Permanent Bracing for
Piggyback Trusses

by Frank Woeste, P. E.

I recently gave a class on bracing of metal-plate-connected roof trusses, attended by about 50 structural engineers. What I learned there about bracing piggyback trusses literally left me sleepless that night. In case you're not familiar with the term, piggyback trusses are used when the height of a required roof truss exceeds the width limit allowed by a state's transportation department for transport on the back of a truck. Depending on the state, the limit is usually around 12 to 14 feet.

So say you need a 12/12 gable roof truss for a 40-foot span. The 20-foot ridge height will force the truss manufacturer to provide the truss in two parts — a bottom truss with a horizontal top chord, and an upper truss that completes the gable triangle (see Figure 1).

Immediately after the presentation on bracing of piggyback systems, one structural engineer produced a set of photographs of a piggyback truss roof in a heap on the hardwood floor of a school gymnasium. The roof collapse occurred with only 5 inches of snow on the roof — there was no ice nor any other load factors. Fortunately, there were no children playing in the gym at the time of the collapse.

All the engineers that spoke up agreed that the sole cause of the collapse was

Incorrect Piggyback Bracing

*Drawings are not to scale. They are intended to illustrate permanent bracing for top chord C only. Permanent bracing for other chord and web members not shown. Engineered connections between piggyback and supporting trusses are not shown.
incomplete bracing of the piggyback system. Lateral bracing had been installed on the supporting truss as depicted in Figure 1. What was lacking was diagonal bracing.

Confusion About Truss Bracing

Temporary and permanent bracing of trusses are critical to the success of a wood truss system from the day of installation through the life of the roof. Yet confusion about bracing is common among builders and architects. The term "truss bracing" is ambiguous because it does not differentiate between temporary bracing and permanent bracing.

To help clear up the confusion, I suggest we avoid using the term "truss bracing" altogether, and refer instead to "temporary bracing" and "permanent bracing." Temporary bracing is the bracing used by the contractor to safely erect the trusses, and it may include elements of the permanent bracing system. Permanent bracing is the bracing required in the roof system to stabilize the trusses throughout the life of the structure.

For example, 2x4s used on the top chords of simple gable trusses to prevent the top chords from buckling during construction are temporary bracing. When the 2x4s come off and the structural sheathing goes down, the sheathing becomes the permanent bracing for the top chord. (Other permanent bracing may be required for other chord and web members.) When trusses are erected in sections on the ground with the plywood already attached, the sheathing serves both functions — temporary bracing and permanent bracing.

Contractors are responsible for determining the necessary temporary bracing, but not the permanent bracing. Permanent bracing should be specified by the building designer.

Back to the Gym

So what happened to the school gym trusses? Under the snow load (as small as it was), truss chord C in Figure 1 is in compression. Imagine what happens when you hold a yardstick on edge between your hands and push your hands together: The yardstick buckles.

The same thing happened to chord C. Chances are the failure happened suddenly, without warning. When the chord buckled, the truss lost its integrity and fell in on itself. The piggyback section fell to the floor with the supporting truss.

Note the lateral braces in Figure 1. This is how some builders incorrectly brace chord C in a piggyback system, assuming perhaps that the roof sheathing on the rest of the roof will prevent lateral movement. This is not the case, however. As the top chord C displaces under load, the lateral braces simply move with the top chord. What is required are additional diagonal braces, nailed to the bottom of chord C, which act with the lateral braces to form a series of triangles stiffening the top chords. Without the diagonals, the lateral braces across the top chords are like...
Retrofit Piggyback Bracing

In some cases, installing strips of plywood between existing piggyback trusses may serve to stabilize the permanent lateral braces. The building designer or engineer should specify the proper nailing.

Proper Permanent Bracing

Figure 2, previous page, illustrates two ways to provide proper permanent bracing for chord C. (Note that this article is concerned only with bracing top chord C, because this is the most common problem area for piggyback systems. Other chord and web members also require permanent bracing.)

The top sketch shows how to use diagonal 2x4s to stabilize lateral braces like the ones used in the gym trusses. The cross-bracing is attached to the bottom of chord C, installed at a 45-degree angle to chord C.

The bottom sketch in Figure 2 shows my preferred method of piggyback bracing — installing structural sheathing across the top chords of the lower truss (you may have to make some allowance for ventilation). After the sheathing is installed on chord C, the piggyback truss can be installed and attached to the supporting truss in accordance with the truss designer’s specifications. It is the responsibility of the truss designer to specify the connection between the piggyback truss and the supporting truss. If this connection has not been spelled out on the truss design drawings, contact the truss manufacturer for the detail.

What If You’ve Built It Wrong?

Over the years, I have heard contractors comment that permanent bracing requirements appear excessive and that there seems to be as much lumber used for bracing as for the trusses. This is an exaggeration, of course, but it is an attitude that may lead builders to ignore the permanent bracing specifications.

So, what if you’ve built a piggyback roof that’s braced like the roof in Figure 1? “If it hasn’t collapsed yet, it’s probably okay” is flawed logic. Whether that top chord C buckles or not is dependent on the compressive load level present in the chord under gravity load. It may take 50 years to get the load level needed to buckle the chord, but it might also happen next week. If you have piggyback trusses with only lateral braces as shown in Figure 1, you should put down this magazine and contact the building designer immediately.

Retrofit Piggyback Bracing

It is possible to install bracing in a piggyback roof that wasn’t braced properly when it was built. You should follow specifications provided by the building designer. The retrofit may take the form of cross-bracing, as in Figure 2, or you might be able to install plywood strips between the trusses, nailed directly to the lateral braces, as in Figure 3. Again, you should follow the designers nailing recommendations, and you may have to provide for roof ventilation.

Summary

Using a piggyback truss system versus a standard truss system is not an incidental framing decision. The contractor should determine the temporary bracing to safely erect the trusses and the building designer should specify the permanent bracing needed to support in-service loads.

A point to remember is that the building designer cannot complete the permanent bracing drawings until he approves the truss design shop drawings used to manufacture the trusses.

It is not uncommon for an architect to specify roof truss bracing “per HIB-91.” This specification should alert you to a problem because HIB-91, “Handling, Installing, & Bracing Metal-Plate-Connected Wood Trusses” (published by the Truss Plate Institute; 608/833-5900), is about temporary bracing, not permanent bracing. Any note by the building designer that does not tell you specifically what permanent truss bracing to install should also be a warning. The truss manufacturer may not be able to respond to your questions since they are not responsible for design of permanent truss bracing.

Do not install piggyback trusses unless the building designer specifies the necessary permanent bracing for the roof system. If you are both the contractor and the building designer, but not an engineer with truss bracing experience, you should contact an engineer familiar with permanent bracing design. Your truss manufacturer should know a local or regional engineer.

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**2% Rule for Bracing**

The column (web) is assumed to be one-inch out of plumb for an assumed story height of 100-inches and pin connected (three pins). A force balance yields the 2% rule with a small angle approximation as follows.

\[
P = \text{column load (pounds)}
\]
\[
F_{br} = \text{brace force (pounds)}
\]
\[
\alpha_1 = \text{angle between brace and vertical plane}
\]
\[
\alpha_2 = \text{angle between brace and vertical plane}
\]

**Force Balance**

Assuming pins at brace ends and at the point of brace attachment:

\[
F_{br} = P \sin(\alpha_1) + P \sin(\alpha_2)
\]

When \(\alpha_1\) and \(\alpha_2\) are small and equal, \(\sin \alpha\) approximately equals \(\tan \alpha\). From Throop (1947), \(\tan \alpha\) was assumed to be 1/100, therefore:

\[
F_{br} \approx (1/100 + 1/100) P
\]
\[
F_{br} \approx 0.02P, \text{ or 2\% or } P
\]

Underwood, Catherine R. 2000. Bracing Design for a MPC Wood Truss Member with Numerous Brace Locations. MS Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.
PERMANENT BRACING DESIGN FOR MPC WOOD ROOF TRUSS WEBS AND CHORDS

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ABSTRACT
Permanent bracing of metal-plate-connected trusses is required to stabilize specific members of each truss throughout the life of the roof structure. The objectives of this research were to: 1) determine the required net lateral restraining force to brace multiple webs or chords in a row braced by one or more continuous lateral braces (CLBs); and 2) develop a methodology for permanent bracing design using a combination of lateral and diagonal braces. Three system analogs used to model multiple truss chords braced by \( n \)-CLBs and one or two diagonal braces were analyzed with a finite element analysis program. Single member analogs were analyzed that represented web members braced by one and two CLBs and chord members braced by \( n \)-CLBs. For analysis and design purposes, a ratio \( R \) was defined as the net lateral restraining force per web or chord divided by the axial compressive load in the web or chord. For both 2 by 4 and 2 by 6 webs braced with one CLB, the \( R \)-value was 2.3 percent for all web lengths studied, and for webs braced with two CLBs, the \( R \)-value was 2.8 percent for all web lengths studied. Calculated \( R \)-values for truss chords braced by \( n \)-CLBs, assumed to be spaced 2 feet on center, ranged from 2.3 to 3.1 percent. A design procedure was offered for determining the net lateral restraining force required for bracing multiple webs or chords in a row based on the results of the single member analogs studied.

Consideration of permanent and temporary bracing of metal-plate-connected (MPC) wood trusses is critical for safety during erection and for reliable performance of a roof structure in service. Temporary bracing is used to position and stabilize trusses until permanent bracing or other building components can be installed. Permanent bracing, the subject of this paper, is required to stabilize specific members of each truss throughout the life of the roof structure. Based on truss design assumptions, various chords and webs require lateral support. The chords that require sheathing or the members that require lateral support at a specified interval are indicated on the truss design drawing (5). Bracing, whether it is temporary or permanent, is required to help prevent the trusses from deflecting laterally and potentially causing the trusses to topple over or collapse (3, 15, 16, 19).

Permanent truss bracing can include several different components, but are typically designed using one of the following options: 1) continuous lateral braces (CLBs) at required brace locations in conjunction with diagonals; or 2) properly nailed sheathing, either oriented strandboard (OSB) or plywood. If carefully designed, some elements of temporary bracing (11) can serve dual roles and become permanent bracing also.

LITERATURE REVIEW
The building designer is responsible for permanent bracing design of MPC wood trusses (12). The Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses (5) provides typical bracing design strategies. The Commentary does not provide a specific design procedure, but it depicts the various locations in roof truss constructions that typically require a permanent bracing design and installation. The Commentary addresses common truss configurations and provided various permanent...
bracing options. Although some permanent lateral bracing information may be shown on truss drawings, the building designer is responsible for the complete bracing design and details needed by the contractor to properly install the permanent bracing system. For the case where a roof truss has a long span or a high pitch and a piggyback truss is needed for shipping purposes, the building designer must provide a permanent bracing design and specifications for the supporting trusses. The Commentary provides several alternatives for bracing the flat compression chord of the supporting truss that includes the use of CLBs in conjunction with diagonal braces. No design guidelines are given for determining the necessary capacity of the diagonals needed to stabilize the CLBs.

Currently, the 2 percent Rule is the most common and accepted design practice used for designing permanent bracing for a compression web that utilizes one CLB with diagonals spaced at some interval. The 2 percent Rule is a strength model based on a percentage of the axial load in a column needed to stabilize the column (17). Designers primarily use strength models during design due to the simplicity of the calculations. In the derivation of the 2 percent Rule, the column was assumed to be pinned at each end and at the center brace location. The column is assumed to be 1 inch out of plumb for an assumed story height of 100 inches (9). A compression web or chord braced at the center of its span will have an initial column slope of 1/100 above and below the brace (9,17). A force balance for a free-body diagram at the brace and chord connection is the basis for the 2 percent Rule (6).

Waltz (17) studied the design requirements for bracing a 2 by 4 compression web member at one brace support point at the center of the web. Waltz tested Select Structural and Standard Douglas Fir-Larch (DFL) specimens axially loaded and braced by his computer-controlled testing apparatus to determine the required brace force. Using finite element (FE) analysis, Waltz estimated the stiffness of the support provided by a lateral and diagonal bracing system for a number of braced 2 by 4 truss webs in a row. His research objective was to determine if one of four existing models could be used to estimate the required brace strength and stiffness. The four existing bracing models considered included Plaut’s method (7,8), Winter’s method (18), the 2 percent Rule (9,10, 11), and Tsien’s method (13). Waltz (17) concluded that either Plaut’s or Winter’s method could be used for web bracing design.

**OBJECTIVES**

For permanent bracing design purposes, a simple “hand calculation” method was desired to approximate the brace force required to stabilize numerous compression webs braced by one or two CLBs or chords braced by CLBs at multiple locations. The objectives of this research were to: 1) determine the required net lateral restraining force to brace multiple webs or multiple chords in a row braced by one or more CLBs; and 2) to develop a methodology for permanent bracing design using a combination of lateral and diagonal braces. In addition to the primary objectives of this research, the impact of lumber specific gravity, modulus of elasticity, and chord lumber size on required bracing forces were investigated.

**ANALYSIS TOOLS AND ASSUMPTIONS**

SAP2000 (2) is a finite element analysis program used for analyzing different types of problems, including structural analysis problems. In this research, SAP2000 was used with the assumption that beam elements were linear elastic. A minimum of three beam elements were used to represent the compression members.

To create the structural analogs used in this research, the following assumptions were made:

- The CLBs and diagonal braces were dry 2 by 4 nominal Stud grade spruce-pine-fir (SPF) lumber;
- The ends of the braced webs or chords were pin connected; therefore, pin or roller reactions were used to support the member ends;
- 2-16d Common nails were used for all wood-to-wood connections;
- Truss chords in a row and multiple CLBs used for bracing the chords were spaced 2 feet on center;
- Truss webs were braced with one or two CLBs at the center or third points of the length, respectively.

The load-displacement relationship for a single nail connection made with SPF lumber was determined by using Equation [1] from Mack (4):

\[ F_n = S_2 d^{1.75} k_s (3.20 \Omega + 0.68) (1 - e^{-75 \Omega})^{0.7} \]

where \( F_n \) = the load per nail (lb.) applied to a single shear nail joint; \( \Omega \) = the slip between the two members of the nail joint (in.); \( d \) = the nail diameter (in.); \( k_s \) = the species constant from Mack (4).

Since Mack’s (4) paper did not provide a species factor for SPF, a linear regression was performed to determine the equation suitable for that particular species using available data. Equation [2] gives the variables and results from a linear regression of 10 pairs of \( k_s \) and specific gravity (SG) data reported in Mack (4):

\[ k_s = 265 \times SG + 3.5 \]

where \( k_s \) = the species factor, and for use in Equation [1]; SG = specific gravity for the species.

From Equation [2], \( k_s \) equals 115 for SPF that has a published SG value of 0.42 (1). The right-hand side of Equation [1] must be doubled because the stiffness of a two-nail joint is assumed to be twice as stiff as a single-nail joint. And finally, substituting \( d \) equals 0.162 inches for a 16d Common nail into Equation [1] and by applying a factor of 1.25 for dry material as referenced in Mack, Equation [3] results:

\[ F_{2n} = 618 \times (3.20 \Omega + 0.68) (1 - e^{-75 \Omega})^{0.7} \]

where \( F_{2n} \) = the load applied to a 2-16d Common nail joint (lb.); \( \Omega \) = the joint slip between the SPF members (in.).

Equation [3] was implemented in the bracing analysis by using the secant modulus at different joint load levels. To find the secant modulus, a slip was calculated using SAP2000 and then the corresponding joint force was calculated using Equation [3]. The joint force \( g(x) \), where \( x \) is slip, was determined by a line drawn from the origin to the point \((x, g(x))\). The slope of the line, called the secant modulus, was the stiffness for the specified joint force and slip.

A linear spring stiffness was estimated for each joint of the structural analog, and then the structural analog was re-analyzed. Calculated spring forces were then compared to the specific force and displacement used to input the lin-
Continuous lateral brace, Top chord IP, compression, Bottom chord 

Figure 1.—Truss webs are braced with one CLB and one diagonal that crosses the truss webs.

ear spring (secant modulus) constants. A new stiffness value for each spring was determined based on the new deflections and entered into SAP2000 (2). The structure was analyzed again. This procedure was repeated until the force in the springs matched the assumed force and displacement used to calculate the secant modulus spring stiffness within a tolerance of 1 percent. This iterative procedure was necessary to accommodate a nonlinear spring (nail connection) in the SAP analysis of the braced system.

For the purpose of discussion and comparison to the 2 percent Rule, the net lateral restraining force from the web and chord analyses were divided by the axial compression load in the web or chord, respectively. This ratio will be referred to as $R$ and is defined by Equation [4]:

$$ R = \frac{\text{Net lateral restraining force (lb.)}}{\text{Axial load level in web or chord (lb.)}} \quad [4] $$

where $R$ = the variable of interest for both webs and chords.

**WEBS BRACED WITH ONE CLB**

The structural analog used to analyze a multiple number of webs in a row ($j$) braced with one CLB is depicted in Figure 1. To simplify the structural analog for the case of multiple truss webs in a row, the diagonal brace was neglected and a single truss web was analyzed. The response of a multiple web case encountered in actual construction can be obtained by multiplying $j$ times the response from the single web case. The structural analog was represented in SAP2000 (2) as depicted in Figure 2, where the length of the web was varied from 3 to 12 feet. The lumber was assumed to be 2 by 4 Stud grade SPF with a modulus of elasticity (MOE), of 1,200,000 psi. The load applied to the web ($P$) varied from 10 to 100 percent of the allowable load based on the assumption that the webs are pin-connected to the chords and that effective web buckling length is 80 percent of the actual web length (1.12).

A second analog was created to determine the effects of web size, if any, on the required bracing force. The lumber was assumed to be 2 by 6 nominal Stud grade SPF with an MOE of 1,200,000 psi. The allowable load ($P$) applied to the web was recalculated based on the web size and the procedures presented in the NDS for column design (1,12).

A third structural analog was created for the one web braced by one CLB case to determine if lumber species (as characterized by MOE and SG) affected the required brace force. Using the analog shown in Figure 2, 2 by 4 nominal No. 2 DFL was assumed for the web and CLB. DFL has a 17 percent higher MOE value than SPF. The published SG value for DFL is 0.50 versus 0.42 for SPF. DFL was chosen as the species to compare to SPF because the nail slip data were available for a DFL joint and because of the higher SG value.

ANSI/TPI 1-1995 (12) installation limits were assumed for all web lengths studied. All web members were assumed to have equal initial curvature in the same direction (for example, all webs bowed left). The assumption for initial curvature of all webs is conservative because in reality truss webs would have initial deflections in both directions (bowed right and bowed left). Initial curvature in both directions (with a CLB installed) would help provide support to the overall structure. Web members were assumed to have an initial maximum deflection of $L/200$ and have initial curvature matching a half sine wave defined by Equation [5]:

$$ \Delta_i = \frac{L}{200} \sin \left( \frac{\pi x}{L} \right) \quad [5] $$

where $\Delta_i$ = assumed initial deflection of the truss webs; $L$ = length of the compression chord, and $x$ = distance from the member end in inches; $\pi$ is in radians.
Continuous lateral braces

**Figure 3.** — Truss webs are braced with two CLBs and one diagonal that crosses the truss webs.

**TABLE 1.** — Summary of truss web cases studied involving one web braced by one CLB.

<table>
<thead>
<tr>
<th>Nominal lumber size</th>
<th>Range of web lengths investigated</th>
<th>Increments</th>
<th>Web lumber species</th>
<th>CLB lumber species</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 by 4</td>
<td>3 ft. to 12 ft.</td>
<td>1 ft.</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 6</td>
<td>3 ft. and 12 ft.</td>
<td>N/A</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 4</td>
<td>3 ft. and 12 ft.</td>
<td>N/A</td>
<td>DFL</td>
<td>DFL</td>
</tr>
</tbody>
</table>

* SPF = spruce-pine-fir; DFL = Douglas-fir-larch.

A deflected web member, by nature, is a smooth curve as opposed to a series of straight lines. Therefore, the half sine configuration was assumed and used in all web and chord analyses. A summary of the cases studied for one web braced by one CLB is given in Table 1.

**RESULTS (WEB WITH ONE CLB)**

*R*-values for the case of one web braced by one CLB were 0.023 (2.3%) for all web lengths and load levels (ranged from 10% to 100% of the allowable compression for the assumed lumber grade). *R*-values were not affected by lumber species (higher MOE and SG) or by using a 2 by 6 versus 2 by 4 web (14). The difference in the 2 percent Rule versus 2.3 percent results from this study, is due to the fact that a flatwise 2 by 4 is very flexible, and thus not dramatically affected by member continuity. The computer analog constructed for this research did not have a pin joint at the center of the web as is assumed for the derivation of the 2 percent Rule.

**WEBS BRACED WITH TWO CLBS**

For the case of one web braced by two CLBs shown in Figure 3, the structural analog included the same modeling assumptions as the structural analogs representing one web braced by one CLB. The structural analog was represented in SAP2000 (2) as depicted in Figure 4, where the length of the web varied from 5 to 12 feet. The lumber was assumed to be 2 by 4 nominal Stud grade SPF with an MOE of 1,200,000 psi. Table 2 contains a summary of the cases studied for one web braced by two CLBs; the study objectives and analysis methods for each case were identical to the case of one web braced by one CLB.

**TABLE 2.** — Summary of truss web cases studied involving one web braced by two CLBs.

<table>
<thead>
<tr>
<th>Nominal lumber size</th>
<th>Range of web lengths investigated</th>
<th>Increments</th>
<th>Web lumber species</th>
<th>CLB lumber species</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 by 4</td>
<td>5 ft. to 12 ft.</td>
<td>1 ft.</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 6</td>
<td>5 ft. and 12 ft.</td>
<td>N/A</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 4</td>
<td>5 ft. and 12 ft.</td>
<td>N/A</td>
<td>DFL</td>
<td>DFL</td>
</tr>
</tbody>
</table>

* SPF = spruce-pine-fir; DFL = Douglas-fir-larch.

**RESULTS (WEB WITH TWO CLBS)**

This case, consisting of one web and two CLBs, produced an *R*-value of particular interest. In the past, one option for design purposes was to assume the bracing force was equal to 2 percent of the applied load, times the number of CLB connections per web, which yields 4 percent of the axial load as the required bracing force per web. The *R*-value was determined to be 0.028 (2.8%) for all lengths and axial load levels studied. The 2.8 percent *R*-value is substantially less than the 4 percent calculated by the assumption that the brace force increases in proportion to the number of CLBs. The *R*-values were not affected by increasing the web lumber MOE and

**Figure 4.** — Structural analog representing one web braced by two CLBs. The nailed connections are represented by springs and an applied load (P) is a compression force determined using design equations provided in the NDS (1).
Truss chord bracing by many CLB's.

Figure 5.—Truss chords are braced using one diagonal that is installed at about 45 degrees to the chords.

Figure 6.—Structural analogs as analyzed by SAP2000 (2) for the case of n-CLBs spaced 24 inches on center, with an applied axial load (P) and the truss chord is length (L).

SG values or increasing the web size to a 2 by 6 (14).

**Chords braced by n-CLBs**

Figure 5 is a line representation of an actual set of j truss chords braced by n-CLBs and one diagonal brace. In this research, the cases of one and two diagonal braces were investigated. A total of 10 structural analogs were developed based on the varying lengths of lumber. The diagonal braces were neglected in the first phase of the investigation. Figure 6 depicts the structural analog models as they were analyzed using SAP2000 (2). Structural analogs were constructed using the same boundary conditions and assumptions for connections between each CLB and chord as were made for the cases of one web with one and two CLBs.

The center panel (or two panels) of a symmetric truss with a flat-top chord under symmetric loading will have the maximum stress interaction according to the NDS (1) when the truss panel lengths are equal. Equation [6] gives the stress interaction criterion for a chord subjected to bending and compression load.

\[
\left( \frac{f_c}{F_c'} \right)^2 + \left( \frac{f_b}{F_b'(1-F_{cb})} \right) \leq 1 \quad [6]
\]

where \( f_c \) = actual compression stress parallel to grain (psi); \( F_c' \) = allowable compression design value parallel to grain (psi); \( f_b \) = actual bending stress (psi); \( F_b' \) = allowable bending design value (psi); \( F_{cb} \) = critical buckling design value for a compression member for the applicable \( l_e/d \) ratio (psi).

From a permanent bracing designer standpoint, one needs to determine the maximum axial force in all the panels of the chord. When an unbraced truss chord is assumed to be continuous in the structural analysis, bending moments will exist in all panels. The amount of bending moments will vary in magnitude from one design to the next. A conservative assumption with respect to permanent bracing design is that the bending moment is zero in all panels. Since the bracing is designed to resist a percentage of the axial load in the web, the higher axial load due to this assumption results in a higher design load in the brace. If the stress interaction is at the maximum equal to 1.0, Equation [6] reduces to Equation [7]:

\[
f_c = F_c' \quad [7]
\]

Equation [7] applies to the center panel (or two symmetrical panels). It is conservative because assuming zero bending moment allows for the maximum applied axial compression load to be present in the assumed chord. The design compression load \( (C) \) in the center panel (or two symmetrical panels) is therefore given by Equation [8]:

\[
C = A \times F_c' \quad [8]
\]

where \( A \) = chord area (in. \(^2\)); \( F_c' \) = allowable compression design value parallel to grain (psi).

Axial load in the outer panels under the symmetry assumptions stated previ-
\[ \Delta_i = 2 \times \sin \left( \frac{\pi x}{L} \right) \]  

ANSI/TPI 1-1995 (12) states that the maximum initial deflection allowed in a truss chord is the lesser of L/200 or 2 inches. In cases where compression chord length \((L)\) is greater than 400 inches, the 2-inch maximum allowance was observed. Table 3 summarizes the truss chords modeled for the investigation of truss chords braced by \(n\)-CLBs.

The same analysis and procedures as were used for the case of a braced web were used to analyze chords with \(n\)-CLBs, except the chords were assumed to be No. 2 grade Southern Pine lumber. For calculations of the nail slip of the 2-16d Common nail connections, it was assumed that both the chord and the CLB were SPF because nail slip data were not available for a joint having mixed species. This assumption will result in lower stiffness values for the connection, which is conservative.

### Table 3. Summary of truss chord cases studied involving chords braced by \(n\)-CLBs.

<table>
<thead>
<tr>
<th>Nominal lumber size</th>
<th>Range of chord lengths investigated</th>
<th>Increments</th>
<th>Chord lumber species</th>
<th>CLB lumber species</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 by 4</td>
<td>4 ft. to 24 ft. including 6 ft.</td>
<td>4 ft.</td>
<td>SP</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 6</td>
<td>4 ft. to 36 ft. including 6 ft.</td>
<td>4 ft.</td>
<td>SP</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 8</td>
<td>4 ft. to 36 ft. including 6 ft.</td>
<td>4 ft.</td>
<td>SP</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 10</td>
<td>4 ft. to 40 ft. including 6 ft.</td>
<td>4 ft.</td>
<td>SP</td>
<td>SPF</td>
</tr>
<tr>
<td>2 by 12</td>
<td>4 ft. to 40 ft. including 6 ft.</td>
<td>4 ft.</td>
<td>SP</td>
<td>SPF</td>
</tr>
</tbody>
</table>

*SP = southern pine; SPF = spruce-pine-fir.*

### Table 4. Net lateral bracing force (lb.) divided by the axial load (lb.) for comparison to the 2 percent Rule for 2 by 4 Southern Pine truss chords.

<table>
<thead>
<tr>
<th>Chord length ((n+2)) (^a)</th>
<th>Lateral force produced in the (n)-web-CLB connections (lb.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft.)</td>
<td>Axial load level in chord (lb.)</td>
</tr>
<tr>
<td></td>
<td>10(^b)</td>
</tr>
<tr>
<td>4</td>
<td>0.023</td>
</tr>
<tr>
<td>6</td>
<td>0.028</td>
</tr>
<tr>
<td>8</td>
<td>0.028</td>
</tr>
<tr>
<td>12</td>
<td>0.03</td>
</tr>
<tr>
<td>16</td>
<td>0.031</td>
</tr>
<tr>
<td>20</td>
<td>0.031</td>
</tr>
<tr>
<td>24</td>
<td>0.031</td>
</tr>
</tbody>
</table>

\(a\) When \(n\)-CLBs are used, one additional brace is typically installed on each end of the chord.

\(b\) Percent of maximum allowable axial compressive load in the truss chords.
$R$-values for the shorter lengths. $R$ was 0.029 (2.9%) for the 36-foot Southern Pine chord with $n$-CLBs spaced 24 inches on center, and $R$ was independent of lumber size. $R$ was equal to 0.026 (2.6%) for the 40-foot Southern pine truss chord with $n$-CLBs spaced 24 inches on center, and $R$ was independent of lumber size. The values for $R$ for chord lengths ($L$) greater than 400 inches were different due to the maximum initial member deflection of 2 inches (14).

In addition, the chord load level as a percent of $F_c'$ did not affect $R$ for any size or length. The SAP2000 analysis was based on linear elastic beam elements with nonlinear springs representing the nail connections, and thus one would expect the system to behave in a non-linear manner. However, the nail connections at low load levels are apparently so stiff that the calculated value of $R$ is not substantially affected by the load level (14).

**SYSTEM ANALOG: EFFECT OF INCLUDING DIAGONALS WITH MULTIPLE CHORDS**

"System" structural analogs were composed of $j$ chords, $n$-CLBs, and diagonal braces. The first system structural analog analyzed with SAP2000 (2) represented five 8-foot roof truss chords spaced 24 inches on center, three continuous lateral braces (CLBs) spaced 24 inches on center, and one diagonal (Fig. 7). For this case and the remaining two cases, the lumber used in the construction of the structural analog consisted of 2 by 4 Stud grade SPF, with an MOE of 1,200,000 psi for the CLBs, and 2 by 4 nominal No. 2 Southern Pine, with an MOE of 1,600,000 psi for the truss chords. The truss chords were assumed to be columns with pin supports at one end and roller reactions supports at the other (14).

An axial chord load was applied to all chords in the analysis. The maximum allowable design load ($4 \times F_c'$) was determined to be 6,842 pounds based on NDS procedures for column design (1). However, 50 percent of the allowable compressive load based on an $I_p/d$ ratio of 16 is a typical load level in a wood truss chord. For the analyses of the required bracing forces, the chord axial load level was increased from 10 to 50 percent of $F_c'$. Maximum allowable axial load was based on the grade, species combination, size of lumber, and the duration of

---

![Figure 7](image7.png)

Figure 7. — The structural analog represents five trusses with an initial curvature, three CLBs, and one diagonal member. The initial curvature is in one direction and is exaggerated for visual purposes.

![Figure 8](image8.png)

Figure 8. — The structural analog represents six trusses with an initial curvature, nine CLBs, and two diagonal bracing members in a V shape. The initial curvature is in one direction and exaggerated for visual purposes.
TABLE 5. — Net lateral forces (lb.) produced by Southern Pine truss n-chords braced by multiple Spruce-Pine-Fir (SPF) CLBs and one or two SPF diagonal(s).

<table>
<thead>
<tr>
<th>Length of chords (ft.)</th>
<th>No. of trusses</th>
<th>No. of CLBs</th>
<th>No. of diagonal braces</th>
<th>Applied axial compressive load from 10% to 50% of allowable load (lb.)a</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>91 182 272 363 453</td>
</tr>
<tr>
<td>20</td>
<td>6</td>
<td>9</td>
<td>2</td>
<td>124 247 367 486 602</td>
</tr>
<tr>
<td>20</td>
<td>11</td>
<td>9</td>
<td>2</td>
<td>221 434 637 824 983</td>
</tr>
</tbody>
</table>

a Based on L/ld equals 16 and 2 by 4 No. 2 Southern Pine truss chords.
b Joint slip was greater than the theoretical failure limit for a 2-16d Common nail connection.

Figure 9. — The structural analog represents 11 trusses with an initial curvature, 9 CLBs, and 2 diagonal bracing members in a V shape. The initial curvature is in one direction and exaggerated for visual purposes.

In summary, for the three cases studied with chord loads from 10 to 50 percent of the allowable $F_c'$, the predicted net lateral bracing force by the single member analysis was greater than the bracing force predicted by the system analog analysis.

CONCLUSIONS

For design purposes, $R$ is the ratio between the NLRF (lb.) and the axial load level in the web or chord (lb.). $R$-values of 2.3 percent for the case of one web braced by one CLB, 2.8 percent for one web braced by two CLBs, and a conser-
ative value of 3.1 percent for j truss chords braced by n-CLBs were determined using structural analyses for single-member analogs. Based on the results of this study, it was concluded that the length of the web member did not affect the R-values; however, the chord length had a significant effect on R-values. The chord length affected the R-values because the numbers of brace locations (24 in. on center) along the chord were based on the length of the chord, whereas the web brace locations were fixed at the center and 1/3 points of the web member.

Based on the three cases studied involving 2 by 4 chords braced as a unit (and believed to be representative of typical truss construction), the bracing force from the single member analog analysis was a conservative estimate for bracing design purposes. Based on other single member analog studies in this research (14) that showed chord size and chord lumber (characterized by MOE and SG) did not affect bracing force ratio, it was concluded that the single member analysis analog will yield approximate bracing forces for chords greater than 2 by 4 and for typical constructions beyond the three cases studied in this research. It is believed that the presence of a diagonal brace(s) stiffens the braced set of j chords and thereby reduces the net lateral force required to brace the j chords compared to the required bracing force from the single member analysis. It is not practical to attempt to analyze all possible combinations of truss lumber and bracing scenarios (j chords braced either by a V diagonal or a single diagonal, and all possible spans and chord load levels). The R-values reported for webs and chords can be used by permanent bracing designers since the R-values are based on a rational engineering analysis.

**Design Considerations**

Based on the results of this study (14), the following design rules may be used for calculating the required NLRF for webs and chords when utilizing CLBs in combination with one or two diagonal braces. R is the ratio between the net lateral restraining force (lb.) and the axial load in the web or chord (lb.).

When designing braces for j webs in a row, the required NLRF for j webs braced by one CLB can be calculated as follows:

\[
NLRF = \frac{j \times 2.3\% \times \text{maximum axial force in web from all load combinations}}{\text{where } j \text{ is the number of identical webs in a row to be braced.}}
\]

When designing braces for j webs in a row, the required NLRF for j webs braced by two CLBs can be calculated as follows:

\[
NLRF = \frac{j \times 2.8\% \times \text{maximum axial force in web from all load combinations}}{\text{where } j \text{ is the number of identical webs in a row to be braced.}}
\]

When designing permanent bracing for j chords in a row, the required NLRF for j chords braced by n-CLBs can be approximated by:

\[
NLRF = j \times R \times \text{maximum axial force in chord from all load combinations}
\]

where j = number of identical truss chords in a row to be braced. An R-value of 3.1 percent is conservative with respect to the variable chord length since for chord lengths between 40 and 40 feet evaluated using the single member analog, 3.1 percent was the maximum R-value obtained (14).

**Literature Cited**

10. Truss Plate Institute. 1989. Recommended design specification for temporary bracing of metal plate connected wood trusses. DSB-89. TPI, Madison, WI.
11. 1991. Commentary and recommendations for handling, installing and bracing metal plate connected wood trusses. HIB-91 Pocketbook. TPI, Madison, WI.
Bracing Summary

We studied 2x4 and 2x6 webs with an assumed bow of \( L/200 \) and 2-16d Common nails per CLB-web connection.

- For one CLB, required bracing force was 2.3\% of \( C \) per truss.

- For two CLB’s, required bracing force was 2.8\% of \( C \), or 1.4\%\( C \) per CLB per truss.

We studied 2x4 through 2x12 chords, braced at 2-ft on-center, with an assumed bow of \( L/200 \). Nail connections were 2-16d Common nails. Chord force, \( C \), was conservatively assumed to be \( A^*F/C \) (no bending assumed).

- For \( n \)-CLB’s at 2-ft. on center, maximum and total required bracing force was 3.1\%\( C \).

Example: Piggyback flat truss chord with 15,000 lbs., 8 CLB’s at 2-ft. oc.

a) Assuming one section of 10 trusses, what is the necessary lateral load design capacity for two \textit{diagonals under the top chord}?

b) Assuming the diagonals are installed at 45-deg., what is the design load in each diagonal?