AN ABSTRACT OF THE THESIS OF

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Abstract approved: Thomas E. McLain

Thomas H. Miller

As part of a roof framing system, light-frame wood trusses often require lateral bracing to reduce the effective length for flexural buckling of compression web members. This observational research investigates the design requirements for discrete compression web bracing intended to provide such lateral support. Four simple brace analysis methods based on rules-of-thumb and theoretical means were compared with the results from a physical test program to determine if any could be used to predict the brace strength and stiffness requirements. These methods include those developed by Plaut, Winter, and Tsien. In addition, the popular "2%" rule-of-thumb was investigated. For the test program, the lateral brace force and deflection were measured at mid-height for 774 2x4 Douglas-fir columns of four different lengths (4, 6, 8, and 10 foot) and two different lumber grades ("Select Structural" and "Standard). The testing incorporated a practical range of brace stiffnesses estimated by finite element analysis to characterize the support offered by a traditional lateral/diagonal bracing system. The brace force and deflection were measured for each column at an axial load equal to the estimated 5% exclusion strength limit for an effectively braced column of a given length and grade.

We found little practical effect of column length and grade on the relative accuracy of the four brace analysis methods. However, a statistically significant length effect was observed for all but the 2% rule-of-thumb. Since brace instability was observed when the lateral load extended into the non-linear range of brace support in about 1% of the tests, we recommend limiting the brace load to a level below the

proportional limit of the brace assembly. The 2% rule-of-thumb was found to be the most conservative method of predicting the brace force in this test program. However, it may not be appropriate for brace design because it does not ensure sufficient brace stiffness to achieve the desired column strength. Given its complexity and comparative inaccuracy as a predictor of the brace support requirements when compared with Winter's method, use of Tsien's equation is not recommended. The results from this study suggest that, with some further modification to achieve design conservatism, the methods developed by Plaut and Winter provide a rational basis for discrete compression web bracing design.

Discrete Compression Web Bracing Design for Light-Frame Wood Trusses

by

Miles E. Waltz, Jr.

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The following symbols find use in this thesis:

A	= member cross-sectional area (in.^{2})
b	= member thickness (inches)
c	= column curve constant (unitless)
C_D	= load duration factor (unitless)
C_F	= size factor (unitless)
C_M	= wet service factor (unitless)
C_t	= temperature factor (unitless)
C_P	= column stability factor (unitless)
C_{PT}	= preservative treatment factor (unitless)
C_{rt}	= fire-retardant treatment factor (unitless)
d	= member depth (inches)
D_{bend}	= center span deflection for the flatwise bending E test (inches)
D_{theory}	= performance variable where *theory* = Plaut, Winter, Tsien, or 2% Rule (unitless)
E	= longitudinal modulus of elasticity of the column (psi)
E_t	= tangent longitudinal modulus of elasticity for the column (psi)
E_{05}	= 5% percentile longitudinal modulus of elasticity for the column (psi)
F_{br}	= brace force (pounds)
$(F_{br})_{est.}$	= brace force predicted by a given brace theory at the observed maximum lateral deflection Δ_{actual} (pounds)
$(F_{br})_{actual}$	= brace force measured for a given test column at the maximum axial test load P (pounds)
$F_{br\text{-}req}$	= required brace strength (pounds)
$F_{br\text{-}ult}$	= ultimate strength of the provided brace at the linear limit (pounds)
F_c	= allowable or reference compression strength parallel to grain depending upon design context (LRFD or ASD) (psi)
F^*_c	= adjusted compression stress multiplied by all applicable design factors except for C_p (psi)
F_{cE}	= allowable buckling stress (psi)
F_{nail}	= load applied to a 2-16d nailed joint (pounds)
F_u	= ultimate compressive stress of the column material (psi)
I	= weak axis area moment of inertia (in^4)
K	= provided brace stiffness (pounds/inch)
K_e	= effective length factor for compression members (unitless)
K_{id}	= "ideal" brace stiffness (pounds/inch)
K_{req}	= required brace stiffness (pounds/inch)
L	= effective length of the unbraced column (inches)
L_{bend}	= test span for the flatwise bending E test (inches)
L_w	= actual compression web length (inches)
M	= weak axis bending moment in the column at the brace (inch-pounds)
N_1	= one-dimensional nailed joint
N_2	= two-dimensional nailed joint

LIST OF SYMBOLS (Continued)

P	= column load (pounds)
P'	= LRFD adjusted compression resistance of the column parallel to grain (pounds)
P_{all}	= allowable column load (pounds)
P_{bend}	= center span load for the flatwise bending E test (pounds)
P_{cr}	= critical column load (pounds)
P_e	= Euler buckling load for the unbraced column (pounds)
$P_{e\text{-}lrfd}$	= LRFD Euler buckling load for the braced web (pounds)
P'_0	= LRFD adjusted member axial parallel to grain resistance of a zero length column, design (pounds)
SF_{column}	= ASD safety factor for the column (unitless)
SF_{brace}	= ASD safety factor for the brace (unitless)
$\alpha_{1,2}$	= slope of column curvature above (1) and below (2) the brace (radians)
α_c	= LRFD factor in the design of columns (unitless)
Δ	= additional lateral deflection at mid-height upon application of axial load (inches)
Δ_{actual}	= the observed maximum lateral deflection (inches)
$\Delta_{criticall}$	= Δ where the 2% Rule and Plaut's method intersect, see Figure 6.1 (inches)
Δ_0	= initial lateral deflection at mid-height (inches)
Δ_T	= total lateral deflection at mid-height ($\Delta_0 + \Delta$) (inches)
λ	= LRFD time-effect factor (unitless)
Ω	= slip between the wood members of a 2-16d nailed joint (inches)
ϕ_b	= LRFD resistance factor for brace strength (unitless)
ϕ_c	= LRFD resistance factor for compressive strength (unitless)
ϕ_s	= LRFD resistance factor for stability (unitless)
σ	= compressive stress in the column (psi)
σ_{cr}	= critical column stress (psi)
θ	= brace angle (radians)

DEDICATION

This thesis is dedicated to my father, Dr. Miles E. Waltz, and my grandfather, Roger A. Laferierre. May I will always strive to achieve their level of personal and professional success with an equal blend of humor, ambition, and integrity.

DISCRETE COMPRESSION WEB BRACING DESIGN FOR LIGHT-FRAME WOOD TRUSSES

1.0 INTRODUCTION

Light-frame wood trusses serve as a popular alternative for roof framing in modern timber construction. These prefabricated building components find favor over traditional stick framing with many industry professionals due to their long-span capabilities, ease of use, rapid installation, and competitive cost for many applications. In addition, computerized truss design and custom fabrication provide fantastic versatility. Contemporary truss manufacturers can fashion wood truss systems to achieve a wide range of roof strengths and configurations.

Light-frame wood trusses must be used properly to provide dependable performance. The wood truss represents just one component of a complete roof system. In addition to the trusses, roof structural systems require proper consideration of connections between trusses, connections between trusses and their supports, and lateral bracing. Depending upon the truss configuration and loading, a number of different types of lateral bracing may be required. This report investigates the design requirements for one specific type of bracing: that intended to reduce the effective length for flexural buckling of compression web members.

1.1 Background

In this paper, the term "light-frame wood truss" refers to a timber truss comprised of two-inch nominal dimension lumber. Figure 1.1 provides a schematic illustration of a "double Fink" truss configuration. An infinite number of truss profile and configuration options exist. Truss manufacturers typically use member sizes that range between 2x4 (1.5 x 3.5 in.) and 2x12 (1.5 x 11.25 in.) for the top and bottom chords, with larger sizes required for longer truss spans. The truss webs, the subject of this research, typically consist of 2x4 or 2x6 (1.5 x 5.5 in.) lumber.

Metal-plate-connectors (MPCs) form the connections that tie the wood truss members together into a structural unit. An MPC consists of a light-gauge metal plate with short protruding teeth. During truss fabrication, the manufacturer embeds the teeth of at least one MPC on each side of every joint. Infrequently, truss manufacturers may substitute plywood or metal gusset plates with nails for MPCs. The bracing principles described in this paper can be applied to trusses with either type of connection. However, trusses with MPCs serve as the focus of this research since they are the most common and the majority of research and trade literature addresses these products.

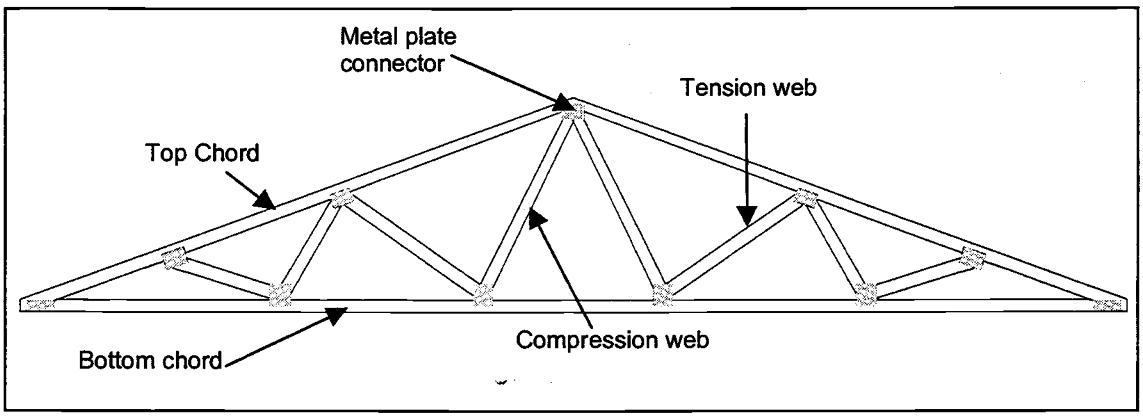


Figure 1.1: Schematic Illustration of a Light-Frame Wood Truss

Depending upon the use, light-frame wood trusses can be installed in single or multiple plies. Truss manufacturers fabricate multiple-ply trusses by nailing together layers of the same truss configuration to form a single truss with greater thickness. Multiple-ply trusses often find application in structures that require longer spans or experience heavier loading conditions. The bracing principles investigated in this paper find relevance for both multiple and single-ply truss situations.

1.1.1 The Role of Lateral Bracing

Lateral bracing provides critical stability for any light-frame wood truss roof. Light-frame wood trusses possess remarkable strength to support vertical loads applied to the top and bottom chords of a vertically oriented truss. In this application, the wood members function primarily in some combination of tension, compression, and strong- axis bending. In stark contrast, light-frame wood trusses possess limited strength to resist out-of-plane loads. Out-of-plane loads induce weak-axis bending on the truss components. Weak-axis bending becomes more critical with longer truss spans. Lateral bracing minimizes weak-axis bending stresses that may not be considered in the truss design by providing supplemental support to resist out-of-plane loads.

In general, two types of lateral bracing exist (Stalnaker and Harris, 1989; Truss Plate Institute, 1991). The first type of bracing resists the tendency for a vertically oriented truss to topple horizontally under lateral loading conditions. Lateral loads generated by wind, earthquake, or construction events can induce this action. In addition, normal construction tolerances for out-of-plumbness can initiate out-of-plane forces under gravity loading conditions. "Overall stability" bracing, if you will, resists lateral truss movement and holds the trusses in their intended position. Depending upon the situation, diaphragm sheathing or a combination of lateral and diagonal braces can be used to provide overall truss stability. Several industry sources provide diagrams that conceptually illustrate these bracing alternatives (Callahan, 1993; Stalnaker and Harris, 1989, Truss Plate Institute, 1976, Truss plate Institute, 1989; Truss Plate Institute, 1995).

The second type of bracing performs an entirely different function; it reduces the effective length for flexural buckling of individual compression members. Compressive forces typically exist in the top chord and in some web members of the truss. In addition, uplift loads sometimes produce compression in the bottom chords. Without continuous lateral support along their entire length, slender compression members like these may buckle flexurally. In other words, the center of any unsupported compression member may laterally translate under axial load. The truss designer must consider this tendency.

For the rectangular wood truss sections discussed in this paper, the "slenderness ratio" of each member is defined as the effective length for flexural buckling about a member cross-sectional axis divided by the section dimension perpendicular to the same axis. The larger the ratio, the greater the risk for flexural buckling. As Figure 1.2 suggests, the truss designer must consider the potential for buckling about two axes for each truss member that experiences compression: "X-X" and "Y-Y". These are also known as "strong" and "weak" axes, respectively. The axis with the largest slenderness ratio governs the compressive design of each truss member.

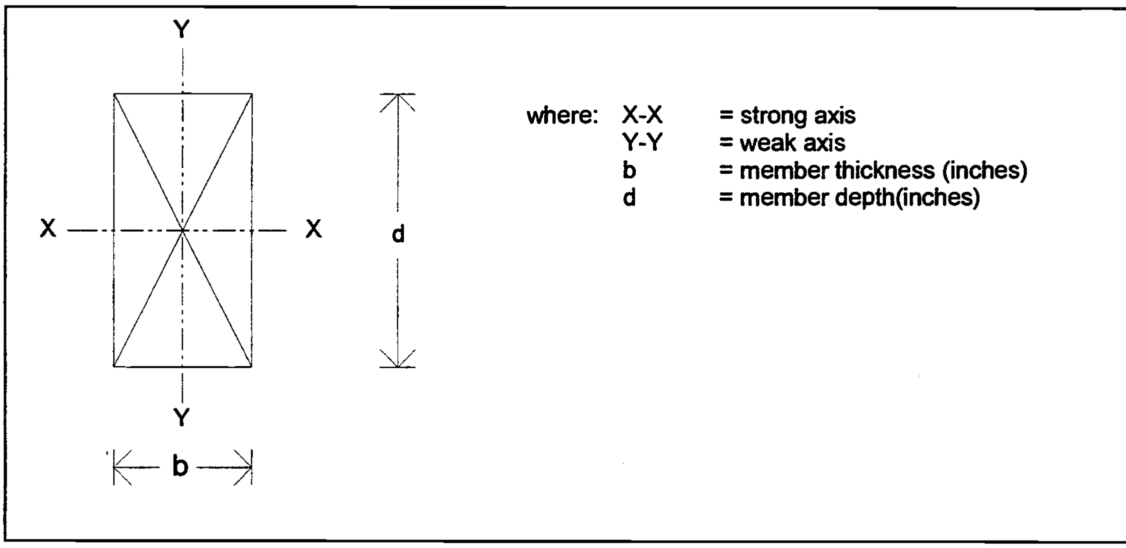


Figure 1.2: Truss Member Cross Section

Lateral bracing provides stability for a truss member by reducing its weak-axis effective length. Lateral bracing does not provide strong-axis support. If the weak axis has the largest slenderness ratio before bracing, then this supplemental support will increase the overall compressive strength of the member. The number and spacing of required lateral braces depends primarily upon the stress level, member dimensions, member grade, and end support conditions used in the truss design. Column design procedures contained within modern wood design codes provide a basis for determining the brace spacing necessary to achieve the desired compressive strength (American Forest Products Association, 1997; Truss Plate Institute, 1995).

For the chords with compressive load, a combination of purlins, lateral braces, diagonal braces, and/or diaphragm sheathing can be used to provide lateral stability for the weak axis. Section 1.2 discusses bracing alternatives for the compression web members that serve as the primary focus of this research.

It is important to realize that some bracing systems can simultaneously provide both "overall" truss stability and member buckling stability. An example of such a dual-purpose brace would be a structural diaphragm attached to the top chord of a pitched truss. In addition to providing "overall stability" for the truss, the diaphragm provides lateral support to limit the potential for weak-axis buckling of the chord.

1.1.2 Temporary vs. Permanent Bracing

The truss industry generally recognizes two further bracing categories: temporary and permanent (AITC, 1994; Callahan, 1993; Kagan, 1993; Truss Plate Institute, 1991; Truss Plate Institute, 1995). Trusses can be especially vulnerable to damage from lateral loads during construction, before attachment of any diaphragm sheathing or other bracing that will form the permanent lateral load resisting system. Temporary bracing provides necessary stability during installation. The Truss Plate Institute and Wood Truss Council of America both provide limited guidance on temporary bracing requirements (Callahan, 1993; Truss Plate Institute, 1976; Truss Plate Institute, 1989; Truss Plate Institute, 1991). Permanent bracing provides lateral stability required for the successful long-term performance of the structure. As will be discussed in Section 2.6, design guidance for permanent bracing is more difficult to obtain than for temporary bracing. Depending upon the roof design, temporary and permanent requirements may exist for both bracing to provide overall stability and bracing to reduce the effective lengths of individual members. The focus of this report will be on permanent bracing, but the principles can be readily extended to the analysis of temporary bracing.

1.2 Compression Web Bracing

This study specifically addresses lateral bracing provided to reduce the weak-axis slenderness of compression web members. It accomplishes this reduction by shortening the effective length for flexural buckling about the weak axis. From this point on, this paper refers to this bracing as "compression web bracing."

1.2.1 Methods

Figures 1.3 and 1.4 illustrate one of the most common methods used to provide compression web bracing when several similar trusses sit adjacent to one another. These two figures reflect a common design assumption that lateral bracing should reduce the weak-axis effective length of the compression web by one-half. After placing the trusses upright on the building and aligning them, the truss installer attaches a continuous "lateral" or "horizontal" brace to the mid-height of the webs to be braced. The lateral brace often consists of 2x4 lumber nailed to each web.

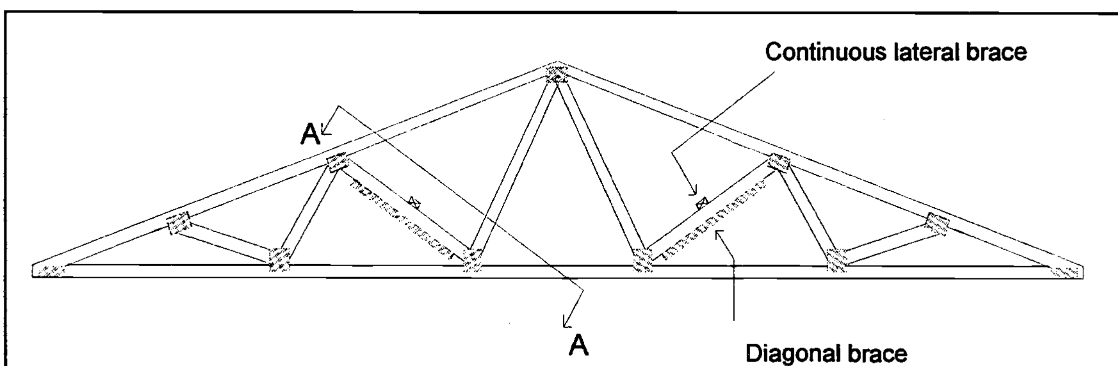


Figure 1.3: Compression Web Bracing Scheme

As indicated in Figure 1.4, the lateral brace effectively ties together the row of compression webs at mid-height. While the continuous lateral brace alone may provide some stability, the potential exists for all of the webs to buckle together as illus-

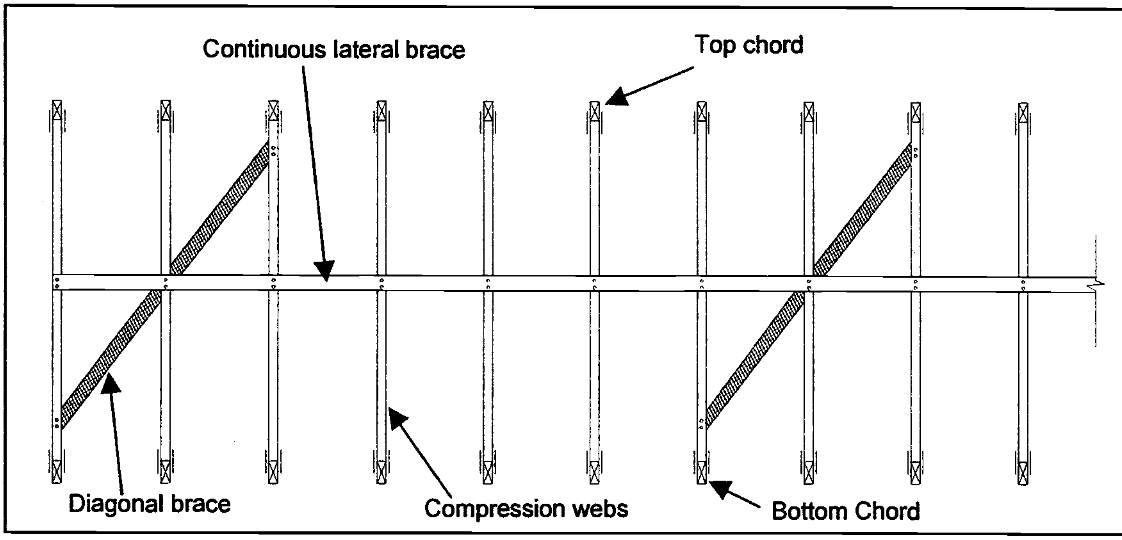


Figure 1.4: Section A-A: Lateral/Diagonal Bracing System

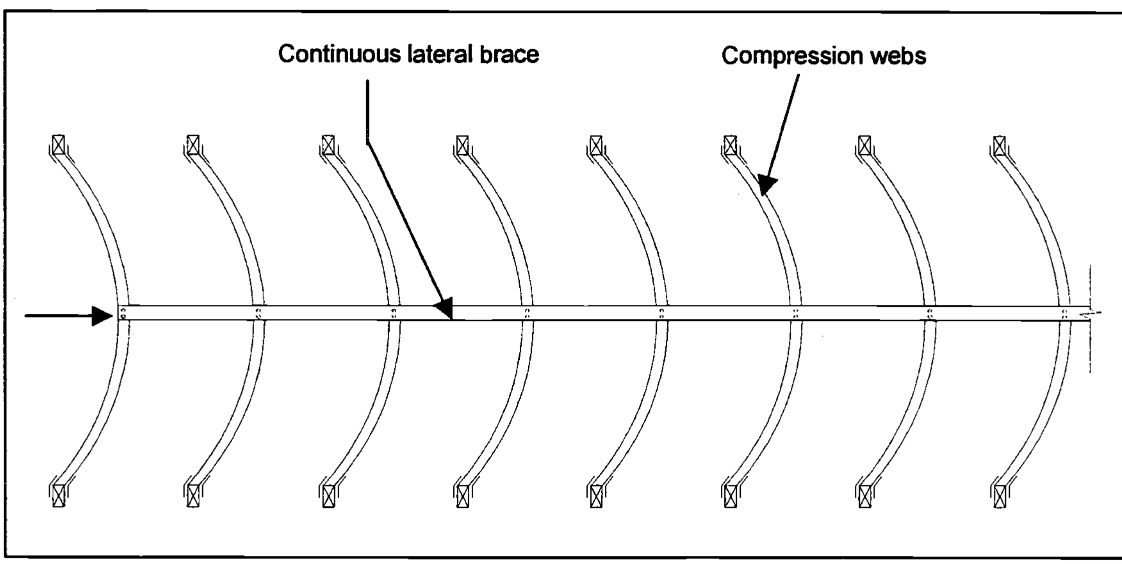


Figure 1.5: Simultaneous Web Buckling Without Diagonal Bracing

trated in Figure 1.5 (Galambos, 1988; Truss Plate Institute, 1991; Woeste, 1998). The trusses require additional lateral resistance to prevent this movement (Callahan, 1993; Stalnaker and Harris, 1989; Truss Plate Institute, 1976; Truss Plate Institute, 1995; Woeste, 1998).

One option may be for the installer to affix an end of the continuous lateral brace to a more rigid portion of the structure, such as a reinforced masonry wall. However, "rigid" attachment points frequently do not exist. A common solution is to install a series of diagonal braces similar to those depicted in Figure 1.4. Periodically placed diagonals help to resist simultaneous web buckling. Diagonal braces usually consist of 2x4 lumber installed at approximately a 45° angle.

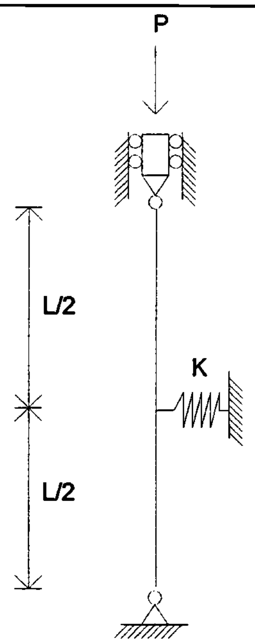
The lateral/diagonal bracing scheme depicted in Figures 1.3 and 1.4 has also been called "discrete" or "point" bracing because it provides discrete rather than continuous lateral support for the full length of the webs. Depending upon the truss design, more than one row of discrete bracing may be required. For simplicity, this research concentrates on the situation where the design requires a single row of lateral bracing.

It is important to note that other bracing methods may be used to resist flexural buckling of a compression web. One of the most popular is to nail a second wood member to the narrow edge of the web over nearly the full length. Called "T" bracing, this creates a composite member with greater resistance to weak axis buckling than that of the web alone. This method often finds application where dissimilar adjacent truss configurations render a lateral/diagonal brace system impractical. When similar trusses exist side-by-side, installers often find it more cost-effective to use lateral/diagonal braces instead of "T" bracing to provide buckling support.

1.2.2 Idealized Model

Figure 1.6 illustrates a simple model used throughout this paper to describe a discretely braced compression web. For analysis purposes, we conceptually remove the compression web from the truss and think of it as an independent column with unbraced effective length "L". Pinned connections attach the ends of the column to the surrounding construction and a single mid-height lateral brace resists flexural buckling about the weak axis. If the brace possesses adequate strength and stiffness, then it reduces the overall weak-axis effective length of the braced column to L/2.

As suggested in Section 1.1.1, the truss designer must consider the slenderness of the compression web about both the weak and strong axes. However, for unbraced compression webs, the truss design process assumes that the connections provide similar end restraint in both directions (Truss Plate Institute, 1995). This means that the effective column lengths are about the same for both axes. Since light-frame wood truss webs typically possess member depths of more than 2.3 times their thickness of 1.5 inches, the largest slenderness ratio and greatest threat for buckling occurs about the weak axis of the unbraced web. In fact, even with a single row of discrete bracing provided to support the weak axis at mid-height, this axis still usually has the largest slenderness ratio and governs the web design.



where: K = provided brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P = column load (pounds)

Figure 1.6: Idealized Discrete Bracing Model for Compression Web Analysis

As stated above, "L" in the conceptual model of Figure 1.6 represents the effective length of the unbraced web about the weak axis. The *National Design Standard for Metal Plate Connected Wood Truss Construction* (Truss Plate Institute, 1995) does not consider the fixity of the end connections to reduce the effective length of a web that receives a lateral brace. In other words, L equals the actual length of the web, L_w for this application. The code does allow the truss designer to use end connection fixity to reduce the effective length of web members that will not receive a lateral brace. In those instances, L is equal to $0.8L_w$. The justification for this inconsistent use of end fixity is not clear to the author. However, to be consistent with the code treatment of braced compression webs, L will be considered equal to L_w for the remainder of this paper.

In reality, no brace provides perfectly rigid support. The spring in Figure 1.6 represents a mid-height brace with finite stiffness "K". The spring describes the combined stiffness of the lateral/diagonal bracing system and its associated connections. Depending upon the analysis technique used, the spring stiffness may be considered linear or nonlinear.

1.3 Current Design Process

In North America, truss manufacturers usually design light-frame wood trusses using computer software provided by the MPC producers. The MPC manufacturer takes responsibility for the truss design by providing the seal of a licensed engineer on the truss shop drawings. For a given truss profile, boundary conditions, wood species, and design load, these computer programs design the truss members and connections in accordance with industry guidelines (Truss Plate Institute, 1995). Some optimization usually takes place. The programs select the size and gauge of each MPC connector plate based on the forces transmitted though that joint. Similarly, they select the wood member sizes and grades based on their stress levels. Since different truss members and connections experience different loads, several different member and connection types may be used in the same truss.

As part of the design process, the truss designer makes assumptions about the effective length of the compression web members. When a trial web design is too slender to provide sufficient strength, the designer has four options: specify compression web bracing, try a different member depth, use a different stress grade of lumber, or try a different truss configuration. When the designer chooses the first option, he/she indicates the number of discrete bracing points or rows assumed on the truss design drawings. However, that's as far as the truss designer normally takes the bracing design.

According to the *National Standard for Metal Plate Connected Wood Truss Construction* (Truss Plate Institute, 1995), the building designer is responsible for designing the actual bracing system to provide the required support. Based on the author's experience, architects, engineers, and installers often do not realize that this breakdown in design responsibility occurs. It is common for them to think that the truss designer provides the design for the entire roof system. In addition, design professionals may not even be involved on many residential projects to make this distinction. Others report similar confusion (Kagan, 1993; Hoyle, 1984). A recent article in the Wood Truss Council of America's newsletter (Pagel, 1997) provides some insight as to why the truss industry insists upon this division in design responsibility:

> "....most truss accidents are caused by inadequate temporary bracing during installation, inadequate bracing connections, overloading during installation, installation of damaged trusses, or inadequate permanent bracing. However, with a properly prepared contract and a clearly defined scope of work, much of the installation and bracing related liability can be avoided or shifted to the customer."

Investigators often implicate missing, improper, or incomplete lateral bracing in performance failures of light-frame wood trusses (Kagan, 1993; Hoyle and Woeste, 1989; Milner, 1996; Pagel, 1997).

The building designer, faced with the problem of designing compression web bracing, needs a methodology to estimate the required strength and stiffness. The Truss Plate Institute does not provide a design method for permanent compression web bracing. Milner (1996) reports that a new British code, due out in 1998, may supply specific design guidance for brace design in Europe. However, at present, building designers in the U.S. must rely on available "rules-of-thumb" and theoretical models in the absence of specific code provisions. The intent behind this research is to investigate the applicability of several potential design aids to see if they provide a rational basis to determine the required strength and stiffness of compression web bracing.

1.4 Project Objective

The primary objective of this research is to evaluate the ability of several existing theoretical analysis models to estimate the discrete brace strength and stiffness required to fully brace a compression web. A secondary objective is to recommend a rational brace design procedure.

1.5 General Research Approach

1.5.1 Part I - Literature Review

The first phase of this research consisted of an extensive literature review. The review primarily concentrated on literature that addressed analysis and design techniques for discrete lateral bracing intended to reduce the effective length for flexural buckling of compression members. The review objective was to determine what information exists to aid with compression web bracing design. In addition to literature specific to light-frame wood trusses, the search included a review of theoretical models that show promise as simplified tools to estimate brace strength and stiffness requirements. Section 2 summarizes the literature review.

1.5.2 Part II - Brace Stiffness Modeling

Many theoretical models predict that the stiffness of the brace system directly affects the critical column load and force that develops within the brace. The objective of this research phase was to estimate the range in brace stiffnesses that may occur when the design incorporates a lateral/diagonal brace system to reduce the effective length of a compression web member. To this end, we developed a series of finite element models to estimate the brace stiffness provided. Section 3 summarizes the finite element analysis and brace stiffness estimates.

1.5.3 Part III - Test Program

The last phase consisted of a physical test program to quantify the mid-height brace requirements of 774 2x4 sample columns. The testing incorporated a range of four different lengths (4', 6', 8', and 10') and two different grades ("Select Structural" and "Standard"). After measuring the brace requirements to achieve the ultimate column strength assumed in the compression web design, we compared the actual brace requirements with those estimated by the theoretical relationships to be evaluated. Sections 4 and 5 summarize the procedure and results.

2.0 LITERATURE REVIEW

Determination of the brace strength and stiffness required to reduce the effective length of a compression web member represents a question of structural stability. Applied mathematicians have toiled with matters of structural stability for centuries. Engesser (1884; 1893) was one of the first investigators to look specifically at the problem of lateral bracing requirements for truss compression members. Engesser, and many researchers after him, concerned themselves with the brace stiffness required to stabilize the top chords of a "Pony" truss. Pony trusses served as the primary structural system for many bridges at that time.

Despite its apparent simplicity, an exact solution to the indeterminate problem of a compression web member with a discrete lateral brace is actually rather complicated. Real-world concerns such as column imperfections, material inelasticity, material non-homogeneity, brace stiffness non-linearity, and normal construction tolerances render exact solution to the problem impractical for routine design. Perhaps due to this complexity, little information exists specific to the design of compression web bracing for light-frame wood trusses. However, several simplified, theoretical models may apply to wood construction. After a brief review of some relevant topics in compression web design, the following sections present these potentially applicable theories.

2.1 Unbraced Columns

2.1.1 Column Buckling

Even perfectly straight, slender columns like that of Figure 2.1A typically cannot support an axial load sufficient to reach the yield stress of the constituent materials. Before the column reaches yield, it tends to buckle flexurally as illustrated in Figure 2.1B. Euler receives credit for first estimation of P_e, the ultimate load at which an elastic column buckles (Van den Broek, 1947):

$$P_e = \frac{\pi^2 EI}{L^2} \tag{1}$$

where: E = longitudinal modulus of elasticity of the column (psi)
I = weak axis moment of inertia (in^4)
L = effective length of the unbraced column (inches)
P_e = Euler buckling load for the unbraced column (pounds)

Derivation of this formula incorporates several assumptions, including: linear elastic column behavior, a prismatic column section, homogeneous and isotropic material properties, load through the cross-section centroid, and pin-ended support conditions.

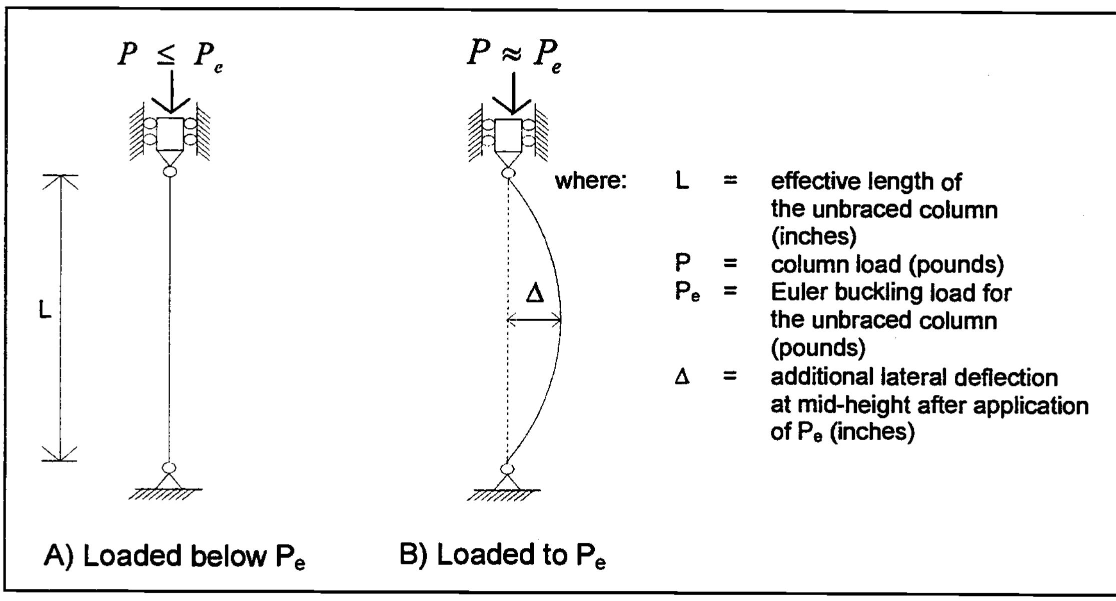


Figure 2.1: Unbraced, Perfect columns

Euler's equation represents the oldest engineering design equation still in use (Johnson, 1983). The theory behind this formula predicts that a perfectly straight, or "perfect," column can be loaded up to P_e without experiencing lateral deflection Δ. At P_e, a "bifurcation" of equilibrium takes place (Brush and Almroth, 1975; Galambos, 1968). The column then behaves in one of two different ways: it can continue to accept load and remain straight, or it can buckle with additional load application. The buckled mode illustrated in Figure 2.1B represents the lower energy state that usually prevails due to normal imperfections in the column.

Analytical buckling models based on the assumption of small column deflections predict an unbounded lateral displacement Δ at mid-height when the column load reaches P_e (Winter, 1960). Models that take into account exact curvature and large-deflection theory predict finite deflection and continued stability at loads above P_e for an elastic column (Gere and Timoshenko, 1990; Timoshenko and Gere, 1961). However, lateral displacements that occur with loads above this benchmark induce vertical displacements and column bending stresses that limit the reliability of the column. Conservative design practice limits the nominal strength of a slender column to some fraction of P_e.

2.1.2 Perfect vs. Imperfect Columns

In reality, no column is perfectly straight. Some initial curvature always results from material, installation, and/or manufacturing tolerances. From this point on, this paper refers to perfectly straight columns as "perfect" and columns with some degree of assumed initial curvature as "imperfect." As indicated in Figure 2.2, we often define the magnitude of an initial imperfection by Δ_0, the assumed initial lateral displacement of the column at mid-height. Since a compression web bracing system supports the web about its weak axis, the initial deflection or "bow" about the member weak axis represents the most important initial curvature for lateral brace design. Figure 2.3 illustrates bow of a compression web member.

The actual shape of the initial lateral column displacement cannot be predicted precisely, especially with wood. Many researchers assume that it approximates one-half of a sine wave, with mid-height amplitude Δ_0 (Clarke and Bridge, 1993; Galambos, 1988; Green et al. 1947; Plaut and Yang, 1993; Timoshenko and Gere, 1961; Tsien, 1942; Zuk, 1956). The *National Design Standard for Metal Plate Connected Wood Truss Construction* (Truss Plate Institute, 1995) does not specifically restrict Δ_0 for compression web members. However, it does limit the overall initial deflection of the chords and panels to the lesser of two inches or 1/200th their span. The grading rules

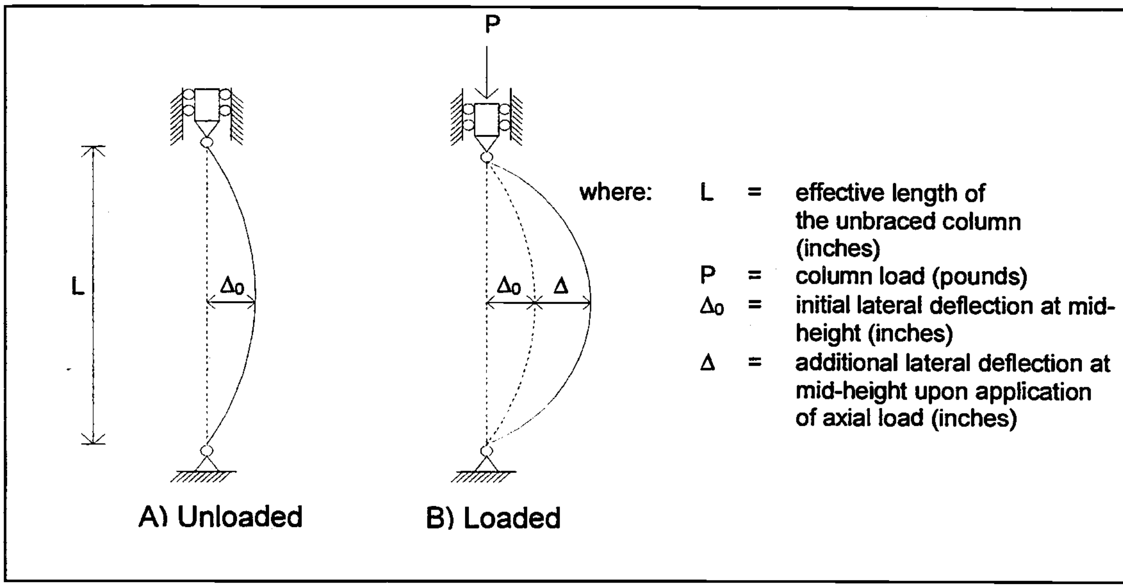


Figure 2.2: Unbraced, Imperfect Columns

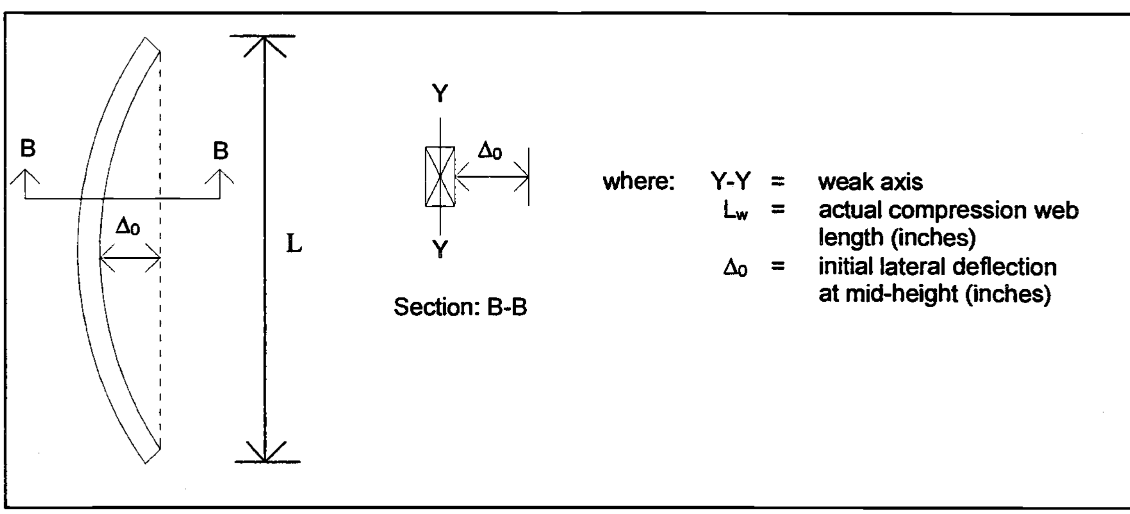


Figure 2.3: Weak Axis Bow

for the lumber that make up the truss provide some restriction on the permitted bow. Table 2.1 illustrates the allowable bow for 2x4 Douglas-fir lumber of "Select Structural" and "Standard" grades (WCLIB, 1996). For comparison, Table 2.1 also includes the bow that would be allowed if an $L_w/200$ limit applied to compression webs. It is important to recognize that these bounds may not be reasonable for an installed truss web. Fabrication, handling, and installation of the truss might impact the Δ_0 that the designer should expect in-service.

Table 2.1: Maximum Value of Δ_0 Allowed in the West Coast Lumber Inspection Bureau Grading Rules for 2x4 Douglas-Fir Structural Lumber (WCLIB, 1996).

Member Length (feet)	Select Structural Grade (inches)	Standard Grade (inches)	$L_w/200$ (inches)
4	0.38	0.50	0.25
6	0.38	0.50	0.36
8	0.50	0.75	0.48
10	1.38	1.50	0.60

Initial deflection dramatically affects the behavior of an unbraced column. As suggested by Figure 2.2B and illustrated in Figure 2.4, columns with initial curvature experience immediate lateral deflection upon application of load (Galambos, 1988). This deflection increases as the column load approaches P_e. Unlike perfect columns, imperfect columns do not undergo a bifurcation of equilibrium. The member experiences immediate axial and secondary bending stresses that increase with the column load. However, if a slender elastic column starts with a small Δ_0, then it should still fail at a critical column load approximately equal to P_e (Brush and Almroth, 1975).

2.1.3 Inelasticity

The engineering community did not initially accept Euler's equation because it overestimates the strength of many columns with lower slenderness ratios (Johnson, 1983). When the axial stress level exceeds the ultimate compressive strength of the constituent material, an extremely short and stout wooden column will crush before it buckles. Columns of intermediate slenderness fail at load levels below both the ultimate compressive strength and the buckling strength predicted by Equation 1. The reason for this is that all or part of the column cross section is stressed beyond its linear elastic limit and the member becomes "inelastic" (Galambos, 1968; Gere and Timoshenko, 1990). Since the modulus of elasticity changes, Equation 1 for the buckling load is also affected.

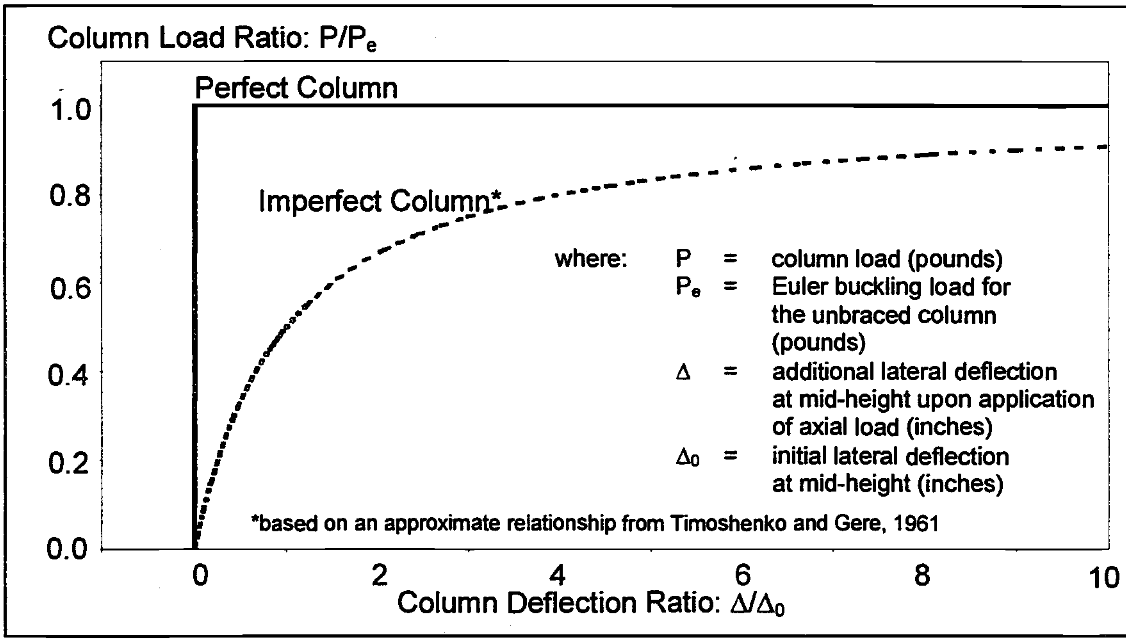


Figure 2.4: Column Load vs. Lateral Deflection – Perfect and Imperfect Columns

After considerable debate, researchers now agree that Equation 1 can still be used to approximate the buckling strength for inelastic columns of intermediate slenderness by substituting the tangent modulus of elasticity, E_t, for E (Gere and Timoshenko, 1990; Johnson, 1983; Shanley, 1947). Some stability may still exist at loads above this modified P_e, but we do not depend upon that stability for design purposes (Bleich, 1952; Galambos, 1988).

2.1.4 Column Curves

Equation 1 requires more than a simple replacement of E_t for E to become a suitable predictor of column strength. Factors other than inelasticity also influence column failure (Brush and Almroth, 1975; Galambos, 1988; Ugural and Fenster, 1995). Material imperfections, non-homogeneity, slight load eccentricities, residual stresses, and secondary bending stresses also affect column performance. To account for factors such as these, structural designers use semi-empirical “column curves” to predict failure. Figure 2.5 schematically illustrates one such curve.

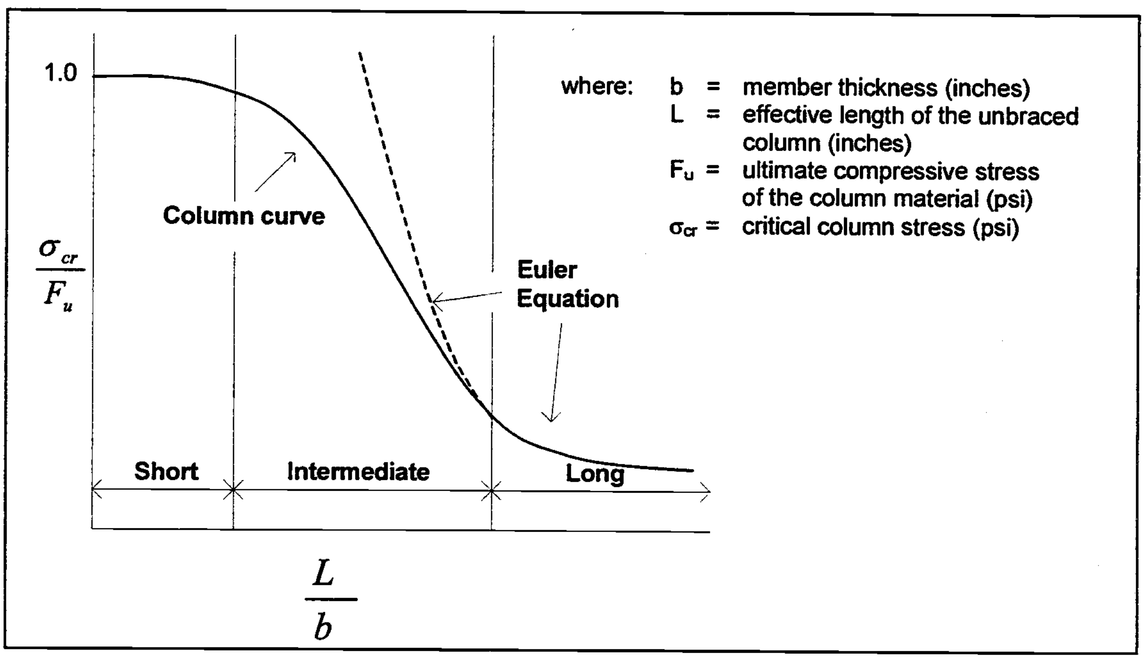


Figure 2.5: Schematic Column Curve

In timber design, the most commonly used column curve can be defined as follows (Forest Products Laboratory, 1987):

$$P_{cr} = \sigma_{cr} A = \left[\frac{\left(F_u + \frac{P_e}{bd}\right)}{2c} - \sqrt{\left(\frac{F_u + \frac{P_e}{bd}}{2c}\right)^2 - \frac{F_u\left(\frac{P_e}{bd}\right)}{c}} \right](bd) \tag{2}$$

where: b = member thickness (inches)
d = member depth (inches)
c = column curve constant (unitless)
A = member cross sectional area (in^2)
F_u = ultimate compressive stress of the column material (psi)
P_{cr} = critical column load (pounds)
P_e = Euler buckling load for the unbraced column (pounds)
σ_{cr} = critical column stress (psi)

Equation 2 predicts the critical strength, P_{cr}, of the column. Zahn (1986; 1991) derived Equation 2 from Equation 1 using a non-linear approximation of a stress-strain curve developed by Ylinen (1956) to define E_t (Forest Products Laboratory, 1987). The "c" term in Equation 2 represents the empirical portion of the equation. The value of c can be adjusted to provide a close match between the column curve and experimental data. A "c" value of approximately 0.8 provides a reasonable fit for sawn lumber (Zahn, 1986; 1991).

Through a column stability factor, C_p, a modified version of Equation 2 supplies a design basis to determine allowable column strengths (American Forest and Paper Association, 1997; Truss Plate Institute, 1995):

$$P_{all} = C_p F_c^* A = \left[\frac{1 + \frac{F_{cE}}{F_c^*}}{2c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_c^*}}{2c}\right)^2 - \frac{\frac{F_{cE}}{F_c^*}}{c}} \right] F_c^* (bd) \tag{3}$$

where: A = member cross sectional area (in^2)
b = member thickness (inches)
d = member depth (inches)
c = column curve constant (unitless)
C_p = column stability factor (unitless)
F^*_c = adjusted allowable compression stress multiplied by all applicable design factors except for C_p (psi)
F_{cE} = allowable buckling stress (psi)
P_{all} = allowable column load (pounds)

In Equation 3, F^*_c represents the allowable axial column stress to prevent crushing. F^*_c takes into account all timber design adjustment factors except for C_p (American Forest and Paper Association, 1997). For softwood lumber, these adjustments include a safety factor reduction of approximately 1.19 applied to the 5% exclusion limit of compression strength for a given lumber grade (Bodig and Jayne, 1982).

F_{cE} in Equation 3 represents an "allowable" stress version of the Euler buckling stress for solid lumber (American Forest and Paper Association, 1997). It is calculated as follows:

$$F_{cE} = \frac{0.3E}{(L/b)^2} \tag{4}$$

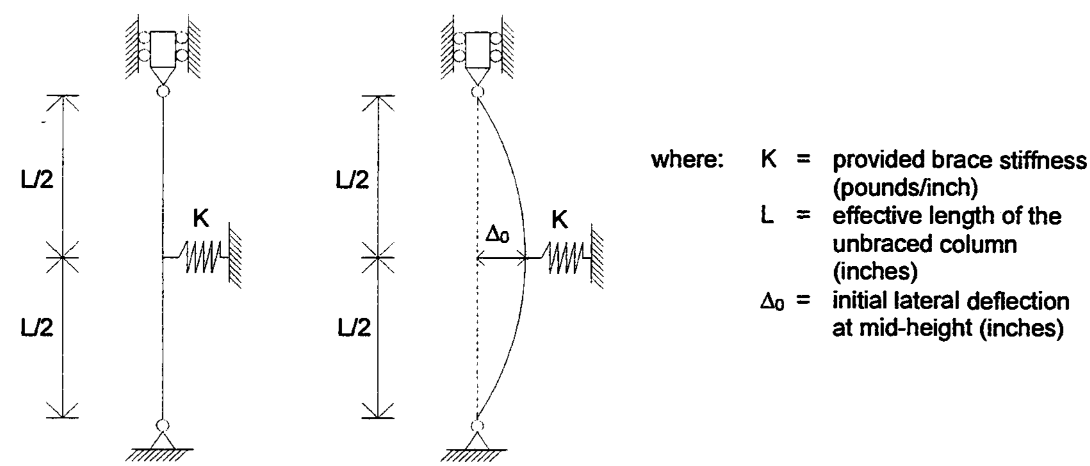
where: b = member thickness (inches)
E = longitudinal modulus of elasticity of the column (psi)
L = effective length of the unbraced column (inches)
F_{cE} = allowable buckling stress (psi)

Equation 4 is derived from Equation 1 by including the assumption of a rectangular cross section, a 1.65 factor reduction to approximate the lower 5% exclusion limit of E, and a 1.66 factor of safety (American Forest and Paper Association, 1997).

2.2 Idealized Braced Column Behavior (Linear Elastic Column and Brace)

One way to increase the strength of a slender column is to reduce its effective length with a discrete lateral brace. Equations 1 and 4 illustrate that the ultimate and allowable buckling strengths are inversely correlated with the square of the effective length. Small reductions in effective length of a long column result in large increases in column strength. Considerable theoretical work exists to address the elastic stability of braced columns. Some might prove suitable to serve as a basis for a design criterion. This section of the review concentrates on models developed for elastic, homogeneous columns with a single elastic brace placed at mid-height.

Figure 2.6 schematically illustrates the conceptual models often used to describe a compression web member with a discrete, mid-height brace. As with unbraced columns, braced columns can be treated as perfect or imperfect depending upon whether the analysis considers initial column curvature. Perfect columns behave differently from imperfect columns, so an assumption of initial curvature becomes very important in determining the required brace strength and stiffness.



A) Perfect column B) Imperfect column

Figure 2.6: Braced Column Configurations

For both perfect and imperfect columns, the linear relationship between the force and deflection in the brace is assumed to be described by:

$$F_{br} = \Delta K \tag{5}$$

where: F_{br} = brace force (pounds)
K = provided brace stiffness (pounds/inch)
Δ = additional lateral deflection at mid-height upon application of axial load (inches)

2.2.1 Perfect Columns

Figure 2.7 illustrates three potential behavior modes for a perfectly straight, braced column. As with an unbraced column, one possible solution is for the braced column to remain straight and continuously accept additional axial load without buckling. Figure 2.7A illustrates this condition. Eventually, the axial load achieves the maximum compressive strength of the material and a wooden column fails by crushing. In this hypothetical case, Δ equals zero and no lateral force exists in the brace. However, this failure mode does not represent the solution with the lowest energy state and would not be conservative to assume for the design of slender columns where buckling represents the critical failure mode.

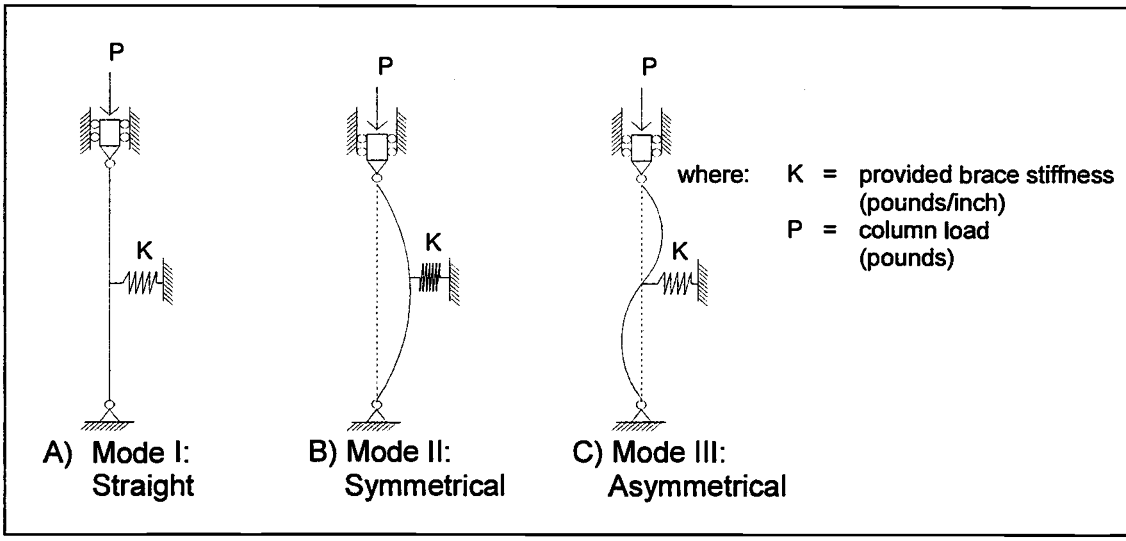


Figure 2.7: Behavior Modes for Perfect Columns with a Mid-height Brace

Figure 2.7B illustrates a second behavior mode where the column carries a "critical" axial load, P_{cr}, somewhere between P_e and $4P_e$ (Timoshenko and Gere, 1961). The brace is not stiff enough to reduce the effective length of the column by one-half. At P_{cr}, a bifurcation occurs: the column can continue to accept load and remain straight or it can buckle symmetrically. When it buckles, the lateral deflection and the force in the brace theoretically become infinitely large (McGuire, 1968). This can be attributed to a small deflection approximation used to solve the problem analytically.

Figure 2.7C represents a third possible behavior mode where the brace supplies enough stiffness to force the column to buckle asymmetrically. When this happens, the critical column load and bifurcation occur at a load of $4P_e$ (Timoshenko and Gere, 1961). In effect, the brace forces the column assembly to buckle with an effective length L/2. We call this column "fully braced" and term the brace stiffness required to achieve this condition in a perfect column the ideal brace stiffness, K_{id}. K_{id} can be determined from equilibrium of the column and defined as (Timoshenko and Gere , 1961):

$$K_{id} = \frac{16\pi^2 EI}{L^3} = \frac{16P_e}{L} \tag{6}$$

where: E = longitudinal modulus of elasticity of the column (psi)
I = weak axis moment of inertia (in^4)
K_{id} = "ideal" brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P_e = Euler buckling load for the unbraced column (pounds)

Since no lateral deflection occurs at the mid-height brace with asymmetrical buckling, the brace force equals zero.

Figure 2.8 graphically illustrates the relationship between the critical load of the perfect column and the stiffness of the mid-height brace. With no brace stiffness, we assume that the column buckles symmetrically at a critical axial load P_e. With a brace stiffness at or above K_{id}, the effective length of the column assembly reduces to L/2 and the column buckles asymmetrically at a critical load of $4P_e$. Theoretically, increasing the brace stiffness beyond K_{id} does not result in increased column strength (Green et al., 1947; McGuire, 1968; Olhoff and Akesson, 1991; Trahair and Nethercot, 1984).

A nearly linear relationship between P_{cr} and K exists for finite brace stiffness between zero and K_{id} (Timoshenko and Gere, 1961). Equation 7 approximates this relationship (Green et al., 1947):

$$P_{cr} = P_e + \frac{3}{16}KL \tag{7}$$

where: K = provided brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P_{cr} = critical column load (pounds)
P_e = Euler buckling load for the unbraced column (pounds)

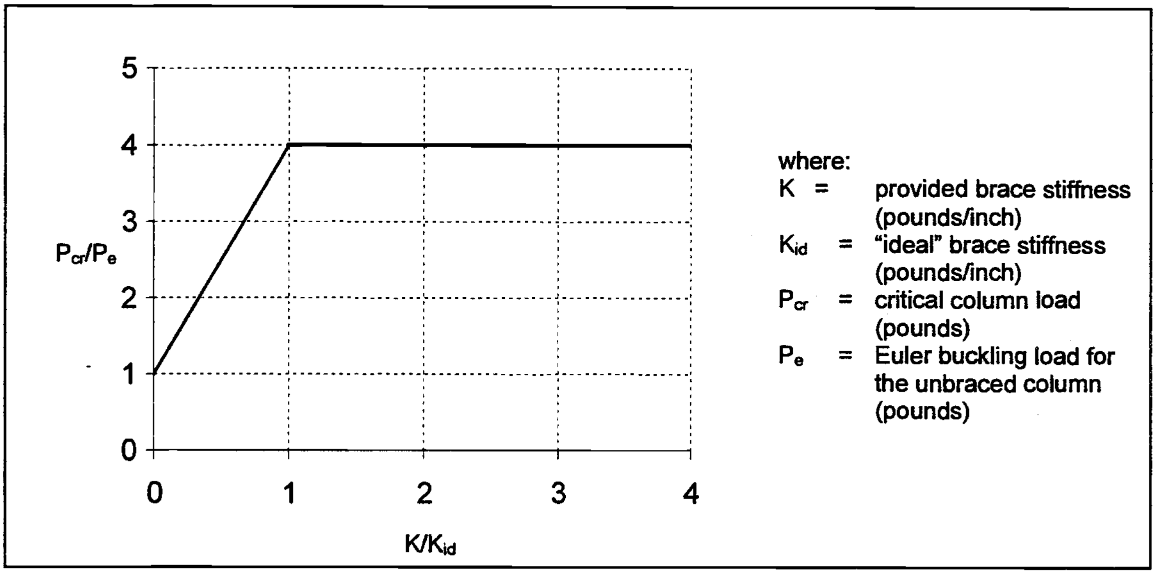


Figure 2.8: Critical Column Load/Brace Stiffness Relationship for Perfect Columns

It is important to note that higher order asymmetric and symmetric buckling modes are theoretically possible (Clarke and Bridge, 1993; Stanway et al., 1992). However, these modes represent higher energy states with larger P_{cr} values and are neither conservative nor realistic to assume for column design.

Engineers typically assume the asymmetrical buckling of Figure 2.7C in the design of braced truss compression webs. The truss design standard establishes that the effective length of the braced column assembly is the distance between the brace and the end of the web (Truss Plate Institute, 1995). In other words, the designer assumes that the discrete bracing system fully braces the column and forces asymmetric buckling. With the substitution of L/2 for L, the web design and analysis process proceeds using the equations of Section 2.1.

Unfortunately, the theoretical solutions for a braced, perfect column do not produce useful approximations for the force in the brace (Bleich, 1952; Plaut and Yang, 1993). The behavior modes of Figure 2.7A and 2.7C imply no required brace strength, even after asymmetric buckling occurs with Mode III. In contrast, the behavior mode of Figure 2.7B requires no brace strength at loads below P_{cr} and infinite strength once buckling initiates. A designer cannot consider any of these approximations to be conservative or useful. To predict realistic brace strength requirements mandates some assumption of initial curvature (Trahair and Nethercot, 1984; Winter, 1960).

2.2.2 Imperfect Column Behavior

For all practical purposes, perfectly straight columns do not exist. All real columns possess some degree of initial curvature. For a column with a mid-height brace, researchers typically assume an initial curvature approximated by one-half of a sine wave as illustrated by Figure 2.9A (Clarke and Bridge, 1993; Galambos, 1988; Green et al., 1947). However, Plaut (1993a) argues that other assumed shapes, such as that defined by a quadratic function, may predict larger brace strength and stiffness requirements. While the appropriate initial shape to be assumed for analysis can be argued, the largest demands for brace strength and stiffness occur when a mid-height braced column possesses some form of "half-wave" initial shape.

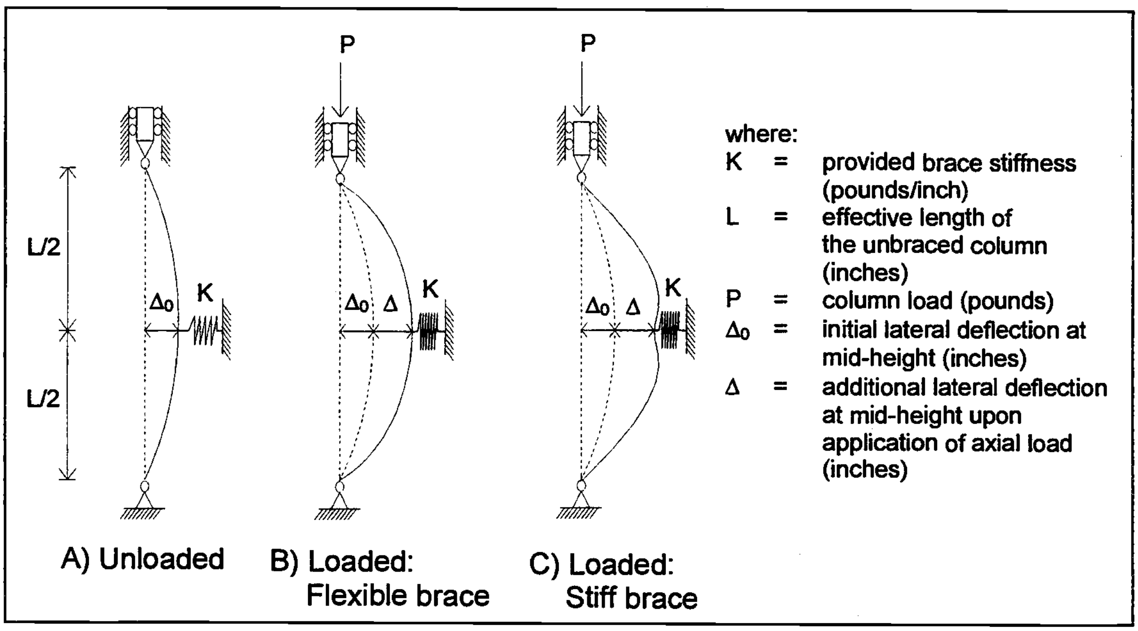


Figure 2.9: Imperfect, Braced Columns

Like their unbraced cousins, imperfect columns with mid-height braces do not normally experience bifurcation (McGuire, 1968). They undergo immediate lateral displacement Δ at the brace location with applied axial load. Equation 5 suggests that a brace force must accompany this displacement. In addition, the column also experiences an immediate, secondary bending moment.

Figure 2.9B illustrates the initial and subsequent lateral position of a loaded column supported by a flexible brace with stiffness much lower than K_{id}. The initial and final curvatures have roughly the same shape, but different amplitudes. The following equation approximates the deflection at the brace (Green et al., 1947):

$$\Delta = \Delta_0 \frac{\left(\frac{P}{P_e}\right)}{\left(1 - \frac{P}{P_e}\right)} \tag{8}$$

where: P = column load (pounds)
P_e = Euler buckling load for the unbraced column (inches)
Δ = additional lateral deflection at mid-height upon application of axial load (inches)
Δ_0 = initial lateral deflection at mid-height (inches)

Using Equation 5 and the approximation of Δ from Equation 8, the force in the brace can be estimated.

Equation 8 will not provide an accurate estimate of Δ when a relatively stiff lateral brace supports the column (Plaut, 1993a). As shown in Figure 2.9C, some researchers suggest that reverse curvature initiates in the brace area with stiffer braces (Clarke and Bridge, 1993; Plaut, 1993a; Stanway et al., 1992). In fact, the column may deflect into a higher order symmetric buckling mode with a P_{cr} larger than $4P_e$. However, a relatively small vertical misplacement of the brace away from the column center could force the column to fail at a lower strength, asymmetrical buckling mode similar to Figure 2.7C (Clarke and Bridge, 1993). Consequently, it is not conservative to assume a higher-order buckling mode or critical load larger than $4P_e$ for column design.

2.2.3 Strength Models for Imperfect Columns

A conceptual description has been given of how an imperfect column, braced at mid-height, behaves when axially loaded. However, the central issue addressed by this research remains unanswered. How should the building designer determine the design requirements for the brace? It would not be realistic to adopt the analytical solutions for perfect columns that imply no required brace strength. One approach to brace design is to use a "strength model," for lack of a better term, to approximate brace strength requirements.

"Strength models" predict the necessary strength of the brace system as a percentage of the axial column load. One of the most commonly used rules-of-thumb is to design the brace for 2% of the compressive load in the column (Throop, 1947):

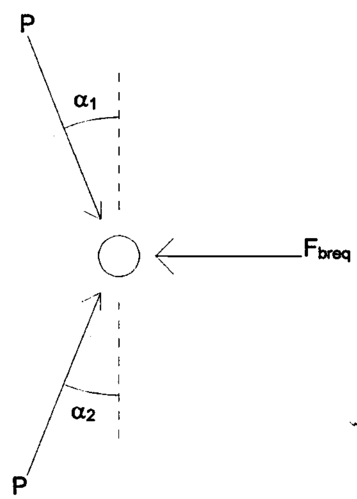
$$F_{br} = .02P \tag{9}$$

where: F_{br} = brace force (pounds)
P = column load (pounds)

Estimates of required strength that range between 1.2% and 2.5% of the compressive load can be found in steel design literature (Clarke and Bridge, 1993; Committee on the Design of Steel Building Structures of the Committee on Metals, ASCE Structural Division, 1992; McGuire, 1968; Medland, 1977; O'Connor, 1979; Throop, 1947; Zuk, 1956). Yura (1996b) reports that most designers use a strength rule to design lateral bracing.

Throop (1942) and Nair (1988; 1992) provide some insight into the origin of these rules. As shown in Figure 2.10, they generally result from a force balance for a free-body diagram at the brace. The idea is to estimate the angle from vertical that will result from the lateral deflection of a loaded column and calculate the brace force accordingly. Throop (1942) arrives at the 2% rule based on an arbitrarily assumed initial column slope of 1/100 above and below the brace. Nair (1988) estimates a 1.2% Rule using a more sophisticated rational that includes rough approximation of Δ in addition to Δ_0. However, Nair incorporates numerous, specific assumptions of steel construction tolerances and detailing into his estimate. The reader should consult his articles for details.

Serious questions arise concerning the use of brace "strength models" for wood compression members. None of the rules reviewed by the author originated from assumptions specific to the problem of compression web bracing design. In addition, the "strength models" do not compensate for the effects of reverse curvature in the column at the brace. Recent research suggests that this curvature can actually increase the force on the brace (Plaut, 1993a; Stanway et al., 1992).



where:

P	=	column load (pounds)
F_{br}	=	brace force (pounds)
α_1	=	slope of column curvature above the brace (radians)
α_2	=	slope of column curvature below the brace (radians)

<u>Force balance:</u>

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

When α_1 and α_2 are considered small and equal:

$$F_{br} \approx (\alpha_1 + \alpha_2)P$$

Figure 2.10: Force Balance for a Strength Model at a Discrete Lateral Brace

One of the biggest concerns with brace strength rules-of-thumb relates to the design of the compression web itself. For the web to be fully braced as assumed in the truss design, the brace must be stiff enough to force the web to buckle asymmetrically about its weak axis. Without adequate stiffness, the column buckles in a lower strength mode and the critical column load falls short of $4P_e$. Using a strength rule alone for brace design does not ensure that the brace has the desired effect on column performance. Even Throop (1942) points out that, in addition to satisfying the strength requirement of Equation 9, the brace designer must ensure that the brace provides "rigid" support. Some authors suggest that a brace strength sufficient to resist 2% to

2.5% of the column load may provide adequate stiffness in steel structures (Galambos, 1988; Mutton and Trahair, 1975; Trahair and Nethercot, 1984). However, similar assurances have not been established for wood. Any thorough check of brace adequacy should examine both strength and stiffness.

2.2.4 Rigid Link Models for Imperfect Columns

George Winter (1960) was one of the first researchers to recognize and practically address the fact that bracing systems require both adequate stiffness and strength. His paper on bracing requirements represents one of the classic works in structural engineering. Many steel design texts use his models and recommendations for discrete bracing design (Galambos, 1988; McGuire, 1968; Salmon and Johnson, 1996; Trahair and Nethercot, 1984).

Based on a series of light-gauge steel column tests completed at Cornell University, Winter recognized that the strength and stiffness requirements to fully brace a column were relatively small (Green et al., 1947). His goal was to conservatively approximate the brace requirements to achieve full bracing in a simple manner. To do this, he used the rigid link model shown in Figure 2.11.

Use of the rigid link model assumes that the column sections above and below the brace point behave as rigid members. A fictitious hinge is assumed in the column at the point of brace attachment. Winter (1960) based this assumption on the fact that an asymmetrically buckled, perfect column will not possess a moment at mid-height due to an inflection of curvature in the buckled shape as depicted in Figure 2.7C. By enforcing static equilibrium on one-half the column in Figure 2.11C, the following equations for the brace force and stiffness can be readily derived (Winter, 1960):

$$K = \frac{4P}{L}\left(1+\frac{\Delta_0}{\Delta}\right) \tag{10}$$

$$F_{br} = K\,\Delta = \frac{4P}{L}\left(\Delta + \Delta_0\right) \tag{11}$$

where: F_{br} = brace force (pounds)
K = provided brace stiffness (pounds/inch)

L = effective length of the unbraced column (inches)
P = column load (pounds)
Δ = additional lateral deflection at mid-height upon application of axial load (pounds)
Δ_0 = initial lateral deflection at mid-height (inches)

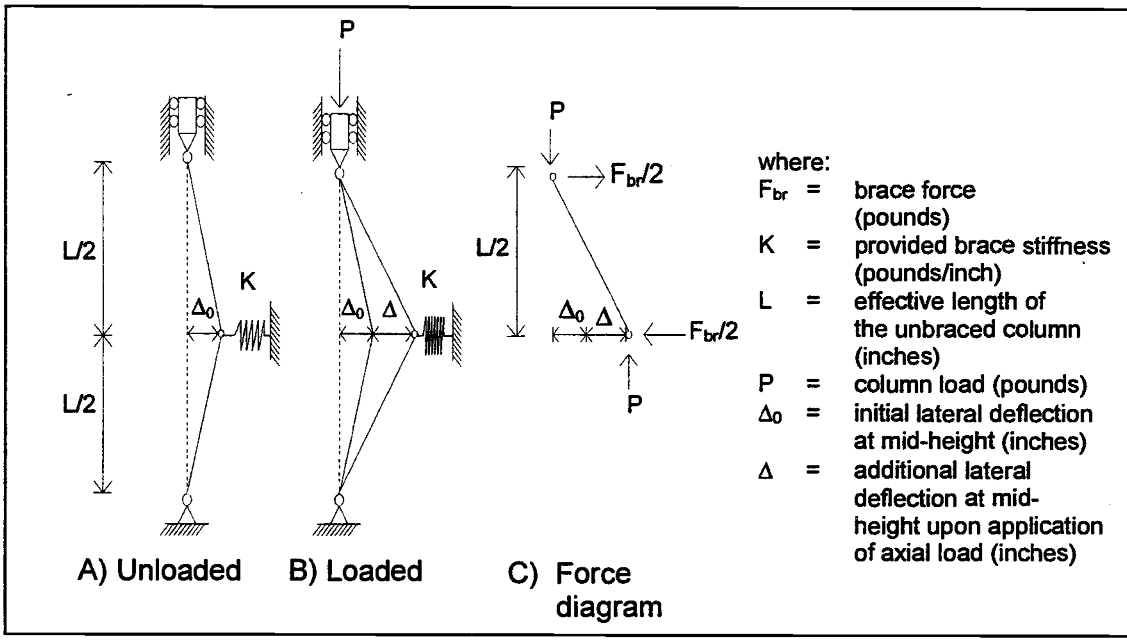


Figure 2.11: Winter's Rigid Link Model for Imperfect, Braced Columns

To provide the full bracing required to achieve a column load of $4P_e$:

$$K_{req} = \frac{4(4P_e)}{L}\left(1+\frac{\Delta_0}{\Delta}\right) = K_{id}\left(1+\frac{\Delta_0}{\Delta}\right) \tag{12}$$

$$F_{br-req} = K_{req}\Delta = K_{id}(\Delta + \Delta_0) \tag{13}$$

where: $F_{br\text{-}req}$ = required brace strength (pounds)
K_{id} = "ideal" brace stiffness (pounds/inch)
K_{req} = required brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P_e = Euler buckling load for the unbraced column (pounds)
Δ = additional lateral deflection at mid-height upon application of axial load (inches)
Δ_0 = initial lateral deflection at mid-height (inches)

Winter (1960) proposed determination of K_{req} and $F_{br\text{-}req}$ using Equations 12 and 13 with assumed values for Δ_0 and Δ. Using steel columns for his examples, Winter suggests that Δ_0 be twice that allowed by code construction tolerances to account for possible load eccentricities. He also suggested that Δ should approximately equal Δ_0 in the absence of other restrictions (McGuire, 1968; Winter, 1960). With these assumptions, the designer has everything needed to design a bracing system.

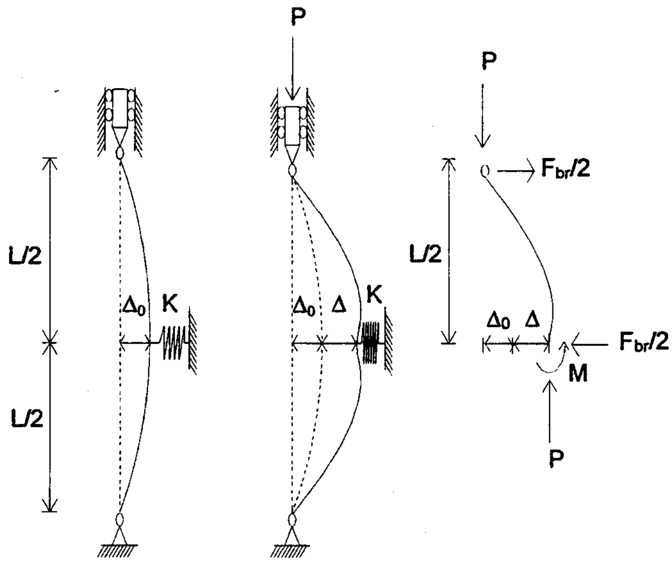
As pointed out by Winter (1960), and expanded by Yura (1996a; 1996b), Winter's model provides some useful insights into brace behavior. Perhaps most important is that Equation 12 predicts that a brace stiffness of K_{id} is not sufficient to fully brace an imperfect column. Yura establishes that with a brace stiffness of K_{id}, the lateral deflections and brace forces become infinitely large as P approaches $4P_e$. With Winter's assumption that Δ equals Δ_0, Equation 15 estimates that a stiffness of $2K_{id}$ will be required. This additional stiffness brings the lateral deflections and brace forces down to manageable levels.

Some authors have provided useful expansions of Winter's work to account for situations when less than full bracing is required to achieve adequate column strength (Green in the Appendix to Winter, 1960; Lutz and Fischer, 1985; Yura, 1996b). However, since full bracing is assumed in compression web bracing design, this expansion will not be addressed in this research.

2.2.5 "Refined" Solutions for Imperfect Columns

Recent investigators suggest that Winter's model may not always provide conservative estimates of the required brace strength and stiffness (Plaut, 1993a; Plaut, 1993b; Stanway et al., 1992). The reason for this lies in Winter's assumption of a perfect hinge in the column at mid-height which does not account for the secondary column bending moment experienced by imperfect columns immediately upon application of axial load. Figure 2.12 illustrates that reverse curvature can occur in the column at mid-height when the brace provides relatively stiff lateral support (Clarke and Bridge, 1993; Stanway et al., 1992).

The free body diagram in Figure 2.12C includes the moment that results from reverse curvature. Enforcing static equilibrium about either end of the column in this indeterminate problem reveals that the force in the brace increases as a result of this moment (Plaut, 1993b; Stanway et al., 1992).



A) Unloaded B) Loaded C) Force diagram

where:

- F_{br} = brace force (pounds)
- K = provided brace stiffness (pounds/inch)
- L = effective length of the unbraced column (inches)
- M = weak axis bending moment in the column at the brace (inch-pounds)
- P = column load (pounds)
- Δ_0 = initial lateral deflection at mid-height (inches)
- Δ = additional lateral deflection at mid-height upon application of axial load (inches)

Figure 2.12: Refined Model for Imperfect, Braced Columns

Refined stability solutions which include the effects of column moment were not generally available at the time of Winter's (1960) article. Recent authors have solved differential equations of equilibrium to analyze the stability of imperfect columns with mid-height braces of finite stiffness (Plaut, 1993a; 1993b; Plaut and Yang, 1993; Stanway et al., 1992). Assuming quadratic and sinusoidal initial column curvatures, Plaut and Yang (1993) derived exact solutions to define the relationships between the column and the brace. Unfortunately, the results are too computationally cumbersome to be useful in design. In addition, the solutions depend upon the assumed shape of initial curvature, which is difficult to determine. In response to these concerns, Plaut (1993a; 1993b) recommends the following inexact but conservative design equations to relate the brace force and stiffness (Plaut, 1993a; Plaut, 1993b):

$$K = \frac{4P}{L}\left(1 + 1.5\frac{\Delta_0}{\Delta}\right) \tag{14}$$

$$F_{br} = K\Delta = \frac{4P}{L}\left(\Delta + 1.5\Delta_0\right) \tag{15}$$

where: F_{br} = brace force (pounds)
K = provided brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P = column load (pounds)
Δ = additional lateral deflection at mid-height upon application of axial load (pounds)
Δ_0 = initial lateral deflection at mid-height (inches)

For the special case when the column load P equals to the critical column load $4P_e$, Plaut approximates the upper bounds on brace strength and stiffness to achieve asymmetric buckling as (Plaut, 1993a):

$$K_{req} = \frac{4(4P_e)}{L}\left(1 + 1.5\frac{\Delta_0}{\Delta}\right) = K_{id}\left(1 + 1.5\frac{\Delta_0}{\Delta}\right) \tag{16}$$

$$F_{br-req} = K_{req}\Delta = K_{id}\left(\Delta + 1.5\Delta_0\right) \tag{17}$$

where: $F_{br\text{-}req}$ = required brace strength (pounds)
K_{id} = "ideal" brace stiffness (pounds/inch)
K_{req} = required brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P_e = Euler buckling load for the unbraced column (pounds)
Δ = additional lateral deflection at mid-height upon application of axial load (pounds)
Δ_0 = initial lateral deflection at mid-height (inches)

These equations are similar to Winter's Equations 12 and 13. Plaut (1993a) suggests that the "1.5" multiplier on Δ_0 should conservatively account for the effects of column moment at the brace with various shapes of half-wave intial curvatures.

Several other authors have used finite element analysis or numerical integration techniques to solve the stability problem for a column with a mid-height brace (eg., Clarke and Bridge, 1993; Mutton and Trahair, 1975; O'Connor, 1979). Some of these models include variations on the solutions already discussed. For example, Mutton and Trahair (1975) study the effects of an initial lack of fit between the column and brace. However, all of these models have assumed material properties and geometry appropriate for steel construction. It is not clear whether they can be directly applied to light-frame wood trusses.

2.3 Departures from Ideal Behavior

To this point, this discussion of braced column behavior has included the following assumptions:

- a single discrete brace placed at mid-height,
- linear elastic columns,
- linear elastic braces,
- brace placement at the shear center of the column, and
- homogeneous, isotropic column properties.

Limited guidance does exist to assist with violations of the first four assumptions.

2.3.1 Off-Centered Bracing

This research focuses on the situation where the brace is placed at the mid-height of a compression web member. A discrete brace placed elsewhere on the web results in reduced capacity of the web (Brush and Almroth, 1975; Galambos, 1988; Plaut and Yang, 1993; Stanway et al., 1992; Olhoff and Akesson, 1991). Should off-centered placement become necessary, several authors address this issue for both perfect and imperfect columns (Brush and Almroth, 1975; Galambos, 1988; O'Connor, 1979; Plaut, 1993a; Plaut, 1993b; Plaut and Yang, 1993; Stanway et al., 1992; Urdal, 1969; Winter, 1960; Yura, 1996b).

2.3.2 Inelastic Column, Linear Elastic Brace

Many, if not most, wood compression web members fall into the category of "intermediate" slenderness. As presented in Sections 2.1.3 and 2.1.4, "intermediate" columns tend to buckle inelastically at a stress level below that predicted by Euler's Equation 1. None of the brace design methodologies presented so far consider inelastic column behavior. Pincus (1964) suggests that column inelasticity could actually increase the brace strength and stiffness requirements. He contends that an inelastic, intermediate column will lose lateral stiffness as the axial load increases. This stiffness loss results in larger lateral deflections, Δ, than would be expected if the column remained elastic. The brace needs to resist these larger deflections. In other words, the inelastic column loses some of its ability to support itself, so the brace must theoretically carry more of the burden. Pincus (1964) contends that rules-of-thumb, like the 2% rule, may not prove conservative for inelastic columns.

Several authors provide limited guidance on bracing inelastic columns. In general, the trend is to suggest that the bracing equations presented so far are still valid with the simple substitution of the tangent modulus of elasticity, E_t, for E (Lutz and Fischer, 1985; McGuire, 1968; Oliveto, 1979; Winter 1960; Yura, 1996a). Recently, Clarke and Bridge (1993) studied the effects of inelasticity on a specific steel column section with assumed residual stresses. However, given their assumptions, it is not clear that their analysis provides useful insight for light-frame wood trusses. Ales and Yura (1993) make an argument that Winter's equations also apply for inelastic columns since they do not depend upon E in their basic form. However, these authors do not address secondary bending moments that develop in the column. The flexibility of wood columns will almost assuredly affect lateral displacement and directly impact the required brace strength and stiffness.

2.3.3 Elastic Column, Nonlinear Elastic Brace

Using an approximate energy method, Tsien (1942) solved the problem where a nonlinear elastic brace supports a linear elastic, imperfect column at mid-height. He derives the following relationship between the column load, the force in the brace, and the initial and subsequent deflections at mid-height:

$$\Delta = \Delta_0\left[\frac{\frac{P}{P_e}}{1-\frac{P}{P_e}}\right]+\left[\frac{\frac{1}{4}-\frac{1}{2\left(\pi\sqrt{\frac{P}{P_e}}\right)}\tan\left(\frac{\pi\sqrt{\frac{P}{P_e}}}{2}\right)}{\frac{P}{P_e}}\right]\frac{F_{br}L}{P_e} \tag{18}$$

where: F_{br} = brace force (pounds)
L = effective length of the unbraced column (inches)
P = column load (pounds)
P_e = Euler buckling load for the unbraced column (pounds)
Δ = additional lateral deflection at mid-height upon application of axial load (pounds)
Δ_0 = initial lateral deflection at mid-height (inches)

Δ and F_{br} are usually both unknowns for a given column. Everything else in Equation 18 generally represents a given property of the column. A second equation relating Δ and F_{br} can be developed to describe the stiffness of the brace as part of the design process. Using these two equations, the brace force and deflection can be approximated.

2.3.4 Inelastic Column, Nonlinear Elastic Brace

To date, no research has been identified which solves the problem where a nonlinear elastic brace provides lateral support for an inelastic column. Ironically, this would probably represent the most rigorously relevant model for bracing design of light-frame wood trusses.

2.3.5 Brace Placement Away From the Column Shear Center

Some authors have examined what happens when the brace placement does not occur exactly at the shear center of the column (Horne andAjmani, 1969; Medland, 1980; O'Connor, 1979). With this condition, the potential exists for the column to buckle torsionally. Strictly speaking, a lateral brace nailed to the side of a compression web will not be attached to the web's shear center. However, it is difficult to evaluate the potential for torsional buckling given the limited amount of information available on the torsional properties of wood, the moment resistance of nailed connections, and the expected range of initial twist possible. Given these limitations and the fact that torsional buckling is generally not considered in wood design, this issue is not addressed here other than to indicate when and if torsion was evident during testing.

2.4 Multiple Braces

Situations may exist where the light-frame truss designer requires more than one row of discrete lateral bracing to reduce the unbraced length of a compression web member. In general, the critical column load and required brace stiffness for a perfect, elastic column with multiple braces can be derived using traditional elastic methods (Timoshenko and Gere, 1961). For the special case with evenly spaced braces, Green et al. (1947) provide approximate solutions. Lutz and Fischer (1985) provide further simplifications to assist with the computer-aided design of bracing.

With each added brace, an additional buckling mode becomes possible. For evenly spaced braces, the maximum practical P_{cr} corresponds with the case where asymmetrical buckling occurs between the braces. In other words, P_{cr} will be based on Euler buckling with the effective column length equal to the distance between the braces. As indicated in Table 2.2, different numbers of braces require different K_{id} values to achieve this buckling mode for a perfect column (Green et al., 1947; Winter, 1960; Urdal, 1969).

Winter's rigid link model equations for imperfect columns also apply to the column with multiple, evenly-spaced braces by simple substitution of the appropriate K_{id} values into Equations 12 and 13 (Salmon and Johnson, 1996; Winter, 1960; Yura, 1996b). Some authors have even extended the applicability of Winter's model to address the condition where less than ideal bracing has been supplied using multiple braces (Green in Appendix to Winter, 1960; Lutz and Fischer, 1985; Yura, 1996b). Refined solutions that consider the moment in the imperfect column at the brace are not currently available. For simplicity, columns with multiple braces will not be specifically addressed by this research.

Table 2.2: Maximum P_{cr} and K_{id} for Perfect Columns with Multiple, Evenly Spaced Braces

Number of Evenly Spaced Braces	K_{id}	P_{cr} for the Highest Practical Buckling Mode
1	$\frac{16P_e}{L}$	$4P_e$
2	$\frac{81P_e}{L}$	$9P_e$
3	$\frac{218.24P_e}{L}$	$16P_e$
4	$\frac{453.75P_e}{L}$	$25P_e$

where: L = effective length of the unbraced column (inches)
P_e = Euler buckling load for the unbraced column (pounds)

2.5 Column Assemblies

Truss compression web members rarely behave as single, braced columns. As illustrated in Figure 1.4, discrete web bracing usually consists of a lateral brace that ties together the webs of several adjacent trusses. Periodic diagonal braces prevent the group of webs from buckling simultaneously. Figure 2.13 illustrates one way to conceptualize such an interbraced column assembly.

Very few authors address the issue of interbraced, multiple column assemblies. Some recommend that all of the braces be designed to meet the strength and stiffness requirements for the most heavily loaded brace (Galambos, 1988). In Figure 2.13, the brace closest to the rigid support would carry the largest lateral load and govern the design. Galambos (1988) suggests that if five columns are in the assembly, then all the braces should be designed with five times the strength and stiffness required to support a single column.

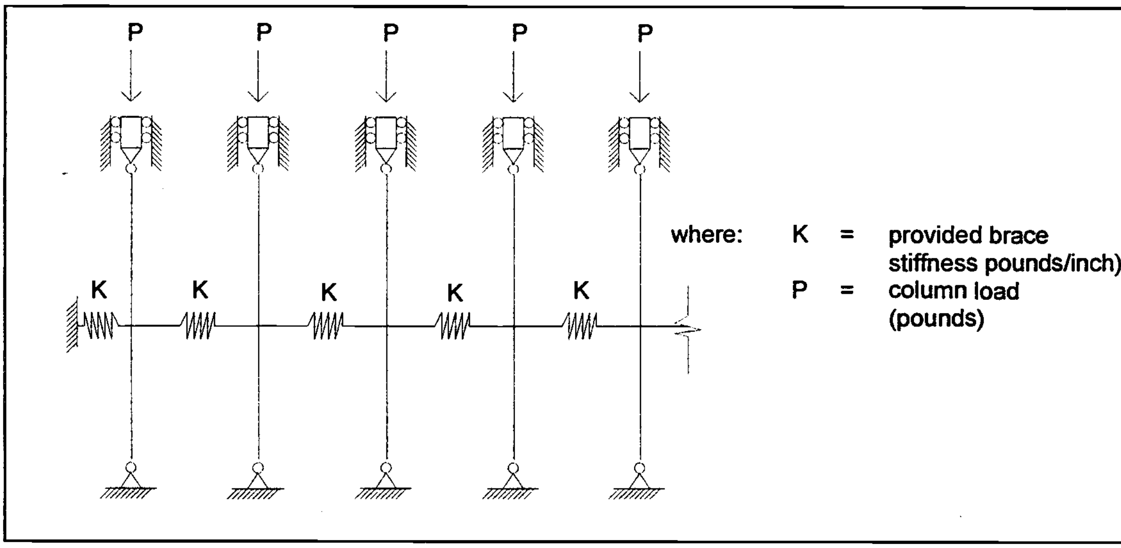


Figure 2.13: Multiple Column Model

Medland and Segedin specifically investigated the brace strength and stiffness requirements for interbraced columns (Medland, 1977; Medland, 1980; Medland and Segedin, 1979; Segedin and Medland; 1978). They concluded that interbraced column assemblies, even with perfect columns, require significantly more brace stiffness than suggested by Galambos (1988).

However, Medland and Segedin's assumed models differ substantially from the braced wood truss web problem. First, Medland and Segedin did not consider material variability between the columns of the assembly. Woeste (1998) suggests that braced wood columns may receive some support from other columns within the assembly simply due to their natural variability in stiffness. Since we design wood columns based on the lower 5% exclusion limit of stiffness, the individual webs within the aggregate group should buckle at a P_{cr} larger than that assumed in the design.

Secondly, the lateral/diagonal bracing scheme often used for compression web bracing in light-frame wood trusses differs from that illustrated in Figure 2.13. In practice, a single lateral brace ties together all of the web members with nailed connections. These nailed connections are more flexible than the lateral brace, so the brace tends to move as a rigid element with approximately equal lateral displacement of the webs. With the conceptual model of Figure 2.13, the lateral deflections between columns would tend to differ across the assembly. Lateral load accumulates in the brace elements closest to the rigid support. Larger loads create larger deflections in accordance with Equation 5. This suspected behavioral difference is enough to question the applicability of Medland and Segedin's modeling for the problem of compression web bracing.

2.6 Current Compression Web Bracing Design Aids For Wood Trusses

All of the above-mentioned theories and solutions represent general approaches and approximations that are not material specific; but nearly all seem to have been directed initially at steel structures. This section summarizes available information in the U.S. to assist with compression web bracing design for light-frame wood trusses.

The current U.S. standard for light-frame wood trusses, the *National Design Standard for Metal Plate Connected Wood Truss Construction* (Truss Plate Institute, 1995), emphasizes the importance of compression web bracing, provides schematic guidelines on how to configure it, and places the responsibility for its design upon the building designer. However, this standard does not provide a method to determine the required brace strength and stiffness. The *National Design Specification for Wood Construction* (American Forest and Paper Association, 1997) also fails to provide specific design guidance.

Modern timber design texts, including Callahan's (1993) book on light-frame wood trusses, emphasize the importance of compression bracing to reduce the effective lengths for wood truss members (AITC, 1994; Hoyle and Woeste, 1989; Stalnaker and Harris, 1989). However, no text provides specific design information. The American Institute of Timber Construction (1994) does state that the strength of a bracing system for compression "chords" should be based on 2% of the force in the chord. That's if the member is in "perfect alignment." No specific guidance covers the situation when the member includes initial curvature from normal construction tolerances. In addition, AITC provides no guidance on the required stiffness of the bracing system.

Design texts often refer to or summarize *BWT-76: Bracing of Wooden Trusses*, a historic document published by the Truss Plate Institute (1976) for bracing design. BWT-76 provided suggested installation guidelines for wood truss bracing and included a section to address compression web bracing. It described the installation of a row of horizontal bracing, like that shown in Figures 1.3 and 1.4, at locations where the truss designer assumed compression web support to reduce the effective length. In addition, BWT-76 required the building designer to provide restraint to stop the webs from buckling simultaneously and displacing the lateral brace. It did not provide direction on calculating the required strength or stiffness of the restraint. Nearly all of this information is now incorporated into the *National Design Standard for Metal Plate Connected Wood Truss Construction.* However, unlike the current truss design standard, BWT-76 further recommended that the diagonal brace spacing "not exceed 20 feet, or twice the horizontal run of the diagonal bracing" and that the diagonal braces be two inch thick lumber.

Two recent Truss Plate Institute (1989; 1996) documents supercede BWT-76: *DSP-89: Specifications for Temporary Bracing of Metal Plate Connected Wood Trusses* and *HIB-91: Commentary and Recommendations for Handling, Installing & Bracing Metal Plate Connected Wood Trusses.* HIB-91 essentially contains the same compression web bracing recommendations as BWT-76.

DSB-89 does not address compression web bracing, but it does require temporary bracing of compression chords to be designed for 2% of the compression chord load. The required brace stiffness is not discussed. When multiple chords are braced together, the bracing system is to be designed for 2% of the cumulative sum of their compressive loads. The number of diagonal braces is determined by the strength of the nailed connection between the webs and the diagonal.

It is interesting to note that some research has been directed toward the design of lateral bracing for the chords of light-frame wood trusses. Ongoing research in England will provide specific information on chord bracing requirements for permanent truss stability of pitched roof trusses (Milner, 1996; Bainbridge et al., 1997a; Bainbridge et al., 1997b). Hoyle (1984) investigated the lateral bracing requirements for the chords of parallel chord wood trusses. However, all of this work differs substantially from the problem at hand since, unlike webs, chords experience axial force magnitudes that vary over their lengths. In addition, they are often continuously sheathed or braced with numerous purlins or lateral ties. In either case, almost continuous lateral support exits. Compression web members usually receive discrete support at only a few locations under the best of conditions.

2.7 Experimental Data

Surprisingly little published experimental data exists to verify the models discussed above. Green et al. (1947) tested a large number of light-gauge steel columns in compression. The columns had discrete braces of various strengths and stiffnesses to test the theoretical relationships between brace stiffness and critical column strength. In general, he found good agreement with his approximations for these relationships. However, other than to note when the discrete braces failed, Green did not measure the force in the brace.

Some authors have measured the brace forces required to stabilize compression chords. From his testing and analysis of pitched roof trusses, Pienaar (1986) estimates that the lateral load at each top chord brace is approximately 10% of the axial force in the chord. Hoyle (1984) measured brace forces in discretely braced chords of parallel chord wood trusses and found that brace forces varied between 0.3 and 5% of the axial force in the chord depending upon the type of lateral restraint provided. Hoyle did not report or discuss the contribution of brace stiffness to the problem. To date, no literature has been found which describes a test program to quantify the strength and stiffness requirements for discrete bracing for compression web members of light-frame wood trusses.

2.8 Literature Review Summary

In summary, clear design guidance does not exist for the wood truss compression web bracing system. Several theoretical models provide simplified methods that may approximate the relationship between the brace and the web. The

2% Rule (Section 2.2.3) represents a common, simple brace strength design criteria. Plaut's method (Section 2.2.5) and Winter's method (Section 2.2.4) both provide simplified ways to estimate both the required brace strength and stiffness. Tsien's equation (Section 2.3.3) goes a step farther in that it also considers the possibility that the brace might supply inelastic support for the column. All four of these simplified models should be questioned for one or more of the following reasons:

- Plaut's method, Winter's method, and Tsien's equation all assume the web member consists of a homogenous, isotropic material. This may not be an appropriate assumption for wood, especially when you consider that truss compression webs often use dimension lumber as low as "Standard" grade. This material can include a significant amount of slope of grain, knots, splits, wane, and other defects that violate the homogenous and isotropic material assumptions.

- Most established guidelines were developed for application in non-wood structures. They might not be appropriate for the detailing, tolerances, and material properties of timber construction.

- Theoretically, the magnitude of the brace force and the effectiveness of a brace depend upon its stiffness. The 2% Rule includes a check of the brace strength, but not stiffness. This rule provides no assurance that the brace provides sufficient support for the column.

- With nailed brace connections and intermediate web slenderness ratios, the compression web bracing problem does not strictly adhere to the assumptions of linear elastic column and brace behavior. The importance of non-linear behavior should be examined.

- Perhaps the principal source of doubt comes from the fact that very limited physical testing has been conducted to validate the available rules-of-thumb and mathematical models for any material.

3.0 STIFFNESS MODELING

3.1 Objective

As discussed in Section 2.2, the stiffness of a discrete mid-height brace directly impacts column performance. When designing a braced compression web, the truss designer assumes that the brace forces the web to buckle asymmetrically with an effective buckling length of half the web length, L_w. For this to happen, the brace must provide both sufficient strength and stiffness.

The overall objective of this project is to determine if one of several simplified analysis models can estimate brace strength and stiffness requirements. To this end, we compared each of the models with the results of a physical test program. For applicability, any brace stiffness used in the testing should be in the range of that expected from a commonly used lateral/diagonal brace system. Figure 1.4, repeated here as Figure 3.1, illustrates such a system. The objective of this project phase was to assess the approximate strength and stiffness of support provided to a compression web by a lateral/diagonal bracing system.

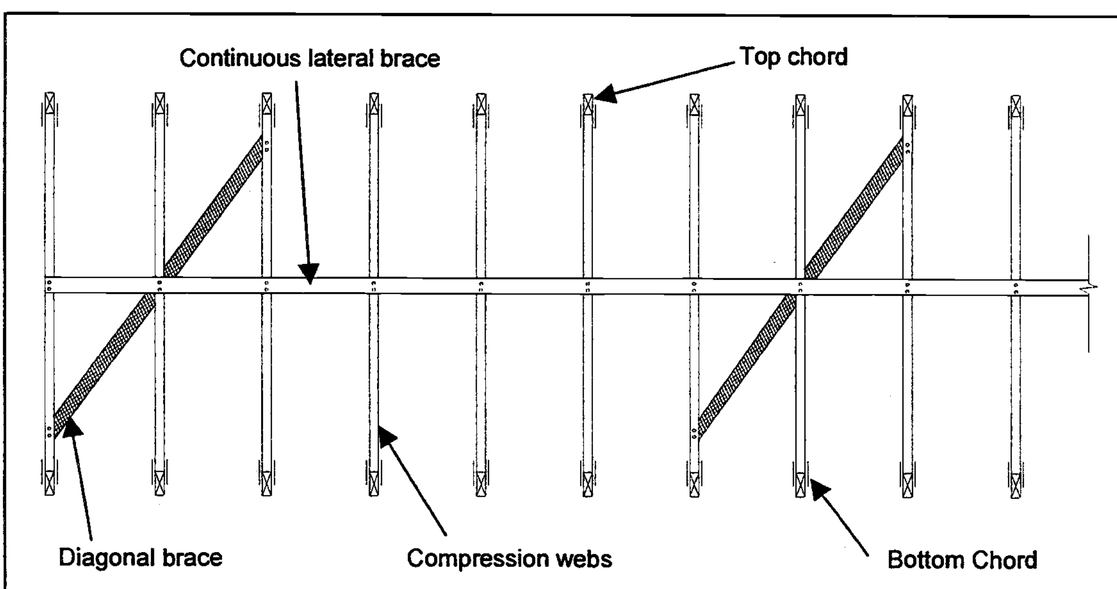


Figure 3.1: Lateral/Diagonal Bracing System

3.2 Major Simplifying Assumptions

Modeling the lateral/diagonal brace system illustrated in Figure 3.1 requires a number of simplifying approximations. The following sections summarize the simplifications used for this research.

3.2.1 Compression Web Loading

Wood truss engineers usually consider uniform roof load conditions for most design purposes. If all of the trusses in the roof system are identically configured and uniformly loaded, then all of the compression web members tied together by a lateral brace support the same axial load. The modeling in this project assumes that all web members attached to a single lateral brace carry an equal axial load. Variations in the roof loading or truss configuration would contradict this assumption.

3.2.2 System Effects

The lateral brace of Figure 3.1 ties together a group of compression web members into a single assembly. Each wood web within the assembly represents an individual member that may vary with respect to its:

- axial strength and stiffness,
- flexural stiffness about the weak axis,
- shape of initial curvature,
- magnitude of initial curvature, and
- direction of initial curvature.

Even if all the webs carry the same axial load, this natural variability creates the potential for each web member to require a different level of brace support. Since the lateral brace ties them together, the support provided to any one depends on the aggregate behavior of the group. An exact solution to the problem requires complete characterization of all the web members. This information is not practically available and a simplified approach was adopted.

This research assumes that all the web members tied together by a lateral brace make equal demands on the bracing system. In other words, each web member requires the same strength and stiffness of support at mid-height in the same lateral direction. This simplification requires all of the web members within the assembly to be identical with respect to their material properties. In addition, all of the webs are assumed to have the same magnitude and direction of initial curvature. If the brace designer uses the upper bound of initial curvature and lower bound of column strength and stiffness to define the webs, then these assumptions should be conservative.

In reality, simply connecting a group of variable columns together with a lateral brace may improve the brace support stiffness over the assumed conditions. For example, some initial lateral web deflections may occur in opposite directions. Rather than deflect as a unit as shown in Figure 1.5, the individual web members might lean upon each other for support through the lateral brace. With less demand for restraint placed on the diagonal brace, this could dramatically improve the stiffness and strength of lateral support provided to any web within the assembly. In addition, as suggested by Woeste (1998), the stiffer webs tied together by the lateral brace might provide support for the more flexible members. Since the compression web design procedure incorporates a lower 5% exclusion limit on E, a high probability exists that most of the webs in the group will be stiffer than assumed by the truss designer. This unaccounted extra stiffness would also reduce the lateral restraint demands on the diagonal brace. Factors such as these suggest the possibility of a "system" effect that may influence the performance of the brace by increasing the stiffness of support provided to any one web.

Eventually, system effects should be evaluated. However, a system effect should primarily impact the brace strength and stiffness provided to a compression web member. It should not affect the relationship between the brace and the column. Since the objective of this project is to investigate the latter, system effects are not included.

3.2.3 Diagonal Brace Restraints

A nailed connection transfers the horizontal force from the braced compression web to the lateral brace. The lateral brace collects the load from all of the attached webs and transfers it to the diagonal brace through a series of nailed connections. As illustrated in Figures 3.1 and 3.2, nailed connections also attach the diagonal member to a series of webs.

From the author's experience, the current industry design practice is to stop defining the load path for the brace force at the ends of the diagonal brace. The reason for this may be that the brace forces are usually very small in comparison with the other forces considered during a building design. After transferring the brace force to the ends of the diagonal brace, a number of different sheathing and bracing systems might be present at the chord level to collect the load and ultimately transmit it to the foundations.

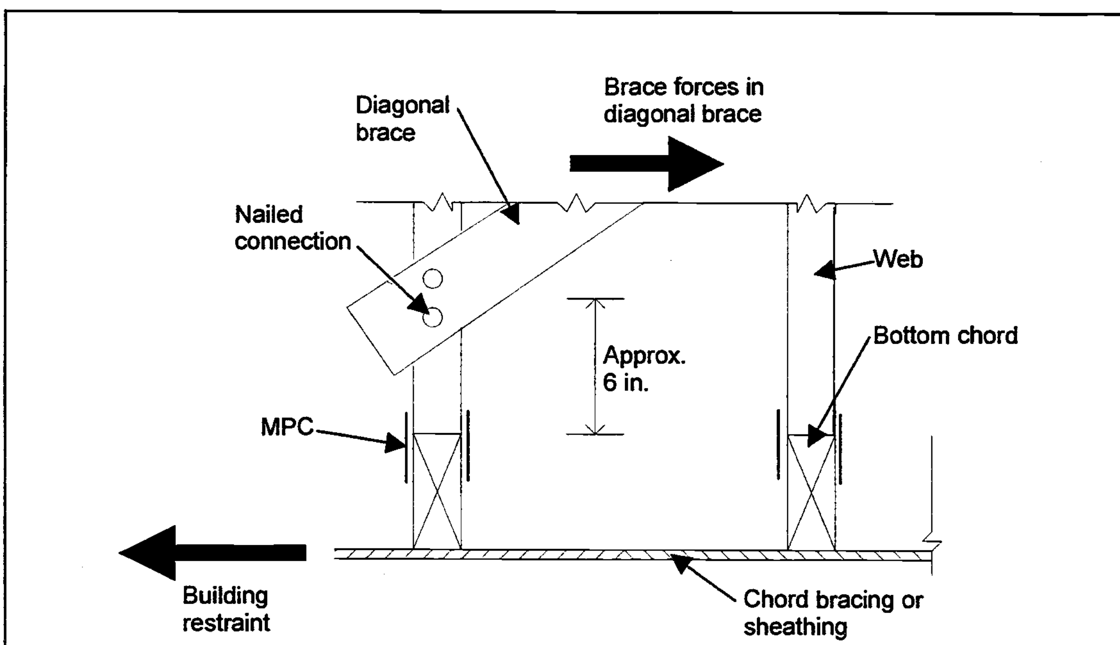


Figure 3.2: Boundary Condition for the Diagonal Brace

To exactly define the lateral support provided to the compression webs would require the load path to be analyzed from the web to the foundation. The process of defining this path would necessitate numerous assumptions concerning the underlying construction and loading conditions. However, the brace loads are generally small and such an endeavor may not be justified. If the building design incorporates diaphragm sheathing or a lateral brace for the chords near the ends of the diagonal brace, then considerable opportunity exists for relatively rigid support to distribute the brace load through the structure. In addition, the ultimate goal of this modeling is only to approximate the magnitude of support stiffness provided, not pinpoint it for a specific building configuration. To that end, this analysis assumes that a reasonable estimate of support rigidity is achieved by conceptually attaching the ends of the diagonal brace to a rigid body with a nailed connection.

3.2.4 Splices

With some larger structures, dozens of compression webs may be tied together with the same lateral brace. This often necessitates splicing of the lateral brace. The location and number of lateral brace splices provided could potentially impact the stiffness of the brace support. For simplicity, lateral brace splices were not considered. In other words, this modeling assumes that at least one diagonal brace has been provided for each length of lateral brace lumber.

3.3 Model Development

Figure 3.3 schematically illustrates the finite element model used to estimate the stiffness of support provided by a lateral/diagonal bracing system.

3.3.1 Geometry

Modeling of the brace system requires some assumptions concerning the probable geometry of the system components. The models for this research assume a horizontal truss spacing of 24 inches on center along the lateral brace of Figures 3.1 and 3.3. This spacing finds frequent application in wood truss roof systems.

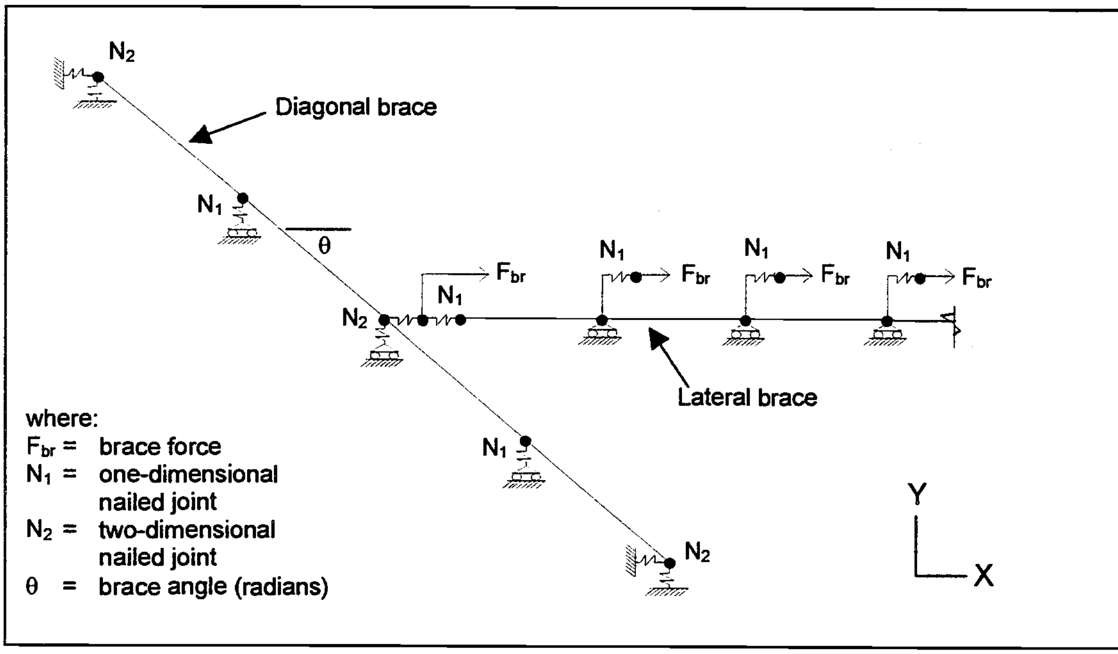


Figure 3.3: Finite Element Model for a Diagonal/Lateral Brace Assembly

The geometry of the diagonal member is defined by the length of the web members, truss spacing, and the number of webs crossed. Truss Plate Institute (1976; 1995; 1991) guidelines suggest that the diagonal brace should be installed at about a 45^0 angle from horizontal. This phase of the project used four finite element models to approximate the stiffness of support provided to a series of 4, 6, 8, and 10 foot long compression web members. Assuming the diagonal brace to be nailed 6 inches from each end of the webs, Table 3.1 describes the diagonal brace orientation that approximates a 45^0 angle for θ. These fixed orientations define the finite element model geometry assumed in Figure 3.3 for the 4, 6, 8, and 10 foot web lengths.

As illustrated in Figures 3.1 and 3.3, all of the models assume that the diagonal and lateral braces intersect at the front and back of the same web member. This orientation avoids the introduction of unnecessary weak-axis bending into the web and agrees with Truss Plate Institute (1991) recommendations.

Table 3.1: Diagonal Brace Geometry

Web Length (feet)	Web Spacing (inches)	Brace Angle θ (degrees)	No. of Webs Crossed
4	24	37	3
6	24	36	5
8	24	41	5
10	24	48	5

3.3.2 Connections

The springs of Figure 3.3 represent nailed connections. Consistent with Truss Plate Institute (1991) recommendations, all wood-to-wood connections in the brace assembly were assumed to consist of two 16d nails. Equation 19, developed by Mack (1966) provides an estimate of load-slip response that can be expected from this type of connection:

$$F_{nail} = 574(3.20\Omega + .68)(1 - e^{-75\Omega})^{0.7} \qquad (19)$$

where: F_{nail} = load applied to a 2-16d nailed joint (pounds)
Ω = slip between the wood members of a 2-16d nailed joint (inches)

Figure 3.4 illustrates this relationship. Equation 20 applies to wood-to-wood member connections of dry, two-inch nominal Douglas-fir. To remain valid, Mack (1966) suggests an upper boundary of about 0.1 inches on Ω. This equation was used to describe the joint behavior for all side-grain connections of the wood members in the brace assembly. The assumed lateral/diagonal brace configuration did not include any end-grain connections.

The springs marked "N_1" in Figure 3.3 provide uniaxial connection behavior. For example, the web members transmit horizontal loads to the lateral brace through a horizontal "N_1" connection. The "N_1" connections between the diagonal brace and the webs provide only vertical restraint. This second assumption conservatively neglects the weak-axis lateral support that might be offered to the diagonal brace by the buckling web member and vice versa.

"N_2" nailed connections occur at the ends and middle of the diagonal brace. The resultant forces at these connections possess both X and Y components. Two component springs model this condition. The stiffness of each component spring within these connections was calibrated so that the resultant force experiences the load-slip behavior of Equation 19 in the resultant direction. While a number of different techniques could be used to accomplish this, Appendix A outlines the trial and error procedure adopted for this research.

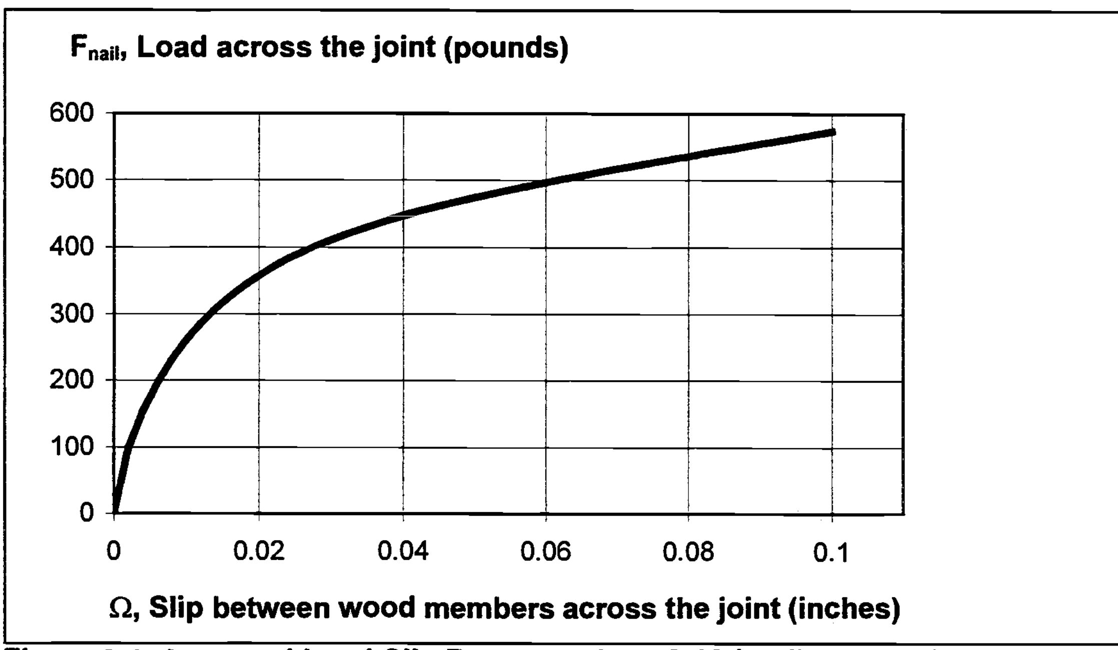


Figure 3.4: Assumed Load-Slip Response for a 2-16d Nail Connection

3.3.3 Members

All of the modeling in this project assumes that the lateral and diagonal braces consist of 2x4 wood members with a modulus of elasticity of approximately 1,200,000 psi. This lumber dimension satisfies Truss Plate Institute (1991) recommendations for truss bracing.

3.3.4 Loads

In Figure 3.3, F_{br} denotes the lateral brace force applied to the brace assembly by a simple compression web member. An F_{br} occurs at each node where the lateral brace attaches to the mid-height of a compression web. As discussed in Section 3.2.2, all of the F_{br} values are considered equal in magnitude and direction.

3.4 Brace Curves

A total of four finite element models were generated to estimate the stiffness of brace support provided to web members of 4, 6, 8, and 10 foot lengths. Each finite element model was repeatedly analyzed using ANSYS® Version 5.4 (ANSYS, Inc., 1997). ANSYS®, a widely-used finite element analysis software package, was selected for its non-linear capabilities and ready access at the Forest Research Laboratory.

For each of the four models, the analysis procedure began by considering the case where only a single web member transmits an F_{br} into the brace assembly at the loaded node closest to the diagonal brace. This corresponds to a condition where a parallel diagonal brace exists for every web member in Figure 3.1. Each of the assumed diagonal braces would have the geometry, including the number of webs crossed, described in Table 3.1. By progressively increasing F_{br} and solving for the horizontal deflection at the node of load application, the horizontal load/deflection response experienced by the web was defined. The curves marked "1 Truss" in

Figures 3.5, 3.6, 3.7, and 3.8 depict these responses for each of the four web length models.

After completing the analysis for one web member, the process was repeated for the case with two web members transmitting equal F_{br} forces into the brace assembly at the two loaded nodes closest to the diagonal brace in Figure 3.3. This corresponds to a condition where a diagonal brace exists for every two web members. Again, each diagonal brace was assumed to have the geometry described in Table 3.1. The horizontal load/deflection response of the brace support was determined by progressively increasing the magnitude of F_{br} and solving for the largest horizontal deflection experienced by a web member. Then another web member was added, and so on. Up to 10 web members were considered for each model. Figures 3.5, 3.6, 3.7, and 3.8 summarize the results.

3.5 Modeling Observations

Figures 3.5 through 3.8 represent the product of this project phase. In essence, each curve within these figures provides an estimate of the mid-height brace support that can be provided to a compression web by a lateral/diagonal bracing system. They summarize the cumulative effects of all the wood members and nailed connections within the assembly. For example, if a lateral/diagonal bracing system ties together ten web members of ten-foot length, then the "10 trusses" curve of Figure 3.8 defines the support stiffness provided to an individual web member within the assembly.

3.5.1 Inelasticity

Inspection of Figures 3.5 through 3.8 reveals that all of the brace curves show a non-linear relationship between the brace load and displacement. In addition, comparison of these figures with Figure 3.4 further reveals that the cumulative brace stiffness provided by the lateral/diagonal assembly is always less than that of a single nailed connection. These observations can be traced to the inclusion of non-linear nailed joints within the brace assembly. They dominate the response. In addition to

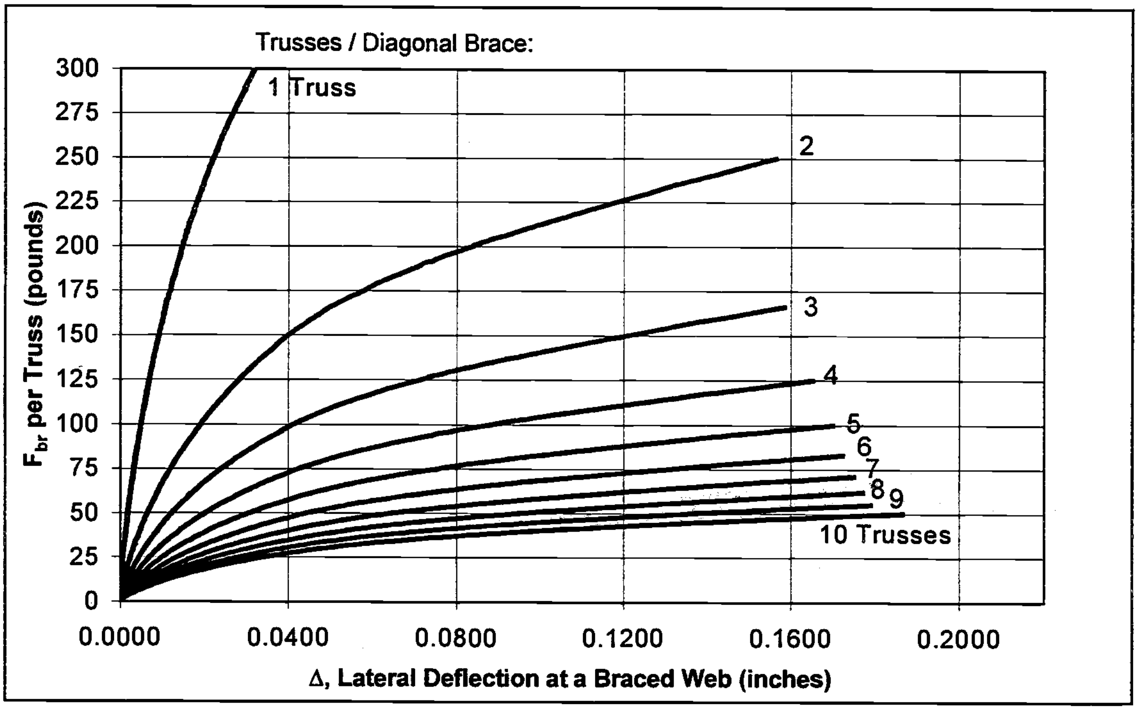


Figure 3.5: Cumulative Brace Support Curves for a 4 Foot Compression Web

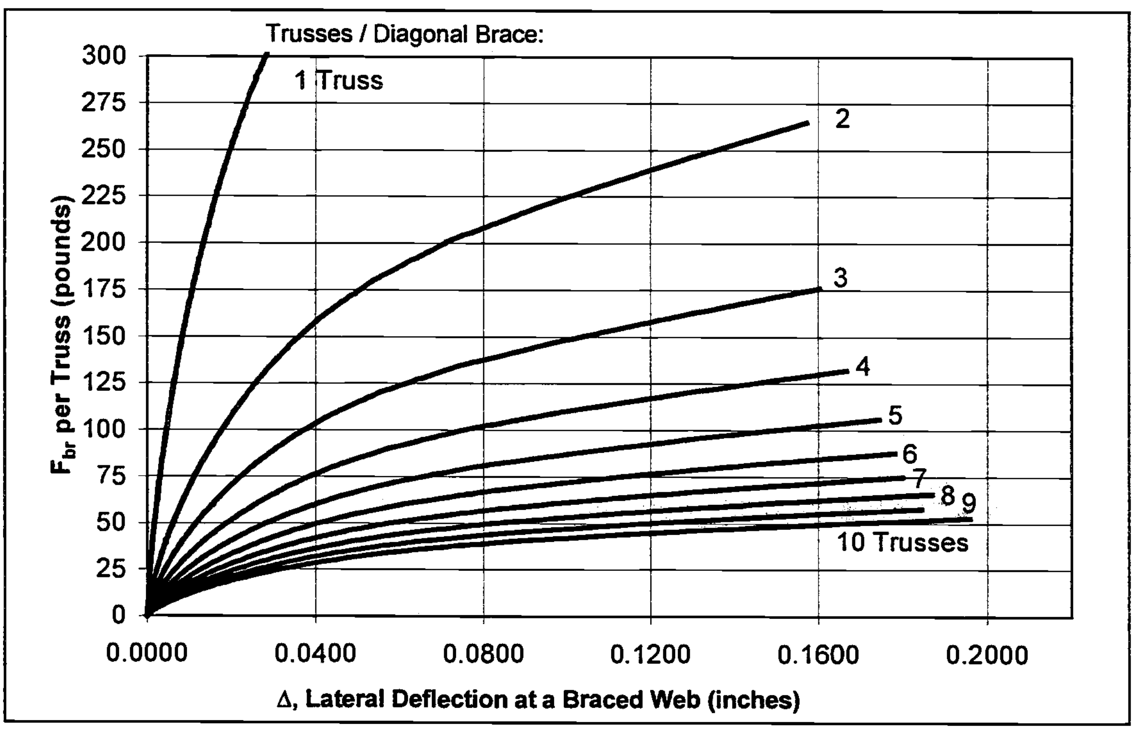


Figure 3.6: Cumulative Brace Support Curves for a 6 Foot Compression Web

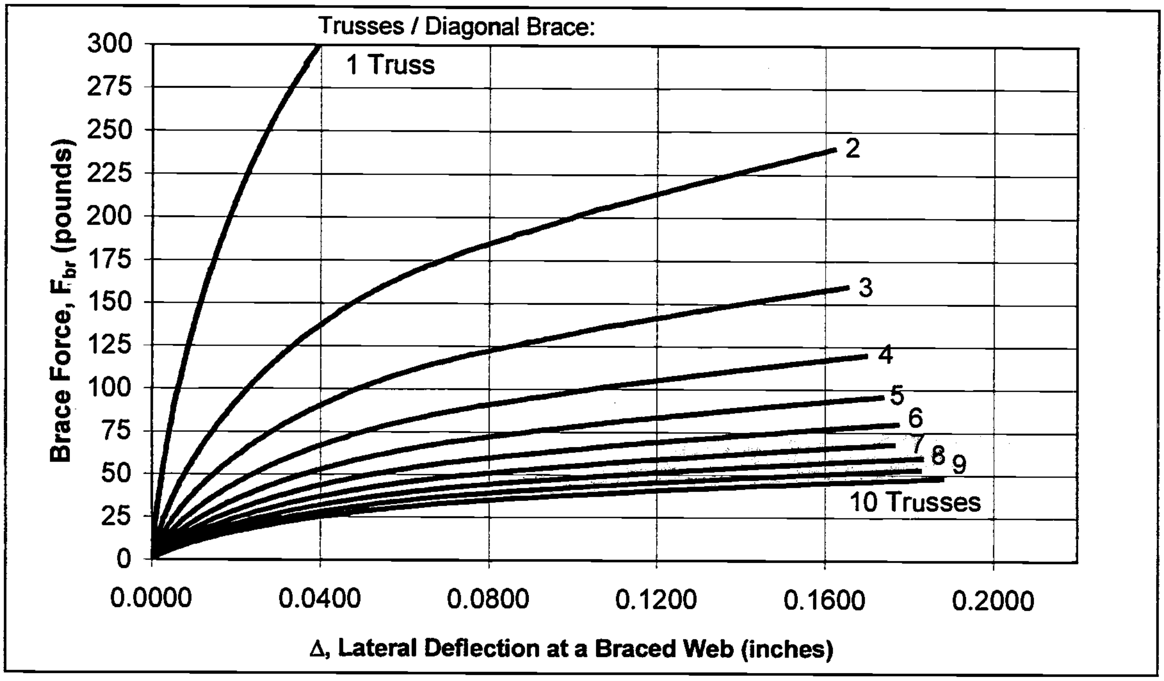


Figure 3.7: Cumulative Brace Support Curves for an 8 Foot Compression Web

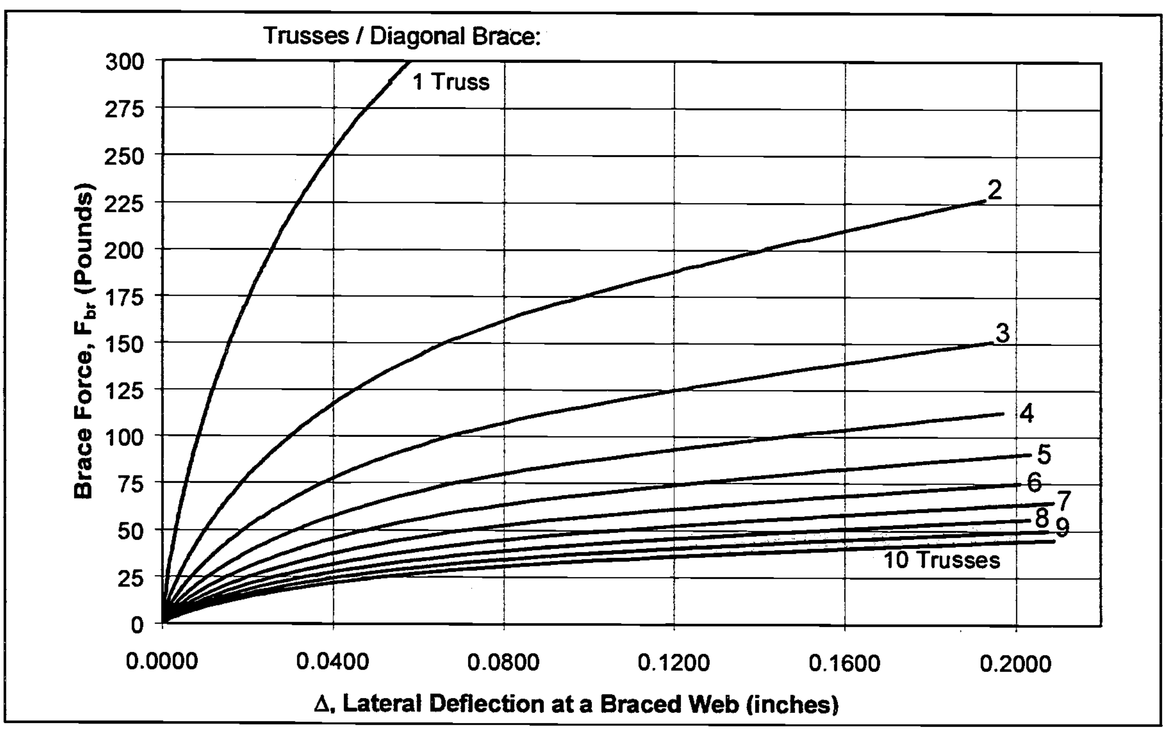


Figure 3.8: Cumulative Brace Support Curves for a 10 Foot Compression Web

being the most flexible components of the brace system, the load path places them in series. This accounts for why their cumulative stiffness is less that that of any single component.

Elastic strain within the wood members does contribute to deflection of the assembly. However, these effects are secondary to the response of the nailed joints. Changing the stiffness of the wood members from 1,200,000 to 1,400,000psi resulted in about a 3% change in the brace force for a given cumulative brace curve deflection.

3.5.2 Multiple Web Members

Figures 3.5 through 3.8 illustrate that the stiffness of brace support provided to any one web member is a function of the number of webs tied together by the same lateral brace. The stiffness provided to any one web decreases with the addition of more webs per diagonal brace. The reason for this is that the load from multiple web members accumulates in the diagonal brace and its connections, which serve as common elements for the brace assembly. With six webs contributing $F_{br,}$ there is twice the load in the common elements than with three web members. The common elements deflect more under the larger load.

3.5.3 Brace Strength

In addition to estimating the cumulative brace stiffness, the brace curves of Figures 3.5 through 3.8 also provide a rough estimate of brace strength. Each curve terminates when the "N_2" connection between the center of the diagonal brace and the backside of the web achieves a resultant displacement of 0.1 inches. This represents the upper bound of Equation 19. Mack (1966) considers this a practical limit on working joint displacement. It also exceeds the 5% offset yield strength of the nailed joint.

The N_2 connection between the diagonal brace and the web member represents the most heavily loaded connection in the assembly. It is the only one that transmits the entire horizontal load imparted on the assembly by the braced webs. In addition, it must resist the tendency for the diagonal brace to bend about its strong-axis in the positive Y direction. For this reason, it is always the connection that reaches Mack's deflection limit first and thus can be used to approximate the strength of the assembly. Compared to the nailed joints, the wood components of the assembly experience light loading conditions and do not govern the brace strength.

4.0 MATERIALS AND METHODS

The following sections summarize the test materials and methodologies used for this thesis investigation. All of this work took place at the Forest Research Laboratory (FRL) of Oregon State University during the first six months of 1998.

4.1 Objective

As discussed in Section 2, several simplified relationships exist which may provide a reasonable basis to estimate discrete compression web bracing requirements. The objective of this phase of work was to physically measure the brace forces required to stabilize a series of individual test columns. A comparison between the estimated and measured brace forces for each column ultimately provides a basis for judging the performance of the proposed analysis methods.

4.2 Experimental Design

The compression web members of light-frame wood trusses serve as the practical focus of this bracing research. In the early spring of 1998, an informal telephone survey of several Oregon truss manufacturers revealed that all of them used 2x4 Douglas-fir lumber of “Standard and Better” grade for their compression web stock. Based on this information, Table 4.1 illustrates the experimental design adopted for this study. This experimental design tests the applicability of the design equations over a reasonable range of compression web lumber. For every replication within the table, the brace force required to stabilize the 2x4 column was measured

Rather than test a single “Standard and Better Grade” mix, we elected to test “Select Structural” and “Standard” grade material. Two justifications support this breakdown. First, the “Standard and Better” grade classification represents a mixture of several 2x4 structural grades. As a mix, the potential exists for a wide range of material variability that may be difficult to characterize with a representative sample.

Testing only the “Select Structural” and “Standard” grades allows for evaluation of the brace analysis methods for the upper and lower bounds of the “Standard and Better” mixture. The second reason for the grade selection relates to the brace analysis methods to be evaluated. They generally envision a homogeneous, isotropic, and prismatic column. By testing a high and a low grade of 2x4, the departures from this assumption can be investigated. In general, the amount of wane, slope of grain, knot sizes, and other lumber defects increase with the lower grade material.

Table 4.1: Target Experimental Design

Column Length (feet)	2x4 Douglas-fir Structural Grades	
	Select Structural	Standard
4	100 pieces	100 pieces
6	100 pieces	100 pieces
8	100 pieces	100 pieces
10	100 pieces	100 pieces

The experimental design also provides the ability to judge the performance of the brace analysis methods over a 4 to 10 foot range of web lengths. Many light-frame wood truss compression webs fall within this range. In addition, the brace analysis methods that consider brace stiffness assume that the column behaves in a linear elastic fashion. As discussed in Section 2.1, this may not be true for shorter web members that experience inelastic buckling. Testing different lengths should provide insight into the performance of the brace design methods with departures from the linear elastic assumption.

The sample sizes of Table 4.1 ensure that the distribution of results for each length and grade could be identified and that tolerance limits, if appropriate, could be established. The experimental design was also limited by time and available resources.

4.3 Sampling

We purchased the test lumber from three sawmills to make the sample more representative of the broader market. The sample mills were:

- Douglas County Forest Products (DCFP), Winchester, Oregon
- Frank Lumber Company (FLC), Mill City, Oregon
- Morton Forest Products Company (MFP), Morton, Washington

Each mill produced 150 pieces of "Select Structural" and 150 pieces of "Standard" grade, 10 foot, 2x4 Douglas-fir lumber for the purpose of this study. The order specified that the mills kiln-dry the lumber to an average moisture content below 19%.

None of these three mills routinely sorts their production for these two visual grades. For the study material, DCFP and MFP graders specifically pulled and graded the sample lumber in accordance with the West Coast Lumber Inspection Bureau grading rules (WCLIB, 1996). With the assistance of WCLIB grading supervisors, FLC also graded the lumber on site to the same standards. The graders pulled all of the lumber over the course of a single day from each mill's normal production. Following production, we transported the lumber to the FRL.

Two significant departures from the specified order occurred. First, a check of the initial moisture contents revealed that the material produced by DCFP and MFP was wetter than specified. Moisture content readings taken with an electric-resistance type meter ranged between 18 and 50%. Every piece of FLC material checkedhad an initial moisture content below 19%. To remedy this problem and minimize the influence of moisture content on the study, all of the lumber was stickered and air-dried in the laboratory for a few weeks prior to testing.

The "Standard" grade material produced by DCFP provided the second departure from the specified order. Instead of providing only "Standard" grade material, the graders at DCFP actually pulled a "Standard and Better" sort which appeared to include some pieces that exceeded the WCLIB requirements for "Standard" grade. While the overall visual quality of the "Standard" material from DCFP was not superior to the "Select Structural" lumber produced by the same mill, this group contained a significant amount of clear, dense lumber. Other than to visually identify and note the lumber that was obviously much better than "Standard" quality, this material was treated the same as the "Standard" lumber produced by the other two mills.

Once the lumber reached the FRL, we assigned a random number to each piece in the overall sample set. Using that random number as an index, we conceptually sorted all of the 10 foot long 2x4's in ascending order. The first 109 pieces of "Select Structural" lumber from that sort were assigned to the "4 foot-Select Structural" test cell of Table 4.1. The first 109 pieces of "Standard" grade lumber were assigned to the "4 foot-Standard" test cell. The second 109 pieces of "Select Structural" grade were assigned to the "6 foot-Select Structural" group, and so on. Eventually, each cell of Table 4.1 contained 109 pieces of 10 foot long lumber. By assigning additional samples, the hope was to ensure that the target of 100 replications within each test cell would be achieved with some sample losses anticipated during testing. We did not use the 28 pieces of lumber left over from the initial sort for this study.

Following the initial sort, we trimmed each sample to its final test length. The final test lengths were four inches shorter than the nominal class length. For example, we cut all of the lumber in the 4 foot length group 44 inches long. This sample length allows for a full 4 feet between the hinges that support the column on the test apparatus. Only one test sample was cut from each initially 10 foot long board. We did not incorporate the spare material into this study. By using knot size, slope of grain, splits, and other defects recognized by the grading rules as a guide, the highest quality end of each "Select Structural" stud was retained as the sample board. The "Standard" grade samples represent the lowest quality end of the "Standard" grade 2x4's.

4.4 Test Apparatus

We constructed a new test machine at the FRL to test the lumber columns. Figures 4.1 through 4.8 illustrate the machine's most important attributes. Appendix B provides a detailed listing of the critical components. The author and FRL staff accomplished all of the design and fabrication of the test equipment.

4.4.1 Test Machine

A rectangular steel frame salvaged from an obsolete tension test machine forms the skeleton of the test apparatus. The long sides of the frame consist of two 6 ¾ in. diameter steel pipes. Heavy steel beam sections serve as the short ends. Continuous welds hold all of the pieces together. As indicated in Figure 4.1, the horizontally-oriented frame rests upon a series of pipe legs.

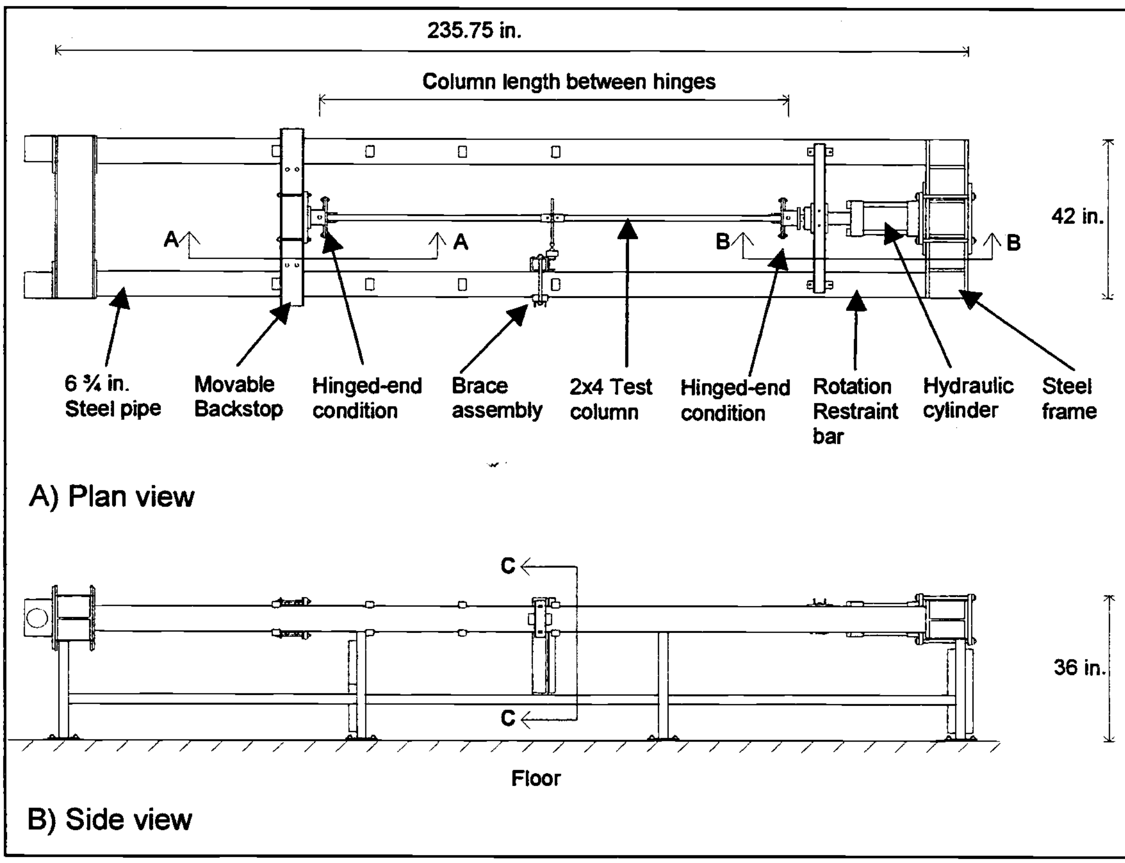


Figure 4.1: Braced Column Test Machine

Within the frame, an 8 inch diameter hydraulic cylinder provides compressive axial load to a 2x4 test column. To allow for testing of different column lengths, a movable steel bridge serves as a backstop at the opposite end of the machine. Figures 4.3 and 4.4 illustrate the backstop construction. The backstop spans between the pipes and rests upon a series of steel channel sections welded to the pipes. The position of the channel sections, or "studs" allow the movable backstop to be positioned to accommodate 4, 6, 8, and 10 foot long 2x4 columns.

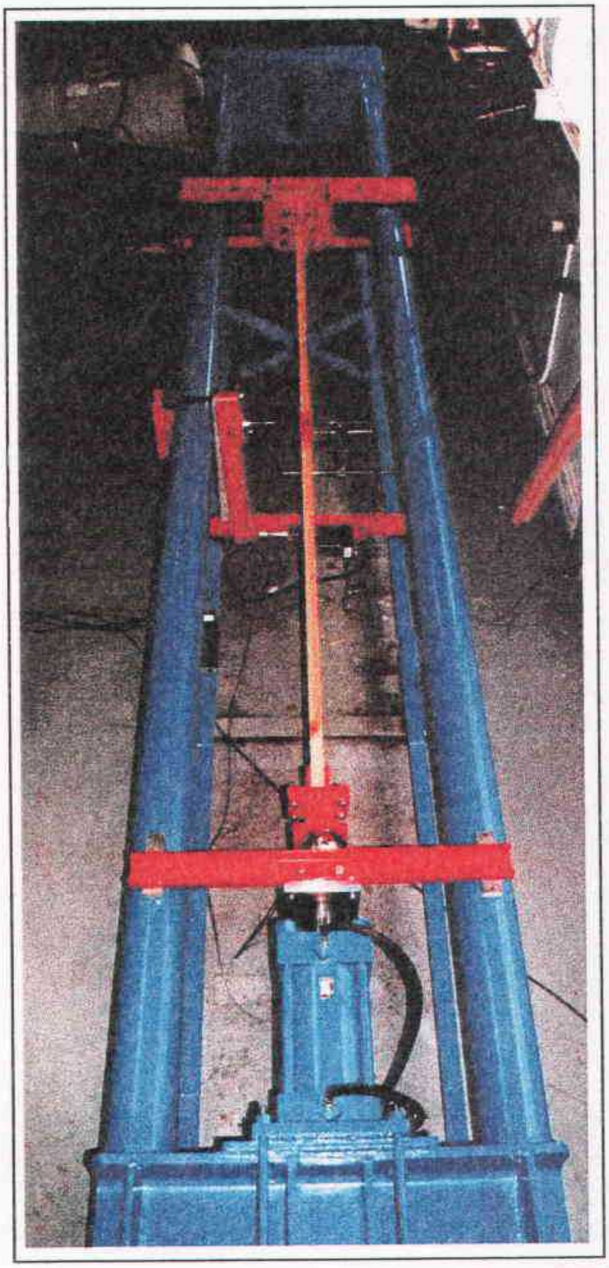


Figure 4.2: Test Machine Photograph

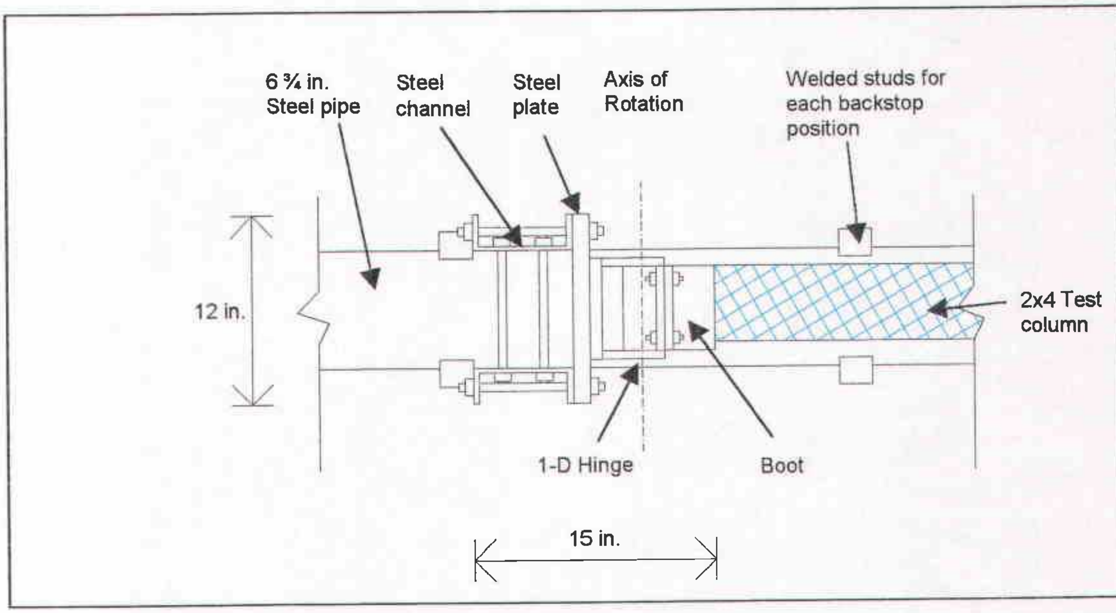


Figure 4.3: Section A-A: Movable Backstop

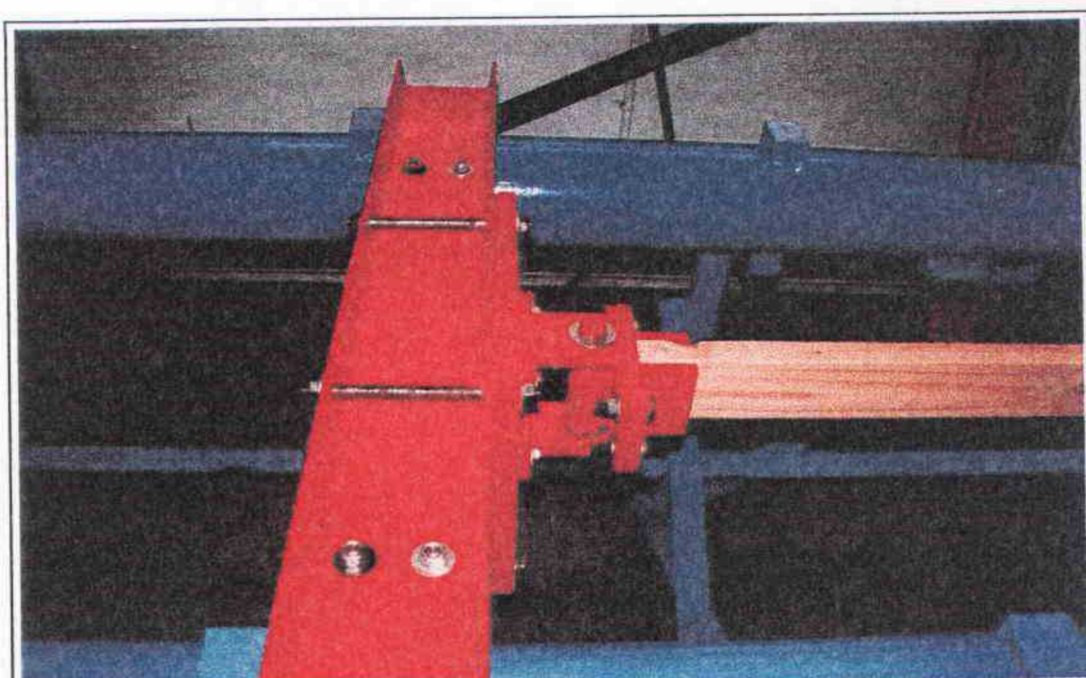


Figure 4.4: Movable Backstop Photograph

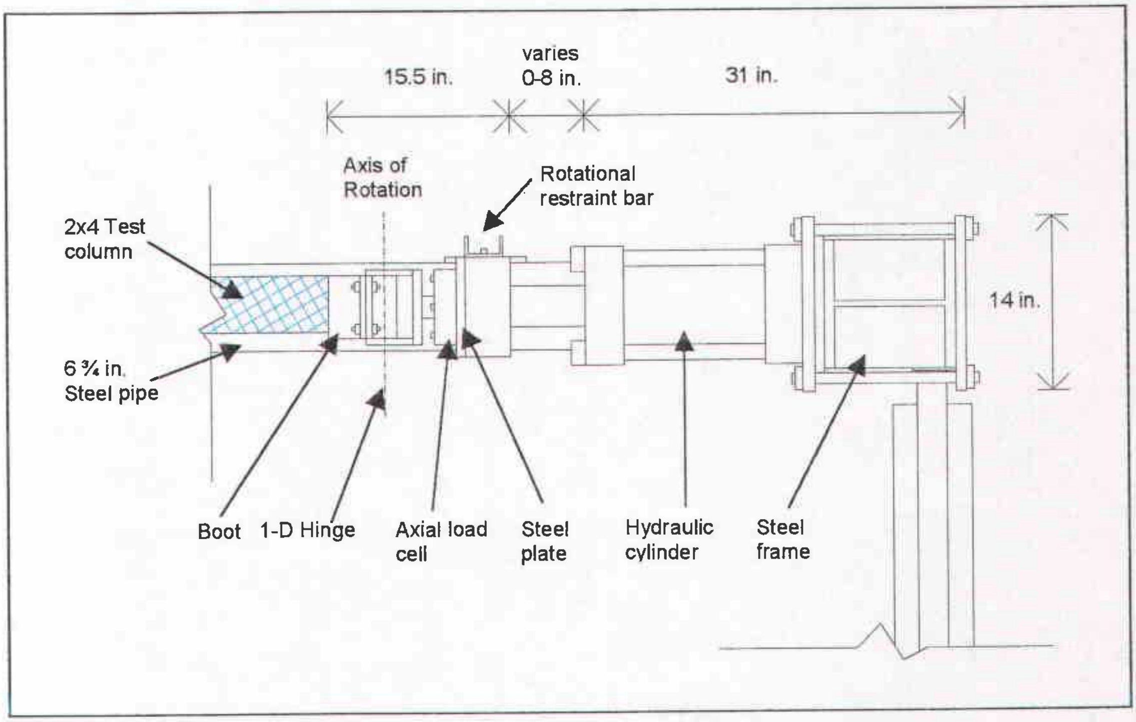


Figure 4.5: Section B-B: Column Load Application Equipment

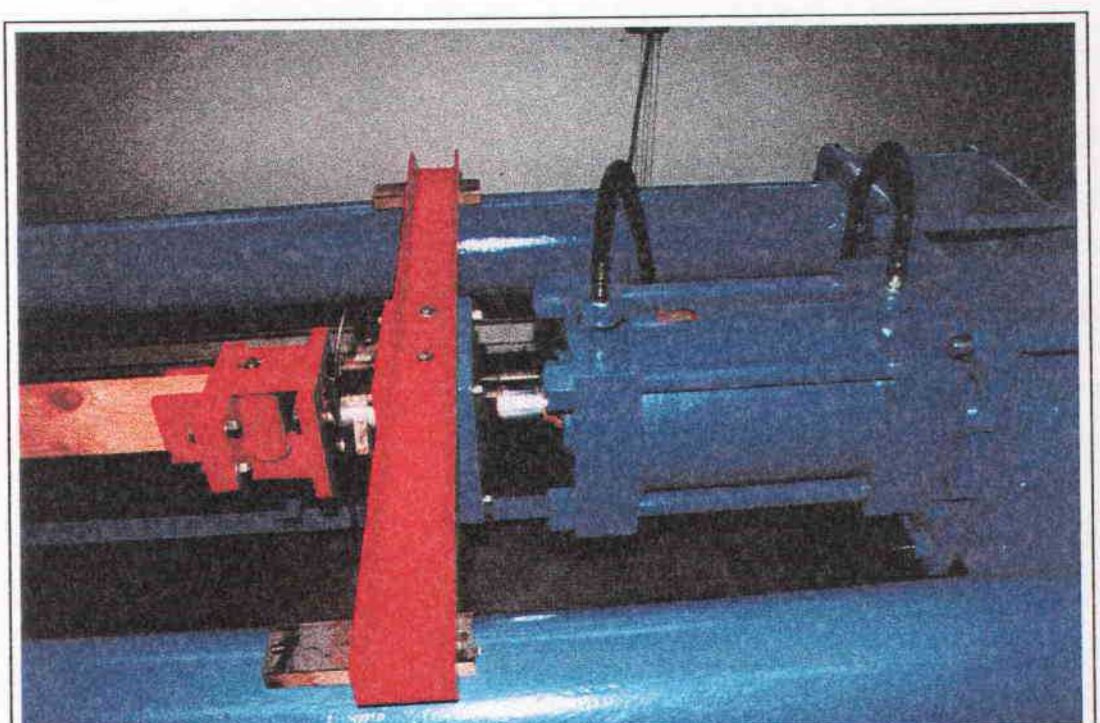


Figure 4.6: Column Load Application Equipment Photograph

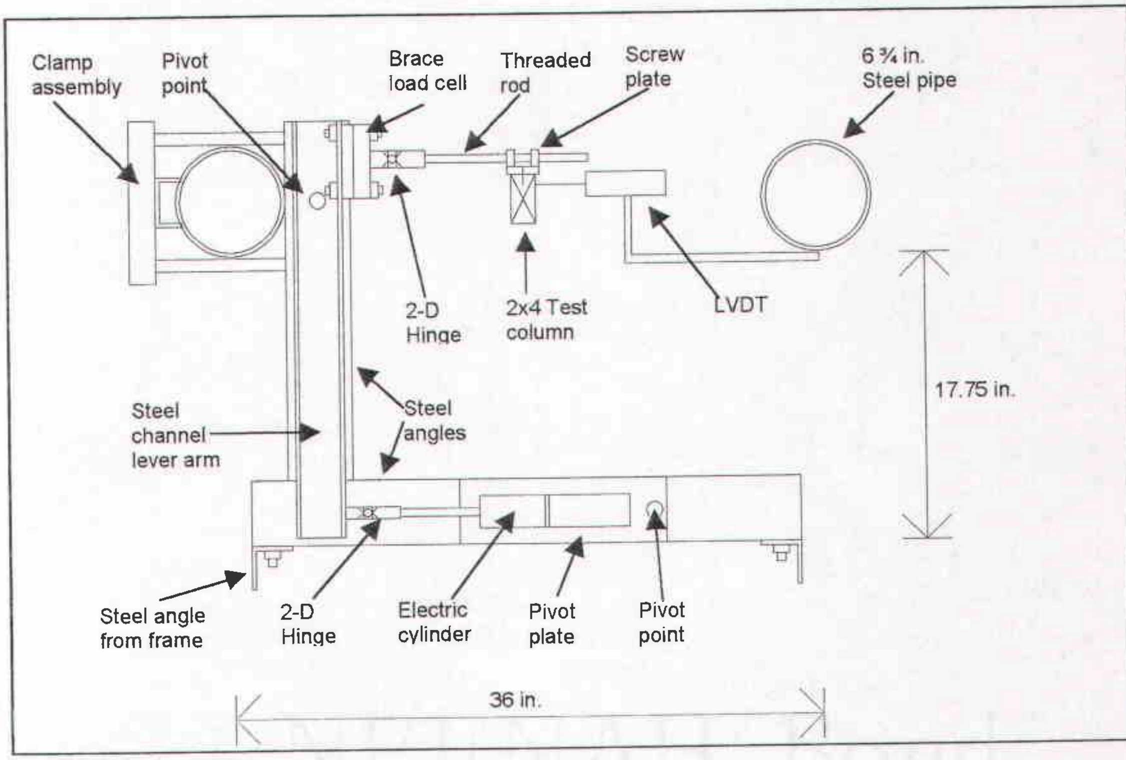


Figure 4.7: Section C-C: Brace Assembly

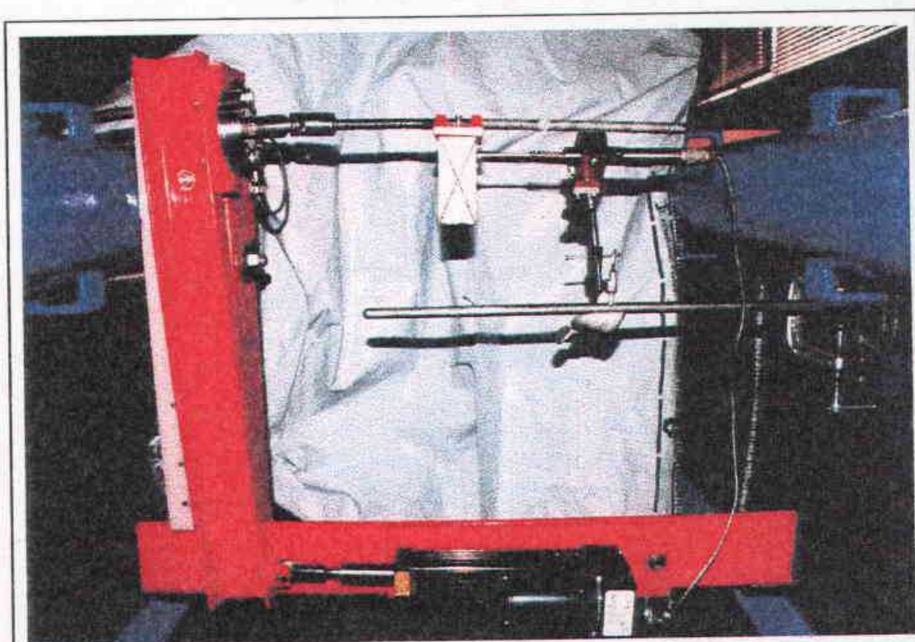


Figure 4.8: Brace Assembly Photograph

At each end of the column, the 2x4 test lumber sits in a "U" shaped steel "boot." Sheet metal shims ensure a tight fit between the 2x4 and the boot. As Figures 4.3 through 4.6 illustrate, the boot bolts to a steel hinge that allows the end of the column to rotate only about the weak axis. The hinge/boot assembly does not allow the end to twist, rotate about the strong axis, or laterally translate. This arrangement mimics the column boundary conditions of Figure 1.6. During the testing, the distance between the center of the hinges at the column ends was exactly 4, 6, 8, or 10 feet, as appropriate. A load cell placed between the hinge and the hydraulic cylinder monitors the axial load applied by the cylinder.

The lateral brace mechanism of Figures 4.7 and 4.8 discretely braces the column at mid-height. As explained in Section 2, the stiffness of the brace may impact both column performance and the brace load that develops. For that reason, we employed a mechanical brace assembly that allowed the brace stiffness to be varied. A threaded steel rod connects one edge of the 2x4 to a load cell mounted on a lever arm. An electric, screw-driven piston powers the lever arm and provides regulated lateral displacement for the column. A linear variable differential transformer (LVDT) measures that displacement at the surface of the 2x4.

4.4.2 Machine Operation and Control

A Windows NT 4.0 computer workstation operates the test machine. Using a data acquisition card developed by National Instruments, Inc., we wrote a custom computer program in LabVIEW® version 4.1 to control the machine and electronically acquire much of the test data. LabVIEW® is a graphical software programming language developed for laboratory applications (National Instruments, 1996).

The computer program controls all of the axial column loads by automatically acquiring the test load levels and cylinder movements input by the test operator. A portable, electric pump provides hydraulic pressure to power the axial load cylinder. The computer interfaces with a hydraulic controller card to operate a servo-valve and regulate cylinder extension. Using the axial load cell between the hinge and hydraulic cylinder, the computer monitors the current load level as it manipulates the servo-valve to achieve the test load.

The LabVIEW® program operates the brace assembly as an independent, closed-loop control system. The computer measures the brace load and lateral column deflection 10 times per second using a load cell and an LVDT. Based on the load cell reading, it calculates a target lateral displacement using a mathematical function input by the operator to describe the desired stiffness of the brace. After checking the current displacement against the target, the computer interfaces with a second controller card to manipulate the electric piston and regulate the lateral deflection. In other words, the machine controls the lateral column deflection based on measurements of the brace force. The software uses a "product-integral-derivative" control loop to equalize the actual and target displacements (Gaddy, 1997). In essence, all the user has to do is to input a mathematical function to describe the relationship between the brace force and displacement. For this research we used non-linear regression equations to describe the curves of Figures 3.5 to 3.8. The independent control system allows the mechanical brace assembly to mimic a mid-height brace spring with the programmed properties.

In addition to controlling the brace and applying axial load, the LabVIEW® program updates a spreadsheet file to record the acquired axial load, brace load, and brace deflection once every two seconds.

4.5 Test Procedure

4.5.1 Initial Column Measurements

Before testing each column, we measured selected column properties that may impact the relationship between the column and brace. All of the following measurements were made after the columns were trimmed to their test length and allowed to air-dry in the laboratory for about three weeks:

Cross-sectional dimensions: Using hand-held calipers, we determined the actual depth and thickness of each column cross section to the nearest 0.01 inches. Both measurements were taken at the column mid-height.

Moisture content: The moisture content of each column was measured to the nearest 0.1% using a Delhmhorst electrical-resistance moisture meter. The probe pins were driven to about ¼ the lumber thickness on the wide face at the mid-height of each sample column.

Modulus of Elasticity: We used a flatwise bending test to measure the longitudinal modulus of elasticity of each 2x4 to the nearest 10,0000 psi. Table 4.2 summarizes the E-test span and load parameters. Each test column was placed on roller supports and loaded at mid-span with about a 5 lb. concentrated preload. With a dial gauge accurate to the nearest 0.001 inch, the initial deflection was measured. The test load of Table 4.2 was then applied at mid-span and the final deflection measured with the dial gauge. D_{bend}, the difference between the initial and final deflections, represents the mid-span deflection under the test load. We calculated E as follows:

$$E = \frac{(P_{bend})(L_{bend})^3}{48(D_{bend})I} \tag{20}$$

where:
- D_{bend} = center span deflection for the flatwise bending E test (inches)
- E = longitudinal column modulus of elasticity (psi)
- I = weak axis moment of inertia (in^4)
- L_{bend} = test span for the flatwise bending E test (inches)
- P_{bend} = center span load for the flatwise bending E test (pounds)

We tested each sample column twice; once with the load applied to each flat surface. The reported E values represent the average of the two tests.

Table 4.2: Test Parameters for Modulus of Elasticity Estimations

Test Column Length (inches)	Actual 2x4 Length (inches)	E Test Parameters	
		Flatwise Bending Span (L_{bend}) (inches)	Center Point Load (P_{bend}) (pounds)
4	44	40	49.95
6	68	48	49.95
8	92	84	17.89
10	116	108	17.89

Weight: To obtain a rough measurement of wood density, the weight of each sample column was measured to the nearest 0.01 lbs. To allow for comparisons between groups, a "dry weight per foot" was calculated using the sample length and moisture content measurements.

Initial Column Deflection at Mid-height (Δ_0): The test apparatus included a projected laser line through the center of the hinges at each end of the test column. This laser provided a reference to approximate a straight centerline for the column weak-axis. After placing the 2x4 in the machine, closing the gaps between the 2x4 and the boots, and wedging the 2x4's into the boots with sheet metal shims, Δ_0 was measured to the nearest 1/16 inch. Δ_0 represents the distance between the center of the column thickness and the center of the laser line at the mid-height brace location.

Initial Shape: Before applying the mid height brace, we noted whether the maximum initial deflection occurred at mid-height and if the initial shape approximated an "S" or a "C" profile about the weak axis.

4.5.2 Column Load Selection

The next stage in the test process was to select an axial test load for each sample column. Table 4.3 summarizes the axial loads used for this research.

Table 4.3: Column Axial Test Loads

Column Length (feet)	Axial Test Load (pounds)	
	Select Structural Grade 2x4	Standard Grade 2x4
4	13,000	9,800
6	7,600	5,600
8	4,600	3,400
10	3,000	2,200

These test loads represent an estimate of the 5% exclusion limit on column strength for each length and grade of sample column. To approximate the test conditions, these values assume that the column experiences a short-term axial load for approximately 10 minutes. They also assume that the 2x4 Douglas-fir columns are sufficiently braced to provide an effective column length of one-half the web length, L_w. In essence, these test loads represent the column strength assumed by the truss designer for a braced compression web member with all safety and load duration factors removed. For the purpose of this research, this strength is the best estimate of the critical column strength, P_{cr}, anticipated by the truss designer.

Calculation of these test loads generally follows the normal allowable stress column design procedure summarized in Section 2.1.4 of this report (American Forest and Paper Association, 1997). Appendix B provides a sample calculation. As the sole exception to the normal design process, we removed the assumed safety factors on F_{cE} and F^*_c before using Equation 3. Section 2.1.4 summarizes these factors. Removal of the safety factors converts Equation 3 into Equation 2 and provides an estimate of the 5% exclusion limit on ultimate column strength.

4.5.3 Brace Stiffness Selection

With the axial test load for each column selected, Equation 9 can be used to directly calculate the brace force estimate for each 2x4 using the 2% rule-of-thumb (Section 2.2.3). To estimate the brace force using the methods developed by Winter (Section 2.2.4), Plaut (Section 2.2.5), and Tsien (Section 2.3.3) requires knowledge about the properties of the brace. Hence, the brace stiffness must be selected for each sample prior to the test.

The brace design methods developed by Plaut and Winter assume that the brace behaves in a linear elastic fashion. If true, then Equations 10 and 14 could be used to estimate the required brace stiffness for each test column using assumed values for Δ and measured values for Δ_0. If K_{req} was supplied as the brace stiffness for the test, then Equations 11 and 15 provide an estimate of the required brace strength.

Unfortunately, three complications limit the applicability of this brace selection approach. First, as discussed in Section 3, a lateral/diagonal bracing system provides non-linear elastic support for the column. Second, the support stiffness supplied by a lateral/diagonal bracing system is deterministic. Traditional brace configurations do not provide enough flexibility for the designer to calculate and then supply a brace with a stiffness of exactly K_{req}. In most cases, the practical brace stiffness modifications available to the designer consist of alteration of the diagonal brace spacing or the number of nails in each connection. Section 3 provides, in the form of load/deflection curves, conservative estimates of the brace support stiffness supplied by a lateral/diagonal brace system with variable spacing of the diagonal braces. Finally, what value should be assumed for Δ in the calculation of K_{req}? No guidance exists for light-frame wood trusses.

To circumvent these problems, we employed a modified brace stiffness selection procedure for this testing. Equations 11, 15, and 18 can be used to describe a theoretical relationship between F_{br} and Δ for the brace analysis methods developed by Winter, Plaut, and Tsien, respectively. Hence, a theoretical F_{br}-Δ function, or "theoretical brace analysis curve" can be defined for each test column and brace theory. This function can be compared to the finite-element derived "brace support curves" from Section 3. By superimposing the theoretical brace analysis curves on the brace support curves, we can select a reasonable brace stiffness for test purposes.

Figure 4.9 graphically illustrates this concept for a 10 foot test column with Δ_0 of 0.5 inches, P of 2,200 lbs., E of 1,190,000 psi, and a weak axis I of 0.911 in^4. The curved black lines in the graph represent the brace support curves from Figure 3.8 for a 10 foot compression web member. Each curve estimates the variable support stiffness that a lateral/diagonal brace system provides for this column with a diagonal brace spacing of between 1 and 10 compression webs per diagonal. The closer the diagonal brace spacing, the stiffer the support. The colored lines in Figure 4.9 describe the theoretical brace analysis relationships between F_{br} and Δ for this column using the methods developed by Winter, Plaut, and Tsien. For reference, Figure 4.9 also illustrates the brace force predicted by the 2% rule.

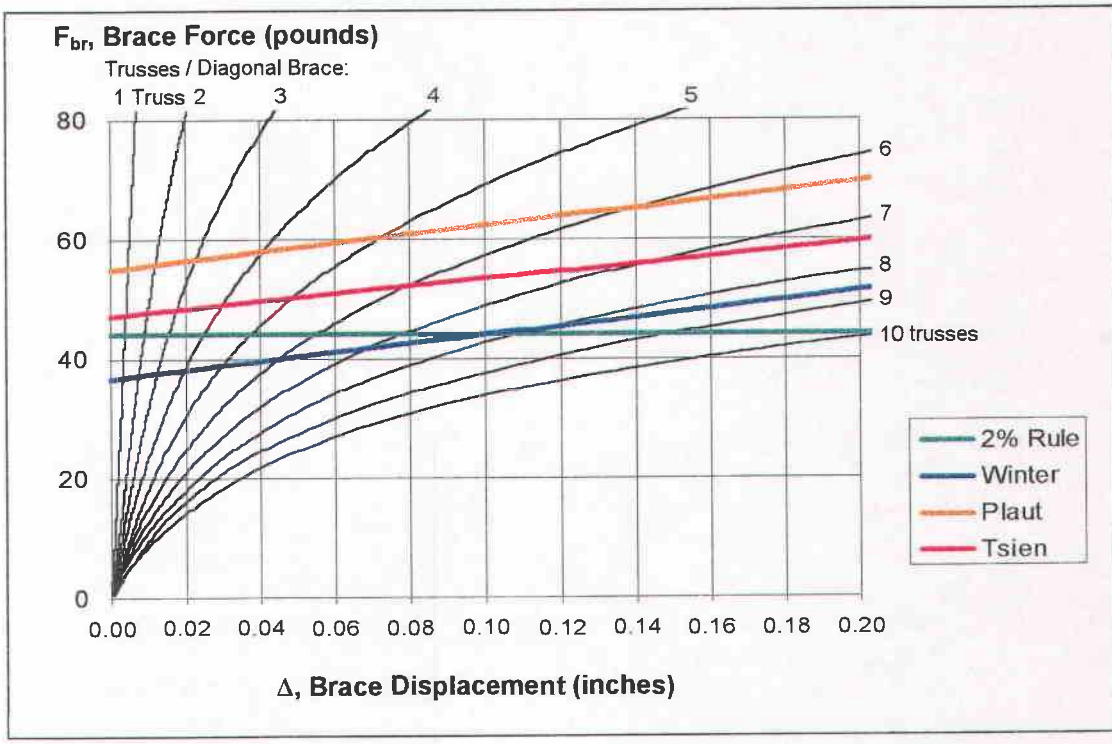


Figure 4.9: Sample Brace Stiffness Selection

Any brace support curve that intersects with a colored theoretical brace analysis curve should provide a sufficient brace according to that theory. With the 2% rule, this comparison incorporates a check of brace strength and not stiffness. For example, Figure 4.9 suggests that the diagonal braces could be spaced up to 10 trusses apart and provide sufficient strength to support this particular test column. For the brace analysis methods developed by Plaut, Winter, and Tsien, this comparison includes a check of both the brace strength and stiffness. For a given Δ, the corresponding point on each of these three theoretical brace analysis curves provides an estimate of the required brace strength for that theory and Δ. The required brace stiffness is the slope of a straight line between that point and the origin. Any brace support curve that intersects with a theoretical brace analysis curve should possess sufficient strength and stiffness that meet that theory's requirements for the Δ of the intersection. Brace support curves that do not intersect with the theoretical brace analysis curve do not meet the theoretical brace requirements. For example, Figure 4.9 suggests that the diagonal braces should not be placed more than 6 webs apart to meet the strength and stiffness requirements of Plaut's method.

Using this graphical procedure, we selected a non-linear brace support curve from Section 3 for every test column on an individual basis. Figures 3.5, 3.6, 3.7, and 3.8 defined the available brace support curve alternatives for test columns of 4, 6, 8, and 10 foot lengths, respectively. Plaut's brace theory, the most conservative of the analysis methods that consider brace stiffness, provided the basis for this selection. We always chose the most flexible brace support curve that intersected with, or came very close to intersecting with, the theoretical brace analysis curve for Plaut's method. Appendix D provides a detailed example of this brace support curve selection procedure for a sample column. Using a regression function to describe the selected brace support curve, we input the curve into the computer to define the properties of the mechanical brace for each test.

4.5.4 Testing

At this stage in a typical test, all of the column properties have been measured, the test load identified for the grade and length combination, and a brace support curve chosen. As the next step, we attached the mechanical brace assembly to the column mid-length using the following procedure. First, as indicated in Figure 4.7, the LVDT was positioned about one inch below the top surface of the column and the screw-plate of the brace loosely placed at the column centerline. Second, the LVDT and brace load cell were zeroed. Third, we fastened the mechanical brace screw-plate to the top edge of the 2x4 using two 2.5 inch long, No. 10 wood screws in pre-drilled pilot holes. After attachment, we input the brace support curve selected for that 2x4 test into the computer workstation. Once we activated the independent control loop that operates the brace, the brace assembly mimicked a non-linear spring with the programmed properties.

Through the control of the LabVIEW® program, we applied a 100 pound axial pre-load load to remove any remaining slack from the system. We always placed the initial column bow away from the mechanical brace assembly, so applying this load induced a small amount of tension on the brace. After turning on the data acquisition subroutine of the software, the operator dialed in the desired test load for that 2x4 length and grade. The computer always applied the axial test load in 200 even load increments, with one increment added every second. In other words, regardless of the test load, it took 200 seconds to fully load the column. As the hydraulic cylinder applied the axial load, the control software for the brace mechanism monitored the brace loads and manipulated the lateral deflections to follow the desired brace support curve. The data acquisition software recorded the column load, brace load, and brace deflection to a spreadsheet file once every two seconds.

Once the test apparatus achieved the full axial test load, it maintained that load until directed otherwise. In most cases, the brace assembly reached an equilibrium position within a minute. When this occurred, the F_{br} and Δ for that position were recorded to the nearest pound and 0.001 inch, respectively.

As will be discussed in Section 5, in some cases the column failed before the full axial test load could be applied. For these tests, the brace load and deflection at failure were recorded. In other instances, the brace failed before the axial test load could be applied. With these "failures," the deflection of the brace assembly exceeded the limit of the brace support curve selected for the test. This signified that at least one of the nailed connections within the brace assembly exceeded 0.1 inches of slip. In addition to noting this failure, we recorded the axial load, brace load, and brace deflection just prior to the failure.

As will be discussed in Section 5, in a few cases the column reached the desired axial test load but the brace did not find an equilibrium position. While thetest apparatus maintained the axial load, the brace slowly accumulated more load and deflection. Eventually, the brace failed by exceeding the Δ limit of the brace support curve. We recorded the column load, brace load, and brace deflection just prior to brace failure for these columns. In addition, it was noted that the failure resulted from an inability of the brace to achieve an equilibrium position.

In general, all of the columns of a single length were tested together. This reduced movement of the movable bridge and ensured better quality control. The testing proceeded by length from the 10 to the 4 foot long material. The grades were typically mixed together within each length class.

5.0 RESULTS

Tables 5.1 and 5.2 summarize, by length and grade, the measurements and observations for all the column tests incorporated into the final analysis of this thesis. Appendix E contains a detailed listing of the results for every test.

5.1 Final Sample Sizes

We originally prepared 872 columns for testing. Data was not collected for 13 columns due to a combination of operator error, equipment malfunction, and electrical power failure. We recorded, but did not use in the final analysis, the data for 85 additional columns. This second group represents the "Standard" grade lumber produced by DCFP that was judged much better than "Standard" quality during testing. Most of these 2x4's consisted of clear, dense material. Preliminary inspection of the results revealed that inclusion of these 2x4's in the final analysis as "Standard" grade created statistically significant grade effects on the performance of the theoretical brace analysis methods evaluated by this research. Possible explanations for this effect might include low values of initial lateral deflection and failure to test this high-grade material at an axial load closer to its 5% exclusion limit on compressive strength. Regardless of the cause, removal of this questionable lumber from the data set eliminated these effects. Since this material was obviously not "Standard" grade, these grade effects would have been both artificial and misleading. Dropping the mis-graded material from the study resulted in a final sample size of 774 columns.

5.2 Initial Column Measurements

Table 5.1 summarizes the column measurements taken prior to each brace test. By inspection, little practical difference exists between the test groups with respect to their member dimensions, weak-axis moment of inertia, moisture contents,

Table 5.1: Summary of Initial Column Measurements by Length and Grade

Criteria	Grade	4 foot	6 foot	8 foot	10 foot
Total replications (DCFP-FLC-MFP)	Select Structural	105 (28 – 41 – 36)	109 (39 – 31 – 39)	107 (39 – 36 – 32)	107 (33 – 39 – 35)
	Standard	83 (11 – 28 – 44)	83 (12 – 41 – 30)	84 (20 – 32 – 32)	96 (18 – 43 – 35)
b average, inches (min. to max, stdev)	Select Structural	1.48 (1.43 to 1.50, 0.01)	1.48 (1.44 to 1.50, 0.01)	1.48 (1.42 to 1.51, 0.02)	1.48 (1.42 to 1.50, 0.02)
	Standard	1.48 (1.38 to 1.51, 0.02)	1.49 (1.45 to 1.51, 0.01)	1.48 (1.42 to 1.51, 0.02)	1.49 (1.45 to 1.52, 0.01)
d average, inches (min. to max, stdev)	Select Structural	3.46 (3.37 to 3.51, 0.03)	3.46 (3.32 to 3.56, 0.04)	3.46 (3.35 to 3.51, 0.03)	3.47 (3.37 to 3.58, 0.03)
	Standard	3.46 (3.36 to 3.54, 0.04)	3.47 (3.35 to 3.59, 0.04)	3.48 (3.37 to 3.57, 0.04)	3.47 (3.38 to 3.57, 0.04)
I average, in^4 (min. to max, stdev)	Select Structural	0.937 (0.842 to 0.980, 0.032)	0.930 (0.843 to 0.993, 0.034)	0.934 (0.808 to 0.999, 0.040)	0.940 (0.821 to 0.993, 0.034)
	Standard	0.935 (0.762 to 1.004, 0.04)	0.950 (0.862 to 1.000, 0.029)	0.948 (0.827 to 1.014, 0.04)	0.952 (0.868 to 1.026, 0.035)
E average, 10^6 psi (min. to max, stdev)	Select Structural	1.63 (0.77 to 2.85, 0.37)	1.81 (1.08 to 2.74, 0.39)	1.91 (1.18 to 2.94, 0.38)	1.97 (1.18 to 2.91, 0.40)
	Standard	1.45 (0.80 to 2.45, 0.34)	1.60 (0.79 to 2.49, 0.40)	1.70 (1.05 to 2.62, 0.38)	1.65 (0.93 to 2.90, 0.39)
MC average, % (min. to max, stdev)	Select Structural	11.5 (9.4 to 14.3, 1.2)	11.7 (9.3 to 14.0, 1.1)	12.3 (9.3 to 15.7, 1.4)	12.9 (9.5 to 16.2, 1.7)
	Standard	11.1 (8.7 to 16.1, 1.3)	10.9 (8.9 to 13.7, 1.1)	11.7 (9.5 to 16.4, 1.3)	12.1 (8.4 to 16.1, 1.7)
Dry wt. average, lbs/ft (min. to max, stdev)	Select Structural	0.996 (0.737 to 1.316, 0.124)	0.997 (0.822 to 1.253, 0.103)	1.006 (0.794 to 1.400, 0.112)	1.022 (0.726 to 1.263, 0.116)
	Standard	0.988 (0.791 to 1.327, 0.115)	1.010 (0.788 to 1.541, 0.121)	1.001 (0.794 to 1.367, 0.108)	1.001 (0.782 to 1.334, 0.106)
Δ_0 average, inches (min. to max, stdev)	Select Structural	0.081 (0.016 to 0.250, 0.049)	0.129 (0.031 to 0.438, 0.071)	0.182 (0.063 to 0.531, 0.103)	0.244 (0.063 to 1.438, 0.175)
	Standard	0.087 (0.016 to 0.250, 0.048)	0.139 (0.031 to 0.438, 0.085)	0.201 (0.031 to 0.531, 0.107)	0.276 (0.063 to 1.188, 0.202)
Initial shape, %"S" / "C" Shaped	Select Structural	15 / 85	13 / 87	6 / 94	13 / 87
	Standard	10 / 90	7 / 93	10 / 90	11 / 89
Maximum initial offset at center, %"yes" / "no"	Select Structural	75 / 25	70 / 30	83 / 17	79 / 21
	Standard	81 / 19	65 / 35	64 / 36	75 / 25

Table 5.2: Summary of Test Results by Length and Grade

Criteria	Grade	4 foot	6 foot	8 foot	10 foot
Failures during test (column–brace– both)	Select Structural	6 (1 – 5 – 0)	7 (1 – 4 – 2)	0 (0 – 0 – 0)	1 (1 – 0 – 0)
	Standard	2 (1 – 1 – 0)	2 (1 – 1 – 0)	3 (1 – 2 – 0)	0 (0 – 0 – 0)
Brace support curve, 1-2-3-4-5-6-7-8-9-10	Select Structural	52-48-5-0-0-0-0-0-0-0	2-15-38-29-21-1-3-0-0-0	0-6-11-11-20-5-31-7-2-14	1-0-3-11-4-11-11-8-25-33
	Standard	22-39-19-3-0-0-0-0-0-0	1-6-13-6-23-17-13-0-3-1	0-0-5-9-6-12-13-6-2-31	0-2-2-1-5-6-7-11-12-50
F_{br} average, pounds (min. to max, stdev)	Select Structural	123 (2 to 499, 96)	72 (0 to 335, 53)	39 (2 to 145, 29)	27 (1 to 175, 25)
	Standard	85 (1 to 481, 68)	52 (1 to 282, 42)	30 (1 to 90, 21)	22 (0 to 105, 18)
Δ_{actual} average, inches (min. to max, stdev)	Select Structural	0.021 (0.000 to 0.161,0.033)	0.029 (0.000 to 0.175, 0.033)	0.026 (0.001 to 0.086, 0.019)	0.027 (0.001 to 0.124, 0.022)
	Standard	0.015 (0.000 to 0.170,0.023)	0.024 (0.001 to 0.163, 0.021)	0.027 (0.000 to 0.141, 0.022)	0.026 (0.000 to 0.078, 0.017)

and dry weights per unit length. In addition, all of the groups contain roughly the same mix of initial column profiles. The most dominant profile was a "C-shaped" column with maximum initial lateral deflection at about the column mid-height, not unlike the conceptual column depicted in Figure 2.3.

A review of Table 5.1 suggests that the mean flatwise modulus of elasticity, E, varies between some length and grade groups of the experimental design. For each column length, the average E of the "Select Structural" lumber exceeds that of the "Standard" grade material and the difference is statistically significant (one-sided p-values less than 0.001 from two-sample t-tests). This observation agrees with industry standards (American Forest and Paper Association, 1997).

Within each grade, the average E for the 4 foot columns was significantly less than that for any other length (one-sided p-values less than 0.005 from two-sample t-tests). In addition, there was a significant mean E difference between the 6 and 10 foot "Select Structural" columns (one-sided p-values less than 0.002 from a two-sample t-test). We did not anticipate a "length effect" on E, especially since we randomized the distribution of samples between length groups. In addition, in all cases, the E tests used flatwise bending test spans with negligible shear influences. However, in retrospect, Madsen (1992) reports a similar length effect. Since the cause of the length effect on E is unknown, some of its variation should be considered in the study results.

Table 5.1 also suggests some mean differences in the initial lateral deflection at mid-height, Δ_0, between the length and grade groups. However, the raw data can be misleading. For example, a Δ_0 of 0.5 inches represents more severe lateral deflection for a 4 foot column than for a 10 foot column. We normalized the data for column length by dividing Δ_0 by L and found no evidence to suggest a statistically significant mean Δ_0/L difference between grades (two-sided probabilities greater than 0.1 from chi-square approximations of Kruskal-Wallis tests). After pooling the data from both grades, no significant differences in mean Δ_0/L were found with column length (two-sided probability greater than 0.1 from a chi-square approximation of a Kruskal-Wallis test). This implies that the magnitude of Δ_0 relative to column length was consistent for all length and grade groups of the experimental design.

5.3 Column and Brace Failures

Twenty-one test samples experienced a column and/or brace failure during the test. All 8 "column" failures occurred when the column asymmetrically buckled. Figure 5.1 illustrates one such failure. The 15 "brace" failures represent instances where the measured F_{br} and Δ exceed the limit of the brace support curve selected for the test. As discussed in Section 3.5.3, this means that the finite element model used to derive the brace support curve predicts that at least one nailed joint in the lateral/diagonal brace assembly exceeds its practical limit on working joint displacement.

Figure 5.2 illustrates a typical relationship between the measured column and brace forces for a 2x4 that experienced a brace failure. Figure 5.3 depicts a typical relationship for a column that did not experience a brace failure.

The column test of Figure 5.2 used the "7 Truss" curve of Figure 3.6 to define the properties of the brace. This brace support curve approximates the lateral support provided to a 6 foot web member when the lateral/diagonal bracing system has seven webs per diagonal brace. Comparison of the brace load levels for the "7 truss" curve in Figure 3.6 with Figure 5.2 reveals that the proportional limit of both curves roughly coincide at a brace force of about 30 pounds. Similar comparison of Figure 5.3 with the brace support curve used for that test suggests that both of these relationships remain at or below their proportional limit. We conducted a similar comparison between the brace support curves and P vs. F_{br} curves for all the tests that experienced a brace failure and about eighty tests that did not. This inspection revealed that the initiation of non-linear behavior between the column and brace force always occurs at about the point where the stiffness of the brace becomes non-linear. Due to time constraints and the consistency of the findings, we did not conduct a similar comparison for all 774 tests.

We noted that the measured brace load and deflection for some columns did not reach a stable equilibrium at the maximum axial test load. With 8 columns, all recorded as brace failures, the brace load and deflection increased despite a constant axial test load. Left alone, F_{br} and Δ grew until they exceeded the limits of the brace support curve. For all 8 columns, the supporting lateral brace was well into the non-linear portion of the brace support curve when the applied axial test load reached the

Figure 5.1: Asymmetrical Buckling of a 10 foot Test Column

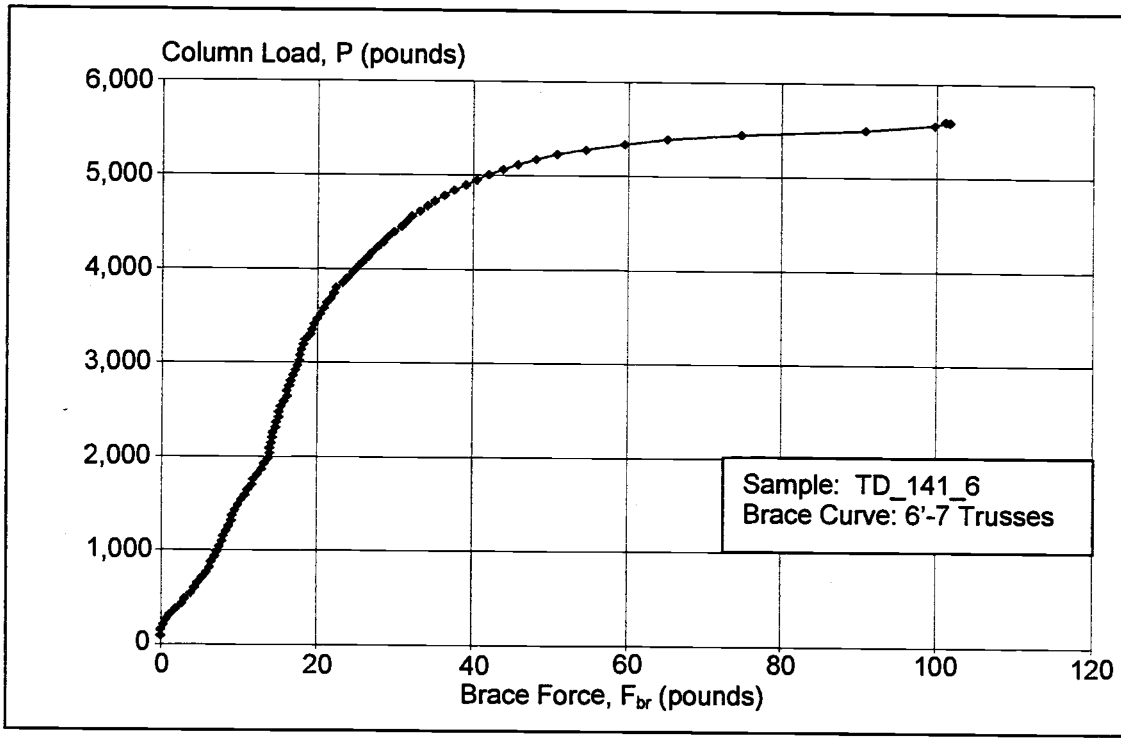


Figure 5.2: Column Load vs. Brace Load for a Column Test with a Brace Failure

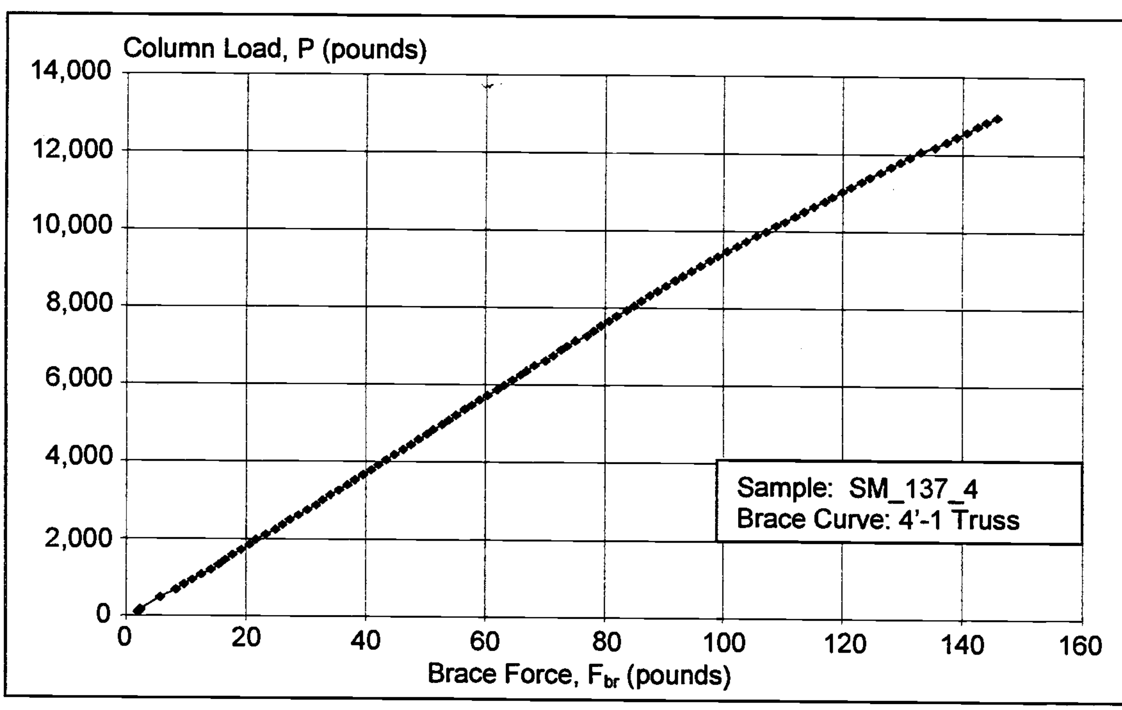


Figure 5.3: Column Load vs. Brace Load for a Column Test without a Brace Failure

maximum level. This suggests that brace "instability" may occur with when the proportional limit of the brace is exceeded.

We attempted to determine if the column or brace failures could be traced back to a common cause. In every case, the "column" failures occurred with 2x4's having an above average Δ_0 and a below average E. However, the "column" failures did not occur for 2x4's with the extreme values of either parameter. No common characteristic was observed for the columns that experienced a "brace" failure.

5.4 Performance Variables

The primary objective of this research is evaluate the ability of several existing, theoretical brace analysis models to estimate the strength and stiffness requirements of a discrete compression web brace. To this end, we compared the lateral support requirements for strength and stiffness observed during the column testing with those estimated by four different analysis models: Plaut's method (Section 2.2.5), Winter's method (Section 2.2.4), Tsien's equation (Section 2.3.3), and the 2% Rule (Section 2.2.3). To simplify comparisons between experimental groups and theories, we calculated the relative deviation between the estimated and experimental brace forces for each column test at the maximum axial test load. This relative deviation is used as a "performance variable" to characterize the prediction performance of each brace theory.

5.4.1 Basis

The performance variables used for this study, D_{theory}, are defined as:

$$D_{theory} = \frac{(F_{br})_{actual} - (F_{br})_{est.}}{(F_{br})_{est.}} \tag{21}$$

where: D_{theory} = performance variable where *theory* = Plaut, Winter, Tsien, or 2% Rule (unitless)

$(F_{br})_{actual}$ = brace force measured for a given test column at the maximum axial test load P (pounds)

$(F_{br})_{est.}$ = brace force estimated by a given brace theory at the observed maximum lateral deflection, Δ_{actual} (pounds)

D_{theory} may range between negative one and positive infinity. D_{theory} less than zero signifies that the brace analysis theory provided a "conservative" overestimate of the lateral support requirements. D_{theory} greater than zero indicates that the experimental measurements exceeded the estimate and the theory was not conservative. For each of the 774 test columns, we computed D_{theory} using the measured data and estimated brace forces from Plaut's method, Winter's method, Tsien's equation, and the 2% Rule.

Calculation of D_{theory} requires, for each test, a common basis of definition for the estimated and measured brace forces. Figure 5.4 illustrates the basis used for this research. As discussed in Section 4.5.3 and Appendix D, a brace support curve developed in Section 3 defines the mid-height brace properties for each test. Using the actual values of P, L, Δ_0, E, and I for each test column, estimated relationships between F_{br} and Δ can be computed for each brace analysis theory using the following:

- Equation 15 for Plaut's method,
- Equation 11 for Winter's method,
- Equation 18 for Tsien's equation, and
- Equation 9 for the 2% Rule.

Figure 5.4 conceptually illustrates these relationships for a test column as "theoretical brace analysis" curves.

One approach to calculate D_{theory} is to use the points of intersection between the brace support curve and the theoretical brace analysis curves, such as point B in Figure 5.4, to define the estimated F_{br} for each brace theory. However, for many test columns, no intersection point exists for one or more brace theories. As depicted in Figure 5.4, this was frequently the case for the 2% Rule. To avoid this problem, we chose an alternative approach and defined D_{theory} based on the estimated and measured brace force at the maximum observed test deflection, Δ_{actual}. This

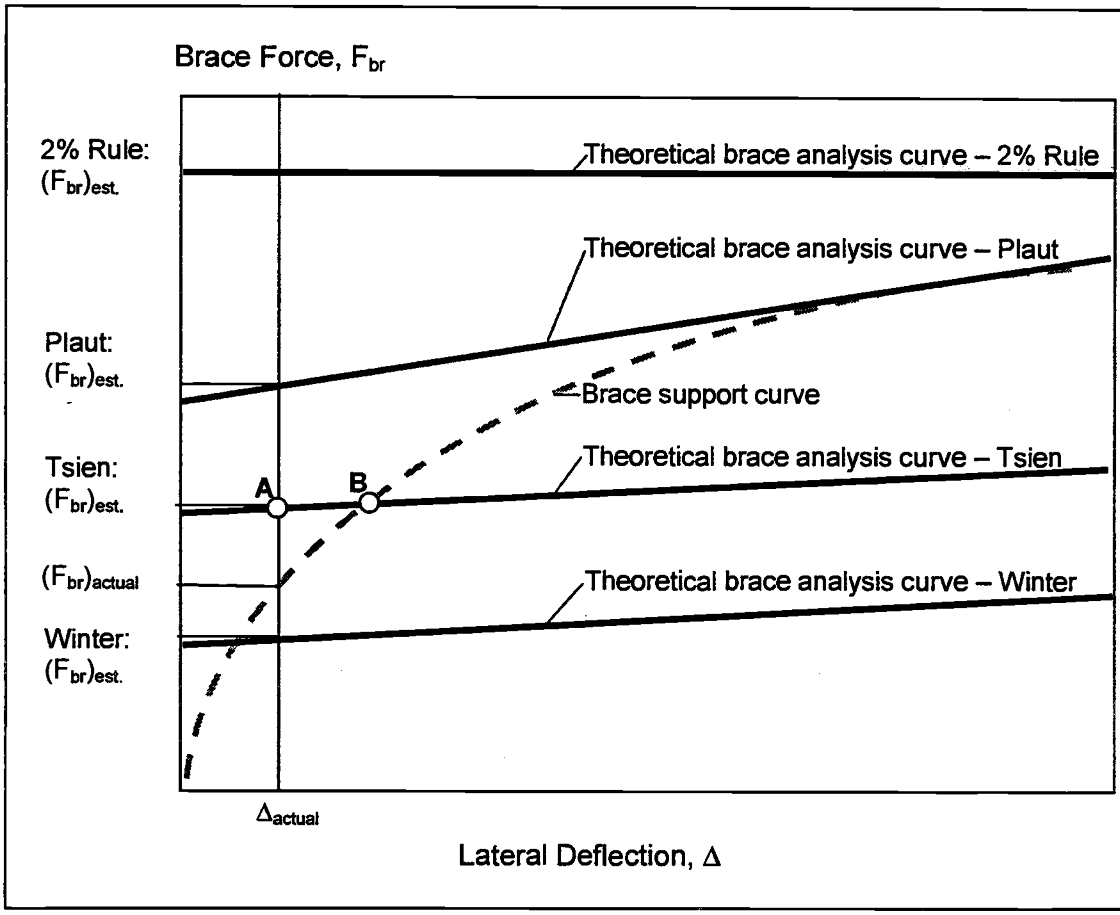


Figure 5.4: Basis for Comparison of Actual vs. Predicted Brace Requirements

comparison can be summarized for each theory as follows: for Δ_{actual} observed during the test, the theory estimated that $(F_{br})_{est.}$ would be required to support the column when $(F_{br})_{actual}$ was necessary. Using this basis, Point A in Figure 5.4 provides the brace force estimated by Tsien's equation for the conceptual column.

It is important to note that, with the exception of the 2% Rule, this comparison basis provides a simultaneous check of both the brace strength and stiffness requirements of each brace analysis theory. The theoretical brace analysis curves of Figure 5.4 implicitly include both support criteria. For example, the intersection of the vertical line of Δ_{actual} and a brace theory curve provides an estimate of the F_{br} required to maintain that deflection. The stiffness required according to that theory will be $(F_{br})_{est.}/\Delta_{actual}$: the slope of a line between the origin and the intersection. If $(F_{br})_{est.}$ exceeds $(F_{br})_{actual}$, then the theory overestimates both the strength and stiffness requirements of that column. If $(F_{br})_{est.}$ is less than $(F_{br})_{actual}$, then it underestimates both requirements. This dual comparison does not apply for the 2% Rule since $(F_{br})_{est.}$ for that rule is not a function of Δ; only the actual and predicted brace forces at the maximum axial test load are compared.

5.4.2 Performance Variable Comparisons with Length and Grade

Table 5.3 summarizes the performance variable results for each brace analysis theory by length and grade of test column. We conducted a statistical comparison of D_{theory} means between the experimental groups to check for length and grade effects.

Table 5.3: Summary of Performance Variable D_{theory} by Length and Grade

Criteria	4 foot	6 foot	8 foot	10 foot
Select Structural				
D_{plaut} average (min. to max, stdev)	-0.22 (-0.96 to 0.56, 0.34)	-0.27 (-1.00 to 0.21, 0.27)	-0.37 (-0.95 to 0.14, 0.23)	-0.39 (-0.95 to 0.24, 0.25)
D_{winter} average (min. to max, stdev)	0.08 (-0.94 to 1.06, 0.44)	0.01 (-1.00 to 0.76, 0.34)	-0.10 (-0.92 to 0.57, 0.31)	-0.12 (-0.92 to 0.71, 0.34)
D_{tsien} average (min. to max, stdev)	-0.04 (-0.95 to 0.99, 0.44)	-0.13 (-1.00 to 0.54, 0.33)	-0.24 (-0.93 to 0.34, 0.28)	-0.27 (-0.94 to 0.47, 0.29)
$D_{2\%}$ average (min. to max, stdev)	-0.52 (-0.99 to 0.92, 0.38)	-0.53 (-1.00 to 1.34, 0.36)	-0.58 (-0.98 to 0.58, 0.31)	-0.56 (-0.98 to 1.92, 0.41)
Standard				
D_{plaut} average (min. to max, stdev)	-0.31 (-0.99 to 0.47, 0.30)	-0.32 (-0.93 to 0.36, 0.22)	-0.37 (-0.93 to 1.47, 0.31)	-0.36 (-1.00 to 0.48, 0.22)
D_{winter} average (min. to max, stdev)	-0.03 (-0.98 to 0.78, 0.39)	-0.05 (-0.90 to 0.99, 0.29)	-0.10 (-0.89 to 2.46, 0.43)	-0.07 (-1.00 to 0.99, 0.31)
D_{tsien} average (min. to max, stdev)	-0.12 (-0.98 to 1.19, 0.42)	-0.16 (-0.92 to 1.11, 0.30)	-0.23 (-0.91 to 1.97, 0.38)	-0.23 (-1.00 to 0.79, 0.27)
$D_{2\%}$ average (min. to max, stdev)	-0.56 (-1.00 to 1.45, 0.35)	-0.54 (-0.99 to 1.57, 0.38)	-0.55 (-0.99 to 0.46, 0.31)	-0.49 (-1.00 to 1.39, 0.43)

Non-parametric statistical methods were selected for all D_{theory} comparisons for three reasons. First, with a fixed lower limit of negative one, all of the D_{theory} distributions in Table 5.3 were positively skewed to varying degrees. Parametric comparison methods based on a normal approximation may be reasonable for D_{plaut}, D_{winter}, and D_{tsien} (Shapiro-Wilk statistics between 0.85 and 0.98). However, the $D_{2\%}$ distributions were notably skewed (Shapiro-Wilk statistics between 0.78 and 0.91).

Non-parametric techniques provide a general comparison basis that applies to all of the distributions. Second, nearly all of the distributions in Table 5.3 contain extreme positive outliers that could not be discredited. Non-parametric statistical methods are more resistant to the influence of outliers (Ramsey and Schafer, 1997). Finally, the lateral brace did not reach a stable equilibrium for the columns that experienced a "column" or "brace" failure. In these instances, the exact magnitude of an "equilibrium" result might differ slightly from what was recorded. Deleting these observations, which all experienced non-conservative results, would unfairly bias the resulting D_{theory} distributions. Due to the rank-ordering procedures incorporated within the non-parametric techniques, the exact magnitude of these observations becomes less important than their relative position within the data set (Ramsey and Schafer, 1997). Use of a non-parametric comparison basis justifies inclusion of these observations.

For each column length, Table 5.4 presents the results of a statistical comparison to test the hypothesis of equal D_{theory} means between grades. For example, Table 5.3 includes a two-sided p-value entry of 0.4553 for a test of equal D_{plaut} means between the "Select Structural" and "Standard" grade columns of 10 foot length. Ramsey and Schafer (1997) provide a detailed discussion of P-values and their meaning. In short, the smaller the P-value, the more likely it is that the mean sample difference has statistical significance. Table 5.4 does not suggest a significant grade effect on the mean performance of Tsien's equation or the 2% rule. Table 5.4 provides some indication that grade may influence D_{plaut} and D_{winter} with shorter column lengths, but the evidence is not convincing.

Table 5.4: Two-Sided Probabilities to Test the Hypothesis of Equal D_{theory} Means Between Grades (n = 774)*

Criteria	4 foot	6 foot	8 foot	10 foot
D_{plaut}	0.0674	0.0650	0.4700	0.4553
D_{winter}	0.0932	0.0439	0.5005	0.3695
D_{tsien}	0.1707	0.1980	0.8629	0.3538
$D_{2\%}$	0.7307	0.5632	0.4611	0.1080

*Chi-square approximation from a Kruskal-Wallis test between grades

Since the data provided no clear proof of a significant grade effect on the mean D_{theory} for any brace analysis method, we pooled the data from both grades to check for length effects. Table 5.5 summarizes the results of a statistical comparison to test the hypothesis of equal D_{theory} means between lengths. For example, Table 5.5 includes a two-sided P-value entry of 0.3431 to summarize a combined-grade comparison of mean D_{plaut} between the 4 and 6 foot column lengths. Table 5.5 does not suggest a length effect on the mean performance of the 2% Rule. However, it does provide convincing evidence of a length effect on D_{theory} for Plaut's method, Winter's method, and Tsien's equation. The analysis suggests that the D_{theory} data for these three theories could be grouped into two similar populations: 4/6 foot columns and 8/10 foot columns.

Table 5.5: Two-Sided Probabilities to Test the Hypothesis of Equal D_{theory} Means Between Lengths (n = 774)*

Criteria	4 vs. 6 foot	4 vs. 8 foot	4 vs. 10 foot	6 vs. 8 foot	6 vs. 10 foot	8 vs. 10 foot
D_{plaut}	0.3431	0.0003	0.0001	0.0010	0.0006	0.9091
D_{winter}	0.2492	0.0003	0.0006	0.0019	0.0046	0.6145
D_{tsien}	0.1605	0.0001	0.0001	0.0008	0.0002	0.9450
$D_{2\%}$	0.6917	0.7321	0.8786	0.4741	0.5870	0.9196

*Chi-square approximation from a Kruskal-Wallis test between lengths

Based on this length and grade effect analysis of performance variable means, we pooled together the data into two groups: 4/6 foot and 8/10 foot columns of both grades. Further combination of these populations into a single length group would be justified for the 2% Rule. However, the split was maintained for the 2% Rule to allow for paired comparisons of mean performance variables between theories. Table 5.6 and Figure 5.5 summarize the pooled performance variable distributions.

Table 5.6 also includes an estimate of the proportion of each performance variable that falls below an upper tolerance limit of zero. Remember that a D_{theory} greater than zero indicates that the theory underestimated the brace strength and stiffness requirements for that test. As an example, Table 5.6 indicates that we have

95% confidence that 80.1% of the D_{plaut} population will be negative for the 4/6 foot column population. Alternatively, we are 95% confident that Plaut's method provides a conservative estimate of the brace support requirements 80.1% of the time for this length group.

To confirm the effect of length, we tested the hypothesis of equal mean performance variables between the pooled length groups of Table 5.6. This check confirmed a statistically significant difference between the 4/6 foot and 8/10 foot column populations for Plaut's method, Winter's method, and Tsien's equation (two-sided probabilities less than 0.0001 from a chi-square approximation of a Kruskal-Wallis test).

Table 5.6: Overall Summary of Performance for Each Brace Analysis Theory

Performance Variable	Length Class	Sample Size	Average	Standard Deviation	Percentage of Population Below an Upper Tolerance Limit Of D_{theory}=0 (95% Conf.) %
D_{plaut}	4' – 6'	380	-0.28	0.29	80.1
	8' – 10'	394	-0.37	0.25	91.3
D_{winter}	4' – 6'	380	0.01	0.38	45.7
	8' – 10'	394	-0.10	0.35	57.9
D_{tsien}	4' – 6'	380	-0.11	0.38	57.8
	8' – 10'	394	-0.25	0.30	76.1
$D_{2\%}$	4' - 6'	380	-0.54	0.36	91.1
	8' - 10'	394	-0.54	0.37	91.1
	Pooled length	774	-0.54	0.37	90.8

Within each of the pooled length classes, we next conducted series of paired statistical tests of a hypothesis of equal D_{theory} means between the theoretical brace analysis methods. As suggested above, these paired comparisons required stratification of the $D_{2\%}$ data by column length. Every comparison provided strong statistical evidence of a mean difference between brace theories (two-sided p-value approximations always less than 0.0001 from a Wilcoxon signed-rank test). That is, each theory yielded distinctly different performance. Additional non-paired tests showed that, with the exception of D_{winter} for the 4/6 foot group, we could reject the hypothesis that any mean D_{theory} actually equals zero (two-sided p-value approximations always less than 0.0001 from a Wilcoxon signed-rank test).

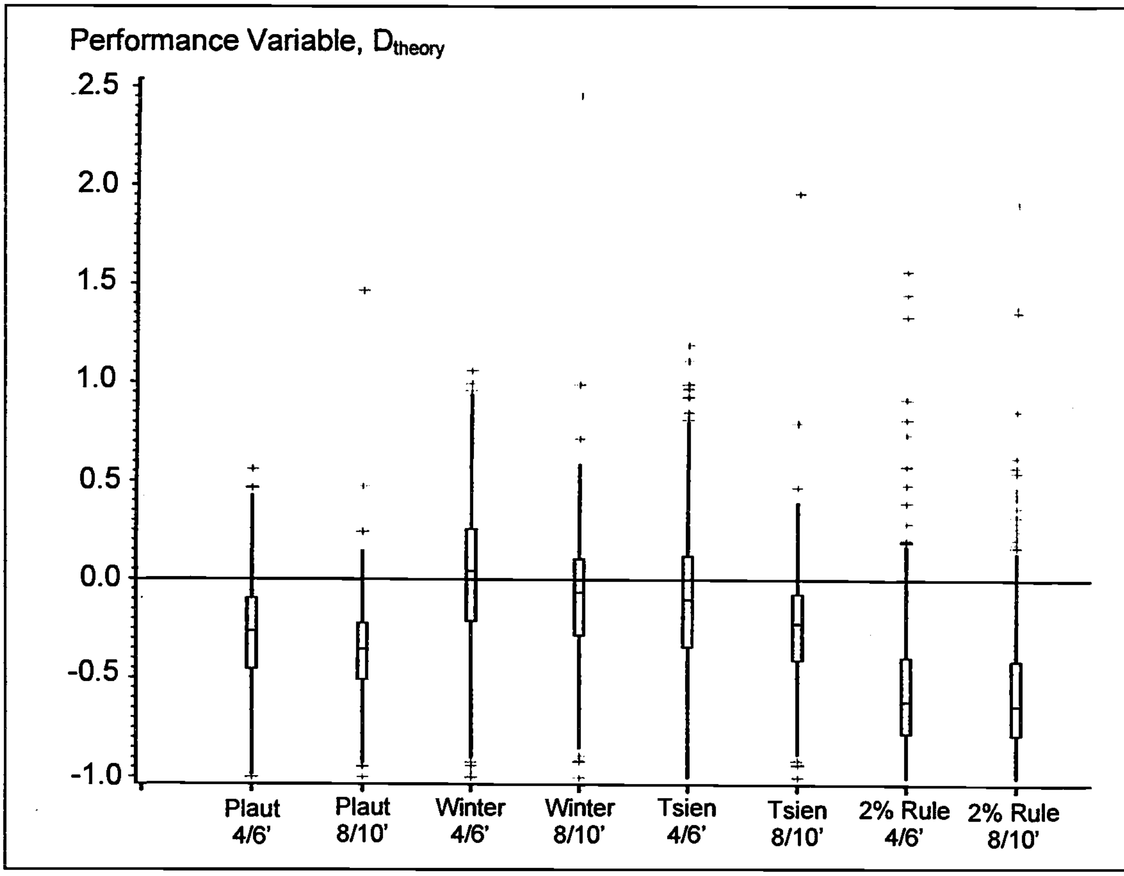


Figure 5.5: Performance Variable Comparison, All Observations

5.4.3 Length and Grade Analysis Summary

Based on the statistical comparisons of D_{theory} means between experimental groups, we found the following:

- No convincing evidence exists to suggest a grade effect on the performance of any brace analysis theory.

- A significant length effect exists between the 4/6 foot and 8/10 foot column groups for Plaut's method, Winter's method, and Tsien's Equation. This length effect was not present for the 2% Rule.

- The mean performance of all the brace analysis theories differed. On average, with the exception of Winter's method for the 4/6 foot column group, none of the brace analysis theories estimated the actual brace support requirements for strength and stiffness.

5.5 Variability Source Observations

As expected, none of the brace theories perfectly estimated the actual brace support requirements. If one of them had, then every D_{theory} observation for that theory would be equal to zero. In this next phase of the analysis, we looked for relationships between the performance variables for each theory and some of the experimental factors that might explain the observed variability.

The observations in this section result from visual inspection of graphs between the performance variables for each brace theory and experimental factors like column load, Δ_0, E, I, and so on. This review includes the pooled performance variable observations from all 774 sample columns. This procedure was similar to a regression analysis of residuals. However, in reality, many experimental factors were not controlled with the experimental design and differ between observations. As a result, detailed statistical analysis of these relationships was neither justified nor attempted. Observed trends should be considered suggestive and not conclusive.

5.5.1 Initial Lateral Deflection, Δ_0

All four of the brace analysis theories establish the initial lateral deflection of the column at the brace, Δ_0, as one of the most important variables of concern. Plaut's method, Winter's method, and Tsien's equation, explicitly include Δ_0 in the calculation of the required brace strength and stiffness. The 2% Rule incorporates an assumed Δ_0 on the order of L/200.

Figure 5.6 illustrates the relationship between D_{plaut} and Δ_0/L. Plots of D_{winter} and D_{tsien} vs. Δ_0/L appear very similar. In general, these graphs suggest little influence of Δ_0/L on D_{plaut}, D_{winter}, and D_{tsien}. More non-conservative observations and variability occur at lower Δ_0/L's, but this can probably be attributed to a greater number of observations in this region. In effect, the inclusion of Δ_0 as a specific parameter in these three theoretical brace analysis methods appears to practically eliminate it as a source of variability in D_{theory}.

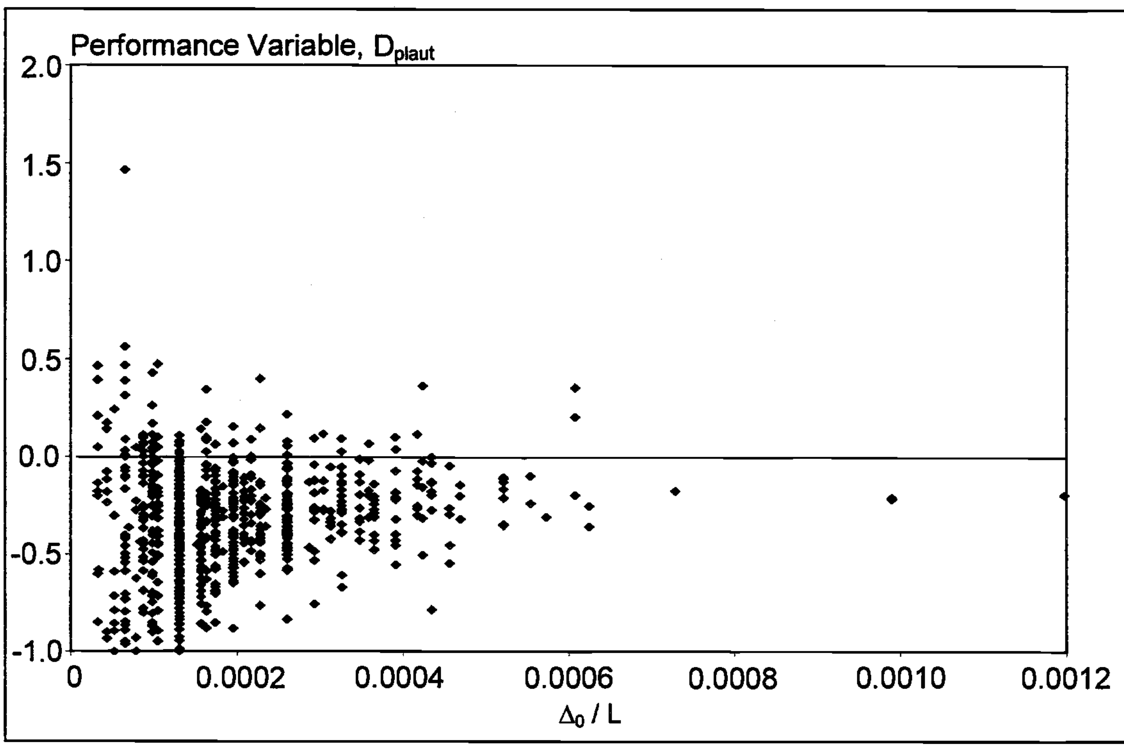


Figure 5.6: D_{plaut} vs. Δ_0/L

For the 2% Rule, as seen in Figure 5.7, the story is different. Here, the data suggests a strong linear relationship between the two variables with $D_{2\%}$, on average, increasing with larger Δ_0/L's. In other words, the 2% rule becomes less conservative with increases in initial lateral deflection. As presented in Section 2.2.3, this behavior can be explained by the fact that the 2% rule incorporates a fixed assumption of Δ_0 equal to L/200. Theoretically, the brace force estimates for columns with Δ_0 below the assumed level should be conservative and those for columns with Δ_0 above it should be non-conservative. However, Figure 5.7 implies that the boundary for conservatism occurs at a Δ_0/L of about 0.0004, an order of magnitude smaller than the assumed L/200. This data suggests that a rule of thumb based on Δ_0 and P alone will not fully describe the brace support requirements.

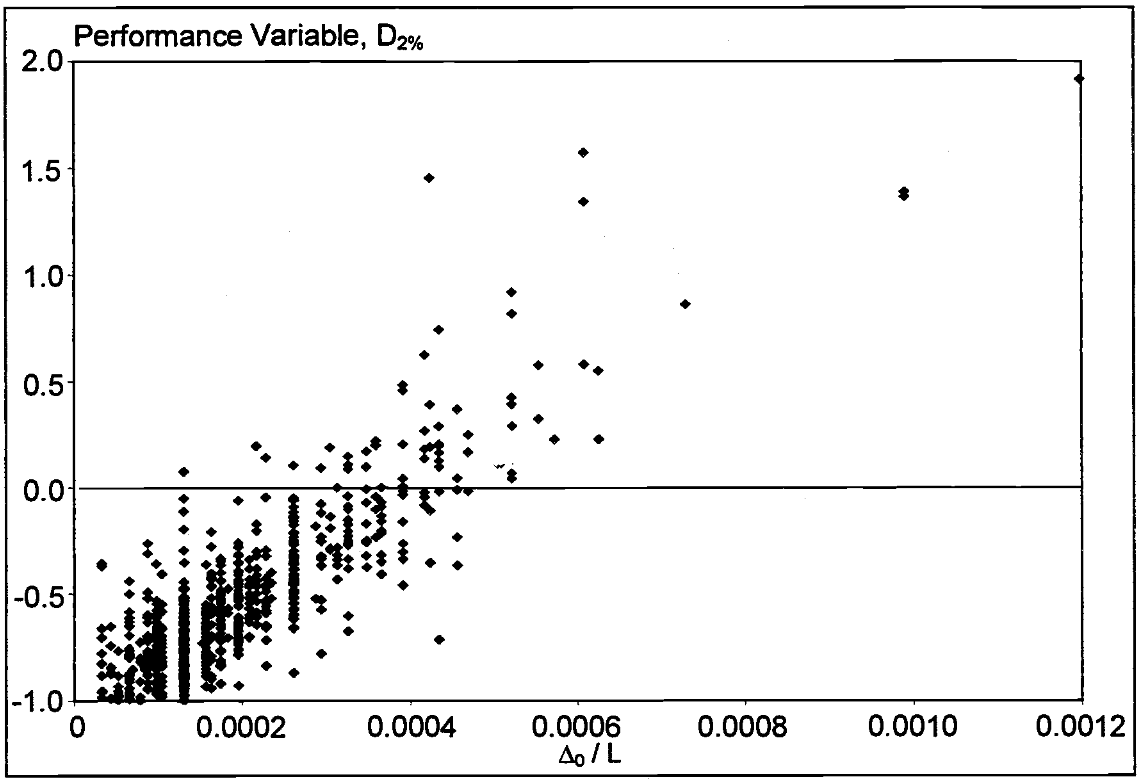


Figure 5.7: $D_{2\%}$ vs. Δ_0/L

5.5.2 Column Load, P

All four of the brace analysis theories investigated in this project include the column load, P, as a specific input parameter. With the exception of those columns that experienced a failure during the test, Table 4.3 defines the maximum axial test load for each column length and grade. For those columns that experienced a column or brace failure, the maximum axial test load is the axial load at failure. Figure 5.8 shows the relationship between the performance variable D_{plaut} and P. Plots of D_{winter}, D_{tsien}, and $D_{2\%}$ have a similar appearance. In general, they suggest a slight increase in the average performance variable with increasing column load. In other words, the theoretical brace analysis theories become slightly less conservative at higher column stress levels. This might be attributed to the tendency for inelastic buckling with higher stresses. However, as suggested by Tables 4.3 and 5.2, the columns with higher axial loads tend to be shorter and supported by stiffer braces. Consequently, it is not clear which experimental factor or combinations of factors account for the observed relationships between the performance variables and column loads. It is also not clear that this potential influence is practically significant.

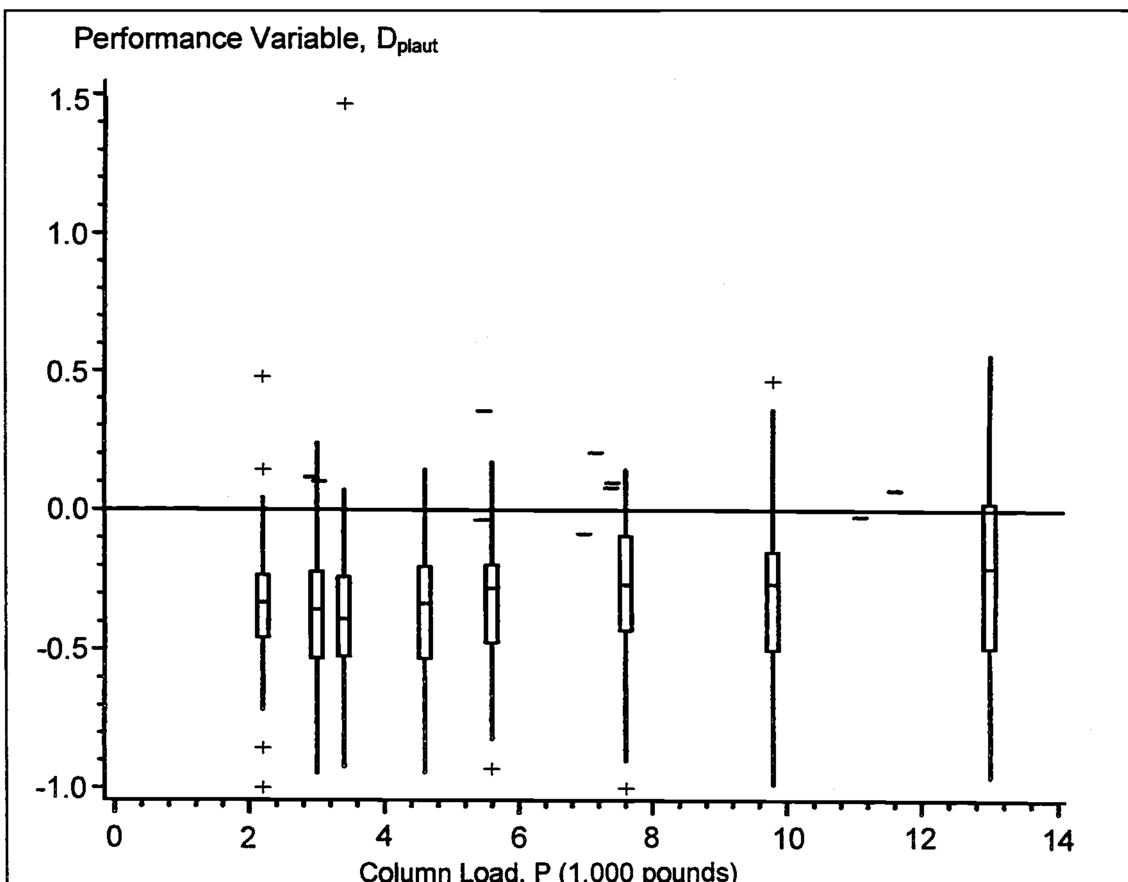


Figure 5.8: D_{plaut} vs. Column Load

5.5.3 Initial Column Profile

All four of the theoretical brace analysis methods assume that the initial curvature about the member weak axis approximates a symmetrical, half-wave shape with maximum initial lateral deflection at the brace. That is, they assume an initial "C-shape" profile rather than an "S–shape". An "S-shape" would correspond to an asymmetrical, full-wave initial curvature with maximum deflection near the quarter points.

Table 5.7 summarizes the performance variables for each brace theory by initial column profile and suggests that all four brace theories are less conservative for "C-shaped" columns than for "S-shaped" columns. This implies that the assumed "C-shape" profile probably represents the worst case for brace design. This is consistent with the concept that a "C-shaped" column will experience immediate lateral deflection at mid-height upon application of an axial load. An "S-shaped" column would intuitively experience the most lateral deflection near the quarter-heights, with less lateral deflection to be restrained at the mid-height brace.

Table 5.7: Performance Variables by Initial Column Profile

Performance Variable	Initial "C-Shaped" Profile (691 Observations)	Initial "S-Shaped" Profile (83 Observations)
D_{plaut} average (min. to max., stdev)	-0.31 (-1.00 to 1.47, 0.26)	-0.44 (-1.00 to 0.46, 0.33)
D_{winter} average (min. to max., stdev)	-0.02 (-1.00 to 2.46, 0.35)	-0.23 (-1.00 to 0.71, 0.42)
D_{tsien} average (min. to max., stdev)	-0.16 (-1.00 to 1.97, 0.33)	-0.31 (-1.00 to 0.97, 0.44)
$D_{2\%}$ average (min. to max., stdev)	-0.51 (-1.00 to 1.92, 0.37)	-0.76 (-1.00 to 0.07, 0.22)

We also recorded whether the maximum initial lateral deflection occurred at the brace height or elsewhere on the column. All of the brace theories make the assumption that the maximum lateral deflection coincides with the point of brace attachment. Table 5.8 suggests little practical difference in the performance variable averages between the two conditions.

Table 5.8: Performance Variables by Maximum Initial Lateral Deflection Location

Performance Variable	Maximum Initial Lateral Deflection at Brace (575 Observations)	Maximum Initial Lateral Deflection not at Brace (199 Observations)
D_{plaut} average (min. to max., stdev)	-0.32 (-1.00 to 1.47, 0.27)	-0.36 (-1.00 to 0.56, 0.28)
D_{winter} average (min. to max., stdev)	-0.03 (-1.00 to 2.46,0.36)	-0.10 (-1.00 to 0.96, 0.37)
D_{tsien} average (min. to max., stdev)	-0.16 (-1.00 to 1.97, 0.35)	-0.22 (-1.00 to 0.99, 0.36)
$D_{2\%}$ average (min. to max., stdev)	-0.53 (-1.00 to 1.92, 0.38)	-0.56 (-1.00 to 0.58, 0.33)

5.5.4 Brace Stiffness

With the exception of the 2% Rule, all of the brace theories prescribe requirements for brace stiffness. Figure 5.9 shows the relationship between D_{plaut} and the brace support curve selected for each test. Plots for D_{winter} and D_{tsien} are comparable. Figure 5.10 depicts the relationship for $D_{2\%}$.

In these plots, the brace support curve identification for each observation corresponds with the brace support curves illustrated in Figures 3.5 through 3.8. As discussed in Section 3, these curves represent an estimate of the strength and stiffness of brace support supplied to the column by a lateral/diagonal bracing system. For example, all of the observations clustered as "2 trusses" within the "6-foot" group of Figure 5.9 were tested with the brace support curve identified as "2 trusses" in Figure 3.6. This curve estimates the cumulative brace support provided to a 6 foot compression web by a lateral/diagonal bracing system with a diagonal brace for every two web members. Section 4.5.3 and Appendix D outline the procedure used to select the brace support curve for each test

For a given column length in Figures 5.9 and 5.10, the brace support stiffness increases from right to left. The box plots suggest that, within a given length, the average D_{theory} tends to increase with increasing brace stiffness. In other words, the stiffer the brace, the less conservative the theoretical brace analysis method. As discussed in Section 2.2.5, one of the concerns about Winter's method is that it does not account for the possibility of moment reversal in the column that may occur with stiffer lateral braces. Failure to account for this reversal would underestimate the brace

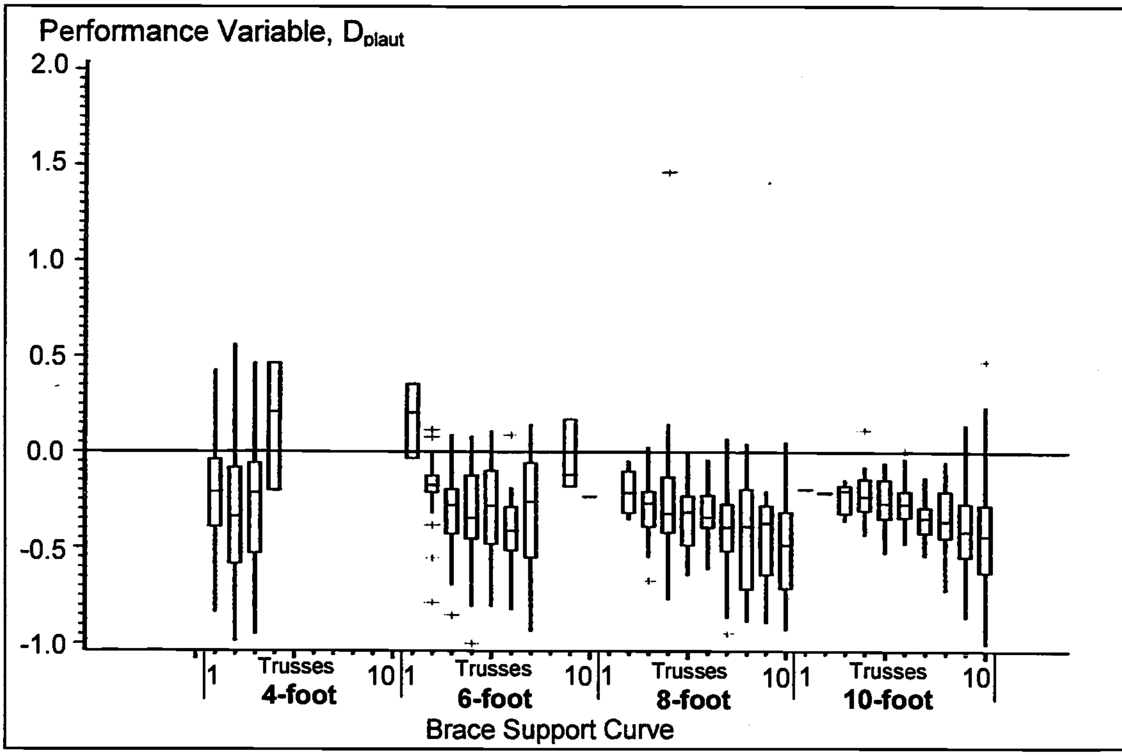


Figure 5.9: D_{plaut} vs. Brace Support Curve

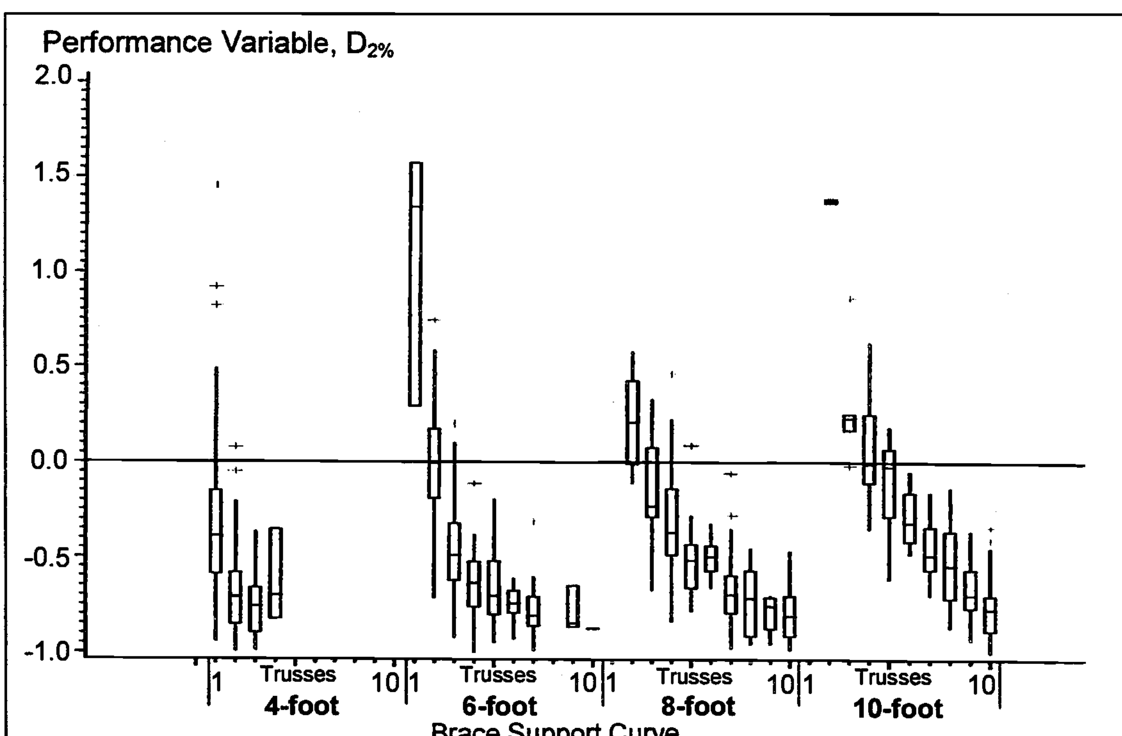


Figure 5.10: $D_{2\%}$ vs. Brace Support Curve

support requirements. The loss of conservatism in our results with stiffer braces would seem to support this assertion.

However, in Figures 5.9 and 5.10, the brace support stiffness is not the only test parameter that changes from right to left with a given column length. As outlined in Section 4.5.3 and Appendix D, we selected the brace support curves for each test based on Plaut's method. This procedure leads to the selection of stiffer brace support curves for columns that have larger Δ_0's and/or axial test loads. However, the effects of P and Δ_0 on D_{plaut} illustrated in Figures 5.6 and 5.8 seem less pronounced than the effect of brace stiffness support in Figure 5.9. On a qualitative level, this suggests that the brace stiffness is the most important of the three factors in terms of estimation variability for Plaut's method. Inspection of plots for D_{winter} and D_{tsien} suggest similar trends.

In Figure 5.10, the effect of brace support stiffness on $D_{2\%}$ seems to be even more pronounced than on D_{plaut} in Figure 5.9. However, Section 5.5.1 suggests that $D_{2\%}$ is positively correlated with Δ_0. This correlation could account for much of the increased effect of "brace stiffness" within a given length group. It is difficult to make any qualitative judgements concerning the relative importance of brace stiffness and Δ_0 on $D_{2\%}$.

5.5.5 Mill

Table 5.9 summarizes the performance variables for each brace analysis theory as affected by the origin of the test lumber. This table suggests little practical influence of mill origin in D_{plaut}, D_{winter}, and D_{tsien}. $D_{2\%}$ proved slightly less conservative for the lumber produced by Morton Forest Products, which can probably be explained by a larger Δ_0 for the lumber from this mill (one-sided probabilities less than 0.0001 from chi-square approximations for a series of Kruskal-Wallis tests). As seen in Section 5.5.1, $D_{2\%}$ tends to increase with increasing initial Δ_0. This implies that the "mill effect" may be attributed to a "Δ_0" effect.

Table 5.9: Performance Variables by Mill

Performance Variable	Douglas County Forest Products (200 Observations)	Frank Lumber Company (291 Observations)	Morton Forest Products (283 Observations)
D_{plaut} average (min. to max., stdev)	-0.32 (-0.95 to 0.47, 0.25)	-0.35 (-1.00 to 0.56, 0.27)	-0.30 (-1.00 to 1.47, 0.29)
D_{winter} average (min. to max., stdev)	-0.05 (-0.92 to 0.93,0.34)	-0.08 (-1.00 to 0.96, 0.35)	-0.01 (-1.00 to 2.46, 0.39)
D_{tsien} average (min. to max., stdev)	-0.18 (-0.94 to 0.93, 0.32)	-0.19 (-1.00 to 1.19, 0.37)	-0.16 (-1.00 to 1.97, 0.35)
$D_{2\%}$ average (min. to max., stdev)	-0.58 (-0.99 to 0.58, 0.31)	-0.60 (-1.00 to 1.36, 0.30)	-0.45 (-1.00 to 1.92, 0.45)

5.5.6 Column and Brace Failures

Table 5.10 summarizes the performance variables by observed failure of the column or brace during testing. On average, all four brace analysis theories proved non-conservative in tests with a failure. In other words, the brace support requirements were underestimated for the failed columns. Given that the brace support curve selection procedure was intended to provide a sufficient brace according to Plaut's method, the most stringent brace analysis theory that considers brace stiffness, this result should be expected. Any failure of the brace by definition means that the selected brace support curve was both insufficient and non-conservative. Note that with few observed failures, this observation may be of limited value.

Table 5.10: Performance Variables by Failure

Performance Variable	Column or Brace Failure (21 Observations)	No Failure (753 Observations)
D_{plaut} average (min. to max., stdev)	0.02 (-0.45 to 0.36, 0.19)	-0.34 (-1.00 to 1.47, 0.27)
D_{winter} average (min. to max., stdev)	0.30 (-0.19 to 0.99,0.33)	-0.06 (-1.00 to 2.46, 0.36)
D_{tsien} average (min. to max., stdev)	0.27 (-0.37 to 0.63, 0.21)	-0.19 (-1.00 to 1.97, 0.34)
$D_{2\%}$ average (min. to max., stdev)	0.26 (-0.37 to 1.57, 0.63)	-0.56 (-1.00 to 1.92, 0.33)

Of the four performance variables listed, D_{plaut} has the lowest average and standard deviation for the tests with some form of failure. These parameters should be lower than the corresponding estimates for Winter's method or Tsien's equation; Plaut's method was intended to be more conservative. Similar comparison of D_{plaut} with $D_{2\%}$ suggests that Plaut's method may provide the closest prediction of the brace support requirements, even in instances where the column experiences some form of failure.

5.5.7 Miscellaneous Factors

As part of the test program, we measured other factors such as the weak-axis moment of inertia, dry weight per unit length, longitudinal modulus of elasticity, and moisture content for each test column. These factors could have an impact on the performance of the theoretical brace analysis methods. However, as suggested by Table 5.1, none of these parameters varied widely between the test samples. Examination of the plots of D_{theory} against each of these variables suggests no discernable correlation.

5.6 Torsional Buckling

As mentioned in Section 2.3.5, torsional buckling of the column could theoretically prove important for some compression webs supported by a lateral/diagonal bracing system. With this bracing scheme, as with our testing, the brace is not attached to the shear center of the column. However, torsional buckling was not observed in any of 859 columns tested.

6.0 DISCUSSION

6.1 Brace and Column Failures

The observed brace "instability" failures in 1% of the test columns represent an unexpected result of this investigation. These were instances where the brace continuously acquired additional lateral load and deflection as the test machine maintained a constant axial test load on the column. Eventually, the measuredF_{br} and Δ exceeded the limits of the brace support curve selected for the test and resulted in a brace "failure".

One possible cause for this instability may be the test machine itself. The computer control software maintains the axial and brace loads as independent operations. At the end of an axial load application ramp, the following theoretical scenario could conceivably explain the observed instability:

1.) The computer slightly increases Δ to adjust to the current brace load, $F_{br.}$
2.) This deflection slightly lowers the axial load, P.
3.) The axial load controller applies additional force to maintain the target load.
4.) The impact from this axial load application slightly increases F_{br}.
5.) Step 1 repeats.

Given that the potential for this chain of events existed for every test, it seems unlikely that it caused an "instability" failure in just 1% of the column samples. If the test machine control software instigated the failure, then we would expect more brace "instability" failures to have occurred. It is possible that an occasional, slow instability failure might have been overlooked. However, the observed instability in the noted "brace" failure columns was dramatic.

A more plausible explanation considers that all of the columns to experience an instability failure were well into the non-linear portion of their brace support curves when the axial load reached the target level. In other words, the brace stiffness had significantly degraded before the test machine attempted to maintain lateral equilibrium. It seems likely that the lateral brace stiffness for these samples fell below what was necessary to stabilize the column before the axial load achieved its maximum test level. This stiffness loss would introduce larger Δ for the same unit increase in F_{br}. Even at a constant axial load, these progressively larger increases in Δ could cause further increases in F_{br} and start a chain of events that induces brace instability.

The full reason for the brace instability failures is not known. However, concerns with it can be minimized if the resulting design procedure limits the ultimate brace load levels to the "linear" range of brace stiffness. We did not observe any instability failures in this range.

Only 8 of 774 columns, or 1%, failed in asymmetrical buckling. As outlined in Appendix B, we selected the axial test loads to approximate a 5% exclusion limit on braced column strength. Since weak-axis buckling represented the major failure risk for all test columns, the lumber in this study may be slightly stiffer than the global populations of the same grades. Comparison of the measured E values in Table 5.1 with the published averages for the same grades supports this assertion (American Forest and Paper Association, 1997). On average, the E of our test lumber generally exceeded the standard values. However, this doesn't affect the conclusions of this study regarding the applicability of the theoretical brace analysis methods. We loaded all of the test columns to the ultimate load levels implicitly assumed by the structural designer. In addition, as Section 5.5.7 suggests, we found little indication that E has a significant impact on the performance of the brace design theories investigated by this research.

6.2 Length and Grade Effects

The lumber grades used for this test program differed with respect to their degree of homogeneity. The "Standard" grade material included more knots of larger size, more wane, increased slope of grain, and more planer skip than the "Select Structural" material. In addition, we found that the "Standard" grade lumber had a lower average modulus of elasticity than the "Select Structural" lumber. We observed little practical difference between grades with respect to their member dimensions, moisture contents, dry weights per foot, and initial curvatures. In the end, lumber grade did not significantly affect the ability of any brace analysis method to estimate the lateral support requirements.

In contrast, we did find statistically significant evidence of a length effect. Plaut's method, Winter's method, and Tsien's equation all proved to be less conservative on average for the 4/6 foot columns than for the 8/10 foot columns. Column length had no significant impact on the average performance of the 2% Rule. The exact cause for the length effect is uncertain. Factors that may contribute include differences in the axial test loads, the brace support stiffness used for each test, and the flatwise column modulus of elasticity.

The axial column load and its resulting axial stress level might impact the relationship between the column and the brace. As discussed in Sections 2.1.3 and 2.1.4, higher axial load levels can stress all or part of the column cross section beyond its proportional limit. This may lead to an overall reduction in the effective modulus of elasticity of the column and induce "inelastic" buckling at a load level below that predicted by Euler's equation (Equation 1). With the exception of Tsien's equation, none of the brace theories affected by length was specifically developed to account for the potential of inelastic buckling.

However, several factors seem to contradict the likelihood that the column stress level or the observed modulus of elasticity differences account for the loss of conservatism with shorter column lengths. First, the overall data in Section 5.5.2 provides only a slight suggestion of a column load effect on the performance of the three analysis methods with a length effect. Second, the inelastic buckling issue can be conceptually reduced to a modulus of elasticity problem. As discussed above, inelastic buckling occurs sometime after the effective modulus of elasticity of the column falls below its initial value. Neither the comparison of performance between grades in Section 5.4.2 nor the overall look at modulus of elasticity effects in Section 5.5.7 suggest that modulus of elasticity variations have a significant impact on the brace analysis methods. Furthermore, compared with the 6 foot columns, the 4 foot columns started with a significantly lower average modulus of elasticity and supported a much larger axial load. If inelastic buckling and/or modulus of elasticity were the most significant factors to account for this length effect, then we should expect aD_{theory} difference between these two lengths. We did not find a statistically significant difference for any performance variable. In contrast, a significant length effect was found between the 6 and 8 foot column lengths, even though both the column loads and modulus of elasticity variations were much smaller than between the 4 and 6 foot lengths. This discrepancy suggests that something other than column load and modulus of elasticity accounts for the observed length effect.

The stiffness of the brace support curve used for the test may also relate to the observed length effect. As Table 5.2 indicates, we generally selected stiffer brace support curves for shorter columns. The reason for this is that Plaut's method was used to select the brace support curve for each column test. This method leads to selection of stiffer braces for columns that experience larger axial loads. As suggested in Section 5.5.4, stiffness of the brace support curve may impact the performance of the analysis methods, especially for columns of the same length. This concept aligns with Plaut's (1993a) assertion that stiffer braces create the potential to induce bending moment reversal in the column at the brace. This reversal theoretically increases the brace strength and stiffness requirements and reduces the conservatism of the design method. This was one of the reasons why Plaut suggested an alteration of Winter's method. However, the relative stiffness differences between the column and the brace may not significantly change for the column length and brace combinations used in this experiment. In other words, the braces used for a 4 foot column were generally stiffer than for a 10 foot column, but the 4 foot column is more resistant to mid-height deflection than a 10 foot column. Column modulus of elasticity and stress level changes with length further complicate the comparisons.

In the end, the data suggest the presence of a statistically significant length effect on the performance of three of the theoretical brace analysis methods. Due to a co-mingling of different possible causes, we did not isolate its source. However, inspection of Table 5.5 and Figure 5.5 suggests that the observed length effect has minor practical importance for any one brace analysis method.

6.3 Brace Design Method Performances

We used performance variables to assess the relative performance of each simplified brace analysis method; Table 5.6 and Figure 5.5 summarize the outcome. All four theories yield significantly different results and are discussed separately.

6.3.1 Plaut's Method

Plaut's brace analysis methodology (Section 2.2.5) provided a conservative estimate of the brace strength and stiffness requirements for 80-90% of the test samples. Consistent conservatism may be a strength of this method. Of equal importance is that D_{plaut} had the lowest standard deviation of its performance variable. This suggests that Plaut's method provided the least residual variability between the actual and estimated brace support requirements. Compared with the 2% Rule, the other conservative method for predicting F_{br}, Plaut's method resulted in the performance variable average closest to zero. In other words, it was the more accurate of the two.

Plaut's method can be used as a reasonable brace design basis. If the degree of conservatism it provides is not sufficient for design applications, then an additional safety factor reduction may be appropriate. In essence, Plaut's method represents a modified version of Winter's procedure with a 1.5 factor applied to the Δ_0 terms of Winter's equations for brace strength and stiffness. Plaut (1993a) recommends the 1.5 factor as an alteration that should be sufficient to renderWinter's method conservative for most brace design situations. This factor accounts for moment reversals in the column at the brace and initial column profiles other than a sinusoidal half wave. This coefficient was not explicitly derived. A code committee could further adjust it to provide the desired degree of conservatism. For example, using the results of this study, Plaut's method could be used with 95% confidence to determine conservative brace design requirements for this application 95% of the time by raising the 1.5 coefficient to 1.75.

6.3.2 Winter's Method

Winter (1960) devised his brace design method (Section 2.2.4) with the intent of creating a conservative basis for determining the required strength and stiffness of discrete lateral support. Based on the results of a theoretical analysis, Plaut (1993a) suggested that Winter's method might not always be conservative. For this application, this test program confirms Plaut's concern. Inspection of the performance variable data of Table 5.5 suggests that Winter's theory provides a conservative estimate of the brace design requirements only about half of the time. From another perspective, however, with performance variable averages close to zero, Winter's method was the best predictor of the actual brace support requirements.

As the best predictor of the lateral brace support requirements, Winter's method could be modified with a safety factor to provide a conservative brace design basis. Plaut made one such modification by applying a coefficient to Δ_0 to make the method more demanding. A second option would be to use Winter's method to calculate the brace strength and stiffness requirements and then apply an overall safety factor. While this would also work, it would probably be less advantageous. With a lower standard deviation of its performance variable, inspection of Table 5.5 and Figure 5.5 suggests that Plaut's modification removed considerable variability from Winter's method. Simply shifting the D_{winter} means downward in Figure 5.5 to achieve the desired level of conservatism for Winter's method would probably result in a more variable and less accurate design method than that offered by Plaut.

At least one possible justification exists for using an overall adjustment to Winter's method instead of a Δ_0 coefficient adjustment similar to that proposed by Plaut. As discussed in Section 2.4, several authors extended Winter's procedures to address cases where multiple discrete braces stabilize a column. The multiple-brace situation lies beyond the scope of Plaut's work and this investigation. Further study of multiple brace applications may prove that an overall safety factor approach provides a more general design solution.

6.3.3 Tsien's Equation

Tsien's equation (Section 2.3.3) represents the most computationally rigorous brace analysis alternative used in this study. We examined it for two reasons. First, it is the only analysis method we found that was intended to be predictive. Second, it was developed specifically for the situation where an inelastic brace supports the column.

With performance variable averages farther from zero, Tsien's equation was less accurate than Winter's method. However, Tsien's equation was not as conservative as Plaut's method or the 2% Rule. Table 5.5 indicates that the standard deviation of D_{tsien} was greater than that of D_{plaut} and comparable to the other two alternatives. In other words, Tsien's equation was not as successful as Plaut's method in reducing the residual variability between the estimated and observed brace support requirements. Given the complexity of Tsien's equation and the observed performance measured by D_{tsien}, little justification exists to support use of Tsien's equation for brace design.

6.3.4 2% Rule

Of the four alternatives, the 2% Rule yielded the most conservative and the most variable estimation of the required brace force. However, it is important to note that the degree of conservatism suggested for this method by this study may be misleading. As discussed in Section 5.4.1, the performance variables provide a simultaneous comparison of both brace strength and stiffness requirements for Plaut's method, Winter's method, and Tsien's equation. All three of these theories address both the strength and stiffness of the brace. The 2% Rule considers only the brace strength and does not explicitly or implicitly consider brace stiffness. The performance variable results for the 2% rule may be misleading since the brace support curves were selected based on Plaut's method. If the decision on how to brace the column had been made based on strength alone, there is no assurance that the results would be as conservative. Figure 6.1 illustrates this issue.

Figure 6.1 uses a brace support curve to depict the relationship between F_{br} and Δ for a non-linear brace. The figure also includes theoretical brace analysis curves for Plaut's method and the 2% Rule. In this example, the brace support curve is sufficient to brace the column based solely on the strength criteria of the 2% Rule. However, the lack of an intersection between the brace support curve and the theoretical brace analysis curve for Plaut's method indicates that the brace may not provide sufficient stiffness to consider the column fully braced. The figure suggests that the brace strength required by the 2% Rule would suffice if the brace has enough stiffness to limit Δ to a level below $\Delta_{critical}$ at the maximum axial column load. $\Delta_{critical}$ in this figure represents the point of intersection between theoretical brace analysis curves for Plaut's method and the 2% Rule. Without checking the required brace stiffness, the designer cannot determine if the brace is appropriate for this column. Without question, designing the brace based on strength alone will result in a brace with finite stiffness. Some probability exists that the provided stiffness will be adequate. However, without specifically evaluating that stiffness, the bracing designer has no way to know. A design method using the functions developed by Plaut and Winter incorporates a check of necessary brace stiffness.

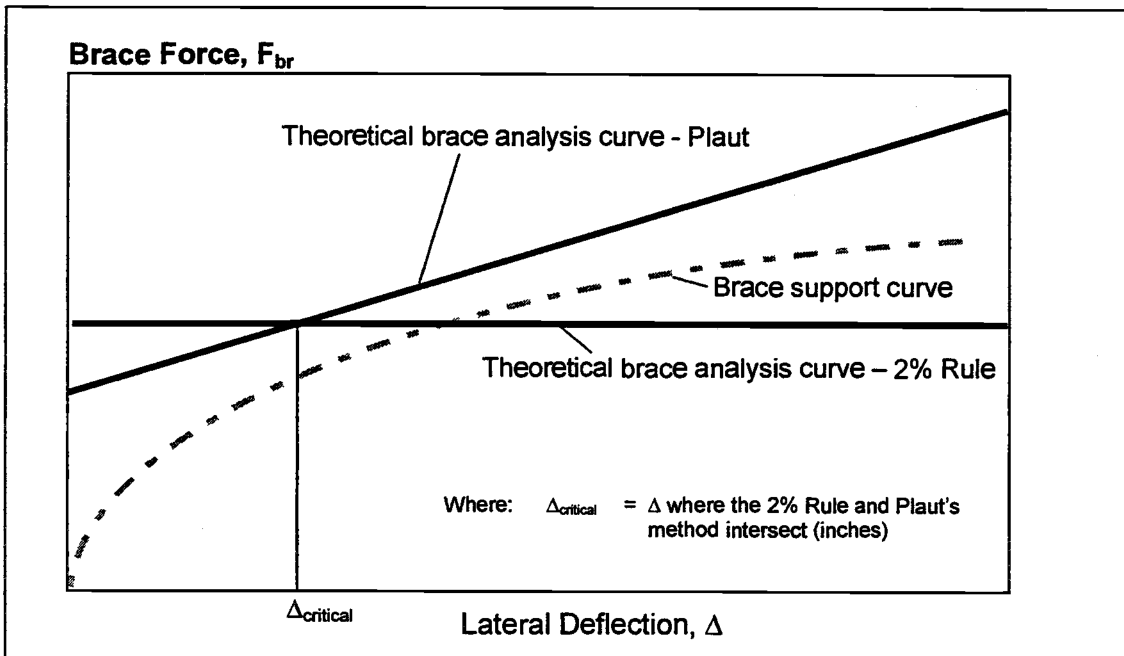


Figure 6.1: Conceptual Design Conflict with the 2% Rule

6.4 Brace Design Procedure

The primary objective of this research was to evaluate the ability of several existing, theoretical analysis models to estimate the brace strength and stiffness requirements for discrete compression web bracing. The problem of discrete compression web bracing design requires a method that balances simplicity and accuracy because of the impractical demands of an exact analysis. We found that, with explicit reductions for safety and variability, either Winter's method or Plaut's method could be used to provide such a basis. This is the primary finding of this investigation.

A secondary objective was to recommend a rational brace design procedure. The following section expands on this objective by exploring the use of Plaut's method for determination of lateral bracing design requirements.

6.4.1 Load and Resistance Factor Design (LRFD)

Figure 6.2 illustrates the objective of the proposed brace design procedure. The linear brace support curve represents the strength and stiffness of the lateral brace assembly to be checked for adequacy. To avoid possible instability, we assume that the brace forces will be kept within the practical linear elastic range of brace stiffness. $F_{br\text{-}ult}$ is the ultimate strength of the brace at the linear limit. With these constraints, the brace support curve can be defined using $F_{br} = K\Delta$ and an upper strength limit of $F_{br\text{-}ult}$.

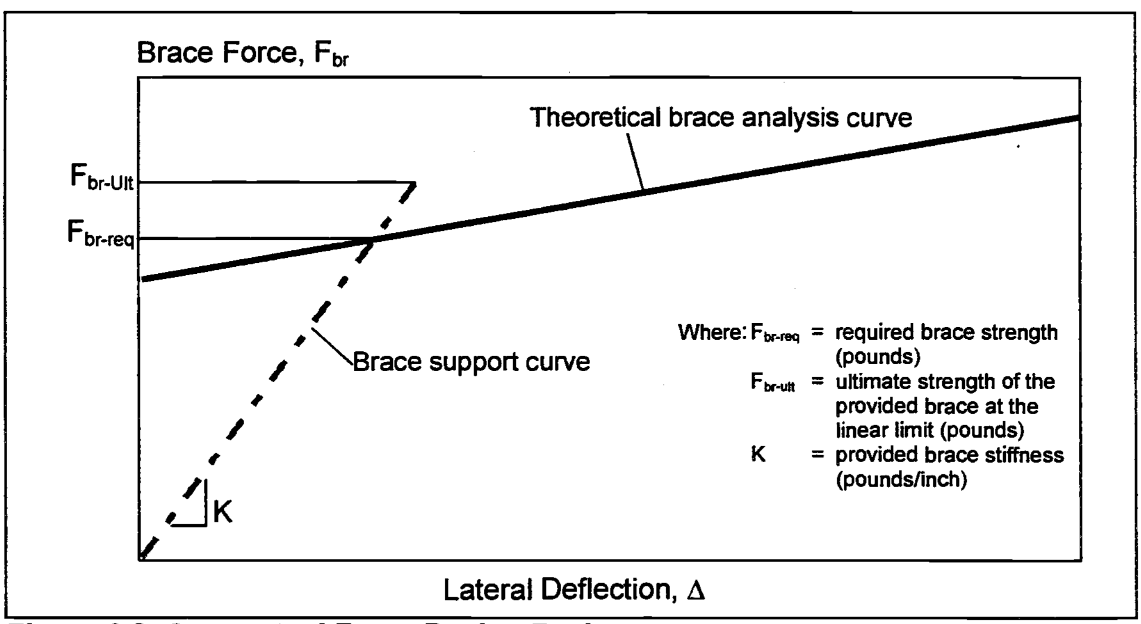


Figure 6.2: Conceptual Brace Design Basis

The theoretical brace analysis curve defines the lateral brace requirements for the column in terms of strength and stiffness. Either of the methods proposedby Plaut and Winter could be used with a modification for safety and variability. The design check is to ensure that the two curves of Figure 6.2 intersect. If the curves do not intersect, then the brace does not have sufficient strength and/or stiffness. Either condition could result in inadequate bracing for the column and the column may buckle with a longer effective length than assumed by the truss designer.

If Plaut's method was adopted for design, Equation 5 and Equation 15 can be combined to obtain the following equation to determine the required brace strength for a given P', K, Δ_0 and L:

$$F_{br-req} = \frac{6\lambda P'\Delta_0 K}{LK - 4\lambda P'} \tag{22}$$

where: $F_{br\text{-}req}$ = required brace strength (pounds)
K = provided brace stiffness (pounds/inch)
L = effective length of the unbraced column (inches)
P' = adjusted column compression resistance of the column parallel to grain (pounds)
Δ_0 = initial lateral deflection at mid-height (inches)
λ = time effect factor (unitless)

Note that Equation 22 does not include any reductions for safety and variability that could be prescribed in LRFD or allowable stress design to address inaccuracies in the brace analysis method. In LRFD, the safety checking equation for brace capacity would be:

$$F_{br-req} \leq \phi_b F_{br-ult} \tag{23}$$

where: $F_{br\text{-}req}$ = required brace strength (pounds)
$F_{br\text{-}ult}$ = ultimate strength of the provided brace at the linear limit (pounds)
ϕ_b = resistance factor for the brace design (unitless)

This procedure would verify that the two curves of Figure 6.2 intersect. It simultaneously checks to ensure satisfaction of both the brace strength and stiffness requirements.

The braced column resistance assumed by the truss designer for the web, P', should be used in Equation 22. The factored load actually imposed on the column should never be substituted for P'. The brace must be capable of supporting the full, column resistance assumed in the check of column adequacy. If the designer determines the brace for something less than that, then the safety factor assumed for the column design would be reduced. In other words, the ultimate column strength would become the factored column load instead of the assumed resistance, P'.

The following design example illustrates the use of this brace design procedure for a compression web member. The web design proceeds in accordance with the American Forest and Paper Association's *LRFD Manual for Engineered Wood Construction* (1996) using the following notation:

A	=	Member cross-sectional area (in^2)
c	=	Column curve constant (unitless)
b	=	Member thickness (inches)
C_M	=	Wet service factor (unitless)
C_t	=	Temperature factor (unitless)
C_F	=	Size factor (unitless)
C_P	=	Column stability factor (unitless)
C_{PT}	=	Preservative treatment factor (unitless)
C_{rt}	=	Fire-retardant treatement factor (unitless)
E_{05}	=	Reference fifth percentile modulus of elasticity (psi)
F_c	=	Reference compression strength parallel to grain (psi)
F^*_c	=	Adjusted compression strength parallel to grain (psi)
$F_{br\text{-}ult}$	=	Ultimate strength of the provided brace (pounds)
$F_{br\text{-}req}$	=	Required brace strength (pounds)
K	=	Provided brace stiffness (pounds/inch)
K_e	=	Effective length factor for compression members (unitless)
L	=	Effective length of the unbraced web (inches)
P'_0	=	Adjusted member axial parallel to grain resistance of a zero length column (pounds)
P'	=	Adjusted compression resistance of the column parallel to grain (pounds)
$P_{e\text{-}lrfd}$	=	Euler buckling load for the braced web (pounds)
α_c	=	Factor in the design of columns (unitless)
Δ	=	Lateral deflection at mid-height upon application of axial load (inches)
ϕ_b	=	Resistance factor for brace design (unitless)
ϕ_s	=	Resistance factor for stability (unitless)
ϕ_c	=	Resistance factor for compression (unitless)
λ	=	Time-effect factor (unitless)

The following will be assumed:

- 10' long compression web member, "Standard" grade, Douglas Fir-Larch, 2x4
- Short-term load duration, λ=1.0

- Δ_0 with a practical upper bound of L/200, or 0.6 in.
- A row of similar trusses requires a lateral brace to reduce the effective column length for flexural buckling about the weak axis by one-half.

From *LRFD Manual for Engineered Wood Construction*, the tabulated properties for the web member are:

F_c=3.24 ksi

E_{05}=850 ksi

Using the column design procedure to define the critical capacity of the web member:

$$F^*_c = F_c C_M C_t C_F C_{PT} C_{RT} = (3.24ksi)(1)(1)(1)(1)(1) = 3.24 \text{ ksi}$$

$$P'_0 = F^*_c A = (3.24ksi)(5.25in.^2) = 17.1 \text{ kips}$$

$$P_{e-lrfd} = \frac{0.822E_{05}A}{(\frac{K_e L}{b})^2} = \frac{0.822(850ksi)(5.25in^2)}{\left[\frac{(0.5)(120in.)}{1.5in.}\right]^2} = 2.29 \text{ kips}$$

$$\alpha_c = \frac{\phi_s P_{e-lrfd}}{\lambda \phi_c P'_0} = \frac{(0.85)(2.29kips)}{(1.0)(0.9)(17.1kips)} = 0.127$$

$$C_P = \frac{1+\alpha_c}{2c} - \sqrt{(\frac{1+\alpha_c}{2c})^2 - \frac{\alpha_c}{c}} = \frac{1+0.127}{2(0.8)} - \sqrt{(\frac{1+0.127}{2(0.8)})^2 - \frac{0.127}{0.8}} = 0.124$$

$$\lambda P' = \lambda C_P P'_o = (1.0)(0.124)(17.1kips) = 2.12 \text{ kips}$$

We did not reduce P' by the resistance factor ϕ_c. Including this factor for the brace design would reduce the implied safety factor for the column/brace assembly.

Section 3 of this report includes estimates of the short-term strength and stiffness of support provided to a 10' compression web member by a lateral/diagonal bracing system. By inspection, Figure 3.8 suggests that with a diagonal brace for every three webs, the proportional limit of brace support provided can be estimatedby F_{br-ult} = 78 lbs. and Δ = 0.04 in. Therefore:

$$K \approx \frac{F_{br-ult}}{\Delta} = \frac{78lbs.}{0.04in.} = 1950 \text{ lbs./in}$$

If an unreduced version of Plaut's method provides sufficient conservatism, then Equation 22 defines the brace strength required with the provided stiffness:

$$F_{br-req} = \frac{6\lambda P' \Delta_0 K}{LK - 4\lambda P'} = \frac{(6)(1.0)(2120lbs.)(0.6in.)(1950lbs./in.)}{(120in.)(1950lbs./in.) - (4)(2120lbs.)} = 66 \text{ lbs.}$$

Checking for adequacy with Equation 24:

$$F_{br-req} \leq \phi_b F_{br-ult}$$

$$66 \leq \phi_b (78) \text{ lbs.}$$

Appropriate values for the brace system resistance factor, ϕ_b, will need to be considered by the governing building code committees. In this case, the brace support curves of Section 3 provide a cumulative estimate of the brace stiffness and strength at the proportional limit of the assembly. ϕ factors for such composite assemblies are not presently available. In this case, where the nailed connections of the lateral/diagonal brace govern the brace support curves of Section 3, a ϕ_b equal to that of a nailed connection might be the most appropriate.

As long as ϕ_b is greater than or equal to 0.85 for this example, this brace configuration provides sufficient support. The solution would be to tie the row of compression webs together with a 2x4 lateral brace as illustrated in Figure 3.1. Diagonal braces that extend over five webs and center over every third web should be installed. All wood-to-wood connections should be made with 2-16d nails.

6.4.2 Comparison with the 2% Rule

For illustrative purposes, the same design could be attempted using the 2% rule. This rule suggests that the required strength of the brace can be defined by Equation 10:

$$F_{br} = 0.02P_{cr} = (0.02)(2120lbs.) = 42 \text{ lbs./truss}$$

Based on the LRFD Manual for Engineered Wood Construction (1996), the factored resistance for a 2-16d nailed connection in two-inch nominal Douglas-fir is about 600 lbs with short-term load duration, dry-use, and single shear conditions. From the author's experience, a common design practice is to assume that the nailed connection between the diagonal brace and the compression web limits the brace strength and carries only a horizontal load. Basing the brace design on the 2% Rule alone suggests that the diagonal braces could be spaced more than 14 webs apart:

$$Spacing = \frac{600lbs.}{42lbs./web} = 14.2 \text{ webs apart}$$

In Section 3, we did not estimate the brace support provided for the 14 truss situation. However, we did estimate the brace properties for the stiffer condition with 10 webs per diagonal. Figure 3.8 suggests that $F_{br\text{-}ult}$ = 22 lbs. and Δ = 0.04 at the proportional limit of this brace system. The required brace strength is:

$$K \approx \frac{F_{br-ult}}{\Delta} = \frac{22lbs.}{0.04in.} = 550 \text{ lbs./in.}$$

$$F_{br-req} = \frac{6\lambda P'\Delta_0 K}{LK - 4\lambda P'} = \frac{(6)(1.0)(2120lbs.)(0.6in.)(550lbs./in.)}{(120in.)(550lbs./in.) - (4)(2120lbs.)} = 73 \text{ lbs.}$$

With a proportional limit capacity of about 22 lbs./truss, the 10 webs per diagonal brace configuration would not provide adequate resistance in accordance with Plaut's method.

By trial and error, we could redo the preceding calculation using progressively smaller assumed values for Δ_0. We would find that it takes a Δ_0 less than 0.188 in for this web member to be fully braced with 10 webs per diagonal brace. Inspection of Table 5.1 suggests that more than half of the 2x4's tested in this experiment started out with Δ_0 's greater than that. In other words, this would not represent a conservative design. In summary, there is a good chance that the brace design using the 2% Rule leads to inadequate support for this web. The dramatic difference in the brace force estimate between the 2% Rule and Plaut's method results because the 2% Rule does not consider brace stiffness. This is the same condition illustrated in Figure 6.1.

There is a second problem with use of the 2% Rule in this case. For simplification, we assumed that the nailed connection between the diagonal brace and the web member carries only a lateral load component. In Section 3, we showed that the nailed connection also carries a significant vertical load component. This vertical component results from the connection's resistance to strong-axis bending of the diagonal brace. The diagonal brace illustrated in Figures 3.1 and 3.3 is structurally indeterminate and cannot be analyzed easily, even for strength. In this case, assuming that the connection carries only a horizontal load component would be non-conservative.

6.4.3 Allowable Stress Design (ASD)

Since the brace must be adequate to preserve the assumed ultimate strength of the column, the brace design problem can be addressed easily with an LRFD procedure. Adaptation for allowable stress design (ASD) will require additional study. For example, if an unmodified version of Plaut's method provides sufficient conservatism, the allowable stress equivalent of Equation 22 might look something like this:

$$F_{br-req} = \left(\frac{SF_{column}}{SF_{brace}}\right)\left(\frac{6P_{all}\Delta_0 K}{LK - 4(SF_{column})P_{all}}\right) \tag{24}$$

where: F_{br-req} = required brace strength (pounds)
K = provided brace stiffness (pounds/inch)

L = effective length of the unbraced column (inches)
P_{all} = allowable column load (pounds)
SF_{column} = safety factor for the column (unitless)
SF_{brace} = safety factor for the brace (unitless)
Δ_0 = initial lateral deflection at mid-height (inches)

With the current ASD design standard for wood construction (American Forest and Paper Association, 1997), the safety factor for the web member in compression may range between 1.19 and 1.66 depending upon the assumed effective length. The safety factor for the brace may range from 1.19 to more than 2.2 depending upon whether a wood member or connection governs the brace capacity. Both the column and brace safety factors vary with each design application. In addition, the safety factor of the brace will be time-consuming to determine. Some simplifying assumptions will need to be made concerning the safety factor interactions before Plaut or Winter's methods can be adapted to allowable stress design.

6.5 Recommendations for Future Work

This research suggests that a modified version of either Plaut or Winter's method could be used to provide a rational basis to determine the strength and stiffness requirements of a mid-height lateral brace. Investigation of the following topics would simplify the conversion of this finding into a practical design procedure.

6.5.1 Conservatism

Neither Plaut nor Winter's method perfectly predicts the strength and stiffness requirements of the brace. With some manipulation, the degree of conservatism offered by either can be adjusted to desired levels. Before adopting either procedure as a design requirement, the level of conservatism should be specified by the appropriate code specification organizations.

6.5.2 Initial Deflection, Δ_0

All four brace analysis methods evaluated in this research require an estimate or assumption of the initial lateral deflection of the column at the brace point. This factor is critical to the analysis. If Δ_0 is underestimated, then the computed brace requirements will be insufficient to support the column. If the assumed value is much larger than actual, then the analysis will be overly conservative. The selection of Δ_0 should be made in concert with the decision to prescribe the overall conservatism in the design process.

At present, the actual range of Δ_0 that can be expected by the truss designer is unknown. One approach may be to use the maximum Δ_0 allowed by the grading rule for the truss lumber. However, as shown in Tables 2.1 and 5.1, this would have significantly overestimated the Δ_0 of our test lumber and may not characterize the in-service conditions. Truss manufacture, handling, and installation will influence the in-service value. A study of in-service Δ_0 boundary conditions should be undertaken to codify the design of compression web bracing.

6.5.3 Brace Stiffness Evaluation

Winter's brace design method was developed in the early 1960's. However, most designers still use strength rules like the 2% Rule for brace design (Yura, 1996). A major reason for this probably lies in the difficulty of characterizing the strength <u>and</u> stiffness of the support offered by the lateral brace system. This becomes more difficult when the designer considers the stiffness of the connections in the analysis. As shown in Section 3, the connections dominate the response of the lateral/diagonal brace for this application and cannot be ignored.

Brace strength and stiffness can be estimated using traditional structural engineering techniques. In fact, using finite element analysis, we produced estimates of the brace support supplied by several different brace configurations in Section 3 of this report. Using only the linear elastic contribution of the brace components should simplify this analysis. Even so, the process can become involved and will not be attractive to most building designers. Further research is needed to simplify the process of estimating the strength and stiffness of brace support for various lateral bracing alternatives. A possible end result could be a series of design aids that characterize the strength and stiffness of support provided by typical bracing details.

Section 3.2.2 discusses the possibility that "system effects" may impact the stiffness and strength provided by any bracing system that services multiple columns. While it would be conservative to ignore the benefits provided by a "system effect," they could be considered in a study to characterize the support provided by typical bracing details.

6.5.4 Multiple Braces

Many situations exist in timber engineering where a compression member requires support from multiple lateral braces to reduce the effective length for flexural buckling. The top chords of light-frame wood trusses are a very common example. In instances where sheathing will not be applied directly to the chord, the lateral support offered by purlins or other tributary elements must be considered. This study was limited to a single lateral brace reducing the effective length of a compression member by half. Similar investigation of multiple brace situations should be undertaken to broaden the applicability as a general basis for lateral bracing design.

6.5.5 Brace Instability

As discussed in Sections 5.3 and 6.1, 1% of the test columns developed structural instability when the brace provided non-linear elastic support for the column. To avoid this problem, we conservatively recommended that the brace load remain below the practical proportional limit of the brace. Future research could explore the source of this instability and define its threshold to allow a less conservative limit on the allowable brace load. However, it is the author's opinion that the degree of extra conservatism added by this requirement would be small. In addition, the computational expense of characterizing the non-linear properties of the brace as part of a regular design procedure probably outweigh the potential structural gain.

7.0 SUMMARY AND CONCLUSIONS

The primary objective of this research was to evaluate the ability of several existing, theoretical models to estimate the brace strength and stiffness requirements for discrete compression web bracing. The methods investigated include: Plaut's method (Section 2.2.5), Winter's method (Section 2.2.4), Tsien's equation (Section 2.3.3), and the 2% Rule (Section 2.2.3). Using a practical range of non-linear brace stiffnesses determined by finite element analysis to estimate the brace support provided by a lateral/diagonal bracing system, we tested 774 2x4 Douglas-fir columns to quantify their actual brace strength and stiffness needs. To assess the analysis methods over a reasonable range of compression web stock, the testing incorporated four different column lengths (4, 6, 8, and 10 foot), two different grades ("Standard" and "Select Structural"), and axial load levels equal to the 5% exclusion limit on braced web strength.

Plaut's method, the most conservative of the approaches that considers brace stiffness, was used to select the brace properties for every test. With 2% of the sample columns, the selected brace failed to provide sufficient strength and/or stiffness to fully reduce the column effective length. In about half of those failures, the non-linear brace failed because it did not reach a stable equilibrium position at the maximum test load. The cause for this instability is uncertain, but review of the test data suggests thatit could be minimized by limiting the brace load to the linear range of brace stiffness.

Using "performance variables" defined in Section 5.4.1, we conducted a comparison between the brace support requirements for each test column and those estimated by the four theoretical brace analysis methods. We found little evidence to suggest an effect of lumber grade on the brace support estimation capability of any brace analysis method. We did find a statistically significant length effect on the performance of Plaut's method, Winter's method, and Tsien's equation. The data suggest that test samples could be pooled into two similar populations: 4/6 foot and 8/10 foot columns of both grades. Possible causes for the length effect might include column inelasticity, an observed length effect on flatwise modulus of elasticity, and/or the stiffness of brace support. Table 5.5 and Figure 5.6 summarize performance of each brace analysis method for the test populations. All three theories affected by length proved less conservative for the shorter column lengths, but the measured difference probably has little practical significance.

Study of the performance variables revealed that all four analysis theories performed differently. In order of decreasing conservatism, they rank as follows: 2% Rule, Plaut's method, Tsien's Equation and Winter's method. The 2% rule was conservative about 90% of the time, while Winter's method was conservative only about 50% of the time. Plaut's method showed the lowest residual variability between the predicted and actual brace support requirements. The three remaining theories all showed about equal variability.

The 2% Rule is not recommended as a brace design basis. Use of a strength criteria alone such as the 2% Rule provides no assurance that the brace supplies sufficient stiffness to fully reduce the column effective length. The implied conservatism of these test results for the 2% Rule may be misleading. The test braces were selected to provide sufficient stiffness based on Plaut's design methodology. It is not clear that the 2% Rule would have proved conservative if the test braces were selected based on strength alone. In addition, the accuracy of the 2% Rule depends upon the assumed degree of initial curvature.

Tsien's equation is not recommended as a brace design basis. It proved less accurate on average than Winter's method and was more variable than Plaut's method. As the most computationally intensive procedure, little reason exists to justify its use.

Both Plaut's method and Winters method supply a means for the designer to determine both the strength and stiffness requirements of the brace. Plaut's method is the most conservative of the two and also possesses the lowest residual variability. Winter's method, although more variable, provided the best average prediction of the actual brace needs. In conclusion, with some adjustment to account for safety and variability, either Plaut's method or Winter's method could be used to provide a rational basis for discrete compression web bracing design.

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APPENDICES

APPENDIX A: OUTLINE PROCEDURE FOR CALIBRATION OF 2-D NONLINEAR NAILED JOINTS

As presented in Section 3, this research used a series of finite element models to estimate the stiffness of brace support offered to a compression web by a lateral/diagonal bracing scheme. Figure 3.3 conceptually illustrates the model used for this analysis.

As mentioned in Section 3.3.2, the nailed connections identified as "N_2" in Figure 3.3 experience resultant forces with both X and Y components. A complication arises in that the author used Equation 19 to describe the joint behavior for all side-grain connections of the wood members in the brace assembly. In other words, it was important for the resultant load and displacement of the N_2 connections to adhere to the stiffness described by Equation 19. Providing each of the X and Y component springs with the non-linear properties of Equation 19 results in an overall load/deflection response that is much stiffer than desired. For a given resultant force, the error in the resultant deflection often exceeded 40%.

This problem could be addressed in a number of different ways. The author elected to use a trial and error procedure to determine non-linear X and Y component spring properties that result in the desired resultant response. The following procedure was used to calibrate the component springs of every N_2 nailed joint in all four finite element models.

Step 1: The geometry of each finite element model was determined. In addition to estimating the node coordinates of Figure 3.3, the member properties were assigned. All of the wood members were given the material properties discussed in Section 3. Every spring in the assembly, even the X and Y component springs of the N_2 connections, were initially provided the non-linear stiffness described by Equation 19.

Step 2: A lateral load was applied to the node which represents the compression web at the intersection between the lateral and diagonal brace. This node is identified with the "F_{br}" farthest to the left in Figure 3.3. The load was progressively increased from 0 to 600 lbs in 20 increments. The max load was selected because it just exceeds the capacity of a 2-16d nailed connection. At each load step, ANSYS was used determine the force in each X and Y component spring of every N_2 connection.

Step 3: At each load step, the component spring forces were combined to determine magnitude and direction of the resultant force for each N_2 connection. Equation 19 was used to calculate the resultant displacement that should correspond with that force. The resultant displacement for each N_2 connection was then broken down into X and Y components and assigned to the component springs based on the proportion of the component force to the computed resultant force. In the end, this resulted in a table of forces and displacements to describe the load/deflection response of every X and Y component spring in each N_2 connection.

Step 4: The progressive load of Step 2 was re-applied and ANSYS was used to determine the load and deflection of each component spring at all the applied load increments. The resultant forces and deflections of each N_2 connection were computed. Using Equation 19, the target deflection that corresponded with each resultant force was also computed. If the target and computed resultant deflections differed by more than 5% for any N_2 connection at any load step, then Steps 2, 3, and 4 were repeated. This was usually not necessary.

APPENDIX B: SAMPLE CALCULATION OF COLUMN TEST LOADS

The axial test loads used for this research represent an estimate of the short-term, 5% exclusion limit on braced column strength. Derivation of these loads assumes that the brace assembly fully braces the 2x4 and reduces the weak-axis effective column length by one-half. Table 4.3 summarizes the axial test loads employed for each length and grade lumber. The following sample calculation illustrates test load estimation procedure used for the specific example of a 4', "Standard" grade test column.

Notation:

A	=	member cross-sectional area (in^2)
b	=	member thickness (inches)
c	=	column curve constant (unitless)
C_D	=	load duration factor (unitless)
C_P	=	column stability factor (unitless)
d	=	member depth (inches)
E	=	longitudinal modulus of elasticity of the column (psi)
F_c	=	allowable compression stress parallel to grain (psi)
F^*_c	=	adjusted compression stress multiplied by all applicable design factors except for C_P (psi)
F_{CE}	=	allowable column stress (psi)
K_e	=	effective length factor for compression members (unitless)
L_w	=	actual compression web length (inches)
P_{all}	=	allowable column load (pounds)
P_{cr}	=	critical column load (pounds)

Step 1 – Obtain Published Data:

From the National Design Specification (NDS) Table 4A entry for "Standard" grade "Douglas Fir-Larch" (American Forest and Paper Association, 1997):

$$F_c = 1{,}400 \text{ psi}$$

$$E = 1{,}400{,}000 \text{ psi}$$

Step 2 - Check Slenderness About Each Axis:

Strong Axis: $$\frac{K_e L_w}{d} = \frac{(0.8)(48in.)}{3.5in.} = 10.97$$

Weak Axis: $$\frac{K_e L_w}{b} = \frac{(0.5)(48in.)}{1.5in.} = 16$$

Section 1.2 includes a detailed discussion of the slenderness check and K_b factors. In this case, as with all the 2x4's in this study, the weak axis slenderness of the braced column controls the stability check.

Step 3 – Calculate the Allowable Compressive and Stability Strengths:

$$F*_c = F_c C_D = (1{,}400\,psi)(1.6) = 2{,}240\text{ psi}$$

$$F_{CE} = \frac{0.3E}{(\frac{K_e L_w}{b})^2} = \frac{(0.3)(1{,}400{,}000\,psi)}{16^2} = 1{,}641\text{ psi}$$

Step 4 - Calculate the Critical Column Strength:

Equation 3 provides the basis for calculating the "allowable" column load:

$$P_{all} = C_p F_c^* A = \left[\frac{1 + \frac{F_{cE}}{F_c^*}}{2c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_c^*}}{2c} \right)^2 - \frac{\frac{F_{cE}}{F_c^*}}{c}} \right] F_c^* (bd)$$

As discussed in Section 2.1.4, the allowable stresses F^*_c and F_{CE} include safety factors of 1.19 and 1.66, respectively. Removal of these factors from each provides an approximation of the 5% exclusion limit of stress for both criteria. Using the 5% exclusion limit stresses in Equation 3 in place of the allowable values should estimate the 5% percentile column strength. For this study, this load is equivalent to assumed critical column strength, P_{cr}:

$$P_{cr} = \left[\frac{1 + \frac{1.66F_{cE}}{1.19F_c^*}}{2c} - \sqrt{\left(\frac{1 + \frac{1.66F_{cE}}{1.19F_c^*}}{2c} \right)^2 - \frac{\frac{1.66F_{cE}}{1.19F_c^*}}{c}} \right] (1.19F_c^*)(bd)$$

$$P_{cr} = \left[\frac{1 + \frac{1.66(1{,}641psi)}{1.19(2{,}240psi)}}{2(0.8)} - \sqrt{\left(\frac{1 + \frac{1.66(1{,}641psi)}{1.19(2{,}240psi)}}{2(0.8)} \right)^2 - \frac{\frac{1.66(1{,}641psi)}{1.19(2{,}240psi)}}{0.8}} \right]$$
$$* (1.19)(2{,}240psi)(5.25in.^2)$$

$$\underline{P_{cr} = 9{,}775 \text{ lbs.}}$$

As indicated in Table 4.3, the testing incorporated a test load of 9,800 lbs. for this length and grade.

APPENDIX C: KEY TEST APPARATUS COMPONENTS

Part	Function/Specifications
Axial load cell	-Function: uses an electronic strain gauge bridge to monitor the axial load -Connections: voltage input from load cell amplifier/power supply, voltage output to load cell amplifier/power supply -Manufacturer: Sensotec -Model 41/573-02 -Properties: 20,000 lb. capacity, 10V DC
Brace load cell	-Function: uses an electronic strain gauge bridge to monitor the axial load -Connections: voltage input from load cell amplifier/power supply, voltage output to load cell amplifier/power supply -Manufacturer: Sensotec -Model 41/572-05 -Properties: 5,000 lb. capacity, 10V DC
Computer workstation:	-Function: Operates the test machine, electronically acquires all data -Software: Custom VI written by Miles E. Waltz, Jr in LabView (National Instruments, 1996) -Platform: Windows NT 5.0 -Machine interface: data acquisition card -Manufacturer: Dell -Model: Dimension XPS P100C -Special: Marked "FRL_NT3"
Data acquisition card	-Function: computer card to interface with Labview software, inputs data from load cells and LVDT in voltage form, sends voltage commands to the electric cylinder control box and hydraulic controller card through the screw terminal -Connections: interfaces with computer, all input and output voltages travel via cable to the screw terminal -Manufacturer: National Instruments -Model: AT-MIO-16XE-10
Electric cylinder	-Function: manipulates the lever arm to control lateral column deflections -Connections: input voltage from electric cylinder control, feedback voltage to electric cylinder control, power from electric cylinder control -Manufacturer: Industrial Devices Corporation -Model: ND 1205A-6-MS6-MT1-L
Electric cylinder control	-Function: control card within the box interfaces between the computer and electric cylinder, computer sends voltages to the card that it uses to find stroke positions of the cylinder, also provides power supply for the cylinder -Connections: input voltage from screw terminal, output voltage to electric cylinder, feedback voltage from electric cylinder -Manufacturer: Industrial Devices Corporation -Model: D series electric cylinder control No. D2502 B -Special: modified by Milo Clauson for 0 Volt input to correspond with half extension of the cylinder.

Part	Function/Specifications
Heat exchanger	-Function: works in-line with the hydraulic pump to cool the hydraulic fluid -Connections: hydraulic input from pump, hydraulic output to oil reservoir, water in from hose, water out to drain -Manufacturer: Thermal Transfer Products Limited -Model: EK-508-0 -Properties: shell capacity 500 psi, core capacity 150 psi
Hydraulic controller card	-Function: works to equalize (opposite sign) the voltage output from the axial load cell and data acquisition card to regulate axial load via the hydraulic valve -Connections: Output voltage to hydraulic valve, input voltage from load cell amplifier/power supply (axial), input voltage from screw terminal -Manufacturer: Continental Hydraulics -Model: ECM5-L2-P1-24C-A
Hydraulic cylinder	-Function: applies axial load to the column -Manufacturer: Miller -Connections: hydraulic input from hydraulic valve, hydraulic output to hydraulic valve -Model: H66 -Properties: 8 in. bore, 8 in. stroke, 5,000 psi capacity
Hydraulic pump	-Function: provides hydraulic pressure to power the axial load cylinder -Connections: hydraulic output to heat exchanger, hydraulic output to hydraulic valve, hydraulic input from heat exchanger, hydraulic input from hydraulic valve -Manufacturer: Continental Hydraulics -Model: ID# 9205752 692, PVR6-4B20-RF-0-612 -Properties: 10 gallon reservoir, 3,000 psi
Hydraulic valve	-Function: controlled by hydraulic controller card to regulate the volume and direction of fluid flow to the hydraulic cylinder -Connections: voltage connection to hydraulic controller card, hydraulic input from hydraulic pump, hydraulic input from hydraulic cylinder, hydraulic output to hydraulic cylinder -Manufacturer: Continental Hydraulics -Model: ID# ED03H-3AGC-6D-24L-B -Capacity: 1,500 psi
Laser line	-Function: provide overhead reference for Δ_0 measurement -Manufacturer: Lasermate -Connections: power from laser power supply -Model: Line generating optic LO-60, diode module LTG-6 357-AH
Laser power supply	-Function: powers the laser line -Connections: power output to laser diode -Manufacturer: Power One -Model: International Series HC5-6/OVD-A
Load cell amplifier/power	-Function: provides continuous power to the load cells, amplifies load cell output to the data acquisition card -Connections: Load cell voltage out to axial and brace load cells, voltages in from axial and brace load cells, amplified voltages from both out to the screw terminals, additional amplified voltage from axial load cell to hydraulic controller card -Manufacturer: FRL staff, marked "HRH/April 86"

Part	Function/Specifications
LVDT	-Function: linear variable differential transformer to measure lateral column deflections at mid-height. -Connections: input voltage from LVDT amplifier/power supply, output voltage to LVDT amplifier/power supply -Manufacturer: Schaevitz -Model: HCA 500
LVDT Amplifier/Power Supply	-Function: supplies power to the LVDT and amplifies output for the data acquisition card -Connections: output voltage to LVDT, input voltage from LVDT, output voltage to screw terminals -Manufacturer: Schaevitz -Model: ATA-101 Analog Transducer Amplifier
Screw Terminals	-Function: provides screw/wire terminal interface for the data acquisition card -Manufacturer: National Instruments -Connections: output voltage to hydraulic controller card, output voltage to electric cylinder control, input voltage from axial load cell amplifier/power supply (2 load cells), input voltage from LVDT amplifier/power supply, interfaces with data acquisition card -Model: SCB-68
Test Frame:	-Function: provides the structural support for all test functions -Manufacturer: in-house by FRL staff -Designer: Miles E. Waltz, Jr. -Specifications: see illustrations in Chapter 4 for more detail

APPENDIX D: SAMPLE SELECTION OF A BRACE TEST CURVE

The test equipment was designed to ensure that adequate and representative brace support was applied to each test sample. The test controller used a non-linear brace support curve to characterize the properties of the mid-height brace. As discussed in Section 4, we selected the brace support curve for each test based on the column's measured properties. Plaut's method of estimating the brace strength and stiffness requirements served as the basis for this selection. We used Plaut's method to choose a brace support curve from the estimates developed using finite element analysis in Section 3. These estimates approximate the stiffness of support offered to the column by a lateral/diagonal bracing system.

The following sample calculation illustrates the brace curve selection procedure for an actual test column: Sample SF44-10.

Notation:

E = longitudinal modulus of elasticity of the column (psi)
F_{br} = brace force (pounds)
I = weak axis moment of inertia (in^4)
L = effective length of the unbraced column (inches)
P = column load (pounds)
P_e = Euler buckling load for the unbraced column (pounds)
Δ = lateral deflection at mid-height upon application of an axial load (inches)
Δ_0 = initial lateral deflection at mid-height (inches)

Step 1 – Measurements and Test Parameters:

Given that sample SF44-10 is a 10' "Select Structural" column that will be tested with a weak axis effective length of L/2, we know that:

$$L = 120\,\text{in.}$$

$$P = 3{,}000 \text{ lbs. (Table 4.3)}$$

Based on preliminary measurements for the column:

$$E = 2{,}260{,}000 \text{ psi}$$
$$I = 0.967 \text{ in}^4$$
$$\Delta_0 = 0.313 \text{ in.}$$

Step 2 – Determine the Column Brace Requirements:

Using Equation 15, we compute the theoretical brace requirements according to Plaut's method:

$$F_{br} = \frac{4P}{L}(\Delta + 1.5\Delta_0) = \frac{4(3{,}000)}{120}[\Delta + (1.5)(0.313)] \text{ lbs.}$$
$$\underline{F_{br} = 100\Delta + 47.0 \text{ lbs}}$$

Equation 11 provides the same type of estimate for Winter's method:

$$F_{br} = \frac{4P}{L}(\Delta + \Delta_0) = \frac{4(3{,}000)}{120}(\Delta + 0.313) \text{ lbs.}$$
$$\underline{F_{br} = 100\Delta + 31.3 \text{ lbs}}$$

Equation 18 summarizes Tsien's requirements:

$$\Delta = \Delta_0 \left[\frac{\frac{P}{P_e}}{1 - \frac{P}{P_e}} \right] + \left[\frac{\frac{1}{4} - \frac{1}{2\left(\pi\sqrt{\frac{P}{P_e}}\right)} \tan\left(\frac{\pi\sqrt{\frac{P}{P_e}}}{2} \right)}{\frac{P}{P_e}} \right] \frac{F_{br} L}{P_e}$$

where:

$$P_e = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 (2{,}260{,}000)(0.967)}{120^2} = 1{,}498 \text{ lbs.}$$

therefore:

$$\Delta = 0.313 \left[\frac{\frac{3{,}000}{1{,}498}}{1 - \frac{3{,}000}{1{,}498}} \right] + \left[\frac{\frac{1}{4} - \frac{1}{2\left(\pi\sqrt{\frac{3{,}000}{1{,}498}}\right)} \tan\left(\frac{\pi\sqrt{\frac{3{,}000}{1{,}498}}}{2} \right)}{\frac{3{,}000}{1{,}498}} \right] \frac{F_{br}(120)}{(1{,}498)}$$

$$\Delta = -.626 + 0.016 F_{br}$$

which simplifies to:

$$\underline{F_{br} = 62.8\Delta + 39.1 \text{ lbs.}}$$

The brace strength requirement of the 2% rule is calculated using Equation 9:

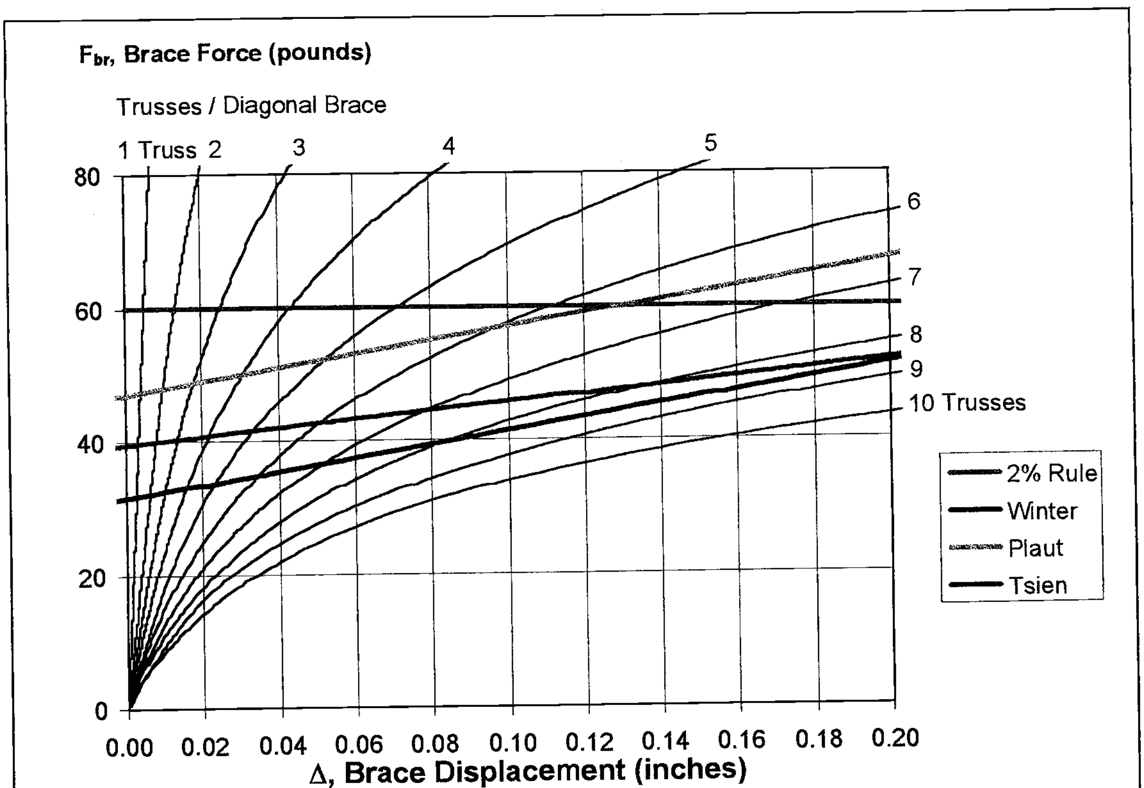
$$F_{br} = .02P = (0.02)(3{,}000)$$

$$\underline{F_{br} = 60 \text{ lbs.}}$$

Step 3 – Select a Brace Curve for the Test

Figure 3.8 graphically summarizes the strength and stiffness of support that a lateral/diagonal bracing system might supply to a 10' compression web member. The various curves in Figure 3.8 correspond to the spacing of the diagonal brace. For example, the "10 trusses" curve approximates the support provided by the brace system when the lateral brace ties together 10 webs for each diagonal brace. Section 3 summarizes the derivation of these curves.

Figure A.1 is a reproduction of Figure 3.8 with adjusted vertical scale. The four theoretical brace analysis relationships calculated in Step 2 are superimposed.



Brace Selection Graph for Column SF44-10

Any brace support curve that intersects with a theoretical brace analysis curve should provide sufficient support according to that theory. Brace support curves that do not intersect with a theoretical brace analysis theory curve may not support the column for the axial test load. For example, Plaut's method predicts that the lateral/diagonal bracing system should adequately support this 2x4 when there are between 1 and 6 similar webs for every diagonal brace. The least stiff curve that provides a solution for Plaut's method was selected for the test: in this instance, the curve that represents 6 compression webs per diagonal brace.

Strictly speaking, not all of the brace load/deflection curves used in the testing satisfied this criteria. In a minority of instances, the curve forPlaut's theory just missed the selected brace curve. The curve was used anyway as the best practical solution.

APPENDIX E: DETAILED LISTING OF ALL TEST DATA

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_o$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	D	2	4	3.489	1.488	0.954	4.27	1.03	1	12.7	1.66E+06	02-Jun-98	2	2	0.125	13,000	1	2	1	89	0.005		209	141	173	260
S	D	5	4	3.489	1.490	0.962	3.78	0.91	1	13.0	1.40E+06	02-Jun-98	1	1	0.125	13,000	1	2	1	205	0.015		219	152	183	260
S	D	7	4	3.472	1.475	0.928	3.72	0.92	1	10.8	1.42E+06	02-Jun-98	1	1	0.109	13,000	1	2	1	141	0.008		186	127	158	260
S	D	13	4	3.476	1.478	0.935	4.20	1.02	1	12.6	1.70E+06	02-Jun-98	1	1	0.047	13,000	2	2	1	105	0.020		98	72	77	260
S	D	15	4	3.438	1.476	0.921	3.79	0.92	1	12.2	1.68E+06	02-Jun-98	1	1	0.141	13,000	1	2	1	240	0.020		250	174	205	260
S	D	26	4	3.487	1.461	0.944	3.83	0.93	1	12.8	1.17E+06	02-Jun-98	1	1	0.094	13,000	1	2	1	126	0.007		160	109	136	260
S	D	31	4	3.482	1.471	0.924	3.53	0.87	1	11.4	1.22E+06	02-Jun-98	1	1	0.141	13,000	1	2	1	284	0.029		260	184	220	260
S	D	33	4	3.511	1.489	0.966	3.81	0.92	1	12.9	1.13E+06	02-Jun-98	1	1	0.094	13,000	1	2	1	98	0.005		156	107	134	260
S	D	40	4	3.500	1.489	0.963	3.69	0.91	1	11.1	1.37E+06	02-Jun-98	2	2	0.063	13,000	2	2	1	17	0.002		104	70	87	260
S	D	42	4	3.514	1.463	0.955	4.07	0.98	1	13.1	1.37E+06	02-Jun-98	1	2	0.125	13,000	1	2	1	193	0.013		217	150	182	260
S	D	44	4	3.435	1.467	0.904	3.71	0.92	1	10.5	1.47E+06	02-Jun-98	1	1	0.063	13,000	2	2	1	70	0.010		112	79	93	260
S	D	49	4	3.443	1.477	0.924	3.63	0.88	1	10.8	1.58E+06	02-Jun-98	1	1	0.078	13,000	1	2	1	168	0.012		140	98	115	260
S	D	52	4	3.478	1.483	0.945	3.29	0.79	1	12.8	1.24E+06	02-Jun-98	1	1	0.031	13,000	2	2	1	32	0.004		55	38	47	260
S	D	57	4	3.474	1.456	0.894	4.41	1.09	1	9.9	1.86E+06	02-Jun-98	1	1	0.168	13,000	1	2	1	271	0.026		333	231	270	260
S	D	63	4	3.442	1.463	0.936	4.29	1.05	1	11.7	1.78E+06	02-Jun-98	2	2	0.047	13,000	1	2	1	63	0.003		79	54	65	260
S	D	73	4	3.477	1.468	0.955	4.03	0.97	1	13.5	1.64E+06	02-Jun-98	2	2	0.047	13,000	2	2	1	37	0.004		81	55	66	260
S	D	74	4	3.471	1.483	0.943	3.51	0.88	1	11.4	1.28E+06	02-Jun-98	1	1	0.109	13,000	1	2	1	92	0.005		183	124	155	260
S	D	85	4	3.474	1.464	0.946	4.08	1.00	1	11.7	1.78E+06	02-Jun-98	1	1	0.078	13,000	1	2	1	125	0.007		135	92	110	260
S	D	91	4	3.442	1.464	0.937	4.06	0.98	1	12.7	1.56E+06	02-Jun-98	1	1	0.094	13,000	1	2	1	63	0.003		156	105	130	260
S	D	108	4	3.465	1.475	0.932	4.04	0.97	1	13.8	1.21E+06	02-Jun-98	2	2	0.031	13,000	2	2	1	33	0.004		55	38	47	260
S	D	113	4	3.431	1.464	0.934	3.60	0.87	1	13.2	1.60E+06	02-Jun-98	1	1	0.063	13,000	2	2	1	45	0.006		108	74	89	260
S	D	117	4	3.403	1.466	0.931	4.18	1.02	1	11.9	1.91E+06	02-Jun-98	1	1	0.063	13,000	1	2	1	69	0.003		105	71	88	260
S	D	125	4	3.428	1.465	0.898	4.10	0.99	1	13.2	1.37E+06	02-Jun-98	1	1	0.094	13,000	1	2	1	83	0.005		156	107	133	260
S	D	129	4	3.434	1.472	0.913	3.89	0.94	1	12.3	1.18E+06	02-Jun-98	1	1	0.188	13,000	1	2	1	386	0.063		373	271	317	260
S	D	137	4	3.368	1.478	0.906	3.17	0.79	1	10.2	1.14E+06	02-Jun-98	1	1	0.031	13,000	2	2	1	61	0.009		61	44	52	260
S	D	138	4	3.431	1.493	0.952	3.33	0.83	1	9.9	1.22E+06	02-Jun-98	1	1	0.063	13,000	2	2	1	73	0.011		113	80	98	260
S	D	141	4	3.465	1.495	0.985	4.89	1.17	1	14.0	1.30E+06	02-Jun-98	1	2	0.109	13,000	1	2	1	110	0.006		184	125	156	260
S	D	149	4	3.464	1.483	0.941	3.61	0.87	1	12.7	1.04E+06	02-Jun-98	1	1	0.203	13,000	1	2	1	310	0.035		368	258	317	260
S	F	2	4	3.493	1.493	0.969	4.20	1.02	1	12.5	1.70E+06	03-Jun-98	1	1	0.250	13,000	1	2	1	473	0.118		534	389	413	260
S	F	3	4	3.494	1.498	0.975	4.12	1.02	1	10.1	1.73E+06	03-Jun-98	1	1	0.031	13,000	2	2	1	96	0.017		69	52	53	260
S	F	6	4	3.474	1.494	0.965	4.68	1.14	1	12.0	2.09E+06	03-Jun-98	1	2	0.047	13,000	2	2	1	100	0.018		98	70	71	260
S	F	8	4	3.497	1.498	0.980	4.24	1.04	1	11.6	1.74E+06	03-Jun-98	2	2	0.033	13,000	2	2	1	38	0.005		60	42	46	260
S	F	9	4	3.483	1.489	0.958	4.13	1.02	1	10.0	1.58E+06	04-Jun-98	1	2	0.125	13,000	1	2	1	34	0.002		205	138	172	260
S	F	20	4	3.478	1.493	0.965	4.88	1.21	1	10.3	1.57E+06	03-Jun-98	1	1	0.125	13,000	2	2	1	125	0.028		233	166	189	260
S	F	26	4	3.483	1.495	0.970	3.48	0.87	1	9.4	1.43E+06	03-Jun-98	1	1	0.031	13,000	2	2	1	108	0.021		74	57	56	260
S	F	27	4	3.474	1.493	0.963	4.42	1.10	1	10.0	2.00E+06	03-Jun-98	1	1	0.047	13,000	2	2	1	50	0.007		84	58	67	260
S	F	28	4	3.465	1.495	0.970	4.93	1.22	1	10.6	1.76E+06	03-Jun-98	1	1	0.063	13,000	2	2	1	54	0.007		109	75	89	260
S	F	30	4	3.490	1.492	0.966	3.71	0.91	1	11.3	1.44E+06	03-Jun-98	1	1	0.094	13,000	1	2	1	176	0.012		165	115	137	260
S	F	34	4	3.471	1.494	0.965	4.59	1.13	1	10.4	1.35E+06	04-Jun-98	2	2	0.016	13,000	3	2	1	11	0.002		28	19	23	260
S	F	38	4	3.495	1.491	0.965	3.79	0.93	1	10.7	1.42E+06	04-Jun-98	1	2	0.031	13,000	2	2	1	130	0.030		83	66	65	260
S	F	39	4	3.478	1.490	0.959	4.33	1.08	1	9.8	1.67E+06	03-Jun-98	1	1	0.125	13,000	1	2	1	148	0.009		213	145	176	260
S	F	40	4	3.453	1.489	0.950	5.01	1.22	1	12.1	2.26E+06	02-Jun-98	1	1	0.141	13,000	1	2	1	57	0.003		232	156	190	260
S	F	41	4	3.465	1.485	0.951	3.84	0.96	1	9.4	1.65E+06	02-Jun-98	1	1	0.031	13,000	2	2	1	32	0.004		55	38	45	260
S	F	45	4	3.465	1.495	0.970	4.30	1.05	1	11.9	1.93E+06	04-Jun-98	2	2	0.018	13,000	3	1	4	164	0.161	Brace unstable at proof load, slow fall	200	191	104	260
S	F	50	4	3.475	1.469	0.958	4.45	1.10	1	10.7	1.83E+06	03-Jun-98	1	1	0.047	13,000	2	2	1	75	0.012		89	64	70	260
S	F	52	4	3.457	1.462	0.938	4.97	1.20	1	13.2	2.49E+06	02-Jun-98	1	1	0.063	13,000	2	2	1	54	0.008		110	76	86	260
S	F	53	4	3.463	1.494	0.968	4.41	1.08	1	11.9	1.97E+06	03-Jun-98	2	2	0.047	13,000	2	2	1	10	0.001		77	52	64	260
S	F	58	4	3.498	1.486	0.957	3.34	0.83	1	10.3	1.27E+06	03-Jun-98	1	2	0.094	13,000	1	2	1	145	0.009		162	111	137	260
S	F	61	4	3.485	1.492	0.965	3.84	0.94	1	12.0	1.52E+06	03-Jun-98	1	1	0.063	13,000	2	2	1	59	0.009		111	77	92	260
S	F	63	4	3.479	1.495	0.969	4.77	1.18	1	10.7	2.15E+06	04-Jun-98	1	1	0.109	13,000	1	2	1	91	0.005		183	124	149	260
S	F	70	4	3.491	1.466	0.955	3.78	0.93	1	10.9	1.48E+06	03-Jun-98	2	2	0.016	13,000	3	2	1	68	0.032		60	52	45	260
S	F	77	4	3.482	1.466	0.952	3.60	0.94	1	9.8	1.57E+06	03-Jun-98	1	1	0.063	13,000	2	1	4	247	0.152	Brace failure	268	232	190	260

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	F	81	4	3.474	1.484	0.946	4.46	1.09	1	11.6	1.84E+06	03-Jun-98	1	1	0.078	13,000	1	2	1	77	0.004		131	89	109	280
S	F	85	4	3.442	1.485	0.938	5.22	1.25	1	14.0	2.49E+06	03-Jun-98	2	2	0.031	13,000	2	2	1	26	0.003		54	37	43	280
S	F	97	4	3.492	1.494	0.970	3.63	0.95	1	9.9	1.49E+06	02-Jun-98	1	1	0.156	13,000	1	2	1	250	0.021		277	192	228	280
S	F	98	4	3.489	1.493	0.968	3.67	0.96	1	10.3	1.53E+06	03-Jun-98	1	1	0.125	13,000	1	2	1	236	0.019		224	156	184	280
S	F	102	4	3.474	1.488	0.954	4.05	1.01	1	9.4	1.75E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	140	0.009		137	94	111	280
S	F	103	4	3.460	1.485	0.950	4.18	1.04	1	10.2	1.85E+06	02-Jun-98	1	1	0.078	13,000	1	2	1	76	0.004		131	89	108	280
S	F	105	4	3.485	1.489	0.959	4.45	1.10	1	10.5	1.98E+06	04-Jun-98	2	2	0.031	13,000	2	2	1	2	0.000		51	34	42	280
S	F	110	4	3.461	1.487	0.948	5.36	1.32	1	11.1	2.42E+06	03-Jun-98	1	1	0.047	13,000	2	2	1	90	0.015		92	67	68	280
S	F	118	4	3.466	1.489	0.954	4.53	1.10	1	12.2	1.91E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	54	0.003		130	88	107	280
S	F	119	4	3.493	1.492	0.967	3.98	0.97	1	11.4	1.64E+06	02-Jun-98	1	1	0.109	13,000	1	2	1	140	0.006		186	127	153	280
S	F	121	4	3.465	1.484	0.944	4.63	1.19	1	10.5	2.06E+06	02-Jun-98	1	1	0.125	13,000	1	2	1	99	0.005		209	141	171	280
S	F	132	4	3.480	1.491	0.961	4.26	1.06	1	9.4	2.04E+06	03-Jun-98	1	1	0.047	13,000	2	2	1	24	0.003		79	54	65	280
S	F	133	4	3.466	1.487	0.950	3.93	0.97	1	10.8	1.78E+06	04-Jun-98	1	2	0.063	13,000	2	2	1	106	0.021		124	90	97	280
S	F	136	4	3.475	1.487	0.952	4.75	1.16	1	12.1	1.54E+06	04-Jun-98	2	2	0.031	13,000	2	2	1	7	0.001		52	35	43	280
S	F	145	4	3.474	1.489	0.956	4.64	1.18	1	11.6	2.28E+06	03-Jun-98	1	1	0.094	13,000	1	2	1	192	0.013		166	116	131	280
S	F	148	4	3.495	1.492	0.967	3.72	0.91	1	11.0	1.42E+06	03-Jun-98	1	1	0.063	13,000	2	2	1	183	0.066		173	139	135	280
S	F	153	4	3.481	1.496	0.971	4.25	1.03	1	12.0	1.85E+06	03-Jun-98	2	2	0.047	13,000	2	2	1	186	0.051		131	106	91	280
S	M	3	4	3.460	1.432	0.847	4.46	1.07	1	13.3	1.75E+06	08-Jun-98	1	1	0.063	13,000	2	2	1	39	0.005		107	73	89	280
S	M	17	4	3.424	1.483	0.931	3.76	0.93	1	10.5	1.51E+06	04-Jun-94	1	1	0.250	13,000	1	1	4	499	0.140	Brace unstable at proof load, slow fail	558	423	445	280
S	M	18	4	3.418	1.455	0.877	3.92	0.97	1	10.3	1.77E+06	04-Jun-94	1	1	0.031	13,000	2	2	1	56	0.008		59	43	48	280
S	M	22	4	3.461	1.490	0.954	3.46	0.85	1	11.2	1.52E+06	04-Jun-98	2	2	0.016	13,000	3	1	4	164	0.152	Brace unstable at proof load, slow fail	190	182	130	280
S	M	24	4	3.415	1.460	0.923	3.32	0.81	1	11.1	7.68E+05	04-Jun-98	1	1	0.203	11,100	1	1	2	309	0.038	Column failure, s-bending	317	223	280	222
S	M	25	4	3.433	1.459	0.889	4.49	1.09	1	12.6	1.81E+06	04-Jun-98	1	1	0.094	13,000	1	2	1	94	0.005		158	107	131	280
S	M	29	4	3.430	1.450	0.871	3.98	0.98	1	10.8	1.48E+06	04-Jun-98	1	1	0.047	13,000	1	2	1	118	0.006		83	57	69	280
S	M	38	4	3.377	1.441	0.842	4.06	1.01	1	10.6	1.82E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	61	0.009		111	77	91	280
S	M	39	4	3.481	1.477	0.935	3.46	0.85	1	11.2	1.06E+06	04-Jun-98	1	2	0.063	11,610	2	1	3	250	0.147	Brace failure first, then column failure in s-bending	233	203	192	232
S	M	47	4	3.418	1.461	0.888	4.81	1.18	1	13.3	2.38E+06	04-Jun-98	1	2	0.063	13,000	2	2	1	44	0.006		108	74	87	280
S	M	51	4	3.466	1.469	0.916	5.26	1.28	1	11.8	2.05E+06	04-Jun-98	1	1	0.094	13,000	1	2	1	121	0.007		160	109	130	280
S	M	55	4	3.492	1.473	0.930	3.81	0.94	1	13.0	1.46E+06	04-Jun-98	1	1	0.047	13,000	2	2	1	101	0.019		97	71	78	280
S	M	58	4	3.452	1.485	0.942	4.06	0.99	1	12.1	1.33E+06	04-Jun-98	1	1	0.109	13,000	1	2	1	297	0.032		212	153	176	280
S	M	59	4	3.455	1.464	0.903	4.61	1.13	1	11.6	1.91E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	106	0.008		133	91	109	280
S	M	64	4	3.472	1.478	0.934	3.65	0.89	1	12.2	1.20E+06	04-Jun-98	1	1	0.172	13,000	1	2	1	312	0.038		318	225	269	280
S	M	66	4	3.480	1.488	0.952	2.97	0.74	1	9.7	1.01E+06	04-Jun-98	1	1	0.047	13,000	2	2	1	110	0.021		99	74	86	280
S	M	69	4	3.387	1.454	0.868	4.06	1.00	1	10.4	1.99E+06	04-Jun-98	1	1	0.047	13,000	2	2	1	42	0.006		83	57	67	280
S	M	72	4	3.497	1.490	0.964	4.05	0.98	1	12.3	1.40E+06	04-Jun-98	1	1	0.031	13,000	2	2	1	52	0.007		58	41	48	280
S	M	73	4	3.465	1.453	0.886	3.51	0.86	1	11.3	1.49E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	31	0.004		106	72	89	280
S	M	78	4	3.451	1.467	0.906	4.12	0.99	1	13.2	1.65E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	168	0.053		159	125	122	280
S	M	79	4	3.467	1.474	0.931	3.20	0.79	1	10.6	1.15E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	38	0.005		107	73	91	280
S	M	83	4	3.446	1.439	0.856	4.84	1.12	1	13.0	2.02E+06	04-Jun-98	1	1	0.047	13,000	2	2	1	122	0.026		104	79	79	280
S	M	84	4	3.441	1.482	0.933	4.54	1.09	1	13.2	1.88E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	150	0.009		137	94	111	280
S	M	86	4	3.402	1.465	0.881	4.00	0.98	1	11.1	1.80E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	78	0.012		115	81	93	280
S	M	87	4	3.453	1.469	0.912	3.40	0.83	1	11.5	1.55E+06	04-Jun-98	1	1	0.094	13,000	1	2	1	158	0.010		163	112	135	280
S	M	100	4	3.429	1.438	0.850	5.23	1.25	1	14.3	2.85E+06	04-Jun-98	1	1	0.125	13,000	1	2	1	159	0.010		214	146	170	280
S	M	101	4	3.498	1.481	0.947	3.28	0.80	1	11.5	1.11E+06	04-Jun-98	1	2	0.125	13,000	1	2	1	132	0.008		212	144	181	280
S	M	103	4	3.496	1.484	0.953	3.76	0.92	1	11.2	1.38E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	148	0.009		137	94	114	280
S	M	106	4	3.434	1.456	0.883	4.39	1.06	1	13.2	2.15E+06	04-Jun-98	1	1	0.063	13,000	2	2	1	33	0.004		106	72	88	280
S	M	114	4	3.508	1.494	0.975	4.21	1.02	1	12.3	1.65E+06	04-Jun-98	1	1	0.125	13,000	1	2	1	287	0.030		236	168	189	280
S	M	118	4	3.477	1.470	0.920	3.71	0.89	1	13.4	1.33E+06	04-Jun-98	1	1	0.047	13,000	2	2	1	84	0.014		91	66	78	280
S	M	125	4	3.392	1.444	0.851	4.47	1.06	1	13.0	2.30E+06	04-Jun-98	1	1	0.172	13,000	1	2	1	249	0.021		302	209	242	280
S	M	128	4	3.465	1.490	0.955	3.75	0.92	1	11.0	1.23E+06	04-Jun-98	1	1	0.016	13,000	3	2	1	58	0.015		42	33	34	280
S	M	129	4	3.484	1.470	0.922	2.98	0.74	1	10.2	1.11E+06	04-Jun-98	1	1	0.047	13,000	1	2	1	15	0.001		77	52	66	280
S	M	135	4	3.404	1.470	0.901	3.81	0.94	1	10.4	1.61E+06	04-Jun-98	2	2	0.031	13,000	2	2	1	58	0.006		59	43	48	280

Measured Test Data Sheet;

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :												Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta₀ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	M	137	4	3.492	1.480	0.943	3.58	0.87	1	11.6	1.03E+06	04-Jun-98	1	1	0.078	13,000	1	2	1	149	0.009		137	94	118	260
T	D	2	4	3.431	1.486	0.938	5.21	1.22	1	16.1	2.18E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	46	0.011		66	47	48	196
T	D	5	4	3.423	1.456	0.880	5.27	1.24	2	15.4	2.04E+06	01-Jun-98	1	2	0.047	9,800	3	2	1	37	0.006		64	45	49	196
T	D	7	4	3.405	1.466	0.935	5.59	1.30	2	17.3	2.18E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	52	0.007		82	57	64	196
T	D	8	4	3.437	1.494	0.956	4.71	1.15	2	12.0	2.11E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	23	0.003		79	53	63	196
T	D	12	4	3.504	1.497	0.980	3.67	0.90	2	11.0	1.36E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	108	0.021		132	94	105	196
T	D	16	4	3.521	1.486	0.963	4.08	0.99	1	11.9	1.21E+06	01-Jun-98	1	2	0.172	9,800	1	2	1	176	0.011		220	149	182	196
T	D	20	4	3.457	1.504	0.980	4.99	1.22	2	11.5	2.30E+06	01-Jun-98	1	1	0.047	9,800	4	2	1	8	0.002		59	40	47	196
T	D	21	4	3.523	1.498	0.987	3.81	0.93	2	11.4	1.42E+06	01-Jun-98	1	1	0.063	9,800	3	2	1	47	0.011		86	60	68	196
T	D	25	4	3.498	1.510	1.004	4.22	1.04	1	10.6	1.54E+06	01-Jun-98	1	1	0.031	9,800	3	2	1	43	0.010		46	34	35	196
T	D	33	4	3.523	1.501	0.993	3.60	0.88	2	11.7	1.16E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	60	0.009		84	58	69	196
T	D	37	4	3.523	1.505	1.001	4.06	0.99	1	11.6	7.97E+05	01-Jun-98	1	1	0.125	9,800	1	2	1	121	0.007		159	108	136	196
T	D	68	4	3.475	1.504	0.965	4.31	1.06	2	10.6	1.80E+06	01-Jun-98	2	2	0.063	9,800	2	2	1	34	0.004		80	54	64	196
T	D	69	4	3.503	1.505	0.995	4.42	1.07	2	13.1	2.19E+06	01-Jun-98	1	2	0.047	9,800	3	2	1	29	0.006		82	43	48	196
T	D	78	4	3.493	1.494	0.971	4.65	1.12	1	13.3	1.96E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	77	0.024		77	56	51	196
T	D	77	4	3.518	1.500	0.989	3.99	0.97	2	12.2	1.60E+06	01-Jun-98	1	1	0.031	9,800	3	2	1	87	0.031		64	51	41	196
T	D	81	4	3.472	1.467	0.913	3.68	0.90	2	11.7	1.53E+06	01-Jun-98	1	1	0.078	9,800	2	2	1	25	0.003		96	66	81	196
T	D	82	4	3.475	1.466	0.954	4.05	0.99	2	11.1	1.41E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	27	0.006		62	43	50	196
T	D	86	4	3.443	1.485	0.940	4.56	1.11	2	11.5	1.92E+06	01-Jun-98	1	1	0.016	9,800	4	2	1	21	0.006		24	18	17	196
T	D	88	4	3.521	1.504	0.998	3.63	0.89	2	11.1	1.40E+06	01-Jun-98	1	1	0.109	9,800	2	2	1	126	0.028		157	112	122	196
T	D	92	4	3.477	1.497	0.972	4.30	1.04	2	13.3	1.18E+06	01-Jun-98	1	1	0.125	9,800	1	2	1	146	0.009		180	109	133	196
T	D	97	4	3.475	1.480	0.939	3.68	0.91	2	9.8	1.42E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	6	0.001		77	52	64	196
T	D	100	4	3.498	1.494	0.972	3.17	0.77	2	11.9	1.06E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	73	0.012		125	86	103	196
T	D	103	4	3.494	1.478	0.936	4.07	0.99	1	12.1	1.37E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	66	0.010		85	59	68	196
T	D	111	4	3.480	1.491	0.961	4.28	1.03	2	13.0	1.90E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	44	0.010		66	46	49	196
T	D	112	4	3.458	1.493	0.959	3.79	0.92	1	12.3	1.09E+06	01-Jun-98	1	1	0.016	9,800	4	2	1	58	0.025		40	33	30	196
T	D	116	4	3.459	1.490	0.954	3.67	0.90	2	11.2	1.54E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	40	0.009		65	46	51	196
T	D	118	4	3.410	1.471	0.905	3.98	0.97	1	11.4	1.47E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	42	0.006		120	81	98	196
T	D	121	4	3.447	1.453	0.881	3.68	0.94	2	12.5	1.78E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	38	0.009		65	46	50	196
T	D	123	4	3.459	1.463	0.903	3.92	0.95	2	12.0	1.70E+06	01-Jun-98	1	1	0.125	9,800	1	2	1	106	0.006		158	107	129	196
T	D	125	4	3.474	1.478	0.935	3.68	0.90	2	11.0	1.47E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	139	0.035		143	105	110	196
T	D	127	4	3.458	1.473	0.921	3.87	0.94	2	12.7	1.45E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	79	0.013		87	62	69	196
T	D	128	4	3.487	1.494	0.969	3.61	0.88	1	12.2	1.11E+06	01-Jun-98	1	1	0.109	9,800	2	2	1	138	0.034		182	117	131	196
T	D	137	4	3.491	1.495	0.972	3.80	0.92	1	12.5	1.34E+06	01-Jun-98	2	2	0.031	9,800	3	2	1	2	0.001		39	26	32	196
T	D	140	4	3.471	1.484	0.945	4.06	0.99	1	11.5	1.44E+06	01-Jun-98	1	1	0.141	9,800	1	2	1	148	0.009		180	122	147	196
T	D	143	4	3.463	1.475	0.926	4.66	1.13	2	12.3	1.89E+06	01-Jun-98	1	2	0.109	9,800	2	2	1	79	0.013		145	100	114	196
T	D	147	4	3.446	1.480	0.931	4.93	1.17	2	14.6	2.04E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	96	0.017		129	90	98	196
T	D	150	4	3.476	1.491	0.960	3.67	0.89	2	12.1	1.31E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	4	0.001		58	39	48	196
T	F	17	4	3.489	1.492	0.966	3.89	0.97	1	9.3	1.28E+06	29-May-98	1	1	0.063	9,800	2	2	1	1	0.000		77	51	64	196
T	F	29	4	3.471	1.489	0.955	4.90	1.21	1	10.6	2.11E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	80	0.013		87	62	65	196
T	F	34	4	3.482	1.495	0.970	4.09	1.00	1	11.8	1.42E+06	29-May-98	1	1	0.031	9,800	3	2	1	6	0.001		39	26	32	196
T	F	42	4	3.492	1.493	0.968	4.48	1.11	1	10.5	1.17E+06	29-May-98	1	1	0.063	9,800	2	2	1	55	0.006		83	58	68	196
T	F	44	4	3.492	1.496	0.974	3.96	0.99	1	9.8	1.57E+06	29-May-98	2	2	0.047	9,800	3	2	1	65	0.018		72	53	54	196
T	F	48	4	3.447	1.481	0.933	4.71	1.18	1	10.4	2.27E+06	29-May-98	1	1	0.078	9,800	2	2	1	41	0.005		100	68	79	196
T	F	56	4	3.484	1.493	0.966	4.22	1.05	1	10.0	1.64E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	60	0.016		70	51	53	196
T	F	58	4	3.473	1.483	0.944	4.51	1.12	1	10.3	1.79E+06	29-May-98	1	1	0.063	9,800	2	2	1	23	0.003		79	53	64	196
T	F	59	4	3.469	1.489	0.954	5.40	1.33	1	10.9	2.45E+06	01-Jun-98	1	1	0.016	9,800	4	2	1	34	0.011		28	22	15	196
T	F	67	4	3.465	1.495	0.970	4.64	1.14	1	11.0	1.65E+06	29-May-98	1	2	0.109	9,800	2	2	1	56	0.006		141	96	113	196
T	F	70	4	3.475	1.491	0.960	4.38	1.09	1	9.3	1.98E+06	29-May-98	1	1	0.063	9,800	2	2	1	54	0.006		83	58	65	196
T	F	74	4	3.479	1.487	0.953	4.49	1.10	1	11.8	1.37E+06	01-Jun-98	1	1	0.047	9,800	3	2	1	72	0.022		75	56	58	196
T	F	82	4	3.488	1.494	0.969	4.16	1.04	1	9.4	1.66E+06	01-Jun-98	1	1	0.063	9,800	2	2	1	102	0.019		92	67	69	196
T	F	87	4	3.498	1.493	0.970	4.06	1.00	1	10.4	1.81E+06	29-May-98	1	1	0.094	9,800	2	2	1	126	0.029		139	100	102	196

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_o$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	F	92	4	3.494	1.489	0.961	3.75	0.93	1	9.8	1.24E+06	29-May-98	1	1	0.141	9,800	1	2	1	130	0.008		179	121	148	196
T	F	93	4	3.495	1.501	0.985	3.98	0.99	1	10.0	1.35E+06	01-Jun-98	1	1	0.031	9,800	3	2	1	76	0.024		58	45	42	196
T	F	101	4	3.475	1.490	0.958	4.46	1.10	1	10.5	1.70E+06	29-May-98	1	1	0.031	9,800	3	2	1	8	0.001		39	26	32	196
T	F	106	4	3.490	1.484	0.950	4.35	1.08	1	10.2	1.75E+06	29-May-98	2	2	0.031	9,800	3	2	1	25	0.005		42	30	33	196
T	F	114	4	3.491	1.492	0.966	4.17	1.04	1	9.8	1.88E+06	29-May-98	1	1	0.125	9,800	1	2	1	150	0.009		160	109	129	196
T	F	116	4	3.495	1.484	0.952	4.05	1.01	1	9.9	1.50E+06	29-May-98	1	1	0.125	9,800	1	2	1	78	0.004		156	105	129	196
T	F	123	4	3.490	1.495	0.972	3.65	0.89	1	11.3	1.11E+06	29-May-98	1	1	0.125	9,800	1	2	1	120	0.007		159	106	132	196
T	F	124	4	3.463	1.493	0.960	4.17	1.05	1	8.7	1.95E+06	29-May-98	1	1	0.031	9,800	3	2	1	25	0.005		42	30	33	196
T	F	126	4	3.477	1.488	0.955	3.75	0.94	1	8.8	1.32E+06	29-May-98	1	2	0.047	9,800	3	2	1	51	0.013		66	49	54	196
T	F	130	4	3.449	1.487	0.945	4.72	1.17	1	9.7	1.66E+06	29-May-98	1	1	0.078	9,800	2	2	1	70	0.011		105	73	83	196
T	F	133	4	3.484	1.491	0.962	4.92	1.20	1	11.4	1.41E+06	29-May-98	2	2	0.078	9,800	2	2	1	155	0.044		132	100	96	196
T	F	134	4	3.475	1.490	0.958	3.52	0.87	1	9.9	1.13E+06	29-May-98	1	1	0.063	9,800	2	2	1	78	0.013		87	62	71	196
T	F	143	4	3.529	1.488	0.969	3.58	0.88	1	10.6	1.49E+06	29-May-98	1	1	0.109	9,800	2	2	1	112	0.022		152	107	120	196
T	F	147	4	3.480	1.415	0.822	4.49	1.11	1	10.2	2.05E+06	01-Jun-98	1	1	0.094	9,800	2	2	1	14	0.002		118	78	98	196
T	M	6	4	3.456	1.491	0.955	3.26	0.81	1	9.7	1.16E+06	28-May-98	1	2	0.125	9,800	1	2	1	118	0.006		158	107	132	196
T	M	9	4	3.544	1.490	0.977	4.69	1.16	1	12.5	1.34E+06	28-May-98	1	1	0.016	9,800	4	1	4	126	0.170	Brace unstable at proof load, slow fail	156	152	90	196
T	M	15	4	3.390	1.477	0.910	4.05	1.00	1	10.7	1.48E+06	28-May-98	1	2	0.094	9,800	2	2	1	101	0.019		130	92	104	196
T	M	18	4	3.474	1.462	0.905	4.34	1.05	1	12.7	1.53E+06	28-May-98	1	1	0.156	9,800	1	2	1	225	0.018		206	142	166	196
T	M	23	4	3.461	1.470	0.916	4.45	1.06	1	12.1	1.99E+06	28-May-98	1	1	0.094	9,800	2	2	1	118	0.025		135	97	100	196
T	M	27	4	3.440	1.461	0.894	3.86	0.94	1	12.3	1.59E+06	28-May-98	1	1	0.047	9,800	3	2	1	57	0.015		70	51	54	196
T	M	33	4	3.465	1.470	0.917	3.68	0.90	1	11.6	1.19E+06	28-May-98	1	1	0.109	9,800	2	2	1	83	0.014		145	101	120	196
T	M	34	4	3.399	1.454	0.871	3.87	0.93	1	13.1	1.55E+06	28-May-98	2	2	0.047	9,800	3	2	1	50	0.012		67	48	53	196
T	M	35	4	3.420	1.471	0.907	4.77	1.17	1	11.5	1.87E+06	28-May-98	1	1	0.063	9,800	2	2	1	71	0.011		86	60	67	196
T	M	36	4	3.413	1.459	0.883	5.28	1.25	1	15.3	2.32E+06	28-May-98	1	1	0.063	9,800	2	2	1	88	0.015		89	63	65	196
T	M	38	4	3.454	1.457	0.890	4.04	0.98	1	12.8	1.70E+06	28-May-98	1	1	0.094	9,800	2	2	1	58	0.008		121	83	98	196
T	M	39	4	3.462	1.486	0.945	4.14	1.01	1	11.9	1.44E+06	28-May-98	1	1	0.063	9,800	2	2	1	19	0.002		79	53	65	196
T	M	42	4	3.453	1.478	0.929	3.62	0.88	1	11.7	1.32E+06	28-May-98	1	1	0.047	9,800	2	2	1	24	0.003		60	41	49	196
T	M	46	4	3.455	1.465	0.905	4.08	1.01	1	10.4	1.29E+06	28-May-98	1	1	0.125	9,800	1	2	1	128	0.007		159	106	132	196
T	M	55	4	3.469	1.483	0.943	3.85	0.95	1	10.5	1.01E+06	28-May-98	1	1	0.094	9,800	2	2	1	107	0.021		132	94	110	196
T	M	58	4	3.465	1.470	0.917	3.91	0.96	1	11.5	1.10E+06	28-May-98	1	1	0.250	9,800	1	2	1	273	0.026		327	225	273	196
T	M	59	4	3.462	1.486	0.947	3.56	0.87	1	11.2	1.19E+06	28-May-98	1	1	0.063	9,800	2	2	1	80	0.013		87	62	71	196
T	M	62	4	3.403	1.456	0.875	4.33	1.06	1	11.0	1.46E+06	28-May-98	1	1	0.063	9,800	2	2	1	11	0.001		77	52	64	196
T	M	64	4	3.491	1.491	0.964	3.20	0.79	1	10.4	1.08E+06	29-May-98	1	1	0.172	9,800	1	2	1	150	0.009		218	148	182	196
T	M	65	4	3.442	1.456	0.889	3.81	0.93	1	11.1	1.28E+06	29-May-98	1	1	0.141	9,800	1	2	1	132	0.008		179	121	148	196
T	M	66	4	3.361	1.463	0.877	3.78	0.94	1	9.9	1.57E+06	28-May-98	2	1	0.063	9,800	2	2	1	72	0.011		86	60	68	196
T	M	68	4	3.438	1.457	0.886	3.62	0.88	1	12.2	1.39E+06	28-May-98	1	1	0.078	9,800	2	2	1	79	0.013		106	74	86	196
T	M	69	4	3.476	1.476	0.932	3.55	0.87	1	11.2	1.20E+06	29-May-98	1	1	0.109	9,800	1	2	1	80	0.004		137	93	114	196
T	M	71	4	3.442	1.474	0.919	3.53	0.87	1	10.1	1.53E+06	28-May-98	1	1	0.141	9,800	2	2	1	150	0.039		204	147	159	196
T	M	75	4	3.473	1.381	0.762	3.23	0.80	1	10.5	1.24E+06	28-May-98	1	1	0.125	9,800	2	2	1	131	0.031		178	127	148	196
T	M	76	4	3.445	1.462	0.897	3.89	0.96	1	10.6	1.56E+06	28-May-98	2	2	0.031	9,800	3	2	1	48	0.012		48	35	37	196
T	M	88	4	3.430	1.469	0.906	4.53	1.10	1	12.7	1.87E+06	28-May-98	1	2	0.047	9,800	3	2	1	17	0.003		60	41	49	196
T	M	89	4	3.486	1.489	0.959	3.53	0.87	1	10.9	8.79E+05	28-May-98	1	1	0.156	9,800	1	2	1	121	0.007		197	133	168	196
T	M	94	4	3.435	1.465	0.900	3.37	0.84	1	9.6	1.31E+06	28-May-98	1	2	0.109	9,800	2	2	1	81	0.013		145	100	119	196
T	M	95	4	3.415	1.478	0.919	3.56	0.88	1	10.3	1.39E+06	28-May-98	1	1	0.125	9,800	1	2	1	121	0.007		159	108	131	196
T	M	100	4	3.489	1.495	0.972	3.96	0.96	1	12.2	1.14E+06	28-May-98	1	2	0.078	9,800	2	2	1	60	0.009		103	71	85	196
T	M	102	4	3.411	1.481	0.923	3.53	0.87	1	10.5	1.17E+06	28-May-98	1	1	0.125	9,800	1	2	1	106	0.005		157	106	131	196
T	M	103	4	3.448	1.471	0.915	3.58	0.86	1	12.9	1.67E+06	29-May-98	1	1	0.078	9,800	2	2	1	23	0.003		98	66	80	196
T	M	104	4	3.425	1.480	0.925	3.74	0.90	1	12.9	1.36E+06	29-May-98	1	1	0.125	9,800	1	2	1	78	0.004		156	105	130	196
T	M	106	4	3.395	1.459	0.879	3.91	0.94	1	12.8	1.49E+06	28-May-98	2	2	0.078	9,800	2	2	1	20	0.002		97	65	81	196
T	M	122	4	3.387	1.474	0.904	3.68	0.90	1	11.9	1.18E+06	29-May-98	1	2	0.109	9,800	2	2	1	67	0.010		142	97	118	196
T	M	125	4	3.512	1.485	0.958	3.67	0.90	1	11.7	1.15E+06	29-May-98	1	1	0.031	9,800	3	2	1	23	0.005		42	30	35	196
T	M	132	4	3.373	1.471	0.895	3.59	0.89	1	10.3	1.19E+06	29-May-98	1	1	0.156	9,800	1	2	1	121	0.007		197	133	165	196

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :												Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	M	135	4	3.427	1.480	0.926	4.21	1.03	1	11.5	1.19E+06	29-May-98	1	1	0.203	9,600	1	2	1	126	0.007		255	172	213	196
T	M	136	4	3.448	1.513	0.995	3.53	0.87	1	10.3	9.36E+05	29-May-98	1	1	0.156	9,600	1	2	1	166	0.011		200	137	169	196
T	M	143	4	3.386	1.476	0.911	3.52	0.87	1	11.0	9.49E+05	29-May-98	1	1	0.203	9,600	1	1	2	461	0.128	Column failure, s-bending at proof load	353	270	295	196
T	M	145	4	3.405	1.461	0.885	3.41	0.84	1	10.9	1.19E+06	28-May-98	1	1	0.063	9,600	2	2	1	55	0.006		83	58	69	196
T	M	149	4	3.414	1.478	0.918	3.78	0.93	1	10.6	1.03E+06	28-May-98	1	1	0.047	9,600	3	2	1	8	0.001		56	39	49	196
T	M	150	4	3.526	1.473	0.939	3.53	0.88	1	9.7	1.27E+06	28-May-98	1	1	0.063	9,600	2	2	1	57	0.006		63	58	66	196
S	D	12	6	3.506	1.480	0.947	5.72	0.90	1	12.4	1.41E+06	26-May-98	1	2	0.188	7,600	12	2	1	111	0.020		127	88	109	152
S	D	16	6	3.442	1.458	0.889	5.63	0.91	1	9.8	1.55E+06	26-May-98	1	2	0.219	7,600	12	2	1	123	0.025		149	103	127	152
S	D	18	6	3.420	1.442	0.855	6.18	0.96	1	11.3	2.07E+06	26-May-98	2	2	0.063	7,600	15	2	1	25	0.010		44	31	36	152
S	D	19	6	3.453	1.477	0.927	5.95	0.93	1	12.6	1.88E+06	26-May-98	1	1	0.094	7,600	14	2	1	50	0.018		67	47	56	152
S	D	21	6	3.474	1.468	0.916	6.20	0.99	1	10.6	1.32E+06	25-May-98	1	1	0.094	7,600	14	2	1	62	0.027		71	51	62	152
S	D	23	6	3.386	1.462	0.918	6.65	1.05	1	12.2	2.26E+06	26-May-98	1	1	0.156	7,600	13	2	1	60	0.015		105	72	87	152
S	D	27	6	3.504	1.463	0.952	6.53	1.02	1	12.9	1.83E+06	26-May-98	1	2	0.125	7,600	13	2	1	25	0.005		81	55	66	152
S	D	29	6	3.488	1.468	0.920	6.67	1.04	1	13.1	1.79E+06	26-May-98	1	1	0.156	7,600	13	2	1	121	0.057		123	90	102	152
S	D	32	6	3.366	1.465	0.888	7.10	1.13	1	11.3	2.38E+06	26-May-98	2	2	0.063	7,600	15	2	1	24	0.009		43	30	35	152
S	D	36	6	3.399	1.467	0.894	5.97	0.95	1	10.8	1.94E+06	26-May-98	1	1	0.094	7,600	14	2	1	43	0.015		66	46	55	152
S	D	46	6	3.488	1.464	0.912	6.79	1.06	1	10.9	2.34E+06	25-May-98	1	1	0.156	7,600	13	2	1	68	0.030		112	79	90	152
S	D	48	6	3.400	1.456	0.875	6.98	1.11	1	11.4	2.41E+06	25-May-98	2	2	0.094	7,600	14	2	1	17	0.004		61	41	51	152
S	D	54	6	3.456	1.490	0.953	5.69	0.91	1	10.5	1.68E+06	24-May-98	1	1	0.094	7,600	14	2	1	65	0.029		72	52	60	152
S	D	59	6	3.478	1.470	0.921	5.25	0.83	1	11.3	1.92E+06	25-May-98	2	2	0.063	7,600	15	2	1	32	0.014		45	32	39	152
S	D	60	6	3.454	1.466	0.944	5.49	0.88	1	12.5	1.58E+06	25-May-98	1	2	0.094	7,600	14	2	1	38	0.012		64	45	55	152
S	D	62	6	3.454	1.474	0.922	5.86	0.93	1	11.2	1.77E+06	26-May-98	2	2	0.031	7,600	17	2	1	39	0.034		34	28	28	152
S	D	65	6	3.460	1.482	0.944	5.34	0.84	1	11.7	1.36E+06	26-May-98	1	1	0.125	7,600	13	2	1	77	0.023		89	62	77	152
S	D	67	6	3.474	1.476	0.931	6.30	0.99	1	12.7	2.06E+06	25-May-98	1	1	0.094	7,600	14	2	1	60	0.044		76	58	62	152
S	D	68	6	3.504	1.493	0.972	5.85	0.92	1	12.2	1.70E+06	25-May-98	1	1	0.094	7,600	14	2	1	74	0.036		75	56	62	152
S	D	70	6	3.468	1.476	0.933	6.00	0.94	1	12.6	1.76E+06	24-May-98	1	1	0.063	7,600	15	2	1	46	0.024		50	37	41	152
S	D	76	6	3.410	1.457	0.879	5.64	0.93	1	10.6	1.75E+06	26-May-98	1	2	0.125	7,600	13	2	1	58	0.014		65	59	72	152
S	D	77	6	3.452	1.479	0.931	5.73	0.90	1	11.7	1.79E+06	26-May-98	2	2	0.031	7,600	17	2	1	24	0.015		26	20	21	152
S	D	78	6	3.474	1.458	0.897	5.82	0.91	1	12.6	1.31E+06	26-May-98	1	1	0.125	7,600	13	2	1	101	0.036		95	69	84	152
S	D	79	6	3.483	1.492	0.964	6.61	1.06	1	13.0	2.15E+06	26-May-98	1	1	0.063	7,600	15	2	1	77	0.071		70	56	51	152
S	D	84	6	3.442	1.476	0.922	6.35	1.01	1	11.5	1.98E+06	26-May-98	1	1	0.094	7,600	14	2	1	37	0.012		64	45	53	152
S	D	93	6	3.474	1.480	0.938	7.04	1.11	1	12.3	1.83E+06	25-May-98	1	1	0.125	7,600	13	2	1	73	0.020		88	61	72	152
S	D	96	6	3.496	1.464	0.952	5.86	0.91	1	13.5	1.26E+06	25-May-98	1	2	0.313	7,600	11	2	1	196	0.012		203	137	176	152
S	D	105	6	3.499	1.464	0.953	6.26	0.97	1	13.4	1.33E+06	24-May-98	1	1	0.094	7,600	14	2	1	12	0.003		61	41	52	152
S	D	111	6	3.350	1.464	0.876	6.21	0.99	1	10.7	1.99E+06	26-May-98	1	1	0.063	7,600	15	2	1	61	0.041		57	44	46	152
S	D	112	6	3.340	1.462	0.870	6.12	0.97	1	10.9	2.04E+06	26-May-98	1	1	0.063	7,600	15	2	1	59	0.036		56	42	45	152
S	D	115	6	3.352	1.456	0.866	5.71	0.91	1	11.1	1.82E+06	26-May-98	1	2	0.094	7,600	14	2	1	55	0.022		69	49	58	152
S	D	123	6	3.413	1.454	0.874	5.55	0.87	1	11.9	1.69E+06	25-May-98	1	1	0.063	7,600	15	2	1	30	0.012		45	31	38	152
S	D	128	6	3.429	1.461	0.928	5.85	0.93	1	11.6	1.59E+06	24-May-98	1	2	0.250	7,600	12	2	1	178	0.053		181	128	153	152
S	D	131	6	3.422	1.474	0.913	6.19	0.98	1	11.2	1.73E+06	24-May-98	1	2	0.156	7,600	14	2	1	63	0.027		110	77	93	152
S	D	140	6	3.324	1.453	0.850	6.87	1.09	1	11.7	2.53E+06	25-May-98	1	2	0.125	7,600	13	2	1	65	0.017		88	60	70	152
S	D	142	6	3.430	1.462	0.893	5.56	0.86	1	11.7	1.19E+06	25-May-98	2	2	0.063	7,402	15	1	6	109	0.146	Column s-bending and brace failure	99	87	92	146
S	D	144	6	3.330	1.468	0.878	5.74	0.91	1	11.7	2.13E+06	25-May-98	1	1	0.156	7,600	13	2	1	85	0.027		110	77	91	152
S	D	146	6	3.398	1.482	0.922	6.11	0.97	1	11.4	2.00E+06	25-May-98	1	1	0.063	7,600	15	2	1	13	0.005		42	29	35	152
S	D	150	6	3.430	1.468	0.904	5.63	0.88	1	12.7	1.52E+06	25-May-98	1	1	0.125	7,600	13	2	1	64	0.017		88	60	74	152
S	F	1	6	3.482	1.490	0.980	7.94	1.24	1	12.9	2.64E+06	20-May-98	1	1	0.094	7,600	14	2	1	72	0.035		74	54	56	152
S	F	11	6	3.486	1.489	0.960	7.53	1.18	1	12.7	2.50E+06	20-May-98	1	1	0.125	7,600	13	2	1	51	0.012		64	58	66	152
S	F	14	6	3.494	1.492	0.967	6.19	1.00	1	9.8	1.65E+06	20-May-98	1	1	0.125	7,600	13	2	1	62	0.016		66	60	72	152
S	F	18	6	3.495	1.499	0.981	6.29	1.00	1	10.5	1.61E+06	20-May-98	1	1	0.094	7,600	14	2	1	24	0.007		62	43	53	152
S	F	22	6	3.480	1.493	0.965	6.90	1.06	1	12.9	1.69E+06	20-May-98	1	1	0.156	7,600	13	2	1	69	0.030		112	79	92	152
S	F	23	6	3.472	1.482	0.942	6.74	1.07	1	10.8	2.31E+06	20-May-98	1	1	0.094	7,600	14	2	1	28	0.006		63	43	52	152
S	F	35	8	3.475	1.489	0.956	7.60	1.21	1	13.9	2.74E+06	20-May-98	1	1	0.125	7,600	13	2	1	46	0.011		64	57	66	152

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_o$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Pfaut (lb)	Winter (lb)	Telen (lb)	2% Rule (lb)
S	F	42	6	3.467	1.489	0.954	7.72	1.21	1	12.6	2.71E+06	20-May-98	1	1	0.125	7,600	13	2	1	43	0.009		83	57	67	152
S	F	43	6	3.475	1.489	0.956	7.06	1.10	1	13.7	1.91E+06	20-May-98	1	2	0.063	7,600	15	2	1	44	0.022		49	36	40	152
S	F	46	6	3.482	1.492	0.964	6.54	1.04	1	10.6	1.87E+06	20-May-98	1	2	0.156	7,600	13	2	1	108	0.043		117	84	96	152
S	F	54	6	3.481	1.489	0.958	7.51	1.18	1	12.1	2.58E+06	18-May-98	1	1	0.063	7,600	15	2	1	34	0.015		46	33	38	152
S	F	56	6	3.464	1.489	0.953	6.30	1.01	1	9.8	2.00E+06	20-May-98	1	1	0.125	7,600	13	2	1	57	0.014		85	59	70	152
S	F	57	6	3.476	1.480	0.939	6.88	1.07	1	13.0	1.66E+06	20-May-98	1	1	0.313	7,600	12	2	1	177	0.053		220	154	183	152
S	F	65	6	3.491	1.483	0.949	6.15	0.96	1	10.7	1.71E+06	20-May-98	2	2	0.094	7,600	13	2	1	56	0.014		65	45	55	152
S	F	82	6	3.497	1.491	0.966	6.06	0.97	1	10.4	1.95E+06	20-May-98	1	1	0.094	7,600	14	2	1	20	0.005		61	42	51	152
S	F	86	6	3.489	1.494	0.970	6.82	1.09	1	10.4	1.73E+06	20-May-98	1	1	0.125	7,600	13	2	1	85	0.027		91	84	75	152
S	F	90	6	3.478	1.493	0.965	6.83	1.09	1	10.2	2.08E+06	20-May-98	1	1	0.188	7,600	13	2	1	96	0.036		134	94	109	152
S	F	94	6	3.496	1.496	0.975	5.37	0.86	1	10.0	1.49E+06	20-May-98	1	1	0.125	7,600	13	2	1	35	0.007		82	56	70	152
S	F	100	6	3.485	1.494	0.968	6.34	1.02	1	10.0	1.87E+06	20-May-98	1	1	0.063	7,600	15	2	1	73	0.062		66	53	51	152
S	F	108	6	3.493	1.494	0.971	6.52	1.02	1	12.7	1.57E+06	20-May-98	1	1	0.250	7,600	12	2	1	151	0.037		174	121	147	152
S	F	113	6	3.491	1.496	0.974	5.90	0.93	1	12.2	1.54E+06	18-May-98	1	1	0.156	7,600	13	2	1	103	0.040		116	83	96	152
S	F	117	6	3.482	1.494	0.968	7.33	1.15	1	12.2	2.43E+06	20-May-98	1	1	0.047	7,600	16	2	1	53	0.045		49	36	34	152
S	F	120	6	3.478	1.496	0.974	6.97	1.10	1	11.5	2.19E+06	20-May-98	1	1	0.156	7,600	13	2	1	54	0.013		104	71	88	152
S	F	127	6	3.482	1.493	0.966	6.97	1.11	1	10.4	2.19E+06	20-May-98	1	1	0.188	7,600	13	2	1	130	0.070		148	109	117	152
S	F	128	6	3.496	1.496	0.975	5.89	0.93	1	11.7	1.53E+06	20-May-98	1	1	0.109	7,600	14	2	1	41	0.013		75	52	63	152
S	F	129	6	3.494	1.494	0.971	6.15	0.98	1	11.1	1.69E+06	20-May-98	2	2	0.125	7,600	14	2	1	52	0.020		88	61	73	152
S	F	134	6	3.480	1.490	0.959	6.33	1.02	1	9.9	2.18E+06	20-May-98	1	1	0.156	7,600	13	2	1	76	0.022		108	75	88	152
S	F	139	6	3.485	1.491	0.963	6.83	1.05	1	11.9	2.34E+06	20-May-98	1	1	0.094	7,600	14	2	1	56	0.022		69	49	55	152
S	F	141	6	3.488	1.492	0.965	7.06	1.11	1	12.1	1.79E+06	20-May-98	1	1	0.125	7,600	14	2	1	54	0.021		88	62	73	152
S	F	142	6	3.491	1.494	0.970	6.16	0.98	1	9.6	1.65E+06	20-May-98	1	1	0.063	7,600	15	2	1	30	0.012		45	31	37	152
S	F	143	6	3.489	1.494	0.970	6.89	1.09	1	11.6	1.83E+06	10-May-98	1	1	0.156	7,600	13	2	1	126	0.064		126	93	105	152
S	M	1	6	3.422	1.470	0.906	5.39	0.87	1	9.3	1.72E+06	22-May-98	1	1	0.250	7,600	12	2	1	114	0.021		167	114	141	152
S	M	2	6	3.437	1.463	0.897	6.00	0.94	1	12.2	2.00E+06	22-May-98	1	2	0.313	7,600	12	2	1	43	0.005		200	134	166	152
S	M	5	6	3.439	1.444	0.883	5.84	0.93	1	10.4	1.64E+06	22-May-98	1	1	0.094	7,600	14	2	1	25	0.007		62	43	53	152
S	M	7	6	3.480	1.489	0.957	5.46	0.88	1	12.0	1.61E+06	22-May-98	1	1	0.188	7,600	13	2	1	83	0.026		130	90	109	152
S	M	8	6	3.506	1.503	0.993	6.30	0.99	1	12.1	1.25E+06	22-May-98	1	2	0.281	7,600	12	2	1	153	0.038		194	135	168	152
S	M	12	6	3.465	1.469	0.915	6.03	0.94	1	12.7	1.51E+06	22-May-98	2	2	0.094	6,977	15	1	3	112	0.175	Brace failure	122	104	100	140
S	M	13	6	3.418	1.477	0.918	5.95	0.94	1	11.2	1.36E+06	22-May-98	1	1	0.313	7,600	12	1	4	265	0.161	Brace failure	266	200	231	152
S	M	14	6	3.447	1.469	0.911	7.37	1.15	1	13.0	2.30E+06	22-May-98	1	1	0.063	7,600	15	2	1	61	0.041		57	44	43	152
S	M	15	6	3.459	1.466	0.946	7.40	1.16	1	12.4	2.06E+06	22-May-98	1	2	0.188	7,600	13	2	1	77	0.023		128	89	106	152
S	M	19	6	3.423	1.477	0.919	7.42	1.16	1	13.3	2.18E+06	22-May-98	1	1	0.125	7,600	13	2	1	36	0.007		82	56	68	152
S	M	20	6	3.418	1.436	0.843	7.96	1.24	1	12.9	2.55E+06	22-May-98	1	1	0.281	7,600	12	2	1	82	0.013		184	124	152	152
S	M	23	6	3.389	1.465	0.888	5.35	0.85	1	11.6	1.36E+06	22-May-98	1	1	0.188	7,600	12	2	1	143	0.033		133	93	115	152
S	M	31	6	3.455	1.472	0.918	6.34	0.99	1	13.3	1.84E+06	23-May-98	1	2	0.188	7,600	12	2	1	76	0.011		123	84	104	152
S	M	32	6	3.431	1.455	0.881	7.77	1.22	1	12.5	2.49E+06	22-May-98	1	1	0.125	7,600	13	2	1	55	0.013		65	58	69	152
S	M	33	6	3.457	1.474	0.923	6.80	1.07	1	12.6	1.96E+06	23-May-98	1	1	0.063	7,600	15	2	1	30	0.012		45	31	37	152
S	M	36	6	3.496	1.480	0.944	5.21	0.82	1	11.6	1.22E+06	23-May-98	1	2	0.188	7,600	12	2	1	125	0.025		129	90	113	152
S	M	40	6	3.446	1.459	0.892	7.25	1.12	1	14.0	2.00E+06	23-May-98	1	1	0.094	7,600	14	2	1	50	0.018		67	47	55	152
S	M	45	6	3.463	1.471	0.919	7.01	1.09	1	13.2	2.12E+06	23-May-98	1	1	0.156	7,600	13	2	1	59	0.015		105	72	87	152
S	M	49	6	3.425	1.474	0.914	6.13	0.96	1	10.4	1.68E+06	23-Jun-98	1	1	0.125	7,600	13	2	1	60	0.015		86	59	72	152
S	M	53	6	3.465	1.473	0.923	7.99	1.25	1	12.5	2.15E+06	23-May-98	1	1	0.063	7,600	15	2	1	9	0.003		41	28	34	152
S	M	88	6	3.451	1.455	0.886	6.10	0.95	1	13.2	1.72E+06	22-May-98	1	1	0.094	7,600	14	2	1	12	0.003		61	41	51	152
S	M	93	6	3.480	1.474	0.923	5.60	0.89	1	10.6	1.34E+06	22-May-98	1	2	0.125	7,600	13	2	1	45	0.010		83	57	72	152
S	M	97	6	3.482	1.487	0.954	6.07	0.95	1	12.3	1.56E+06	23-May-98	1	1	0.125	7,600	13	2	1	65	0.017		88	60	73	152
S	M	98	6	3.499	1.491	0.966	5.75	0.90	1	12.6	1.27E+06	23-May-98	2	2	0.063	7,600	15	2	1	31	0.013		45	32	39	152
S	M	102	6	3.487	1.485	0.952	5.57	0.89	1	11.0	1.29E+06	23-May-98	1	1	0.125	7,600	13	2	1	83	0.016		86	60	75	152
S	M	104	6	3.393	1.469	0.896	5.81	0.93	1	10.6	1.66E+06	23-May-98	1	1	0.125	7,600	13	2	1	12	0.002		80	54	68	152
S	M	112	6	3.469	1.471	0.920	5.55	0.88	1	11.8	1.42E+06	23-May-98	1	1	0.094	7,600	14	2	1	71	0.035		74	54	64	152
S	M	113	6	3.499	1.475	0.936	5.60	0.91	1	12.9	1.61E+06	23-May-98	2	2	0.031	7,600	17	2	1	2	0.001		20	14	17	152

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta₀ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	M	115	6	3.479	1.481	0.942	5.48	0.87	1	11.4	1.31E+06	23-May-98	1	1	0.094	7,600	14	2	1	73	0.037		75	56	65	152
S	M	117	6	3.556	1.477	0.955	6.44	1.00	1	13.3	1.91E+06	23-May-98	1	1	0.094	7,390	14	1	3	131	0.155	Brace failure	121	102	91	148
S	M	123	6	3.482	1.481	0.943	5.53	0.87	1	12.2	1.31E+06	22-May-98	1	1	0.219	7,600	12	2	1	131	0.027		150	104	130	152
S	M	126	6	3.479	1.478	0.936	6.00	0.95	1	11.7	1.61E+06	23-May-98	1	1	0.094	7,600	14	2	1	36	0.012		64	45	55	152
S	M	130	6	3.464	1.459	0.897	6.06	0.96	1	11.6	1.93E+06	23-May-98	1	1	0.094	7,600	14	2	1	79	0.044		78	58	63	152
S	M	131	6	3.451	1.465	0.904	6.57	1.04	1	11.6	1.22E+06	23-May-98	1	1	0.438	7,157	11	1	2	335	0.043	Column failure, s-bending	278	191	243	143
S	M	133	6	3.447	1.474	0.920	5.76	0.92	1	10.3	1.23E+06	23-May-98	1	2	0.219	7,600	12	1	4	161	0.056	Brace unstable, slow failure at proof load	162	116	143	152
S	M	138	6	3.482	1.481	0.943	5.92	0.94	1	11.3	1.21E+06	23-May-98	1	1	0.063	7,600	15	2	1	28	0.011		44	31	39	152
S	M	140	6	3.507	1.489	0.965	5.26	0.84	1	10.5	1.08E+06	23-May-98	1	1	0.156	7,600	13	1	5	162	0.162	Column s-bending and brace failure	167	134	154	152
S	M	145	6	3.475	1.457	0.896	6.37	1.00	1	12.4	1.86E+06	12-May-98	2	2	0.094	7,600	14	2	1	0	0.000		59	40	50	152
S	M	150	6	3.466	1.464	0.906	6.33	1.00	1	11.9	1.50E+06	22-May-98	2	2	0.063	7,600	15	2	1	6	0.002		40	27	35	152
T	D	1	6	3.441	1.490	0.949	8.46	1.30	2	14.9	2.58E+06	26-May-98	2	2	0.063	5,600	15	2	1	15	0.005		31	21	24	112
T	D	3	6	3.434	1.500	0.966	5.99	0.96	1	10.3	1.61E+06	27-May-98	2	2	0.063	5,600	17	2	1	13	0.007		31	22	26	112
T	D	4	6	3.458	1.491	0.955	8.13	1.24	2	15.4	2.27E+06	27-May-98	1	2	0.094	5,600	16	2	1	41	0.028		52	38	39	112
T	D	6	6	3.439	1.485	0.938	8.51	1.28	2	17.6	2.45E+06	27-May-98	1	1	0.125	5,600	16	2	1	49	0.039		70	51	51	112
T	D	9	6	3.481	1.511	1.001	6.61	1.04	2	12.4	1.67E+06	27-May-98	1	2	0.094	5,600	18	2	1	31	0.017		49	34	39	112
T	D	17	6	3.423	1.464	0.895	8.39	1.26	2	17.7	2.60E+06	27-May-98	1	1	0.125	5,600	17	2	1	27	0.019		64	45	49	112
T	D	18	6	3.478	1.507	0.992	6.99	1.10	2	12.2	1.78E+06	27-May-98	1	1	0.375	5,600	12	2	1	140	0.032		185	127	151	112
T	D	24	6	3.479	1.511	1.000	5.94	0.95	1	10.8	1.62E+06	27-May-98	1	1	0.094	5,600	16	2	1	32	0.018		49	35	40	112
T	D	26	6	3.459	1.506	0.985	7.24	1.13	1	13.2	2.02E+06	27-May-98	1	1	0.125	5,600	15	2	1	31	0.013		62	43	50	112
T	D	27	6	3.414	1.471	0.906	7.35	1.11	2	16.5	2.31E+06	27-May-98	2	2	0.063	5,600	17	2	1	16	0.009		32	22	25	112
T	D	32	6	3.525	1.503	0.997	6.19	0.97	1	12.1	1.54E+06	27-May-98	1	2	0.125	5,600	15	2	1	63	0.044		72	53	57	112
T	D	45	6	3.510	1.505	0.997	7.28	1.16	2	10.4	1.96E+06	27-May-98	1	1	0.188	5,600	13	2	1	91	0.031		97	68	77	112
T	D	46	6	3.521	1.503	0.996	6.23	0.98	2	12.2	1.48E+06	27-May-98	1	1	0.188	5,600	13	2	1	37	0.008		90	61	75	112
T	D	52	6	3.523	1.505	1.001	6.07	0.96	2	11.6	1.60E+06	27-May-98	1	1	0.125	5,600	15	2	1	28	0.012		62	43	51	112
T	D	53	6	3.504	1.489	0.964	6.66	1.03	2	14.4	1.77E+06	27-May-98	1	2	0.156	5,600	14	2	1	50	0.019		79	55	64	112
T	D	55	6	3.518	1.502	0.993	6.42	1.00	1	13.3	1.85E+06	27-May-98	1	2	0.094	5,600	16	2	1	23	0.011		47	33	38	112
T	D	60	6	3.496	1.501	0.985	7.60	1.18	2	13.6	2.20E+06	27-May-98	1	1	0.094	5,600	16	2	1	34	0.021		50	36	38	112
T	D	63	6	3.525	1.507	1.005	5.70	0.91	2	10.5	1.14E+06	17-May-98	2	2	0.063	5,600	17	2	1	11	0.005		31	21	26	112
T	D	67	6	3.443	1.477	0.924	6.63	1.06	2	11.8	2.11E+06	27-May-98	2	1	0.063	5,600	17	2	1	15	0.008		32	22	25	112
T	D	72	6	3.429	1.446	0.864	6.60	1.05	1	10.9	1.60E+06	27-May-98	1	1	0.094	5,600	16	2	1	37	0.023		51	36	42	112
T	D	74	6	3.417	1.477	0.917	6.73	1.07	1	11.1	2.06E+06	27-May-98	1	1	0.094	5,600	17	2	1	14	0.007		48	31	37	112
T	D	75	6	3.521	1.505	1.000	5.56	0.88	1	12.3	1.42E+06	27-May-98	1	1	0.156	5,600	14	2	1	52	0.020		79	55	65	112
T	D	79	6	3.520	1.511	1.012	6.39	1.00	2	12.6	1.15E+06	27-May-98	1	2	0.094	5,600	16	2	1	63	0.070		66	51	54	112
T	D	80	6	3.526	1.514	1.020	5.62	0.88	2	12.3	1.45E+06	26-May-98	1	1	0.125	5,600	15	2	1	42	0.021		65	45	53	112
T	D	99	6	3.500	1.488	0.961	5.49	0.85	2	14.0	1.27E+06	16-May-98	1	1	0.063	5,600	17	2	1	33	0.025		37	27	30	112
T	D	101	6	3.467	1.489	0.959	6.33	1.00	2	12.0	1.80E+06	26-May-98	1	1	0.063	5,600	17	2	1	29	0.021		38	26	28	112
T	D	104	6	3.471	1.486	0.953	5.84	0.92	2	11.9	1.71E+06	26-May-98	2	2	0.094	5,600	16	2	1	38	0.021		50	36	40	112
T	D	106	6	3.372	1.482	0.915	6.31	1.00	1	11.8	1.61E+06	27-May-98	1	1	0.125	5,600	15	2	1	56	0.034		69	49	55	112
T	D	107	6	3.462	1.472	0.920	6.19	0.96	1	11.2	1.31E+06	26-May-98	2	2	0.063	5,600	17	2	1	26	0.018		35	25	29	112
T	D	117	6	3.482	1.485	0.950	6.47	1.04	1	10.3	1.22E+06	26-May-98	1	2	0.313	5,600	12	2	1	135	0.030		155	107	131	112
T	D	141	6	3.481	1.477	0.935	5.17	0.81	1	12.2	1.08E+06	27-May-98	2	2	0.063	5,449	17	1	3	75	0.163	Brace failure	78	68	65	109
T	D	144	6	3.490	1.463	0.911	5.53	0.88	2	11.8	1.32E+06	26-May-98	1	1	0.188	5,600	13	2	1	82	0.025		95	66	80	112
T	D	146	6	3.476	1.490	0.958	5.67	0.89	2	12.4	1.24E+06	26-May-98	1	1	0.094	5,600	18	2	1	11	0.005		45	31	38	112
T	D	146	6	3.394	1.462	0.884	6.78	1.06	2	12.5	1.90E+06	26-May-98	1	1	0.125	5,600	15	2	1	37	0.017		64	44	51	112
T	F	7	6	3.472	1.479	0.936	8.12	1.29	1	10.7	2.26E+06	16-May-98	1	2	0.094	5,600	16	2	1	26	0.013		48	33	38	112
T	F	8	6	3.492	1.494	0.970	6.15	0.96	1	10.3	1.76E+06	24-May-98	1	2	0.031	5,600	20	2	1	14	0.012		18	13	14	112
T	F	9	6	3.498	1.498	0.980	5.55	0.89	1	9.5	1.30E+06	24-May-98	1	2	0.250	5,600	13	2	1	83	0.026		125	86	104	112
T	F	13	6	3.478	1.486	0.951	7.37	1.16	1	12.0	1.61E+06	22-May-98	1	2	0.094	5,600	16	2	1	29	0.016		48	34	40	112
T	F	18	6	3.491	1.496	0.974	6.23	1.01	1	9.4	1.86E+06	24-May-98	1	2	0.125	5,600	15	2	1	49	0.026		66	47	52	112
T	F	20	6	3.490	1.492	0.968	6.11	0.99	1	9.4	1.69E+06	19-May-98	1	1	0.094	5,600	16	2	1	32	0.018		49	35	40	112
T	F	21	6	3.486	1.493	0.967	6.25	1.01	1	9.6	2.07E+06	19-May-98	1	1	0.094	5,600	16	2	1	14	0.008		46	31	37	112

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	F	22	6	3.479	1.494	0.967	6.76	1.09	1	9.4	1.94E+06	19-May-98	1	1	0.094	5,600	16	2	1	27	0.014		48	34	38	112
T	F	23	6	3.474	1.490	0.958	6.48	1.01	1	12.7	2.29E+06	19-May-98	2	2	0.094	5,600	18	2	1	23	0.011		47	33	37	112
T	F	31	6	3.468	1.488	0.952	7.59	1.20	1	11.6	2.49E+06	19-May-98	1	1	0.094	5,600	16	2	1	43	0.030		53	39	38	112
T	F	35	6	3.451	1.485	0.942	6.95	1.11	1	10.2	2.31E+06	19-May-98	1	1	0.125	5,600	15	2	1	32	0.013		62	43	50	112
T	F	41	6	3.461	1.491	0.958	7.49	1.20	1	9.8	2.41E+06	19-May-98	1	1	0.031	5,600	19	2	1	17	0.015		19	14	13	112
T	F	49	6	3.492	1.495	0.972	6.98	1.12	1	10.1	1.49E+06	19-May-98	1	2	0.094	5,600	16	2	1	33	0.019		50	35	40	112
T	F	50	6	3.486	1.491	0.963	6.53	1.04	1	11.0	2.06E+06	18-May-98	1	1	0.063	5,600	17	2	1	17	0.019		35	25	27	112
T	F	53	6	3.474	1.486	0.950	6.74	1.07	1	11.6	2.29E+06	18-May-98	1	1	0.031	5,600	17	2	1	1	0.001		15	10	12	112
T	F	54	6	3.457	1.489	0.951	6.92	1.11	1	10.4	1.74E+06	18-May-98	1	1	0.094	5,600	16	2	1	28	0.015		48	34	39	112
T	F	55	6	3.478	1.493	0.965	6.94	1.11	1	10.5	2.13E+06	18-May-98	1	2	0.063	5,600	17	2	1	44	0.044		43	33	29	112
T	F	57	6	3.488	1.491	0.963	6.19	0.99	1	10.0	1.38E+06	18-May-98	1	1	0.125	5,600	15	2	1	18	0.007		81	41	51	112
T	F	60	6	3.491	1.489	0.960	6.68	1.06	1	9.3	1.58E+06	18-May-98	1	1	0.063	5,600	17	2	1	31	0.022		36	26	29	112
T	F	61	6	3.489	1.492	0.968	6.90	1.11	1	9.5	2.09E+06	18-May-98	1	1	0.094	5,600	16	2	1	25	0.013		48	33	38	112
T	F	64	6	3.493	1.495	0.973	5.95	0.95	1	10.8	1.71E+06	19-May-98	1	1	0.063	5,600	17	2	1	25	0.016		34	24	27	112
T	F	66	6	3.483	1.491	0.962	6.09	0.97	1	10.3	1.80E+06	19-May-98	1	1	0.125	5,600	15	2	1	51	0.029		67	48	53	112
T	F	71	6	3.494	1.500	0.983	6.75	1.08	1	9.9	1.44E+06	18-May-98	1	1	0.031	5,600	19	2	1	15	0.012		18	13	15	112
T	F	84	6	3.496	1.495	0.974	6.97	1.12	1	9.5	1.78E+06	19-May-98	1	1	0.094	5,600	16	2	1	8	0.003		45	30	37	112
T	F	95	6	3.490	1.498	0.974	6.12	0.98	1	9.9	1.42E+06	18-May-98	1	1	0.250	5,600	13	2	1	76	0.022		124	85	103	112
T	F	96	6	3.470	1.490	0.957	7.16	1.15	1	10.2	1.91E+06	18-May-98	1	1	0.188	5,600	13	2	1	64	0.017		93	64	75	112
T	F	97	6	3.455	1.489	0.950	6.73	1.08	1	10.4	2.14E+06	18-May-98	2	2	0.063	5,600	17	2	1	18	0.010		32	23	25	112
T	F	98	6	3.490	1.490	0.962	6.91	1.11	1	9.8	2.16E+06	18-May-98	2	2	0.063	5,600	17	2	1	41	0.036		41	31	28	112
T	F	99	6	3.478	1.494	0.966	6.22	1.00	1	10.0	1.58E+06	18-May-98	1	1	0.125	5,600	15	2	1	19	0.007		61	41	50	112
T	F	111	6	3.486	1.491	0.963	5.37	0.86	1	10.3	1.46E+06	19-May-98	1	1	0.313	5,600	12	2	1	110	0.020		152	103	127	112
T	F	115	6	3.470	1.489	0.955	5.57	0.90	1	9.3	1.19E+06	19-May-98	1	2	0.250	5,600	13	2	1	70	0.019		123	84	104	112
T	F	117	6	3.498	1.497	0.978	5.77	0.93	1	8.9	1.24E+06	18-May-98	1	1	0.094	5,600	16	2	1	22	0.011		47	33	40	112
T	F	122	6	3.470	1.491	0.958	7.52	1.19	1	11.7	2.22E+06	19-May-98	1	1	0.031	5,600	19	2	1	39	0.080		33	28	18	112
T	F	125	6	3.491	1.496	0.974	6.06	0.96	1	12.0	1.34E+06	19-May-98	1	1	0.125	5,600	15	2	1	71	0.059		77	57	62	112
T	F	129	6	3.489	1.497	0.975	5.98	0.96	1	10.2	1.05E+06	19-May-98	1	1	0.188	5,600	13	2	1	45	0.010		91	61	77	112
T	F	132	6	3.485	1.485	0.951	5.46	0.88	1	9.6	1.47E+06	19-May-98	1	1	0.125	5,600	15	2	1	73	0.062		78	58	62	112
T	F	138	6	3.489	1.489	0.960	6.95	1.11	1	10.7	2.05E+06	19-May-98	1	1	0.125	5,600	15	2	1	33	0.014		63	43	50	112
T	F	139	6	3.477	1.491	0.960	6.63	1.06	1	10.7	2.01E+06	19-May-98	1	1	0.063	5,600	17	2	1	17	0.010		32	23	26	112
T	F	140	6	3.456	1.486	0.945	7.77	1.24	1	10.7	2.30E+06	19-May-98	1	1	0.094	5,600	16	2	1	36	0.022		51	36	38	112
T	F	142	6	3.492	1.492	0.968	5.59	0.90	1	9.7	1.75E+06	19-May-98	1	1	0.156	5,600	14	2	1	69	0.033		83	59	66	112
T	F	146	6	3.488	1.495	0.971	5.89	0.93	1	11.7	1.85E+06	18-May-98	1	1	0.156	5,600	13	2	1	60	0.015		78	53	63	112
T	M	1	6	3.400	1.488	0.933	5.27	0.84	1	10.8	1.23E+06	22-May-98	1	1	0.125	5,600	15	2	1	53	0.031		68	49	57	112
T	M	2	6	3.378	1.452	0.862	6.45	1.03	1	11.0	1.62E+06	22-May-98	1	1	0.094	5,600	16	2	1	42	0.029		53	38	43	112
T	M	3	6	3.481	1.492	0.963	6.19	0.97	1	12.8	1.50E+06	22-May-98	1	2	0.125	5,600	15	2	1	26	0.010		61	42	51	112
T	M	7	6	3.432	1.501	0.967	4.96	0.79	1	11.0	1.21E+06	20-May-98	1	1	0.063	5,600	17	2	1	34	0.028		38	28	31	112
T	M	10	6	3.568	1.482	0.968	7.02	1.09	1	13.7	1.14E+06	22-May-98	1	2	0.125	5,600	15	2	1	38	0.018		64	44	54	112
T	M	11	6	3.539	1.478	0.952	6.84	1.07	1	13.0	1.08E+06	22-May-98	1	2	0.438	5,600	12	2	1	177	0.053		221	153	188	112
T	M	12	6	3.409	1.466	0.895	6.59	1.05	1	10.6	1.79E+06	22-May-98	1	2	0.219	5,600	13	2	1	79	0.024		110	78	90	112
T	M	14	6	3.456	1.480	0.934	6.24	1.00	1	10.6	1.32E+06	22-May-98	1	1	0.313	5,600	12	2	1	134	0.028		155	108	130	112
T	M	17	6	3.401	1.480	0.919	5.59	0.89	1	10.4	1.48E+06	21-May-98	1	1	0.250	5,600	13	2	1	93	0.032		127	88	105	112
T	M	28	6	3.442	1.450	0.874	6.24	0.99	1	11.4	2.09E+06	21-May-98	1	1	0.125	5,600	15	2	1	33	0.014		63	43	51	112
T	M	29	6	3.423	1.478	0.921	5.32	0.85	1	10.8	7.91E+05	22-May-98	1	2	0.156	5,600	14	2	1	63	0.027		81	57	73	112
T	M	30	6	3.496	1.490	0.964	4.93	0.79	1	10.5	9.83E+05	21-May-98	1	2	0.125	5,600	15	2	1	87	0.051		74	55	64	112
T	M	48	6	3.476	1.480	0.940	6.07	0.96	1	12.2	1.51E+06	22-May-98	1	1	0.125	5,600	15	2	1	55	0.033		69	49	56	112
T	M	49	6	3.458	1.474	0.923	6.79	1.08	1	11.2	1.40E+06	22-May-98	1	2	0.219	5,600	13	2	1	80	0.024		110	76	91	112
T	M	52	6	3.459	1.484	0.942	5.95	0.95	1	10.9	1.03E+06	21-May-97	1	1	0.188	5,600	13	2	1	75	0.022		94	65	81	112
T	M	78	6	3.390	1.462	0.883	5.67	0.91	1	9.8	1.25E+06	21-May-98	1	2	0.063	5,600	13	2	1	14	0.003		30	20	26	112
T	M	80	6	3.421	1.462	0.891	5.93	0.95	1	10.7	1.77E+06	22-May-98	1	1	0.125	5,600	15	2	1	58	0.037		70	50	55	112
T	M	85	6	3.470	1.468	0.915	6.33	1.00	1	11.4	1.20E+06	20-May-98	1	2	0.313	5,600	12	2	1	126	0.025		154	105	130	112

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_o$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	M	90	6	3.438	1.492	0.952	6.77	1.09	1	9.1	1.68E+06	22-May-98	1	2	0.125	5,600	15	2	1	21	0.008		61	41	50	112
T	M	93	6	3.499	1.492	0.968	9.84	1.54	1	12.7	8.71E+05	21-May-98	1	1	0.438	5,462	11	1	2	282	0.027	Column failure in s-bending	208	141	181	110
T	M	98	6	3.459	1.461	0.936	6.02	0.96	1	11.1	1.39E+06	21-May-98	1	1	0.313	5,600	12	2	1	123	0.024		153	105	128	112
T	M	113	6	3.474	1.460	0.901	5.14	0.82	1	10.3	1.24E+06	21-May-98	1	1	0.125	5,600	15	2	1	20	0.007		61	41	51	112
T	M	116	6	3.569	1.487	0.983	5.94	0.93	1	12.7	1.33E+06	22-May-98	1	1	0.125	5,600	14	2	1	52	0.020		65	45	54	112
T	M	127	6	3.543	1.494	0.965	6.24	0.98	1	12.4	1.26E+06	22-May-98	1	2	0.250	5,600	13	2	1	123	0.061		136	97	113	112
T	M	128	6	3.354	1.496	0.940	5.05	0.80	1	11.2	9.07E+05	22-May-98	1	2	0.125	5,600	15	2	1	26	0.010		61	42	54	112
T	M	131	6	3.559	1.471	0.944	6.20	0.97	1	12.7	1.23E+06	22-May-98	1	1	0.125	5,600	15	2	1	53	0.032		68	49	57	112
T	M	133	6	3.355	1.478	0.903	5.69	0.91	1	10.2	1.43E+06	22-May-98	1	1	0.125	5,600	15	2	1	49	0.027		67	47	56	112
T	M	139	6	3.416	1.491	0.944	5.98	0.94	1	11.4	1.47E+06	22-May-98	1	1	0.250	5,600	13	2	1	104	0.041		129	91	107	112
T	M	144	6	3.437	1.489	0.946	5.55	0.88	1	11.2	1.10E+06	22-May-98	1	1	0.188	5,600	14	2	1	51	0.019		93	64	80	112
T	M	146	6	3.415	1.475	0.913	6.36	1.01	1	11.2	1.39E+06	22-May-98	1	1	0.188	5,600	14	2	1	61	0.026		96	66	80	112
S	D	3	8	3.446	1.460	0.931	8.71	1.02	1	11.9	2.13E+06	17-May-98	1	1	0.094	4,600	30	2	1	20	0.028		32	23	26	92
S	D	6	8	3.460	1.480	0.940	7.44	0.85	1	13.5	1.31E+06	17-May-98	1	1	0.313	4,600	23	2	1	102	0.051		100	70	87	92
S	D	9	8	3.358	1.461	0.906	8.67	1.00	1	12.6	2.26E+06	17-May-98	1	2	0.125	4,600	27	2	1	18	0.013		38	26	32	92
S	D	10	8	3.448	1.479	0.930	8.77	1.01	1	13.7	1.63E+06	17-May-90	1	1	0.125	4,600	27	2	1	26	0.023		40	28	34	92
S	D	11	8	3.411	1.470	0.903	7.81	0.92	1	12.2	2.07E+06	15-May-98	1	1	0.094	4,600	29	2	1	27	0.037		34	25	28	92
S	D	20	8	3.454	1.473	0.920	7.08	0.81	1	13.2	1.89E+06	15-May-98	1	1	0.125	4,600	27	2	1	21	0.018		39	27	33	92
S	D	24	8	3.437	1.466	0.902	8.99	1.02	1	14.7	2.11E+06	18-May-98	1	1	0.125	4,600	27	2	1	26	0.021		40	28	33	92
S	D	25	8	3.412	1.478	0.916	8.47	0.99	1	11.2	1.74E+06	18-May-98	1	1	0.313	4,600	23	2	1	67	0.023		94	64	80	92
S	D	30	8	3.462	1.470	0.916	9.26	1.05	1	15.1	1.70E+06	18-May-98	1	1	0.188	4,600	25	2	1	52	0.038		61	43	52	92
S	D	34	8	3.427	1.470	0.907	9.49	1.11	1	11.7	2.44E+06	17-May-98	1	1	0.125	4,600	27	2	1	51	0.080		51	39	39	92
S	D	38	8	3.452	1.469	0.912	7.95	0.93	1	11.9	1.44E+06	17-May-98	2	2	0.125	4,600	27	2	1	39	0.045		45	33	39	92
S	D	43	8	3.512	1.491	0.970	8.06	0.93	1	13.1	1.63E+06	17-May-98	1	1	0.125	4,600	27	2	1	33	0.032		42	30	36	92
S	D	45	8	3.509	1.479	0.946	9.41	1.10	1	11.4	1.53E+06	17-May-98	1	2	0.125	4,600	27	2	1	19	0.014		39	27	33	92
S	D	47	8	3.443	1.474	0.919	8.04	0.92	1	13.9	1.77E+06	17-May-98	1	1	0.125	4,600	27	2	1	30	0.028		41	29	35	92
S	D	50	8	3.408	1.471	0.903	7.48	0.88	1	11.3	1.82E+06	15-May-98	1	1	0.125	4,600	27	2	1	10	0.005		37	25	31	92
S	D	51	8	3.480	1.479	0.938	9.05	1.04	1	13.8	1.86E+06	15-May-98	1	1	0.125	4,600	27	2	1	27	0.023		40	28	34	92
S	D	55	8	3.464	1.478	0.926	7.92	0.93	1	11.7	1.54E+06	15-May-98	1	1	0.313	4,600	23	2	1	69	0.025		95	65	81	92
S	D	56	8	3.430	1.445	0.862	8.46	0.99	1	11.3	2.07E+06	17-May-98	1	1	0.063	4,600	30	2	1	10	0.010		20	14	17	92
S	D	61	8	3.469	1.473	0.924	8.87	1.04	1	11.7	2.24E+06	18-May-98	1	1	0.125	4,600	27	2	1	21	0.018		39	27	32	92
S	D	66	8	3.442	1.474	0.919	8.04	0.95	1	10.8	1.61E+06	18-May-98	1	2	0.313	4,600	23	2	1	68	0.024		94	64	81	92
S	D	80	8	3.488	1.468	0.958	9.11	1.06	1	11.7	1.85E+06	17-May-98	1	1	0.156	4,600	26	2	1	55	0.065		57	42	47	92
S	D	81	8	3.478	1.474	0.928	10.65	1.20	1	15.7	2.32E+06	15-May-98	1	1	0.125	4,600	27	2	1	29	0.026		41	29	33	92
S	D	82	8	3.442	1.473	0.917	8.74	1.01	1	13.1	1.65E+06	15-May-98	1	1	0.500	4,600	22	2	1	98	0.022		148	100	125	92
S	D	88	8	3.479	1.481	0.942	7.96	0.92	1	12.2	1.65E+06	15-May-98	1	1	0.125	4,600	27	2	1	18	0.013		36	26	32	92
S	D	89	8	3.463	1.494	0.962	8.62	0.98	1	14.7	2.30E+06	15-May-98	1	1	0.094	4,600	28	2	1	7	0.004		26	19	23	92
S	D	94	8	3.466	1.470	0.917	8.26	0.95	1	13.7	1.88E+06	15-May-98	1	1	0.125	4,600	30	2	1	36	0.066		52	40	43	92
S	D	100	8	3.445	1.462	0.897	9.06	1.04	1	14.3	2.33E+06	15-May-98	1	1	0.188	4,600	25	2	1	25	0.012		56	38	47	92
S	D	101	8	3.368	1.451	0.857	8.42	0.96	1	12.1	2.05E+06	15-May-98	1	1	0.313	4,600	23	2	1	30	0.007		91	61	77	92
S	D	102	8	3.440	1.496	0.964	8.97	1.03	1	13.3	1.90E+06	15-May-98	1	1	0.531	4,600	22	2	1	145	0.045		161	110	135	92
S	D	104	8	3.424	1.476	0.916	7.97	0.92	1	13.3	1.43E+06	17-May-98	2	2	0.125	4,600	27	2	1	22	0.017		39	27	34	92
S	D	106	8	3.482	1.499	0.977	9.44	1.09	1	13.3	2.01E+06	17-May-98	1	1	0.125	4,600	27	2	1	27	0.023		40	28	33	92
S	D	109	8	3.493	1.491	0.965	7.63	0.89	1	12.0	1.88E+06	17-May-98	1	1	0.125	4,600	27	2	1	24	0.019		40	28	33	92
S	D	118	8	3.464	1.498	0.966	9.57	1.10	1	13.5	2.20E+06	17-May-98	1	1	0.063	4,600	30	2	1	28	0.048		27	21	21	92
S	D	121	8	3.421	1.460	0.924	7.35	0.85	1	12.4	1.47E+06	17-May-98	1	1	0.063	4,600	30	2	1	19	0.025		23	17	20	92
S	D	124	8	3.394	1.462	0.884	9.24	1.08	1	13.6	2.39E+06	17-May-98	1	1	0.125	4,600	27	2	1	40	0.045		45	33	36	92
S	D	127	8	3.346	1.452	0.854	9.21	1.06	1	11.1	2.63E+06	17-May-98	1	1	0.188	4,600	27	2	1	46	0.063		66	46	52	92
S	D	139	8	3.411	1.471	0.905	8.18	0.93	1	14.4	2.10E+06	18-May-98	1	1	0.188	4,600	25	2	1	45	0.030		60	42	49	92
S	D	146	8	3.381	1.456	0.870	9.65	1.11	1	13.0	2.57E+06	18-May-98	1	1	0.250	4,600	24	2	1	47	0.021		78	52	63	92
S	D	147	8	3.446	1.466	0.905	7.41	0.85	1	13.2	1.41E+06	18-May-98	1	2	0.313	4,600	23	2	1	76	0.029		95	65	83	92
S	F	4	8	3.496	1.497	0.978	9.41	1.11	1	10.5	2.02E+06	12-May-98	1	1	0.063	4,600	30	2	1	5	0.004		19	13	16	92

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	F	5	8	3.494	1.500	0.983	8.72	1.03	1	10.4	1.87E+06	12-May-98	1	1	0.094	4,600	28	2	1	29	0.032		33	24	27	92
S	F	19	8	3.496	1.498	0.979	8.47	0.99	1	11.5	1.80E+06	13-May-98	1	1	0.188	4,600	25	2	1	44	0.029		59	41	50	92
S	F	25	8	3.493	1.500	0.982	8.43	0.98	1	11.8	1.69E+06	11-May-98	1	1	0.281	4,600	24	2	1	61	0.060		92	65	78	92
S	F	32	8	3.469	1.492	0.960	10.12	1.17	1	13.1	2.60E+06	12-May-98	2	2	0.188	4,600	25	2	1	24	0.012		56	38	46	92
S	F	33	8	3.482	1.493	0.966	8.22	0.95	1	12.3	2.08E+06	13-May-98	1	1	0.188	4,600	25	2	1	46	0.031		60	42	49	92
S	F	36	8	3.467	1.492	0.960	9.20	1.08	1	11.1	2.40E+06	12-May-98	1	1	0.125	4,600	27	2	1	30	0.028		41	29	33	92
S	F	37	8	3.476	1.498	0.974	10.65	1.21	1	15.0	2.60E+06	12-May-98	1	2	0.125	4,600	27	2	1	19	0.014		39	27	31	92
S	F	47	8	3.482	1.490	0.960	9.58	1.11	1	12.7	1.84E+06	13-May-98	1	1	0.250	4,600	24	2	1	52	0.025		77	53	64	92
S	F	55	8	3.498	1.498	0.976	8.94	1.07	1	9.3	1.83E+06	13-May-98	1	1	0.156	4,600	26	2	1	38	0.032		51	36	43	92
S	F	60	8	3.484	1.493	0.966	8.81	1.01	1	13.3	1.86E+06	13-May-98	1	1	0.188	4,600	25	2	1	30	0.015		57	39	48	92
S	F	64	8	3.496	1.508	0.999	6.97	0.83	1	9.5	1.43E+06	13-May-98	1	1	0.219	4,600	24	2	1	15	0.004		64	43	55	92
S	F	66	8	3.496	1.503	0.989	9.00	1.08	1	10.3	1.75E+06	13-May-98	1	1	0.313	4,600	23	2	1	64	0.035		97	67	81	92
S	F	71	8	3.498	1.502	0.988	8.58	0.99	1	12.9	1.63E+06	13-May-98	1	1	0.125	4,600	27	2	1	23	0.018		39	27	33	92
S	F	72	8	3.482	1.500	0.982	7.93	0.93	1	11.1	1.74E+06	12-May-98	1	1	0.188	4,600	25	2	1	32	0.017		57	39	46	92
S	F	78	8	3.478	1.492	0.962	12.21	1.40	1	13.8	2.94E+06	11-May-98	1	1	0.250	4,600	24	2	1	42	0.018		75	51	61	92
S	F	80	8	3.491	1.498	0.978	8.79	1.03	1	11.1	1.90E+06	13-May-98	1	2	0.125	4,600	27	2	1	10	0.008		37	25	31	92
S	F	86	8	3.472	1.498	0.969	11.62	1.37	1	10.4	2.73E+06	12-May-98	1	1	0.125	4,600	27	2	1	17	0.012		38	26	31	92
S	F	92	8	3.491	1.501	0.984	9.23	1.08	1	11.8	2.33E+06	12-May-98	1	1	0.094	4,600	28	2	1	11	0.008		28	20	23	92
S	F	95	8	3.484	1.492	0.964	8.29	0.98	1	10.3	2.09E+06	13-May-98	1	1	0.063	4,600	30	2	1	5	0.004		19	13	16	92
S	F	104	8	3.455	1.490	0.952	11.63	1.33	1	14.0	2.54E+06	12-May-98	1	2	0.219	4,600	24	2	1	32	0.012		65	44	54	92
S	F	107	8	3.487	1.494	0.969	9.51	1.12	1	10.8	2.22E+06	13-May-98	1	1	0.313	4,600	23	2	1	61	0.020		94	64	78	92
S	F	108	8	3.492	1.494	0.970	7.70	0.90	1	12.0	1.69E+06	13-May-98	1	1	0.188	4,600	25	2	1	24	0.011		56	38	47	92
S	F	109	8	3.502	1.498	0.981	7.44	0.87	1	11.7	1.18E+06	13-May-98	1	1	0.219	4,600	24	2	1	88	0.073		77	56	69	92
S	F	112	8	3.486	1.489	0.959	8.44	1.00	1	9.7	2.02E+06	12-May-98	2	2	0.063	4,600	30	2	1	5	0.003		19	13	16	92
S	F	114	8	3.495	1.492	0.967	7.64	0.93	1	10.1	1.47E+06	12-May-98	1	1	0.063	4,600	30	2	1	2	0.001		18	12	16	92
S	F	123	8	3.491	1.487	0.978	8.00	0.94	1	10.7	1.61E+06	12-May-98	1	1	0.125	4,600	27	2	1	5	0.002		36	24	31	92
S	F	124	8	3.484	1.499	0.978	8.73	1.04	1	9.7	1.79E+06	12-May-98	1	1	0.094	4,600	28	2	1	43	0.075		41	32	34	92
S	F	126	8	3.481	1.496	0.971	9.86	1.15	1	12.3	2.36E+06	11-May-98	1	1	0.094	4,600	28	2	1	26	0.028		32	23	26	92
S	F	131	8	3.486	1.494	0.969	9.73	1.13	1	12.7	2.29E+06	13-May-98	1	1	0.125	4,600	27	2	1	2	0.001		36	24	30	92
S	F	137	8	3.486	1.498	0.973	9.03	1.06	1	10.8	2.11E+06	12-May-98	1	1	0.094	4,600	28	2	1	8	0.005		26	19	23	92
S	F	140	8	3.489	1.493	0.968	9.19	1.07	1	12.3	1.85E+06	13-May-98	1	2	0.406	4,600	22	2	1	82	0.018		120	81	101	92
S	F	147	8	3.491	1.500	0.982	8.58	1.01	1	10.3	1.51E+06	11-May-98	1	2	0.375	4,600	23	2	1	111	0.063		120	84	102	92
S	F	149	8	3.487	1.498	0.973	8.38	0.97	1	12.6	2.01E+06	13-May-98	1	1	0.188	4,600	25	2	1	20	0.008		55	37	46	92
S	F	151	8	3.490	1.499	0.980	9.18	1.08	1	10.9	1.46E+06	11-May-98	1	1	0.094	4,600	29	2	1	19	0.021		31	22	27	92
S	F	152	8	3.491	1.498	0.974	8.35	0.99	1	10.1	1.66E+06	13-May-98	1	1	0.125	4,600	27	2	1	29	0.027		41	29	35	92
S	M	10	8	3.472	1.453	0.888	9.95	1.14	1	13.4	2.09E+06	15-May-98	1	1	0.125	4,600	27	2	1	34	0.035		43	31	35	92
S	M	21	8	3.435	1.472	0.913	7.99	0.94	1	11.2	1.71E+06	15-May-98	1	1	0.125	4,600	27	2	1	9	0.005		37	25	31	92
S	M	34	8	3.474	1.484	0.946	7.76	0.90	1	13.1	1.62E+06	15-May-98	1	1	0.188	4,600	25	2	1	54	0.042		62	44	53	92
S	M	35	8	3.436	1.464	0.898	7.62	0.89	1	12.1	1.71E+06	15-May-98	1	1	0.188	4,600	25	2	1	66	0.065		66	48	56	92
S	M	37	8	3.507	1.461	0.949	6.63	0.78	1	12.2	1.34E+06	15-May-98	1	1	0.313	4,600	23	2	1	71	0.026		95	65	83	92
S	M	41	8	3.481	1.456	0.899	7.96	0.91	1	13.8	1.47E+06	15-May-98	2	2	0.063	4,600	30	2	1	5	0.004		19	13	16	92
S	M	42	8	3.448	1.442	0.862	10.03	1.14	1	15.0	2.73E+06	15-May-98	1	2	0.250	4,600	24	2	1	39	0.016		75	51	62	92
S	M	46	8	3.465	1.466	0.913	8.28	0.96	1	13.0	1.73E+06	15-May-98	1	1	0.188	4,600	25	2	1	49	0.034		60	42	51	92
S	M	50	8	3.472	1.460	0.938	10.60	1.22	1	12.9	2.40E+06	15-May-98	1	1	0.250	4,600	24	2	1	51	0.024		76	53	63	92
S	M	52	8	3.464	1.470	0.917	9.34	1.07	1	14.2	1.86E+06	15-May-98	1	1	0.063	4,600	30	2	1	6	0.007		19	13	16	92
S	M	60	8	3.442	1.434	0.846	10.06	1.15	1	14.8	2.26E+06	15-May-98	1	1	0.125	4,600	30	2	1	19	0.024		41	29	33	92
S	M	61	8	3.479	1.446	0.877	8.39	0.97	1	12.5	1.45E+06	15-May-98	1	1	0.094	4,600	28	2	1	39	0.058		38	29	34	92
S	M	63	8	3.422	1.444	0.859	8.41	0.97	1	12.7	1.87E+06	14-May-98	1	1	0.063	4,600	30	2	1	9	0.009		20	14	17	92
S	M	76	8	3.437	1.470	0.910	8.28	0.96	1	12.4	2.04E+06	15-May-98	1	1	0.063	4,600	30	2	1	11	0.012		20	14	17	92
S	M	90	8	3.475	1.449	0.881	8.94	1.03	1	13.2	1.88E+06	15-May-98	1	2	0.219	4,600	24	2	1	47	0.022		67	46	57	92
S	M	96	8	3.393	1.419	0.806	10.22	1.18	1	13.4	2.86E+06	15-May-98	1	1	0.375	4,600	23	2	1	84	0.022		112	78	93	92
S	M	105	8	3.462	1.421	0.828	8.84	1.00	1	12.8	2.10E+06	17-May-98	1	2	0.188	4,600	25	2	1	63	0.058		65	47	54	92

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	M	107	8	3.456	1.468	0.908	8.44	0.98	1	12.9	2.14E+06	11-May-98	1	1	0.156	4,600	26	2	1	33	0.025		50	35	41	92
S	M	108	8	3.509	1.476	0.940	7.67	0.90	1	11.2	1.97E+06	11-May-98	2	2	0.125	4,600	27	2	1	16	0.010		38	26	32	92
S	M	109	8	3.458	1.461	0.899	7.97	0.93	1	11.5	1.89E+06	11-May-98	1	1	0.219	4,600	25	2	1	50	0.036		70	49	59	92
S	M	110	8	3.490	1.475	0.933	6.83	0.80	1	10.8	1.46E+06	11-May-98	1	1	0.156	4,600	26	2	1	54	0.061		57	42	49	92
S	M	119	8	3.472	1.473	0.925	8.47	0.97	1	13.8	1.52E+06	15-May-98	1	1	0.188	4,600	25	2	1	41	0.025		59	41	51	92
S	M	122	8	3.472	1.456	0.893	7.97	0.92	1	12.7	1.43E+06	15-May-98	1	1	0.438	4,600	22	2	1	126	0.034		132	90	115	92
S	M	127	8	3.478	1.472	0.924	8.60	0.98	1	14.0	1.42E+06	15-May-98	1	1	0.250	4,600	24	2	1	58	0.032		78	54	68	92
S	M	132	8	3.477	1.486	0.951	9.40	1.08	1	14.0	1.58E+06	15-May-98	1	1	0.438	4,600	22	2	1	91	0.019		129	87	111	92
S	M	139	8	3.469	1.470	0.924	8.03	0.93	1	13.2	1.49E+06	17-May-98	1	1	0.500	4,600	22	2	1	131	0.036		151	103	130	92
S	M	141	8	3.478	1.469	0.918	7.98	0.94	1	11.2	1.60E+06	15-May-98	1	1	0.125	4,600	27	2	1	13	0.008		37	25	32	92
S	M	143	8	3.468	1.484	0.944	7.05	0.83	1	10.6	1.48E+06	11-May-98	1	1	0.188	4,600	25	2	1	38	0.021		58	40	50	92
S	M	144	8	3.458	1.472	0.919	7.59	0.88	1	12.7	1.32E+06	11-May-98	1	1	0.188	4,600	25	2	1	46	0.031		60	42	53	92
S	M	146	8	3.440	1.473	0.916	8.93	1.03	1	12.8	2.02E+06	11-May-98	1	1	0.188	4,600	25	2	1	28	0.012		56	38	47	92
S	M	147	8	3.396	1.468	0.892	7.89	0.91	1	10.2	1.82E+06	11-May-98	1	1	0.156	4,600	26	2	1	49	0.051		55	40	46	92
S	M	148	8	3.426	1.463	0.931	7.30	0.85	1	11.6	1.71E+06	14-May-98	1	1	0.168	4,600	25	2	1	31	0.017		57	39	49	92
T	D	11	8	3.528	1.511	1.014	7.76	0.90	1	11.9	1.10E+06	17-May-98	1	1	0.250	3,400	26	2	1	37	0.031		58	40	49	68
T	D	13	8	3.433	1.501	0.967	7.52	0.89	2	10.1	1.61E+06	17-May-98	1	1	0.281	3,400	25	2	1	40	0.024		63	43	52	68
T	D	14	8	3.482	1.499	0.977	8.33	0.97	2	11.7	1.75E+06	18-May-98	1	1	0.125	3,400	30	2	1	3	0.001		27	18	22	68
T	D	22	8	3.518	1.495	0.980	7.39	0.87	1	11.3	1.57E+06	18-May-98	1	1	0.125	3,400	30	2	1	22	0.032		31	22	25	68
T	D	23	8	3.530	1.502	0.997	8.10	0.94	1	12.3	1.51E+06	18-May-98	1	1	0.313	3,400	24	2	1	61	0.033		71	49	59	68
T	D	28	8	3.466	1.500	0.975	8.71	1.02	2	11.4	2.27E+06	18-May-98	1	1	0.188	3,400	27	2	1	24	0.019		43	29	34	68
T	D	29	8	3.436	1.486	0.940	10.60	1.17	2	17.8	2.60E+06	17-May-98	1	1	0.188	3,400	27	2	1	26	0.022		43	30	34	68
T	D	30	8	3.418	1.491	0.944	10.66	1.16	2	19.8	2.74E+06	17-May-98	1	2	0.188	3,400	27	2	1	32	0.032		44	31	34	68
T	D	31	8	3.524	1.509	1.009	8.34	0.99	1	9.6	1.67E+06	18-May-98	2	2	0.094	3,400	30	2	1	3	0.002		20	14	17	68
T	D	35	8	3.514	1.505	0.998	7.84	0.92	1	11.5	1.34E+06	18-May-98	1	1	0.094	3,400	30	2	1	2	0.001		20	13	17	68
T	D	38	8	3.517	1.493	0.975	8.29	0.96	1	13.2	1.11E+06	18-May-98	2	2	0.031	3,400	30	2	1	8	0.007		8	5	7	68
T	D	39	8	3.427	1.510	0.983	9.98	1.15	1	12.7	2.51E+06	18-May-98	2	2	0.031	3,400	30	2	1	1	0.001		7	4	6	68
T	D	43	8	3.500	1.506	0.996	10.47	1.19	2	14.6	2.31E+06	17-May-98	1	1	0.313	3,400	24	2	1	68	0.039		72	50	57	68
T	D	44	8	3.513	1.449	0.891	9.47	1.09	1	13.0	2.09E+06	18-May-98	1	1	0.219	3,400	26	2	1	30	0.021		49	34	40	68
T	D	49	8	3.490	1.494	0.970	10.95	1.27	1	12.7	2.22E+06	18-May-98	1	1	0.188	3,400	27	2	1	30	0.028		44	31	35	68
T	D	51	8	3.491	1.487	0.957	9.55	1.11	2	12.2	2.60E+06	18-May-98	1	1	0.125	3,400	30	2	1	15	0.018		29	20	23	68
T	D	54	8	3.509	1.501	0.989	9.63	1.13	2	11.3	2.43E+06	17-May-98	1	1	0.094	3,400	30	2	1	24	0.038		25	19	18	68
T	D	56	8	3.520	1.510	1.010	9.70	1.12	1	12.6	2.16E+06	18-May-98	1	2	0.250	3,400	28	2	1	37	0.052		60	43	47	68
T	D	58	8	3.521	1.500	0.990	8.59	1.00	1	12.1	1.90E+06	17-May-98	1	1	0.094	3,400	26	2	1	3	0.002		20	14	17	68
T	D	61	8	3.485	1.505	0.990	7.99	0.94	1	10.5	1.89E+06	18-May-98	1	1	0.125	3,400	30	2	1	23	0.034		31	23	25	68
T	D	65	8	3.501	1.492	0.969	8.88	1.04	1	10.9	2.06E+06	18-May-98	1	1	0.125	3,400	30	2	1	10	0.009		28	19	23	68
T	D	73	8	3.469	1.480	0.937	8.43	0.99	1	11.6	1.57E+06	18-May-98	1	2	0.188	3,400	27	2	1	26	0.022		43	30	36	68
T	D	78	8	3.520	1.507	1.004	7.71	0.90	1	11.9	1.54E+06	18-May-98	1	1	0.125	3,400	30	2	1	2	0.001		27	18	22	68
T	D	83	8	3.449	1.466	0.906	8.53	0.98	2	13.1	1.92E+06	17-May-98	1	2	0.063	3,400	30	2	1	12	0.012		15	11	12	68
T	D	85	8	3.474	1.482	0.942	8.94	1.02	2	14.4	1.83E+06	17-May-98	1	1	0.168	3,400	27	2	1	30	0.027		44	30	35	68
T	D	91	8	3.432	1.457	0.885	8.92	1.03	1	13.4	2.24E+06	17-May-98	2	2	0.125	3,400	30	2	1	14	0.015		29	20	23	68
T	D	93	8	3.461	1.486	0.946	9.00	1.03	2	14.2	1.80E+06	17-May-98	1	2	0.219	3,400	26	2	1	32	0.023		50	34	41	68
T	D	94	8	3.447	1.468	0.909	7.61	0.88	1	12.5	1.28E+06	17-May-98	1	1	0.500	3,400	23	2	1	68	0.038		112	78	95	68
T	D	95	8	3.407	1.471	0.904	8.72	1.01	1	13.0	1.79E+06	17-May-98	1	1	0.188	3,400	27	2	1	44	0.057		48	35	38	68
T	D	96	8	3.382	1.456	0.870	8.41	0.98	2	11.5	2.16E+06	18-May-98	2	2	0.125	3,400	30	2	1	13	0.014		29	20	23	68
T	D	102	8	3.422	1.474	0.913	8.64	1.01	2	11.4	2.09E+06	18-May-98	1	1	0.063	3,400	30	2	1	2	0.001		13	8	11	68
T	D	105	8	3.458	1.460	0.898	8.10	0.91	1	18.4	1.70E+06	18-May-98	1	2	0.281	3,400	25	2	1	48	0.031		64	44	53	68
T	D	113	8	3.437	1.468	0.902	9.68	1.10	1	14.8	2.10E+06	18-May-98	1	1	0.188	3,400	27	2	1	41	0.048		47	33	37	68
T	D	114	8	3.462	1.464	0.948	8.96	1.02	2	14.3	1.52E+06	17-May-98	1	2	0.375	3,400	24	2	1	96	0.092		93	66	76	68
T	D	119	8	3.494	1.479	0.942	7.60	0.89	2	11.6	1.27E+06	17-May-98	1	2	0.250	3,400	26	2	1	41	0.035		58	40	49	68
T	D	122	8	3.474	1.492	0.962	9.27	1.08	2	12.0	2.32E+06	17-May-98	2	2	0.063	3,400	30	2	1	1	0.001		13	9	11	68
T	D	126	8	3.452	1.468	0.910	9.01	1.05	2	11.7	2.27E+06	17-May-98	1	1	0.313	3,400	25	2	1	32	0.017		69	47	56	68

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta₀ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	D	130	8	3.492	1.474	0.932	7.76	0.90	2	12.5	1.22E+06	17-May-98	1	1	0.375	3,400	24	2	1	67	0.040		86	59	73	68
T	D	131	8	3.462	1.478	0.931	8.10	0.93	2	13.5	1.75E+06	17-May-98	1	1	0.188	3,400	27	2	1	20	0.015		42	29	35	68
T	D	132	8	3.452	1.460	0.895	8.76	1.00	2	14.2	1.81E+06	17-May-98	1	1	0.188	3,400	27	2	1	34	0.033		45	31	36	68
T	D	133	8	3.456	1.474	0.922	8.92	1.02	2	13.5	2.15E+06	18-May-98	1	1	0.125	3,400	30	2	1	28	0.048		33	25	25	68
T	D	135	8	3.426	1.452	0.874	8.26	0.95	2	13.0	2.32E+06	17-May-98	1	1	0.125	3,400	30	2	1	14	0.015		29	20	23	68
T	D	138	8	3.480	1.520	1.018	9.28	1.07	2	13.3	1.93E+06	18-May-98	1	1	0.125	3,400	30	2	1	11	0.011		28	19	23	68
T	D	139	8	3.492	1.478	0.940	8.03	0.93	2	12.9	1.29E+06	17-May-98	1	1	0.406	3,400	24	2	1	71	0.045		93	64	78	68
T	D	149	8	3.456	1.466	0.907	9.27	1.05	2	14.7	1.93E+06	17-May-98	1	2	0.250	3,400	26	2	1	39	0.033		58	40	47	68
T	F	4	8	3.496	1.501	0.985	8.98	1.06	1	10.7	1.35E+06	13-May-98	1	2	0.438	3,400	23	2	1	52	0.018		95	64	80	68
T	F	11	8	3.472	1.492	0.961	9.08	1.06	1	10.1	2.02E+06	11-May-98	1	1	0.250	3,400	26	2	1	46	0.045		60	42	47	68
T	F	15	8	3.484	1.493	0.966	9.11	1.08	1	11.6	1.84E+06	11-May-98	1	1	0.125	3,400	30	2	1	20	0.026		30	21	24	68
T	F	16	8	3.498	1.492	0.968	9.12	1.08	1	10.1	2.14E+06	11-May-98	1	1	0.125	3,400	30	2	1	7	0.007		28	19	22	68
T	F	24	8	3.492	1.501	0.984	8.36	0.99	1	10.3	1.56E+06	11-May-98	1	1	0.032	3,400	30	2	1	3	0.002		7	5	6	68
T	F	26	8	3.490	1.496	0.974	8.39	0.99	1	10.1	1.55E+06	11-May-98	1	1	0.125	3,400	30	2	1	21	0.029		31	22	25	68
T	F	37	8	3.490	1.493	0.968	8.77	1.04	1	10.3	1.58E+06	11-May-98	1	1	0.125	3,400	30	2	1	36	0.083		38	29	30	68
T	F	39	8	3.490	1.498	0.974	8.63	1.01	1	11.8	1.97E+06	11-May-98	1	1	0.219	3,400	26	2	1	46	0.046		53	38	42	68
T	F	46	8	3.496	1.490	0.964	10.85	1.28	1	10.9	2.24E+06	11-May-98	1	1	0.156	3,400	28	2	1	4	0.002		33	22	28	68
T	F	51	8	3.480	1.494	0.967	8.04	0.95	1	10.0	1.72E+06	11-May-98	1	1	0.156	3,400	28	2	1	29	0.034		38	27	31	68
T	F	69	8	3.468	1.491	0.963	8.06	0.95	1	10.5	1.98E+06	11-May-98	1	1	0.188	3,400	27	2	1	40	0.047		47	33	38	68
T	F	73	8	3.492	1.492	0.966	8.21	0.97	1	10.1	1.39E+06	11-May-98	1	1	0.313	3,400	25	2	1	74	0.064		78	56	65	68
T	F	76	8	3.481	1.494	0.970	9.17	1.09	1	10.1	1.91E+06	13-May-98	1	1	0.188	3,400	27	2	1	26	0.021		43	30	35	68
T	F	79	8	3.473	1.496	0.969	10.48	1.24	1	10.8	2.21E+06	11-May-98	1	1	0.188	3,400	27	2	1	20	0.015		42	29	34	68
T	F	83	8	3.483	1.494	0.968	9.10	1.08	1	11.6	2.23E+06	11-May-98	1	1	0.125	3,400	30	2	1	24	0.038		32	23	24	68
T	F	86	8	3.500	1.495	0.975	8.35	0.99	1	9.8	1.76E+06	11-May-98	1	2	0.188	3,400	27	2	1	21	0.015		42	29	35	68
T	F	90	8	3.496	1.496	0.975	8.69	1.03	1	10.2	2.17E+06	11-May-98	1	2	0.063	3,400	30	2	1	14	0.016		16	11	12	68
T	F	100	8	3.490	1.493	0.968	7.91	0.93	1	10.5	1.93E+06	11-May-98	1	1	0.156	3,400	28	2	1	30	0.035		38	27	30	68
T	F	102	8	3.490	1.489	0.960	9.41	1.09	1	12.3	1.79E+06	11-May-98	1	2	0.344	3,400	24	2	1	83	0.033		78	53	64	68
T	F	103	8	3.482	1.498	0.971	9.77	1.13	1	12.3	2.47E+06	11-May-98	1	1	0.125	3,400	30	2	1	17	0.021		30	21	23	68
T	F	104	8	3.492	1.494	0.970	8.31	0.99	1	9.7	1.64E+06	11-May-98	2	2	0.094	3,400	30	2	1	12	0.012		22	15	18	68
T	F	107	8	3.500	1.498	0.977	8.01	0.94	1	11.0	1.48E+06	13-May-98	1	1	0.125	3,400	29	2	1	19	0.021		30	21	24	68
T	F	109	8	3.466	1.500	0.960	7.92	0.94	1	10.2	1.53E+06	11-May-98	1	1	0.188	3,400	27	2	1	22	0.017		42	29	35	68
T	F	110	8	3.476	1.490	0.958	9.86	1.16	1	10.4	2.62E+06	11-May-98	1	2	0.125	3,400	30	2	1	20	0.026		30	21	23	68
T	F	116	8	3.490	1.490	0.962	8.10	0.96	1	10.1	1.86E+06	11-May-98	1	1	0.125	3,400	30	2	1	15	0.017		29	20	23	68
T	F	120	8	3.490	1.488	0.958	9.59	1.14	1	10.1	1.70E+06	11-May-98	1	1	0.125	3,400	30	2	1	17	0.022		30	21	24	68
T	F	121	8	3.490	1.486	0.954	10.26	1.19	1	12.9	1.75E+06	11-May-98	1	2	0.438	3,400	23	2	1	71	0.028		97	66	80	68
T	F	131	8	3.430	1.489	0.944	8.87	1.03	1	12.1	2.08E+06	11-May-98	1	1	0.219	3,400	26	2	1	38	0.028		50	35	41	68
T	F	136	8	3.468	1.494	0.964	10.75	1.25	1	12.4	2.39E+06	13-May-98	1	1	0.281	3,400	25	2	1	32	0.017		62	42	50	68
T	F	137	8	3.498	1.498	0.975	8.53	1.02	1	9.5	1.86E+06	13-May-98	1	2	0.250	3,400	25	2	1	33	0.018		56	38	46	68
T	F	141	8	3.483	1.489	0.958	9.40	1.11	1	10.9	2.06E+06	13-May-98	1	2	0.188	3,400	27	2	1	25	0.021		43	30	35	68
T	F	144	8	3.479	1.490	0.959	11.66	1.37	1	11.2	2.32E+06	13-May-98	1	2	0.281	3,400	25	2	1	43	0.028		64	44	51	68
T	M	18	8	3.446	1.448	0.872	8.99	1.03	1	13.8	1.71E+06	14-May-98	1	2	0.375	3,400	24	2	1	57	0.030		84	57	70	68
T	M	19	8	3.480	1.464	0.948	7.82	0.92	1	11.2	1.05E+06	14-May-98	1	2	0.375	3,400	24	1	2	45	0.019	Column failure in s-bending at proof load	82	56	71	68
T	M	20	8	3.498	1.460	0.907	7.24	0.84	1	12.2	1.30E+06	14-May-98	2	2	0.063	3,400	30	2	1	4	0.002		14	9	12	68
T	M	25	8	3.398	1.482	0.922	7.77	0.90	1	12.9	1.47E+06	14-May-98	1	1	0.125	3,400	30	2	1	28	0.050		34	25	28	68
T	M	32	8	3.472	1.472	0.923	7.83	0.91	1	12.0	1.28E+06	14-May-98	1	1	0.188	3,400	28	2	1	18	0.013		42	28	35	68
T	M	37	8	3.462	1.457	0.892	7.79	0.90	1	12.8	1.58E+06	13-May-98	1	1	0.219	3,400	28	2	1	32	0.023		50	34	41	68
T	M	40	8	3.483	1.481	0.943	7.86	0.90	1	11.5	1.24E+06	13-May-98	1	1	0.313	3,400	24	2	1	54	0.027		70	48	60	68
T	M	44	8	3.430	1.462	0.893	7.84	0.92	1	11.2	1.71E+06	13-May-98	1	1	0.250	3,400	28	2	1	29	0.020		56	38	46	68
T	M	53	8	3.452	1.476	0.925	7.93	0.92	1	12.2	1.15E+06	14-May-98	2	2	0.125	3,400	30	2	1	11	0.012		28	19	24	68
T	M	57	8	3.462	1.493	0.960	8.15	0.96	1	10.5	1.55E+06	14-May-98	1	1	0.125	3,400	30	2	1	22	0.031		31	22	25	68
T	M	61	8	3.472	1.482	0.942	7.26	0.85	1	12.0	1.39E+06	13-May-98	1	1	0.250	3,400	26	2	1	23	0.014		55	37	46	68
T	M	70	8	3.429	1.502	0.968	8.68	1.01	1	12.5	1.63E+06	13-May-98	1	1	0.250	3,400	26	2	1	38	0.031		58	40	47	68

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	M	73	8	3.456	1.489	0.951	7.95	0.93	1	11.6	1.19E+06	13-May-98	1	2	0.313	3,400	24	2	1	42	0.018		69	47	59	68
T	M	74	8	3.479	1.470	0.921	8.10	0.94	1	12.2	1.15E+06	13-May-98	1	2	0.375	3,031	24	1	3	88	0.073	Column failure in s-bending at proof load	80	57	68	61
T	M	77	8	3.472	1.462	0.904	8.74	0.79	1	10.8	1.26E+06	13-May-98	2	2	0.188	3,400	27	1	4	64	0.141	Brace failure	80	47	50	68
T	M	79	8	3.412	1.459	0.883	8.14	0.95	1	11.4	1.54E+06	13-May-98	1	1	0.125	3,400	29	2	1	3	0.002		27	18	23	68
T	M	81	8	3.432	1.466	0.901	8.22	0.97	1	10.6	1.48E+06	13-May-98	1	1	0.375	3,400	24	2	1	50	0.024		83	57	70	68
T	M	82	8	3.387	1.471	0.898	7.02	0.82	1	11.8	1.39E+06	13-May-98	1	1	0.125	3,400	30	2	1	15	0.018		29	20	25	68
T	M	83	8	3.440	1.476	0.922	8.27	0.97	1	11.2	1.57E+06	13-May-98	1	2	0.125	3,400	30	2	1	31	0.061		35	26	28	68
T	M	87	8	3.441	1.456	0.888	8.76	1.00	1	14.0	1.58E+06	14-May-98	1	1	0.531	3,400	23	2	1	90	0.040		119	81	99	68
T	M	97	8	3.493	1.486	0.955	7.90	0.93	1	11.1	1.19E+06	13-May-98	1	2	0.188	3,400	27	2	1	49	0.073		50	37	43	68
T	M	105	8	3.506	1.468	0.925	8.37	0.95	1	14.5	1.36E+06	14-May-98	1	1	0.438	3,400	23	2	1	43	0.012		95	64	80	68
T	M	107	8	3.457	1.472	0.919	8.60	1.01	1	11.5	1.43E+06	14-May-98	1	1	0.063	3,400	24	2	1	38	0.015		15	11	13	68
T	M	112	8	3.518	1.464	0.958	8.16	0.94	1	13.4	1.41E+06	14-May-98	1	1	0.281	3,400	25	2	1	29	0.015		62	42	52	68
T	M	116	8	3.530	1.477	0.948	8.92	1.04	1	12.0	1.69E+06	14-May-98	1	1	0.188	3,400	27	2	1	32	0.030		44	31	36	68
T	M	117	8	3.432	1.469	0.907	7.37	0.85	1	12.7	1.20E+06	14-May-98	1	1	0.125	3,400	30	2	1	8	0.007		28	19	24	68
T	M	121	8	3.470	1.483	0.943	8.49	0.96	1	13.0	1.15E+06	13-May-98	1	2	0.250	3,400	26	2	1	34	0.026		57	39	49	68
T	M	123	8	3.466	1.420	0.827	9.01	1.05	1	12.2	2.13E+06	13-May-98	1	1	0.156	3,400	30	2	1	13	0.013		35	24	29	68
T	M	130	8	3.572	1.504	1.013	9.74	1.11	1	14.6	1.79E+06	14-May-98	1	2	0.219	3,400	26	2	1	31	0.022		50	34	40	68
T	M	134	8	3.372	1.465	0.884	8.26	0.95	1	13.6	1.58E+06	14-May-98	1	2	0.125	3,400	30	2	1	18	0.023		30	21	25	68
T	M	140	8	3.398	1.442	0.849	7.89	0.92	1	12.3	1.94E+06	14-May-98	1	2	0.313	3,400	26	2	1	27	0.018		69	47	57	68
T	M	147	8	3.427	1.492	0.949	7.54	0.88	1	12.4	1.10E+06	14-May-98	1	1	0.125	3,400	30	2	1	6	0.005		27	18	23	68
S	D	1	10	3.492	1.480	0.943	12.25	1.09	1	16.1	1.84E+06	08-May-98	1	1	0.063	3,000	40	2	1	2	0.001		9	6	8	60
S	D	4	10	3.454	1.474	0.922	11.31	1.01	1	15.7	2.34E+06	10-May-98	1	1	0.188	3,000	39	2	1	26	0.043		32	23	26	60
S	D	6	10	3.465	1.486	0.947	11.07	0.99	1	15.3	2.01E+06	10-May-98	1	1	0.125	3,000	40	2	1	27	0.058		25	18	20	60
S	D	14	10	3.504	1.476	0.939	10.57	0.97	1	12.8	1.56E+06	10-May-98	1	1	0.750	3,000	34	2	1	93	0.119		124	87	107	60
S	D	22	10	3.468	1.456	0.896	10.93	0.96	1	15.1	1.85E+06	10-May-98	1	1	0.375	3,000	35	2	1	43	0.036		60	41	51	60
S	D	35	10	3.474	1.453	0.888	10.82	0.98	1	13.6	1.73E+06	09-May-98	1	1	0.250	3,000	37	2	1	24	0.024		40	27	34	60
S	D	37	10	3.421	1.466	0.898	11.37	1.04	1	13.3	1.87E+06	09-May-98	1	1	0.313	3,000	36	2	1	31	0.028		50	34	42	60
S	D	39	10	3.456	1.471	0.917	9.92	0.91	1	12.4	1.63E+06	09-May-98	1	1	0.125	3,000	40	2	1	1	0.001		19	13	16	60
S	D	41	10	3.492	1.472	0.928	10.70	0.96	1	15.1	1.78E+06	08-May-98	1	1	0.125	3,000	40	2	1	17	0.026		21	15	18	60
S	D	53	10	3.484	1.480	0.941	11.21	1.01	1	14.6	2.22E+06	08-May-98	2	2	0.125	3,000	40	2	1	1	0.001		19	13	16	60
S	D	64	10	3.440	1.478	0.928	10.00	0.92	1	12.2	1.72E+06	08-May-98	1	1	0.313	3,000	36	2	1	33	0.031		50	34	42	60
S	D	69	10	3.470	1.458	0.896	11.21	1.01	1	14.6	1.98E+06	08-May-98	1	1	0.438	3,000	34	2	1	39	0.029		69	47	58	60
S	D	71	10	3.479	1.481	0.942	10.39	0.94	1	14.2	2.03E+06	10-May-98	1	1	0.156	3,000	40	2	1	14	0.021		26	18	21	60
S	D	72	10	3.504	1.498	0.982	9.61	0.87	1	14.0	1.41E+06	10-May-98	1	1	0.156	3,000	40	2	1	15	0.023		26	18	22	60
S	D	86	10	3.502	1.493	0.971	9.79	0.89	1	14.2	1.43E+06	09-May-98	1	1	0.313	3,000	36	2	1	47	0.080		53	37	46	60
S	D	87	10	3.426	1.448	0.867	12.24	1.11	1	14.3	2.57E+06	09-May-98	1	1	0.250	3,000	37	2	1	28	0.033		41	28	33	60
S	D	90	10	3.454	1.493	0.958	11.63	1.07	1	12.6	2.25E+06	09-May-98	1	1	0.250	3,000	37	2	1	32	0.041		42	29	34	60
S	D	92	10	3.500	1.490	0.965	10.12	0.93	1	12.9	1.89E+06	08-May-98	1	1	0.188	3,000	39	2	1	16	0.021		30	21	25	60
S	D	95	10	3.449	1.483	0.937	11.02	0.99	1	14.6	2.00E+06	08-May-98	1	1	0.188	3,000	39	2	1	9	0.009		29	20	24	60
S	D	96	10	3.469	1.492	0.966	9.43	0.86	1	13.6	1.60E+06	08-May-98	2	2	0.125	3,000	40	2	1	10	0.013		20	14	17	60
S	D	97	10	3.454	1.478	0.929	12.34	1.11	1	14.8	1.99E+06	08-May-98	1	1	0.313	3,000	35	2	1	23	0.013		48	33	41	60
S	D	99	10	3.457	1.482	0.938	9.98	0.90	1	14.6	1.56E+06	08-May-98	1	1	0.188	3,000	39	2	1	14	0.017		30	20	26	60
S	D	103	10	3.446	1.486	0.942	11.80	1.07	1	11.7	2.17E+06	10-May-98	2	2	0.188	3,000	39	2	1	12	0.012		29	20	24	60
S	D	107	10	3.479	1.472	0.925	13.17	1.18	1	15.0	2.58E+06	09-May-98	1	1	0.125	3,000	40	2	1	15	0.021		21	15	17	60
S	D	114	10	3.446	1.458	0.890	9.84	0.89	1	14.7	1.42E+06	08-May-98	1	1	0.563	3,000	33	2	1	59	0.024		87	59	76	60
S	D	116	10	3.470	1.484	0.945	9.48	0.86	1	13.6	1.35E+06	10-May-98	1	1	0.125	3,000	40	2	1	25	0.051		24	18	21	60
S	D	119	10	3.460	1.482	0.938	9.63	0.87	1	14.2	1.60E+06	08-May-98	1	1	0.188	3,000	39	2	1	20	0.028		31	22	26	60
S	D	122	10	3.399	1.482	0.922	10.95	0.98	1	15.3	2.02E+06	09-May-98	1	1	0.094	3,000	40	2	1	12	0.015		16	11	13	60
S	D	126	10	3.447	1.472	0.916	10.78	0.97	1	14.2	1.72E+06	09-May-98	1	1	0.313	3,000	36	2	1	31	0.028		50	34	42	60
S	D	130	10	3.443	1.490	0.949	10.94	1.00	1	13.2	1.88E+06	09-May-98	1	1	0.125	3,000	40	2	1	12	0.016		20	14	17	60
S	D	132	10	3.394	1.483	0.922	10.98	1.01	1	12.0	1.83E+06	08-May-98	1	1	0.125	3,000	40	2	1	11	0.013		20	14	17	60
S	D	133	10	3.374	1.482	0.915	10.61	0.97	1	13.5	2.18E+06	08-May-98	1	2	0.188	3,000	39	2	1	10	0.013		29	20	25	60

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
S	D	135	10	3.443	1.458	0.889	11.90	1.10	1	11.7	2.24E+06	06-May-98	1	1	0.563	3,000	33	2	1	75	0.035		68	60	73	60
S	F	7	10	3.494	1.493	0.969	13.04	1.23	1	9.9	2.58E+06	04-May-98	1	1	0.469	3,000	34	2	1	56	0.040		74	51	61	60
S	F	10	10	3.485	1.494	0.968	11.73	1.08	1	12.4	2.12E+06	04-May-98	2	2	0.094	3,000	40	2	1	1	0.001		14	9	12	60
S	F	12	10	3.493	1.489	0.961	11.80	1.09	1	10.4	2.47E+06	04-May-98	1	1	0.188	3,000	39	2	1	10	0.011		29	20	24	60
S	F	13	10	3.487	1.492	0.965	13.51	1.26	1	10.7	2.55E+06	05-May-98	2	2	0.188	3,000	39	2	1	4	0.004		29	19	24	60
S	F	15	10	3.488	1.491	0.963	13.61	1.25	1	14.3	2.58E+06	04-May-98	1	2	0.281	3,000	38	2	1	33	0.031		45	31	37	60
S	F	16	10	3.492	1.493	0.968	13.62	1.23	1	14.2	2.91E+06	04-May-98	1	1	0.188	3,000	39	2	1	14	0.017		30	20	24	60
S	F	17	10	3.494	1.496	0.975	11.56	1.06	1	10.6	2.15E+06	04-May-98	2	2	0.188	3,000	38	2	1	22	0.027		31	21	25	60
S	F	21	10	3.478	1.491	0.961	12.39	1.14	1	12.0	2.10E+06	04-May-98	1	1	0.188	3,000	38	2	1	17	0.019		30	21	25	60
S	F	24	10	3.458	1.492	0.957	13.16	1.21	1	12.3	2.88E+06	04-May-98	1	1	0.125	3,000	40	2	1	15	0.021		21	15	17	60
S	F	29	10	3.469	1.491	0.958	13.42	1.25	1	11.1	2.51E+06	05-May-98	1	1	0.188	3,000	38	2	1	16	0.017		30	20	24	60
S	F	31	10	3.478	1.491	0.960	12.58	1.18	1	10.3	2.67E+06	05-May-98	1	1	0.188	3,000	38	2	1	8	0.005		29	19	24	60
S	F	44	10	3.487	1.493	0.967	12.46	1.14	1	13.5	2.26E+06	04-May-98	1	2	0.313	3,000	36	2	1	40	0.044		51	36	42	60
S	F	48	10	3.490	1.497	0.976	9.65	0.93	1	9.8	1.91E+06	05-May-98	1	1	0.188	3,000	38	2	1	17	0.017		30	20	25	60
S	F	49	10	3.489	1.494	0.970	11.16	1.04	1	10.7	1.81E+06	05-May-98	1	2	0.250	3,000	37	2	1	23	0.022		40	27	33	60
S	F	51	10	3.492	1.497	0.976	10.47	0.96	1	12.3	1.80E+06	05-May-98	1	1	0.156	3,000	40	2	1	17	0.025		26	18	22	60
S	F	59	10	3.494	1.494	0.971	10.23	0.96	1	10.4	1.69E+06	05-May-98	1	1	0.188	3,000	38	2	1	24	0.032		31	22	26	60
S	F	62	10	3.486	1.488	0.957	11.63	1.09	1	10.9	2.02E+06	05-May-98	2	2	0.125	3,000	39	2	1	17	0.021		21	15	17	60
S	F	66	10	3.478	1.491	0.961	11.37	1.04	1	13.0	2.31E+06	05-May-98	2	2	0.188	3,000	39	2	1	13	0.015		30	20	24	60
S	F	67	10	3.492	1.498	0.978	10.63	0.99	1	11.5	1.79E+06	05-May-98	1	1	0.125	3,000	40	2	1	12	0.015		20	14	17	60
S	F	69	10	3.464	1.460	0.936	12.28	1.14	1	11.7	2.49E+06	05-May-98	1	1	0.094	3,000	40	2	1	1	0.001		14	9	12	60
S	F	73	10	3.478	1.494	0.967	12.21	1.12	1	13.1	1.87E+06	05-May-98	1	1	0.375	3,000	35	2	1	38	0.028		59	40	50	60
S	F	74	10	3.497	1.496	0.976	11.04	1.04	1	10.2	1.79E+06	04-May-98	1	2	0.469	3,000	34	2	1	59	0.042		75	51	63	60
S	F	76	10	3.473	1.491	0.959	11.85	1.09	1	12.0	2.25E+06	05-May-98	1	1	0.281	3,000	36	2	1	36	0.036		46	32	38	60
S	F	79	10	3.487	1.495	0.971	10.89	1.01	1	11.2	1.96E+06	05-May-98	1	2	0.250	3,000	37	2	1	32	0.040		42	29	34	60
S	F	83	10	3.487	1.490	0.961	10.45	0.96	1	12.3	1.76E+06	05-May-98	1	1	0.188	3,000	39	2	1	7	0.007		29	19	24	60
S	F	84	10	3.493	1.495	0.973	10.52	0.99	1	9.7	1.97E+06	05-May-98	1	1	0.438	3,000	34	2	1	47	0.027		68	46	57	60
S	F	89	10	3.496	1.496	0.975	11.75	1.10	1	10.4	2.43E+06	05-May-98	1	1	0.188	3,000	38	2	1	25	0.034		32	22	25	60
S	F	93	10	3.469	1.493	0.962	13.19	1.24	1	10.1	2.88E+06	05-May-98	1	1	0.125	3,000	40	2	1	4	0.003		19	13	18	60
S	F	96	10	3.490	1.496	0.974	12.12	1.12	1	11.6	2.03E+06	05-May-98	1	1	0.250	3,000	37	2	1	26	0.028		40	28	33	60
S	F	101	10	3.487	1.491	0.963	12.73	1.18	1	11.6	2.44E+06	05-May-98	2	2	0.188	3,000	39	2	1	9	0.009		29	20	24	60
S	F	111	10	3.495	1.493	0.969	11.05	1.02	1	12.4	1.71E+06	05-May-98	1	1	0.156	3,000	40	2	1	8	0.009		24	17	21	60
S	F	115	10	3.496	1.493	0.970	10.08	0.95	1	10.0	1.80E+06	05-May-98	1	2	0.500	3,000	34	2	1	78	0.073		62	57	69	60
S	F	118	10	3.494	1.494	0.971	12.78	1.18	1	11.4	2.85E+06	05-May-98	1	2	0.250	3,000	37	2	1	28	0.028		40	28	33	60
S	F	125	10	3.487	1.492	0.965	12.03	1.11	1	12.1	2.19E+06	05-May-98	1	1	0.125	3,000	40	2	1	18	0.023		21	15	17	60
S	F	130	10	3.489	1.494	0.970	11.63	1.09	1	12.2	2.29E+06	05-May-98	1	1	0.250	3,000	37	2	1	22	0.021		40	27	33	60
S	F	135	10	3.486	1.490	0.961	10.33	0.97	1	10.4	2.04E+06	05-May-98	1	1	0.250	3,000	37	2	1	26	0.027		40	28	33	60
S	F	138	10	3.493	1.496	0.975	13.68	1.25	1	12.8	2.28E+06	05-May-98	2	2	0.188	3,000	40	2	1	11	0.015		30	20	24	60
S	F	144	10	3.476	1.494	0.966	12.91	1.18	1	15.8	2.72E+06	05-May-98	2	2	0.188	3,000	38	2	1	15	0.014		30	20	24	60
S	F	150	10	3.492	1.493	0.968	10.21	0.96	1	9.5	1.70E+06	05-May-98	1	1	0.063	3,000	40	2	1	4	0.004		10	7	8	60
S	M	4	10	3.486	1.497	0.975	9.35	0.86	1	12.4	1.30E+06	06-May-98	1	1	0.188	3,000	39	2	1	27	0.048		33	24	29	60
S	M	6	10	3.484	1.490	0.960	12.64	1.17	1	13.7	2.24E+06	07-May-98	2	2	0.188	3,000	40	2	1	21	0.038		32	23	26	60
S	M	9	10	3.472	1.416	0.821	10.29	0.93	1	14.1	1.73E+06	06-May-98	1	1	0.125	3,000	40	2	1	2	0.002		19	13	18	60
S	M	11	10	3.411	1.475	0.912	10.19	0.91	1	18.2	1.78E+06	06-May-98	1	1	0.063	3,000	40	2	1	14	0.019		11	8	10	60
S	M	27	10	3.500	1.473	0.932	9.42	0.88	1	10.9	1.26E+06	06-May-98	2	2	0.063	3,000	40	2	1	1	0.001		9	6	8	60
S	M	28	10	3.484	1.486	0.953	9.81	0.89	1	13.5	1.57E+06	06-May-98	1	1	0.563	3,000	33	2	1	70	0.033		68	60	75	60
S	M	30	10	3.442	1.470	0.911	12.18	1.12	1	13.0	2.30E+06	06-May-98	1	2	0.375	3,000	35	2	1	60	0.072		63	45	52	60
S	M	44	10	3.453	1.460	0.896	9.70	0.89	1	13.3	1.85E+06	07-May-98	1	1	0.188	3,000	39	2	1	14	0.017		30	20	25	60
S	M	46	10	3.463	1.460	0.898	10.58	0.97	1	13.0	1.92E+06	07-May-98	1	1	0.250	3,000	37	2	1	37	0.053		43	30	36	60
S	M	54	10	3.493	1.486	0.955	11.14	1.00	1	15.1	1.68E+06	06-May-98	1	1	0.125	3,000	40	2	1	4	0.004		19	13	18	60
S	M	56	10	3.456	1.474	0.922	11.45	1.03	1	14.7	2.09E+06	06-May-98	1	1	0.313	3,000	36	2	1	35	0.035		50	35	42	60
S	M	57	10	3.436	1.455	0.882	10.84	0.99	1	14.2	2.13E+06	06-May-98	1	1	0.125	3,000	40	2	1	20	0.034		22	18	18	60

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Talen (lb)	2% Rule (lb)
S	M	62	10	3.483	1.486	0.952	7.77	0.73	1	10.7	1.21E+06	07-May-98	1	1	0.500	3,000	34	2	1	55	0.036		79	54	70	60
S	M	65	10	3.480	1.462	0.906	10.56	0.96	1	14.2	1.92E+06	07-May-98	1	1	0.125	3,000	40	2	1	18	0.028		22	15	18	60
S	M	67	10	3.444	1.462	0.897	12.40	1.11	1	15.8	2.55E+06	07-May-98	1	1	0.188	3,000	39	2	1	14	0.015		30	20	24	60
S	M	68	10	3.418	1.470	0.905	12.76	1.18	1	13.6	2.44E+06	07-May-98	1	1	0.188	3,000	39	2	1	20	0.029		31	22	25	60
S	M	70	10	3.464	1.466	0.909	11.02	1.01	1	12.8	1.83E+06	07-May-98	1	1	0.313	3,000	36	2	1	42	0.048		52	36	44	60
S	M	71	10	3.461	1.462	0.939	11.14	1.00	1	15.0	1.90E+06	07-May-98	1	1	0.188	3,000	39	2	1	9	0.009		29	20	24	60
S	M	74	10	3.492	1.489	0.961	11.55	1.05	1	14.3	1.35E+06	07-May-98	1	1	0.313	3,000	36	2	1	57	0.097		57	41	50	60
S	M	75	10	3.476	1.495	0.968	8.18	0.77	1	10.3	1.24E+06	07-May-98	1	1	0.250	3,000	37	2	1	18	0.016		39	27	34	60
S	M	77	10	3.486	1.464	0.912	10.52	0.95	1	14.4	1.39E+06	07-May-98	1	1	0.313	3,000	36	2	1	53	0.061		55	39	48	60
S	M	80	10	3.480	1.464	0.910	10.11	0.93	1	12.7	1.64E+06	08-May-98	1	1	0.438	3,000	34	2	1	52	0.033		69	47	59	60
S	M	81	10	3.472	1.472	0.923	10.80	0.98	1	13.5	1.58E+06	07-May-98	1	1	0.438	3,000	34	2	1	60	0.044		70	48	60	60
S	M	82	10	3.470	1.490	0.957	9.46	0.87	1	11.9	1.41E+06	08-May-98	1	1	1.438	3,000	31	2	1	175	0.020		218	149	188	60
S	M	85	10	3.438	1.442	0.859	13.64	1.26	1	11.7	2.47E+06	07-May-98	1	1	0.188	3,000	39	2	1	22	0.033		31	22	26	60
S	M	89	10	3.455	1.428	0.838	10.94	0.99	1	14.4	2.18E+06	07-May-98	2	2	0.063	3,000	40	2	1	4	0.004		10	7	8	60
S	M	91	10	3.463	1.469	0.913	9.01	0.83	1	12.2	1.67E+06	08-May-98	1	1	0.188	3,000	39	2	1	15	0.019		30	21	26	60
S	M	92	10	3.463	1.461	0.905	9.85	0.90	1	12.9	1.58E+06	08-May-98	1	1	0.188	3,000	39	2	1	25	0.040		32	23	28	60
S	M	95	10	3.405	1.461	0.885	10.60	0.97	1	13.5	1.91E+06	07-May-98	1	1	0.188	3,000	39	2	1	14	0.015		30	20	25	60
S	M	99	10	3.409	1.466	0.895	11.63	1.07	1	13.9	2.55E+06	07-May-98	1	1	0.063	3,000	40	2	1	7	0.007		10	7	8	60
S	M	118	10	3.443	1.463	0.898	11.47	1.04	1	14.3	2.47E+06	08-May-98	1	1	0.438	3,000	34	2	1	60	0.042		70	48	58	60
S	M	134	10	3.575	1.494	0.993	12.38	1.11	1	14.7	1.62E+06	07-May-98	1	1	0.188	3,000	39	2	1	18	0.023		30	21	26	60
S	M	136	10	3.492	1.463	0.949	10.82	0.99	1	13.4	1.74E+06	07-May-98	1	1	0.100	3,000	40	2	1	9	0.010		16	11	14	60
S	M	142	10	3.450	1.492	0.955	9.21	0.85	1	11.6	1.63E+06	07-May-98	1	1	0.188	3,000	39	2	1	18	0.023		30	21	28	60
S	M	149	10	3.493	1.480	0.944	9.37	0.87	1	12.0	1.18E+06	07-May-98	1	1	0.500	2,920	34	1	2	95	0.124	Column failure in S-bending	85	61	76	58
T	D	10	10	3.450	1.473	0.919	14.64	1.28	2	20.5	2.76E+06	09-May-98	1	1	0.219	2,200	40	2	1	19	0.031		26	18	20	44
T	D	15	10	3.490	1.507	0.995	11.26	1.05	2	11.1	1.66E+06	09-May-98	1	1	0.188	2,200	40	2	1	13	0.017		22	15	18	44
T	D	19	10	3.486	1.511	1.002	9.90	0.92	1	10.8	1.37E+06	10-May-98	1	1	0.125	2,200	40	2	1	10	0.012		15	10	12	44
T	D	34	10	3.530	1.515	1.023	10.06	0.94	1	11.2	1.19E+06	09-May-98	1	2	0.375	2,200	37	2	1	29	0.034		44	30	37	44
T	D	40	10	3.460	1.468	0.950	14.61	1.25	2	21.1	2.70E+06	09-May-98	1	1	0.063	2,200	40	2	1	2	0.002		7	5	6	44
T	D	41	10	3.506	1.520	1.026	11.76	1.09	1	11.5	1.89E+06	09-May-98	1	1	0.063	2,200	40	2	1	2	0.002		7	5	6	44
T	D	42	10	3.504	1.502	0.989	11.63	1.04	1	16.1	1.81E+06	09-May-98	2	2	0.063	2,200	40	2	1	1	0.001		7	5	6	44
T	D	47	10	3.516	1.496	0.981	11.91	1.07	1	14.9	2.11E+06	09-May-98	1	1	0.125	2,200	40	2	1	5	0.005		14	10	12	44
T	D	48	10	3.454	1.498	0.968	13.45	1.24	2	12.5	2.68E+06	09-May-98	1	1	0.313	2,200	38	2	1	25	0.034		37	25	29	44
T	D	50	10	3.500	1.496	0.977	13.18	1.18	2	15.2	2.70E+06	10-May-98	1	1	0.188	2,200	40	2	1	9	0.010		21	14	17	44
T	D	57	10	3.512	1.510	1.008	11.52	1.06	1	12.9	1.90E+06	10-May-98	1	1	0.125	2,200	40	2	1	7	0.008		14	10	12	44
T	D	59	10	3.520	1.506	1.002	12.13	1.11	1	13.5	2.10E+06	09-May-98	1	1	0.125	2,200	40	2	1	10	0.012		15	10	12	44
T	D	62	10	3.520	1.506	1.002	12.17	1.11	1	13.6	2.08E+06	09-May-98	1	1	0.125	2,200	40	2	1	7	0.008		14	10	12	44
T	D	64	10	3.487	1.512	1.004	10.25	0.95	2	11.8	1.74E+06	10-May-98	2	2	0.094	2,200	40	2	1	0	0.000		10	7	9	44
T	D	66	10	3.500	1.500	0.984	12.19	1.11	2	13.7	2.18E+06	09-May-98	1	1	0.125	2,200	40	2	1	12	0.016		15	10	12	44
T	D	70	10	3.516	1.506	1.001	13.31	1.19	2	15.9	2.04E+06	09-May-98	1	1	0.188	2,200	40	2	1	13	0.019		22	15	18	44
T	D	71	10	3.443	1.482	0.934	10.68	0.97	1	14.0	1.91E+06	09-May-98	1	1	0.063	2,200	40	2	1	1	0.001		7	5	6	44
T	D	84	10	3.456	1.473	0.920	12.03	1.07	1	15.9	2.02E+06	09-May-98	1	1	0.250	2,200	39	2	1	20	0.028		30	20	24	44
T	D	87	10	3.523	1.450	0.895	9.61	0.90	2	12.8	1.95E+06	09-May-98	2	2	0.063	2,200	40	2	1	7	0.007		7	5	6	44
T	D	98	10	3.482	1.469	0.958	8.75	0.80	1	12.5	1.24E+06	09-May-98	2	2	0.125	2,200	40	2	1	15	0.022		15	11	13	44
T	D	106	10	3.456	1.483	0.939	9.40	0.85	1	14.5	1.28E+06	10-May-98	1	1	0.250	2,200	39	2	1	19	0.026		29	20	25	44
T	D	109	10	3.410	1.480	0.921	9.01	0.84	1	11.4	1.50E+06	10-May-98	1	1	0.188	2,200	40	2	1	15	0.022		22	15	19	44
T	D	110	10	3.492	1.484	0.951	10.42	0.96	2	12.2	1.71E+06	09-May-98	1	1	0.250	2,200	38	2	1	10	0.012		28	19	24	44
T	D	120	10	3.460	1.480	0.935	10.25	0.93	1	14.0	1.86E+06	09-May-98	1	1	0.125	2,200	40	2	1	11	0.014		15	10	12	44
T	D	124	10	3.473	1.482	0.942	8.55	0.76	1	13.1	9.29E+05	09-May-98	1	1	0.250	2,200	39	2	1	25	0.040		30	21	27	44
T	D	129	10	3.467	1.486	0.948	11.62	1.06	1	14.9	2.06E+06	08-May-98	1	1	0.313	2,200	38	2	1	20	0.023		36	25	29	44
T	D	134	10	3.462	1.487	0.949	12.29	1.10	1	16.1	1.94E+06	09-May-98	1	1	0.125	2,200	40	2	1	7	0.007		14	10	12	44
T	D	136	10	3.480	1.483	0.946	9.91	0.90	1	13.6	1.54E+06	08-May-98	2	2	0.125	2,200	40	2	1	8	0.008		14	10	12	44
T	D	142	10	3.444	1.470	0.912	12.77	1.15	2	14.4	2.55E+06	08-May-98	1	1	0.063	2,200	40	2	1	3	0.002		7	5	6	44

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test:											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	$Delta_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tsien (lb)	2% Rule (lb)
T	D	145	10	3.452	1.470	0.914	11.68	1.05	2	14.4	1.91E+06	09-May-98	1	1	0.125	2,200	40	2	1	13	0.017		15	10	12	44
T	F	1	10	3.468	1.497	0.975	12.46	1.14	1	12.9	1.74E+06	04-May-98	1	1	0.094	2,200	40	2	1	0	0.000		10	7	9	44
T	F	2	10	3.489	1.493	0.968	10.17	0.96	1	10.0	1.73E+06	04-May-98	1	1	0.250	2,200	39	2	1	24	0.037		30	21	25	44
T	F	3	10	3.494	1.498	0.979	9.32	0.87	1	11.3	1.04E+06	20-Apr-98	1	2	0.281	2,200	38	2	1	21	0.025		33	22	28	44
T	F	5	10	3.475	1.500	0.977	12.25	1.13	1	11.6	2.11E+06	20-Apr-98	1	1	0.094	2,200	40	2	1	8	0.010		11	8	9	44
T	F	6	10	3.500	1.496	0.977	11.18	1.05	1	9.8	1.82E+06	05-May-98	1	1	0.313	2,200	38	2	1	33	0.057		39	27	31	44
T	F	10	10	3.498	1.498	0.979	10.90	1.01	1	11.1	1.79E+06	05-May-98	1	2	0.500	2,200	35	2	1	52	0.052		59	40	48	44
T	F	12	10	3.480	1.492	0.963	12.10	1.12	1	11.4	1.72E+06	05-May-98	1	1	1.188	2,200	32	2	1	104	0.031		133	89	111	44
T	F	14	10	3.463	1.491	0.957	12.93	1.20	1	11.1	1.83E+06	05-May-98	1	1	0.313	2,200	38	2	1	21	0.025		36	25	30	44
T	F	19	10	3.498	1.500	0.983	9.98	0.94	1	9.8	1.77E+06	05-May-98	1	1	0.344	2,200	37	2	1	21	0.020		39	27	32	44
T	F	25	10	3.494	1.498	0.979	10.29	0.96	1	11.5	1.78E+06	05-May-98	1	1	0.188	2,200	40	2	1	13	0.018		22	15	18	44
T	F	27	10	3.494	1.484	0.952	10.65	0.99	1	11.0	2.25E+06	05-May-98	1	1	0.313	2,200	38	2	1	27	0.038		37	26	30	44
T	F	28	10	3.499	1.493	0.970	9.23	0.88	1	8.4	1.29E+06	04-May-98	1	1	0.438	2,200	36	2	1	41	0.048		52	36	44	44
T	F	30	10	3.474	1.496	0.969	12.65	1.17	1	11.7	2.46E+06	04-May-98	1	1	0.188	2,200	40	2	1	13	0.016		22	15	17	44
T	F	32	10	3.487	1.499	0.979	10.77	1.00	1	11.9	1.92E+06	30-Apr-98	1	1	0.188	2,200	40	2	1	8	0.009		21	14	18	44
T	F	33	10	3.460	1.495	0.963	14.46	1.33	1	12.1	2.90E+06	30-Apr-98	1	1	0.250	2,200	39	2	1	22	0.035		30	21	23	44
T	F	36	10	3.492	1.492	0.966	9.87	0.89	1	12.1	1.48E+06	04-May-98	1	1	0.438	2,200	36	2	1	40	0.046		51	35	43	44
T	F	38	10	3.484	1.494	0.968	10.57	0.99	1	10.3	1.34E+06	04-May-98	1	2	0.375	2,200	37	2	1	29	0.033		44	30	37	44
T	F	40	10	3.494	1.497	0.977	11.14	1.04	1	11.1	1.56E+06	04-May-98	1	1	0.313	2,200	36	2	1	28	0.042		37	26	31	44
T	F	45	10	3.501	1.490	0.965	10.52	0.99	1	10.0	1.56E+06	04-May-98	1	1	0.250	2,200	39	2	1	13	0.014		29	19	24	44
T	F	47	10	3.491	1.490	0.962	11.06	1.04	1	9.8	1.64E+06	04-May-98	1	1	0.219	2,200	40	2	1	19	0.031		26	18	22	44
T	F	52	10	3.481	1.493	0.965	12.37	1.15	1	11.2	2.32E+06	30-Apr-98	1	1	0.125	2,200	40	2	1	7	0.007		14	10	12	44
T	F	62	10	3.498	1.496	0.978	11.99	1.12	1	10.6	1.78E+06	30-Apr-98	1	2	0.375	2,200	37	2	1	30	0.036		44	30	36	44
T	F	63	10	3.490	1.494	0.970	11.18	1.04	1	10.6	1.78E+06	04-May-98	1	2	0.344	2,200	37	2	1	36	0.050		41	29	34	44
T	F	65	10	3.496	1.495	0.973	13.34	1.25	1	10.6	2.77E+06	04-May-98	1	2	0.188	2,200	40	2	1	12	0.015		22	15	17	44
T	F	66	10	3.491	1.497	0.976	13.12	1.23	1	10.2	1.33E+06	30-Apr-98	1	2	0.438	2,200	37	2	1	37	0.055		52	36	44	44
T	F	72	10	3.486	1.492	0.965	11.26	1.07	1	9.4	1.47E+06	04-May-98	1	1	0.438	2,200	36	2	1	26	0.021		50	34	42	44
T	F	75	10	3.492	1.495	0.972	10.69	1.02	1	10.0	1.28E+06	04-May-98	1	2	0.500	2,200	35	2	1	42	0.032		57	39	49	44
T	F	77	10	3.493	1.491	0.965	10.96	1.04	1	9.5	1.57E+06	04-May-98	2	2	0.188	2,200	40	2	1	10	0.012		22	15	18	44
T	F	78	10	3.493	1.488	0.959	9.69	0.90	1	11.2	1.50E+06	04-May-98	1	1	0.063	2,200	40	2	1	2	0.002		7	5	6	44
T	F	81	10	3.478	1.480	0.940	12.12	1.14	1	9.7	1.67E+06	04-May-98	1	1	0.188	2,200	40	2	1	15	0.022		22	15	18	44
T	F	85	10	3.481	1.495	0.969	11.42	1.06	1	11.0	2.10E+06	30-Apr-98	1	1	0.375	2,200	37	2	1	25	0.027		43	29	35	44
T	F	88	10	3.473	1.493	0.963	10.66	1.00	1	10.4	1.72E+06	04-May-98	1	1	0.219	2,200	40	2	1	18	0.029		26	18	21	44
T	F	89	10	3.493	1.493	0.969	12.18	1.15	1	10.0	2.12E+06	04-May-98	1	1	0.188	2,200	40	2	1	12	0.016		22	15	18	44
T	F	91	10	3.492	1.500	0.982	10.30	0.97	1	9.8	1.56E+06	04-May-98	1	1	0.125	2,200	40	2	1	9	0.010		14	10	12	44
T	F	94	10	3.494	1.498	0.979	10.06	0.94	1	10.2	1.69E+06	04-May-98	1	2	0.219	2,200	40	2	1	23	0.043		27	19	22	44
T	F	105	10	3.482	1.492	0.964	11.97	1.11	1	11.4	2.29E+06	04-May-98	1	2	0.313	2,200	38	2	1	30	0.045		38	26	30	44
T	F	106	10	3.473	1.494	0.965	10.79	1.02	1	9.8	1.79E+06	04-May-98	1	1	0.250	2,200	39	2	1	21	0.031		30	21	24	44
T	F	112	10	3.491	1.498	0.978	10.02	0.94	1	10.2	1.25E+06	04-May-98	1	1	0.438	2,200	36	2	1	37	0.038		51	35	43	44
T	F	113	10	3.462	1.492	0.964	12.06	1.13	1	10.4	1.85E+06	04-May-98	2	2	0.094	2,200	40	2	1	4	0.004		11	7	9	44
T	F	119	10	3.479	1.493	0.965	12.98	1.21	1	10.6	2.23E+06	05-May-98	1	1	0.250	2,200	39	2	1	17	0.021		29	20	23	44
T	F	127	10	3.481	1.493	0.965	12.04	1.13	1	10.4	2.23E+06	04-May-98	1	1	0.125	2,200	40	2	1	4	0.004		14	9	12	44
T	F	128	10	3.484	1.496	0.972	11.65	1.11	1	10.8	2.06E+06	04-May-98	1	1	0.094	2,200	40	2	1	12	0.018		11	8	9	44
T	F	135	10	3.489	1.500	0.981	11.06	1.05	1	9.1	1.66E+06	04-May-98	1	2	0.188	2,200	40	2	1	16	0.023		22	15	18	44
T	M	8	10	3.400	1.474	0.907	9.41	0.87	1	12.1	1.24E+06	06-May-98	1	1	0.188	2,200	40	2	1	18	0.030		23	16	19	44
T	M	13	10	3.458	1.452	0.882	10.68	0.98	1	12.8	1.46E+06	06-May-98	1	1	0.125	2,200	40	2	1	9	0.011		15	10	12	44
T	M	21	10	3.398	1.455	0.872	10.53	0.98	1	13.3	1.83E+06	06-May-98	2	2	0.188	2,200	40	2	1	17	0.026		23	16	18	44
T	M	24	10	3.565	1.484	0.971	9.93	0.91	1	12.7	1.25E+06	06-May-98	1	1	0.188	2,200	40	2	1	14	0.020		22	15	18	44
T	M	26	10	3.462	1.476	0.933	9.84	0.88	1	12.9	1.06E+06	06-May-98	1	1	0.250	2,200	40	2	1	24	0.046		31	22	27	44
T	M	31	10	3.448	1.456	0.888	11.47	1.05	1	13.0	1.64E+06	06-May-98	1	1	0.250	2,200	39	2	1	25	0.042		31	21	25	44
T	M	41	10	3.450	1.489	0.949	10.64	0.99	1	13.4	1.89E+06	06-May-98	2	2	0.188	2,200	40	2	1	12	0.016		22	15	18	44
T	M	43	10	3.424	1.466	0.899	10.83	0.99	1	13.5	1.80E+06	06-May-98	1	1	0.188	2,200	40	2	1	13	0.019		22	15	18	44

Measured Test Data Sheet:

Discrete Bracing Design for Light-Frame Wood Trusses

Miles E. Waltz, Jr.
Department of Forest Products
Oregon State University
Spring 1998

Identification:				General Properties:								Buckle Test :											Constant Deflection Estimates:			
Grade ID	Mill ID	No.	Test Length (feet)	Width d (in.)	Thick b (in.)	Weak Axis I (in.^4)	Board Weight (lbs)	Dry Weight (lbs/ft)	Grade Check 1=OK 2=Better	Moisture Content (%)	E (psi)	Date Tested	Initial Profile "C" = 1 "S" = 2	Max Curve Center "Y" = 1 "N" = 2	Delta$_0$ (in)	Test Load (lb)	Brace Curve ID	Test Failure "Y" = 1 "N" = 2	Failure Code	Brace Load (lb)	Brace Deflection (in)	Comments	Plaut (lb)	Winter (lb)	Tolen (lb)	2% Rule (lb)
T	M	45	10	3.466	1.470	0.917	10.23	0.94	1	13.1	1.30E+06	06-May-98	1	1	0.250	2,200	40	2	1	29	0.069		33	23	27	44
T	M	47	10	3.482	1.496	0.971	9.34	0.85	1	13.1	1.31E+06	06-May-98	1	1	0.438	2,200	36	2	1	35	0.035		51	35	43	44
T	M	51	10	3.482	1.484	0.948	11.30	1.02	1	14.8	1.37E+06	06-May-98	1	1	0.500	2,200	35	2	1	43	0.035		56	39	49	44
T	M	54	10	3.444	1.476	0.923	9.72	0.90	1	11.7	1.33E+06	06-May-98	2	2	0.188	2,200	40	2	1	3	0.003		21	14	18	44
T	M	56	10	3.524	1.497	0.985	10.79	0.97	1	14.7	1.04E+06	06-May-98	2	1	0.625	2,200	35	2	1	47	0.042		72	49	62	44
T	M	63	10	3.493	1.487	0.957	8.85	0.82	1	11.8	1.10E+06	06-May-98	1	1	0.438	2,200	36	2	1	30	0.027		50	34	43	44
T	M	67	10	3.529	1.476	0.946	9.45	0.87	1	12.9	1.46E+06	06-May-98	1	2	0.313	2,200	36	2	1	37	0.069		39	28	33	44
T	M	72	10	3.446	1.496	0.962	10.85	0.99	1	13.8	1.09E+06	06-May-98	1	1	0.188	2,200	40	2	1	9	0.010		21	14	18	44
T	M	91	10	3.381	1.462	0.880	11.93	1.08	1	14.5	2.33E+06	06-May-98	1	1	0.188	2,200	40	2	1	12	0.015		22	15	18	44
T	M	92	10	3.486	1.483	0.947	9.95	0.91	1	13.6	1.06E+06	06-May-98	1	1	0.313	2,200	36	2	1	36	0.078		40	29	35	44
T	M	96	10	3.412	1.464	0.892	10.45	0.96	1	13.1	1.50E+06	06-May-98	2	2	0.125	2,200	40	2	1	7	0.007		14	10	12	44
T	M	99	10	3.461	1.465	0.907	10.26	0.94	1	13.2	1.33E+06	06-May-98	2	2	0.125	2,200	40	2	1	11	0.013		15	10	12	44
T	M	101	10	3.466	1.470	0.923	10.62	0.96	1	12.6	1.47E+06	06-May-98	1	1	0.125	2,200	40	2	1	26	0.053		18	13	14	44
T	M	106	10	3.392	1.462	0.920	10.75	0.97	1	14.2	1.07E+06	06-May-98	1	1	0.188	2,200	40	2	1	11	0.014		22	15	19	44
T	M	109	10	3.410	1.464	0.892	10.39	0.95	1	13.5	1.28E+06	06-May-98	1	2	0.500	2,200	35	2	1	50	0.048		59	40	50	44
T	M	110	10	3.473	1.460	0.901	10.22	0.94	1	12.0	1.68E+06	06-May-98	1	1	0.063	2,200	40	2	1	0	0.000		7	5	6	44
T	M	111	10	3.441	1.463	0.886	10.12	0.93	1	13.0	1.49E+06	06-May-98	1	1	0.875	2,200	33	2	1	82	0.045		100	67	84	44
T	M	114	10	3.420	1.454	0.876	9.94	0.90	1	13.8	1.84E+06	06-May-98	1	1	0.250	2,200	39	2	1	17	0.021		29	20	24	44
T	M	115	10	3.570	1.490	0.984	11.03	0.99	1	15.5	1.47E+06	06-May-98	1	1	0.188	2,200	39	2	1	28	0.053		25	18	20	44
T	M	119	10	3.458	1.468	0.912	10.46	0.96	1	13.1	1.77E+06	06-May-98	1	1	0.313	2,200	36	2	1	19	0.021		36	24	30	44
T	M	120	10	3.505	1.488	0.958	10.34	0.95	1	12.5	1.46E+06	06-May-98	1	1	0.219	2,200	40	2	1	13	0.017		25	17	21	44
T	M	124	10	3.460	1.496	0.965	10.52	0.95	1	14.3	1.19E+06	06-May-98	1	1	1.188	2,200	32	2	1	105	0.032		133	89	113	44
T	M	126	10	3.410	1.457	0.879	10.34	0.98	1	11.6	1.39E+06	07-May-98	1	1	0.688	2,200	34	2	1	54	0.035		78	53	66	44
T	M	129	10	3.462	1.493	0.960	9.81	0.89	1	14.8	1.57E+06	07-May-98	1	1	0.313	2,200	36	2	1	22	0.026		36	25	30	44
T	M	137	10	3.430	1.446	0.866	10.46	0.95	1	13.7	1.30E+06	06-May-98	1	1	0.750	2,200	33	2	1	54	0.021		64	57	72	44
T	M	142	10	3.387	1.460	0.878	9.77	0.90	1	12.3	1.26E+06	07-May-98	1	1	0.250	2,200	39	2	1	22	0.032		30	21	26	44
T	M	146	10	3.399	1.478	0.911	9.62	0.89	1	12.1	1.19E+06	07-May-98	1	1	0.188	2,200	40	2	1	19	0.032		23	16	20	44

Failure Codes:

1 - No Failure Observed
2 - Column failure, test load not achieved
3 - Brace failure below proof load
4 - Brace failure, unstable at proof load
5 - Column and brace failure at proof load
6 - column and brace failure below proof load