B\right)$$

$$M_{BA} = \frac{2EI}{L_1}\left(\theta_A + 2\theta_B\right)$$

$$FEM_{BC} = \frac{wL_2^{\ 2}}{12}$$

$$FEM_{CB} = \frac{-wL_2^{\ 2}}{12}$$

$$M_{BC} = \frac{2EI}{L_2}\left(2\theta_B + \theta_C\right) + \frac{wL_2^{\ 2}}{12}$$

$$M_{CB} = \frac{2EI}{L_2}\left(\theta_B + 2\theta_C\right) - \frac{wL_2^{\ 2}}{12}$$

3. Recognize boundary conditions:
   $v_A, v_B, v_C = 0$

4. Apply conditions of equilibrium to joints:

   M_{AB}=0

   M_{BA}+M_{BC}=0

   M_{CB}=0

5. Solve for unknown degrees of freedom:
   M_{AB}=0:

$$M_{AB} = \frac{2EI}{L_1}\left(2\theta_A + \theta_B\right) = 0$$

$$\frac{4EI\theta_A}{L_1} + \frac{2EI\theta_B}{L_1} = 0$$

$$\theta_A = \frac{-\theta_B}{2}$$

M_{CB}=0:

$$M_{CB} = \frac{2EI}{L_2}(\theta_B + 2\theta_C) - \frac{wL_2^{\ 2}}{12} = 0$$

$$\frac{2EI\theta_B}{L_2} + \frac{4EI\theta_C}{L_2} - \frac{wL_2^{\ 2}}{12} = 0$$

$$\frac{4EI\theta_C}{L_2} = \frac{-2EI\theta_B}{L_2} + \frac{wL_2^{\ 2}}{12}$$

$$\theta_C = \frac{-\theta_B}{2} + \frac{wL_2^{\ 3}}{48EI}$$

θ_A is now know in terms of θ_B. M_{BA} can then be solved for in terms of θ_B.

$$M_{BA} = \frac{2EI}{L_1}(\theta_A + 2\theta_B)$$

$$M_{BA} = \frac{2EI}{L_1}\left(\frac{-\theta_B}{2} + 2\theta_B\right)$$

$$M_{BA} = \frac{3EI\theta_B}{L_1}$$

θ_C is now know in terms of θ_B. M_{BC} can then be solved for in terms of θ_B.

$$M_{BC} = \frac{2EI}{L_2}(2\theta_B + \theta_C) + \frac{wL_2^{\ 2}}{12}$$

$$M_{BC} = \frac{2EI}{L_2}\left[2\theta_B + \left(\frac{-\theta_B}{2} + \frac{wL_2^{\ 3}}{48EI}\right)\right] + \frac{wL_2^{\ 2}}{12}$$

$$M_{BC} = \frac{2EI}{L_2}\left[\frac{3\theta_B}{2} + \frac{wL_2^{\ 3}}{48EI}\right] + \frac{wL_2^{\ 2}}{12}$$

$$M_{BC} = \frac{3EI\theta_B}{L_2} + \frac{wL_2^{\ 2}}{24} + \frac{2wL_2^{\ 2}}{24}$$

$$M_{BC} = \frac{3EI\theta_B}{L_2} + \frac{wL_2^{\ 2}}{8}$$

With M_{BC} and M_{BA} both in terms of θ_B, the conditions of equilibrium: $M_{BA}+M_{BC}=0$ can be applied, and θ_B can be solved for.

$M_{BA}+M_{BC}=0$:

$$\frac{3EI\theta_B}{L_1} + \left[\frac{3EI\theta_B}{L_2} + \frac{wL_2^{\ 2}}{8}\right] = 0$$

$$\theta_B\left[\frac{3EI}{L_2} + \frac{3EI}{L_1}\right] = \frac{-wL_2^{\ 2}}{8}$$

$$\theta_B = \frac{\frac{-wL_2^{\ 2}}{8}}{\frac{3EI}{L_2} + \frac{3EI}{L_1}}$$

6. Compute member end moments:

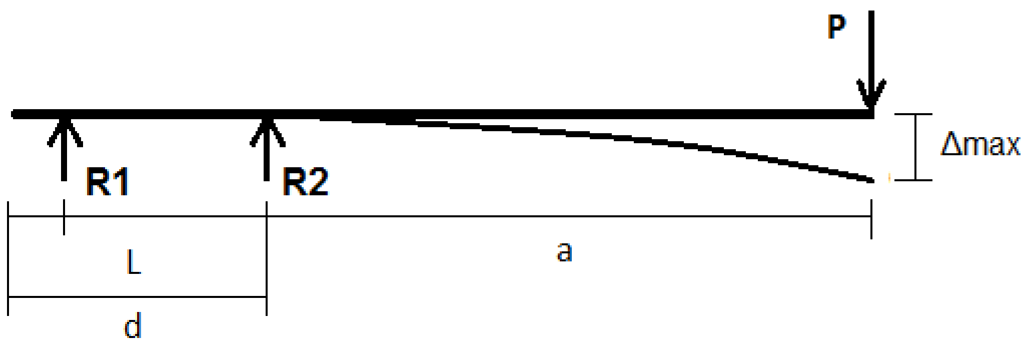
This equation for θ_B can then be substituted into the equation for M_{BA}, resulting in the equation for the moment at support B (the ground-line) moment.

$$M_{BA} = \frac{3EI\left[\frac{\frac{-wL_2^{\ 2}}{8}}{\frac{3EI}{L_2} + \frac{3EI}{L_1}}\right]}{L_1}$$

In order for the terms of this equation to match those used throughout this paper: L_2=h_w and L_1=.7(d), where d is the post embedment depth. The resulting equation is:

$$M_{BA} = \frac{3EI\left[\dfrac{\dfrac{-wh_w^{\,2}}{8}}{\dfrac{3EI}{h_w}+\dfrac{3EI}{.7(d)}}\right]}{.7(d)} \qquad \text{[Eq. 9]}$$

Derivation of deflection for pin/roller supported cantilever analog:



Appendix Figure 2. Pin/Roller supported cantilever post analog.

For the following derivation it is assumed that the embedment (d-L) of the post to the left of the pin support, A, does not have any significant affect on the deflection at the end of the cantilever.

Equation for beam overhanging one support with a concentrated point load at the end of the overhang:

$$\Delta_{max} = \frac{Pa^2}{3EI}(L+a) \qquad \text{[AISC Table 3-23, pg. 3-220]}$$

$$M_{max} = Pa$$

Solve deflection equation for P:

$$P = \frac{3EI\Delta}{a^2(L+a)}$$

Substitute $P = \frac{M_{max}}{a}$ into the above equation for deflection:

$\frac{M_{max}}{a} = \frac{3EI\Delta}{a^2(L+a)}$. Solving this equation for M_{max} results in:

$$M_{max} = \frac{3EI\Delta}{a\ (L+a)}$$

In order for the terms of this equation to match those used throughout this paper: $a=h_w$ and

$L=.7(d)$, where d is the post embedment depth. The resulting equation is:

$$M_{max} = \frac{3EI\Delta}{h_w[(.7(d))+h_w]} \qquad \text{[Eq. 11]}$$

APPENDIX B: POST DESIGN EXAMPLE

Post Design Using Standard Method:

COLUMN DESIGN: Bending and Axial Compression

Lumber properties:

$grade := 1$

$b := 4.31$ in

$d := 7.19$ in

$l_u := 192$ in

$F_b := 1500$ psi

$F_c := 1650$ psi

$E := 1700000$ psi

$E_{min} := 620000$ psi

Adjustment Factors

$C_D := 1.6$

$C_M := 1$

$C_t := 1$

$C_F := 1$

$C_{fu} := 1$

$C_i := 1$

$C_r := 1$

Member Forces:

$C := 7516$ lb

$M := 23634$ lb − in

These represent results (factored) from the standard method

$$f_b := \frac{6 \cdot M}{b \cdot d^2} = 636.434 \quad \text{psi}$$

$$f_c := \frac{C}{b \cdot d} = 242.538 \quad \text{psi}$$

Bending Calculations:

$$l_e := \begin{cases} 2.06 \cdot l_u & \text{if } \frac{l_u}{d} < 7 \\ 1.63 \cdot l_u + 3 \cdot d & \text{otherwise} \end{cases} = 334.53 \quad \text{in}$$

$$R_B := \left(\frac{l_e \cdot d}{b^2}\right)^{.5} = 11.379$$

$C_T := 1$

$$E_{min'} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 6.2 \times 10^5 \quad \text{psi}$$

$$F_{bE} := \frac{1.2 \cdot E_{min'}}{R_B^2} = 5.746 \times 10^3$$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{bs}}\right)}{1.9} - \sqrt{\left[\frac{1 + \left(\frac{F_{bE}}{F_{bs}}\right)}{1.9}\right]^2 - \frac{\left(\frac{F_{bE}}{F_{bs}}\right)}{.95}} = 0.967$$

$$F_{b'} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 2.321 \times 10^3 \quad \text{psi}$$

Compression Calculations:

$K_e := .8$ From Table G1 of Appendix G

$l_{ep} := K_e \cdot l_u = 153.6$ in

$$F_{cE} := \frac{.822 \cdot E_{min'}}{\left(\frac{l_{ep}}{d}\right)^2} = 1.117 \times 10^3 \quad \text{psi}$$

$$F_{cs} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 2.64 \times 10^3 \quad \text{psi}$$

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F_{cs}}\right)}{2 \cdot .8} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{cs}}\right)}{2 \cdot .8}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{cs}}\right)}{.8}} = 0.377$$

$$F_{c'} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_p = 996.017$$

$$\left(\frac{f_c}{F_{c'}}\right)^2 + \frac{f_b}{F_{b''} \left[1. - \left(\frac{f_c}{F_{cE}}\right)\right]} = 0.41$$

Post Embeddment Calculation:

$$M_g := \frac{23634}{12} = 1.97 \times 10^3 \quad \text{lb} - \text{ft}$$

$$b_p := \frac{\left(b^2 + d^2\right)^{.5}}{12} = 0.699 \quad \text{ft}$$

$S_{prime} := 200$ $\frac{\text{psf}}{\text{ft}}$ Allowable lateral soil-bearing pressure. From Table 1806.2 (IBC 2009)

$$d_{embedd} := \left(\frac{4.25 \cdot M_g}{S_{prime} \cdot b_p}\right)^{\frac{1}{3}} = 3.913 \quad \text{ft}$$

Equation from ASAE EP 486.1 Shallow Post Foundation Design.

Post Design Using Simplified Method:

Lumber properties:

$grade := 1$

$b := 4.31$ in

$d := 7.19$ in

$l_u := 192$ in

$F_b := 1500$ psi

$F_c := 1650$ psi

$E := 1700000$ psi

$E_{min} := 620000$ psi

Adjustment Factors

$C_D := 1.6$

$C_M := 1$

$C_t := 1$

$C_F := 1$

$C_{fu} := 1$

$C_i := 1$

$C_r := 1$

Member Forces:

$C := 7516$ lb

$M := 33644$ lb − in

These represent results (factored) from the simplified method

$$f_b := \frac{6 \cdot M}{b \cdot d^2} = 905.991 \quad \text{psi}$$

$$f_c := \frac{C}{b \cdot d} = 242.538 \quad \text{psi}$$

Bending Calculations:

$$l_e := \begin{cases} 2.06 \cdot l_u & \text{if } \frac{l_u}{d} < 7 \\ 1.63 \cdot l_u + 3 \cdot d & \text{otherwise} \end{cases} = 334.53 \quad \text{in}$$

$$R_B := \left(\frac{l_e \cdot d}{b^2}\right)^{.5} = 11.379$$

$C_T := 1$

$$E_{min'} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 6.2 \times 10^5 \quad \text{psi}$$

$$F_{bE} := \frac{1.2 \cdot E_{min'}}{R_B^2} = 5.746 \times 10^3$$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F_{bs}}\right)}{1.9} - \sqrt{\left[\frac{1 + \left(\frac{F_{bE}}{F_{bs}}\right)}{1.9}\right]^2 - \frac{\left(\frac{F_{bE}}{F_{bs}}\right)}{.95}} = 0.967$$

$$F_{b'} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 2.321 \times 10^3 \quad \text{psi}$$

Compression Calculations:

$K_e := .8$ From Table G1 of Appendix G

$l_{ep} := K_e \cdot l_u = 153.6$ in

$$F_{cE} := \frac{.822 \cdot E_{min'}}{\left(\frac{l_{ep}}{d}\right)^2} = 1.117 \times 10^3 \quad \text{psi}$$

$F_{cs} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 2.64 \times 10^3$ psi

$$C_p := \frac{1 + \left(\frac{F_{cE}}{F_{cs}}\right)}{2 \cdot .8} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{cs}}\right)}{2 \cdot .8}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{cs}}\right)}{.8}} = 0.377$$

$F_{c'} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_p = 996.017$

$$\left(\frac{f_c}{F_{c'}}\right)^2 + \frac{f_b}{F_{b''} \left[1. - \left(\frac{f_c}{F_{cE}}\right)\right]} = 0.558$$

Post Embeddment Calculation:

$M_g := \frac{33644}{12} = 2.804 \times 10^3$ lb – ft

$$b_p := \frac{\left(b^2 + d^2\right)^{.5}}{12} = 0.699 \quad \text{ft}$$

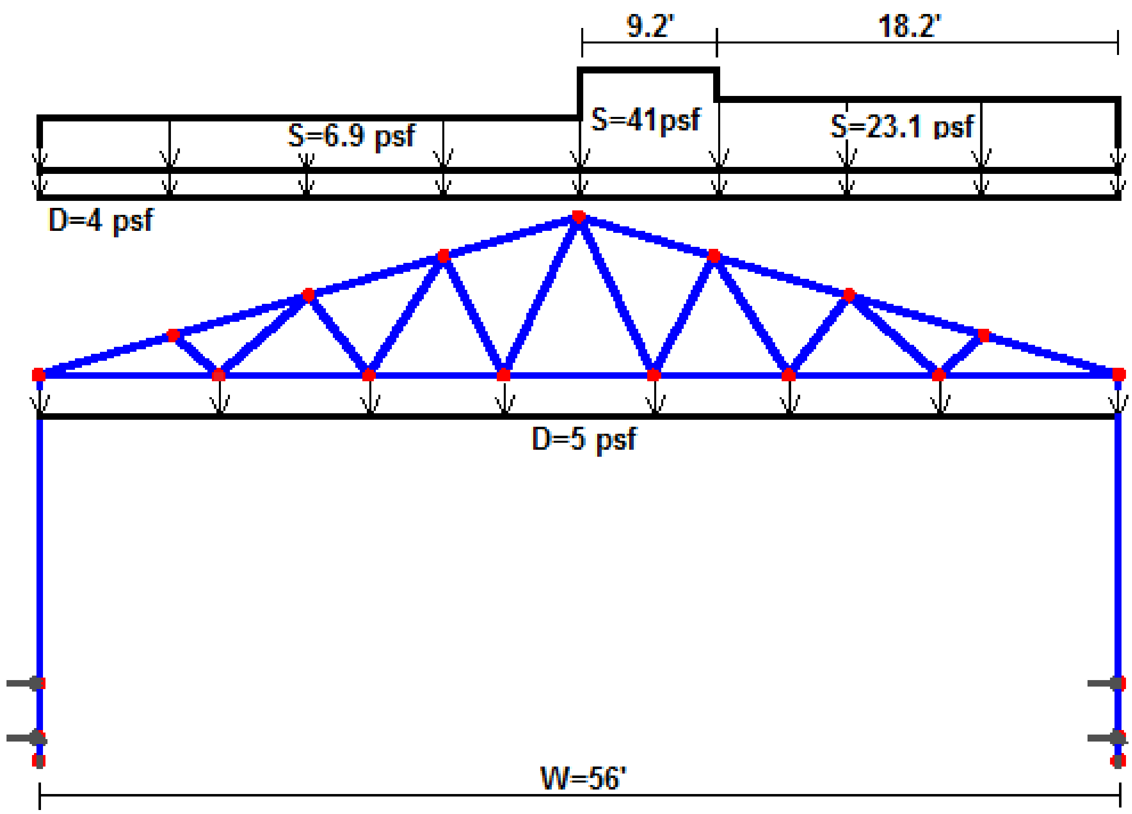
$S_{prime} := 200$ $\frac{psf}{ft}$ Allowable lateral soil-bearing pressure. From Table 1806.2 (IBC 2009)

$$d_{embedd} := \left(\frac{4.25 \cdot M_g}{S_{prime} \cdot b_p}\right)^{\frac{1}{3}} = 4.402 \quad \text{ft}$$

Equation from ASAE EP 486.1 Shallow Post Foundation Design.

Determining governing post axial load:

Snow loads were calculated following the procedures outlined in chapter 7 of the ASCE7-05. The calculated snow loads are shown in Appendix Figure 3 below. The governing load combination for post design is: D+.75(W) +.75(S). The maximum post axial load was a result of an unbalanced snow load that resulted from the 41 and 23.1 psf loads shown on the frame below. Dead load on the truss was estimated to be 4 and 5 psf on the top and bottom truss chords, respectively. The total dead weight of the truss was 420 lb. Uplift from section on the roof was conservatively ignored for the post design.



Appendix Figure 3. Resulting dead and snow loads on post-frame.

Max dead load acting on single post:

9psf(8 ft x 30 ft) + 420/2 = 2370 lb

(The 30 pounds accounts for half of the building width + a 2 ft eave overhang)

Max snow load acting on single post:

S_{unbal} = 41 psf(8 ft x 9.2 ft) + 23.1 psf(8 ft x 20.8 ft) = 6861 lb

From the governing load combo of: D+.75(W) +.75(S), the maximum axial force to design the posts is: 2370 + .75(6861) = **7516 lb.**

APPENDIX C: ANSI/ASAE EP484.2 METHOD EXAMPLE PROBLEM

Building Properties:

$L := 112$ ft. Building Length

$W := 56$ ft. Building Width

$h_w := 16$ ft. Post/Wall Height

$s := 8$ ft. Post/Bay Spacing

$f := \frac{3}{8}$ frame base fixity factor

$G := 4.7 \ \frac{kips}{in}$ Apparent diaphragm shear stiffness

$G_a := 19.3 \ \frac{kips}{in}$ Apparent shearwall stiffness

$pitch := \frac{3.5}{12}$

$\theta := atan\left(\frac{3.5}{12}\right) \cdot \frac{180}{\pi} = 16.26$ Roof Incline, degrees

$overhang := 2$ ft

$h_r := \frac{W}{2} \cdot pitch = 8.167$ ft

Post Properties:

$b := 4.31$ in

$d := 7.19$ in

$E := 1700000$ psi

$I := \frac{b \cdot d^3}{12} = 133.5$ in^4

Wind Pressures:

Wind pressures were calculated based on the procedures outlined in section 6.5 of ASCE 7-05. Note: Based on the calculated wind pressures, higher diaphragm forces were calculated if the roof pressures were not considered. So, for a more conservative design, and according to Note 6 of Figure 6-10, only the windward and leeward wall pressures will be used.

$K_z := .625$ Table 6-3

$K_{zt} := 1.0$ Section 6.5.7

$K_d := .85$ Table 6-4

$V := 90$ mph Figure 6-1

$I := 1.0$ Table 6-1

$q_h := .00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot I \cdot V^2 = 11.016$ psf

From Figure 6-10:

For building surface 1 (windward wall):
By interpolating between 5 and 20 degrees, GC.pf for a 16.26 roof angle is .5.

$GC_{pfww} := .5$

For building surface 2 (leeward wall):
By interpolating between 5 and 20 degrees, GC.pf for a 16.26 roof angle is -.4.

$GC_{pflw} := -.4$

It was assumed for this example that the internal pressure of the building was 0.

$GC_{pi} := 0$

$p_{ww} := q_h \cdot (GC_{pfww} - GC_{pi}) = 5.508$ psf

$p_{lw} := q_h \cdot (GC_{pflw} - GC_{pi}) = -4.406$ psf

Wind Calculation Results:

$q_{ww} := 5.5$ psf

$q_{lw} := -4.4$ psf

$q_{wr} := 0$ psf

$q_{lr} := 0$ psf

Standard Method:

Calculating Inputs for DAFI program:

1. Number of Building Bays:

$$bays := \frac{L}{s} = 14$$

2. Diaphragm Stiffness:

$$k_{dia} := G \cdot \left[\cos\left(\theta \cdot \frac{\pi}{180}\right) \cdot \frac{\left(\frac{W}{2} + overhang\right)}{s} \right] \cdot 2 \cdot 1000 = 3.384 \times 10^4 \quad \frac{k}{in}$$

3. Endwall Stiffness:

$$k_{sw} := G_a \cdot 1000 \cdot \left(\frac{W}{h_w}\right) = 6.755 \times 10^4 \quad \frac{k}{in}$$

4. Frame stiffness

The stiffness of the interior post-frames was found using visual analysis software in which an arbitrary eave load was applied to the frame and the stiffness was found by the ratio P/Δ. With P=100 lb and the corresponding Δ=.612 in., the frame stiffness was found to be 163.4 lb/in.

$$k_{frame} := 163.4 \quad \frac{lb}{in}$$

5. Eave load:

The eave load input to DAFI may be calculated using a different frame fixity factor which has be found using a plane-frame analysis program. In this case, Visual Analysis was used and determined the frame fixity factor, f, to be .393, meaning that based on the post fixity and applie wind loads, 39.3% of the applied lateral load on the walls is transferred into the building diaphragm.

$$f := .393$$

$R := s \cdot \left[h_r \cdot (q_{wr} - q_{lr}) + h_w \cdot f \cdot (q_{ww} - q_{lw}) \right] = 498.01$

The previous five values were input into the DAFI program. The resulting DAFI output is shown the the array below. Columns 1-6 represent the frame #, frame stiffness (lb/in), applied load (lb), horizontal displacement (in), load resisted by frame (lb), and fraction of load applied (lb), respectively.

1	67550	249	0.047223	3189.92	12.8109
2	163.4	498	0.13413	21.92	0.044
3	163.4	498	0.206968	33.82	0.0679
4	163.4	498	0.266089	43.48	0.0873
5	163.4	498	0.311779	50.94	0.1023
6	163.4	498	0.344257	56.25	0.113
7	163.4	498	0.363682	59.43	0.1193
8	163.4	498	0.370147	60.48	0.1214
9	163.4	498	0.363682	59.43	0.1193
10	163.4	498	0.344257	56.25	0.113
11	163.4	498	0.311779	50.94	0.1023
12	163.4	498	0.266089	43.48	0.0873
13	163.4	498	0.206968	33.82	0.0679
14	163.4	498	0.13413	21.92	0.044
15	67550	249	0.047223	3189.92	12.8109

From this output, the maximum unit shear and deflection can be found:

$v_{max} := \frac{3189.92}{W} = 56.963 \qquad \frac{lb}{ft}$

$\Delta_{max} := .370147 \qquad in$

The maximum post moment can then be found by applying the calculated wind loads to the post-frame of interest in Visual Analysis. In order to account for diaphragm action, which is the force resisting the lateral loads via the diaphram, a load Q must applied to the frame,in the opposite direction the wind force is acting. Q is found by subtracting the load resisted by the frame from the load applied to the frame. To design the posts conservatively the Q of the center frame is used because this frame has the smallest contribution of diaphragm action.

$Q := R - 60.48 = 437.53 \qquad lb.$

After applying this diaphragm force to the post-frame in Visual Analysis, the maximum resulting moment is found at the groundline:

$M_{max} := -20364 \qquad lb - in$