STRENGTH OF BUILT-UP TIMBER COLUMNS

by

DAVID BIR VAN DYER

A Thesis Submitted to the

Faculty of Graduate Studies

in Partial Fulfillment of the Requirements

for the Degree of

DOCTOR OF PHILOSOPHY

Major Subject: Civil Engineering

Approved

S. K. Malhotra

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NOVA SCOTIA TECHNICAL COLLEGE

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ERRATA SHEET

for

Ph.D. Dissertation "Strength of Built-Up Timber Columns", by D.B. Van Dyer, Nova Scotia Technical College, Halifax, Nova Scotia, 1976.

1.	Page xvii	Add the following footnote: Some of the symbols as given under "NOTATION" are redefined in Chapters 2 and 3. For these chapters, the definitions are to be taken as indicated therein.
2.	Pages xvii and 33	Replace the definition of "a" with the following: Length of column laminate divided by the number of connectors piercing a joint (or interface).
3.	Page xix	Replace s with S
		Replace "$=4\sqrt{\frac{k}{4EI}}$" with "$\sqrt[4]{\frac{\bar{k}}{4EI}}$" in second line from bottom.
4.	Pages xvii, 11, 12, 16, 17, 46, 47, 283, 285, 286, 287, 288 290, 291 and 292	Replace k_1 and k_2 with $\bar{k}_1$ and $\bar{k}_2$, respectively.
5.	Page 18	Replace $k_{1,2}$ and k with $\bar{k}_{1,2}$ and $\bar{k}$, respectively.
6.	Page 29	Replace the expressions for symbols G, G_K, T and δ with the following:

6. Page 29 (cont'd)

G = average connector modulus
$= \frac{1}{m}(G_1 + G_2 + \ldots + G_m)$ (lb./in.^{2})

G_K = connector modulus for the joint between the K^{th} and $(K^{th}+1)$ layers $= \frac{T_K}{\delta_K}$ (lb./in.^{2})

T_K = connector shear flow between the K^{th} and $(K^{th}+1)$ layers (lb./in.)

δ_K = connector displacement between the K^{th} and $(K^{th}+1)$ layers (in.)

Replace n with m in the definition of Ω.

7. Page 33 — First line should read as follows:
k = slip modulus = shear flow = Ga (lb./in.)

8. Page 46 — Replace λ_2 with λ_1^2 in last line.

9. Page 112 — Change sign for all values in last column of Table 7.2.

10. Page 129 — Second line from top...."(mean + " should read...."(mean $\pm$ ".

11. Pages 150 and 170 — Replace $F_{cr}4^*$ with $F_{cr}4$ in the third line.

12. Page 159 — Title of figure should read as follows:
Figure 8.C1.2 Column Stress.....Curves - Column Type C1

13. Page 169 — Replace 2½" with 2¼" under the heading "Connector Data".

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*Plates given in Appendix C

NOTATION

A	Area of the entire cross section
A_r	$= \frac{A_1 Z_1}{r_1} + \frac{A_1 Z_1 + A_2 Z_2}{r_2} + \text{---} + \frac{A_1 Z_1 + \text{---} + A_m Z_m}{r_m}$ (see Fig. 2.8)
a	Length of the laminate divided by the total number of fasteners
b	Width of laminate
B	A factor in the approximate solution for the column buckling stress equation
c	A parameter in the stress-strain function
d	Connector diameter
E	Modulus of elasticity
$E_{c_{11}}$	Compression modulus of elasticity parallel to grain
$E_{c_{1}}$	Compression modulus of elasticity perpendicular to grain
E_t	Tangent modulus of elasticity
E_r	Reduced modulus of elasticity
F	Stress
F_{cr}	Column buckling stress
F_t	Tangent modulus column stress
F_u	Ultimate compressive stress
I, I_s	Moment of inertia of the solid section
i_i	Moment of inertia of member No. i with respect to its own centroidal axis parallel to the joints

i	Sum of moments of inertia of the individual laminations about their weak axis through their centroids
J_1, J_2 K_1, K_2 L_1, L_2	Coefficients in Kuenzi's formula for connector modulus (Eq. 2.4)
C_1, C_2 C_3, C_4	Coefficients in the exact inelastic column formula (Eq. 3.20)
k	Connector modulus
$\bar{k}$	Foundation modulus
k_1, k_2	Foundation modulus in members 1 and 2 respectively
L	Buckling length of the column
	Length of connection member
L_1	Laminate length
l	Length of the compression specimen
M	Total external moment acting on the laminated column
M_1	Moment of the internal forces bending the equivalent solid column
M_2	Moment of the internal forces causing additional deflection of the column members
m	Number of joints (or interfaces) in the system
P	Column load or load on connection
P_e	Euler critical load
P_{cr}	Column critical load
P_t	Tangent modulus column load
P_r	Reduced modulus column load

Q	External shear force acting on the laminated column
Q_1	Shear force acting on the equivalent solid column
Q_2	Shear force acting on the members of the laminated column
r	Radius of gyration $= \sqrt{\frac{I}{A}}$
s	Nail spacing
x	Distance measured along column length
	Factor in connector modulus formula, $k = tdE/x$
y	Lateral deflection of the column at point x
y_1	Deflection corresponding to M_1
y_2	Deflection corresponding to M_2
α	$= \frac{i}{I}$
β	Buckling coefficient
δ, ϕ	Joint slip
ε	Strain
λ	Slenderness ratio of the column, $(\frac{L}{r})$.
	$= 4\sqrt{\frac{k}{4EI}}$ (Eq. 2.2)
σ	Standard deviation

ACKNOWLEDGMENTS

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ABSTRACT

At the present only a limited amount of information is provided by various timber codes and specifications for the design of built-up timber columns. Very little information is provided for designing layered timber columns and no information is given for braced columns. The design method provided for spaced columns is based on empirical tests and formulas.

The objective of the present investigation is to develop a rational procedure for the analysis and design of mechanically connected built-up timber columns including layered, braced and spaced columns. The theory developed takes into account the effect of interlayer slip and is applicable to columns failing in the elastic as well as inelastic ranges of stress.

The test material is Construction Grade No. 1 Eastern Spruce lumber. Connector types used are common wire nails, steel bolts and split ring connectors. The philosophy and the procedure presented herein are applicable to other species of wood as well.

The investigation is conducted in three phases. The primary aim of phase one is a general theory for predicting

the load-slip behaviour of timber joints subjected to interlayer slip. Some 250 connections fabricated from three to seven members and fastened with various types of connectors are tested. Good agreement is observed between the theory and the experimental results; the overall average difference between the two is about five percent. In addition, 2,130 compression tests are conducted to evaluate physical, strength and elastic properties of the test materials.

Phase two provides a general theory, incorporating the results of phase one, for predicting the buckling stresses of built-up timber columns. To verify the theory, a comprehensive test program is conducted on some 400 columns, including layered, braced and spaced columns of various dimensions built up from two to seven members and covering the range of slenderness ratio values from thirty-four to one hundred and fifty. The cross-sections investigated have one or two axes of symmetry. Statistical techniques are applied to analyse the test data. Good agreement is observed between the theoretical predictions and the experimental results. The overall average difference observed between the predictions and the experimental results is about six and one-half percent.

Incorporating the results of phases one and two in phase three, a rational procedure, using dimensionless coefficients called 'Buckling Coefficients', is developed for the design of built-up timber columns. This design method is simple to apply and is applicable to elastic as well as inelastic columns.

CHAPTER 1

INTRODUCTION

1.1. General

Timber columns are quite frequently built up from laminations fastened together with nails, bolts, split ring connectors and glue or a combination thereof. This practice is necessary for practical as well as economic reasons. For example, columns can be built up from material that is too small in cross-section to be structurally useful otherwise. In addition, lower grade material can be placed in locations where stresses are low and the better quality material can be used where higher stresses occur, thereby utilizing lumber more efficiently.

In addition to the layered columns discussed above, built-up columns, as referred to herein, also include spaced and braced columns.

The buckling strength of a mechanically connected built-up column has an upper bound which is that of an equivalent solid column, that is, a solid column of identical dimensions and a lower bound which is that of a corresponding unconnected assembly, that is, a similar built-up column without connectors . The type and number of connectors used in the column will determine the actual buckling strength of the column between the upper and lower bounds.

In addition to the buckling phenomenon characteristic of axially loaded compression members, the strength of a mechanically connected built-up timber column is affected by the load-deformation behaviour of the type of connector used in the built-up column.

At the instant any lateral deflection occurs in the built-up column, shearing forces are induced across the column cross-section and transferred across the interfaces of the column laminates by the connectors. The magnitude of the shearing force acting on each connector depends on the number of connectors used in the built-up column. At this point the behaviour of the connector in the wood members is similar to that of a mechanically fastened timber connection in which the connector is subjected to a lateral load.

In the process of transferring the shearing forces across the column laminates the connector deforms and compresses the wood adjacent to it,resulting in an interlayer slip. The magnitude of this interlayer slip depends on the magnitude of the shearing force, the number of connectors, the stiffness of the connectors and the strength properties of the wood members.

Various column theories, which take into account the effect of interlayer slip, have been developed for predicting the buckling stresses in mechanically fastened built-up timber columns. These theories have the limitation of being applicable only to long columns, that is, columns which fail at stresses within the elastic range. Furthermore, in the tests conducted to verify these theories for elastic columns, the connector modulus, which is indicative of the interlayer slip behaviour of laminated columns, has been determined empirically by conducting shearing tests on connections fabricated with similar wood members and connectors as used in the built-up columns. In addition, the experimental verification of the theories developed have been very limited in the column dimensions, types of column cross-sections and the types of connectors investigated.

1.2. Existing Design Procedure

At the present the basic Canadian sources of information (1, 2, 3) on the design of mechanically fastened built-up columns are very limited.

The design methods currently in use for built-up timber columns are based primarily on empirical tests and formulas. For a given connector type and connector spacing the built-up column strength is determined as a percentage of the strength of a solid column of similar dimensions to that of the built-up column considered. Guidelines are given only for specific connector types. An example of this kind is the Timber Design Manual published by the Laminated Timber Insitute of Canada(2), which recommends that "........ the strength of a built-up timber column be taken as eighty percent of the value of a corresponding solid column if the fastener, (bolts and split ring connectors), spacing does not exceed six times the thickness of each member; if the column is nailed or bolted together the load carrying capacity of the column is determined as the sum of the capacities of the individual pieces considered as independent compression members."

U. S. Department of Agriculture, Wood Handbook (4), devotes a paragraph on built-up timber columns with mechanical connectors. No guidelines are given on design methods or recommendations on connector types and arrangements.

The British Standard Code of Practice CP 112 (5) gives no information with regard to the design of built-up layered columns. However, a design procedure, incorporating the work of Pleskov (6) and Brock (7), is provided for spaced columns.

1.3. Objectives of the Present Investigation

The broad objectives of the present investigation are as outlined in the following:

1. To provide a rational method for determining the connector modulus for any given combination of member sizes, material properties and connector type.
2. To provide a rational method for the analysis of the buckling stresses of mechanically fastened built-up timber columns. This method should apply to elastic as well as inelastic columns.
3. To develop a rational and practical method for the design of mechanically fastened built-up timber columns.

1.4. Scope

The present study is conducted in three phases as described below:

Phase 1

A comprehensive experimental and theoretical investigation is conducted on some two hundred and fifty (250) double and multiple shear connections, fabricated using various combinations of member sizes and connector types. The test material is Construction Grade No. 1 Eastern Spruce lumber fasted with common wire nails, bolts and split ring connectors. Also, some two thousand one hundred and thirty tests are conducted to evaluate the physical, strength and elastic properties of the test material. A brief layout of the tests is given in Table 7.1 (Chapter 7)

Phase 2

Using similar test materials as described in Phase 1, a comprehensive test program is conducted on some four hundred columns of various dimensions built up from two to seven members and covering the range of slenderness ratio values from thirty-four to one hundred and fifty. The columns investigated have one or two axes of symmetry and consist of layered, spaced and braced columns. The theoretical investigation of this second phase incorporates the results obtained in Phase 1. The results of Phase 2 are subjected to statistical analysis in order to determine the reliability of the proposed theory to predict the strength of built-up timber columns. The various column types investigated are shown in detail in Chapter 8.

Phase 3

Utilizing the results of Phases 1 and 2 in the third phase of the study, a rational design procedure for mechanically connected built-up timber columns is developed.

CHAPTER 2

REVIEW OF LITERATURE

2.1. Introduction

This literature review is divided into two separate sections dealing with information relevant to the present study. The first section presents the literature on timber shear connections. The second section presents the literature on built-up timber columns.

2.2. Timber Shear Connections

2.2.1. General

A review of the literature on timber shear connections indicates that most of the research in this area has been directed towards evaluating the lateral shear strength of the connectors while very little has been done specifically to investigate the initial load-slip behaviour of the connection. A method for predicting this initial load-slip behaviour is essential for dealing with the problem of interlayer slip which occurs in mechanically connected built-up timber columns.

A good comprehensive literature review on timber connections subjected to lateral load is given by Wilson (8). This work is mentioned here as it provides good background information on connection behaviour.

Studies by Kuenzi (9) and Wilkinson (10) are presented in some detail below as they have direct relevance to the present investigation. Kuenzi applied the beam on an elastic foundation concept to develop a theory for predicting the strength of nailed and bolted timber connections. Wilkinson's study was based on the work of Kuenzi.

2.2.2. Analytical Procedures

The concept of beams on an elastic foundation was first introduced by Winkler (11). Several solutions for beams of finite length, under varying load conditions, have been formulated by Hetenyi (12). The basic assumptions are:

1. The materials are elastic and obey Hooke's Law.
2. Reaction force at any point is proportional to deflection at that point.
3. Reaction forces are vertical at every cross-section.
4. The foundation transmits no shear.

These assumptions lead to the basic differential equation, for the deflection curve of a beam supported on an elastic foundation, as given by

$$EI\frac{d^4y}{dx^4} = -\bar{k}y \qquad (2.1)$$

where

EI^* = stiffness of the beam, lb-in^2

E = modulus of elasticity, lb/in^2

I = moment of inertia, in^4

y = deflection at point x, in

$\bar{k}$ = foundation modulus, lb/in^2

The solution of Equation (2.1) finally results in expressions involving a characteristic parameter

$$\lambda = \sqrt[4]{\frac{\bar{k}}{4EI}} \qquad (2.2)$$

and expressions for deflections, moments and shears depend on the value of this characteristic parameter.

Kuenzi

Kuenzi (9) made the following additional assumptions in applying the beam on an elastic foundation theory to nailed timber connections.

1. No friction forces develop between the timber members.
2. The effective foundation depth is one inch deep.

---

* Symbols are defined where they first appear.

Kuenzi defines the foundation modulus as

$$\bar{k} = \frac{E_{c_{\parallel} \text{ or } \perp} D}{D_e} \qquad (2.3)$$

where

$\bar{k}$ = foundation modulus, lb/in^2

$E_{c_{\parallel}}$ or $\perp$ = compression modulus of elasticity parallel or perpendicular to grain respectively, lb/in^2

D = bolt or nail diameter, in

D_e = effective foundation depth, in (assumed to be one inch)

Based on these assumptions and the concept of a beam on an elastic foundation, Kuenzi provided a theoretical solution for single shear and double shear frictionless nailed timber connections.

Kuenzi's relevant expressions for deflections, moments and shears are provided in detail in Appendix B.

Of primary concern for nailed and bolted joints in built-up timber columns is the relation between load and the relative displacement (slip) of the wood members. This relationship, developed by Kuenzi, is presented below for single shear and double shear timber connections.

Two member joint (nailed or bolted)

$$\delta = P\left[2(L_1 + L_2) - \frac{(J_1 - J_2)^2}{K_1 + K_2}\right] \text{ - - - - - - - - - } (2.4)$$

where

δ = joint slip, in

P = load, lbs.

The factors L_1, L_2, J_1, J_2, K_1, and K_2 are combinations of hyperbolic and trigonometric functions and are equal to

$$L_1 = \frac{\lambda_1}{k_1}\left(\frac{\sinh\lambda_1 a \cosh\lambda_1 a - \sin\lambda_1 a \cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right) \text{ - - - } (2.5)$$

$$L_2 = \frac{\lambda_2}{k_2}\left(\frac{\sinh\lambda_2 b \cosh\lambda_2 b - \sin\lambda_2 b \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right) \text{ - - - } (2.6)$$

$$J_1 = \frac{\lambda_1^2}{k_1}\left(\frac{\sinh^2\lambda_1 a + \sin^2\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right) \text{ - - - - - - - - - - - } (2.7)$$

$$J_2 = \frac{\lambda_2^2}{k_2}\left(\frac{\sinh^2\lambda_2 b + \sin^2\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right) \text{ - - - - - - - - - - } (2.8)$$

$$K_1 = \frac{\lambda_1^3}{k_1}\left(\frac{\sinh\lambda_1 a \cosh\lambda_1 a + \sin\lambda_1 a \cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right) \text{ - - - } (2.9)$$

$$K_2 = \frac{\lambda_1^{\ 3}}{k_2} \left(\frac{\sinh\lambda_2 b \cosh\lambda_2 b + \sin\lambda_2 b \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right) \quad - - - \ (2.10)$$

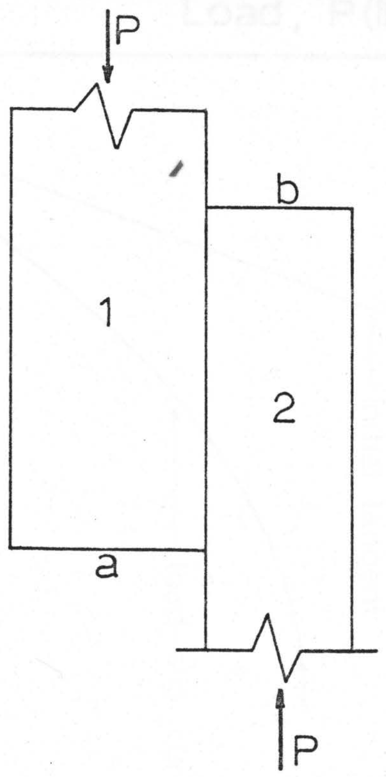
in which a and b equals the member thickness (Figure 2.1) or if the connector does not completely penetrate the member, then a or b equals the depth of penetration. The subscripts refer to either member 1 or 2.

Rearranging Equation (2.4) the initial slip modulus, k, or the initial tangent to the load-slip curve is directly obtained

$$k = \frac{P}{\delta} = \frac{(K_1 + K_2)}{2(L_1 + L_2)(K_1 + K_2) - (J_1 - J_2)^2} \quad - - - \ (2.11)$$

Figure 2.2 shows a typical load-slip curve for a nailed or bolted timber connection. The slope of the initial tangent defines the slip modulus for that particular joint. Equation (2.11) when plotted coincides with the initial tangent drawn. This is reasonable since Equation (2.11) is based on the assumption of elastic behaviour.

Figure 2.3 shows a graph reproduced from a recent paper by Foschi (13). The materials used in his study were Glulam Rivets and Douglas Fir lumber. In Figure 2.3, Equation (2.11) is plotted along with other theoretical curves and as shown



NOTE a or b equals member thickness or amount of nail penetration

FIGURE 2.1 Notation for Single Shear Connections

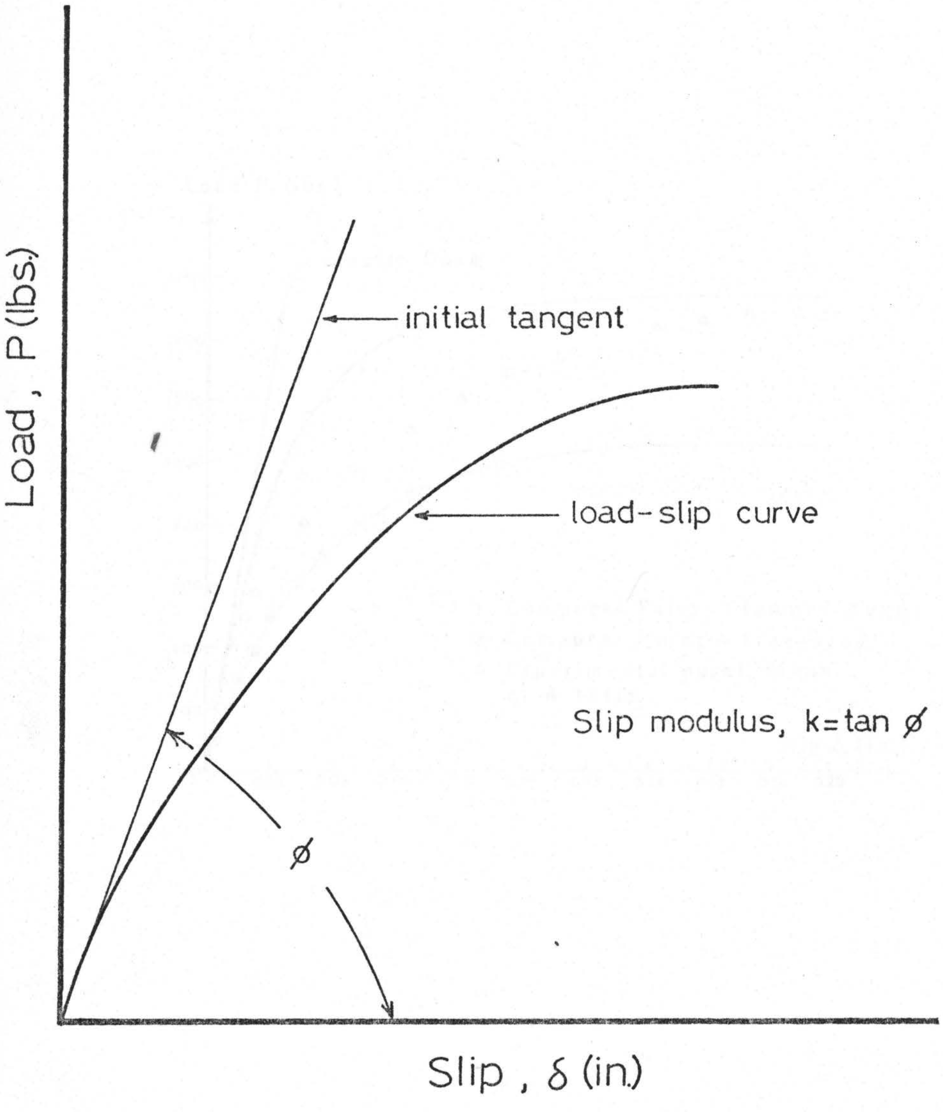


FIGURE 2.2 Typical Load-Slip Curve for Nailed or Bolted Timber Joint

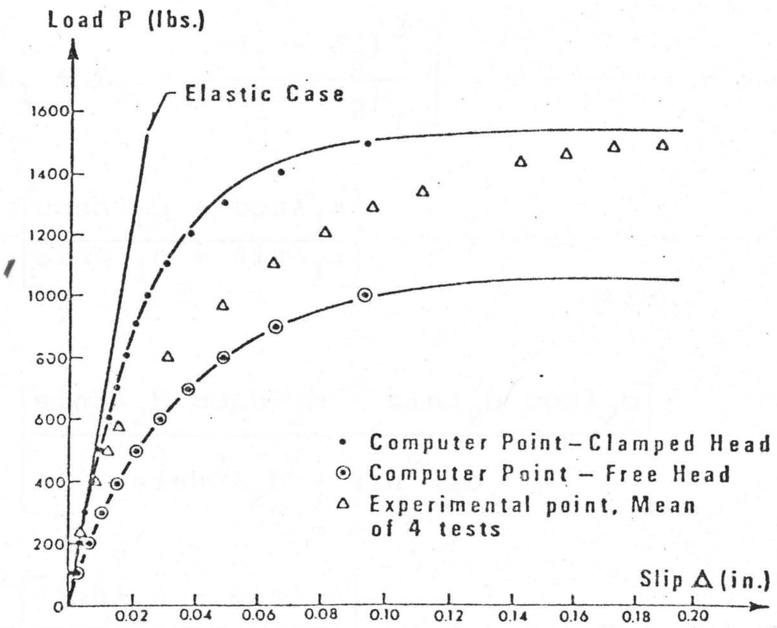


FIGURE 2.3 Load-Slip Curves - Foschi

it describes the initial tangent of the load-slip curve. Foschi concluded that although the linear approximations based on the theory of beams on elastic foundations are not accurate to predict ultimate loads, they are, however, valid for predicting the initial behaviour of nailed or bolted timber connections.

Three member joint (nailed or bolted)

$$\delta = P\left[L_1 + L_2 - \frac{(J_1 - J_2)^2}{2(K_1 + K_2)}\right] \qquad (2.12)$$

where

$$L_1 = \frac{\lambda_1}{k_1}\left[\frac{\cosh\lambda_1 a + \cos\lambda_1 a}{\sinh\lambda_1 a + \sin\lambda_1 a}\right] \qquad (2.13)$$

$$L_2 = \frac{\lambda_2}{k_2}\left[\frac{\sinh\lambda_2 b\,\cosh\lambda_2 b - \sin\lambda_2 b\,\cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right] \qquad (2.14)$$

$$J_1 = \frac{\lambda_1^{\,2}}{k_1}\left[\frac{\sinh\lambda_1 a - \sin\lambda_1 a}{\sinh\lambda_1 a + \sin\lambda_1 a}\right] \qquad (2.15)$$

$$J_2 = \frac{\lambda_2^{\,2}}{k_2}\left[\frac{\sinh^2\lambda_2 b + \sin^2\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right] \qquad (2.16)$$

$$K_1 = \frac{\lambda_1^{\,3}}{k_1}\left[\frac{\cosh\lambda_1 a - \cos\lambda_1 a}{\sinh\lambda_1 a + \sin\lambda_1 a}\right] \qquad (2.17)$$

$$K_2 = \frac{\lambda_2^{\;3}}{k_2} \left[\frac{\sinh \lambda_2 b \cosh\lambda_2 b + \sin\lambda_2 b \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b} \right] \quad \text{- - (2.18)}$$

Equation (2.12) can be rearranged to provide the slip modulus for a three member joint as shown below

$$k = \frac{2(K_1 + K_2)}{2(L_1 + L_2)\ (K_1 + K_2) - (J_1 - J_2)^2} \quad \text{- - - - - - - (2.19)}$$

Wilkinson

Based on the work of Kuenzi, Wilkinson (10) developed an approximate relationship between load and joint slip for a two member joint.

Figure 2.4 shows curves of L, J and K for a two member joint. As shown in the figure when $\lambda_1 a$ and $\lambda_2 b$ exceed 2 the value of the ordinate becomes unity. For the joint where $\lambda_1 a$ and $\lambda_2 b$ are both greater than 2, as is the case for relatively long fastener such as nails, and both members are of the same species of wood

$$k_1 = k_2 = \overline{k}$$

and $\lambda_1 = \lambda_2 = \lambda$

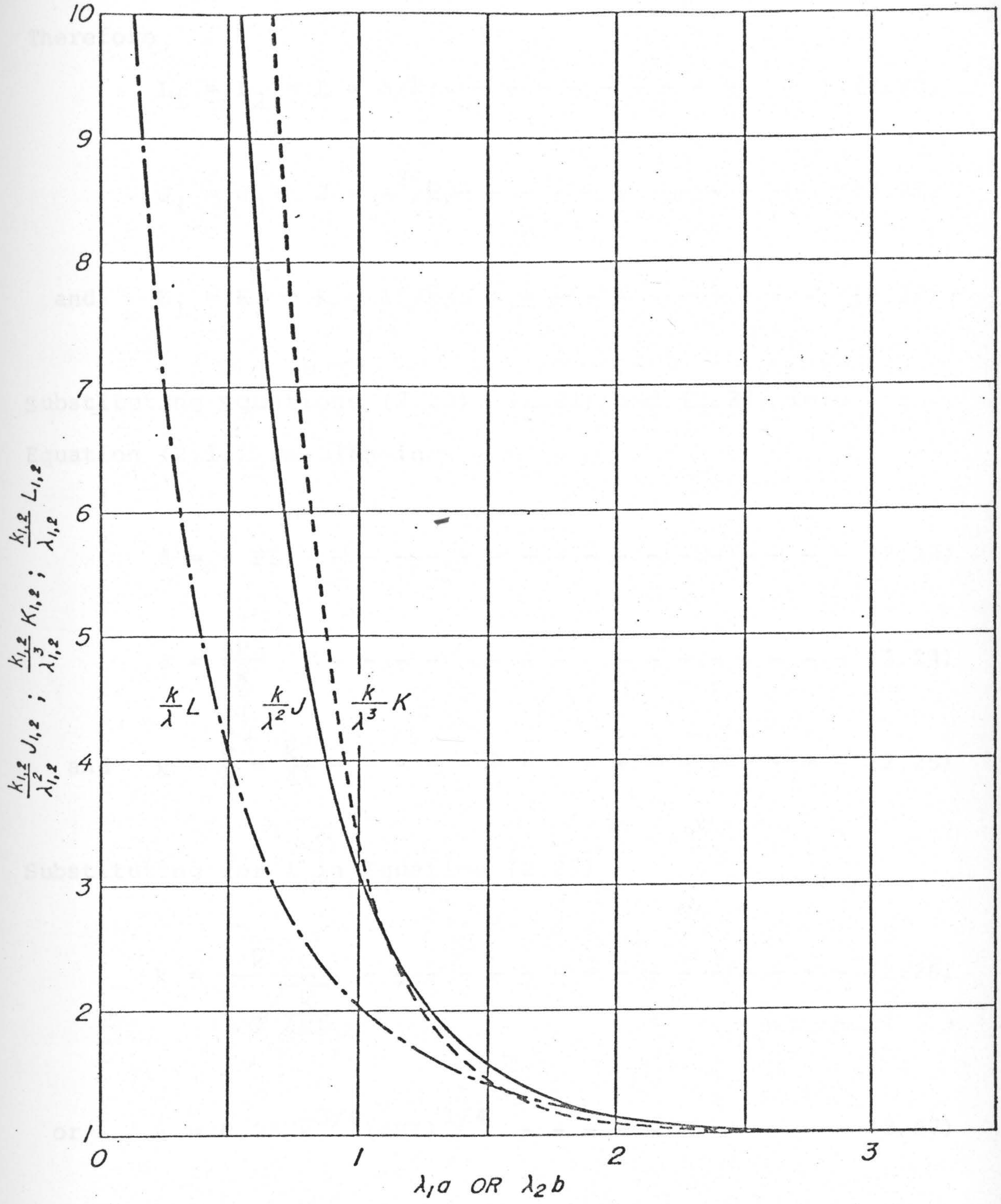


FIGURE 2.4 Curves for Computation of J_1, K_1, and L_1 for Joints of Two Members and J_2, K_2 and L_2 for Joints of Two or Three Members - Kuenzi.

Therefore,

$$L_1 = L_2 = L = \lambda/\bar{k}, \quad (2.20)$$

$$J_1 = J_2 = J = \lambda^2/\bar{k}, \quad (2.21)$$

$$\text{and } K_1 = K_2 = K = \lambda^3/\bar{k}, \quad (2.22)$$

Substituting Equations (2.20), (2.21) and (2.22) into Equation (2.11) results in

$$\delta = 4\,PL \quad (2.23)$$

$$\delta = \frac{4P\lambda}{\bar{k}} \quad (2.24)$$

$$\text{and } \quad k = \frac{P}{\delta} = \frac{\bar{k}}{4\lambda} \quad (2.25)$$

Substituting for λ in Equation (2.25),

$$k = \frac{\bar{k}}{4\sqrt[4]{\frac{\bar{k}}{4EI}}} \quad (2.26)$$

$$\text{or } \quad k = 0.25\,\bar{k}^{3/4}(4EI)^{1/4} \quad (2.27)$$

For joints having short, thick connectors the complete expressions, Equations (2.5) through (2.10), must be used in place of Equation (2.27).

2.2.3. Summary

Kuenzi's theory, though it was developed for the design of nailed or bolted timber joints under lateral load, appears, from the literature, to be more applicable to the problem of predicting the initial load-slip behaviour of nailed or bolted timber joints. Kuenzi's theory is valid only for elastic stress conditions as his assumptions dictate and these assumptions are most closely realized for a nailed or bolted timber joint when that joint is in the initial stages of loading.

2.3. Built-Up Timber Columns

2.3.1. A Brief Review of Column Theories

The first mathematical analysis of the load carrying capacity of columns is attributed to Euler. In his book published in 1757 (14), Euler presented the following differential equation relating the column load P and the deflection of the elastic curve y to the distance x along the column (Fig. 2.5).

$$EI \frac{d^2y}{dx^2} = -Py \text{ - } (2.28)$$

The solution of Eq. (2.28), for columns hinged at both ends, provides the value of a load, P_e, known as the Euler critical load. It is given by

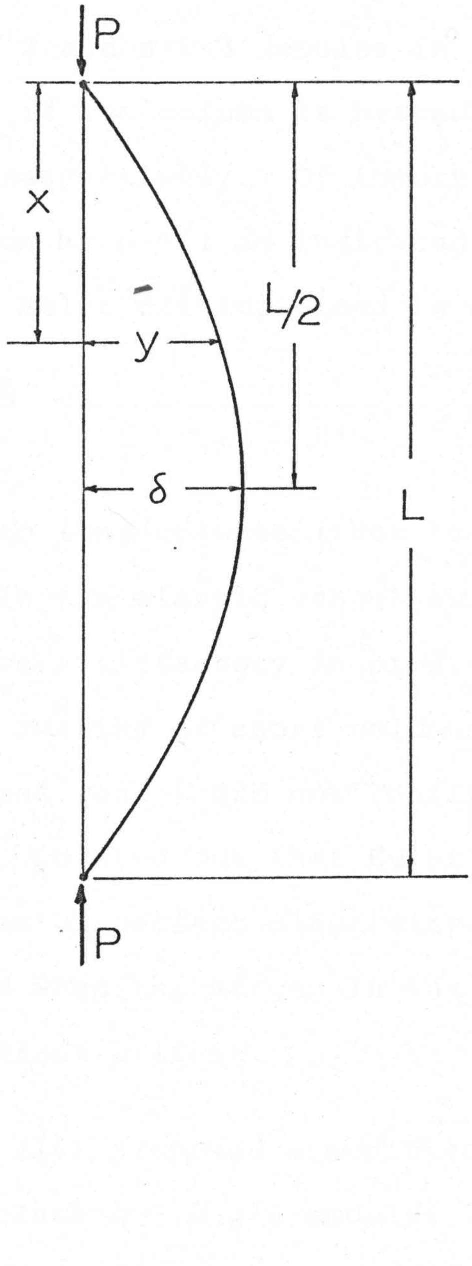


FIGURE 2.5 Buckling of a Column

$$P_e = \frac{n^2\pi^2 EI}{L^2} \qquad (2.29)$$

where n is an integer indicating the buckling mode and L is the column buckling length. Figure 2.6 shows the buckling modes corresponding to values of n equal to 1, 2 and 3. Values of n = 2 and n = 3 results in higher loads and are possible only if the column is braced at the middle or third points respectively. Of importance here is the buckling mode given by n = 1 as indicated in Fig. 2.6(a). In this case the Euler critical load is expressed as

$$P_e = \frac{\pi^2 EI}{L^2} \qquad (2.30)$$

Applied to long columns (that is, columns buckling at stresses within the elastic range) Euler's buckling formula was found to be satisfactory in predicting buckling stresses. However, test results of short columns (buckling within the inelastic stress range) did not confirm his theory. Considere (15), in 1891, pointed out that Euler's theory was based on the assumptions of perfect elasticity and, therefore, could not be applied when the stress in any portion of the column exceeded the elastic limit.

Engesser (16) proposed a modified form of Euler's formula by replacing Young's modulus, E, with the tangent modulus, E_t. He defined E_t as the slope of the stress-stain

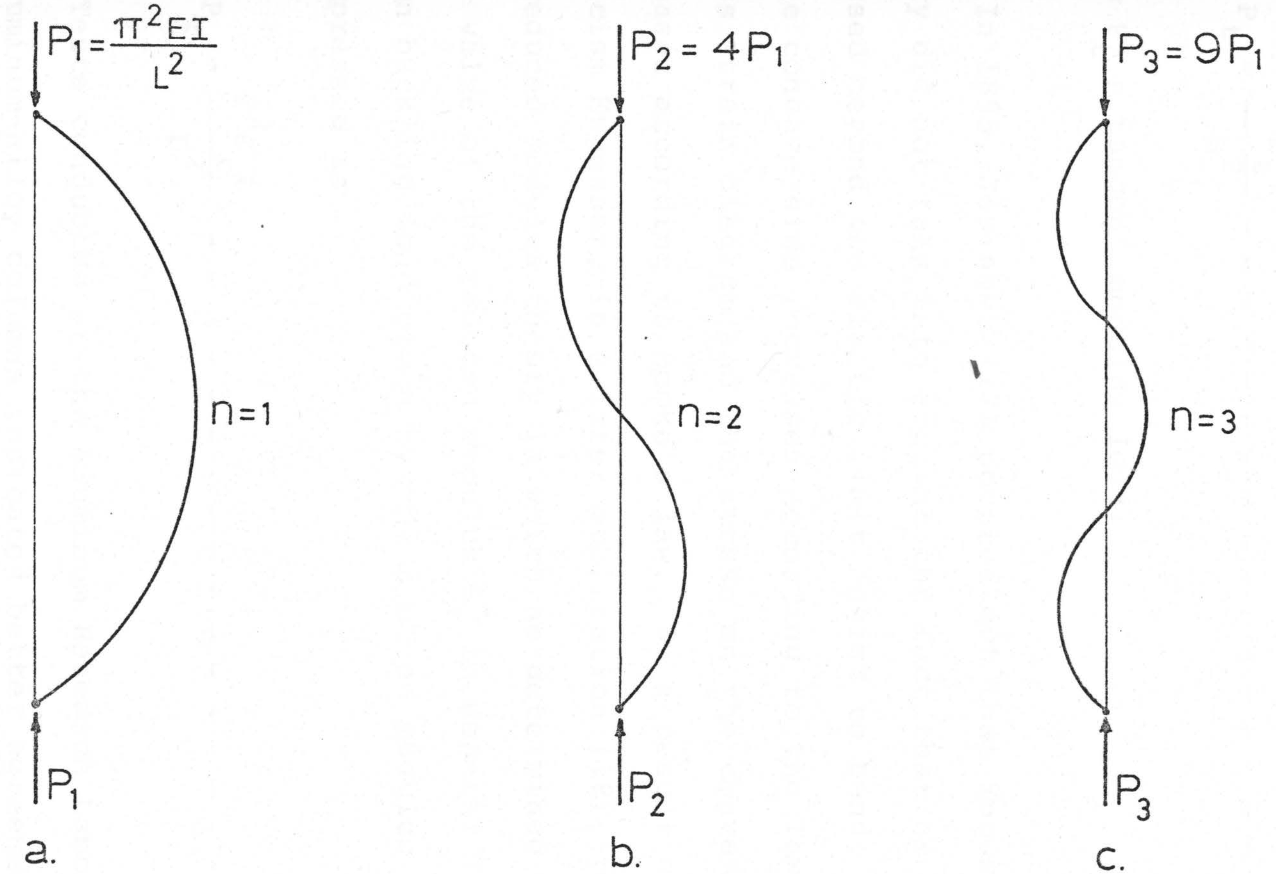


FIGURE 2.6 Possible Buckling Modes

curve corresponding to the average stress. Engesser's formula is expressed as

$$P_t = \frac{\pi^2 E_t I}{L^2} \qquad (2.31)$$

where P_t = tangent modulus load.

In 1895, Jasinski (17) pointed out that Engesser's theory did not take into account the fact that as a column stressed beyond the elastic limit begins to bend, the stress on the concave side increases according to the law of the stress strain diagram and the stress on the convex side decreases according to Hooke's law. As a result of this criticism Engesser, in a later publication (18), presented his reduced modulus theory in which he determined the theoretical value of the reduced modulus E_r^* in general form. The column buckling load given by the reduced modulus concept, P_r, is expressed as

$$P_r = \frac{\pi^2 E_r I}{L^2} \qquad (2.32)$$

Tests conducted at the Aluminum Research Laboratory (19) on aluminum-alloy columns indicated better agreement between the experimental results and the theoretical results given by Engesser's tangent modulus formula rather than those given by the reduced modulus formula.

*E_r depends on E_t and the column cross section.

The reduced modulus theory assumes that the column will remain straight up to the calculated maximum load but it also states that some strain reversal has to occur in order to provide the additional stiffness required beyond the tangent modulus load. These two assumptions contradict each other because strain reversal is not possible in a straight column.

On the basis of mathematical analysis and experimental verification using a buckling model, Shanley (20,21) solved this paradox by showing that it is possible for a column to bend simultaneously with increasing load, without strain reversal, and it was reasonable to conclude that such bending would start at the tangent modulus load. He stated further that the reduced modulus load represents the upper limit for the load that can theoretically be reached as the column continues to bend with increasing load. In practice, though, the maximum column load would have a value between the tangent modulus load and the reduced modulus load. The final conclusion of Shanley's investigation is that the Engesser tangent modulus equation should be used as the basis for determining the buckling strength of inelastic columns.

The above conclusions have since been more rigorously confirmed by Duberg and Wilder (22) and Johnston (23). An elaborate discussion of the above mentioned theories is given by Bleich (24) and Timoshenko and Gere (25).

2.3.2. Review of Existing Theories for Built-Up Timber Columns

A brief review of the works of Granholm (26), Pleskov (6), Niskanen (27), Newmark, Siess and Viest (28), Goodman (29), Rassam (30), Rassam and Goodman (31,32,33) and Malhotra (34) are presented here because of their relevance to this study.

Granholm

In his paper "On Composite Beams and Columns with Particular Regard to Nailed Timber Structures", Granholm (26) developed a theory for layered systems which takes into account the effect of interlayer slip. Granholm's theory was developed for cross sections having double symmetry and assumes a constant connector spacing. His theory further assumes that the effect of the connector can be uniformly distributed along the length of the beam and that the relationship between the force carried by the connector and its deformation is linear.

For the case of a two layered system (see Fig. 2.7), Granholm derived the following relationships

$$\frac{d^2\phi}{dx^2} - \frac{2bK}{EA}\phi = r\frac{d^3y}{dx^3} \qquad (2.33)$$

$$\frac{d^2y}{dx^2} - \frac{EAr}{2EI_s}\frac{d\phi}{dx} = -\frac{M}{EI_s} \qquad (2.34)$$

where

ϕ = relative slip between the layers, (in).

b = width of each layer, (in).

K = displacement modulus, (lb/in^2), related to ϕ by the relation $\tau = K\phi$

τ = shear flow between the layers, (lb/in)

r = distance between the centroids of the two layers, (in).

E = modulus of elasticity of the matenal of the layers, (lb/in^2)

I_s = moment of inertia of an equivalent solid section (in^4).

A = cross section area of each layer (in^2).

M = external moment at the section(in-lb).

The solution for the two layered beam shown in Fig. 2.7 is obtained by simultaneously solving Equations (2.33) and (2.34).

Granholm further developed this theory to deal with layered beams and columns of more than two layers.

Pleskov

In his book "Theoretical Studies of Composite Wood Structures", Pleskov (6) developed a theory for layered systems including the effect of interlayer slip. The effect of the connector is assumed to be uniformly distributed over the

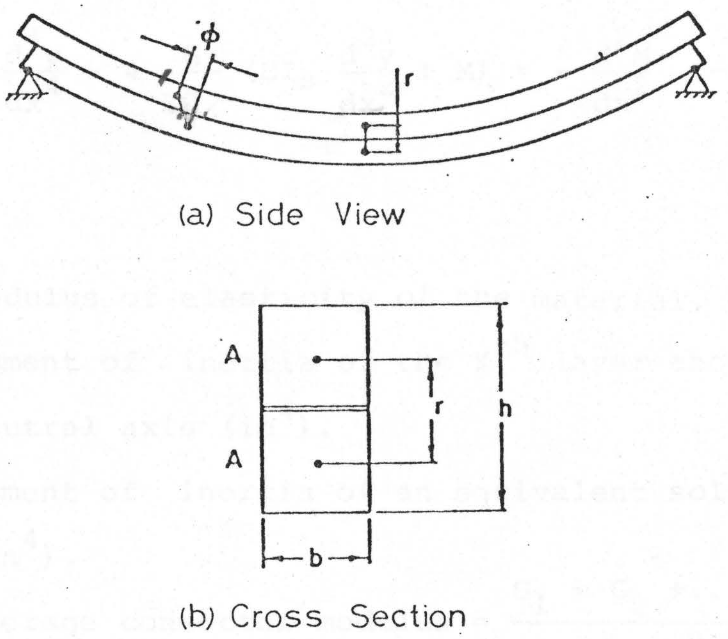


FIGURE 2.7 Layered Beam with Interlayer Slip - Granholm

length of the member and the relationship between the force on the connector and its deformation is assumed to be linear. Pleskov's theory, however, is applicable to members having one axis of symmetry (see Fig. 2.8).

The governing differential equation of Pleskov's theory is given by

$$E\ \Sigma i_K \frac{d^4y}{dx^4} - 4\frac{G}{E\Omega}\left(EI_s\frac{d^2y}{dx^2} + M\right) = -\frac{d^2M}{dx^2} \quad ----(2.35)$$

where

E = modulus of elasticity of the material, (lb/in^2).

i_K = moment of inertia of the K^{th} layer about its own neutral axis (in^4).

I_s = moment of inertia of an equivalent solid section (in^4).

G = average connector modulus = $\dfrac{G_1 + G_2 + \ldots + G_n}{n} = T/\delta$

T = connector shear flow (lb/in).

δ = connector displacement (in).

G_k = connector modulus for the joint between the K^{th} and $(K^{th}+1)$ layers (lb/in^2).

$$\Omega = \frac{4}{m}\left(\frac{A_1Z_1}{r_1} + \frac{A_1Z_1 + A_2Z_2}{r_2} + \frac{A_1Z1 + A_2Z_2 + \ldots + A_nZ_n}{r_n}\right)$$

m = number of joints in the system

= number of layers minus 1

A_K = area of K^{th} layer (in^2)

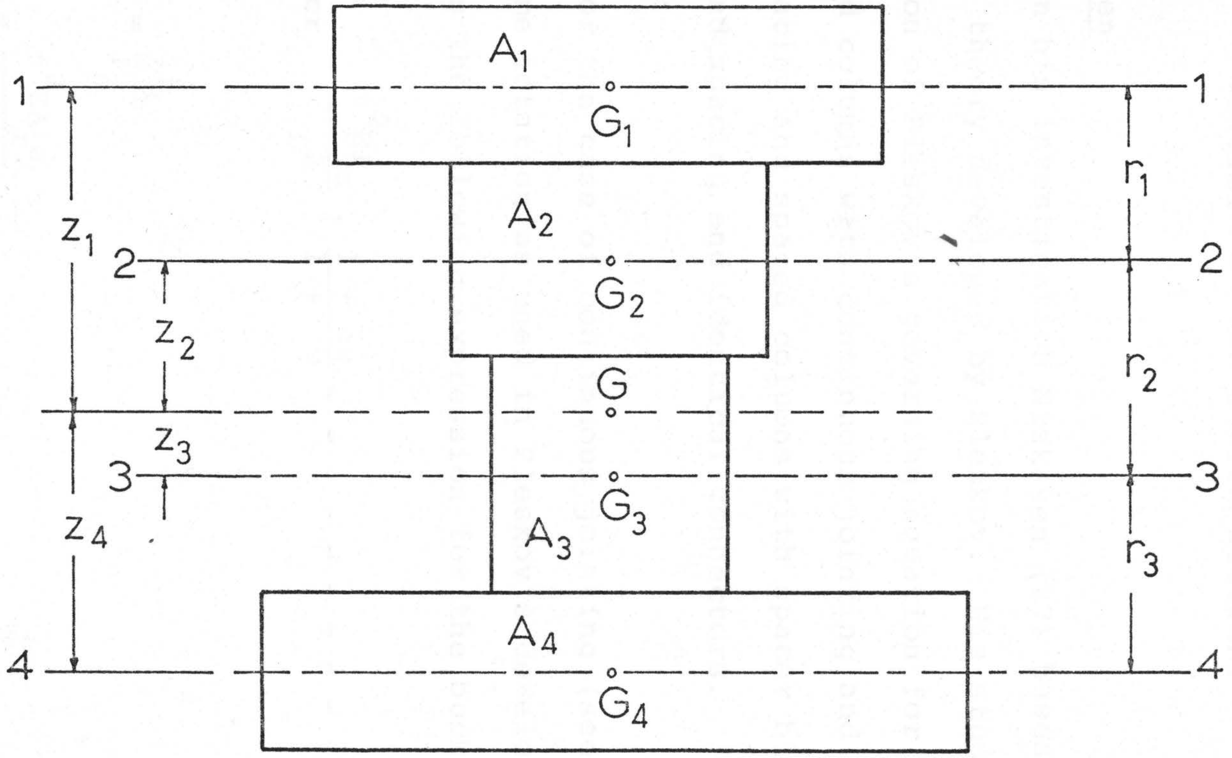


FIGURE 2.8 Cross Section - Pleskov Theory

r_K = distance from centroid of K^{th} layer to centroid of $(K^{th}+1)$ layer.

Z_K = distance from the centroid of the K^{th} layer to the centroid of the entire section.

Niskanen

In his investigation Niskanen (27) bases most of his work on the theory developed by Pleskov. His study deals with the solution of Pleskov's governing equation for the cases of layered columns with continuous jointing and constant connector spacing and spaced columns with spacer blocks of equal size and spacing and identical connectors.

For the case of continuous jointing (see Fig. 2.9) using the same notation as used in Pleskov's development, Niskanen derives the following expression for the buckling load

$$P_{cr} = \frac{\pi^2 EI_s}{L^2} \cdot \frac{1 + \alpha\nu}{1 + \nu} \qquad (2.36)$$

where

$$\alpha = \frac{\Sigma i_K}{I_s}$$

$$\nu = \frac{\pi^2 EA_r a}{2mkL^2}$$

$$A_r = \frac{m}{4}\,\Omega$$

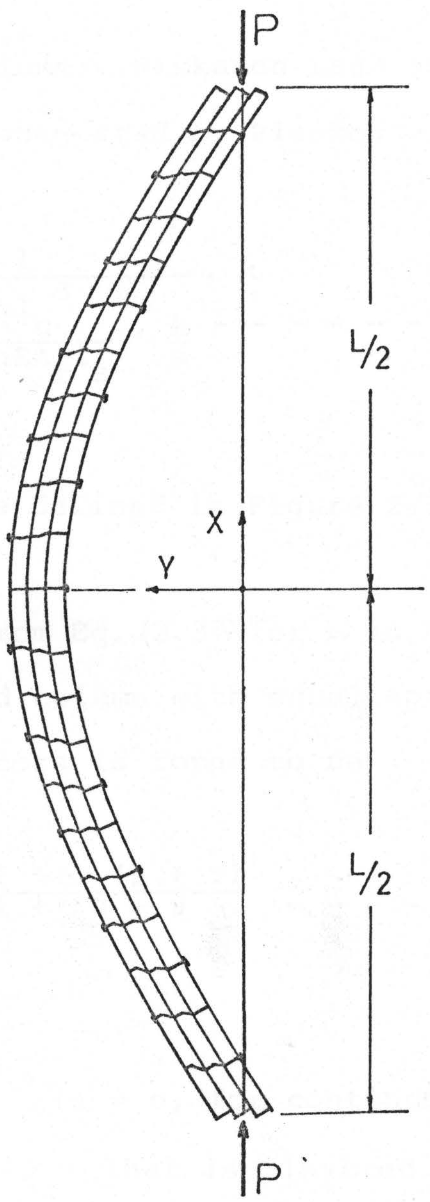


FIGURE 2.9 Buckling of Layered Column with Interlayer Slip

k = shear flow (lb/in), $k = Ga$

a = connector spacing (in)

For spaced columns, Niskanen uses an equivalent slip modulus, $k_{eq.}$, as suggested by Pleskov

$$k_{eq.} = \frac{1}{1 + \frac{m l_c^3}{12 \alpha E A_r l_s} \cdot \frac{k}{a}} \cdot k \qquad (2.37)$$

where l_c and l_s are defined in Figure 2.10.

Substituting k_{eq} from Eq. (2.37) for k in Eq. (2.36) the buckling load for the spaced column with equal spacings and identical spacers and connectors is found to be

$$P_{cr} = \frac{\pi^2 E I_s}{L_2} \; \frac{1 + \alpha\ (\mu + \nu)}{1 + \mu + \nu} \qquad (2.38)$$

where

$$\mu = \frac{\pi^2 l_c^3}{12 \alpha l_s L^2}$$ (μ = o, for continuous jointing, that is, layered columns)

$$\nu = \frac{\pi^2 E A_r a}{m k L^2}$$ (ν = o, for glued connection)

Niskanen's main objective in his study was to verify the formula (2.36) for columns with continuous jointing and formula

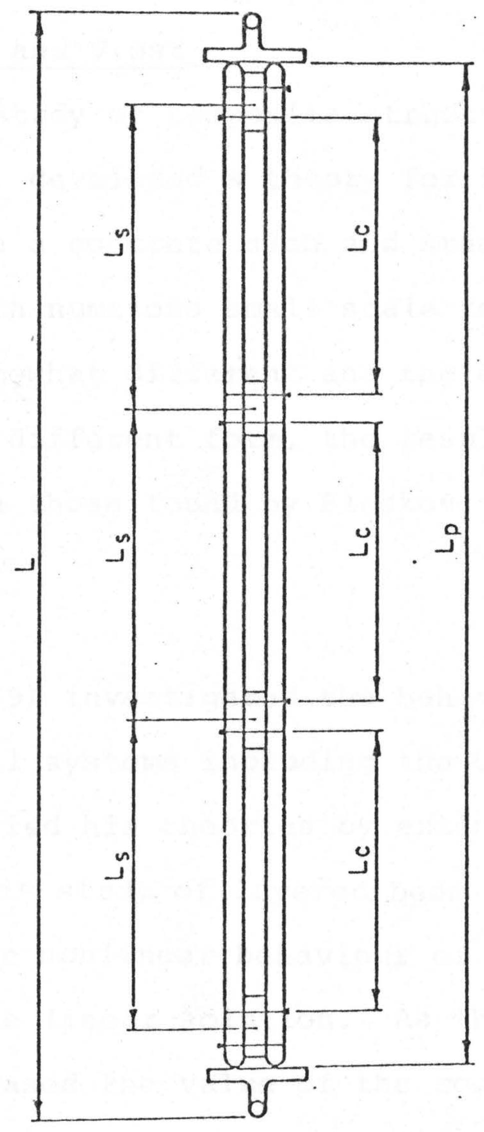


FIGURE 2.10 Spaced Column Showing Dimensions - Niskanen

(2.38) for spaced columns as described above.

Newmark, Siess and Viest

In their study of composite structures Newmark, Siess and Viest (28, 35) developed a theory for treating the problem of slip between a concrete slab and steel girder and verified the results with numerous small scale tests. Although the approach is somewhat different and the derived governing equations have different form, the results of their development agree with those found by Pleskov.

Goodman

Goodman (29) investigated the behaviour of layered beams, plates and shell systems including the effects of interlayer slip. He verified his theories by extensive experimental investigation. In his study of layered beam systems, Goodman took into account the nonlinear behaviour of the connector by using a stepwise linear solution. As the force on the connector is increased the value of the connector stiffness is decreased to correspond to the tangent of the load-slip curve of the connector at that load.

Rassam, and Rassam and Goodman

In his investigation Rassam (30) developed a theory for layered columns with interlayer slip. His theory was developed along the same lines as those used by Goodman (29) in the development of his theory for layered beams with interlayer slip. The results of this investigation have been reported by Rassam and Goodman (31,32,33).

The theoretical development assumes linear elastic materials, therefore, Rassam's theory is applicable only to long columns, that is, elastic columns.

Governing differential equations were derived and solved for columns with two or three layers and for cross-sections having one or two axes of symmetry.

For three equal layers (see Fig. 2.11) and for the case of constant section properties, connector type, and spacing, the governing differential equation has the form

$$\frac{d^4y}{dx^4} + AP\frac{d^2y}{dx^2} - B\frac{d^2y}{dx^2} - C\,P\,y = 0 \qquad (2.39)$$

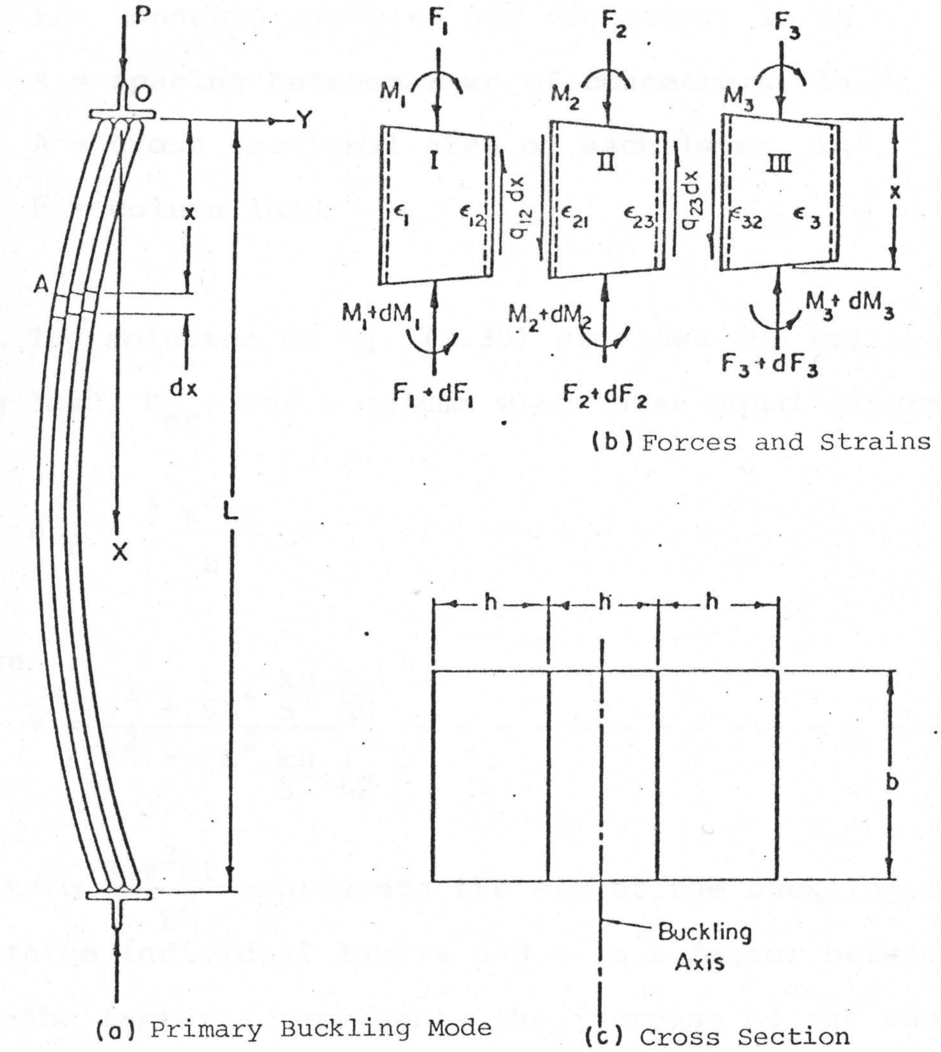
where

$$A = \frac{1}{3EI}$$

$$B = 9\,\frac{nk}{S}\;\frac{1}{AE}$$

$$C = \frac{nk}{S}\;\frac{1}{AE}\cdot\frac{1}{3EI}$$

I = moment of inertia of each layer about its own neutral axis, in^4



(a) Primary Buckling Mode

(b) Forces and Strains

(c) Cross Section

FIGURE 2.11 Three Layered Column - Rassam n = number of connectors per row

k = connector modulus per connector, lb/in.

S = spacing between rows of connectors, in.

A = cross sectional area of each layer, in^2.

P = column load

The solution of Eq. (2.39) provides the critical buckling load, P_{cr}, for a column with three equal layers.

$$P_{cr} = \frac{3\,\pi^2 EI}{L^2}\,\Psi \qquad (2.40)$$

where

$$\Psi = \frac{\pi^2 + 9L^2 \frac{kn}{S}\frac{1}{AE}}{\pi^2 + L^2 \frac{kn}{S}\frac{1}{AE}} \qquad (2.41)$$

The term $\frac{3\,\pi^2 EI}{L^2}$ represents the sum of the buckling loads of the three individual layers and Ψ is a factor between 1 and 3^2. The factor Ψ represents the increase of the buckling load of the layered column due to the presence of the connector.

For a built-up column having three equal layers and uniform connector spacing, Rassam's and Pleskov's equations for critical buckling load are identical. Rewriting Eq. (2.36) in notations used in Eq. (2.40),it yields

$$P_{cr} = \frac{27\pi^2 EI_i}{L^2} \cdot \frac{1 + \frac{1}{9} \cdot \frac{\pi^2 Ea_1 a}{kL^2}}{1 + \frac{\pi^2 Ea_1 a}{kL^2}} \quad \text{---------} \quad (2.42)$$

where the following values have been substituted by definition

$$I = 27\ I_i$$

$$a = \frac{i}{I} = \frac{1}{9}$$

$$A_r = 2a_1$$

If the numerator and demoninator in the second term of Eq. (2.40) are multiplied by $a_1 SE/9Lnk$, it becomes

$$P_{cr} = \frac{3\pi^2 EI_i}{L^2} \cdot 9 \left(\frac{1 + \frac{1}{9} \cdot \frac{\pi^2 Ea_1 S}{kL^2 n}}{1 + \frac{\pi^2 Ea_1 S}{kL^2 n}} \right) \quad \text{---------} \quad (2.43)$$

It can be seen that Eqs. (2.42) and (2.43) are almost the same except for the terms a and S/n. These two terms have values very close to each other. The discrepancy lies in the end distances provided for the nails at the two ends of the column. The values of a and S/n are the same if the sum of the two end distances is equal to the uniform nail spacing in the column.

Malhotra

In a recent study on solid timber columns, conducted by Malhotra (34), buckling stress formula was developed for solid timber columns failing in elastic and inelastic ranges of stress. This was achieved by adopting the tangent modulus column buckling theory. The variation of the tangent modulus, E_t, with stress was elucidated by means of a stress-strain function proposed by Ylinen (36) and verified experimentally for wood by Malhotra (34). Details of this procedure will be given in Chapter 3.

In a further study by Malhotra and Kwan (37) the above mentioned procedure was applied to built-up timber columns. The experimental program was limited to columns built-up of three equal layers and fastened with 2 inch common wire nails. An empirical method, as suggested by Rassam (30), was used to provide high and low estimates of slip modulus.

2.3.3. Discussion

Each of the theories described in the preceding section made its contribution towards better understanding of the behaviour of layered systems. However, these theories have the common limitation that they are applicable only to elastic (or long) columns. In addition, the experimental verification of the theories developed have been rather limited.

The theories developed by Pleskov, as reported by Niskanen , and Rassam provide two rational means for the evaluation of critical stress of laminated columns in the elastic range. In applying these theories to built-up timber columns slip modulus has been determined by empirical methods.

Rassam conducted tests on shear connections similar to the cross-section of column investigation in his study. Slip modulus so obtained was reported in terms of modulus of elasticity and density of the wood. Rassam used a high and low estimate for slip modulus as given below

High estimate: $k \text{ (lb/in)} = -38000 + 80000\, F_1$ - - (2.44)

Low estimate: $k \text{ (lb/in)} = -48000 + 8000\, F_1$ - - (2.45)

where

$$F_1 = \frac{D_1 + D_2}{110} + \frac{E_1 + E_2}{10^7} \qquad (2.46)$$

D_1 = average density of first layer, (lb/ft^3)

D_2 = average density of second layer, (lb/ft^2)

E_1 = modulus of elasticity of first layer, (lb/in^2)

E_2 = modulus of elasticity of second layer, (lb/in^2)

Niskanen, applying Pleskov's theory to continuously jointed layered timber columns and spaced timber columns, estimated the magnitude of the slip modulus in the following manner. For a nailed joint, for which, according to the

"Proposition for New Finnish Design Specifications for Timber Structures" (38), the load per one nail in a single shear joint at well air dry moisture content and within the range of normal nail thickness, S, has an allowable load in kilograms of

$$q_{all.} \approx 440\ S^2 \qquad (2.47)$$

In the said condition of moisture content the relative slip in the joint is about S/10. Therefore slip modulus

$$k \approx 440\ S^2 \div S/10$$

and $$k \approx 4400\ S \qquad (2.48)$$

Since slip modulus has the dimension kg/cm, the numerical factor 4400 must have the dimension kg/cm^2.

2.4. Summary

The method developed by Kuenzi provides a rational means of predicting the initial slip modulus for timber joints. This method seems to be directly applicable in the solution of the problem of interlayer slip which occurs in built-up timber columns.

Two rational formulas are provided by Pleskov and Rassam for the evaluation of critical stresses of built-up timber columns

These formulae are valid only for long columns, that is, columns which fail at stresses within the elastic range.

In a recent study on solid timber columns, Malhotra developed a general procedure whereby the elastic buckling formula for timber columns can be extended to inelastic timber columns.

CHAPTER 3

THEORETICAL CONSIDERATIONS

3.1. General

Jam.

Subject to experimental verification, the works of Kuenzi, Pleskov and Malhotra are adopted as the theoretical basis of this investigation. These theories provide rational procedures for determining slip modulus and column strength, based on fundamental engineering principles and basic material properties, and therefore satisfy the broad objectives as outlined in Chapter 1.

Kuenzi's theory for nailed and bolted timber joints will be applied to the problem of interlayer slip in built-up timber columns. Pleskov's formula will be adopted for determining critical stresses in built-up timber columns and the method proposed by Malhotra, for solid timber columns, will be used to extend Pleskov's elastic column formula to inelastic columns.

3.2. Slip Modulus Formula

Figure 3.1 shows, in an exaggerated manner, the type of connector deformation which occurs when a built-up timber column buckles. Figure 3.1 also shows the connector deformation in single and double shear timber connections. Comparing

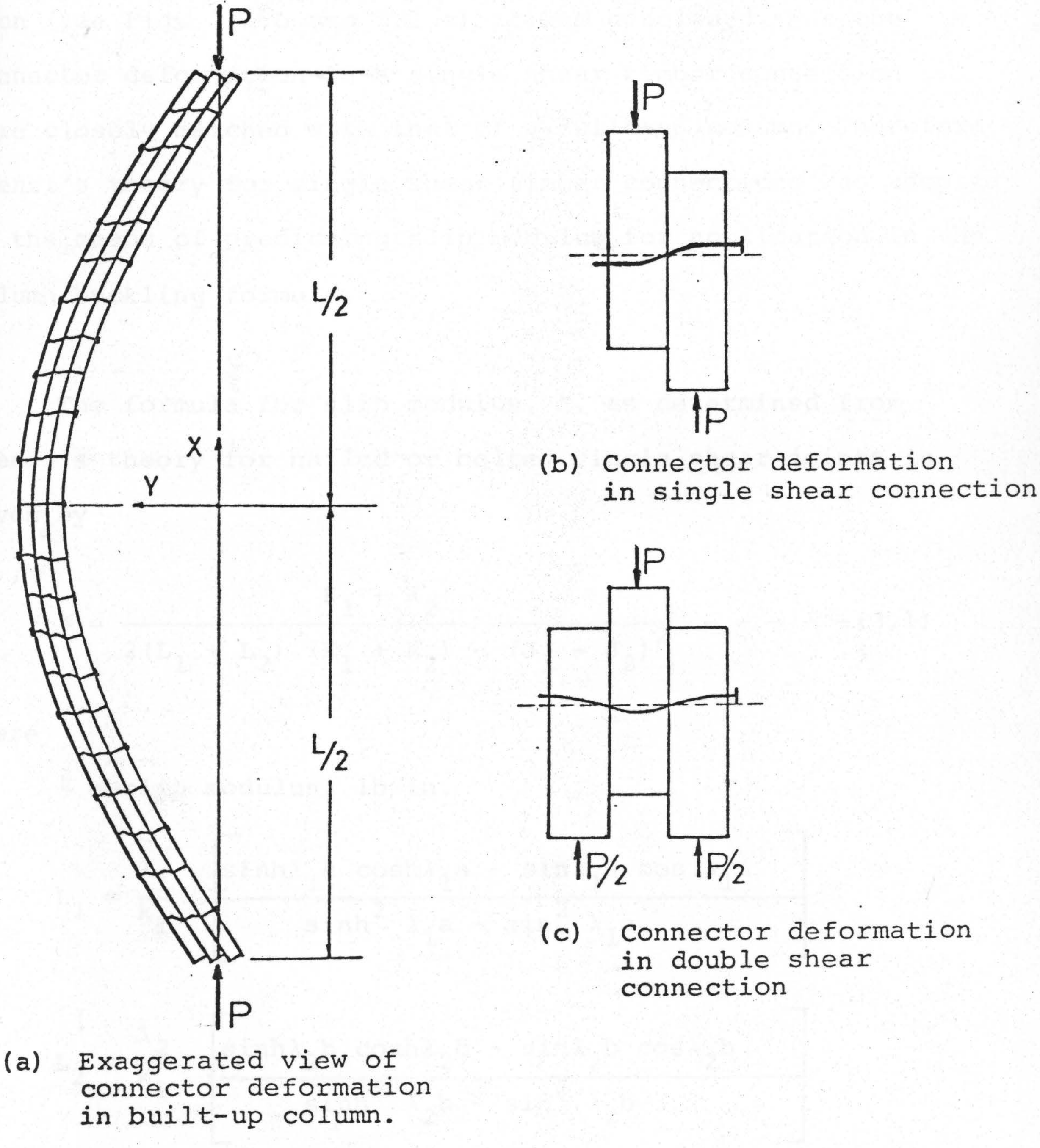


FIGURE 3.1 Connector Deformation in Timber Columns and Timber Shear Connections

the connector deformation in the built-up column (see Fig. 3.1.a) with that of a single or double shear timber connection (see Figs. 3.1.b and 3.1.c), it was concluded that the connector deformation in a single shear timber connection more closely matched with that of a built-up column. Therefore Kuenzi's theory for single shear timber connections was adopted as the means of predicting slip modulus for application in the column buckling formula.

The formula for slip modulus, k, as determined from Kuenzi's theory for nailed or bolted single shear joints is given by

$$k = \frac{K_1 + K_2}{2(L_1 + L_2)(K_1 + K_2) - (J_1 - J_2)^2} \quad \text{- - - - -(3.1)}$$

where

k = slip modulus, lb/in.

$$L_1 = \frac{\lambda_1}{k_1}\left[\frac{\sinh\lambda_1 a \cosh\lambda_1 a - \sin\lambda_1 a \cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right]$$

$$L_2 = \frac{\lambda_2}{k_2}\left[\frac{\sinh\lambda_2 b \cosh\lambda_2 b - \sin\lambda_2 b \cos\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right]$$

$$J_1 = \frac{\lambda_2}{k_1}\left[\frac{\sinh^2\lambda_1 a + \sin^2\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right]$$

$$J_2 = \frac{\lambda_2^{\,2}}{k_2}\left[\frac{\sinh^2\lambda_2 b + \sin^2\lambda_2 b}{\sinh^2\lambda_2 b - \sin^2\lambda_2 b}\right]$$

$$K_1 = \frac{\lambda_1^{\,3}}{k_1}\left[\frac{\sinh\lambda_1 a\,\cosh\lambda_1 a + \sin\lambda_1 a\,\cos\lambda_1 a}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right]$$

$$K_2 = \frac{\lambda_2^{\,3}}{k_2}\left[\frac{\sinh\lambda_2 b\,\cosh\lambda_2 b + \sin\lambda_2 b\,\cos\lambda_2 b}{\sinh^2\lambda_1 a - \sin^2\lambda_1 a}\right]$$

in which a or b equals the member thickness (see Fig. 2.1.) or the depth of penetration of the connector. The subscripts 1 and 2 refer to members 1 and 2 respectively.

In addition

$$\lambda = \sqrt[4]{\frac{\bar{k}}{4EI}}$$

EI = connector stiffness, lb-in^2

$\bar{k}$ = foundation modulus, lb/in^2

$= \dfrac{Ed}{D_e}$, lb/in^2

E = wood modulus of elasticity, lb/in^2

d = connector diameter, in

D_e = foundation depth assumed as 1 inch.

3.3 Stress-Strain Function

To evaluate the buckling strength of columns using the tangent modulus theory it is necessary to approximate the stress-strain diagram of the uniaxial state of stress occurring in the column with a suitable expression. Several investigators (36, 40) have proposed different equations to express the stress-strain diagram. In the present study, the following stress-strain function given by Ylinen (36) is adopted as it is most suitable for elucidating column buckling problems. This function has been verified experimentally for wood by Malhotra (34).

$$\varepsilon = \frac{1}{E} \{cF - (1 - c) \log_e (1 - \frac{F}{F_u}) \} \quad \text{----------(3.2)}$$

where

ε = strain

F = stress

F_u = ultimate compressive stress

c = a constant depending on E, F_u, and the shape of the stress-strain curve beyond the elastic stress limit

Equation (3.2) is derived from an expression of the form

$$E_t = \frac{dF}{d\varepsilon} = \frac{A - F}{B - C.F} \quad \text{--------------------(3.3)}$$

where F is the stress, ε the strain and A, $\overline{B}$, C, are three parameters the values of which depend on the material properties. At the ultimate stress, $F = F_u$ and $\frac{dF}{d\varepsilon} = 0$. Substituting these values into Eq. (3.3),

$$A = F_u \quad \text{-----------------------} \quad (3.4)$$

At the point where F = 0 and $\varepsilon = 0$, $\frac{dF}{d\varepsilon} = E$. Applying these conditions to Eq. (3.3),

$$E = \frac{A}{\bar{B}} = \frac{F_u}{\bar{B}} \quad \text{-----------------------} \quad (3.5)$$

or

$$\bar{B} = \frac{F_u}{E} \quad \text{-----------------------} \quad (3.6)$$

From the values of A and $\bar{B}$, and putting c = CE, Eq. (3.3) becomes

$$E_t = \frac{dF}{d\varepsilon} = E \; \frac{F_u - F}{F_u - cF} \quad \text{---------------} \quad (3.7)$$

or

$$d\varepsilon = \frac{1}{E} \left(\frac{F_u - cF}{F_u - F} \right) dF$$

$$= \frac{1}{E} \left\{ c + \frac{(1-c) F_u}{(F_u - F)} \right\} dF \quad \text{-------------} \quad (3.8)$$

Integrating both sides of Eq. (3.8),

$$\varepsilon = \frac{1}{E} \left\{ cF - (1 - c) F_u . \log_e (F_u - F) + \xi \right\} \text{-} \quad (3.9)$$

where ξ is a constant of integration. Applying the conditions, $\varepsilon = 0$ and $F = 0$, at the origin to Eq. (3.9)

$$\xi = (1 - c)\ F_u .\ \log_e F_u$$

When this value of ε is substituted into Eq. (3.9), it becomes identical to Eq. (3.2).

The shape of the stress strain curve according to Eq. (3.2) is shown in Fig. 3.2. Parameter c can be evaluated by suitably selecting a point "A" anywhere between the elastic and ultimate strength points on the stress-strain diagram. Point "A" should preferably be chosen at a location close to the mid-point between the elastic and ultimate strength points. In Fig. 3.2, point "A" is defined by the slope of the line joining point "A" and the origin. The tangent of the angle is referred to as E_A. δ_A is denoted by

$$\delta_A = \varepsilon_A - \varepsilon_e \text{ - } (3.10)$$

Putting F_A/E_A for ε_A and F_A/E for ε_e, Eq. (3.10) becomes

$$\delta_A = \frac{F_A}{E_A E}\ (E - E_A) \text{ - - - - - - - - - - - - - - - - } (3.11)$$

From Eq. (3.2),

$$\varepsilon_A = \frac{1}{E}\ \{cF_A - (1-c) F_u .\ \log_e (1 - \frac{F_A}{F_u})\} \text{ - - - - - - } (3.12)$$

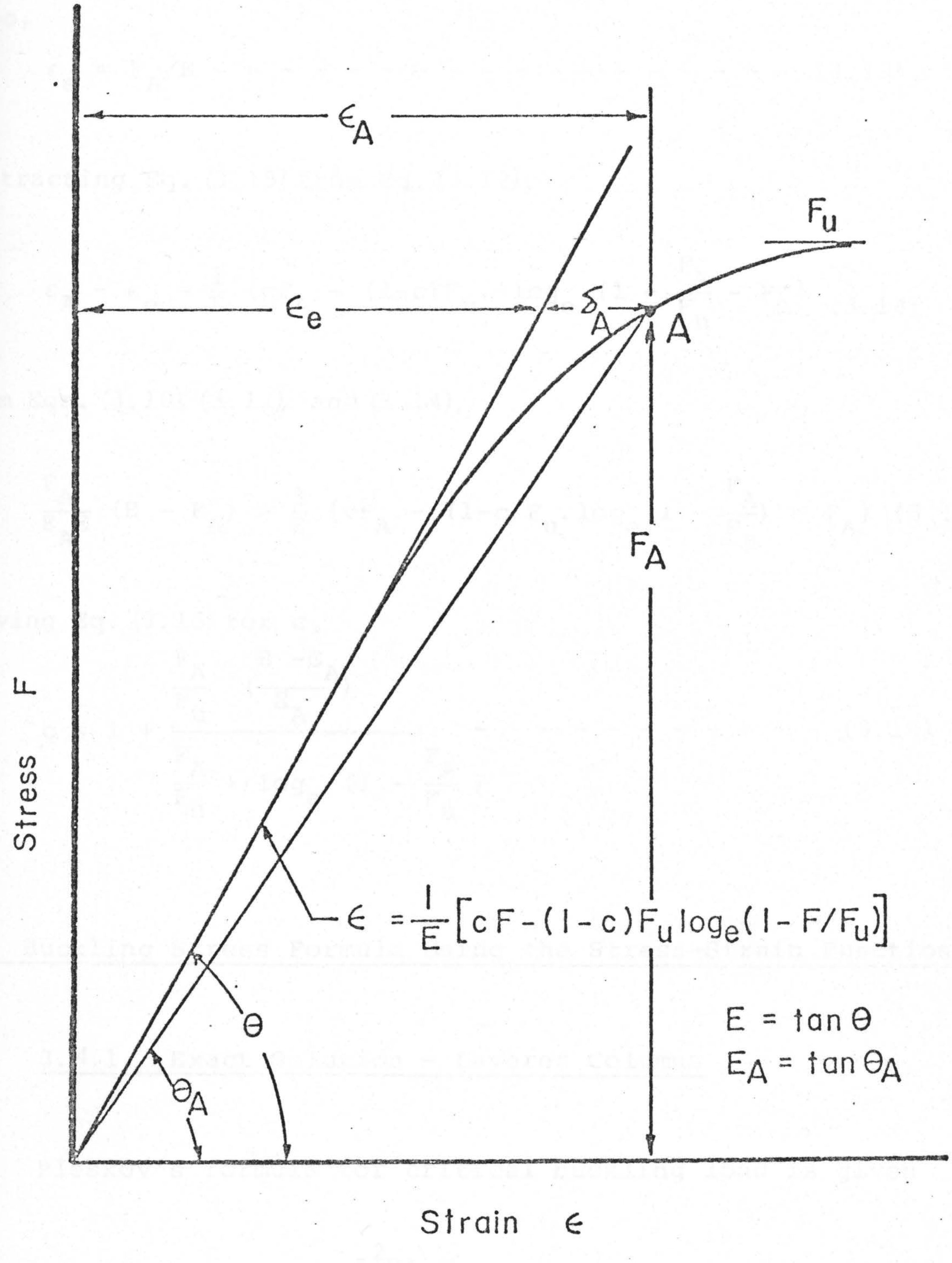


FIGURE 3.2 Stress-Strain Curve

Also,

$$\varepsilon_e = F_A/E \text{ - } (3.13)$$

Subtracting Eq. (3.13) from Eq. (3.12),

$$\varepsilon_A - \varepsilon_e = \frac{1}{E}\{cF_A - (1-c)F_u \cdot \log_e (1 - \frac{F_A}{F_u} - F_A\} \quad (3.14)$$

From Eqs. (3.10), (3.11), and (3.14),

$$\frac{F_A}{E_A E}(E - E_A) = \frac{1}{E}\{cF_A - (1-c)F_u . \log_e (1 - \frac{F_A}{F_u}) - F_A\} \quad (3.15)$$

Solving Eq. (3.15) for c,

$$c = 1 + \frac{\frac{F_A}{F_u}\left(\frac{E - E_A}{E_A}\right)}{\frac{F_A}{F_u} + \log_e\left(1 - \frac{F_A}{F_u}\right)} \text{ - - - - - - - - - - - - } (3.16)$$

3.4. Buckling Stress Formula Using the Stress-Strain Function

3.4.1. Exact Solution - Layered Columns

Pleskov's formula for critical buckling load is given by

$$P_{cr} = \frac{\pi^2 EI}{\lambda^2} \cdot \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2mkL^2}} \text{ - - - - - - - - - - - } (3.17)$$

To make Eq. (3.17) applicable to layered columns in elastic and inelastic ranges of stress, E_t is substituted in place of E, where E_t=tangent modulus of elasticity=slope of stress-strain curve at any value of stress. Now, dividing both sides of Eq. (3.17) by A,

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot \frac{1 + \alpha \dfrac{\pi^2 E_t A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E_t A_r a}{2mkL^2}} \qquad (3.18)$$

where

$F_{cr} = \frac{P_{cr}}{A}$ = critical column buckling stress, lb/in^2

A = area of entire cross-section, in^2

$\lambda = \frac{L}{r}$ = slenderness ratio of the column

$r = \sqrt{\frac{I}{A}}$ = radius of gyration, in

I = moment of inertia of column cross section, in^4

Substituting the value of E_t from Eq. (3.7) (with F replaced by F_{cr}) into Eq. (3.17) results in

$$F_{cr} = \frac{\pi^2}{\lambda^2} \cdot E\left(\frac{F_u - F_{cr}}{F_u - cF_{cr}}\right) \cdot \frac{1 + \alpha \cdot \dfrac{\pi^2 A_r a}{2mkL^2} \cdot E\left(\dfrac{F_u - F_{cr}}{F_u - cF_{cr}}\right)}{1 + \dfrac{\pi^2 A_r a}{2mkL^2} \cdot E\left(\dfrac{F_u - F_{cr}}{F_u - cF_{cr}}\right)} \quad \text{- - - - - - -(3.19)}$$

On rearranging the terms in Eq. (3.19), the following polynomial of third order is obtained.

$$C_1 F_{cr}^3 - C_2 F_{cr}^2 + C_3 F_{cr} - C_4 = 0 \quad \text{- - - - - - - - (3.20)}$$

where

$$C_1 = c^2\lambda^2 + c.e.\lambda^2. \frac{\pi^2 A_r.a}{2mkL^2}$$

$$C_2 = 2c.\lambda^2 F_u + \pi^2 c.E. + (1+c)\lambda^2.E.F_u. \frac{\pi^2 A_r a}{2mkL^2} + \pi^2.\alpha.E^2 \frac{\pi^2 A_r a}{2mkL^2}$$

$$C_3 = \left(1 + E . \frac{\pi^2 A_r a}{2mkL^2}\right).\lambda^2 F_u^2 + (1+c).\pi^2.E.F_u + 2\pi^2.E^2.F_u\,\alpha. \frac{\pi^2 A_r a}{2mkL^2}$$

$$C_4 = (1 + \alpha . E \frac{\pi^2 A_r a}{2mkL^2}) . \pi^2 . E . F_u{}^2$$

Equation (3.19) is an exact formula obtained by incorporating the tangent modulus concept into Pleskov's theory for layered timber columns. Equation (3.19) is applicable to layered timber columns of any slenderness ratio.

3.4.2. Approximate Solution - Layered Columns

Equation (3.19) can be solved quite easily if the factor

$$\frac{1 + \alpha\ \pi^2 E_t A_r a / 2mkL^2}{1 + \ \ \pi^2 E_t A_r a / 2mkL^2}$$ is taken to be not significantly affected

by replacing E_t by E. With this assumption, Eq. (3.19) is reduced to

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} . B \text{ - } (3.21)$$

where

$$B = \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2mkL^2}}$$

Substituting the value of E_t from Eq. (3.7) (with F replaced by F_{cr}) into Eq. (3.21) results in

$$F_{cr} = \frac{\pi^2}{\lambda^2} BE \left(\frac{F_u - F_{cr}}{F_u - cF_{cr}}\right)$$

or

$$c\lambda^2 . F_{cr}^{\ 2} - (B\pi^2 E + F_u \lambda^2) F_{cr} + B\pi^2 E F_u = 0 \quad --- \quad (3.22)$$

Solving Eq. (3.22) for F_{cr} yields

$$F_{cr} = \frac{B\pi^2 E + F_u \lambda^2}{2c\lambda^2} - \frac{\sqrt{(B\pi^2 E + F_u \lambda^2)^2 - 4B.c\lambda^2 \pi^2 E.F_u}}{2c\lambda^2} \quad (3.23)$$

Since the buckling stress must vanish when $\lambda \to \infty$, only negative sign has been selected for the second term in Eq. (3.23).

Theoretical column stress predictions will be determined using both exact and approximate solutions.

It might be mentioned here that the results obtained by Malhotra and Kwan (37) indicate an insignificant difference between the exact and approximate solutions.

3.4.3. Approximate Solution - Spaced Columns

For spaced columns, Pleskov (6) shows that the buckling stress, F_{cr}, can be determined if the slip modulus, k, in Eq. (3.21) is replaced with the "equivalent slip modulus"

$$k_{eq.} = \frac{1}{1 + \frac{m l_c^3}{12\alpha E A_r l_s} \cdot \frac{k}{a}} \cdot k \text{ ------------ } (3.24)$$

The significance of l_c and l_s can be seen from Fig.2.10. Substitution of $k_{eq.}$ into Eq. (3.21) yields

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot \frac{1 + \alpha\ (\mu+\nu)}{1 + \mu + \nu} \text{ -------------- } (3.25)$$

$$\text{where } \mu = \frac{\pi^2 l_c^3}{12\alpha l_s L^2} \text{ --------------------- } (3.25a)$$

($\mu = 0$ for continuously jointed columns)

$$\text{and } \nu = \frac{\pi^2 E A_r a}{m k L^2} \text{ ---------------------- } (3.25b)$$

($\nu = 0$ for glued columns)

Applying the same procedure used to obtain Eq. (3.23), the critical stress for a spaced column with equal spacings and identical connectors

$$F_{cr} = \frac{B\pi^2 E + F_u \lambda^2}{2c\lambda^2} - \frac{\sqrt{(B\pi^2 E + F_u \lambda^2)^2 - 4Bc.\lambda^2 \pi^2 E.F_u}}{2c\lambda^2} \quad (3.26)$$

where

$$B = \frac{1 + \alpha\ (\mu+\nu)}{1 + \mu + \nu},$$ μ and ν as given in Eqs. (3.25a) and (3.25b).

3.4.4. Approximate Solution - Braced Column with 45° Braces

Equation (3.26), for spaced columns, is applied to braced columns with 45° braces. At the intersection of the braces, points a, b, c in Fig. 3.3(a), the action of the braces, at the instant of buckling, is assumed to be similar to spacer blocks in a spaced column.

Therefore, the formula for braced columns with 45° braces is given by

$$F_{cr} = \frac{B\pi^2 E + F_u\lambda^2}{2c\lambda^2} - \frac{\sqrt{(B\pi^2 E + F_u\lambda^2)^2 - 4Bc\lambda^2\pi^2 EF_u}}{2c\lambda^2} \quad \text{--- (3.27)}$$

where

$$B = \frac{1 + \alpha \dfrac{\pi^2 EA_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 l_s L^2}}{1 + \dfrac{\pi^2 EA_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12\alpha l_s L^2}} \quad \text{----------- (3.28)}$$

The quantities l_c and l_s are determined as for spaced columns (see Fig. 3.3). Two interfaces are assumed as is the case for spaced columns, therefore, m = 2 in Eq.(3.28). The

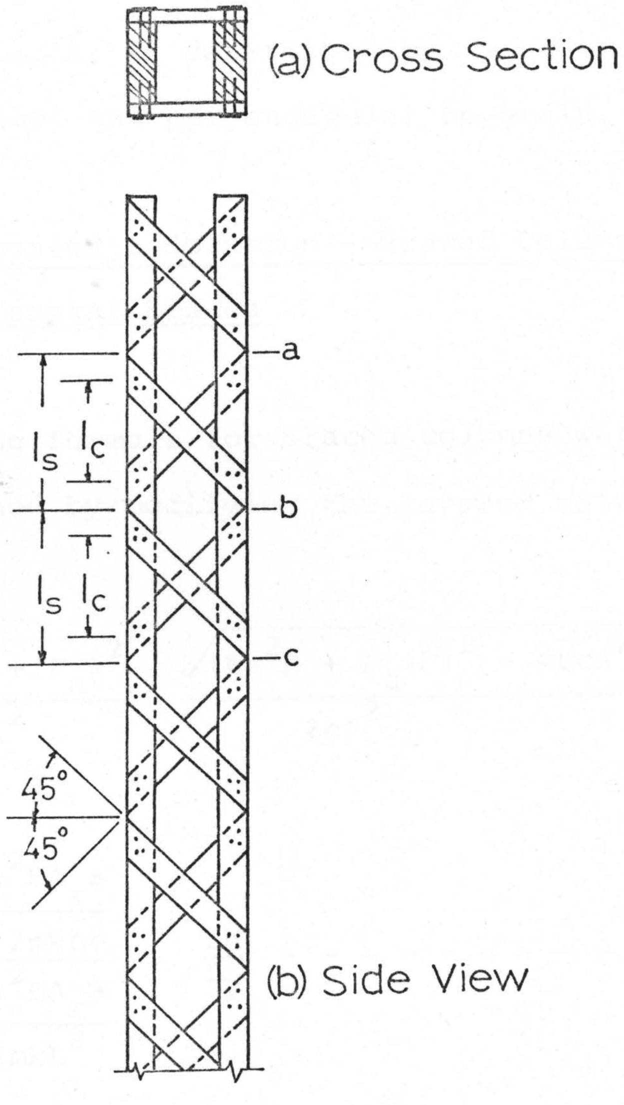


FIGURE 3.3 Braced Columns with 45° Braces

quantity a in Eq. (3.28) is given by

$$a = \frac{\text{laminate length}}{\frac{1}{2} \times \text{total number of connectors}}$$

Connector modulus, k, is determined by using the average value of E parallel and perpendicular to grain.

3.4.5. Approximate Solution - Braced Columns with Horizontal Braces

The buckling formula for braced columns with horizontal braces is obtained by modifying the layered column formula and is given by

$$F_{cr} = \frac{B\pi^2 E + F_u\lambda^2}{2c\lambda^2} - \frac{\sqrt{(B\pi^2 E + F_u\lambda^2)^2 - 4Bc\lambda^2\pi^2 EF_u}}{2c\lambda^2} \quad (3.29)$$

where

$$B = \frac{1 + \alpha \dfrac{\pi^2 EA_r a}{2mkL^2}}{1 + \dfrac{\pi^2 EA_r a}{2mkL^2}} \cdot \frac{\Sigma h}{L_1} \text{ - - - - - - - - - - - - - - } (3.30)$$

The significance of h and L_1 is shown in Fig. 3.4(b). The factor B given in Eq. (3.30) is obtained from the layered column formula and reduced by the factor $\Sigma h/L_1$ to account for the discontinuity in the bracing along the length of the column. The number of interfaces, m, is taken as 2 (see

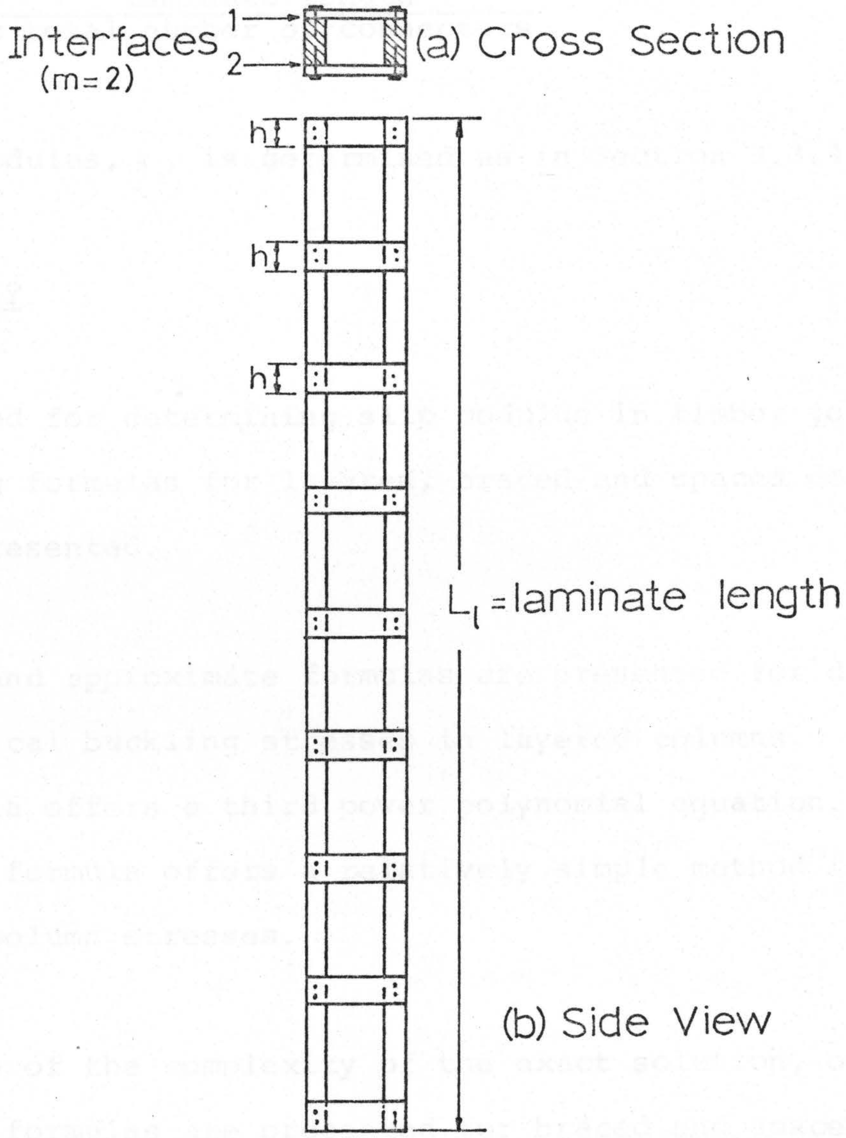


FIGURE 3.4 Braced Column with Horizontal Braces

Fig. 3.4) and the quantity a in Eq. (3.30) is given by

$$a = \frac{\text{laminate length}}{\frac{1}{2} \times \text{total number of connectors}}$$

Connector modulus, k , is determined as in Section 3.3.4.

3.5. Summary

A method for determining slip modulus in timber joints and buckling formulas for layered, braced and spaced columns have been presented.

Exact and approximate formulas are presented for determining critical buckling stresses in layered columns. The exact formula offers a third power polynomial equation. The approximate formula offers a relatively simple method for evaluating column stresses.

Because of the complexity of the exact solution, only approximate formulas are presented for braced and spaced columns. It is assumed at this point that the results to be obtained using the exact and approximate formulas will differ only a few percent, as reported by Malhotra and Kwan (34) for layered columns.

CHAPTER 4

SELECTION AND FABRICATION OF TEST SPECIMENS

4.1. Materials

4.1.1. Connectors

The types of connectors used in this investigation include ten sizes of common wire nails, eight sizes of hexagonal head steel bolts and one size of split ring connectors. A complete list of these connectors is given in Tables 4.1 and 4.2.

4.1.2. Lumber

Three separate lots of Construction Grade No. 1 Eastern Spruce lumber, each approximately 3,000 board foot measure and consisting of nominal sizes of 1 X 4, 1 X 6, 2 X 4, 2 X 6, 3 x 4 and 4 X 4, were purchased from a central Nova Scotian mill.

The lumber, which was received in a wet condition, was open air dried to approximately 25 percent moisture content before it was cut to the desired lengths for the connection or column specimens. The first lot of lumber was used to fabricate the connection specimens; the second and third lots

TABLE 4.1. COMMON WIRE NAILS SELECTED FOR PROPERTY INVESTIGATION

NOMINAL NAIL LENGTH (in.)	NOMINAL GAUGE SIZE	NOMINAL NAIL DIAMETER (in.)
2	12	0.104
2 1/4	11	0.116
2 1/2	10	0.128
3	9	0.144
3 1/2	7	0.176
4	6	0.192
4 1/2	5	0.213
5	4	0.232
5 1/2	3	0.253
6	2	0.276

TABLE 4.2. STEEL BOLTS SELECTED FOR PROPERTY INVESTIGATION

NOMINAL SIZE	NOMINAL DIAMETER (in.)
1/4"ø x 6"	0.250
3/8"ø x 5"	0.375
3/8"ø x 6"	0.375
3/8"ø x 7"	0.375
3/8"ø x 8"	0.375
1/2"ø x 6"	0.500
1/2"ø x 7"	0.500
1/2"ø x 8"	0.500

were used to fabricate the columns. After cutting, each length was visually inspected and all severely knotted and cross-grained pieces were rejected. Each acceptable length was given an identification number which was then given to subsequent specimens obtained from these original lengths.

Each piece of lumber selected was of such a length as to provide enough material for the required length of the main test specimens (connections or columns), two compression test specimens plus a few extra inches to facilitate the squaring off of the ends of each test specimen. This is shown schematically in Fig. 4.1. The connection members were of such lengths that relatively clear material could be obtained for the connection specimens. Because of the length of the columns, knots could not be avoided in the column laminates. However, the diameter of the largest knot or knots at any section was measured and recorded for later use.

From each piece of lumber previously selected and identified a four inch length was removed from the lumber of one inch nominal thickness and a seven inch length was removed from the lumber having a nominal thickness of two inches or more. Each four and seven inch length so obtained was given the same identification as the original length from which it was removed. These four and seven inch lengths were required for fabricating the compression test specimens necessary for

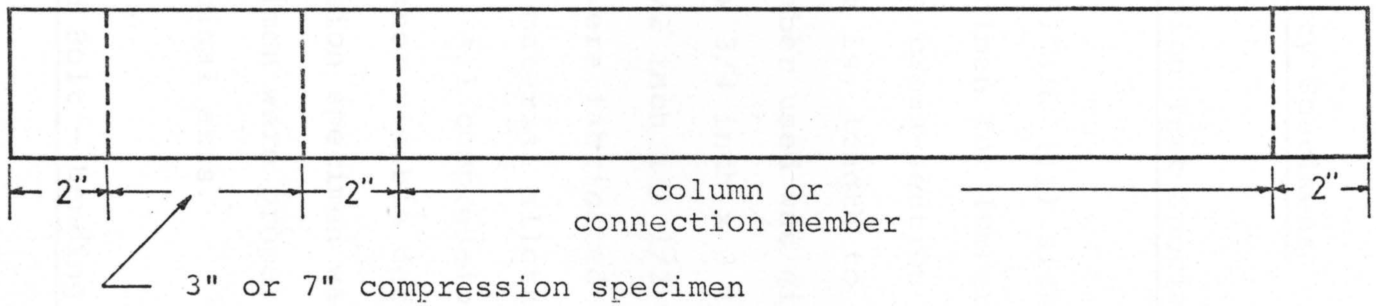


FIGURE 4.1 Selection of Column or Connection Members and Corresponding Material Property Specimens

determining the strength properties of the corresponding connection members or column laminates.

4.2. Material Property Specimens

4.2.1. Compression Test Specimens

The standard A.S.T.M. (41) size parallel to grain compression test specimen for lumber has a length (L) of 8 inches and a square cross-section 2 inches x 2 inches (depth, d = 2"), that is, length to depth ratio (l/d) equals 4. Since most of the lumber used was either 3/4 or 1 1/2*inches thick, two 3/4 inch x 3/4 inch x 3 inch compression specimens (l/d = 4) or two 1 1/2 inch x 1 1/2 inch x 6 inch compression specimens (l/d = 4) were fabricated from each four inch or seven inch length of material allotted for determination of strength properties. All compression specimens selected were free from knots or other visible defects. The longitudinal axis of each compression specimen was parallel to grain and the ends of the specimen were properly squared off perpendicular to the longitudinal axis.

4.2.2. Nail and Bolt - Bending Test Specimens

To determine the mechanical and physical properties of each size of common wire nail or steel bolt, a sample consist-

*Actual dimensions (Nominal dimensions: 1 or 2 inches).

ting of ten of each size of common wire nails and ten of each size of steel bolts were randomly and independently chosen from the total quantity of nails and bolts purchased. In addition a sample of ten 2 1/2 inch diameter split ring connectors was selected for determining the physical properties of the connectors.

4.3. Fabrication of Connections

To minimize splitting during fabrication the individual connection members were initially conditioned in the moisture control chamber to a uniform moisture content of approximately 12 percent.

At the time of fabrication 0.025 inch thick spacers were placed between the members to satisfy the assumption of no friction between the wood members. These spacers were removed from the connection prior to testing.

Common wire nails 3 1/2 inches or smaller were driven directly into the wood members. Connections fabricated with steel bolts or common wire nails larger than 3 1/2 inches were prebored in such a manner as to provide a snug fit for the connector. To ensure that the connections were symmetrically loaded the connections were properly squared off at the ends.

All severely split connections were rejected and replaced. In addition, the corresponding compression test specimens were also replaced.

After fabrication the connections were returned to the moisture control chamber to maintain a steady moisture content of about 12 percent. A total of 250 connections were fabricated including connections with 3, 5 and 7 members, that is, double and multiple shear connections. A breakdown of the connections fabricated is given in Table 7.1 (Chapter 7).

4.4. Fabrication of Columns

Column laminates were initially conditioned to a moisture content of approximately 12 percent in the moisture control chamber.

Prior to fabrication of the built-up columns any laminate found to be warped was rejected and replaced. The corresponding compression test specimen was also replaced. The laminates were clamped together and the nailing pattern was laid out on the outer members. Holes were prebored in laminates which were fastened with bolts or common wire nails larger than 3 1/2 inches. The laminates were then fastened together with nails or bolts and the ends of the built-up column were squared off, perpendicular to the longitudinal axis, to the required column length. One set of columns was fabricated with

$2\frac{1}{2}$ inch diameter split ring connectors used in conjunction with $\frac{1}{2}$ inch diameter bolts.

The column specimens were conditioned to approximately 12 percent moisture content in the moisture control chamber. In total 400 columns, covering the range of slenderness ratios from 34 to 150, were fabricated. These included layered columns having one or two axes of symmetry and built-up from two to seven laminates, braced columns with 45 degree or horizontal braces, with or without end blocks, and one set of spaced columns. A breakdown of the columns fabricated is given in Chapter 8.

The compression test specimens matched with corresponding columns and connections were also conditioned to about 12 percent moisture content. The strength properties so obtained from the compression tests could then be used directly in the theoretical analysis of the main test specimens.

CHAPTER 5

INSTRUMENTATION AND TEST PROCEDURE

5.1. General

Special gauges, designed and made using metal foil electrical resistance strain gauges bonded to spring steel or to beryllium copper metal, were used in combination with an X-Y Chart Drive Amplifier, to measure deformations.

The gauges were calibrated using a Huggenberger portable calibrator having a least count of $\pm$ 0.001 centimeters or using a Batty dial gauge having a least count of $\pm$ 0.0001 inches. All gauges were found to provide a linear relationship between the chart movement and the deformation being measured.

Prior to testing, all connections, columns and their corresponding compression test specimens were conditioned to and maintained at a moisture content of approximately 12 percent.

5.2. Nail and Bolt - Bending Tests

The diameter of the nails and bolts were measured to an accuracy of $\pm$ 0.001 inches.

Nail and bolt bending tests were conducted using a simply supported nail or bolt with load applied at mid-span. Deflection were measured using a Batty dial gauge ($\pm$ 0.0001 inches). The test set-up is shown in Plate 5.1.

A load, as shown in Tables 5.1 and 5.2 was manually applied to the nail or bolt. The resulting deflection was determined as the difference between the maximum dial gauge reading and the initial zero reading. This deflection was used to compute nail and bolt stiffness.

5.3. Compression Tests

5.3.1. Instrumentation and Test Set-Up

The relevant details of the compression gauge are shown in Figure 5.1. This gauge was calibrated using a portable Huggenberger calibrator ($\pm$ 0.001 cm.) as shown in Plate 5.2 and resulted in a strain of 0.00108 inch per inch for one centimeter of chart movement.

Plate 5.3 shows the gauge being used during a compression test. The entire test set-up is shown in Plate 5.4. The tests were conducted using an Instron Universal Testing Machine having a maximum load capacity of 10,000 Kilograms.

TABLE 5.1 SPAN AND LOADS USED TO DETERMINE MATERIAL PROPERTIES OF THE COMMON WIRE NAILS

NAIL LENGTH (in)	SPAN (in)	LOAD APPLIED FOR DETERMINATION OF STIFFNESS (lb)
2	1.0	22.0
2 1/4	1.5	22.0
2 1/2	1.5	44.0
3	2.0	44.0
3 1/2	2:5	44.0
4	3.0	44.0
4 1/2	3.5	44.0
5	4.0	44.0
5 1/2	4.5	44.0
6	5.0	44.0

Table 5.2 SPAN AND LOADS USED TO DETERMINE MATERIAL PROPERTIES OF THE STEEL BOLTS

NOMINAL BOLT SIZE	SPAN (in)	LOAD APPLIED FOR DETERMINATION OF STIFFNESS (lb)
1/4"ø x 6"	4.0	44.0
3/8"ø x 5"	3.0	44.0
3/8"ø x 6"	4.0	44.0
3/8"ø x 7"	5.0	44.0
3/8"ø x 8"	6.0	44.0
1/2"ø x 6"	4.0	66.0
1/2"ø x 7"	5.0	66.0
1/2"ø x 8"	6.0	66.0

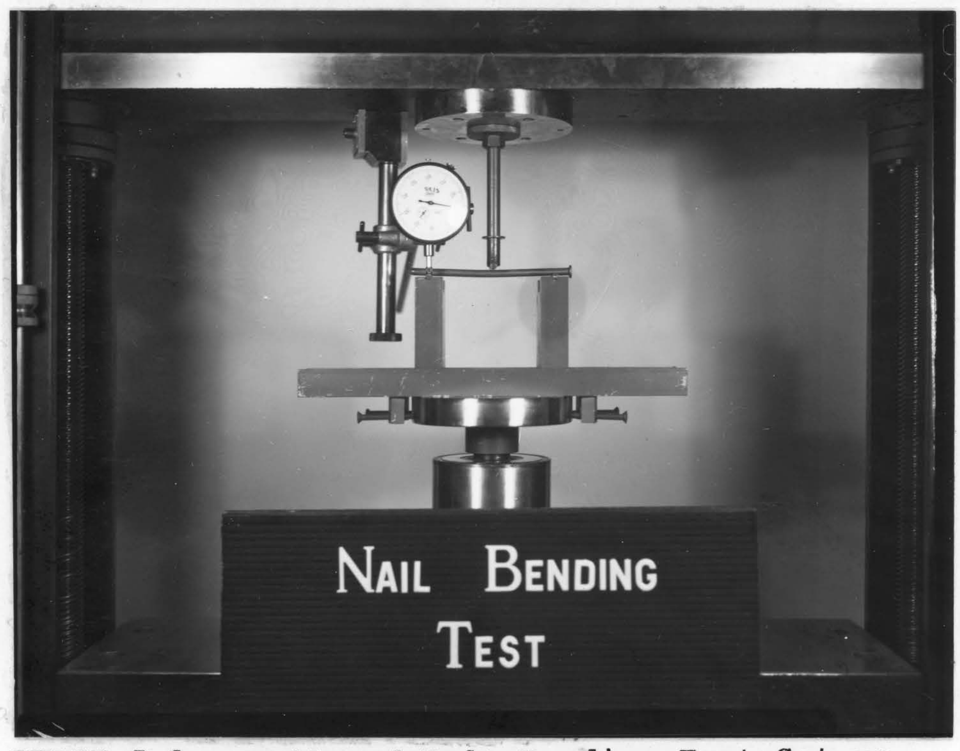


PLATE 5.1 Nail and Bolt Bending Test Set-up

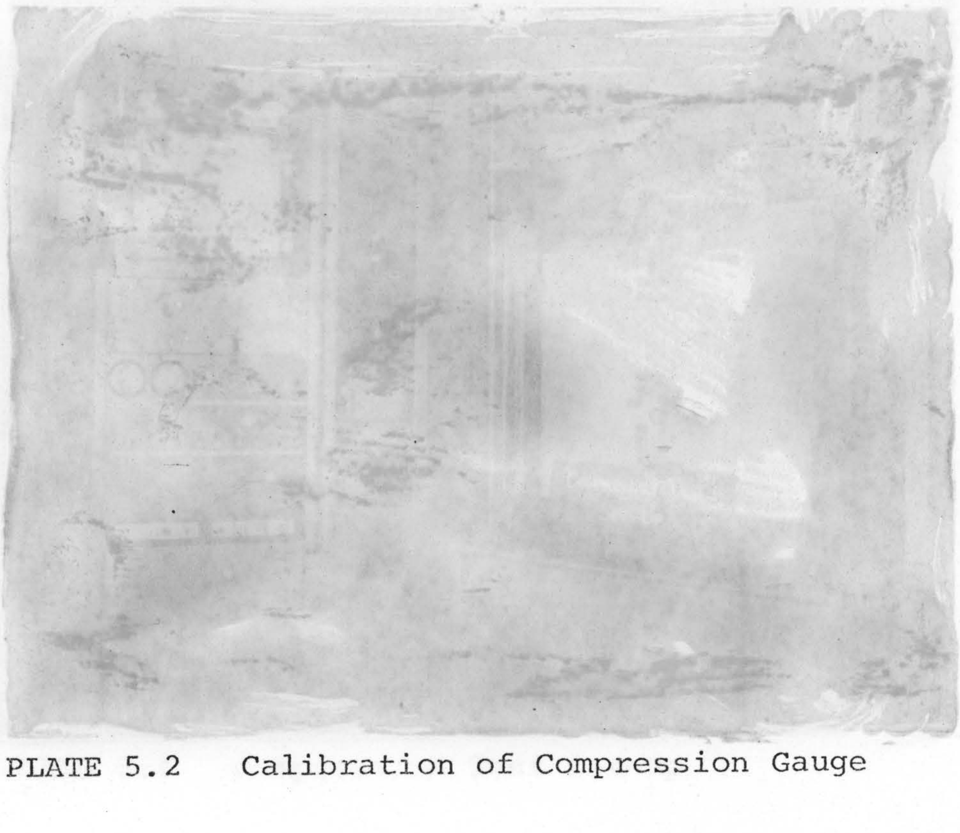


PLATE 5.2 Calibration of Compression Gauge

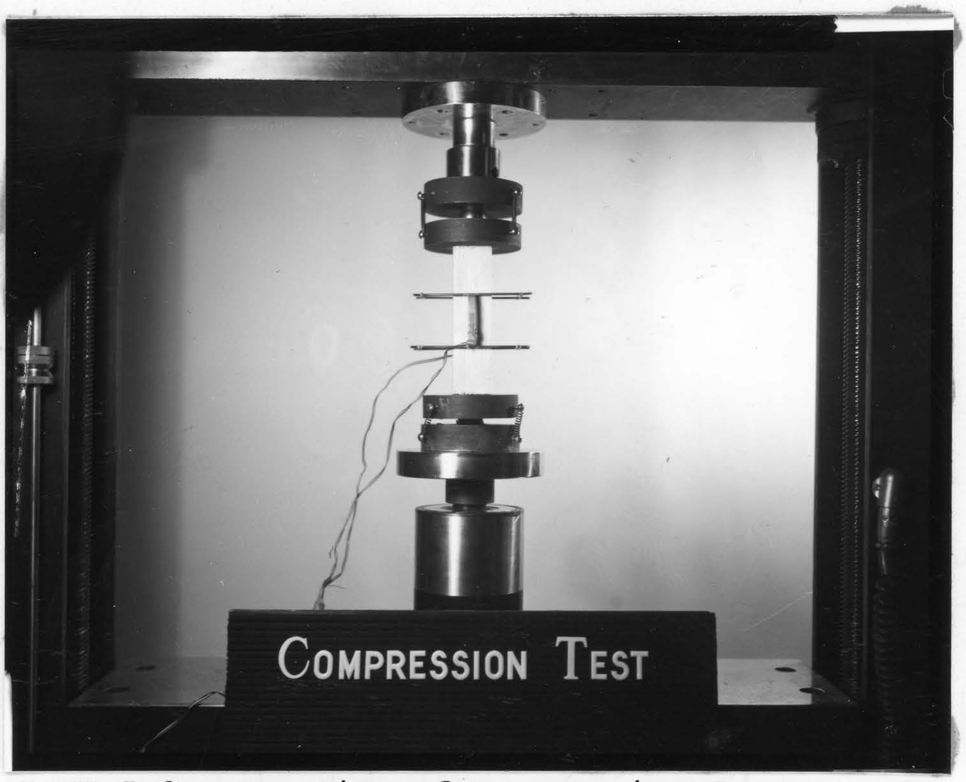
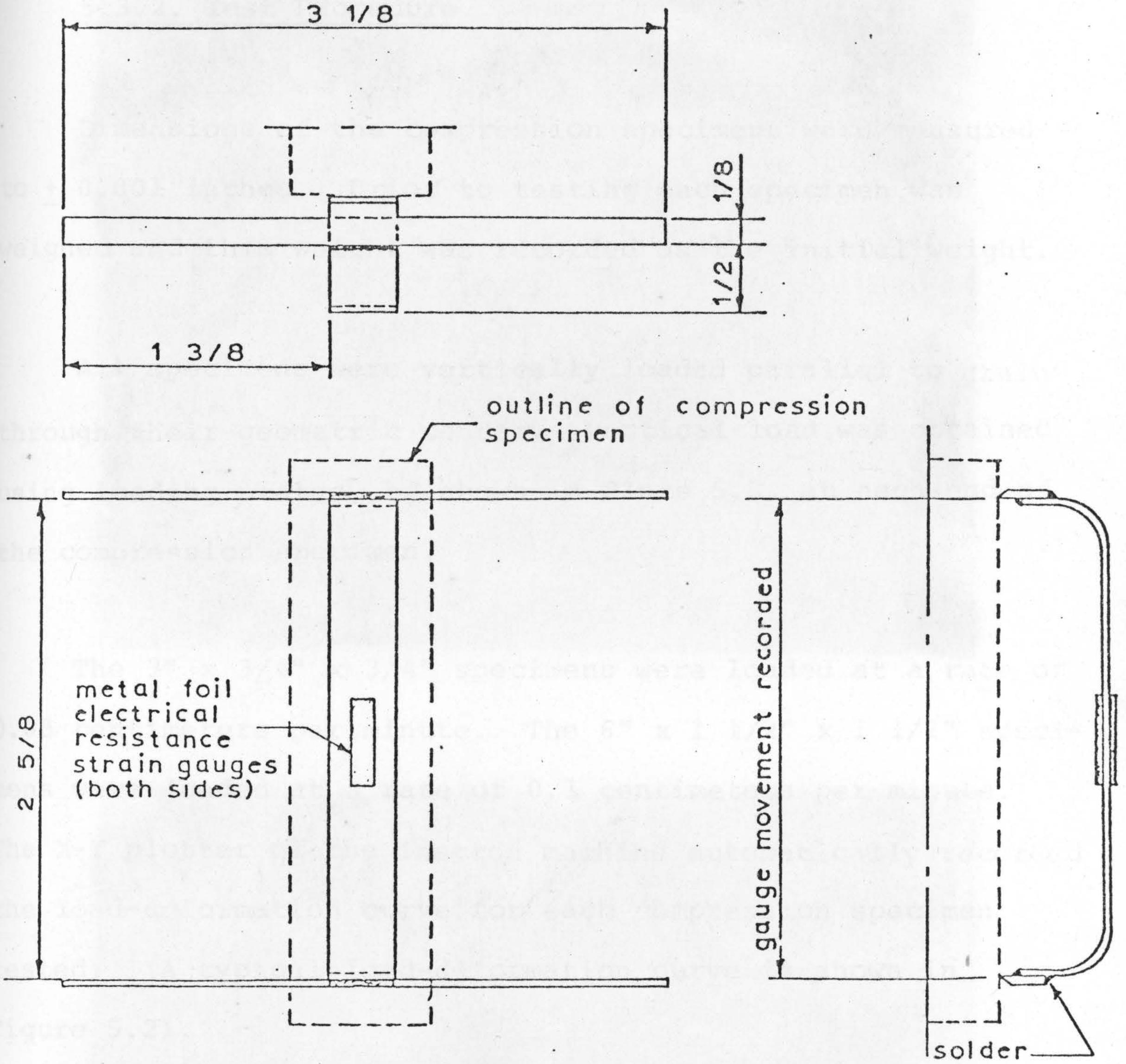


PLATE 5.3 Location of Compression Gauge

PLATE 5.4 Compression Test Set-up



NOTE
- only important dimensions given
- gauge shaped from 3/8 x 0.035 inch spring steel
- 2 required for compression gauge

FIGURE 5.1 Details of Compression Gauge

5.3.2. Test Procedure

Dimensions of the compression specimens were measured to ± 0.001 inches. Prior to testing each specimen was weighed and this weight was recorded as the initial weight.

All specimens were vertically loaded parallel to grain through their geometric center. Vertical load was obtained using loading plates, as shown in Plate 5.3, at each end of the compression specimen.

The 3" x 3/4" x 3/4" specimens were loaded at a rate of 0.05 centimeters per minute. The 6" x 1 1/2" x 1 1/2" specimens were loaded at a rate of 0.1 centimeters per minute. The X-Y plotter of the Instron machine automatically recorded the load-deformation curve for each compression specimen tested. (A typical load-deformation curve is shown in Figure 5.2).

5.4. Moisture Content and Specific Gravity Test

The compression specimens were oven-dried (105°C) after testing and the final weights recorded. Moisture content at test was obtained using the difference in weight at test and oven-dried weight*. Specific gravity

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$$\text{*Moisture content, \%} = \frac{\text{Weight at test - oven-dried weight}}{\text{oven-dried weight}} \times 100$$

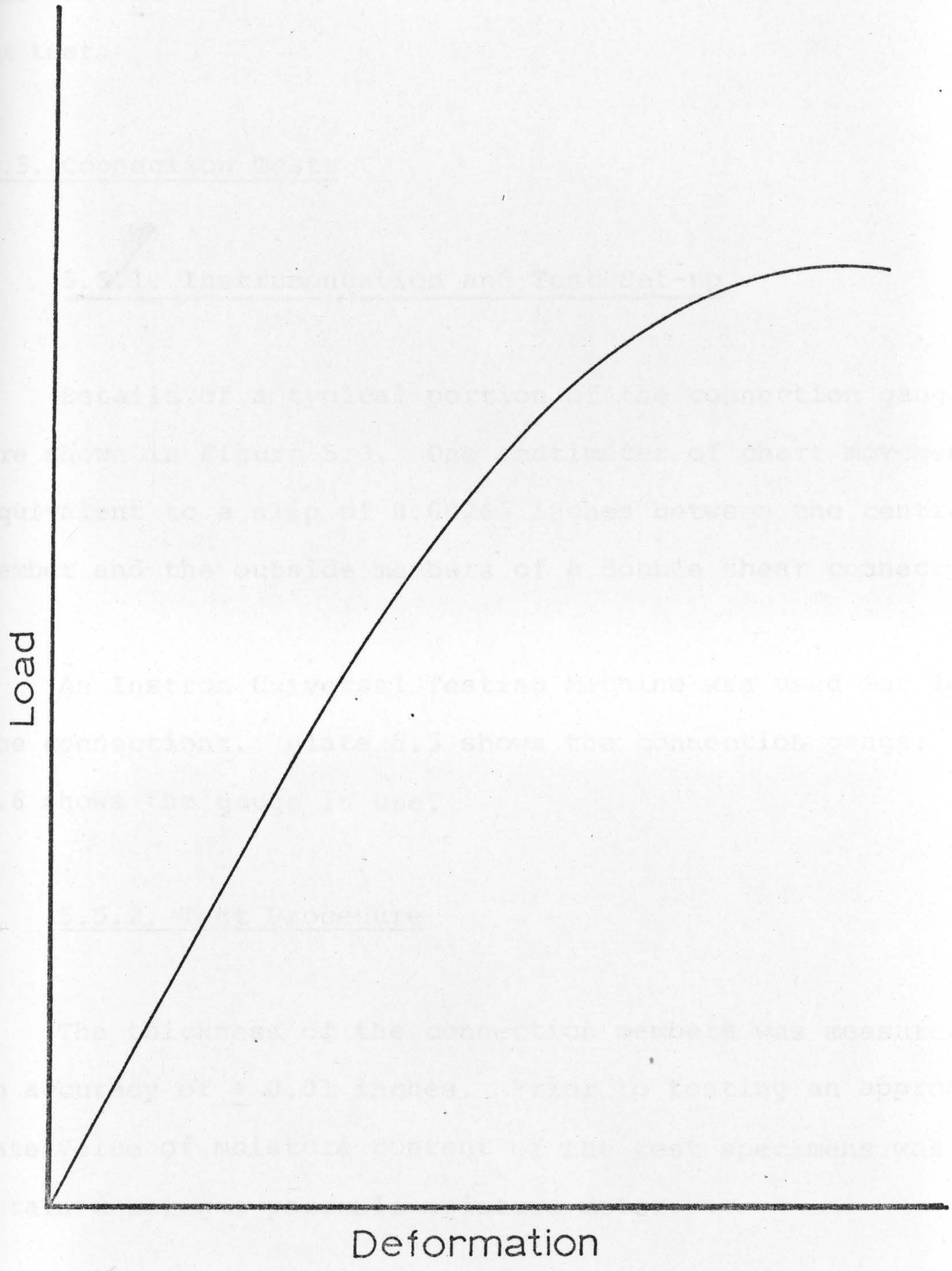


FIGURE 5.2 Typical Load-Deformation Curve-
Compression Test

was determined using the oven-dried weight and the volume at test.

5.5. Connection Tests

5.5.1. Instrumentation and Test Set-up

Details of a typical portion of the connection gauge are shown in Figure 5.3. One centimeter of chart movement was equivalent to a slip of 0.00263 inches between the central member and the outside members of a double shear connection.

An Instron Universal Testing Machine was used for loading the connections. Plate 5.5 shows the connection gauge. Plate 5.6 shows the gauge in use.

5.5.2. Test Procedure

The thickness of the connection members was measured to an accuracy of $\pm$ 0.01 inches. Prior to testing an approximate value of moisture content of the test specimens was obtained using a portable moisture meter.

All connections were loaded parallel to grain at a rate of 0.01 centimeters per minute.

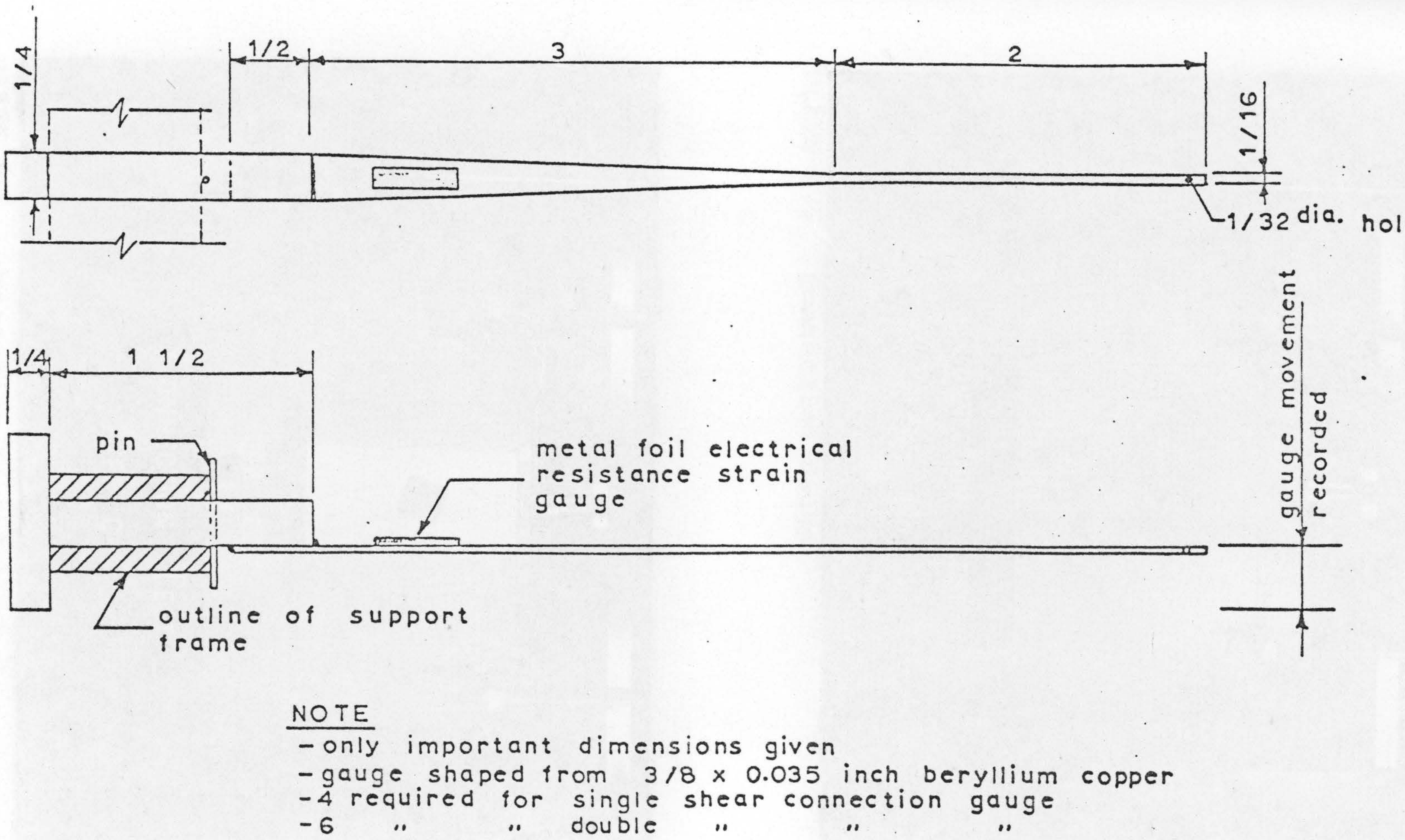


FIGURE 5.3 Typical Portion of Connection Gauge

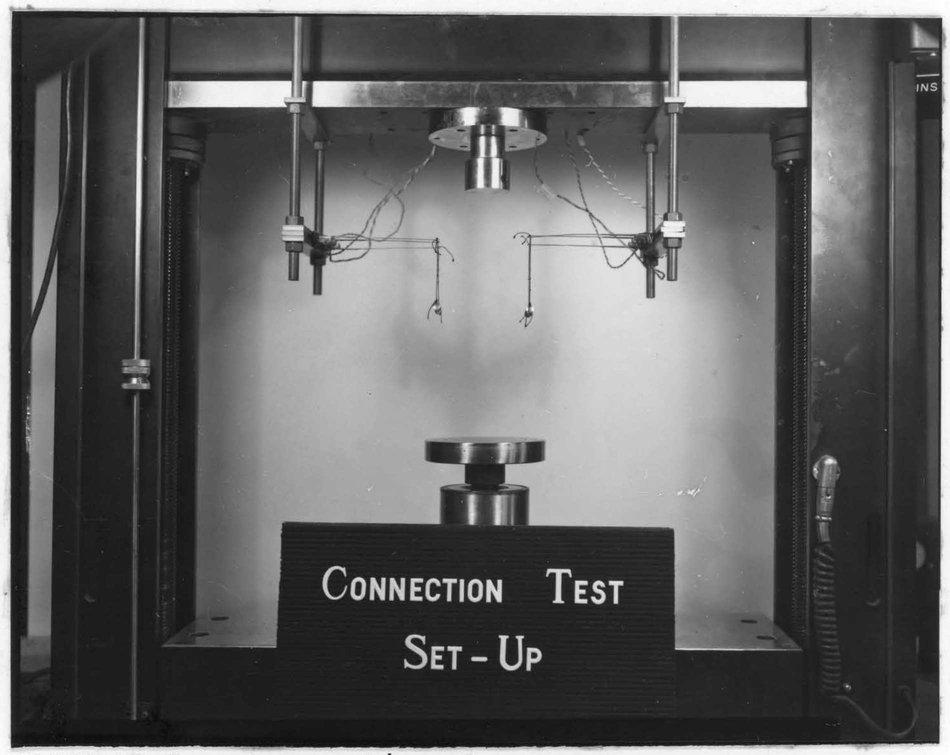


PLATE 5.5   Connection Gauge

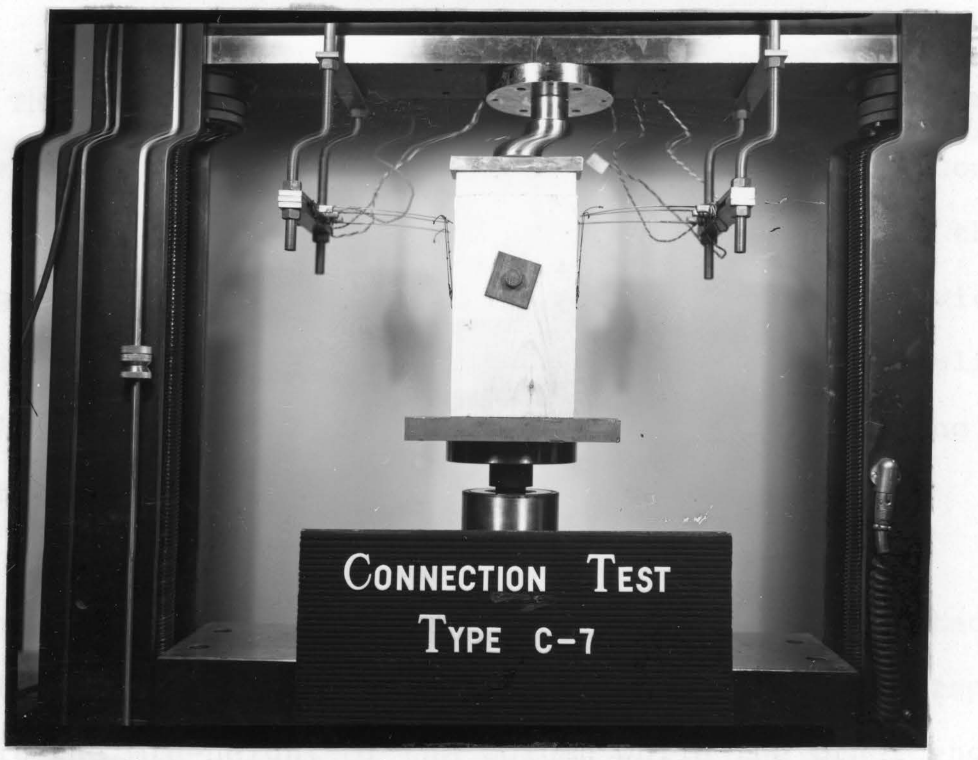


PLATE 5.6   Connection Test Set-up

Load-slip curves were obtained for each connection using the connection gauge in combination with the X-Y plotter of the Instron Testing Machine. A typical load-slip curve is shown in Figure 5.4.

Plate 5.6 shows a connection test in progress.

5.6. Column Tests

5.6.1. Instrumentation and Test Set-Up

Column load and lateral deflection were obtained using transducers in combination with an X-Y plotter which provided a continuous plot of load versus lateral deflection.

The apparatus used to measure column load consisted of a transducer in which one end of the sliding core was connected to the Y-axis of an X-Y plotter while the other end of the sliding core was fixed directly to a load measuring device on the column testing machine. The load transducer was calibrated by applying a known load. Details of the load measuring apparatus are shown in Plate 5.7.

The lateral deflection measuring apparatus consisted of a flexible metal wire one end of which was hooked to a staple fixed at the mid-height of the column while the other end

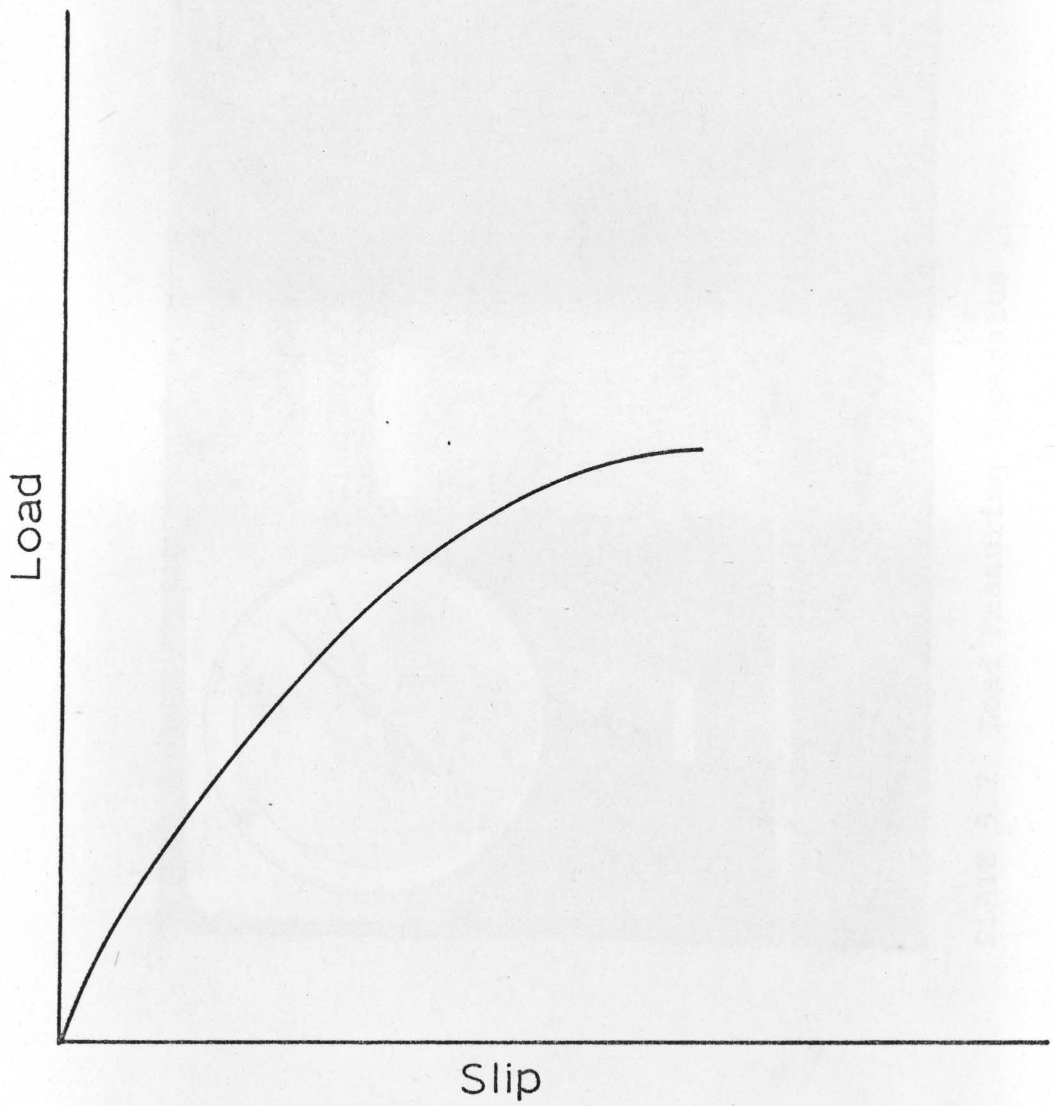


FIGURE 5.4 Typical Load-Slip Curve - Connection Test

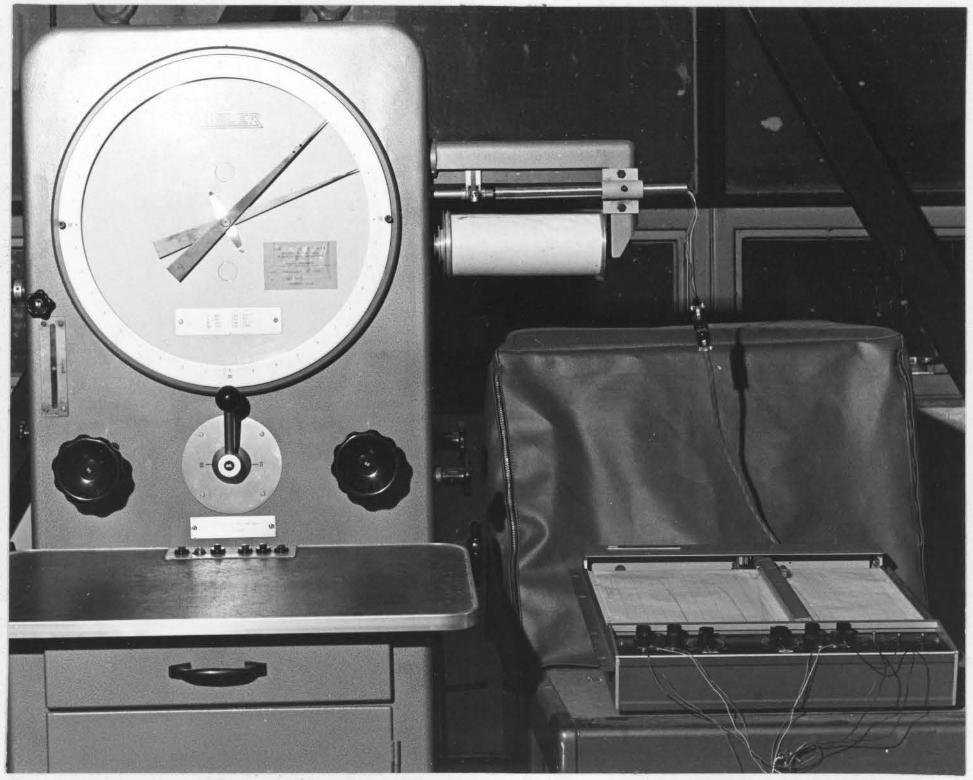


PLATE 5.7 Load Measuring Apparatus

running over a pulley was attached to the sliding core of a transducer. The transducer was connected to the X-axis of an X-Y plotter. The deflection transducer was calibrated by applying a known deflection with the aid of a Batty dial gauge ($\pm$ 0.0001 in.). Details of the apparatus used to measure lateral deflections are shown in Figure 5.5 .

An Amsler Column Testing Machine shown in Plate 5.8 was used for testing the columns. The testing machine has four load ranges with a maximum capacity of 400,000 pounds and a maximum cross-head speed of about 5 inches per minute. The machine can accommodate columns up to about 20 feet in length.

5.6.2. Test Procedure

The cross-section dimensions of each column were measured prior to testing. A staple was applied to each column at mid-height for attaching the lateral deflection apparatus to the column.

Each column was centered in the testing machine and load was applied at a rate of 0.003 inches per inch of column length per minute. The loading plates used provided pinned-end conditions.

The column was adjusted in the end fixtures when necessary

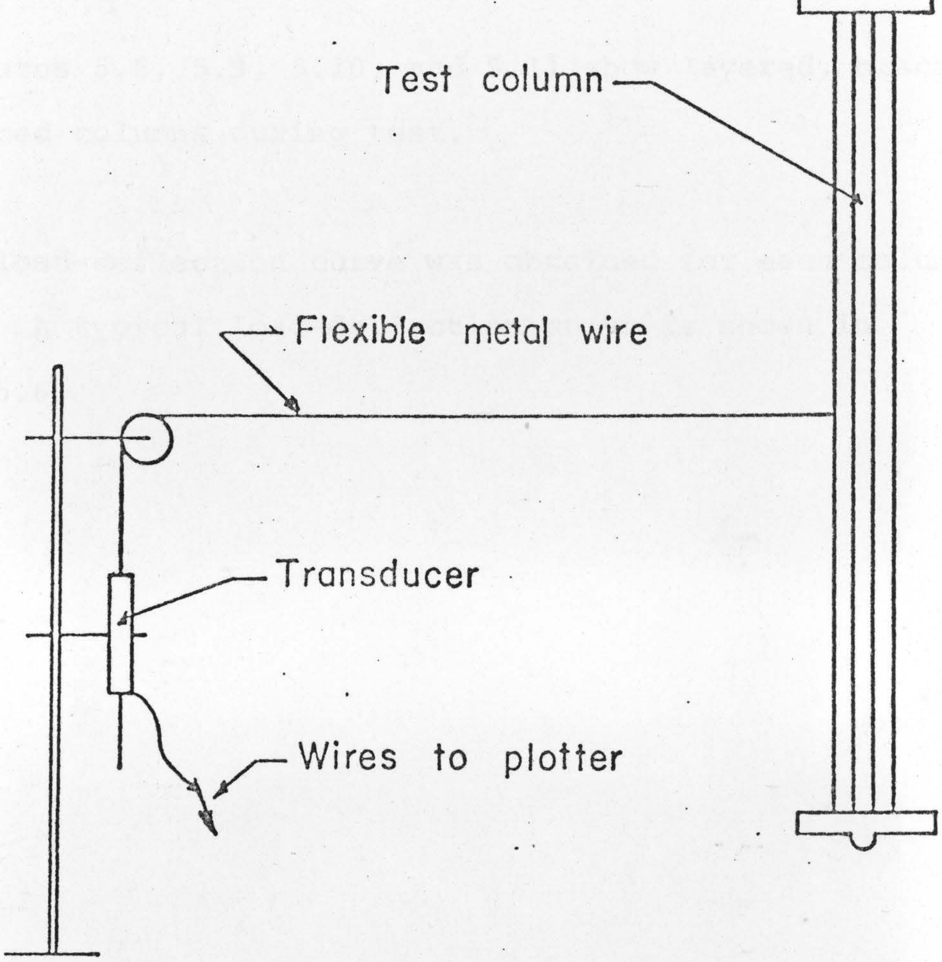


FIGURE 5.5 Apparatus for Measuring Lateral Deflection of Column

to ensure zero lateral deflection up to about 40 percent of the expected buckling load.

Plates 5.8, 5.9, 5.10, and 5.11 show layered, braced and spaced columns during test.

A load-deflection curve was obtained for each column tested. A typical load-deflection curve is shown in Figure 5.6.

PLATE 5.8   Layered Column Test

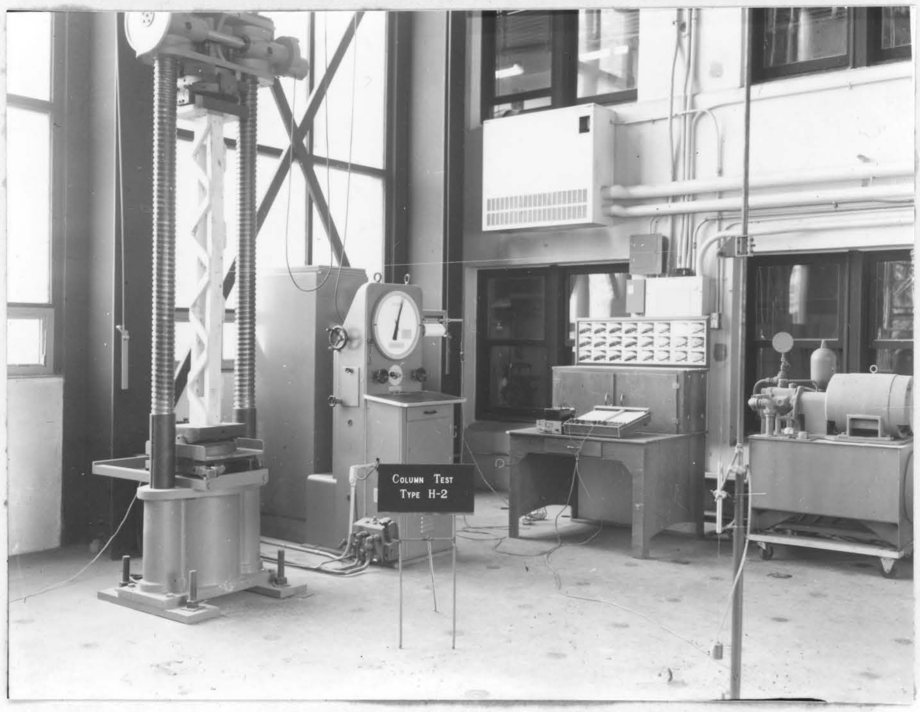


PLATE 5.9   Braced Column (45^{o} Braces) Test

PLATE 5.10 Braced Column (Horizontal Braces) Test

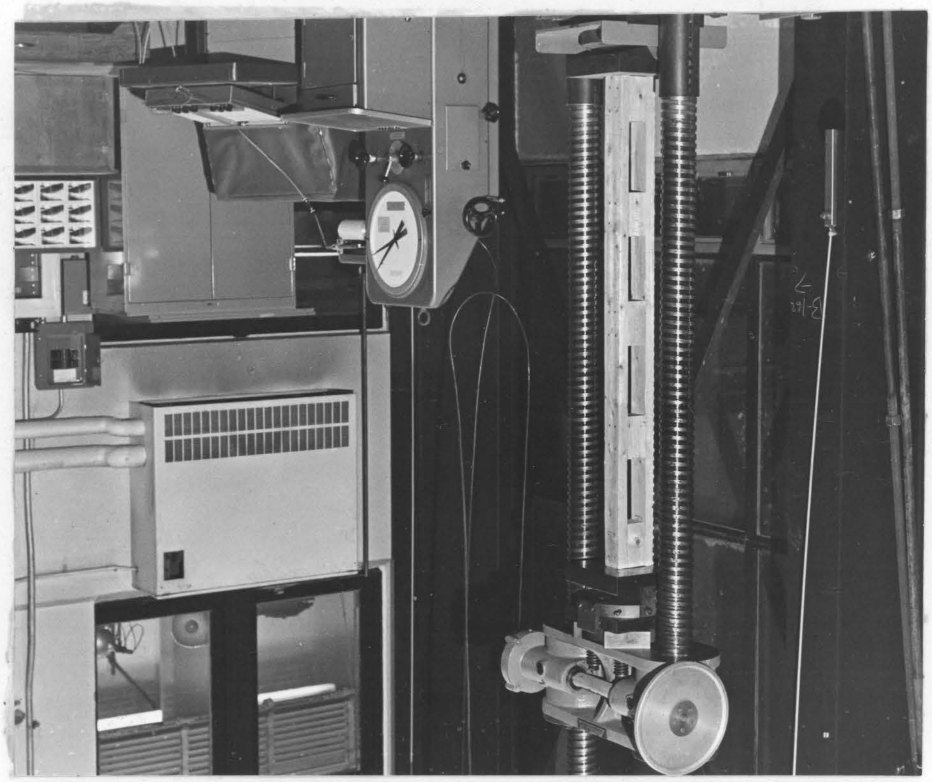


PLATE 5.11 Spaced Column Test

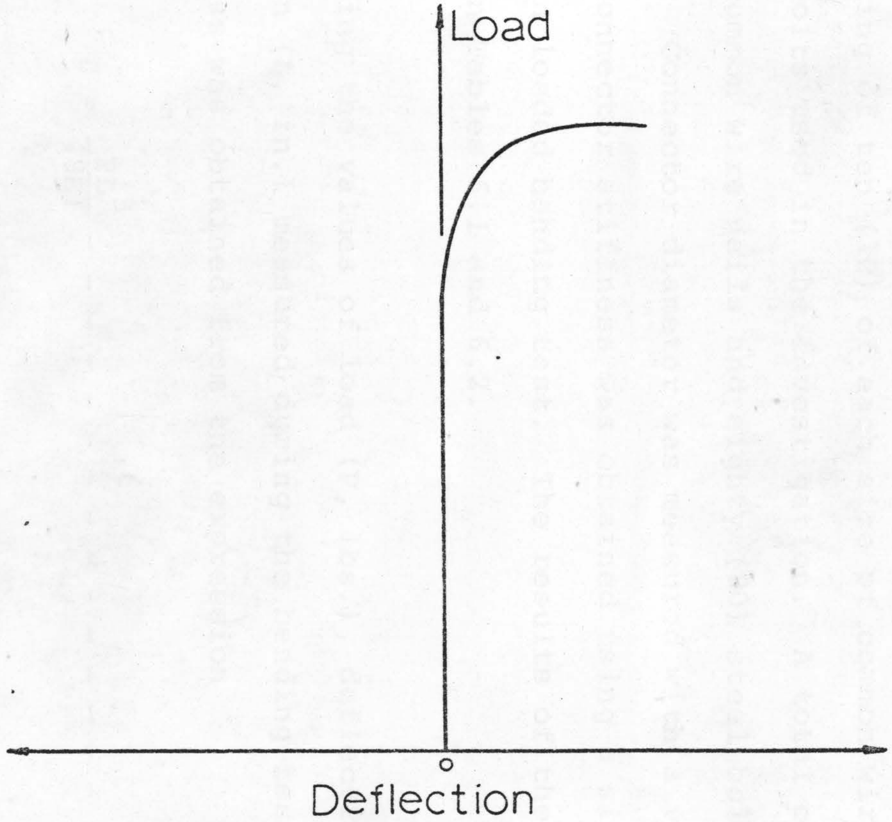


FIGURE 5.6 Typical Load-Deflection Curve-Column Test

CHAPTER 6

PRESENTATION OF MATERIAL PROPERTY TEST RESULTS

6.1. Nail and Bolt - Bending Tests

Physical and material properties (that is, diameter and stiffness) of the connectors were obtained from samples consisting of ten (10) of each size of common wire nails and steel bolts used in the investigation. A total of one hundred (100) common wire nails and eighty (80) steel bolts were tested. Connector diameter was measured with a vernier calliper. Connector stiffness was obtained using a simply supported, mid-span loaded bending test. The results of these tests are shown in Tables 6.1 and 6.2.

Using the values of load (P, lbs.), deflection (Δ, in.), and span (L, in.) measured during the bending tests, connector stiffness was obtained from the expression

$$\Delta = \frac{PL^3}{48EI} \qquad (6.1)$$

where

$$\text{stiffness, } EI = \frac{PL^3}{48\Delta} \qquad (6.2)$$

TABLE 6.1 DIAMETER AND STIFFNESS OF COMMON WIRE NAILS

NAIL LENGTH (in)	AVERAGE * DIAMETER (in)	AVERAGE * STIFFNESS ($lb-in^2$)
2	0.104	153.8
2 1/4	0.114	217.1
2 1/2	0.131	358.6
3	0.144	598.4
3 1/2	0.176	1220.6
4	0.193	1920.6
4 1/2	0.213	2708.0
5	0.231	3927.5
5 1/2	0.253	5069.2
6	0.278	8326.5

* Average of ten (10) tests.

TABLE 6.2 DIAMETER AND STIFFNESS OF STEEL BOLTS AND PHYSICAL PROPERTIES OF SPLIT RING CONNECTORS

NOMINAL BOLT SIZE	AVERAGE * DIAMETER (in)	AVERAGE * STIFFNESS ($lb\text{-}in^2$)
1/4"ø x 6"	0.247	4787.
3/8"ø x 5"	0.368	16667.
3/8"ø x 6"	0.371	18581.
3/8"ø x 7"	0.369	21763.
3/8"ø x 8"	0.356	18939.
1/2"ø x 6"	0.493	37021.
1/2"ø x 7"	0.491	62340.
1/2"ø x 8"	0.499	45744.

SPLIT RING CONNECTORS

NOMINAL SIZE	INSIDE * DIAMETER (in)	OUTSIDE * DIAMETER (in)	THICKNESS (in)	WIDTH* (in)
2 1/2"ø	2.523	2.843	0.32	0.64

* Average of ten (10) tests.

6.2. Split Ring Connectors

Only physical properties of the 2 1/2"ϕ split ring connectors were measured. These are shown in Table 6.2. To determine its stiffness a value of modulus of elasticity, $E = 25.0 \times 10^6$ pounds per square inch (p.s.i.), was estimated as the average value of E obtained for the nails and bolts. The moment of inertia, I, was calculated from the measured physical dimensions of the split rings. Stiffness was then obtained as the product of E and I (stiffness = EI).

6.3. Compression Tests

6.3.1. General

Compression tests, parallel to grain, were conducted on each connection and column member to evaluate modulus of elasticity values for connection members and modulus of elasticity and ultimate stress values for column members. The justification of the use of 3/4" x 3/4" x 3" and 1 1/2" x 1 1/2" x 6" compression specimens instead of the A.S.T.M. standard 2" x 2" x 8" compression specimen has been discussed in section 4.2.1.

To minimize any difference in moisture content between the compression specimens and the main test specimens (connections and columns) the compression and corresponding main test specimens were maintained simultaneously at the same moisture conditions (approximately 12 percent moisture content) for at least three weeks. By this procedure the strength properties obtained for the compression specimens can be used directly in the theoretical investigation of the main test specimens.

Figure 6.1 presents a typical stress-strain curve for a compression test parallel to grain. The location of the modulus of elasticity and ultimate stress is shown.

6.3.2. Compression Specimens - Connections

The results of the compression tests conducted on the connection compression specimens are given in Table A-1 (Appendix A). Eight hundred (800) compression specimens were tested. However, the values of specific gravity, moisture content and modulus of elasticity listed represent average values for each connection. That is, an average based on the number of members per connection.

The overall average values (based on 800 tests) of specific gravity, moisture content and modulus of elasticity are given on page 99.

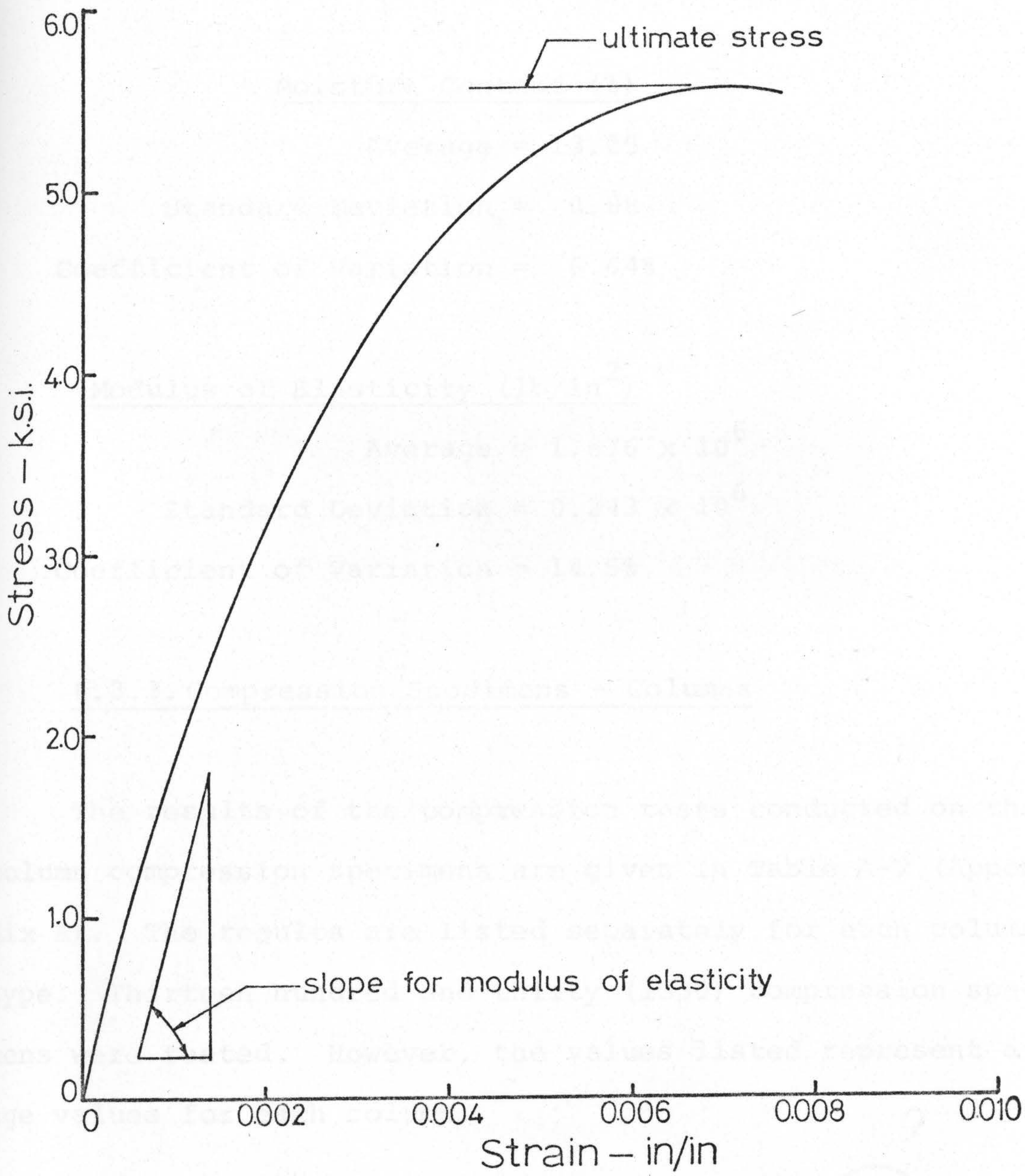


FIGURE 6.1 Typical Stress-Strain Curve

Specific Gravity

Average = 0.401

Standard Deviation = 0.035

Coefficient of Variation = 8.73%

Moisture Content (%)

Average = 13.25

Standard Deviation = 0.88

Coefficient of Variation = 6.64%

Modulus of Elasticity (lb/in^2)

Average = 1.676×10^6

Standard Deviation = 0.243×10^6

Coefficient of Variation = 14.5%

6.3.3. Compression Specimens - Columns

The results of the compression tests conducted on the column compression specimens are given in Table A-2 (Appendix A). The results are listed separately for each column type. Thirteen hundred and thirty (1330) compression specimens were tested. However, the values listed represent average values for each column.

?
34

Adopting the recommendations of Malhotra (31), the value of ultimate stress, F_u is reduced by the average knot ratio of the column laminates and the value of modulus of elasticity E, is reduced by 50 percent of the knot ratio. The knot ratio is defined as the ratio of the largest knot diameter or the sum of knot diameters at one section and the width of the column laminate.

A summary of these results, corrected for knot ratio and representing total averages per column type, is given in Table 6.3.

6.3.4. Summary

The physical and strength properties of the test materials (nails, bolts, split ring connectors and Construction Grade No. 1 Eastern Spruce lumber) have been presented. These results agree closely with those reported by Wilson (8) who used similar test materials in his investigation. These material properties are required for the theoretical calculations of slip modulus of the connections and column strength.

TABLE 6.3 SUMMARY OF STRENGTH PROPERTIES - COLUMN COMPRESSION SPECIMENS

COLUMN SERIES	NUMBER OF TESTS	MODULUS OF ELASTICITY,* E (psi)		ULTIMATE STRESS,* F_u (psi)	
		AVERAGE ($\times 10^6$)	STD. DEV. ($\times 10^6$)	AVERAGE	STD. DEV.
A1	90	1.577	0.295	4208.	600.
A2	90	1.513	0.309	4182.	672.
B1	125	1.560	0.342	4545.	720.
B2	125	1.616	0.331	4668.	713.
B3	125	1.669	0.349	5178.	779.
C1	75	1.426	0.304	4566.	713.
C2	75	1.517	0.327	4660.	703.
C3	75	1.506	0.297	4612.	587.
D	175	1.615	0.294	5043.	687.
E1	45	1.787	0.363	5274.	802.
E2	45	1.594	0.315	5117.	857.
F	60	1.483	0.371	4321.	796.
G	45	1.407	0.280	4202.	556.
H1	30	1.518	0.315	4826.	895.
H2	20	1.545	0.371	4599.	728.
H3	30	1.563	0.315	5055.	1047.
H4	20	1.693	0.473	4635.	553.
H5	20	1.386	0.342	4292.	438.
I1	30	1.559	0.339	4961.	787.
I2	30	1.645	0.351	5088.	727.

* Reduced for Knot ratio

CHAPTER 7

PRESENTATION AND DISCUSSION OF CONNECTION TEST RESULTS

7.1. Connection Types

A summary of the connections tested is presented in Table 7.1. The types and quantity of connectors and the number of connections are indicated. Seven different cross-sections were investigated as shown in Figures 7.1. to 7.7. Each cross-section is identified by one of the letters A, B, C, D, ...,G. The connector types and quantities are indicated by the numerals 1, 2, 3, etc., as shown in Table 7.1. In each of Figures 7.1. to 7.7., connector details are shown for only one connector type.

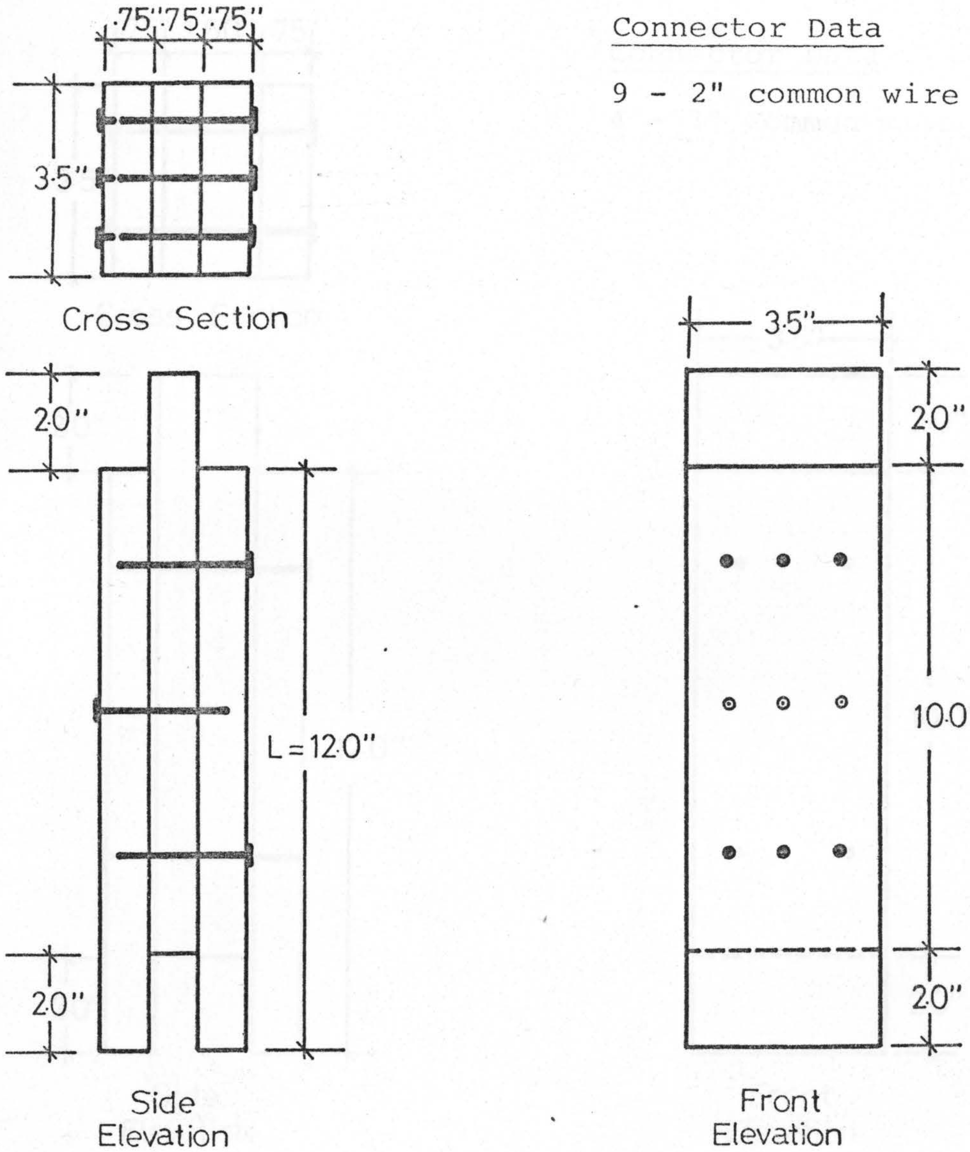
7.2. Results of Connection Tests

The theoretical and experimental results of the connection tests are summarized in Table 7.2. The values shown represent the average results of either five or ten similar tests. Theoretical results are given for double shear connections (as conducted in the experimental program) and for corresponding single shear connections. Percent difference between the theoretical double shear results and the experimental results are also shown in Table 7.2.

TABLE 7.1 SUMMARY OF CONNECTIONS TESTED

Connection		Connector Data	
Type	Quantity	Number	Type
A-1	10	4	2" Common Wire Nail
A-2	5	9	2" " " "
A-3	5	15	2" " " "
A-4	10	4	2 1/4" " " "
A-5	10	6	2 1/4" " " "
A-6	5	9	2 1/4" " " "
A-7	5	12	2 1/4" " " "
A-8	5	15	2 1/4" " " "
A-9*	5	18	2 1/4" " " "
A-10*	5	18	2 1/4" " " "
A-11*	5	18	2 1/4" " " "
A-12	5	4	2 1/2" " " "
A-13	5	4	3" " " "
A-14	5	4	3 1/2" " " "
A-15	5	4	4" " " "
A-16	5	4	4 1/2" " " "
A-17	5	4	5" " " "
A-18	5	4	5 1/2" " " "
B-1	10	8	2" Common Wire Nail
B-2	5	4	3" " " "
C-1	10	6	3" Common Wire Nail
C-2	10	4	4 1/2" " " "
C-3	5	4	5" " " "
C-4	5	4	5 1/2" " " "
C-5	5	4	6" " " "
C-6	5	2	1/4"øx6" Hexagonal Head Steel Bolt
C-7	5	1	3/8"øx6" " " " "
C-8	5	1	1/2"øx6" " " " "
C-9	5	2	1/2"øx6" " " " "
C-10	10	1 2	1/2"øx6" Hex. Head Steel Bolt and 2 1/2"ø Split Ring Connectors
D-1	5	4	5 1/2" Common Wire Nail
D-2	5	1	3/8"øx7" Hexagonal Head Steel Bolt
D-3	5	2	3/8"øx7" " " " "
D-4	5	1	1/2"øx7" " " " "
E-1	5	1	3/8"øx8" Hexagonal Head Steel Bolt
E-2	5	2	3/8"øx8" " " " "
E-3	5	1	1/2"øx8" " " " "
F-1	10	4	3 1/2" Common Wire Nail
F-2	10	2	3/8"øx5" Hexagonal Head Steel Bolt
G	10	4	5" Common Wire Nail

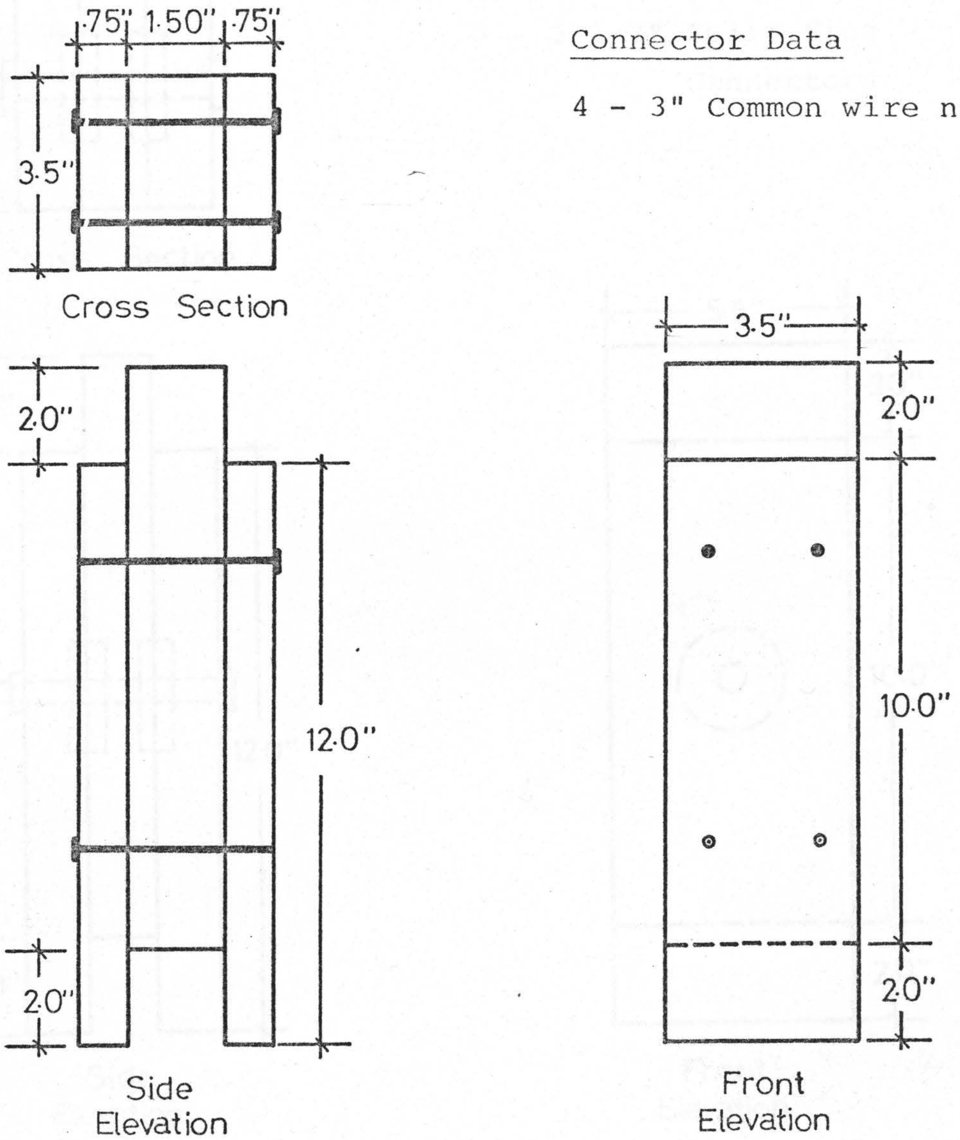
*Length of connection members, L=12.0" for all connection types except A-9 (L=18"), A-10 (L=24"), A-11 (L=36").



Connector Data

9 - 2" common wire nails

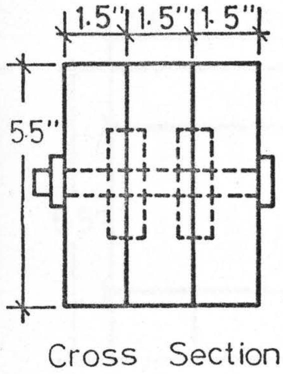
FIGURE 7.1 Dimensions and Connector Details for Type A-2 Connections



Connector Data

4 - 3" Common wire nails

FIGURE 7.2 Dimensions and Connector Details for Type B-2 Connections



Connector Data

1 - 1/2"ø 6" Steel Bolt

2 - 2 1/2" Split Ring Connectors

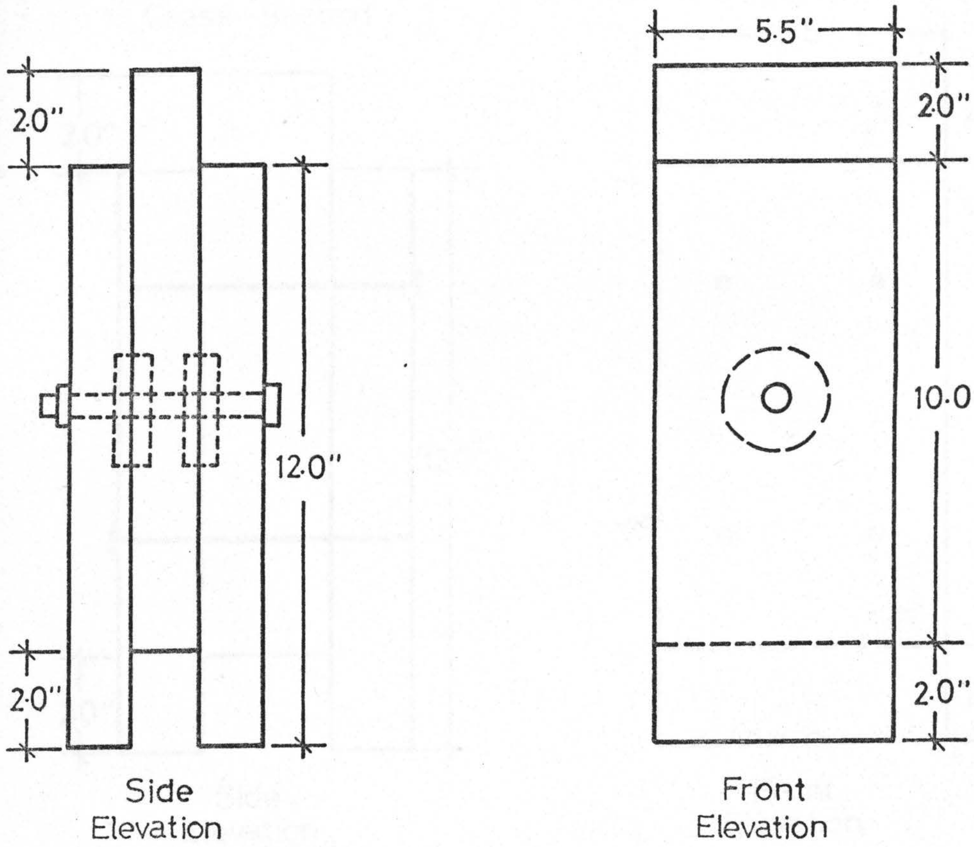
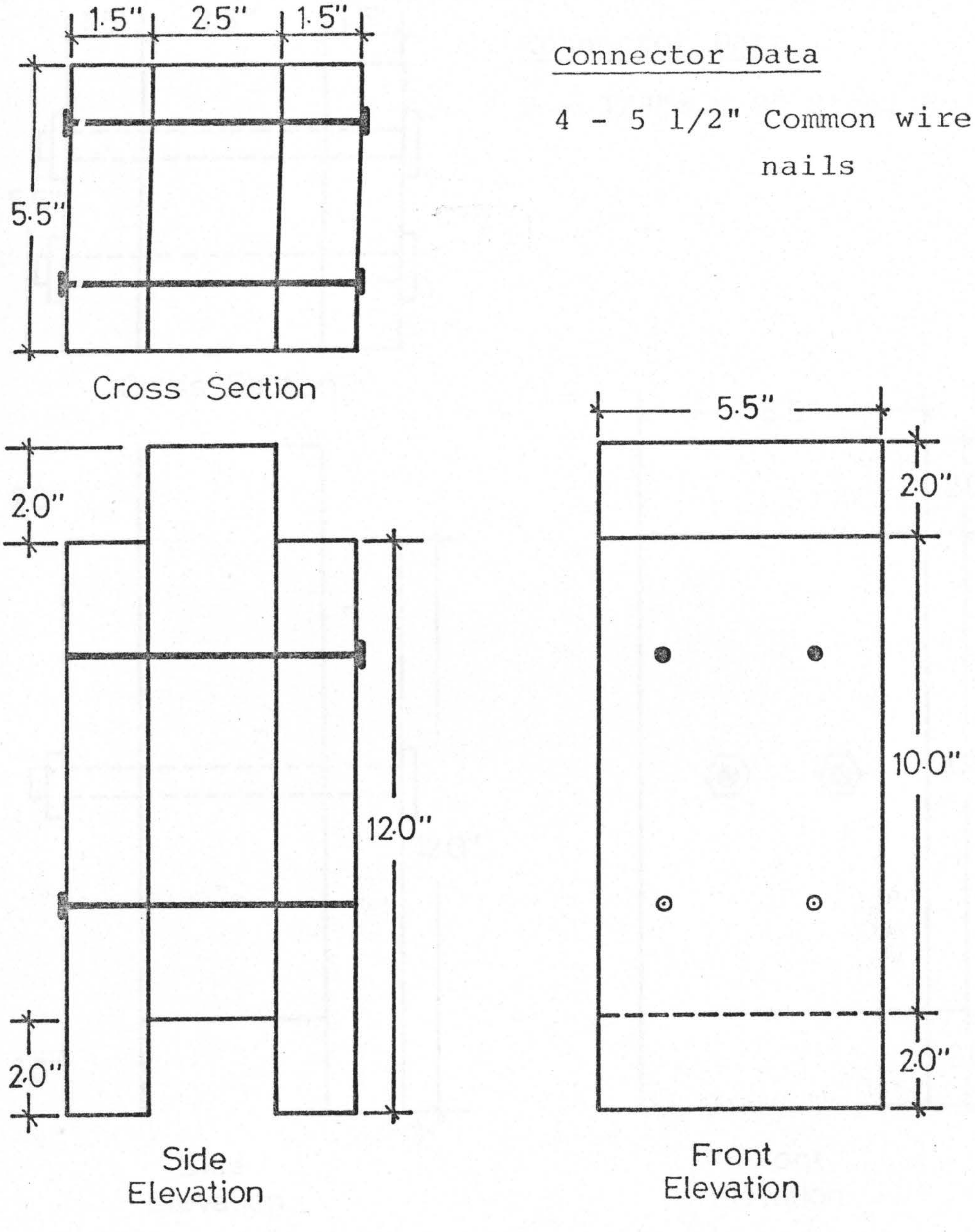


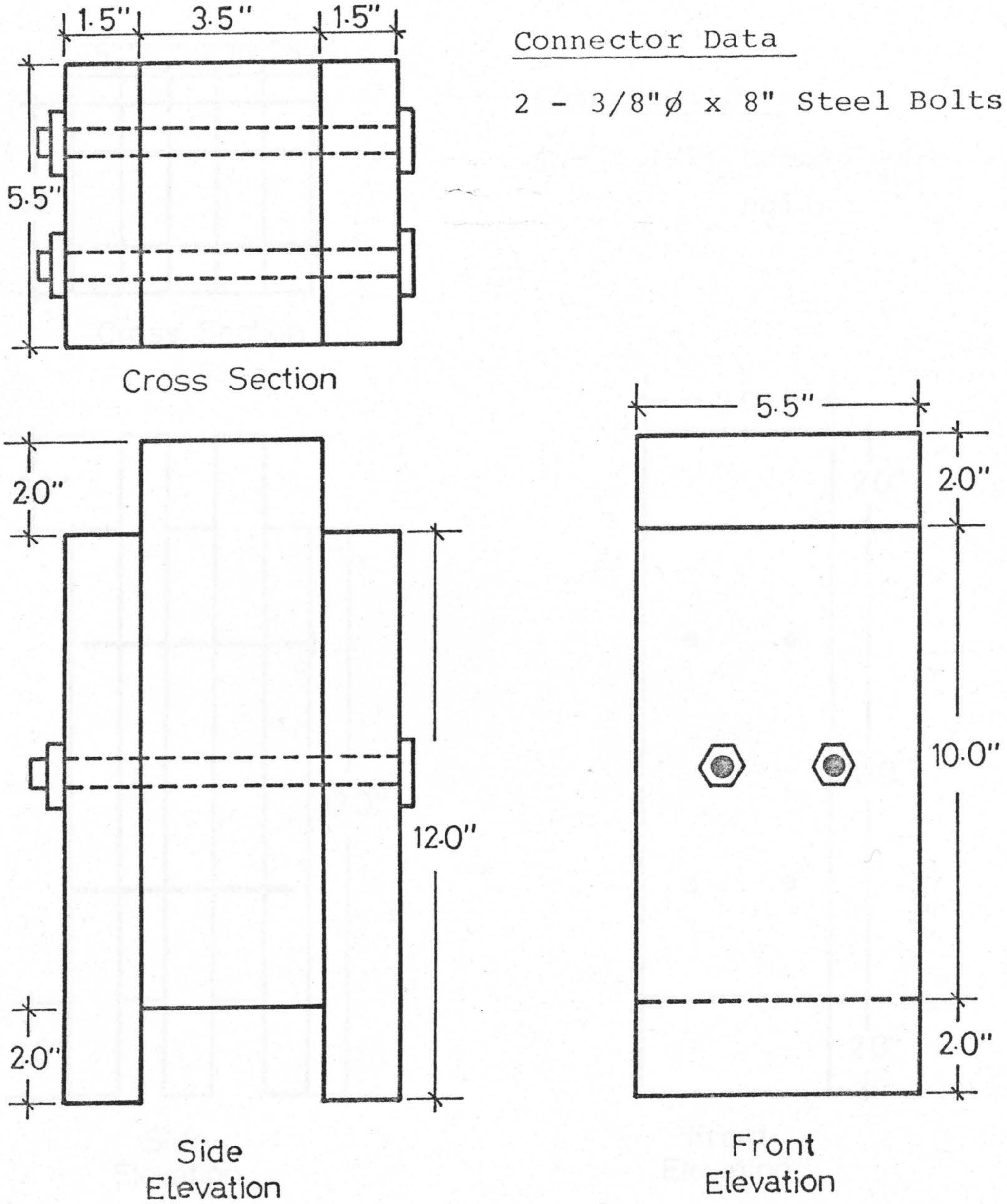
FIGURE 7.3 Dimensions and Connector Details for Type C-10 Connections



Connector Data

4 - 5 1/2" Common wire nails

FIGURE 7.4 Dimensions and Connector Details for Type D-1 Connections



Connector Data

2 - 3/8"ø x 8" Steel Bolts

FIGURE 7.5 Dimensions and Connector Details for Type E-2 Connections

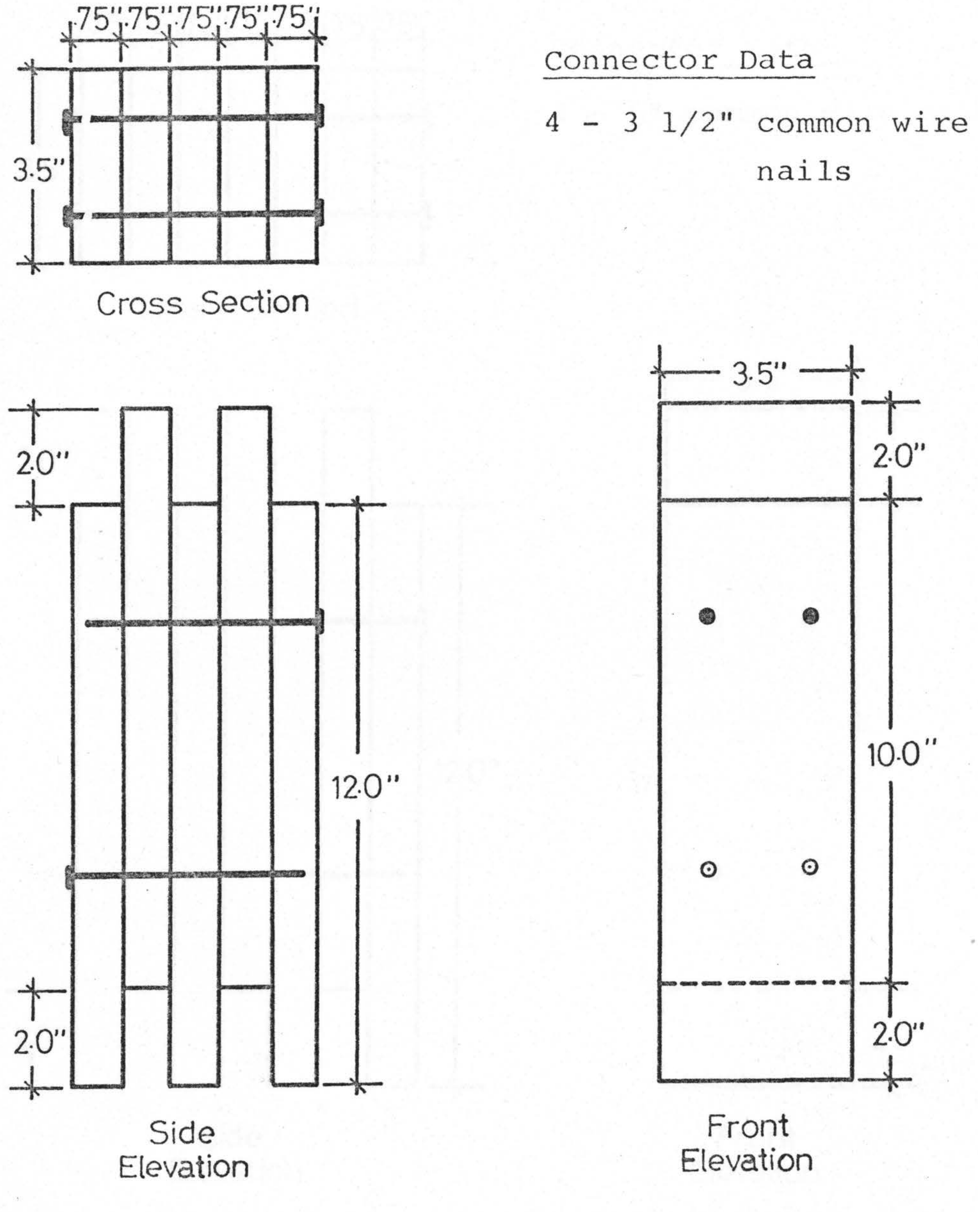


FIGURE 7.6 Dimensions and Connector Details for Type F-1 Connections

Connector Data

4 - 5" common wire nails

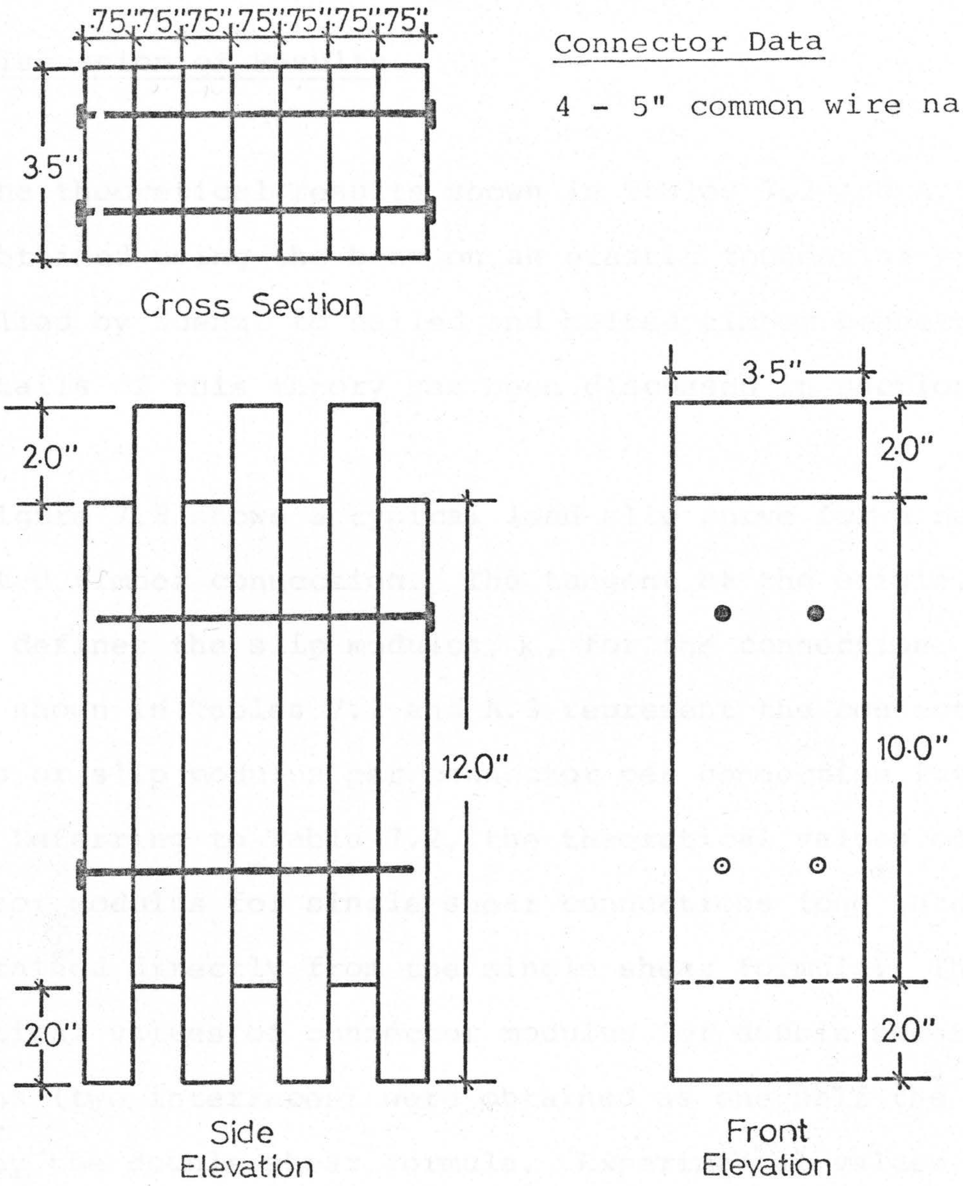


FIGURE 7.7 Dimensions and Connector Details for Type G Connections

A complete list of the connection results is given in Table A.3 (Appendix A)

7.3. Discussion of Results

The theoretical results shown in Tables 7.2 and A.3 were obtained using the beam on an elastic foundation theory as applied by Kuenzi to nailed and bolted timber connections. The details of this theory has been discussed in section 3.2.

Figure 7.8 shows a typical load-slip curve for a nailed or bolted timber connection. The tangent at the origin, as shown, defines the slip modulus, k, for the connection. The values shown in Tables 7.2 and A.3 represent the connector modulus or slip modulus per connector per connection interface. Referring to Table 7.2, the theoretical values of connector modulus for single shear connections (one interface) are obtained directly from the single shear formula. The theoretical values of connector modulus for double shear connections (two interfaces) were obtained as one half the value given by the double shear formula. Experimental values of connector modulus were obtained by dividing the slip modulus of each connection by the product of the number of connectors and the number of interfaces in each connection.

It has been pointed out in section 3.2 that the theoreti-

TABLE 7.2 SUMMARY OF RESULTS OF CONNECTION TESTS

Connection Type	Connector Modulus per Connector, k (lb/in.)					Per Cent Diff.**
	Single Shear			Double Shear		
	Kuenzi	Wilkinson	k=tdE/x	Kuenzi	Experimental	
A-1	9801*	9965	9502	10408	10548	-1.33
A-2	11080	11194	11079	11620	12073	-3.75
A-3	11534	11609	11604	12030	12448	-3.36
A-4	13509	13513	13595	14236	14554	-2.18
A-5	13434	13564	13482	14246	14755	-3.33
A-6	12620	12767	12468	13493	13979	-3.48
A-7	11861	11943	11434	12728	13343	-4.61
A-8	14135	14251	14452	14940	14808	0.89
A-9	14066	13950	14041	14725	14704	0.14
A-10	14813	14746	15070	15476	15278	1.30
A-11	11885	12221	11729	12884	11907	8.21
A-12	16448	16732	16724	18031	17264	4.44
A-13	18485	19856	18698	21514	21204	1.46
A-14	23287	26151	23392	28844	29505	-2.24
A-15	30459	34741	30776	38442	37737	1.87
A-16	29303	36262	29404	39871	40809	-2.30
A-17	36167	45379	35496	49921	47562	4.96
A-18	41350	53792	41838	58206	56814	2.45
B-1	10043	10401	10009	10734	10105	6.22
B-2	19602	20079	19124	22405	22735	-1.45
C-1	21194	21225	21054	21449	21278	-1.05
C-2	37015	37052	36552	37496	37118	1.02
C-3	43531	43307	41958	44601	42430	5.12
C-4	49162	49143	48388	50992	52163	-2.24
C-5	57157	59590	56926	62497	61782	1.16
C-6	47256	47941	46902	49253	50482	-2.43
C-7	87461	87039	87697	94538	96323	-1.85
C-8	120908	128336	125112	141326	134936	4.74
C-9	132866	149564	135841	166215	158243	5.09
C-10	153175		159855		152636	
D-1	52073	52937	53454	54058	53040	1.92
D-2	91893	94021	91943	105030	105831	-0.76
D-3	85262	88845	85308	98194	95814	2.48
D-4	146698	159325	147284	183736	177628	3.44
E-1	92742	92955	91997	103360	97323	6.20
E-2	96412	93834	93448	103350	102110	1.21
E-3	129408	132867	130309	154051	144759	6.42
F-1	24790	27366	26043	30335	28706	5.68
F-2	57922	95746	56461	102822	97318	5.66
G	30973	42214	31677	42779	40695	5.12

* Average of 5 or 10 values

** (Exp. — Kuenzi, double shear) x 100/Exp.

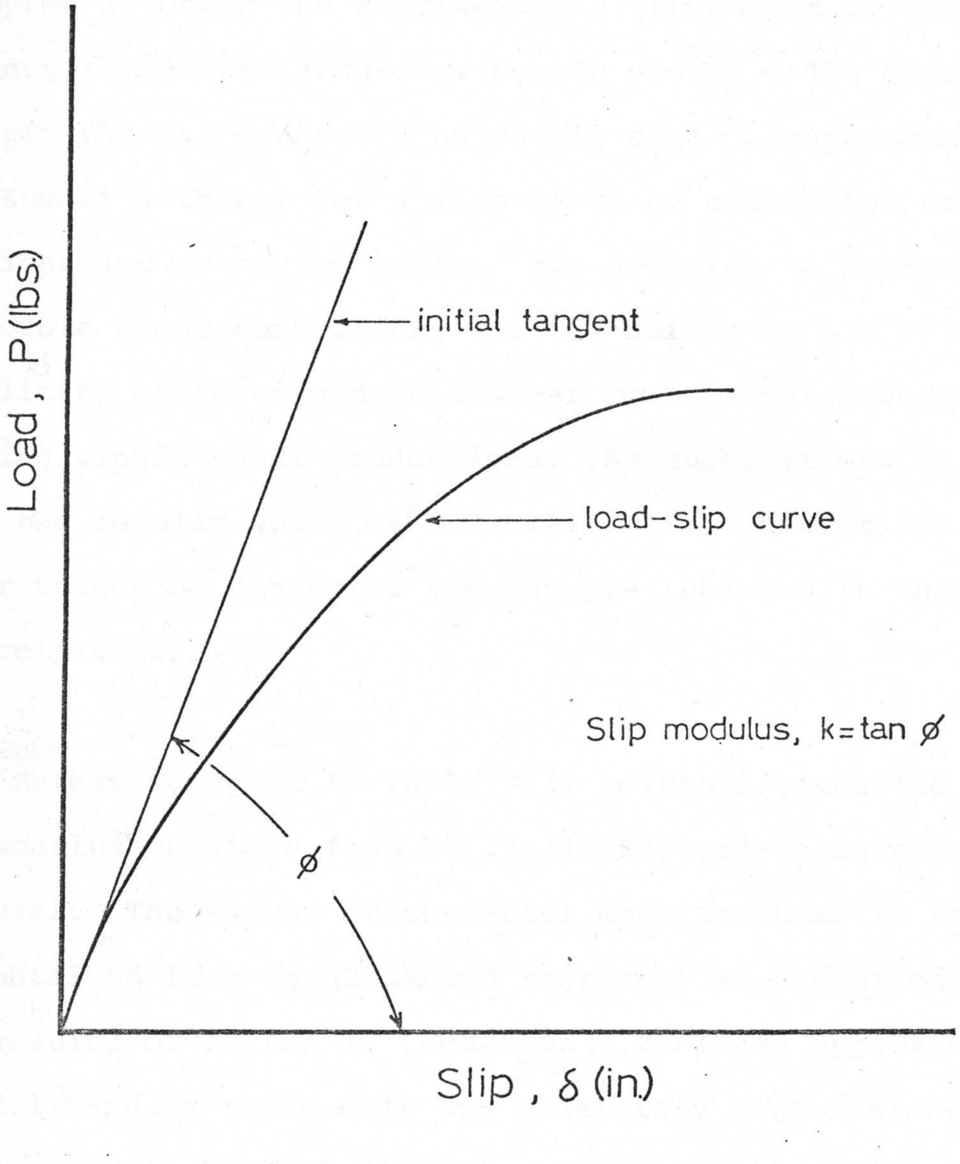


FIGURE 7.8 Typical Load-Slip Curve

cal single shear value of connector modulus would be adopted as the basis of determining connector modulus in built-up timber columns. Therefore, a rational method is required for determining single shear values of connector modulus. Based on the literature review, Kuenzi's theory was tentatively accepted as being the most suitable method for determining connector modulus (single or double shear). The experimental program was conducted on double shear connections to verify Kuenzi's theory for a wide range of connection cross-sections and connector types. The decision to perform tests on double shear connections was due mainly to the relative simplicity of testing double shear connection as compared to testing single shear connections. As such, it was thought that the results would be more reliable. Theoretical single shear values of connector modulus are included in the results for comparison.

Referring again to Table 7.2, column 1 gives the connector modulus obtained from Eq. (2.11) (Kuenzi's single shear formula). The values of connector modulus shown in column 2 are obtained from Eq. (2.26) and represent an approximation of the results of column 1. Equation (2.26) is an approximation of Eq. (2.11) and is valid only for relatively long fasteners, as can be seen in Table 7.2. The results of column 3 were obtained using the formula

$$k = \frac{tdE}{x} \quad \text{- -} \quad (7.1)$$

where

t = least thickness of connection members, in

d = connector diameter, in

E = average modulus of elasticity of connection members, lb/in^2

$1/x$ = factor depending on ratio t/d

Equation (7.1) expresses Kuenzi's single shear formula in terms of material properties of the connection - t, d and E. Values of 1/x are obtained from Figure 7.9 (for nominal 1 inch thick wood) and Figure 7.10 (for nominal 2 inch thick wood). Equation (7.1), together with Figures 7.9 and 7.10, provides a simple and accurate method of determining connector modulus. Column 4, lists Kuenzi's double shear values of slip modulus, using Eq. (2.18). Column 5 lists the experimental results and column 6 shows the percent difference between columns 4 and 5 based on the results of column 5.

Comparison of the results shown in columns 4 and 5 indicates very good agreement between the experimental and predicted results. Based on the average results shown in Table 7.1, the difference between the experimental and predicted values of connector modulus has a range varying from - 4.61 percent to 8.21 percent.

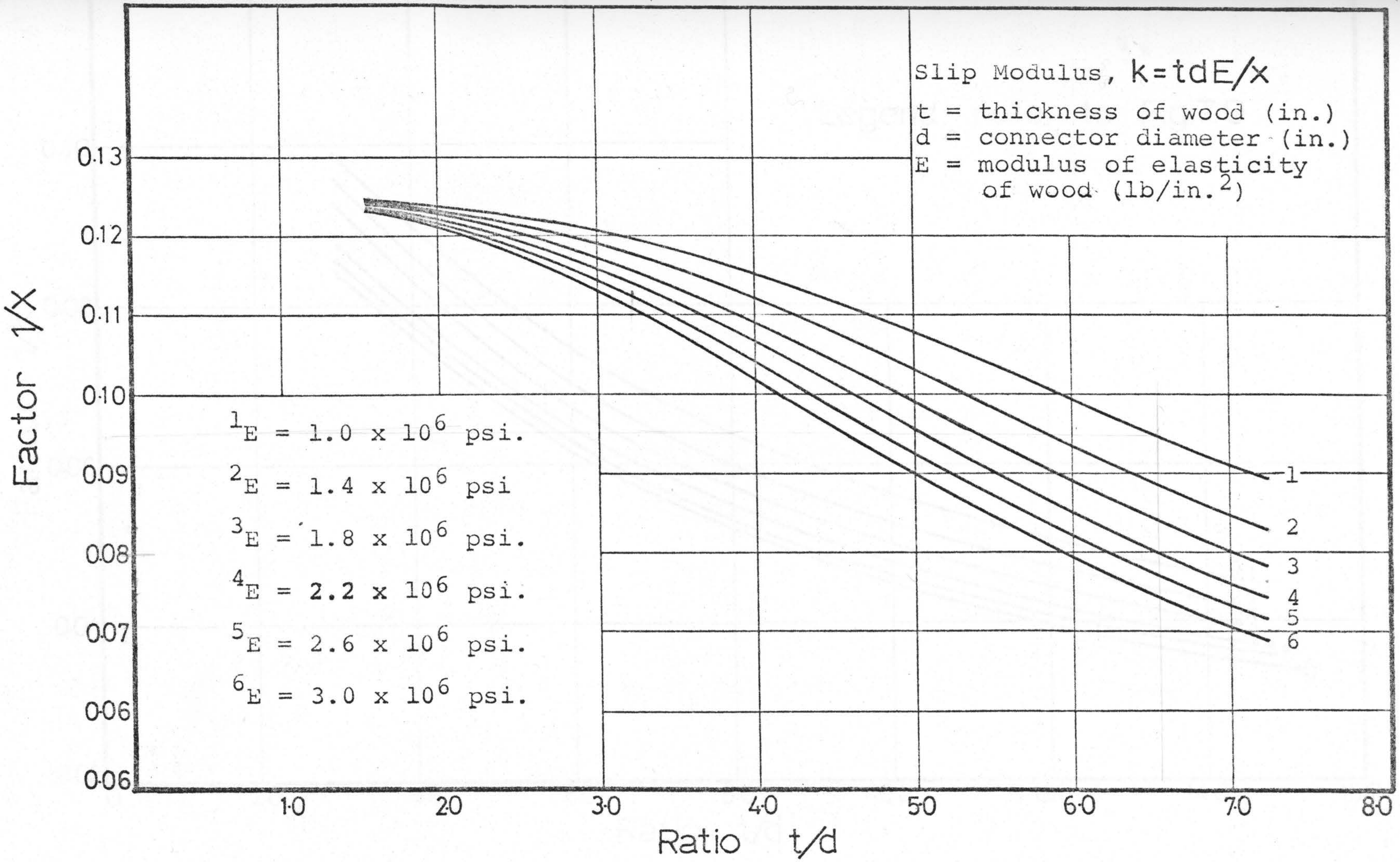


FIGURE 7.9 Variation of Factor 1/x with Ratio t/d in Slip Modulus Formula (k = tdE/x) for nominal 1-inch Thick Wood

.75
.113

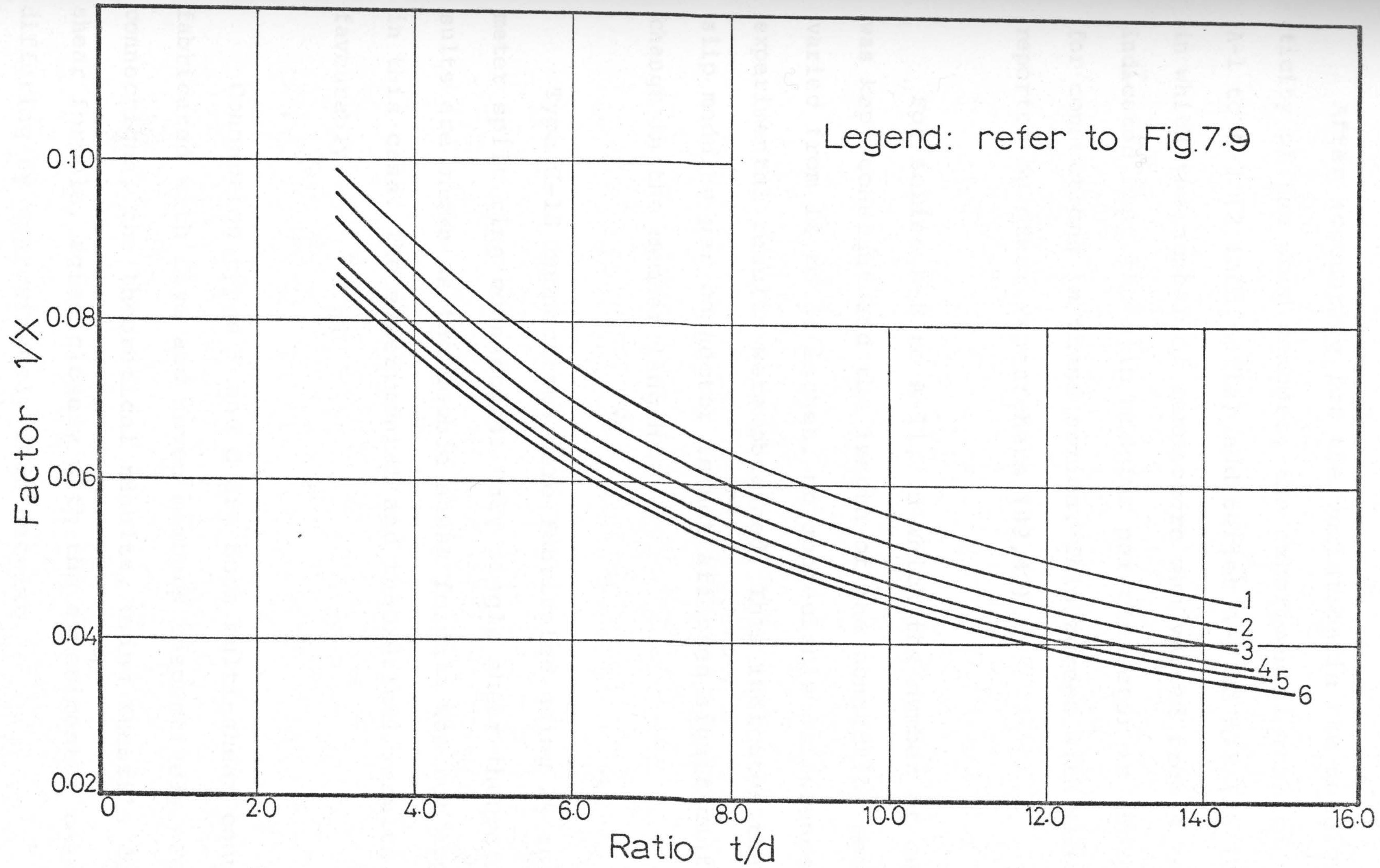


FIGURE 7.10 Variation of Factor 1/x with Ratio t/d in Slip Modulus Formula (k = tdE/x) for Nominal 2-inch Thick Wood

After accounting for the variation in the modulus of elasticity of the wood members, the experimental results of series A-1 to A-3 (2 inch nails) and series A-4 to A-8 (2¼ inch nails), in which the number of connectors was varied from 4 to 15, indicated that the slip modulus per connector is about the same for connections in these series. This agrees with the results reported by other researchers (42,43).

For series A-8 to A-11, in which the number of connectors was kept constant and the length of the connection members was varied from 12 to 36 inches, no appreciable differences in the experimental results were obtained. This indicates that the slip modulus per connector is not affected significantly by the change in the member length.

Type C-10 connections were fabricated using 2½ inch diameter split ring connectors. Only single shear theoretical results are shown as the double shear formula was not applicable in this case. The experimental and theoretical results compare favourably.

Connection types F and G are both multi-shear connections, fabricated with five and seven members respectively. For these connections, the theoretical results, using Kuenzi's double shear formula, agree closely with the experimental results, differing by approximately four percent.

7.4. Summary

The connection results have been presented. Very good agreement was observed between the experimental and theoretical results, the difference between the two having a range varying from - 4.61 percent to 8.21 percent, based on the average results.

These results indicate that the beam on an elastic foundation theory, as applied by Kuenzi, is satisfactory for predicting the connector modulus in mechanically fastened timber connections.

Equation (7.1), ($k = tdE/x$), together with Figs. 7.9 and 7.10 provides a simple and accurate method for determining the connector modulus in mechanically fastened timber joints fabricated with nominal 1 inch or 2 inch thick lumber.

CHAPTER 8

PRESENTATION AND DISCUSSION OF COLUMN TEST RESULTS

8.1. Presentation of Results

The column results for each of the twenty (20) column types (layered, spaced and braced columns) are presented in the following manner. For each column type, three figures and one table are presented. To illustrate the order in which the figures and tables are presented, column type A1 will be discussed in detail as an example.

Figure 8.A1.1* provides longitudinal and cross-section sketches to indicate the nailing pattern used. Additional information provided includes connector type, quantity and spacing, column lengths and number of columns fabricated.

Table 8.A1.1 presents a complete list of the theoretical and experimental column stresses for each slenderness ratio investigated. Average column stresses at each slenderness ratio are also indicated. The significance of the various theoretical column stresses will be discussed in the separate sections dealing with layered, spaced and braced columns.

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*All figures, tables and graphs are presented at the end of this chapter.

Figure 8.A1.2 is a graphical presentation of the average results listed in Table 8.A1.1. This graph shows a plot of column stress, F_{cr} (Y-axis), versus slenderness ratio, λ (X-axis).

Figure 8.A1.3 presents the results of a statistical analysis for Column Type A1.

8.2. Layered Columns

8.2.1. Buckling Stress Formulas

The proposed buckling formulas for layered columns have been discussed in Section 3.3. The various formulas used to provide theoretical prediction curves for the layered columns are summarized below. The notations used are the same as in Chapters 2 and 3.

Inelastic Buckling Formula- Exact Solution

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot B \qquad (8.1)$$

$$\text{where}\quad B = \frac{1 + \alpha \dfrac{\pi^2 E_t A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E_t A_r a}{2mkL^2}} ,$$

$$\text{and}\qquad E_t = E\left(\frac{F_u - F_{cr}}{F_u - cF_{cr}}\right)$$

Solution of Eq. (8.1) for F_{cr} is given in Eq. (3.20).

<u>Inelastic Buckling Formula - Approximate Solution</u>

[E used instead of E_t in Factor B of Eq. (8.1)]

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \; \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2mkL^2}} \qquad (8.2)$$

Solution of Eq. (8.2) for F_{cr} is given in Eq. (3.23).

Elastic Buckling Formula

[E used instead of E_t in Eq. (8.1)]

$$F_{cr} = \frac{\pi^2 E}{\lambda^2} \cdot \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2mkL^2}} \quad \text{-------------} \quad (8.3)$$

Tangent Modulus Formula

(Used to calculate equivalent solid column strength)

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \qquad [B = 1 \text{ in Eq. (8.1)}] \text{ ---------} \quad (8.4)$$

Solution of Eq. (8.4) for F_{cr} is given in Eq. (3.23), with B = 1.

8.2.2. Discussion of Layered Column Results

The results of the tests on layered columns (column types A1, A2, B1, B2, B3, C1, C2, C3, D, E1, E2, F and G) are presented in Figures 8.A1.1 to 8.G.3 in the manner described in section 8.1.

The theoretical column stresses listed in columns 1 to 4 of Tables 8.A1.1, 8.A2.1, 8.B1.1, etc., were obtained from Eqns. (8.1) to (8.4) respectively, using the average strength properties, E and F_u, for each column. Slip modulus, k, was calculated from Eq. (2.11) (Kuenzi's single shear formula).

The column stress versus slenderness ratio curves, numbered 1, 2, 3 and 4, shown in Figures 8.A1.2, 8.A2.2, 8.B1.2, etc., were obtained by plotting the average theoretical column stresses listed in Tables 8.A1.1, 8.A2.1, 8.B1.1, etc., respectively. The average experimental results are superimposed on the theoretical column curves.

Curves 1 and 2 represent the exact and approximate inelastic column formulas, Eqns. (3.20) and (3.23), obtained after substituting the value of E_t from Eq. (3.7) into Eqns. (8.1) and (8.2). No appreciable difference exists between the theoretical predictions obtained from the exact and approximate inelastic column formulas as shown by these curves. For slenderness ratios, λ, greater than approximately 90, curves 1 and 2 coincide since $E_t \simeq E$ in this range.

Except for column types B2 and D, good agreement is observed between the experimental results and the theoretical predictions obtained from either the exact or approximate inelastic buckling formulas. For any column type, the maximum difference between the predictions, based on the approximate formula, and the experimental values is 26.87 percent, and the average difference is 11.58 percent. The overall average difference for layered columns is 6.87 percent.

For Type B2 and Type D columns the cross-sections shown in Figures 8.B2.1 and 8.D.1 indicate the values of m (number of interfaces) to be 4 and 6 respectively. In view of the fact that shearing forces are higher at the interfaces closer to the neutral axis of the column and considering the nail size and nailing pattern, it seems that the critical slip planes exist at the two inner interfaces of Type B2 columns and at the inner four interfaces of Type D columns. This gives rise to values of m equal to 2 and 4 for column types B2 and D respectively. In Figures 8.B2.3 and 8.D.3, curve 2 (approximate inelastic formula) is replotted using $m = 2$ (Type B2) and $m = 4$ (Type D) and identified as curve 5. In both cases, assuming the above explanation to be valid, the theoretical predictions are observed to be in very good agreement with the experimental results.

Referring to Figures 8.A1.2, 8.A2.2, 8.B1.2 etc., curve 3 represents the elastic buckling formula given by Eq. (8.3). This formula is similar to the inelastic buckling formula used to plot curves 1 and 2 except that the initial modulus of elasticity, E, is used throughout, instead of the tangent modulus of elasticity, E_t.

Comparison of curve 3 (elastic buckling formula) with either of curves 1 or 2 (inelastic buckling formula) illustrates that the elastic buckling formula is theoretically valid only for layered columns having values of slenderness ratio, λ, greater than approximately 120, that is, very long columns.

However, the validity of the elastic buckling formula is not in question here. It is expected that the elastic buckling formula would be valid for long columns in which the buckling stresses are within the elastic range. Furthermore, for such long columns, the inelastic buckling formula becomes identical to the elastic buckling formula since the tangent modulus of elasticity, E_t, is equal to the initial modulus of elasticity, E, within the elastic stress range.

The point to be made from a comparison of the results provided by the elastic and inelastic buckling formulas is that the inelastic buckling formula accurately predicts the buckling stresses for columns of any slenderness ratio and is not restricted to elastic (or long) columns as is the case with the elastic buckling formula. In addition, referring to Eq. (3.23), the inelastic buckling formula presents no increased inconvenience in its application as compared to the elastic buckling formula.

Referring again to Figures 8.A1.2, 8.A2.2, 8.B1.2 etc., a theoretical prediction curve for equivalent solid column strength, corresponding to each layered column, is given by the column curve identified as curve 4. This curve was obtained from the tangent modulus formula given by Eq. (8.4). Curve 4 represents the upper bound of the inelastic buckling formulas given by Eq. (8.1) or (8.2). A comparison of curve 4 with either of curves 1 or 2 provides a graphical appreciation of the efficiency, considering the connector type and quantity, of each layered column type.

8.2.3. Statistical Analysis of Layered Column Results

A statistical analysis was conducted to determine the reliability of the proposed inelastic formulas to predict buckling stresses in layered columns in various probability ranges. The results are shown in Figures 8.A1.3, 8.B1.3, ..., 8.G.3 for column types A1, B1, ..., G, respectively.

The theoretical prediction curves shown, indicating mean, and the probability ranges (mean $\pm$ 1.00σ), (mean $\pm$ 1.96σ) and (mean $\pm$ 3.09σ) were generated from the approximate inelastic formula using the mean and the upper and lower values of E and F_u, for each column type, in the ranges (mean $\pm$ 1.00σ), (mean $\pm$ 1.96σ) and (mean $\pm$ 3.09σ). Individual test results are also plotted and the percentage of test result in each probability range is indicated.

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Based on the results obtained by Malhotra (31), the strength properties, E and F_u, are taken to follow a normal distribution. In his investigation, Malhotra conducted a large number of tests on compression specimens fabricated from Construction Grade No. 1 Eastern Spruce lumber and subjected the results to a rigorous statistical analysis. The results of the analysis indicated that the strength properties he obtained closely followed a normal distribution. Since the test material used in this study is Construction Grade No. 1 Eastern Spruce lumber it seemed reasonable to accept the results obtained by Malhotra.

The reason for selecting the above mentioned ranges is that the values of area under the normal probability curve bounded within these ranges are of special interest. The deviation of $\pm\ \sigma$ from the mean as given in the range (mean $\pm$ 1.00σ) is referred to as a standard error, and is often quoted as a measure of precision. The range (mean $\pm$ 1.96σ), that is, 2½ percent exclusion limit (one-tail distribution), is extensively used in statistical treatment of data. The range (mean $\pm$ 3.09σ), that is, 0.1 percent exclusion limit (one-tail distribution), is useful for arriving at design curves. The statistical probability of encountering a test result falling below the lower limit of this range is extremely low, 1 in 1000. A design based on this lower limit value will, therefore, have adequate safety.

The theoretical probabilities of the results falling in the ranges (mean $\pm$ 1.00σ), (mean $\pm$ 1.96σ) and (mean + 3.09σ) are 68.26%, 95% and 99.8%, respectively. For most of the layered column types, the percentages of the actual test results in the various ranges compare reasonably well with the probability predictions. This indicates that the proposed inelastic column formula is a satisfactory predictor of column stresses in various probability ranges.

8.2.4. Summary

The results of the layered column tests have been presented.

Based on the mean results, very good agreement was observed between the experimental results and the theoretical predictions. The overall average difference between the predictions and the experimental values is 6.87 percent.

The results of a statistical analysis indicated the proposed inelastic column formula to be a satisfactory predictor of column stresses in various probability ranges.

8.3. Braced and Spaced Columns

8.3.1. General

Seven sets of braced and spaced columns, as summarized below, were fabricated from 2-2x4's with center to center spacing, S = 4.5" (Types H1 to H5) or S = 5.5" (Types I1 and I2).

Column Type	Description
H1	Braced, 45° braces, S = 4.5"
H2	Braced, 45° braces and end blocks, S = 4.5"
H3	Braced, horizontal braces, S = 4.5"
H4	Braced, horizontal braces and end blocks, S = 4.5"
H5	Spaced, five spacer blocks, S = 4.5"
I1	Braced, 45° braces, S = 5.5"
I2	Braced, horizontal braces, S = 5.5"

The results of the braced and spaced column tests are grouped together in Figures 8.H1.1 to 8.I2.3 and presented in the manner described in section 8.1.

8.3.2. Buckling Stress Formulas for Braced Columns (45° braces) and Spaced Columns

The buckling stress formulas for braced (45^o braces) and spaced columns have been discussed in section 3.3. The buckling formulas used to obtain the various theoretical column curves are summarized below. The notations used are the same as in Chapters 2 and 3.

Inelastic Buckling Formula- Exact Solution

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot B \qquad (8.5)$$

$$\text{where } B = \frac{1 + \alpha \dfrac{\pi^2 E_t A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 l_s L^2}}{1 + \dfrac{\pi^2 E_t A_r a}{mkL^2} + \dfrac{\pi^2 L_c^3}{12\alpha l_s L^2}}$$

Inelastic Buckling Formula - Approximate Solution

[E used instead of E_t in Factor B of Eq. (8.4)]

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 l_s L^2}}{1 + \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12\alpha l_s L^2}} \qquad (8.6)$$

Elastic Buckling Formula

[E used instead of E_t throughout Eq. (8.5)]

$$F_u = \frac{\pi^2 E}{\lambda^2} \cdot \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 l_s L^2}}{1 + \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 \alpha l_s L^2}} \quad (8.7)$$

Buckling stresses for an equivalent layered column are determined from Eq. (8.2). An equivalent layered column refers to a layered column fabricated with 2-2x4's having the same length and fastened with the same type and number of connectors of any particular braced or spaced column.

Buckling stresses for an equivalent solid column are determined from Eq. (8.4). An equivalent solid column refers to a solid column with cross-sectional dimensions equivalent to 2-2x4's and having the same length of the corresponding braced or spaced column.

8.3.3. Buckling Stress Formulas for Braced Columns with Horizontal Braces

The proposed buckling stress formula for braced columns with horizontal braces has been discussed in section 3.3. Theoretical column curves generated from the inelastic (exact and approximate) and elastic buckling formulas, are plotted using Eqns. (8.1), (8.2) and (8.3) (layered column formulas) respectively, each reduced by the factor $\Sigma h/L_1$; where h equals the vertical height of each brace and L equals the length of the column member. Buckling stress formulas for equivalent layered and equivalent solid columns are the same as described in section 8.3.2.

8.3.4. Discussion of Braced and Spaced Column Results

The results of the braced and spaced column tests, as shown in Figures 8.H1.1 to 8.I2.3, indicate good agreement between the experimental results and the theory. The overall average difference between the predictions and the experimental values is 6.73 percent. The general discussion given for layered columns is applicable here except for the differences pointed out in the following paragraphs.

Referring to Figures 8.H1.2, 8.H2.2,...., 8.I2.2, curves 1, 2 and 3 are theoretical prediction curves using the inelastic (approximate) and elastic buckling formulas, respectively, as given in sections 8.3.2. or 8.3.3. Curve 4 is a theoretical curve for an equivalent layered column obtained using the approximate inelastic formula for layered columns, Eq. (8.2). Curve 5 represents the theoretical buckling stresses for an equivalent solid column obtained from Eq. (8.4). Equivalent layered and equivalent solid column have been defined in section 8.3.2.

The slenderness ratios of the equivalent layered and equivalent solid columns are greater than those of the corresponding braced, or spaced columns. In Figs. 8.H1.2, 8.H2.2,....., 8.I2.2, the slenderness ratios indicated apply only to the spaced and braced columns. However, curves 4 and 5 which are theoretical curves for equivalent layered and equivalent solid columns, are plotted at the same slenderness ratios of the corresponding braced and spaced columns in order to show, on the same graph, the comparative buckling stresses between the braced or spaced columns and the equivalent layered or equivalent solid columns of the same length.

Figures 8.H3.2, 8.H4.2 and 8.I2.2 show the results for the braced columns with horizontal braces. For these three column types good agreement is observed between the experimental results and the theory (curves 1 or 2), except for the shortest columns tested, in which case the theoretical predictions are approximately 17 percent lower than the experimental results. Probably for such short columns the analogy between a braced column (horizontal braces) and a layered column does not apply.

8.3.5. Statistical Analysis of Braced and Spaced Column Results

As explained for layered columns in section 8.2.3., the results of the braced and spaced column tests were subjected to a statistical analysis to determine the reliability of the proposed column formulas to predict column stresses in various probability ranges.

The results of this statistical analysis are presented in Figures 8.H1.3, 8.H2.3,....., 8.I2.3 for Column Types H1, H2, ..., I2, respectively. The theoretical prediction curves were generated using the approximate inelastic formulas for braced and spaced columns.

It is observed, for Column Types H3, H4, H5 and I2, that 100 percent of the test results lie within the range (Mean $\pm$ 1.96σ). For Column Types H1, H4 and I1., 100% of the test results lie within the range (Mean $\pm$ 1.00σ). This distribution of the experimental results could possibly be attributed to the small number of braced and spaced columns tested.

8.3.6. Summary

The results of the braced and spaced column tests have been presented.

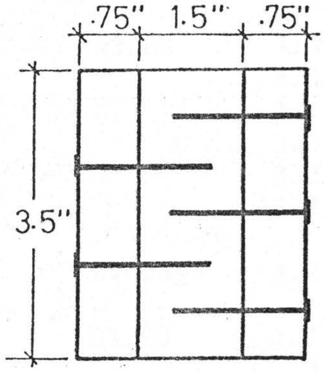
Good agreement was observed between the experimental results and the predictions by applying the spaced column formula to both spaced columns and braced columns with 45°

braces.

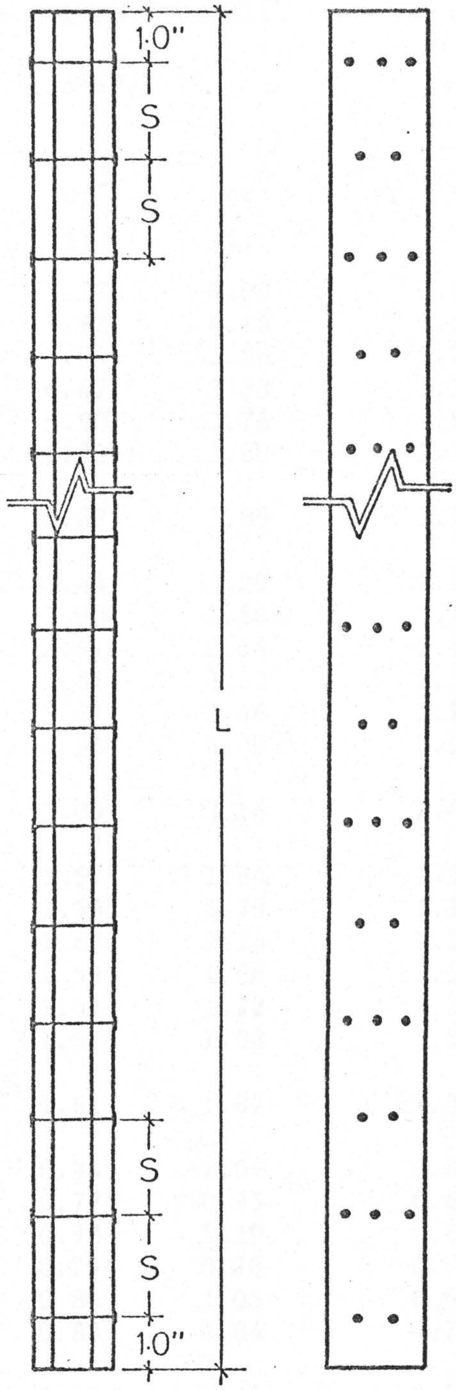
Similar results were obtained by modifying the layered column formula to predict column stresses in braced columns with horizontal braces.

The overall average difference between the predictions and the experimental values, for braced and spaced columns, is 6.73 percent.

The results of the statistical analysis indicate that the proposed inelastic buckling formulas are satisfactory for predicting column stresses in braced and spaced timber columns.



Cross Section



Connector Data

Type: 2" common wire nails

Quantity: 144

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
30.0	6	1.00
45.0	6	1.50
70.0	6	2.50
96.0	6	3.50
130.0	6	4.50

FIGURE 8.A1.1 Details of Column Type A1

TABLE 8.A1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE A1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
A1-1	43.88	3.39	3.51	5.37	4.00	2.98
A1-2		3.55	3.67	5.92	4.14	2.66
A1-3		2.98	3.09	4.64	3.56	3.22
A1-4		3.34	3.45	6.47	3.83	3.37
A1-5		3.24	3.36	5.80	3.76	3.54
A1-6		3.25	3.38	5.83	3.80	3.10
	Average	3.27	3.41	5.67	3.85	3.15
A1-7	61.20	2.74	2.81	3.44	3.29	2.50
A1-8		2.36	2.42	2.90	2.86	2.47
A1-9		2.31	2.35	2.61	2.94	2.79
A1-10		2.42	2.47	2.81	3.03	2.34
A1-11		2.82	2.88	3.41	3.46	2.10
A1-12		2.76	2.82	3.34	3.39	2.50
	Average	2.57	2.63	3.09	3.16	2.45
A1-13	90.07	1.48	1.49	1.57	1.84	1.52
A1-14		1.44	1.46	1.54	1.79	1.16
A1-15		1.72	1.74	1.87	2.14	1.16
A1-16		1.49	1.51	1.59	1.86	1.52
A1-17		1.39	1.41	1.51	1.72	1.32
A1-18		1.50	1.52	1.60	1.88	1.32
	Average	1.51	1.52	1.61	1.87	1.33
A1-19	120.09	0.83	0.83	0.84	1.04	0.87
A1-20		0.76	0.76	0.77	0.95	0.85
A1-21		0.95	0.95	0.98	1.19	0.87
A1-22		0.77	0.77	0.79	0.96	0.71
A1-23		0.84	0.84	0.86	1.05	0.68
A1-24		0.82	0.82	0.83	1.04	0.76
	Average	0.83	0.83	0.85	1.04	0.79
A1-25	159.35	0.56	0.56	0.57	0.65	0.58
A1-26		0.49	0.49	0.50	0.57	0.49
A1-27		0.45	0.45	0.45	0.51	0.55
A1-28		0.56	0.56	0.57	0.65	0.58
A1-29		0.52	0.52	0.53	0.60	0.50
A1-30		0.54	0.54	0.56	0.62	0.55
	Average	0.52	0.52	0.53	0.60	0.54

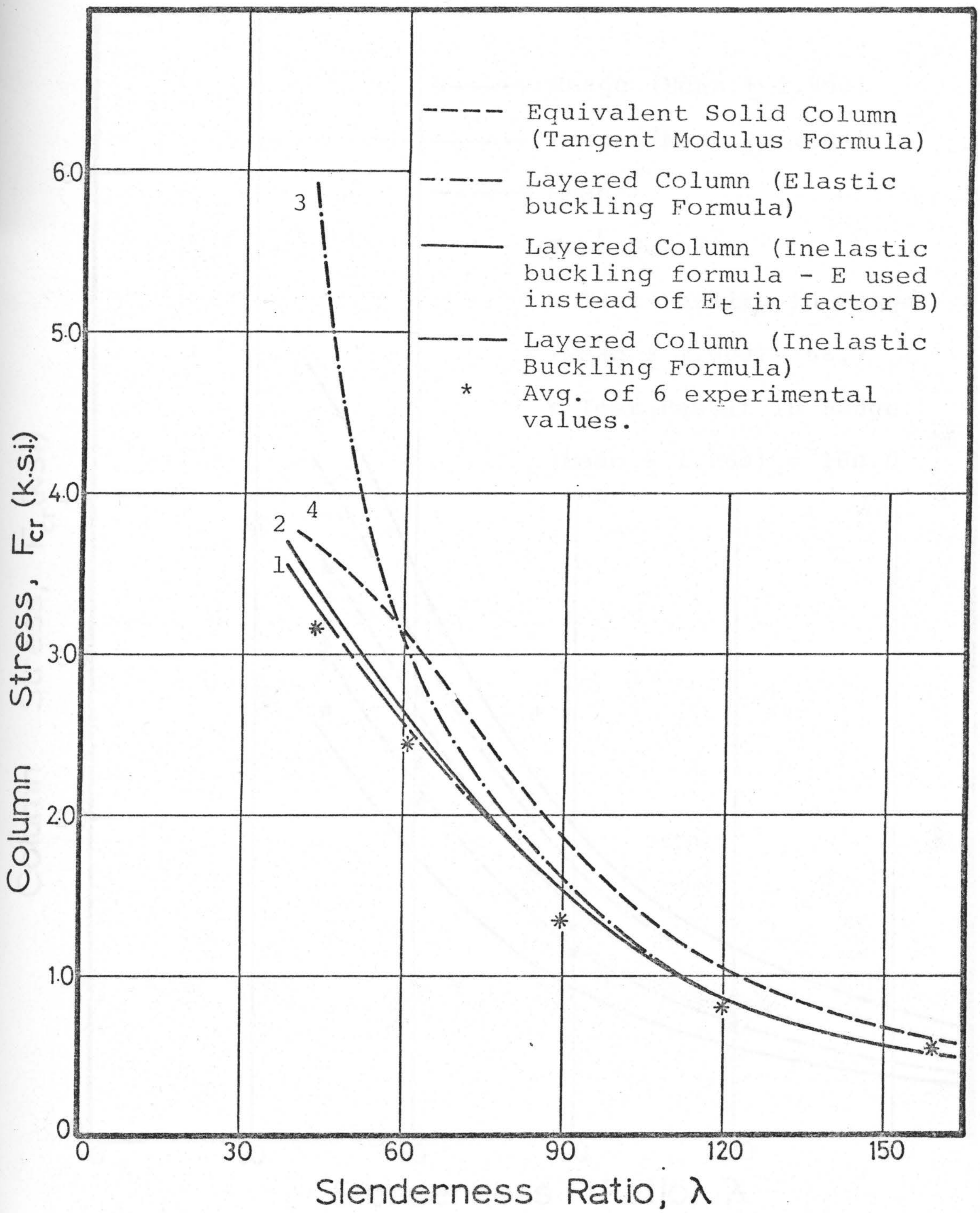


FIGURE 8.A1.2 Column Stress versus Slenderness Ratio Curves - Column Type A1

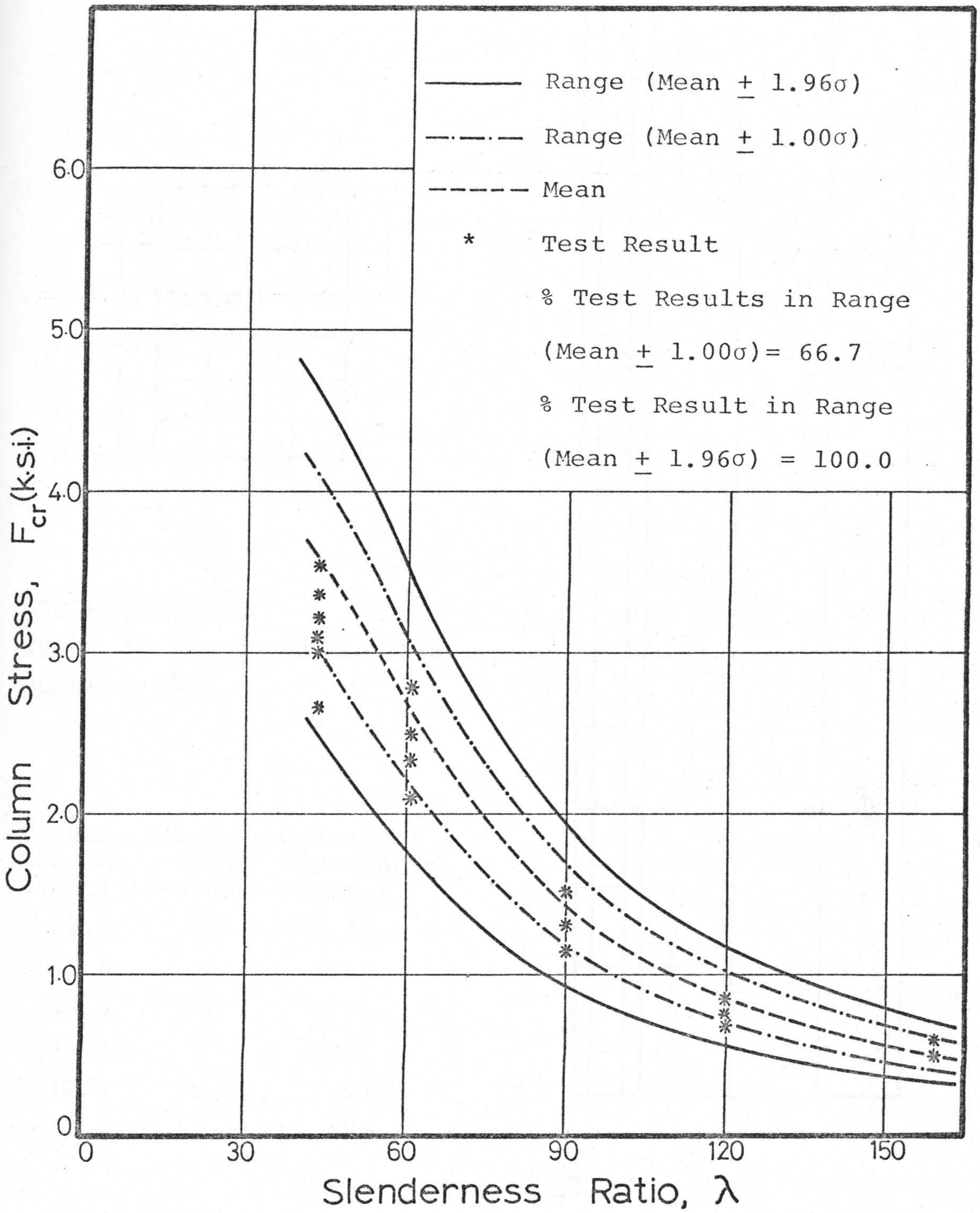
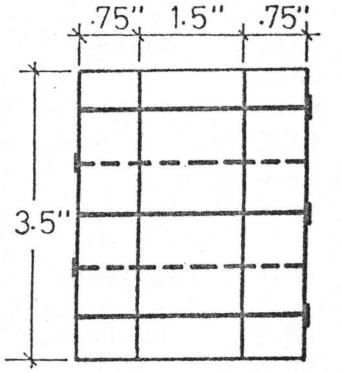
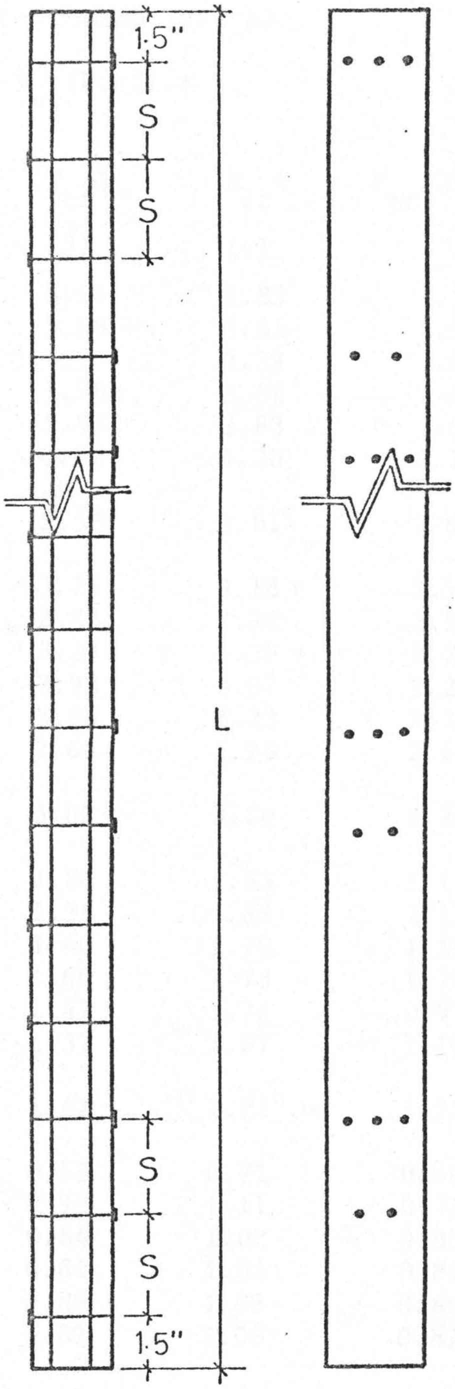


FIGURE 8.A1.3 Statistical Analysis Curves- Column Type A1



Cross Section



Connector Data

Type: 3" common wire nails

Quantity: 48

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
30.0	6	1.50
45.0	6	2.50
70.0	6	4.00
96.0	6	5.50
130.0	6	7.50

FIGURE 8.A2.1 Details of Column Type A2

TABLE 8.A2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE A2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
A2-1	43.88	3.44	3.56	5.26	3.85	3.54
A2-2		3.46	3.57	5.68	3.81	4.12
A2-3		2.98	3.06	4.77	3.29	3.41
A2-4		3.49	3.61	5.98	3.82	3.48
A2-5		3.45	3.55	5.55	3.83	3.65
A2-6		3.65	3.98	6.22	4.26	3.76
	Average	3.44	3.56	5.58	3.81	3.66
A2-7	61.20	2.60	2.65	3.15	3.18	2.11
A2-8		2.39	2.44	2.83	2.94	2.55
A2-9		2.68	2.74	3.19	3.29	2.21
A2-10		2.50	2.55	2.98	3.07	2.23
A2-11		2.67	2.73	3.26	3.23	2.25
A2-12		2.32	2.36	2.66	2.90	2.18
	Average	2.53	2.58	3.02	3.10	2.26
A2-13	90.07	1.19	1.20	1.26	1.53	1.13
A2-14		1.31	1.31	1.38	1.68	1.13
A2-15		1.32	1.33	1.40	1.70	1.19
A2-16		1.51	1.52	1.60	1.93	1.70
A2-17		1.38	1.39	1.47	1.76	0.97
A2-18		1.30	1.30	1.37	1.67	1.19
	Average	1.34	1.35	1.42	1.72	1.22
A2-19	120.09	0.55	0.55	0.56	0.71	0.69
A2-20		0.89	0.89	0.92	1.11	0.71
A2-21		0.82	0.82	0.84	1.02	0.83
A2-22		0.79	0.79	0.81	1.04	0.81
A2-23		0.87	0.87	0.89	1.08	0.85
A2-24		0.87	0.87	0.89	1.08	0.87
	Average	0.80	0.80	0.82	1.00	0.79
A2-25	159.35	0.54	0.54	0.54	0.62	0.49
A2-26		0.47	0.47	0.48	0.54	0.51
A2-27		0.51	0.51	0.52	0.58	0.55
A2-28		0.49	0.49	0.50	0.57	0.67
A2-29		0.47	0.47	0.48	0.55	0.47
A2-30		0.55	0.55	0.56	0.63	0.56
	Average	0.50	0.50	0.51	0.58	0.53

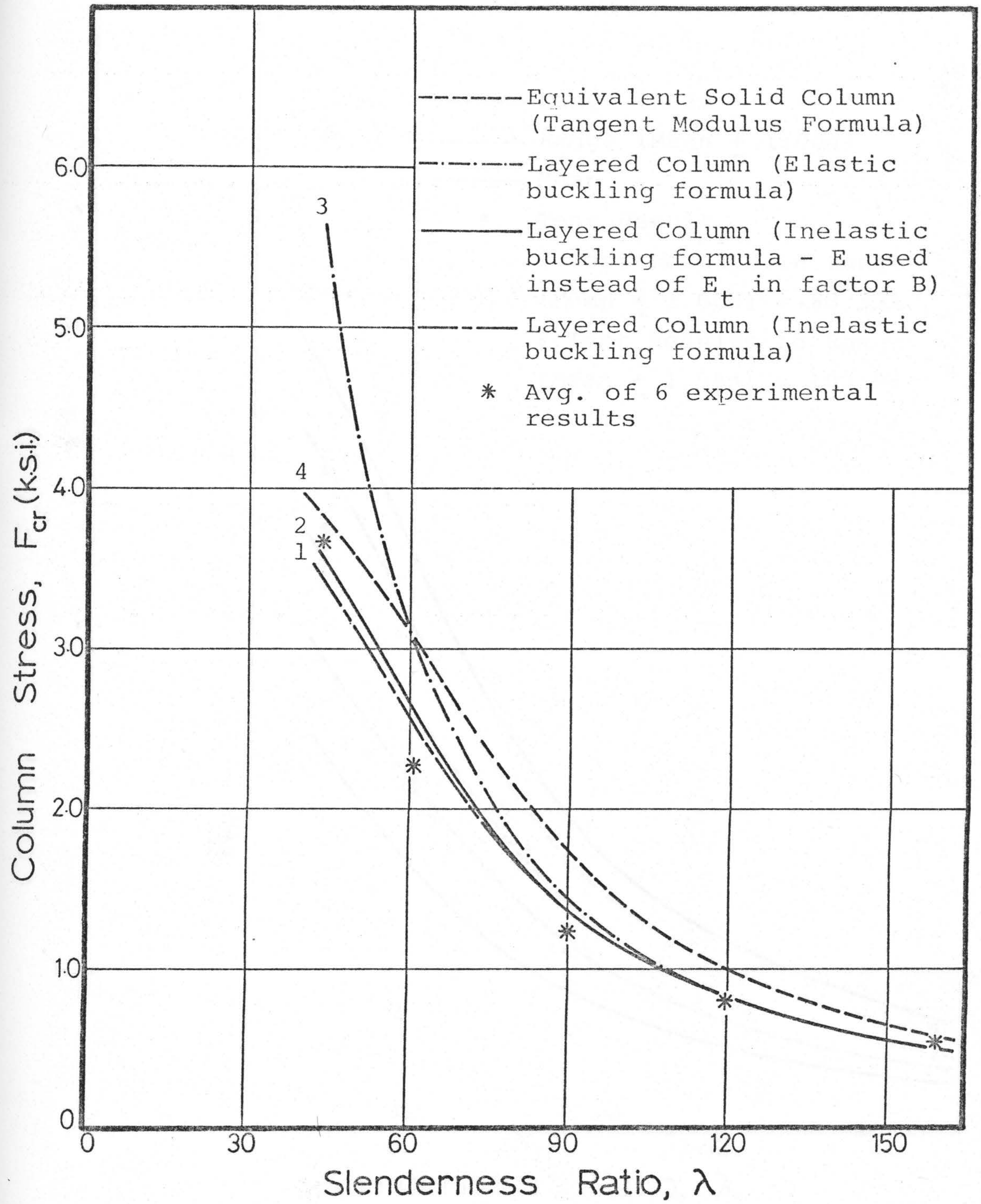


FIGURE 8.A2.2 Column Stress versus Slenderness Ratio Curves - Column Type A2

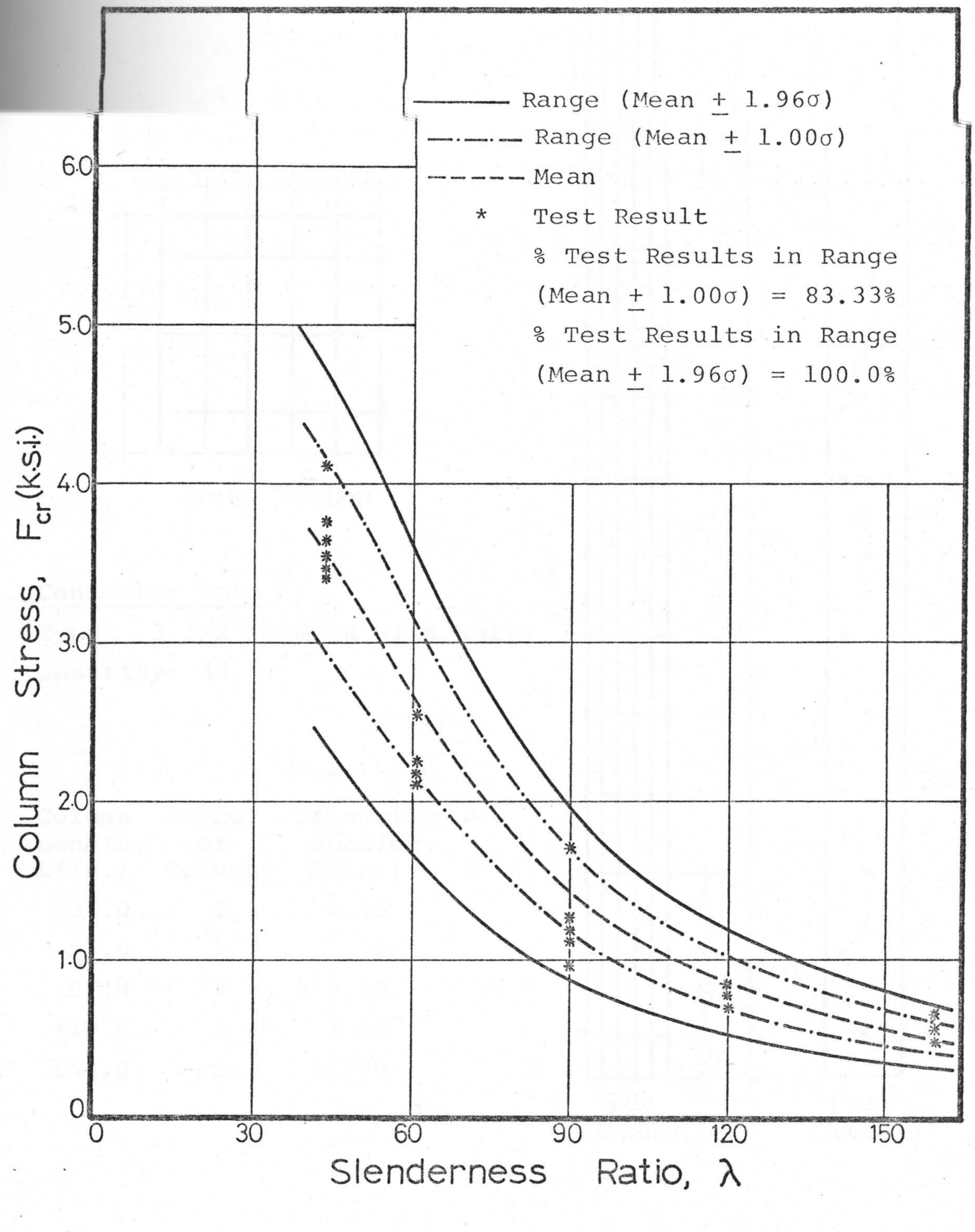
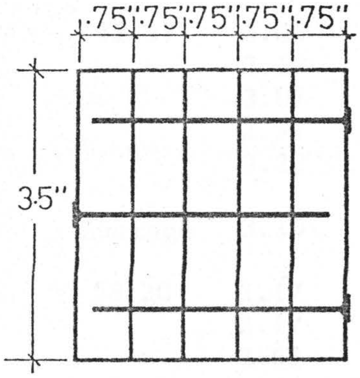
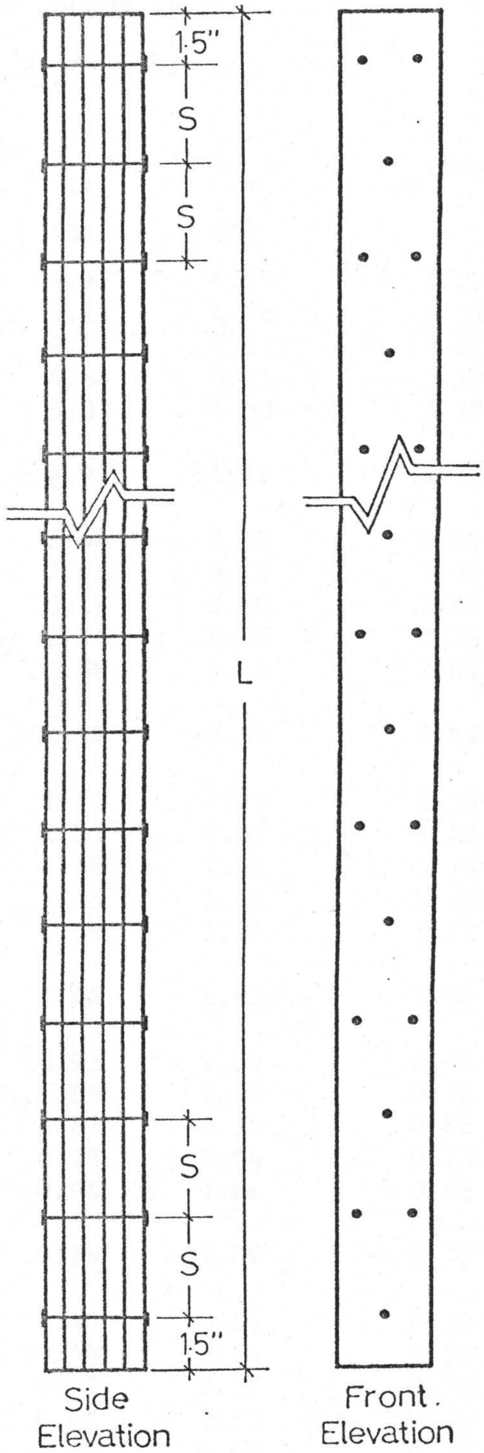


FIGURE 8.A2.3 Statistical Analysis Curves - Column Type A2





Connector Data

Type: 3 1/2" common wire nails

Quantity: 48

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
37.0	5	2.25
55.0	5	3.50
87.0	5	5.50
120.0	5	7.50
158.0	5	10.00

FIGURE 8.B1.1 Details of Column Type B1

TABLE 8.B1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE B1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		F_{cr}^1 (1)	F_{cr}^2 (2)	F_{cr}^3 (3)	F_{cr}^4 (4)	F_{cr}(Exp.)
B1-1	41.57	3.06	3.28	4.92	4.12	3.15
B1-2		3.19	3.41	5.17	4.28	3.66
B1-3		3.07	3.29	4.81	4.17	2.88
B1-4		2.99	3.20	4.84	4.01	3.30
B1-5		3.20	3.42	5.03	4.33	2.52
	Average	3.10	3.32	4.96	4.18	3.10
B1-6	58.20	2.17	2.23	2.64	3.17	2.83
B1-7		2.17	2.23	2.64	3.17	2.39
B1-8		2.62	2.70	3.19	3.80	2.22
B1-9		2.24	2.30	2.74	3.25	2.73
B1-10		2.35	2.40	2.88	3.40	2.01
	Average	2.31	2.38	2.82	3.36	2.44
B1-11	87.76	1.37	1.39	1.44	1.93	1.54
B1-12		1.47	1.49	1.56	2.07	1.13
B1-13		1.40	1.41	1.48	1.96	1.33
B1-14		1.29	1.30	1.35	1.82	1.26
B1-15		1.31	1.30	1.38	1.85	1.62
	Average	1.37	1.38	1.44	1.93	1.38
B1-16	118.24	0.92	0.93	0.95	1.06	0.74
B1-17		1.02	1.03	1.05	1.20	0.75
B1-18		0.85	0.85	0.87	0.96	0.68
B1-19		0.91	0.91	0.93	1.04	0.73
B1-20		0.92	0.92	0.94	1.06	0.83
	Average	0.92	0.92	0.95	1.07	0.75
B1-21	153.35	0.45	0.45	0.46	0.56	0.40
B1-22		0.43	0.43	0.44	0.54	0.54
B1-23		0.47	0.47	0.47	0.58	0.56
B1-24		0.42	0.42	0.43	0.53	0.34
B1-25		0.53	0.53	0.54	0.66	0.39
	Average	0.46	0.46	0.47	0.57	0.44

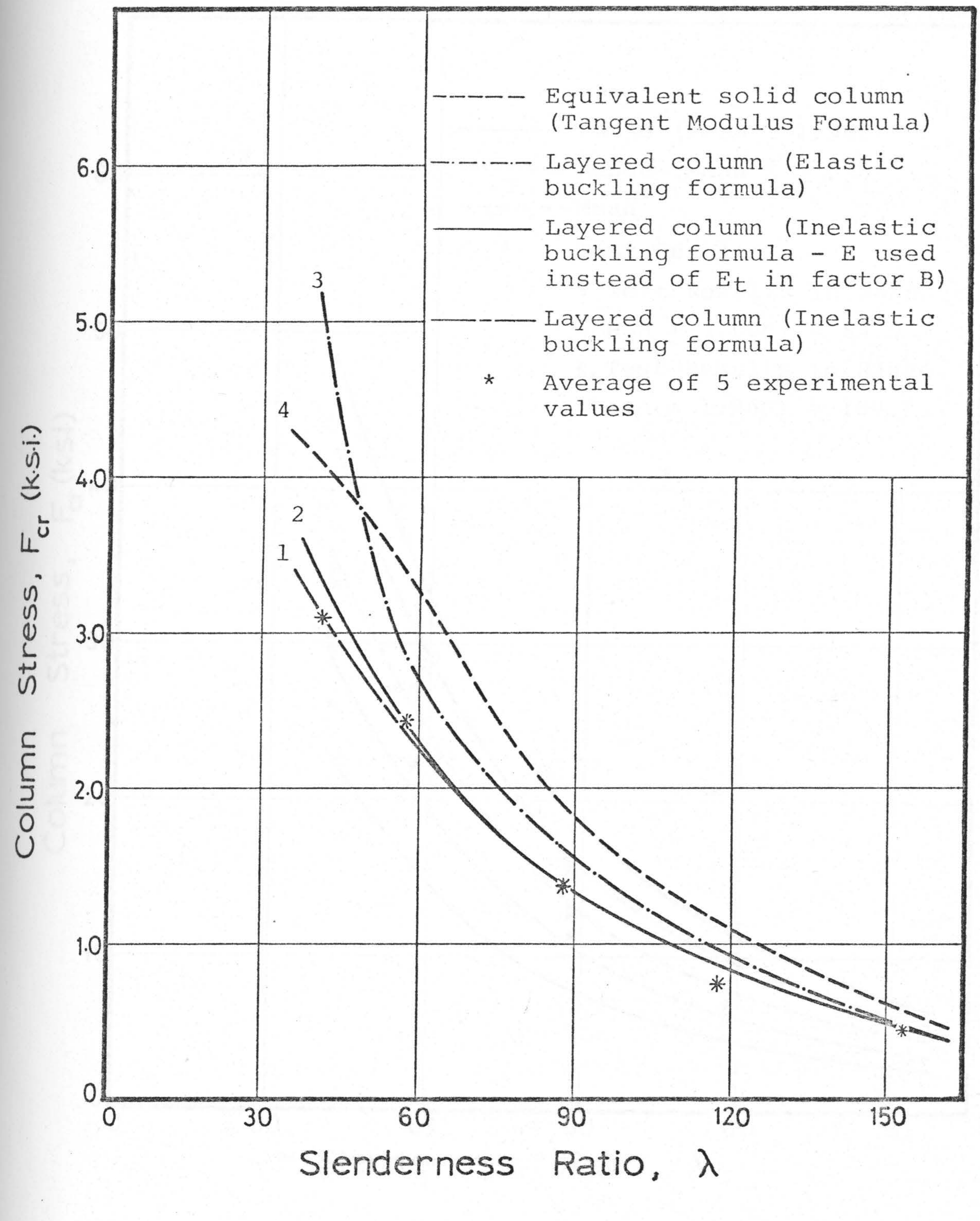


FIGURE 8.B1.2 Column Stress versus Slenderness Ratio Curves - Column Type B1

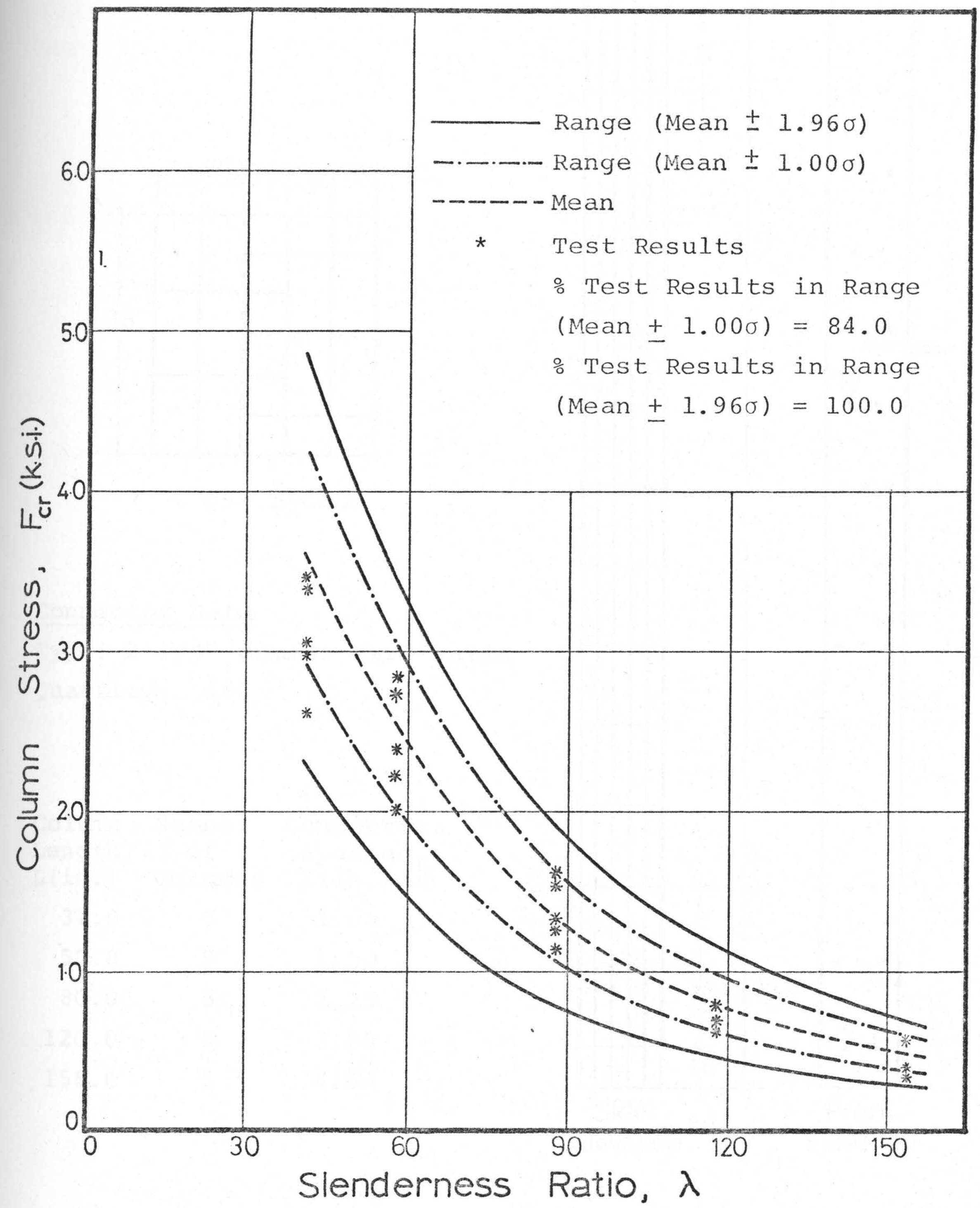
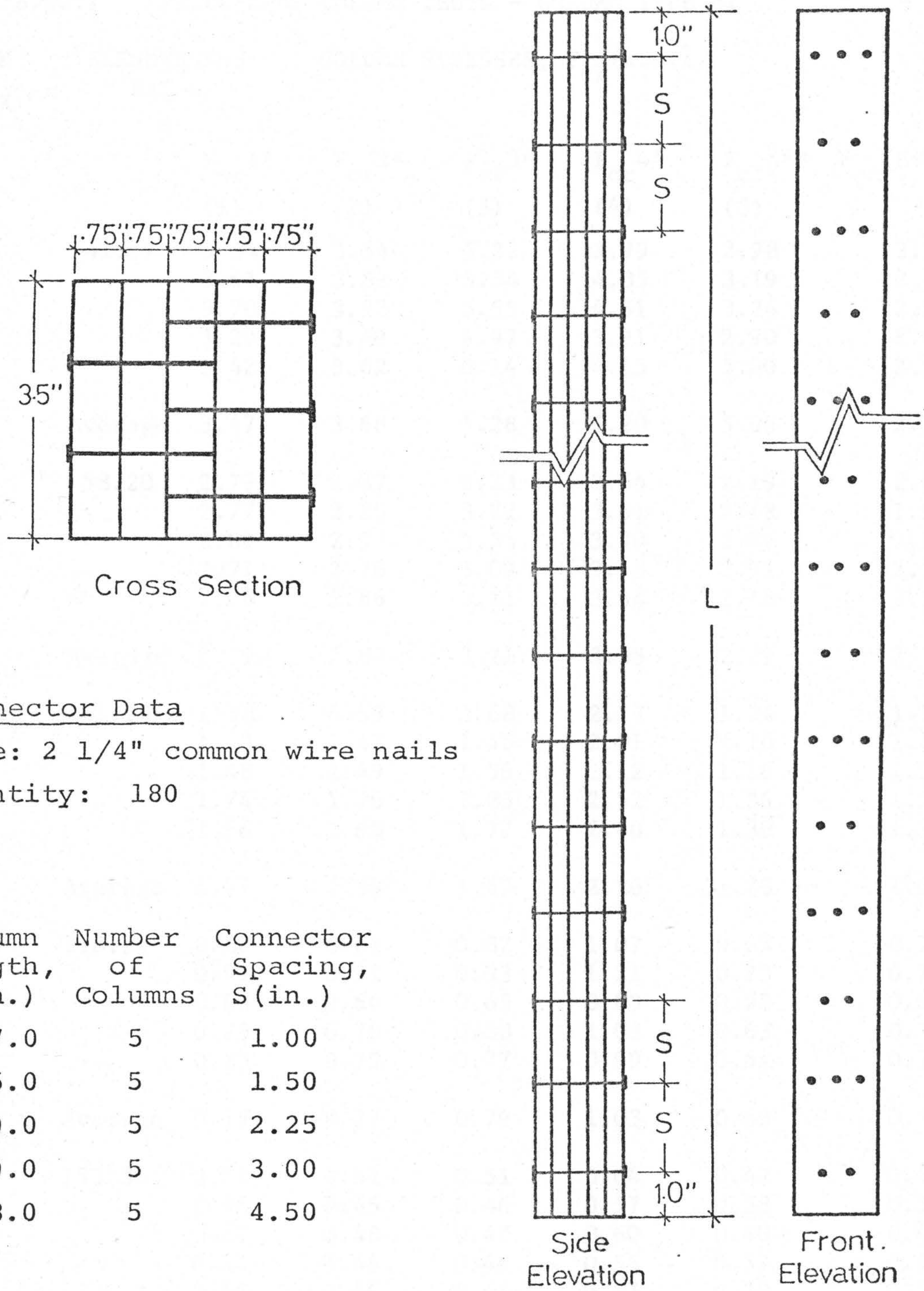


FIGURE 8.B1.3 Statistical Analysis Curves - Column Type B1



Connector Data

Type: 2 1/4" common wire nails

Quantity: 180

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
37.0	5	1.00
55.0	5	1.50
80.0	5	2.25
120.0	5	3.00
158.0	5	4.50

FIGURE 8.B2.1 Details of Column Type B2

TABLE 8.B2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE B2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)					
		$F_{cr}1$* (1)	$F_{cr}2$* (2)	$F_{cr}3$* (3)	$F_{cr}4$* (4)	$F_{cr}5$** (5)	F_{cr}(Exp.)
B2-1	41.57	3.34	3.54	5.22	3.99	2.98	3.12
B2-2		3.61	3.82	5.54	4.35	3.19	2.88
B2-3		3.70	3.92	5.55	4.51	3.24	2.98
B2-4		3.29	3.49	4.97	3.91	2.90	2.66
B2-5		3.42	3.62	5.14	4.15	3.00	2.50
	Average	3.47	3.68	5.28	4.20	3.06	2.83
B2-6	58.20	2.79	2.87	3.23	3.54	2.19	2.05
B2-7		2.77	2.85	3.22	3.51	2.18	1.96
B2-8		2.89	2.97	3.35	3.70	2.27	2.10
B2-9		2.71	2.78	3.09	3.48	2.11	2.17
B2-10		2.78	2.86	3.21	3.54	2.18	2.22
	Average	2.79	2.87	3.22	3.55	2.19	2.10
B2-11	81.29	1.58	1.59	1.68	2.27	1.24	1.37
B2-12		1.40	1.42	1.48	2.01	1.10	1.47
B2-13		1.48	1.49	1.56	2.12	1.16	1.37
B2-14		1.74	1.76	1.85	2.52	1.36	1.28
B2-15		1.66	1.68	1.77	2.40	1.30	1.34
	Average	1.57	1.59	1.67	2.26	1.23	1.37
B2-16	118.24	0.80	0.81	0.82	1.07	0.65	0.72
B2-17		0.91	0.91	0.93	1.21	0.73	0.75
B2-18		0.63	0.64	0.65	0.83	0.52	0.82
B2-19		0.78	0.78	0.80	1.03	0.63	0.70
B2-20		0.75	0.75	0.77	0.99	0.61	0.77
	Average	0.77	0.77	0.79	1.03	0.63	0.75
B2-21	153.35	0.51	0.51	0.51	0.64	0.42	0.47
B2-22		0.45	0.45	0.46	0.57	0.38	0.37
B2-23		0.47	0.48	0.48	0.60	0.40	0.44
B2-24		0.44	0.44	0.45	0.55	0.37	0.46
B2-25		0.46	0.46	0.46	0.57	0.38	0.38
	Average	0.47	0.47	0.47	0.59	0.39	0.42

*M = 4 in Eqns. 8.1 to 8.3

**M = 2 in Eq. 8.2

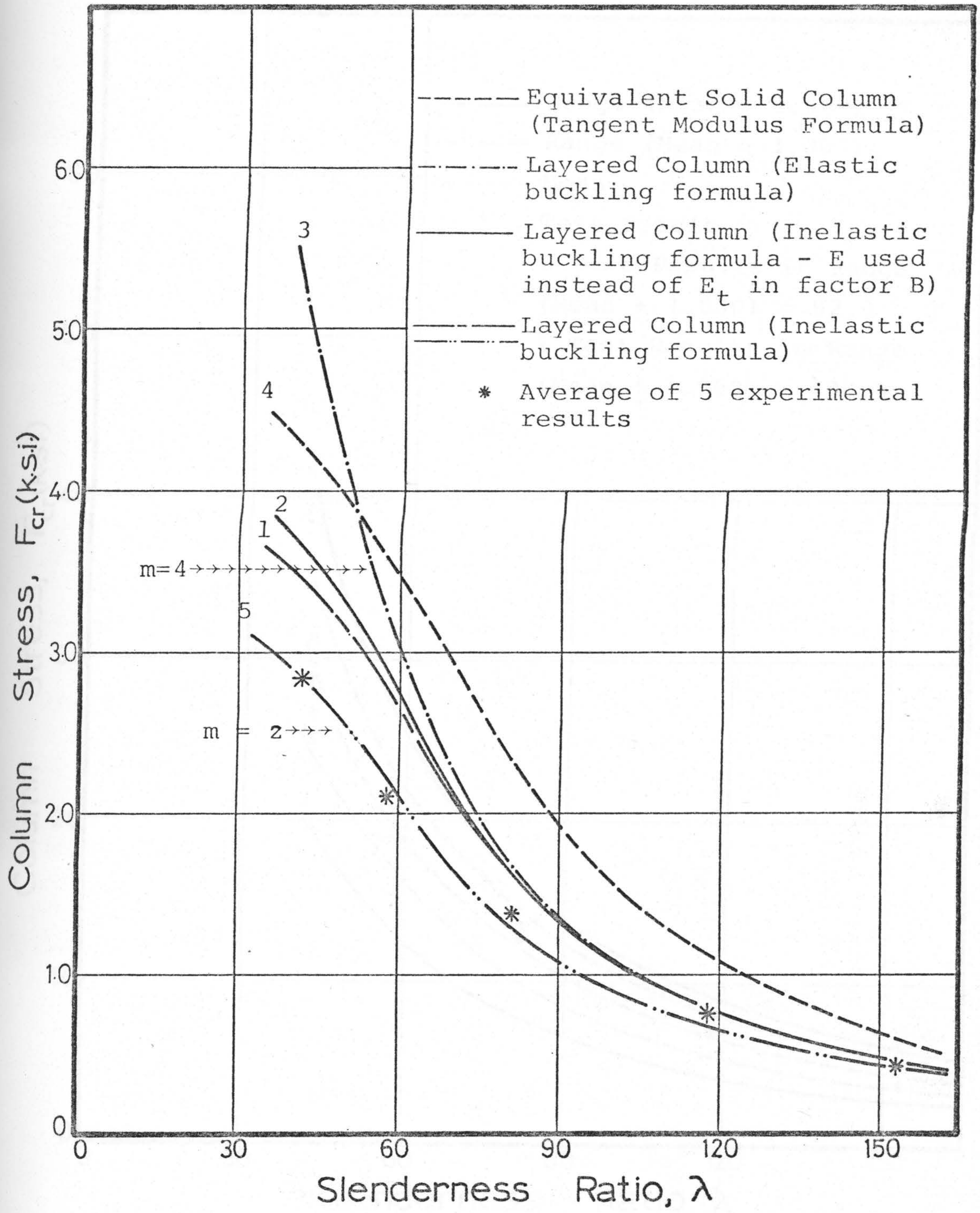


FIGURE 8.B2.2 Column Stress versus Slenderness Ratio Curves - Column Type B2

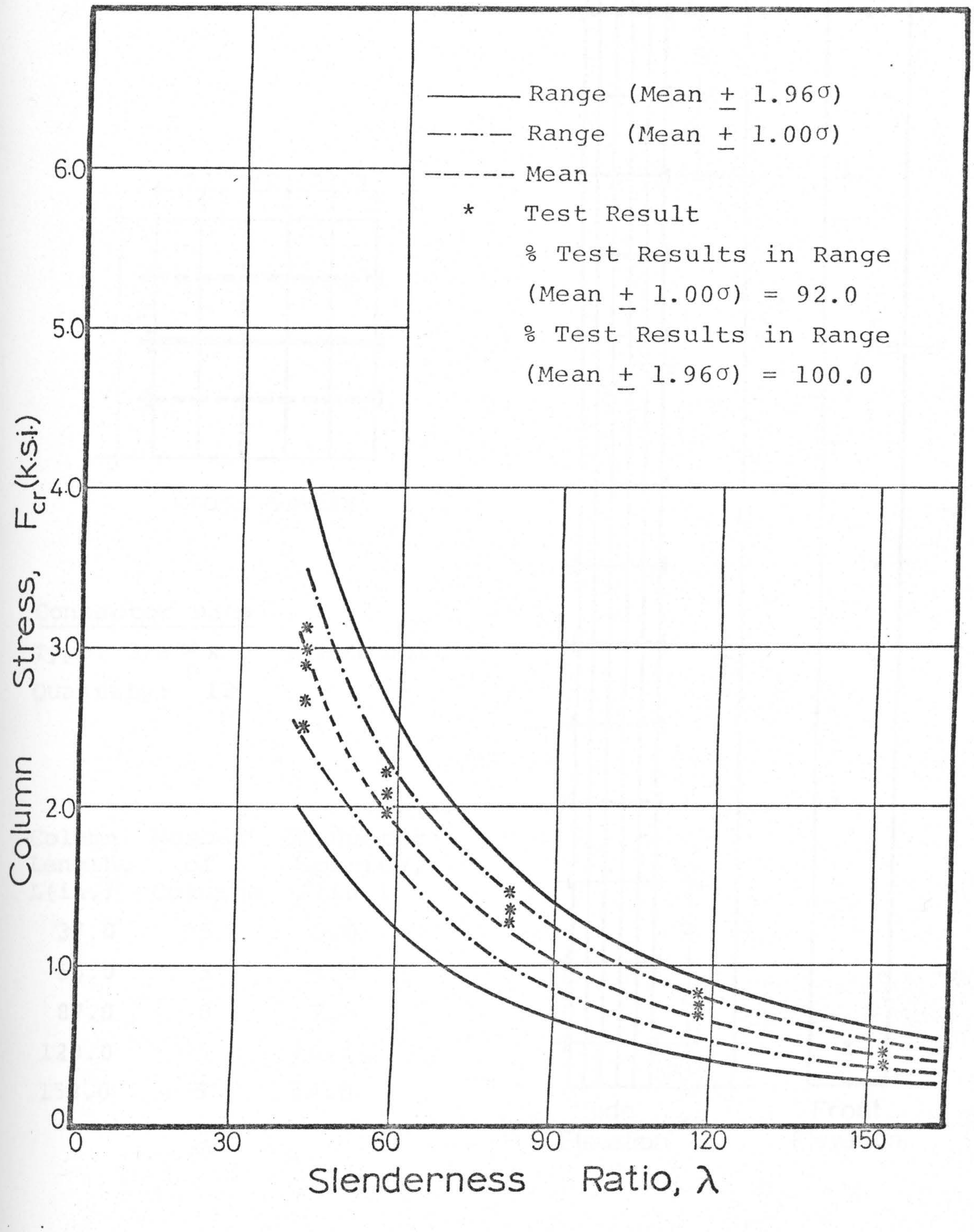
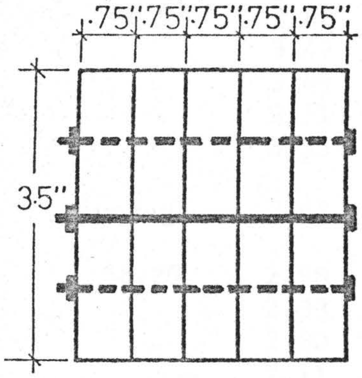
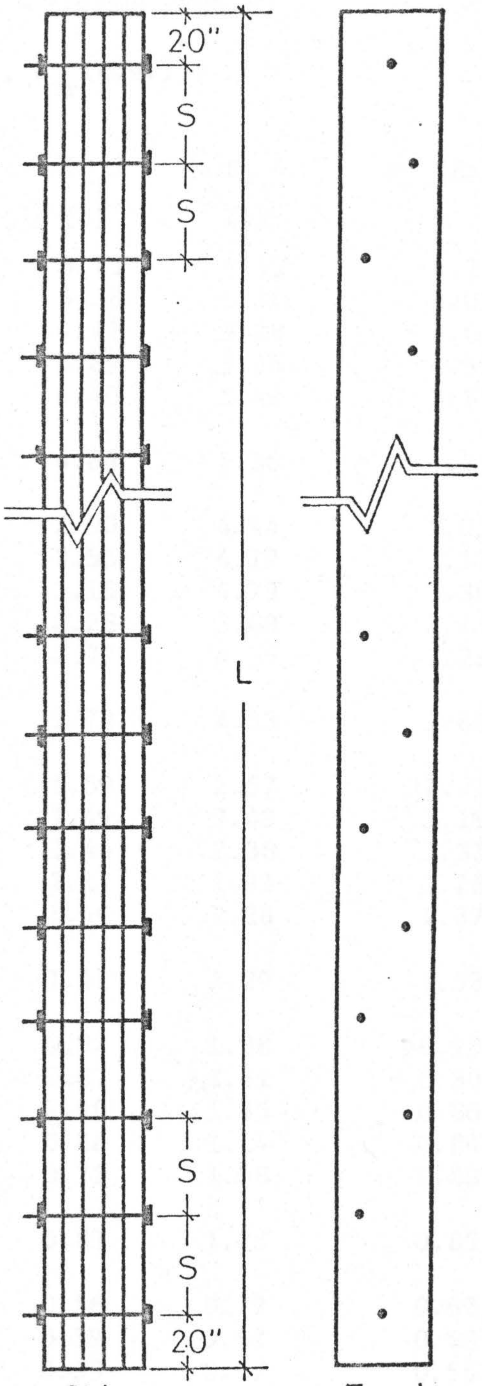


FIGURE 8.B2.3 Statistical Analysis Curves - Column Type B2



Cross Section



Side Elevation

Front Elevation

Connector Data

Type: 3/8"∅ x 5" steel bolt

Quantity: 12

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
37.0	5	3.0
55.0	5	4.5
87.0	5	7.5
120.0	5	10.5
158.0	5	14.0

FIGURE 8.B3.1 Details of Column Type B3

TABLE 8.B3.1 RESULTS OF COLUMN TESTS - COLUMN TYPE B3

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i) $F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
B3-1	41.57	3.86	4.16	5.13	5.53	4.17
B3-2		3.47	3.70	4.38	5.24	4.05
B3-3		3.49	3.73	4.41	5.28	4.66
B3-4		3.71	3.99	4.89	5.36	4.51
B3-5		3.53	3.76	4.40	5.42	3.88
	Average	3.63	3.89	4.64	5.36	4.25
B3-6	58.20	2.58	2.67	2.90	4.44	2.05
B3-7		2.33	2.39	2.55	4.19	2.17
B3-8		2.80	2.90	3.15	4.79	3.36
B3-9		2.11	2.16	2.28	3.89	2.13
B3-10		2.49	2.57	2.77	4.35	3.28
	Average	2.46	2.54	2.73	4.33	2.60
B3-11	87.76	1.44	1.47	1.59	2.57	1.71
B3-12		1.35	1.38	1.49	2.43	1.19
B3-13		1.32	1.35	1.46	2.38	1.37
B3-14		1.05	1.07	1.17	1.91	1.28
B3-15		1.26	1.29	1.40	2.26	1.37
	Average	1.29	1.31	1.41	2.29	1.38
B3-16	118.24	0.89	0.89	0.92	1.38	0.90
B3-17		0.85	0.85	0.87	1.31	0.86
B3-18		0.87	0.87	0.90	1.35	0.86
B3-19		0.73	0.73	0.86	1.14	0.84
B3-20		0.70	0.70	0.72	1.08	0.80
	Average	0.81	0.81	0.83	1.25	0.85
B3-21	153.35	0.56	0.56	0.56	0.79	0.62
B3-22		0.60	0.60	0.59	0.82	0.68
B3-23		0.55	0.55	0.55	0.77	0.56
B3-24		0.55	0.55	0.54	0.77	0.68
B3-25		0.48	0.48	0.47	0.67	0.55
	Average	0.53	0.53	0.54	0.77	0.62

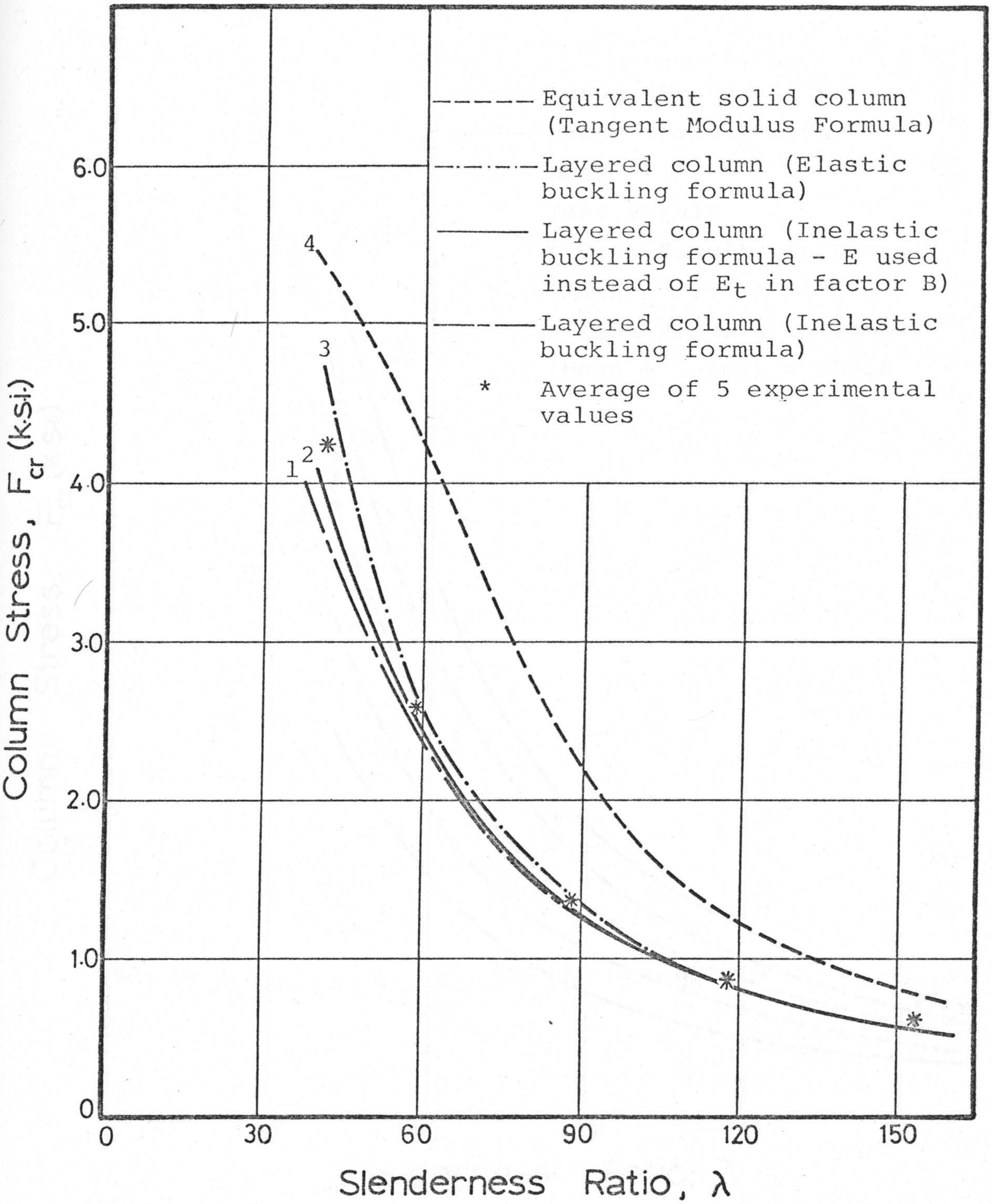


FIGURE 8.B3.2 Column Stress versus Slenderness Ratio Curves - Column Type B3

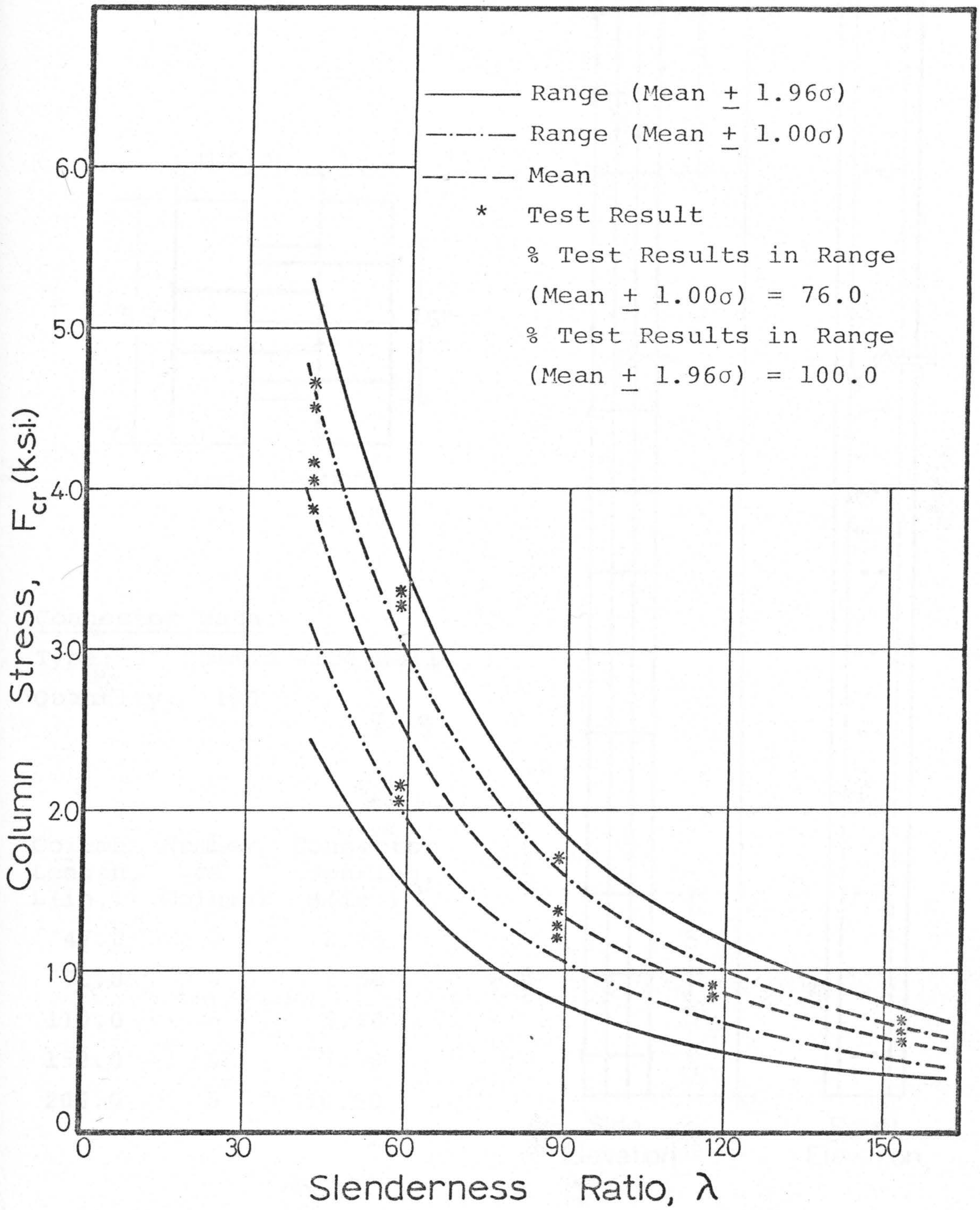
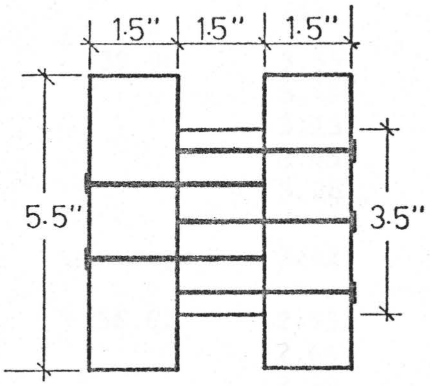
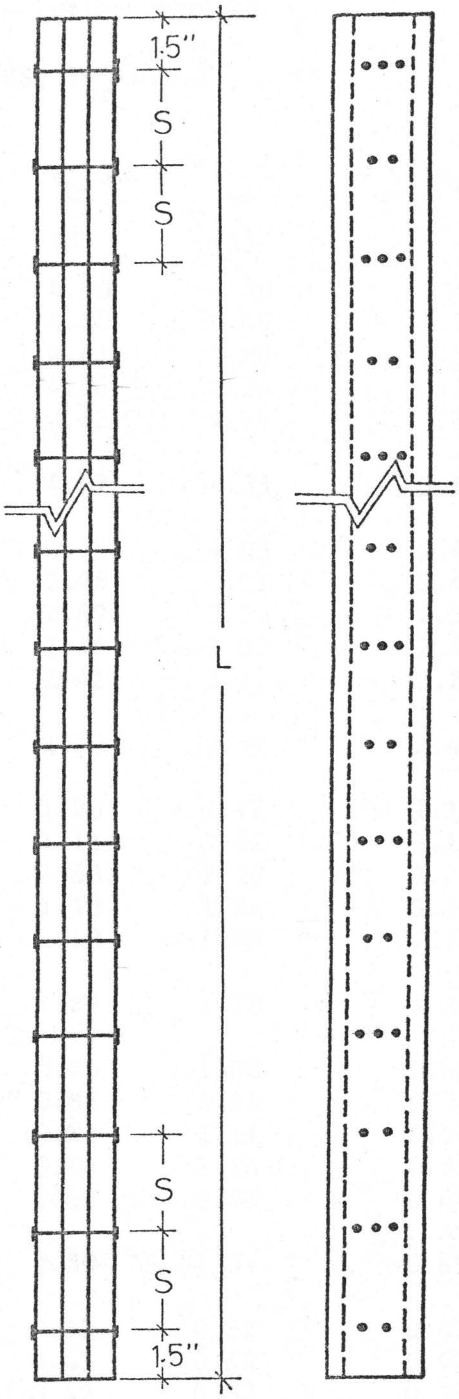


FIGURE 8.B3.3 Statistical Analysis Curves - Column Type B3



Cross Section



Side Elevation

Front Elevation

Connector Data

Type: 3" common wire nails

Quantity: 100

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
47.0	5	2.25
70.0	5	3.50
110.0	5	5.50
152.0	5	7.50
206.0	5	10.50

FIGURE 8.C1.1 Details of Column Type C1

TABLE 8.C1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE C1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
C1-1	39.96	3.65	3.83	4.90	4.70	3.40
C1-2		3.49	3.66	4.68	4.49	3.38
C1-3		3.13	3.28	4.41	3.89	3.50
C1-4		3.45	3.26	4.76	4.36	3.74
C1-5		3.38	3.55	4.62	4.29	3.52
	Average	3.42	3.59	4.67	4.35	3.51
C1-6	56.67	2.75	2.83	3.15	4.03	2.45
C1-7		2.64	2.70	2.98	3.91	2.46
C1-8		2.38	2.44	2.69	3.51	2.39
C1-9		2.04	2.08	2.26	3.03	2.48
C1-10		2.13	2.18	2.41	3.11	2.27
	Average	2.39	2.44	2.70	3.52	2.41
C1-11	85.73	1.20	1.21	1.26	1.72	1.16
C1-12		1.07	1.08	1.11	1.52	1.14
C1-13		1.56	1.57	1.63	2.28	1.28
C1-14		1.09	1.10	1.13	1.56	1.15
C1-15		1.26	1.28	1.33	1.82	1.27
		1.24	1.25	1.29	1.78	1.20
C1-16	116.25	0.77	0.77	0.80	1.08	0.64
C1-17		0.83	0.83	0.85	1.15	0.73
C1-18		0.80	0.80	0.82	1.11	0.64
C1-19		0.82	0.82	0.84	1.14	0.81
C1-20		0.64	0.64	0.66	0.88	0.64
	Average	0.77	0.77	0.80	1.07	0.69
C1-21	155.48	0.41	0.41	0.42	0.52	0.40
C1-22		0.43	0.43	0.43	0.54	0.42
C1-23		0.41	0.41	0.42	0.52	0.38
C1-24		0.44	0.44	0.44	0.56	0.43
C1-25		0.47	0.47	0.48	0.60	0.51
	Average	0.43	0.43	0.44	0.55	0.43

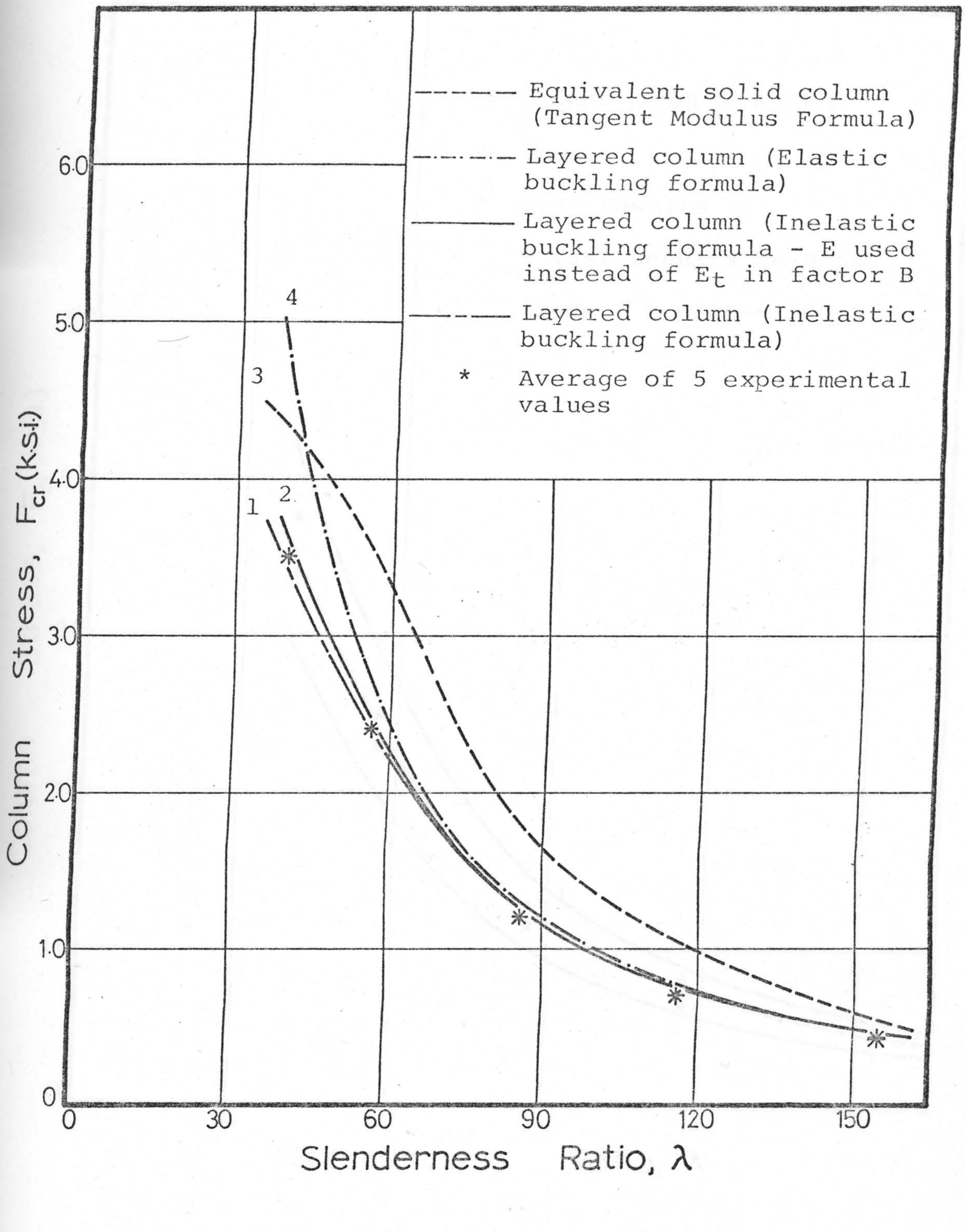


FIGURE 8.B3.2 Column Stress versus Slenderness Ratio Curves - Column Type B3

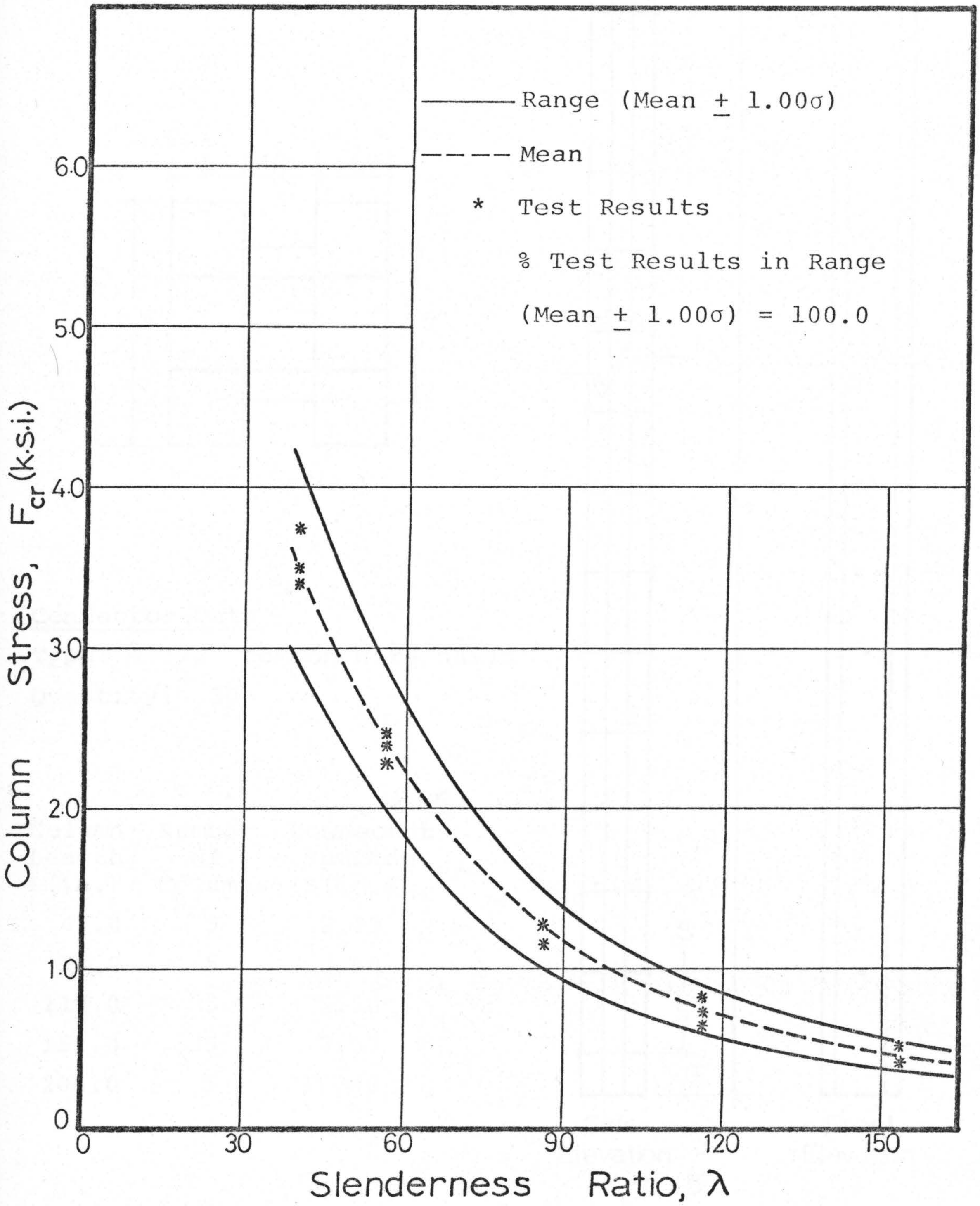
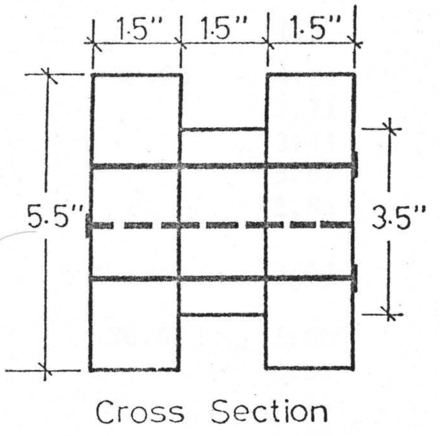
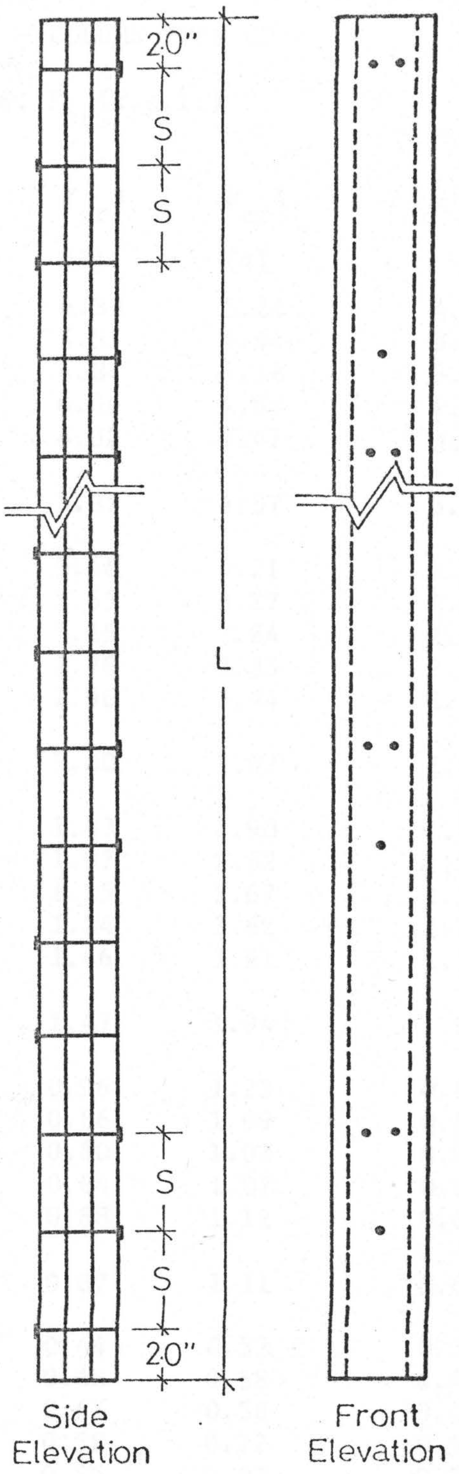


FIGURE 8.C1.3 Statistical Analysis Curves - Column Type C1





Connector Data

Type: 4 1/2" common wire nails

Quantity: 30

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
47.0	5	2.25
70.0	5	3.50
110.0	5	5.50
152.0	5	7.50
206.0	5	10.50

FIGURE 8.C2.1 Details of Column Type C2

TABLE 8.C2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE C2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
C2-1	39.96	4.31	4.53	6.39	5.21	4.11
C2-2		3.71	3.89	5.55	4.44	3.52
C2-3		3.41	3.57	4.84	4.18	3.57
C2-4		3.87	4.07	6.06	4.56	4.09
C2-5		2.84	2.97	4.02	3.47	3.57
	Average	3.63	3.81	5.37	4.37	3.77
C2-6	56.67	3.00	3.12	3.48	4.21	2.55
C2-7		2.32	2.41	2.63	3.27	2.20
C2-8		2.74	2.85	3.15	3.84	2.59
C2-9		2.40	2.50	2.76	3.35	2.58
C2-10		2.52	2.63	2.96	3.44	2.29
	Average	2.60	2.70	3.00	3.62	2.44
C2-11	85.73	1.37	1.38	1.43	1.90	1.28
C2-12		1.86	1.88	1.97	2.62	1.29
C2-13		1.21	1.22	1.25	1.67	1.34
C2-14		1.18	1.19	1.24	1.62	1.17
C2-15		1.38	1.39	1.46	1.91	1.33
	Average	1.40	1.41	1.47	1.94	1.28
C2-16	116.25	0.94	0.94	0.96	1.23	0.66
C2-17		0.84	0.84	0.86	1.09	0.67
C2-18		0.78	0.78	0.80	1.01	0.68
C2-19		0.82	0.82	0.84	1.07	0.68
C2-20		0.86	0.86	0.88	1.12	0.64
	Average	0.85	0.85	0.87	1.11	0.67
C2-21	155.48	0.43	0.43	0.44	0.53	0.31
C2-22		0.47	0.47	0.48	0.58	0.48
C2-23		0.47	0.47	0.47	0.58	0.40
C2-24		0.58	0.58	0.58	0.72	0.50
C2-25		0.49	0.49	0.50	0.61	0.51
	Average	0.49	0.49	0.49	0.60	0.44

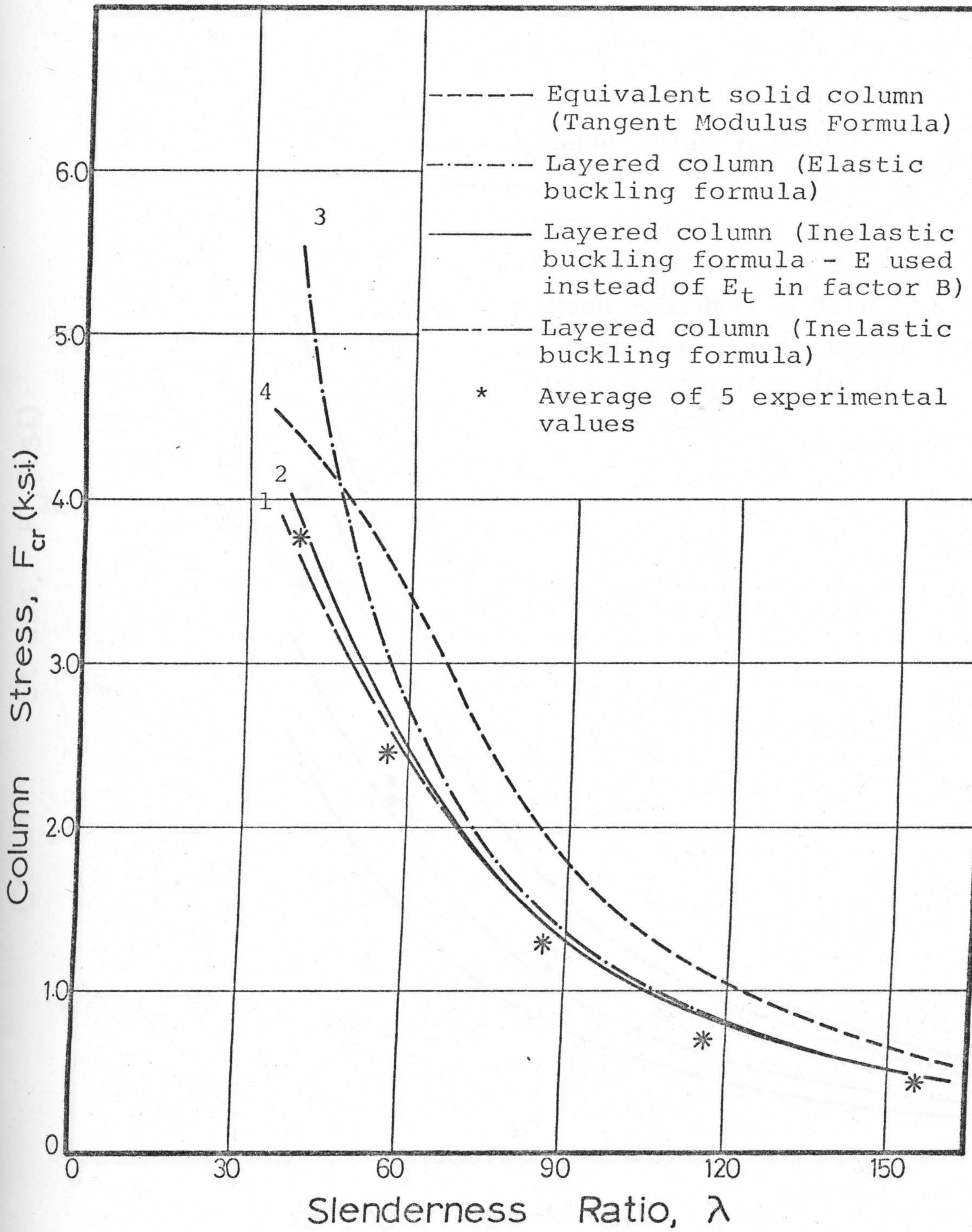


FIGURE 8.C2.2 Column Stress versus Slenderness Ratio Curves - Column Type C2

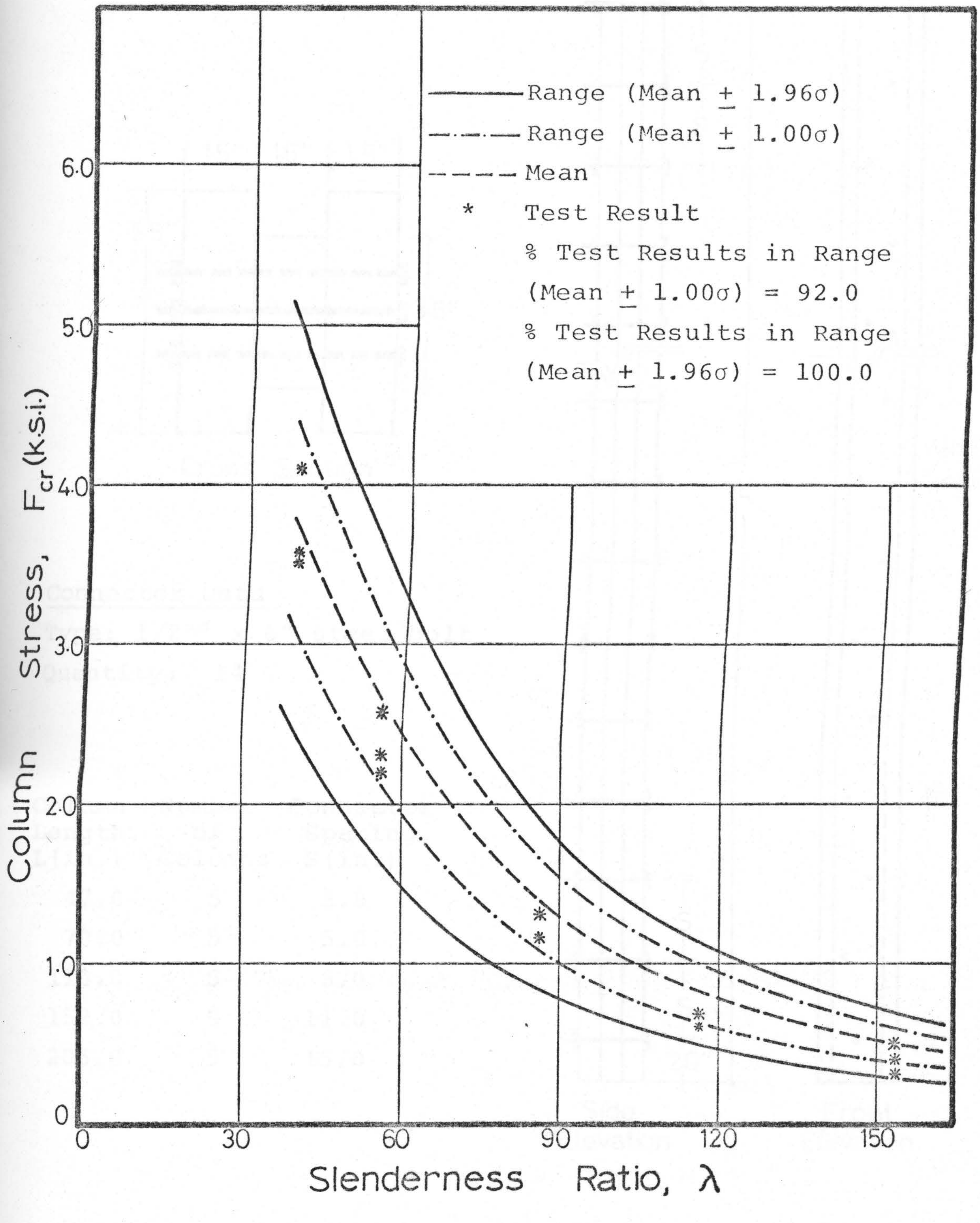
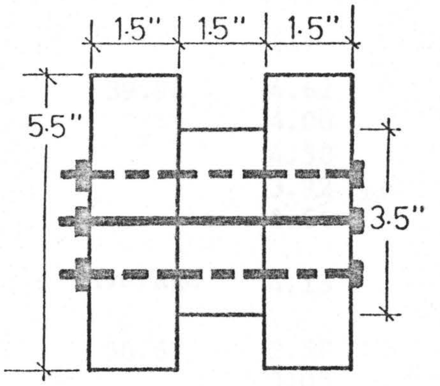


FIGURE 8.C2.3 Statistical Analysis Curves - Column Type C2



Connector Data

Type: 1/2"Ø x 6" steel bolt

Quantity: 14

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
47.0	5	3.0
70.0	5	5.0
110.0	5	8.0
152.0	5	11.0
206.0	5	15.0

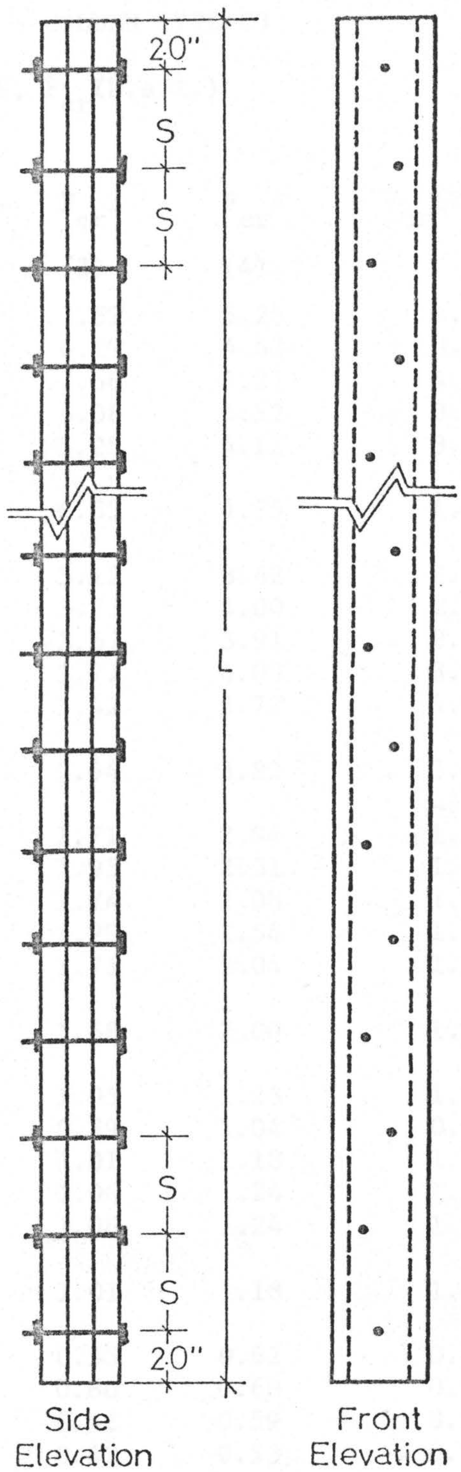


FIGURE 8.C3.1 Details of Column Type C3

TABLE 8.C3.1 RESULTS OF COLUMN TESTS - COLUMN TYPE C3

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
C3-1	39.96	4.62	4.84	7.59	5.26	4.37
C3-2		4.00	4.19	6.19	4.62	3.80
C3-3		4.58	4.80	7.56	5.21	4.22
C3-4		3.92	4.11	6.08	4.52	3.66
C3-5		3.54	3.71	5.25	4.12	3.94
	Average	4.13	4.33	6.53	4.75	4.00
C3-6	56.67	2.58	2.76	3.12	3.42	2.68
C3-7		3.05	3.05	3.73	4.00	2.81
C3-8		2.99	3.19	3.67	3.91	2.72
C3-9		3.08	3.28	3.77	4.03	3.09
C3-10		2.82	3.01	3.42	3.72	4.02
	Average	2.90	3.09	3.54	3.82	3.06
C3-11	85.73	1.63	1.64	1.71	2.04	1.57
C3-12		1.84	1.86	1.95	2.31	1.72
C3-13		1.66	1.67	1.74	2.08	1.79
C3-14		1.23	1.24	1.29	1.54	1.83
C3-15		1.63	1.64	1.71	2.04	1.60
	Average	1.60	1.61	1.68	2.00	1.70
C3-16	116.25	1.02	1.03	1.05	1.23	1.06
C3-17		0.87	0.87	0.89	1.04	0.91
C3-18		0.98	0.98	1.01	1.18	1.39
C3-19		1.03	1.03	1.06	1.24	1.25
C3-20		1.03	1.03	1.06	1.24	1.35
	Average	0.99	0.99	1.01	1.18	1.19
C3-21	155.48	0.54	0.54	0.55	0.62	0.42
C3-22		0.59	0.59	0.60	0.68	0.54
C3-23		0.51	0.51	0.52	0.59	0.41
C3-24		0.46	0.46	0.47	0.53	0.41
C3-25		0.50	0.50	0.51	0.57	0.39
	Average	0.52	0.52	0.53	0.60	0.43

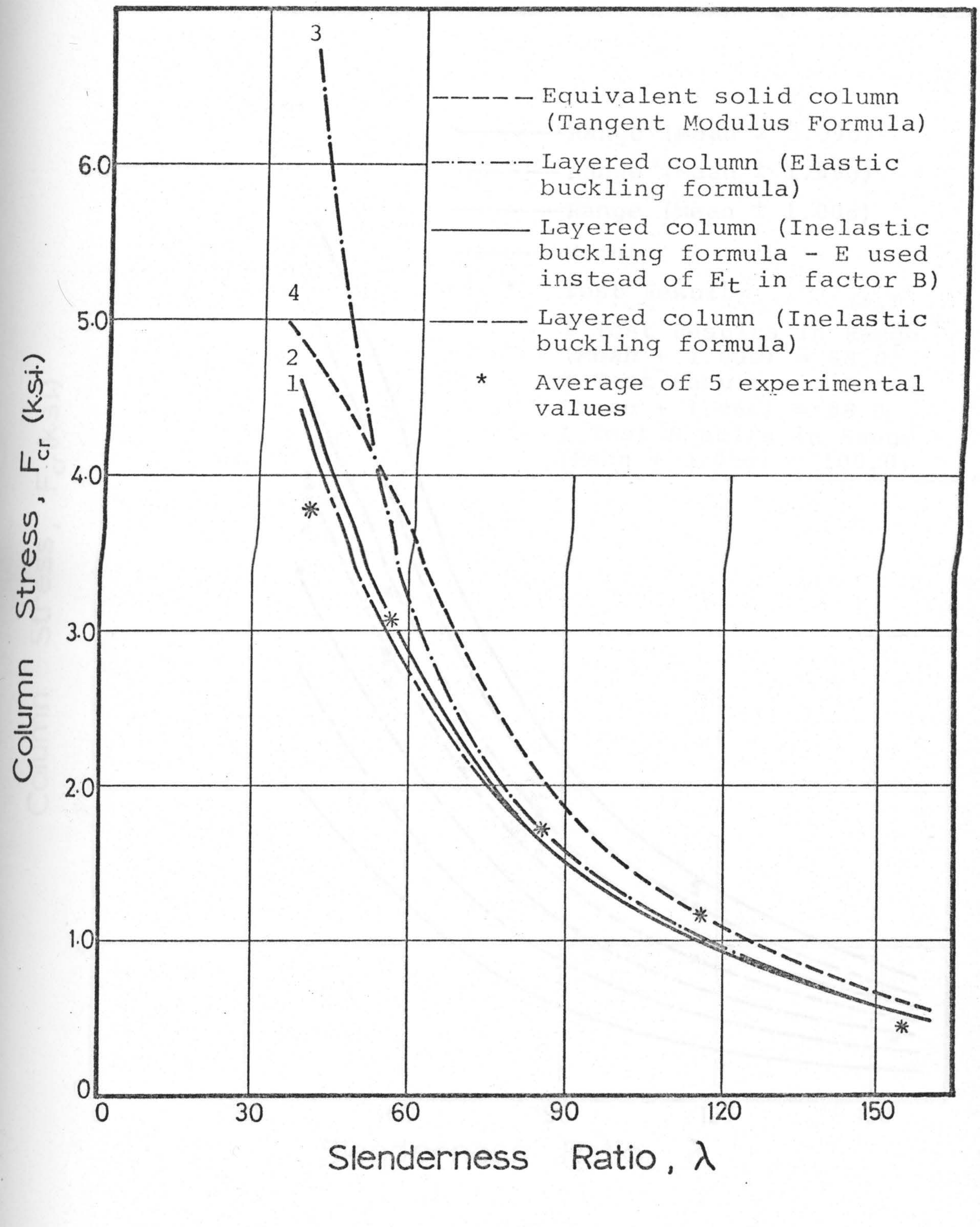


FIGURE 8.C3.2 Column Stress versus Slenderness Ratio Curves - Column Type C3

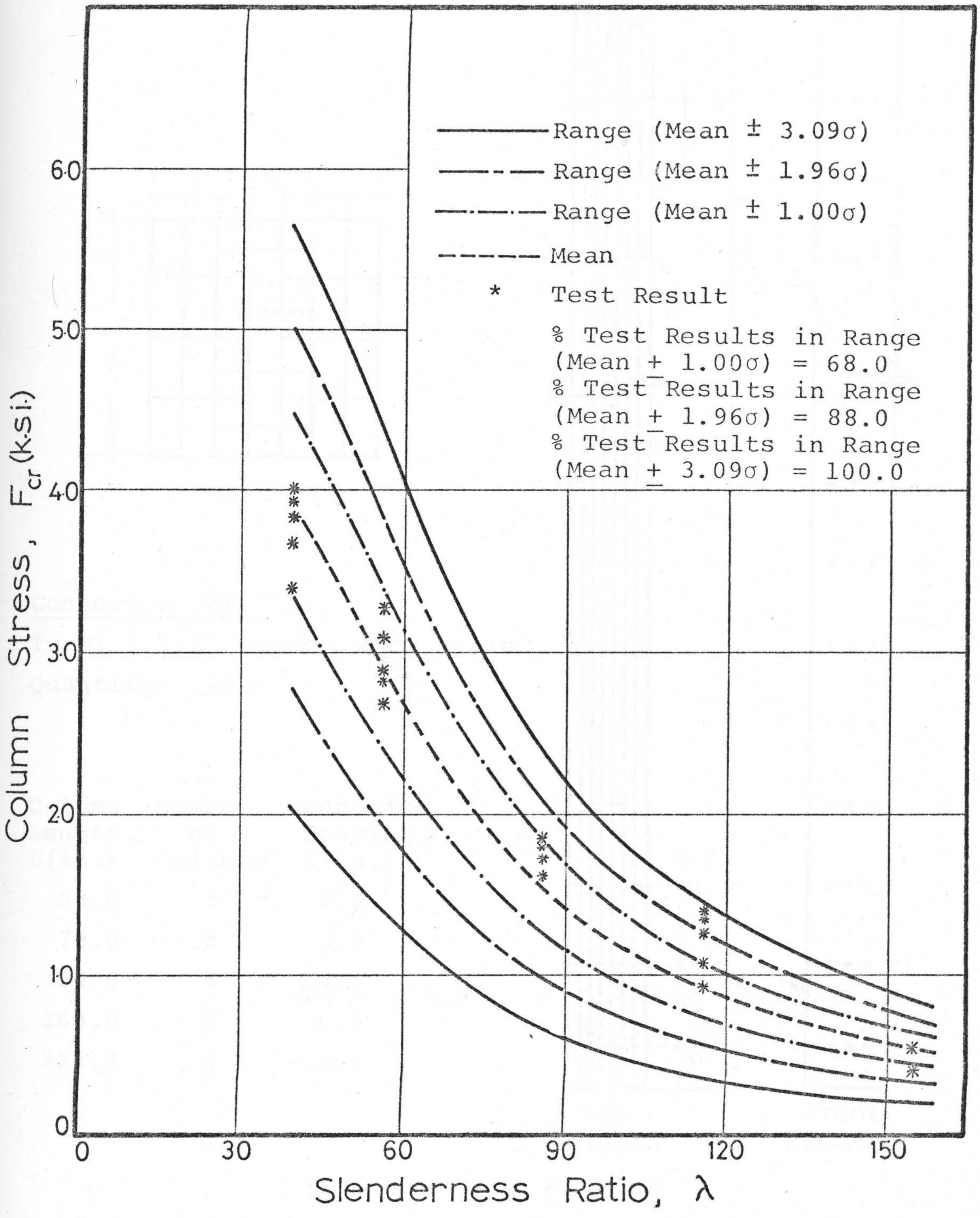
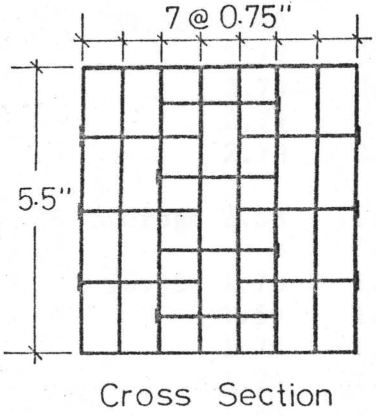
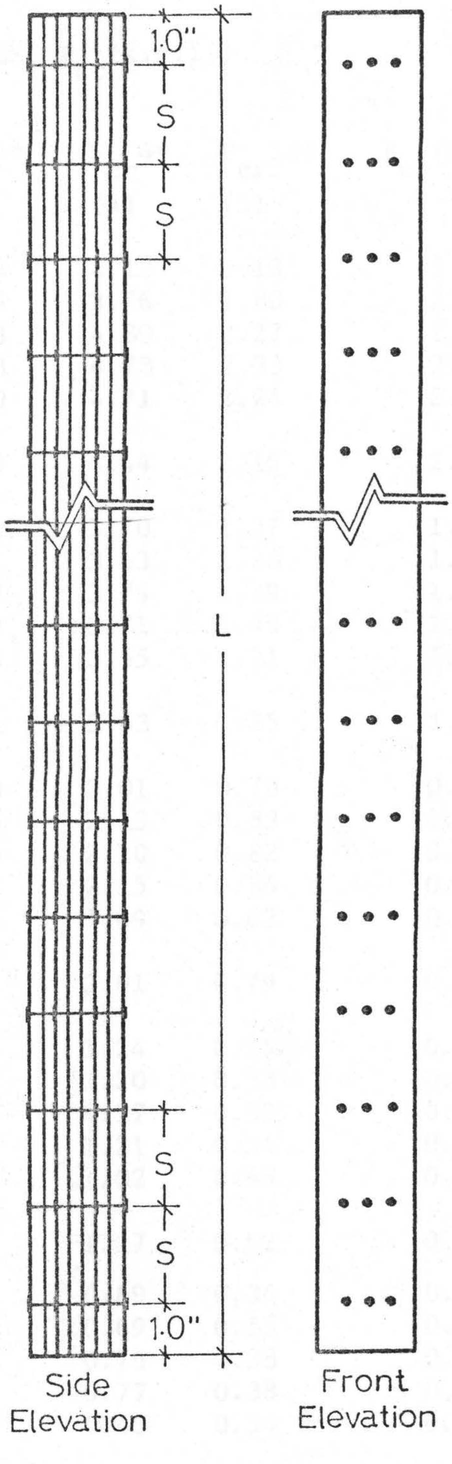


FIGURE 8.C3.3 Statistical Analysis Curves - Column Type C3





Connector Data

Type: 2 1/2" common wire nails

Quantity: 260

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
52.0	5	2.0
78.0	5	3.0
122.0	5	4.5
168.0	5	6.5
222.0	5	8.5

FIGURE 8.D.1 Details of Column Type D

TABLE 8.D.1 RESULTS OF COLUMN TESTS - COLUMN TYPE D

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)					
		$F_{cr}1^*$ (1)	$F_{cr}2^*$ (2)	$F_{cr}3^*$ (3)	$F_{cr}4^*$ (4)	$F_{cr}5^{**}$ (5)	F_{cr}(Exp.)
D-1	39.58	2.52	2.66	3.08	4.12	2.13	2.33
D-2		2.17	2.28	2.56	3.76	1.80	2.47
D-3		2.74	2.87	3.23	4.80	2.27	2.45
D-4		2.79	2.94	3.33	4.78	2.33	2.14
D-5		2.70	2.84	3.20	4.71	2.24	2.25
	Average	2.58	2.72	3.08	4.44	2.15	2.33
D-6	56.73	1.70	1.74	1.84	3.70	1.37	1.50
D-7		1.59	1.62	1.71	3.43	1.28	1.35
D-8		1.73	1.78	1.87	3.79	1.39	1.58
D-9		1.74	1.78	1.89	3.71	1.40	1.28
D-10		1.63	1.67	1.76	3.55	1.31	1.30
	Average	1.68	1.72	1.81	3.63	1.35	1.40
D-11	85.75	0.97	0.98	1.00	2.01	0.78	0.91
D-12		1.02	1.03	1.06	2.13	0.83	0.84
D-13		1.01	1.02	1.05	2.10	0.82	0.75
D-14		1.03	1.04	1.07	2.15	0.84	0.80
D-15		0.83	0.84	0.86	1.69	0.67	0.70
	Average	0.97	0.98	1.01	2.01	0.79	0.80
D-16	116.10	0.67	0.67	0.68	1.24	0.55	0.54
D-17		0.65	0.65	0.66	1.20	0.53	0.51
D-18		0.64	0.64	0.65	1.17	0.52	0.57
D-19		0.65	0.66	0.67	1.21	0.54	0.62
D-20		0.56	0.56	0.57	1.02	0.46	0.57
	Average	0.63	0.64	0.65	1.17	0.52	0.56
D-21	151.72	0.42	0.42	0.42	0.69	0.35	0.31
D-22		0.41	0.42	0.42	0.69	0.35	0.30
D-23		0.45	0.45	0.46	0.76	0.38	0.29
D-24		0.46	0.46	0.46	0.77	0.38	0.45
D-25		0.41	0.41	0.41	0.68	0.34	0.35
	Average	0.43	0.43	0.43	0.72	0.36	0.34

*m = 6 in Eqns. (8.1) to (8.3)

**m = 4 in Eqn. (8.2)

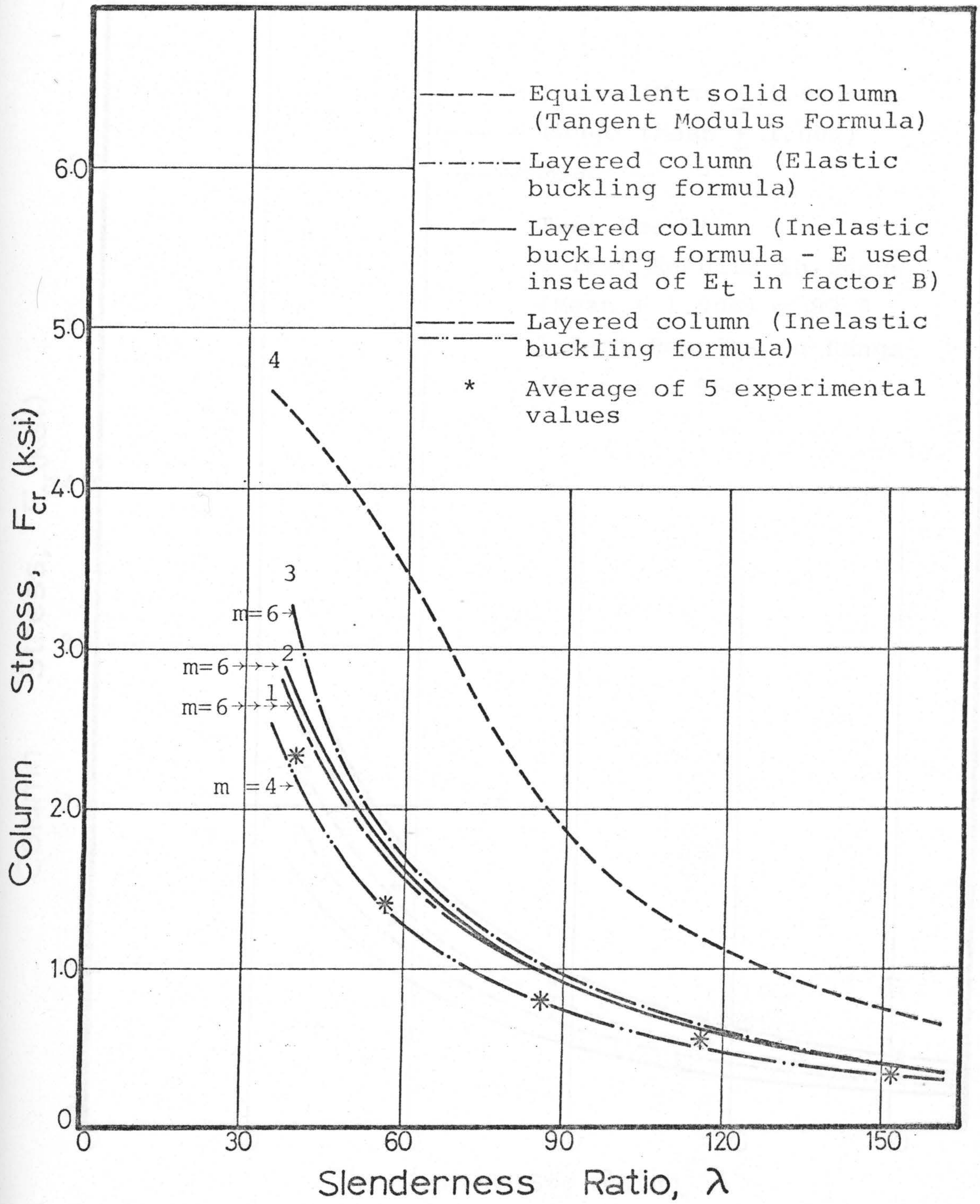


FIGURE 8.D.2 Column Stress versus Slenderness Ratio Curves - Column Type D

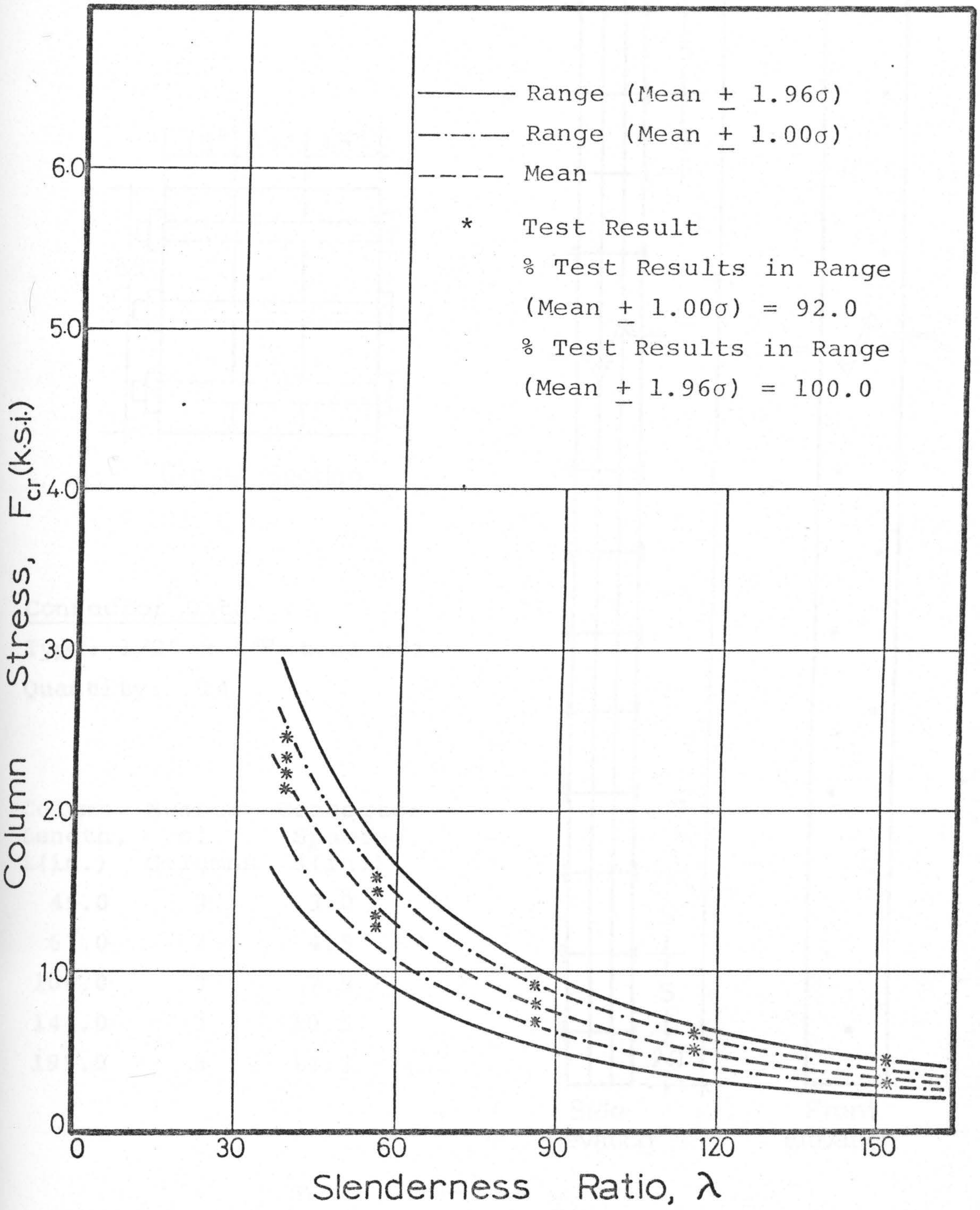
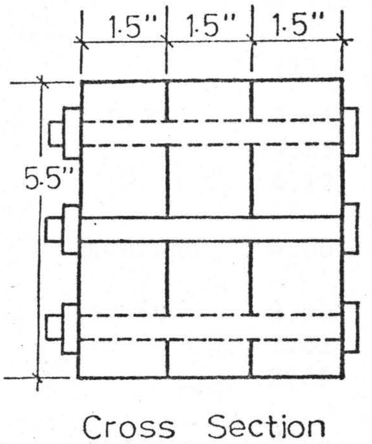
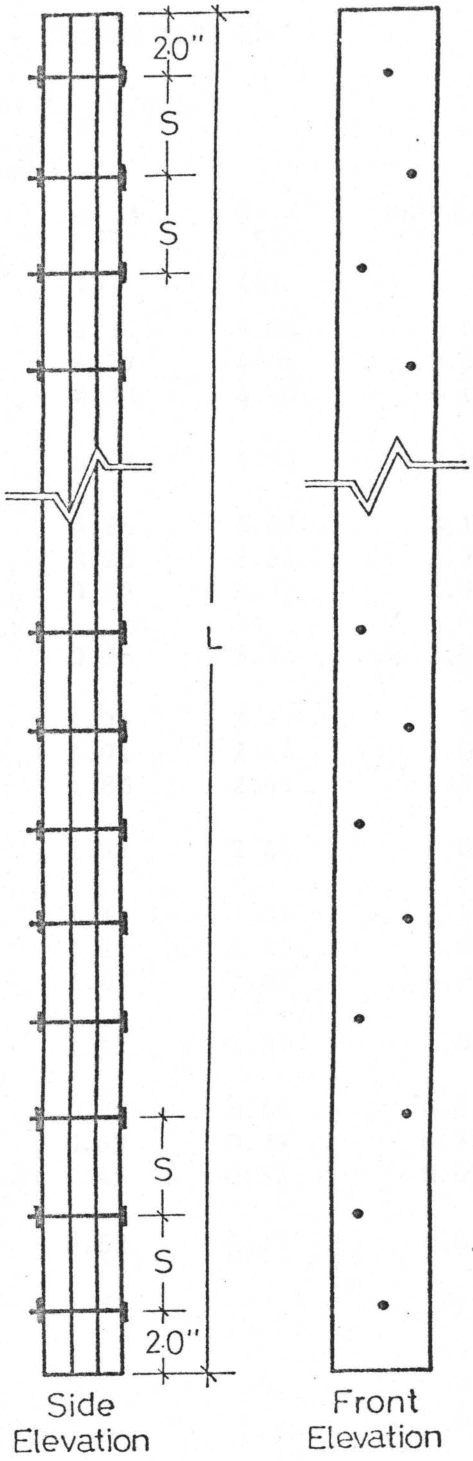


FIGURE 8.D.3 Statistical Analysis Curves - Column Type D





Connector Data

Type: 1/2" x 6"Ø steel bolt

Quantity: 14

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
45.0	3	3.0
67.0	3	4.5
104.0	3	7.5
144.0	3	10.5
195.0	3	14.5

FIGURE 8.E1.1 Details of Column Type E1

TABLE 8.E1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE E1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)				
		F_{cr}^1 (1)	F_{cr}^2 (2)	F_{cr}^3 (3)	F_{cr}^4 (4)	F_{cr}(Exp.)
E1-1	40.77	3.64	3.81	6.36	4.08	3.69
E1-2		4.25	4.44	6.77	4.86	3.97
E1-3		4.12	4.32	6.73	4.69	4.09
	Average	4.00	4.19	6.62	4.54	3.92
E1-4	57.69	3.93	4.05	4.86	4.87	3.17
E1-5		2.82	2.89	3.26	3.61	2.72
E1-6		2.66	2.73	3.15	3.37	2.94
	Average	3.14	3.22	3.76	3.95	2.94
E1-7	86.15	1.83	1.84	1.94	2.47	1.84
E1-8		1.80	1.81	1.91	2.44	2.08
E1-9		1.79	1.79	1.88	2.41	1.55
	Average	1.81	1.82	1.91	2.44	1.82
E1-10	116.92	1.11	1.11	1.14	1.34	1.10
E1-11		1.10	1.10	1.13	1.33	1.00
E1-12		1.05	1.05	1.07	1.27	0.98
	Average	1.09	1.09	1.11	1.31	1.03
E1-13	156.15	0.72	0.72	0.73	0.84	0.67
E1-14		0.68	0.68	0.69	0.79	0.70
E1-15		0.45	0.45	0.45	0.52	0.69
	Average	0.62	0.62	0.62	0.72	0.68

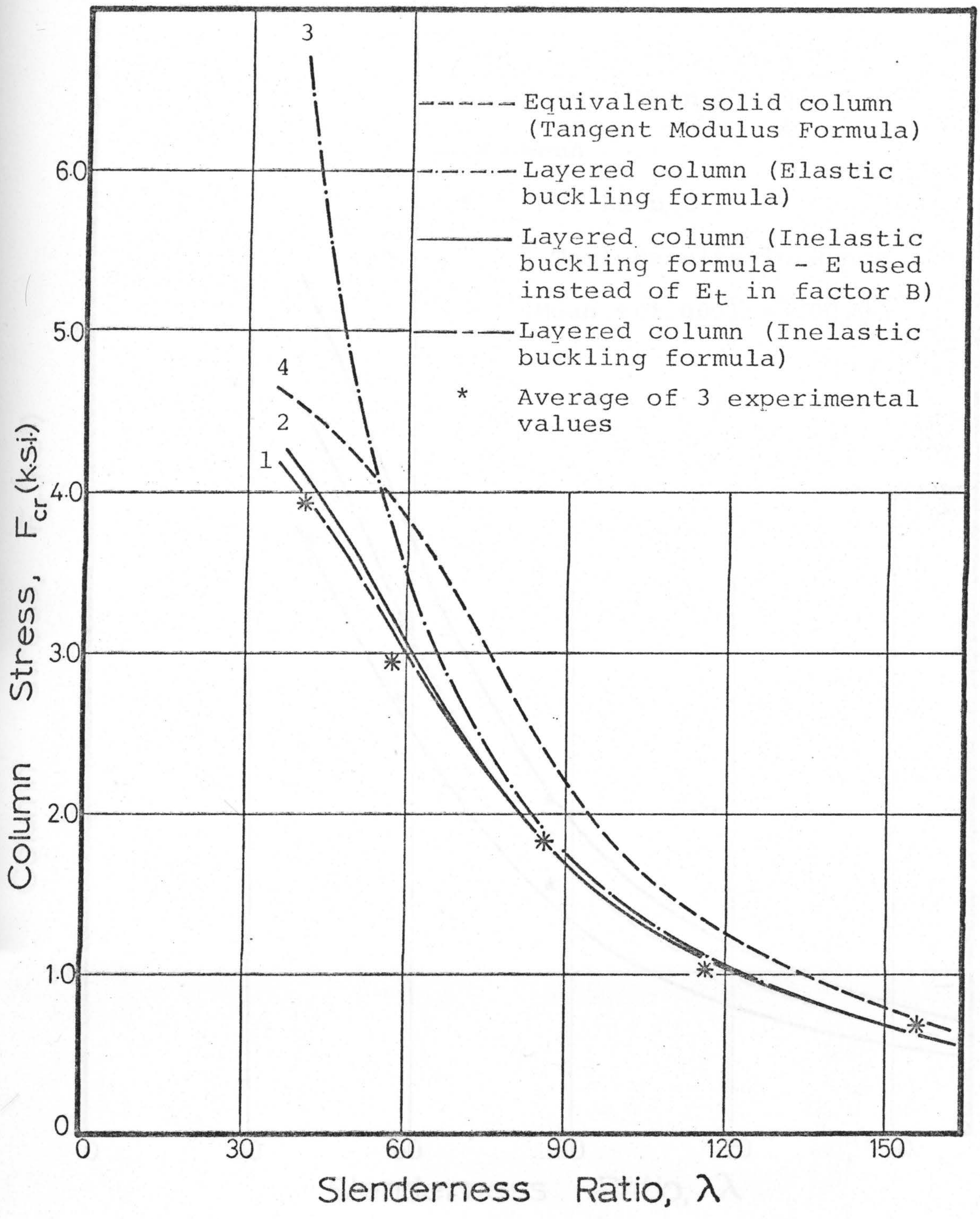


FIGURE 8.E1.2 Column Stress versus Slenderness Ratio Curves-

Column Type E1

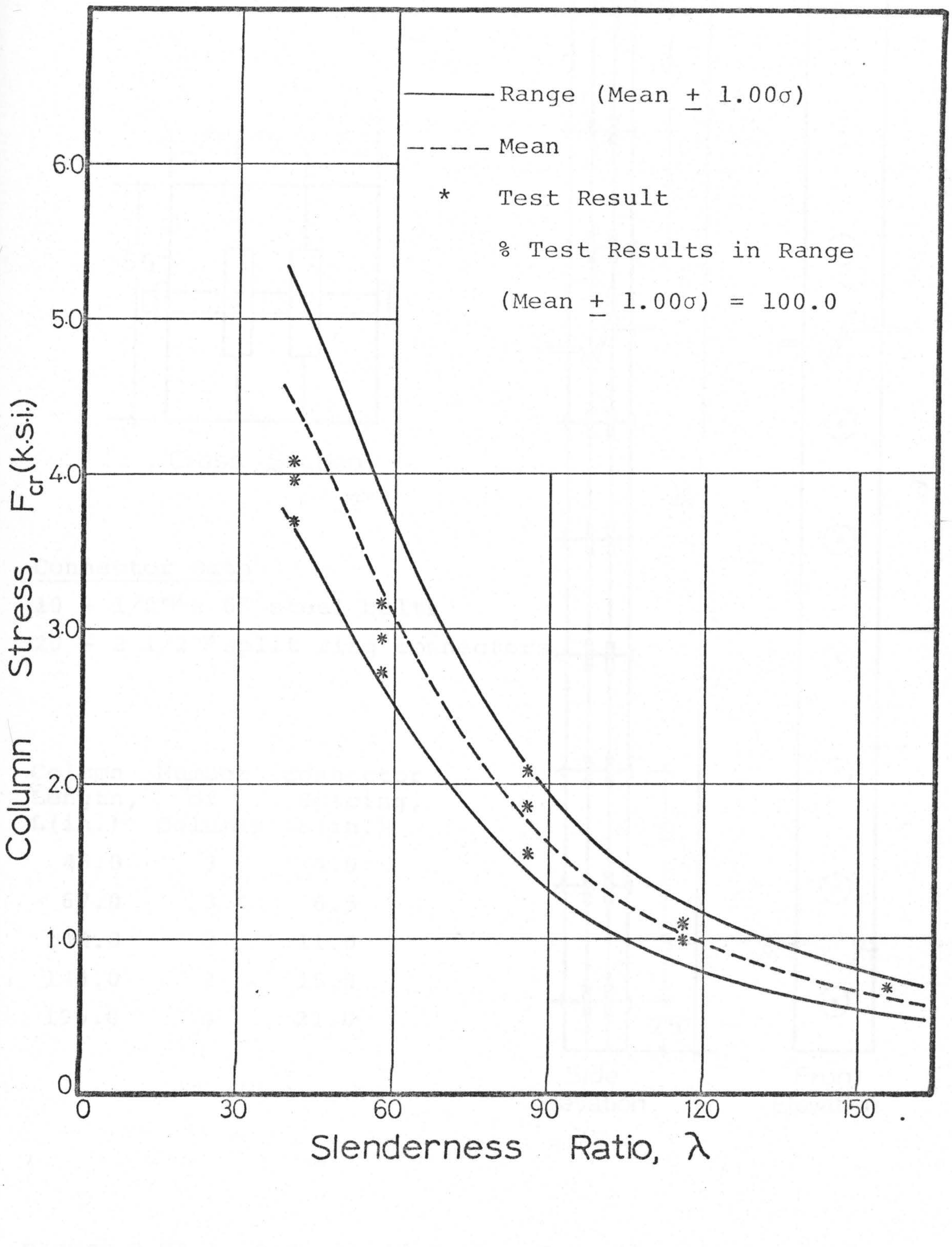
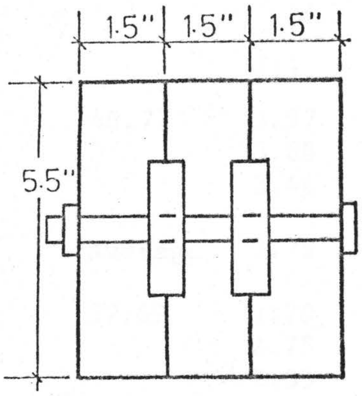


FIGURE 8.E1.3 Statistical Analysis Curves - Column Type E1



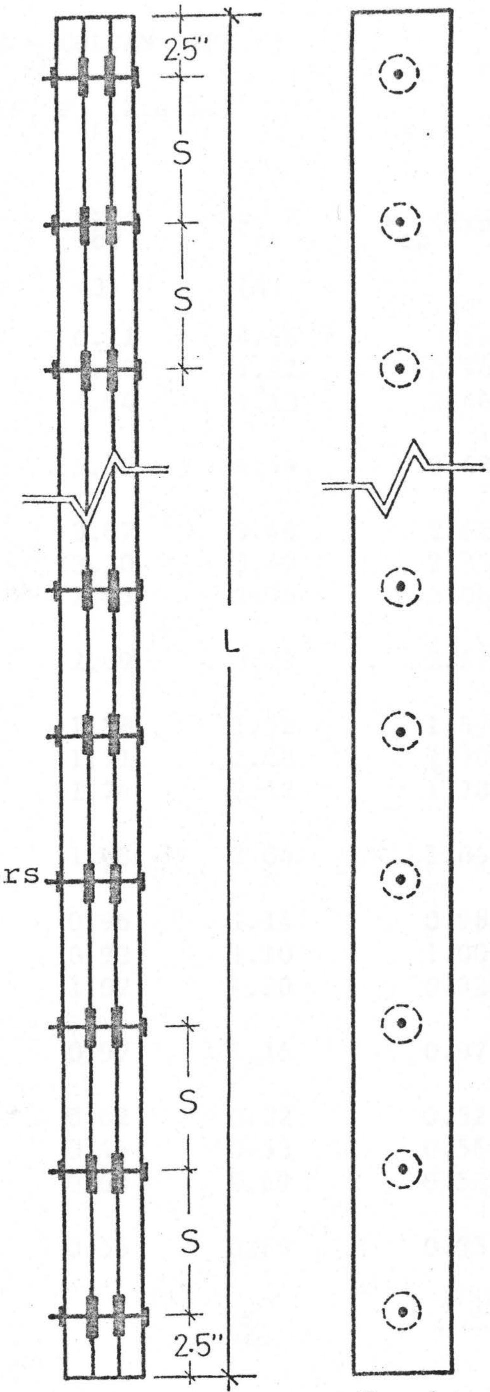
Cross Section

Connector Data

10 - 1/2"ø x 6" steel bolts

20 - 2 1/2"ø split ring connectors

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
45.0	3	4.0
67.0	3	6.5
104.0	3	11.0
144.0	3	15.0
195.0	3	21.0



Side Elevation

Front Elevation

FIGURE 8.E2.1 Details of Column Type E2

TABLE 8.E2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE E2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
E2-1	40.77	3.97	4.20	6.11	4.66	3.64
E2-2		3.85	4.07	5.92	4.52	3.94
E2-3		3.44	3.63	4.94	4.13	3.46
	Average	3.75	3.96	5.66	4.44	3.68
E2-4	57.69	2.70	2.76	3.07	3.46	2.61
E2-5		2.75	2.82	3.20	3.47	2.35
E2-6		2.33	2.39	2.69	2.95	3.06
	Average	2.59	2.66	2.99	3.29	2.67
E2-7	86.15	1.50	1.51	1.58	1.92	1.57
E2-8		1.63	1.64	1.71	2.08	1.70
E2-9		1.66	1.67	1.74	2.12	1.70
	Average	1.60	1.61	1.68	2.04	1.66
E2-10	116.92	0.94	0.94	0.96	1.14	0.98
E2-11		0.91	0.91	0.93	1.10	1.00
E2-12		1.00	1.00	1.02	1.20	0.92
	Average	0.95	0.95	0.97	1.15	0.97
E2-13	156.15	0.62	0.62	0.62	0.72	0.52
E2-14		0.46	0.46	0.46	0.53	0.56
E2-15		0.60	0.60	0.60	0.69	0.52
	Average	0.56	0.56	0.56	0.65	0.53

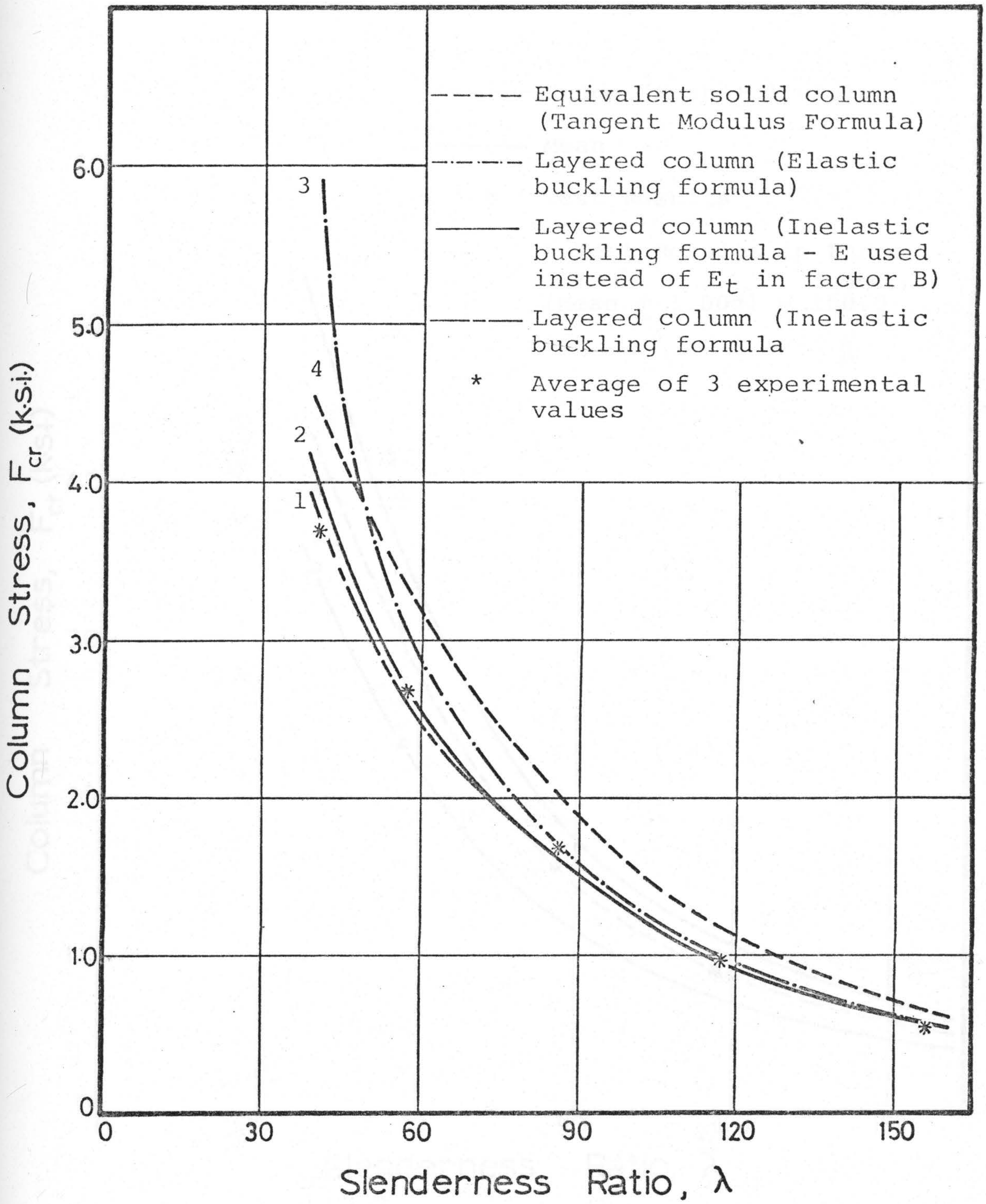


FIGURE 8.E2.2 Column Stress versus Slenderness Ratio Curves - Column Type E2

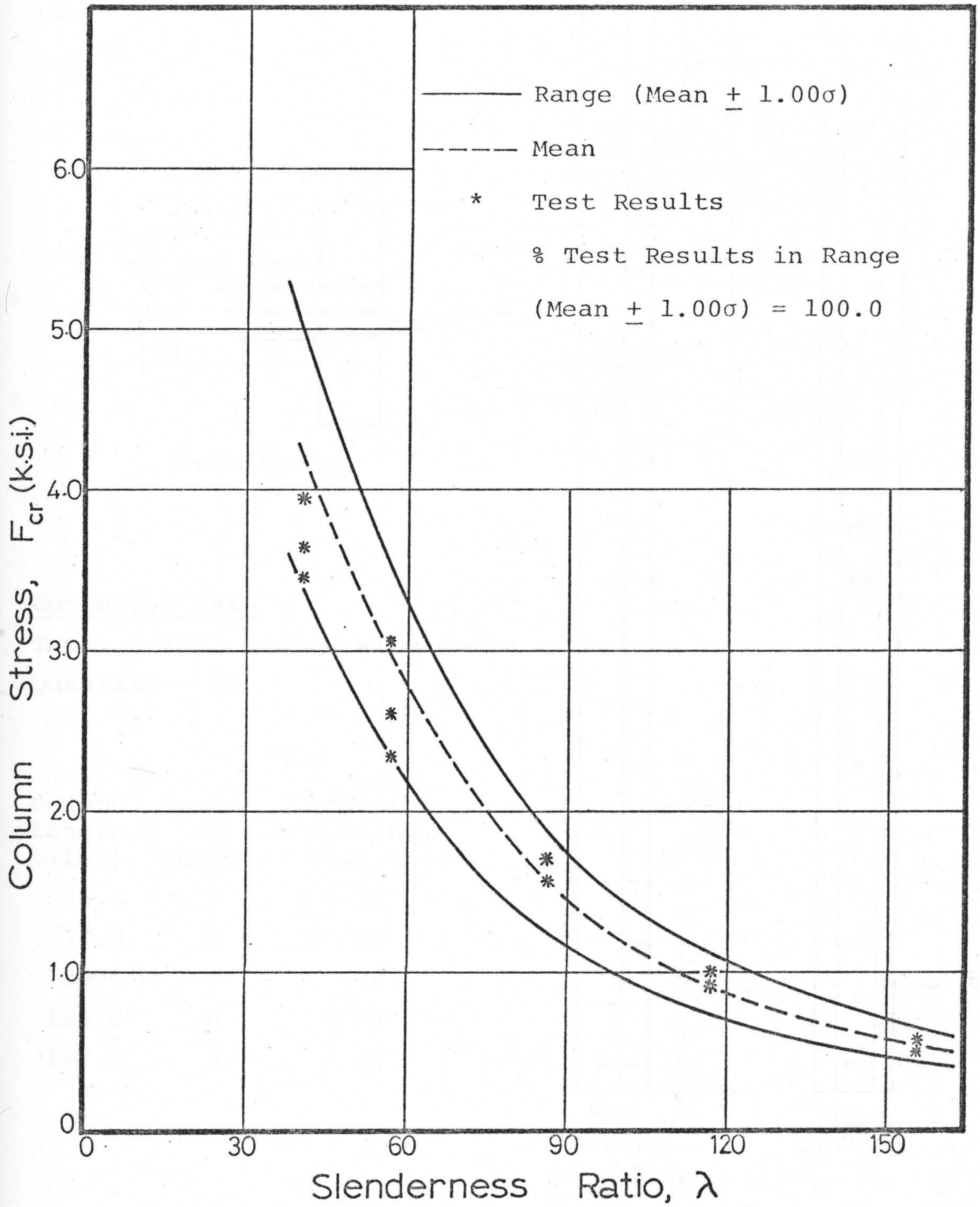
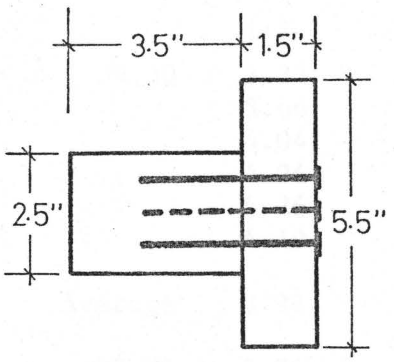
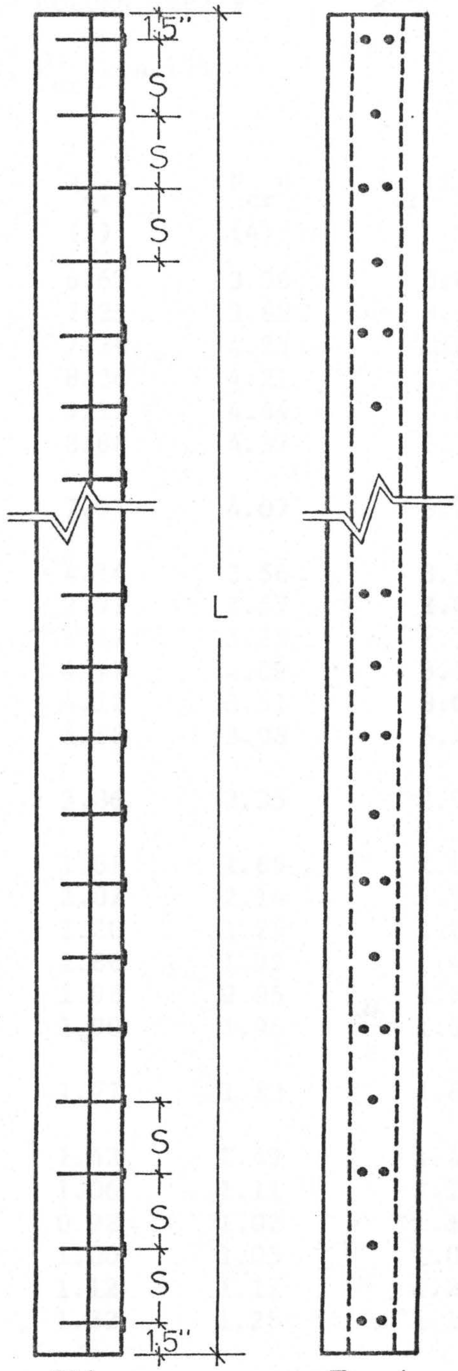


FIGURE 8.E2.3 Statistical Analysis Curves - Column Type E2



Cross Section



Side Elevation

Front Elevation

Connector Data

Type: 3 1/2" common wire nails

Quantity: 60

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
50.0	6	1.25
77.0	6	2.00
120.0	6	3.00
158.0	6	4.00
192.0	6	4.50

FIGURE 8.F.1 Details of Column Type F

TABLE 8F.1 RESULTS OF COLUMN TESTS - COLUMN TYPE F

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
F-1	39.30	3.22	3.28	6.67	3.36	3.65
F-2		3.66	3.72	7.21	3.82	3.53
F-3		4.04	4.11	7.38	4.25	3.90
F-4		4.04	4.11	8.36	4.21	4.07
F-5		4.26	4.34	9.17	4.44	3.81
F-6		4.19	4.26	8.68	4.37	3.71
	Average	3.90	3.97	7.91	4.07	3.75
F-7	57.59	3.26	3.30	4.14	3.56	3.11
F-8		2.35	2.38	2.91	2.57	2.64
F-9		2.96	3.00	3.67	3.25	2.76
F-10		3.74	3.79	4.77	4.09	3.10
F-11		3.23	3.27	4.12	3.51	3.02
F-12		2.82	2.85	3.56	3.08	3.14
	Average	3.06	3.10	3.86	3.35	2.96
F-13	86.72	1.50	1.52	1.61	1.65	1.80
F-14		1.93	1.95	2.07	2.14	1.98
F-15		1.14	1.15	1.20	1.25	1.86
F-16		1.74	1.76	1.86	1.92	1.49
F-17		1.86	1.87	1.98	2.05	1.82
F-18		1.77	1.79	1.90	1.96	1.98
	Average	1.66	1.67	1.77	1.83	1.82
F-19	112.47	1.37	1.37	1.42	1.49	1.40
F-20		1.02	1.02	1.06	1.11	1.19
F-21		0.94	0.94	0.98	1.02	1.16
F-22		0.97	0.97	1.00	1.05	1.09
F-23		1.09	1.09	1.12	1.17	1.21
F-24		1.18	1.18	1.22	1.28	1.13
	Average	1.10	1.10	1.13	1.19	1.20
F-25	135.50	0.75	0.75	0.76	0.81	0.75
F-26		0.64	0.64	0.65	0.70	0.78
F-27		0.83	0.83	0.85	0.91	0.90
F-28		0.63	0.63	0.64	0.69	0.78
F-29		0.74	0.74	0.74	0.80	0.76
F-30		0.85	0.85	0.87	0.93	0.91
	Average	0.74	0.74	0.75	0.81	0.81

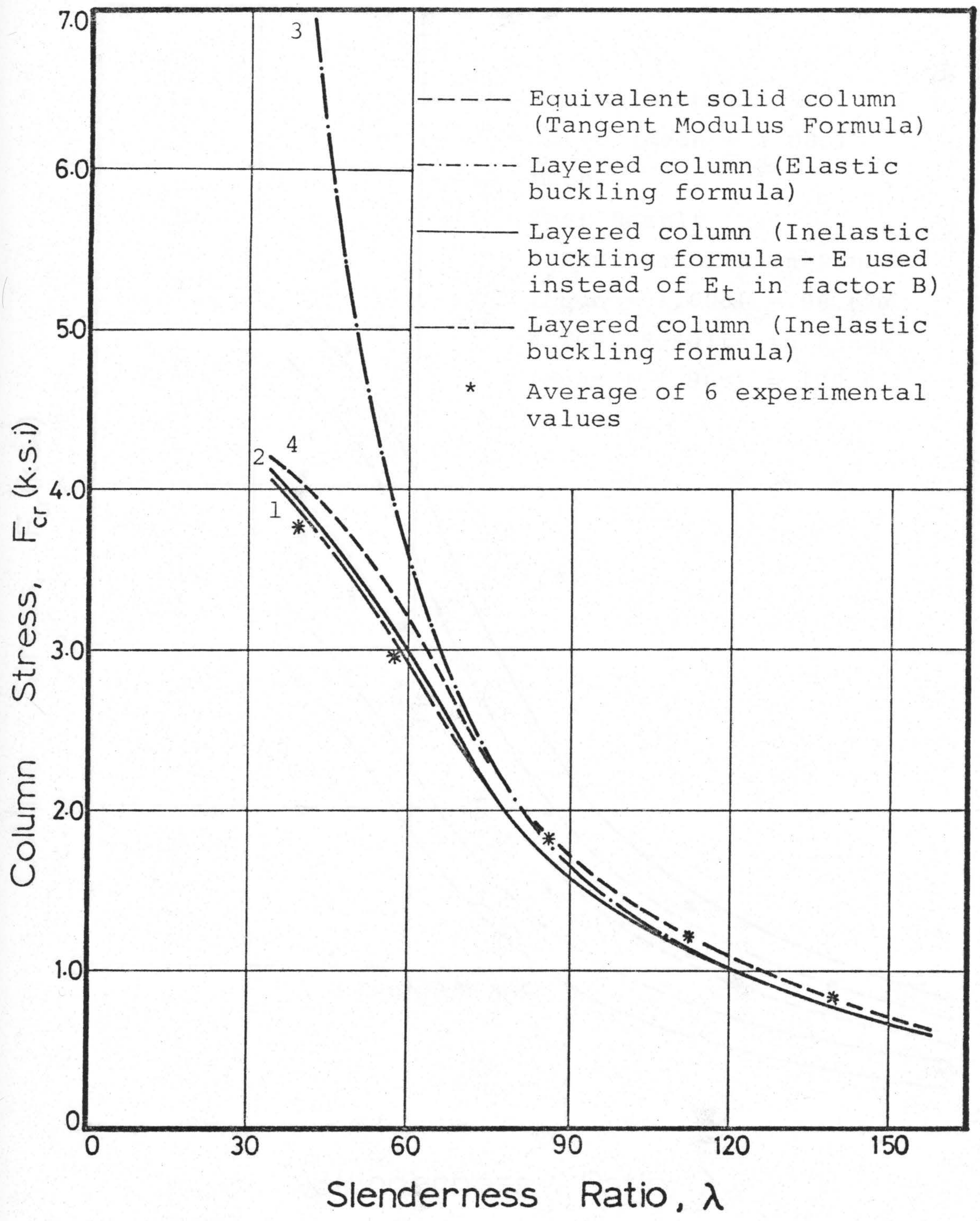


FIGURE 8.F.2 Column Stress versus Slenderness Ratio Curves - Column Type F

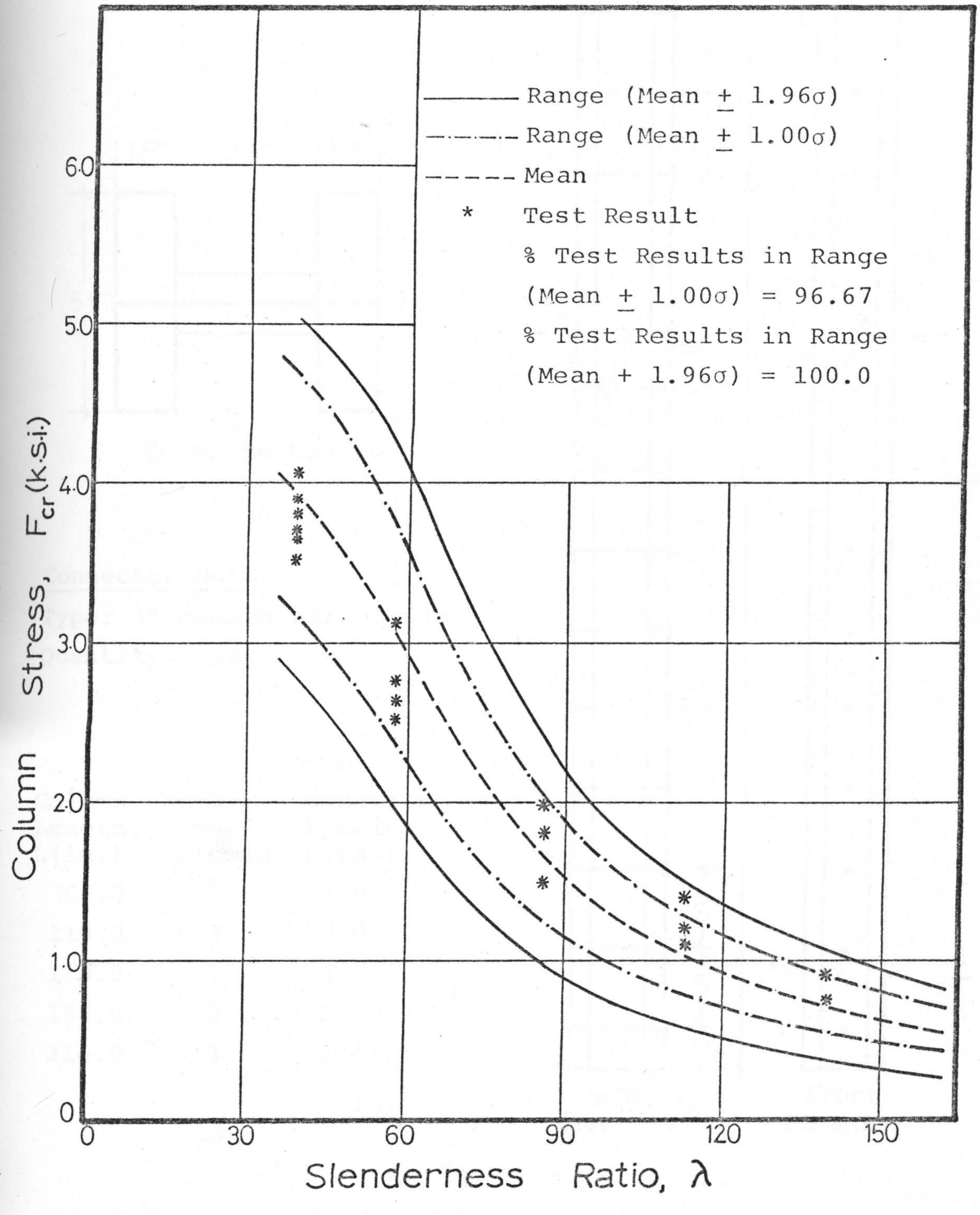
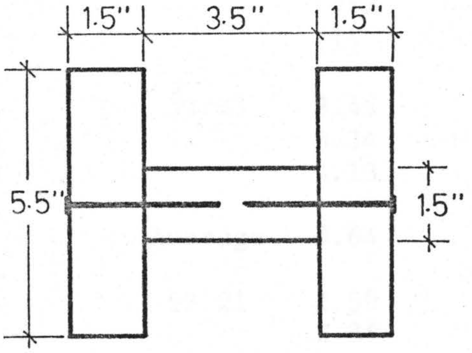
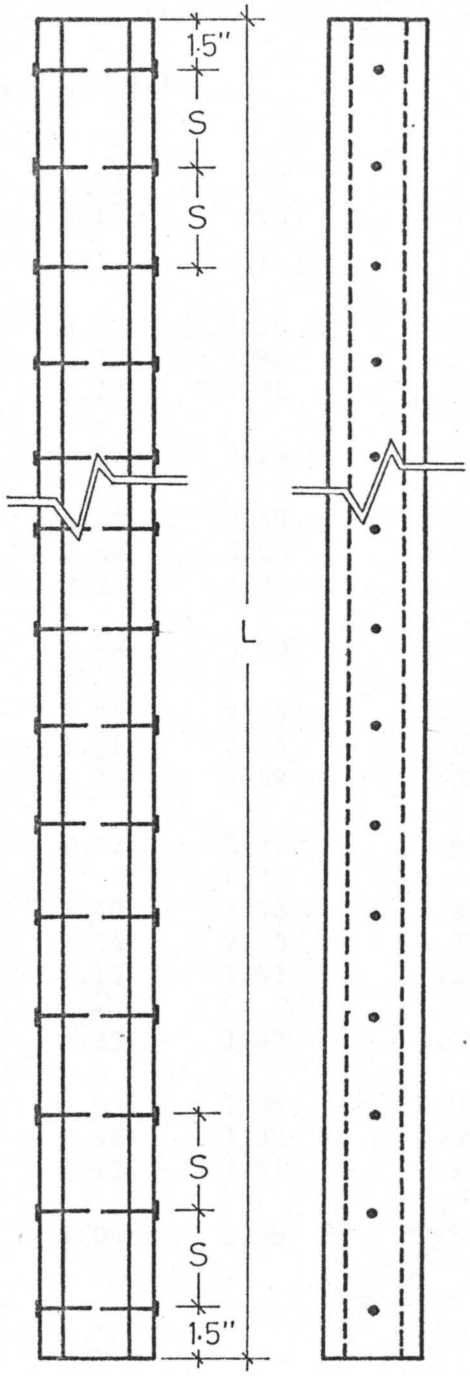


FIGURE 8.F.3 Statistical Analysis Curves - Column Type F



Cross Section



Side Elevation

Front Elevation

Connector Data

Type: 3" common wire nails

Quantity: 68

Column Length, L(in.)	Number of Columns	Connector Spacing, S(in.)
68.0	3	2.0
110.0	3	3.0
144.0	3	4.0
180.0	3	5.0
216.0	3	6.0

FIGURE 8.G.1 Details of Column Type G

TABLE 8.G.1 RESULTS OF COLUMN TESTS - COLUMN TYPE G

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)				
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	$F_{cr}4$ (4)	F_{cr}(Exp.)
G-1	33.63	3.45	3.64	6.18	3.98	3.82
G-2		3.34	3.53	5.39	3.96	3.51
G-3		4.13	4.37	7.23	4.81	3.87
	Average	3.64	3.85	6.27	4.25	3.73
G-4	52.21	2.59	2.67	3.03	3.69	2.36
G-5		2.28	2.35	2.69	3.20	2.65
G-6		2.75	2.83	3.23	3.91	2.32
	Average	2.54	2.62	2.98	3.60	2.44
G-7	67.26	1.87	1.90	2.04	2.75	1.54
G-8		1.74	1.77	1.91	2.55	1.89
G-9		1.96	1.99	2.13	2.89	1.52
	Average	1.86	1.89	2.02	2.73	1.65
G-10	83.19	1.38	1.39	1.40	1.98	1.25
G-11		1.51	1.53	1.54	2.18	1.39
G-12		1.13	1.14	1.13	1.61	1.12
	Average	1.34	1.35	1.35	1.92	1.25
G-13	99.12	0.99	0.99	1.03	1.36	1.00
G-14		0.95	0.95	0.98	1.30	0.89
G-15		1.09	1.09	1.13	1.51	0.91
	Average	1.01	1.01	1.04	1.39	0.93

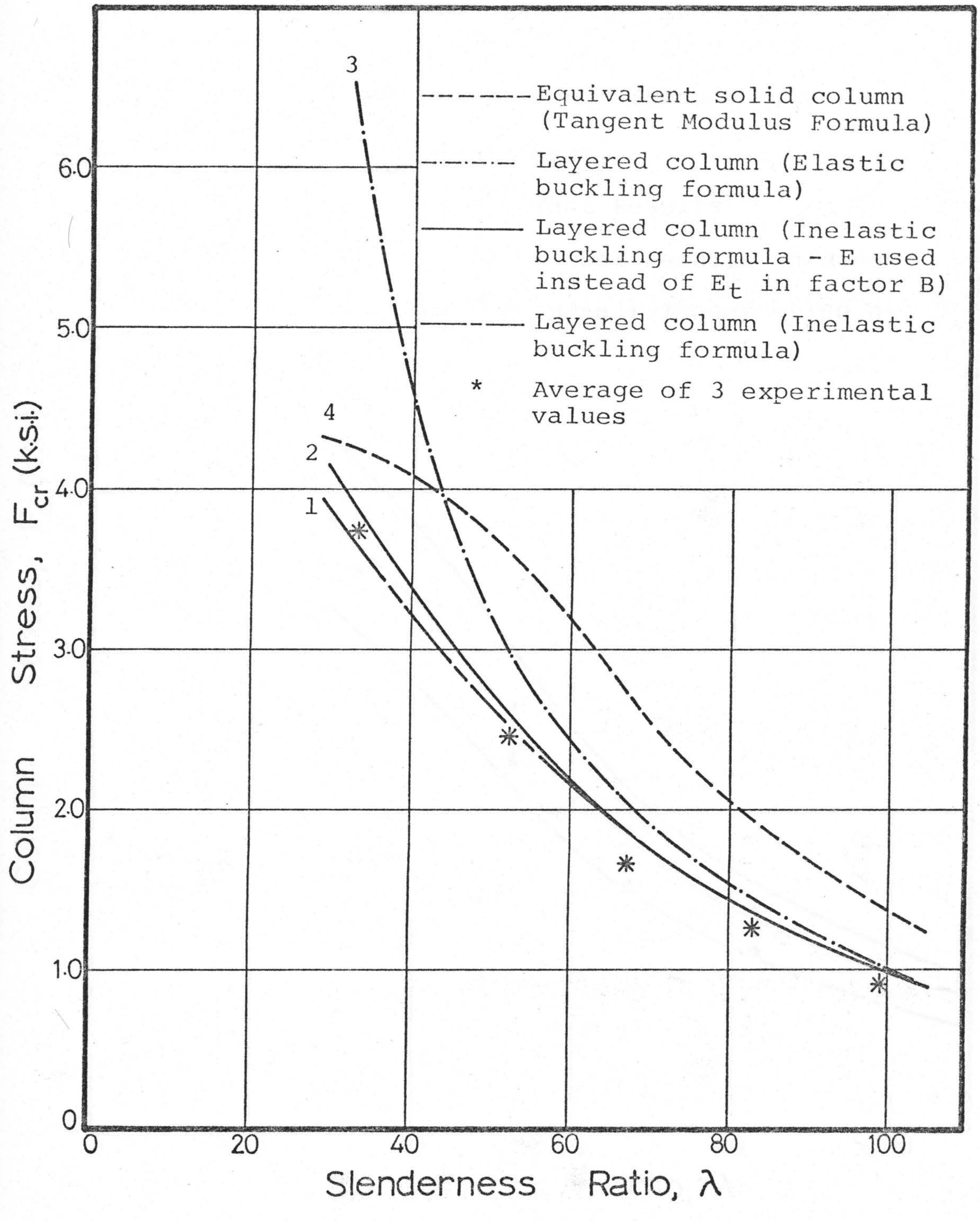


FIGURE 8.G.2 Column Stress versus Slenderness Ratio Curves - Column Type G

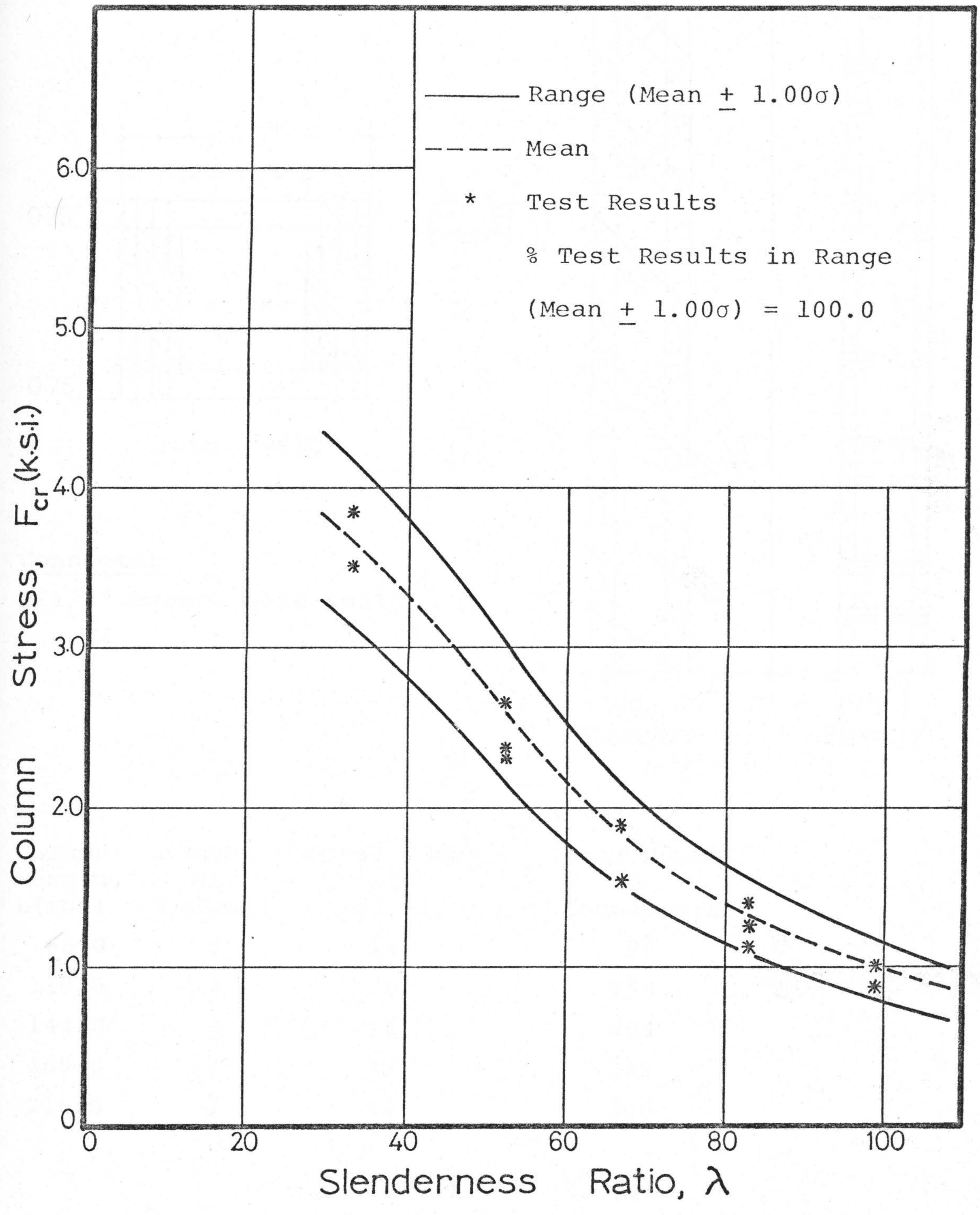
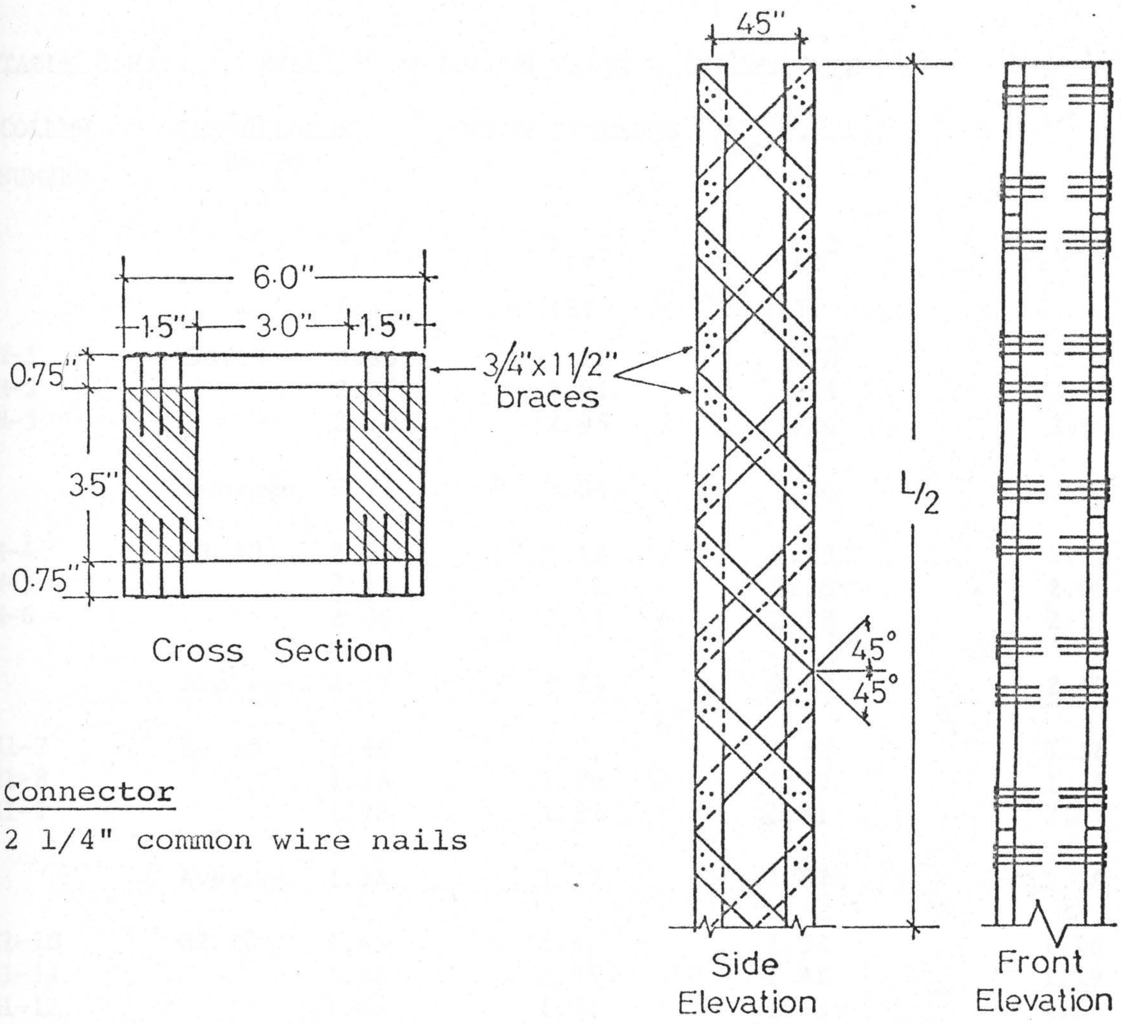


FIGURE 8.G.3 Statistical Analysis Curves - Column Type G



Connector

2 1/4" common wire nails

Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
68.0	3	16	96
110.0	3	26	156
144.0	3	34	204
180.0	3	42	252
216.0	3	50	300

FIGURE 8.H1.1 Details of Column Type H1

TABLE 8.H1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE H1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
H-1	33.19	3.00	3.28	3.70	2.99
H-2		2.66	2.91	3.25	3.11
H-3		2.70	2.95	3.26	2.69
	Average	2.79	3.04	3.40	2.93
H-4	51.53	2.04	2.10	2.25	2.20
H-5		2.42	2.51	2.76	2.09
H-6		2.05	2.11	2.28	2.11
	Average	2.17	2.24	2.43	2.13
H1-7	66.38	1.69	1.72	1.82	1.56
H1-8		1.76	1.79	1.89	1.54
H1-9		1.76	1.82	1.91	2.01
	Average	1.74	1.77	1.87	1.70
H1-10	82.10	1.45	1.46	1.52	1.15
H1-11		1.34	1.35	1.42	1.19
H1-12		1.31	1.32	1.37	1.54
	Average	1.37	1.38	1.44	1.29
H1-13	97.82	1.07	1.07	1.11	0.87
H1-14		1.12	1.12	1.16	1.19
H1-15		1.08	1.08	1.12	0.92
	Average	1.09	1.09	1.13	1.00

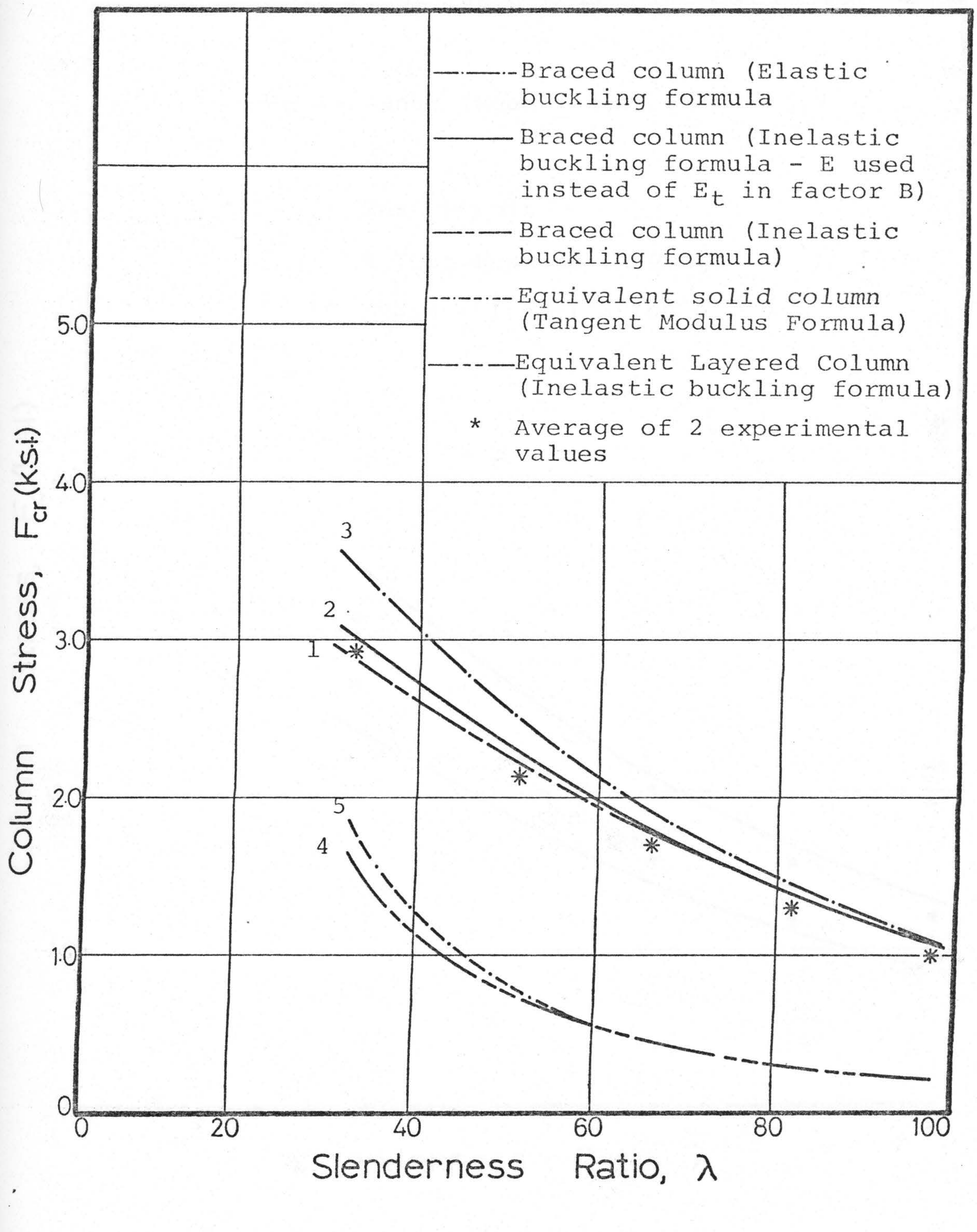


FIGURE 8.H1.2 Column Stress versus Slenderness Ratio Curves -

Column Type H1

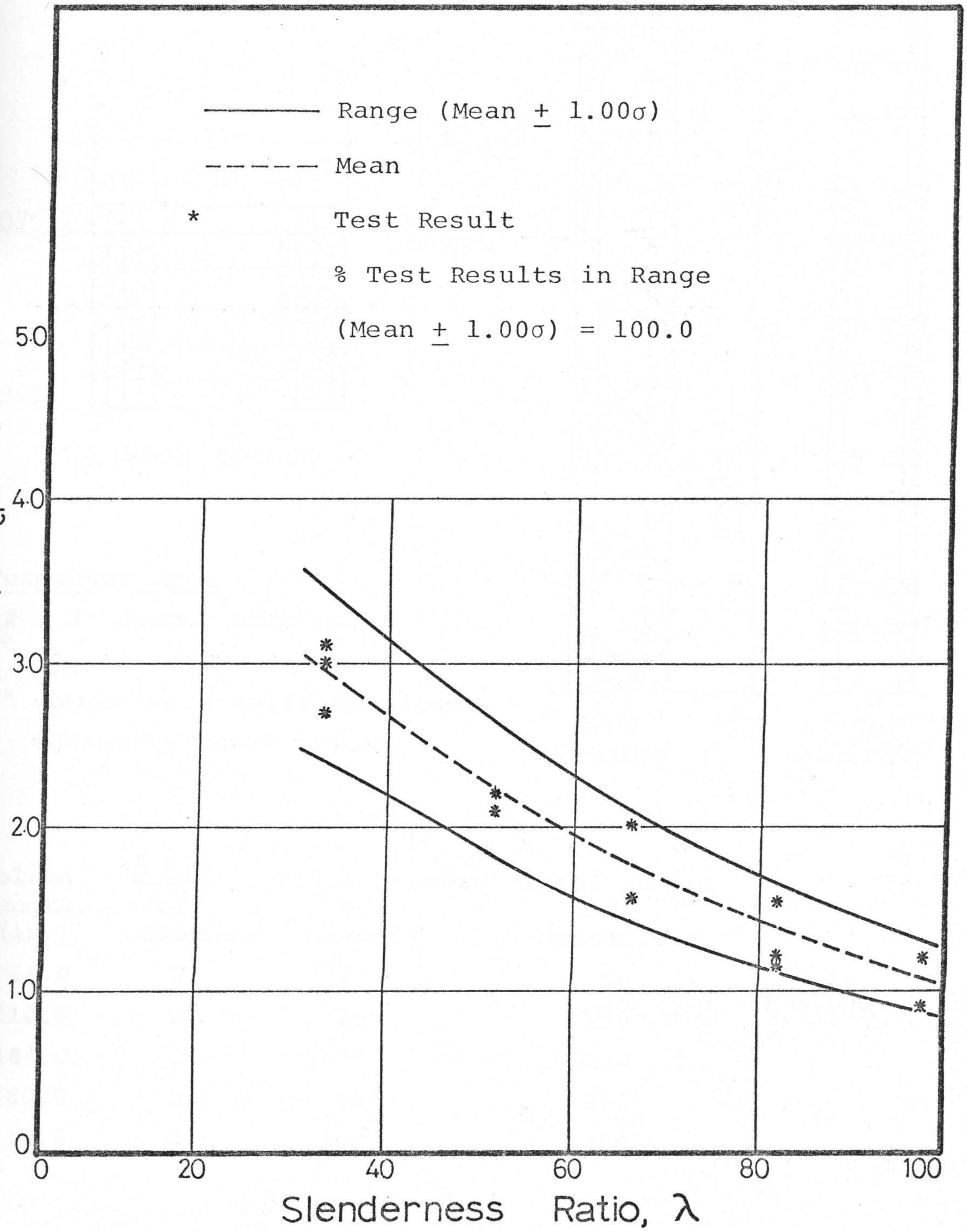
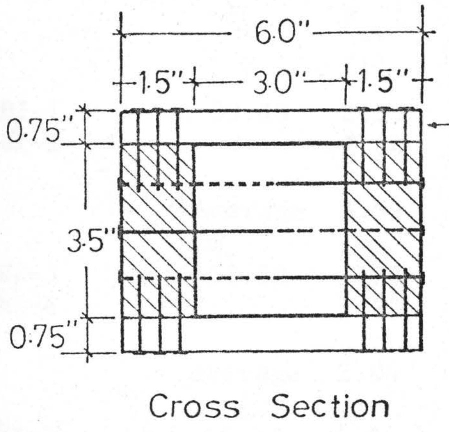
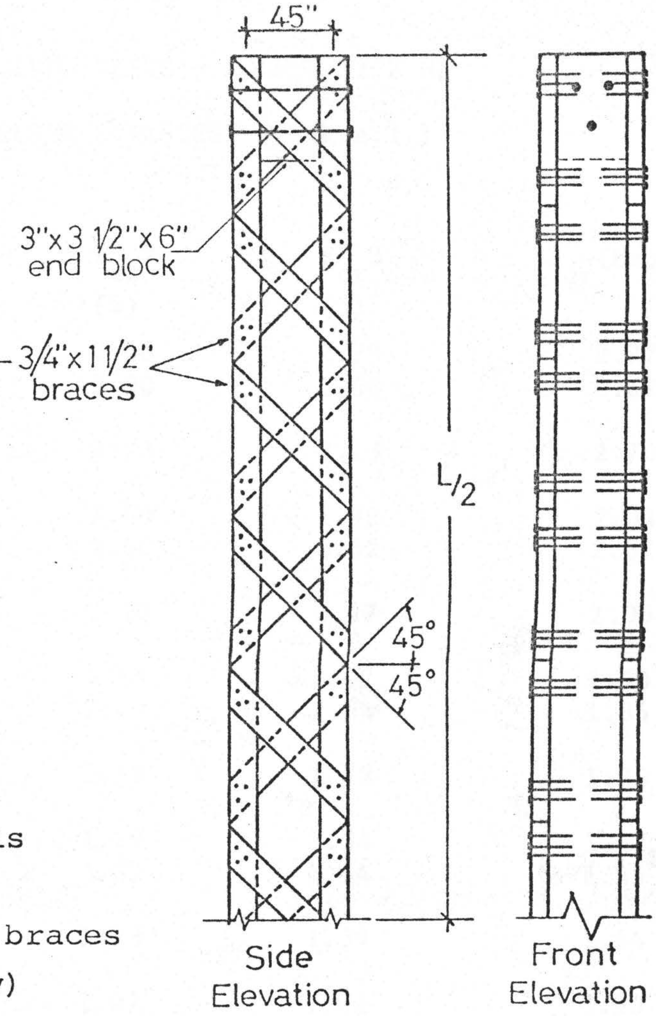


FIGURE 8.H1.3 Statistical Analysis Curves - Column Type H1





Connector Data

12 - 3" common wire nails
in end blocks

2" common wire nails in braces
(quantity shown below)

Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
68.0	2	16	96
110.0	2	26	156
144.0	2	34	204
180.0	2	42	252
216.0	2	50	300

FIGURE 8.H2.1 Details of Column Type H2

TABLE 8.H2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE H2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
H2-1	33.19	2.35	2.46	2.79	2.77
H2-2		2.84	3.00	3.48	2.62
	Average	3.59	2.73	3.13	2.70
H2-3	51.53	1.76	1.80	1.92	2.03
H2-4		2.33	2.40	2.62	2.07
	Average	2.04	2.10	2.27	2.05
H2-5	66.38	1.84	1.88	2.02	1.60
H2-6		1.46	1.48	1.54	1.86
	Average	1.65	1.68	1.78	1.73
H2-7	82.10	1.28	1.29	1.34	1.43
H2-8		1.36	1.37	1.44	1.45
	Average	1.32	1.33	1.39	1.44
H2-9	97.82	1.06	1.07	1.09	0.90
H2-10		1.06	1.08	1.11	1.17
	Average	1.06	1.07	1.10	1.03

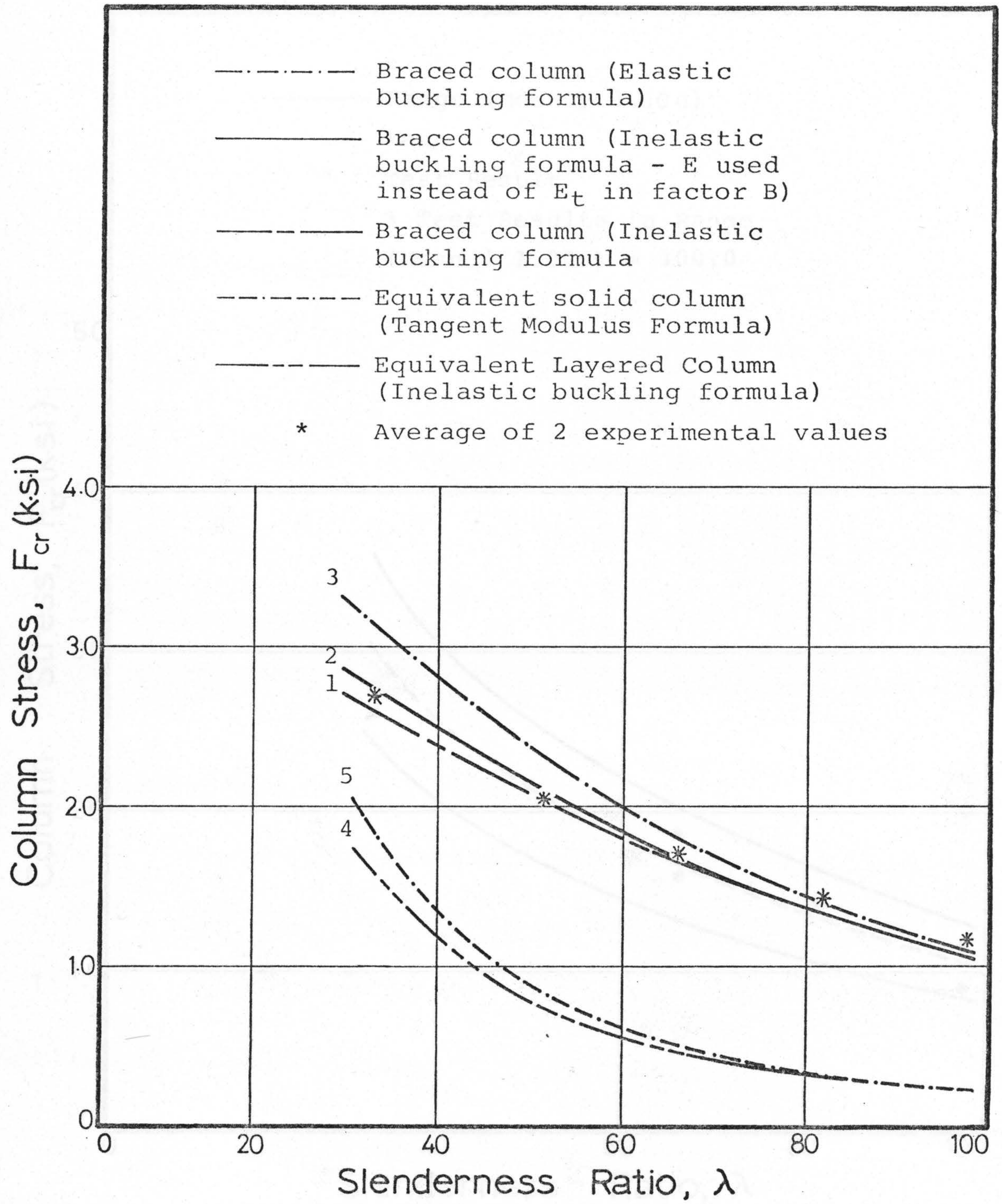


FIGURE 8.H2.2 Column Stress versus Slenderness Ratio Curves - Column Type H2

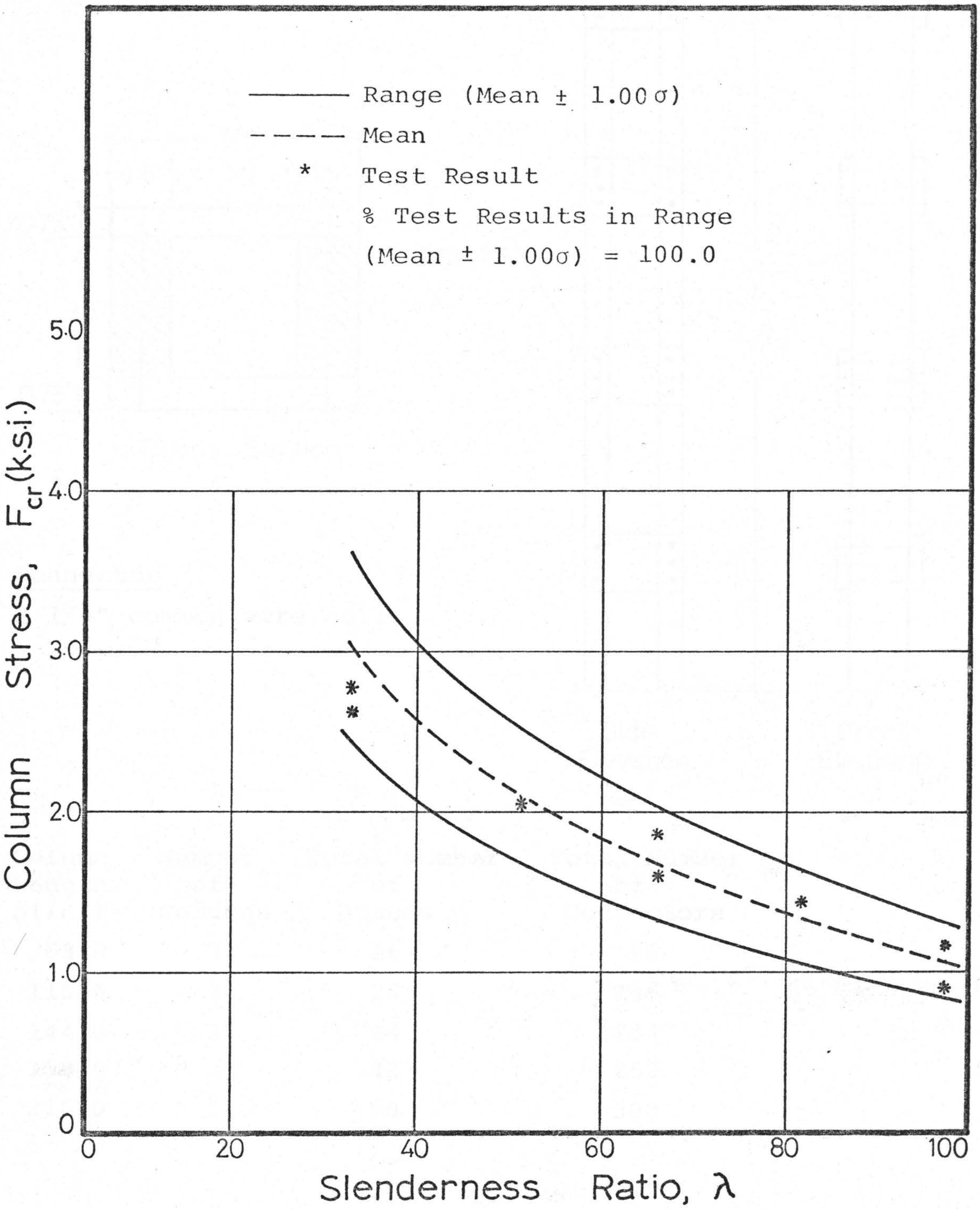
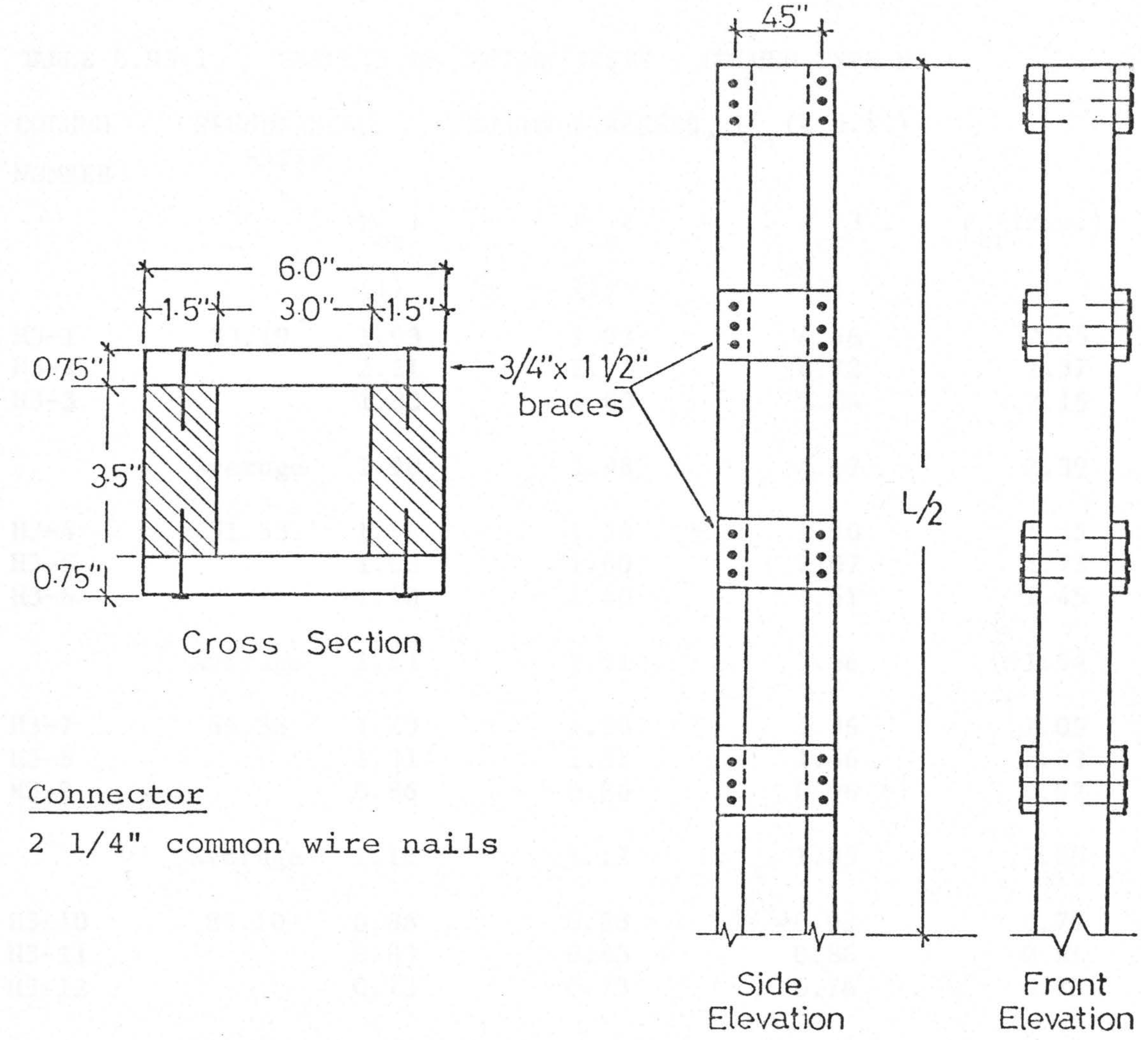


FIGURE 8.H2.3 Statistical Analysis Curves - Column Type H2



Connector

2 1/4" common wire nails

Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
68.0	3	16	96
110.0	3	26	156
144.0	3	34	204
180.0	3	42	252
216.0	3	50	300

FIGURE 8.H3.1 Details of Column Type H3

TABLE 8.H3.1 RESULTS OF COLUMN TESTS - COLUMN TYPE H3

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
H3-1	33.19	1.93	1.93	4.46	2.65
H3-2		2.21	2.21	4.22	2.37
H3-3		1.80	1.80	3.84	2.15
	Average	1.98	1.98	4.17	2.39
H3-4	51.53	1.54	1.54	1.90	1.45
H3-5		1.60	1.60	2.07	1.73
H3-6		1.40	1.40	1.61	1.45
	Average	1.51	1.51	1.86	1.54
H3-7	66.38	1.20	1.20	1.36	1.05
H3-8		1.31	1.31	1.46	1.03
H3-9		0.86	0.86	0.96	0.92
	Average	1.12	1.12	1.25	1.00
H3-10	82.10	0.88	0.88	0.93	0.79
H3-11		0.83	0.83	0.86	0.81
H3-12		0.73	0.73	0.76	0.75
	Average	0.81	0.81	0.85	0.78
H3-13	97.82	0.64	0.64	0.67	0.60
H3-14		0.53	0.53	0.56	0.55
H3-15		0.60	0.60	0.64	0.68
	Average	0.59	0.59	0.62	0.61

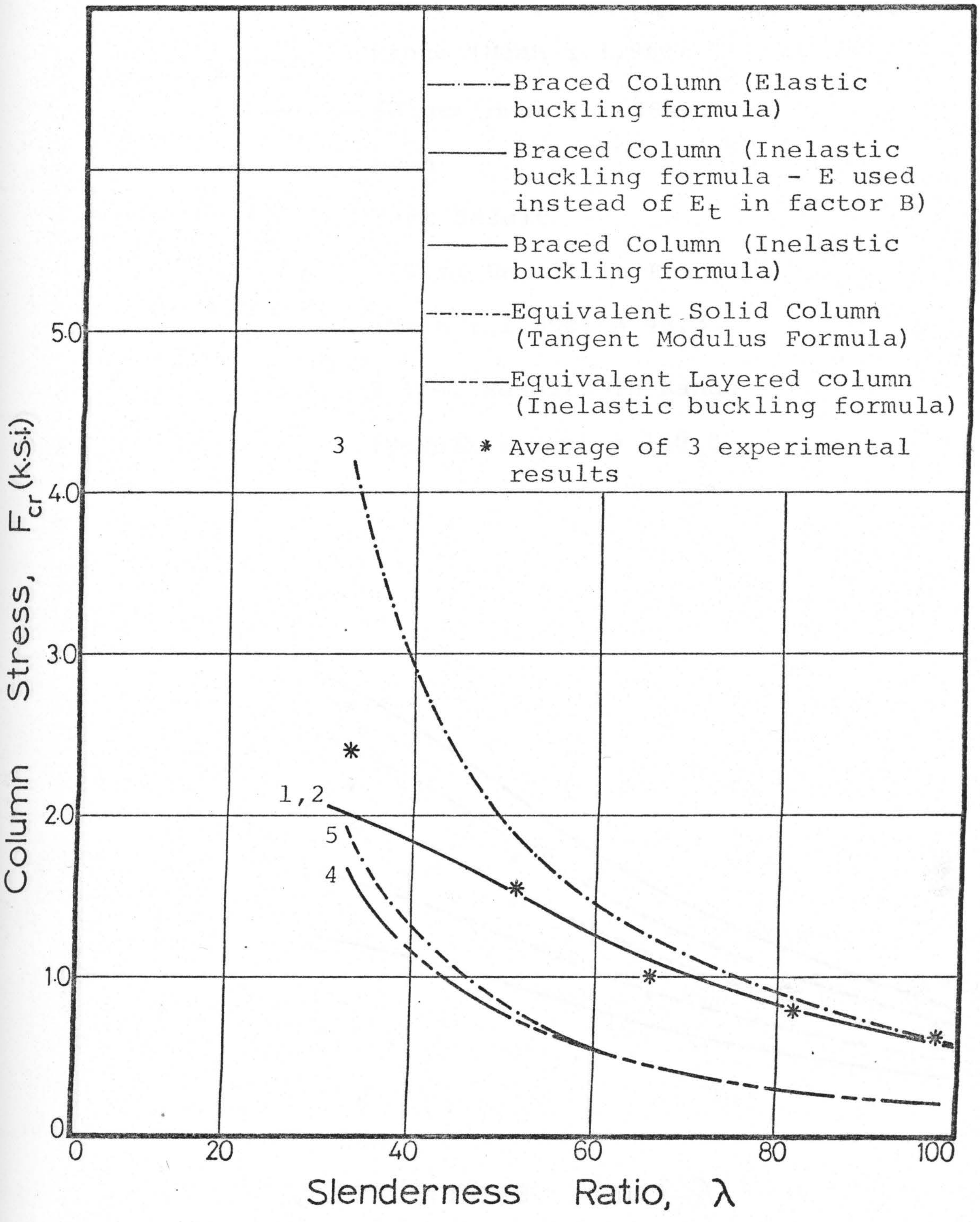


FIGURE 8.H3.2 Column Stress versus Slenderness Ratio Curves - Column Type H3

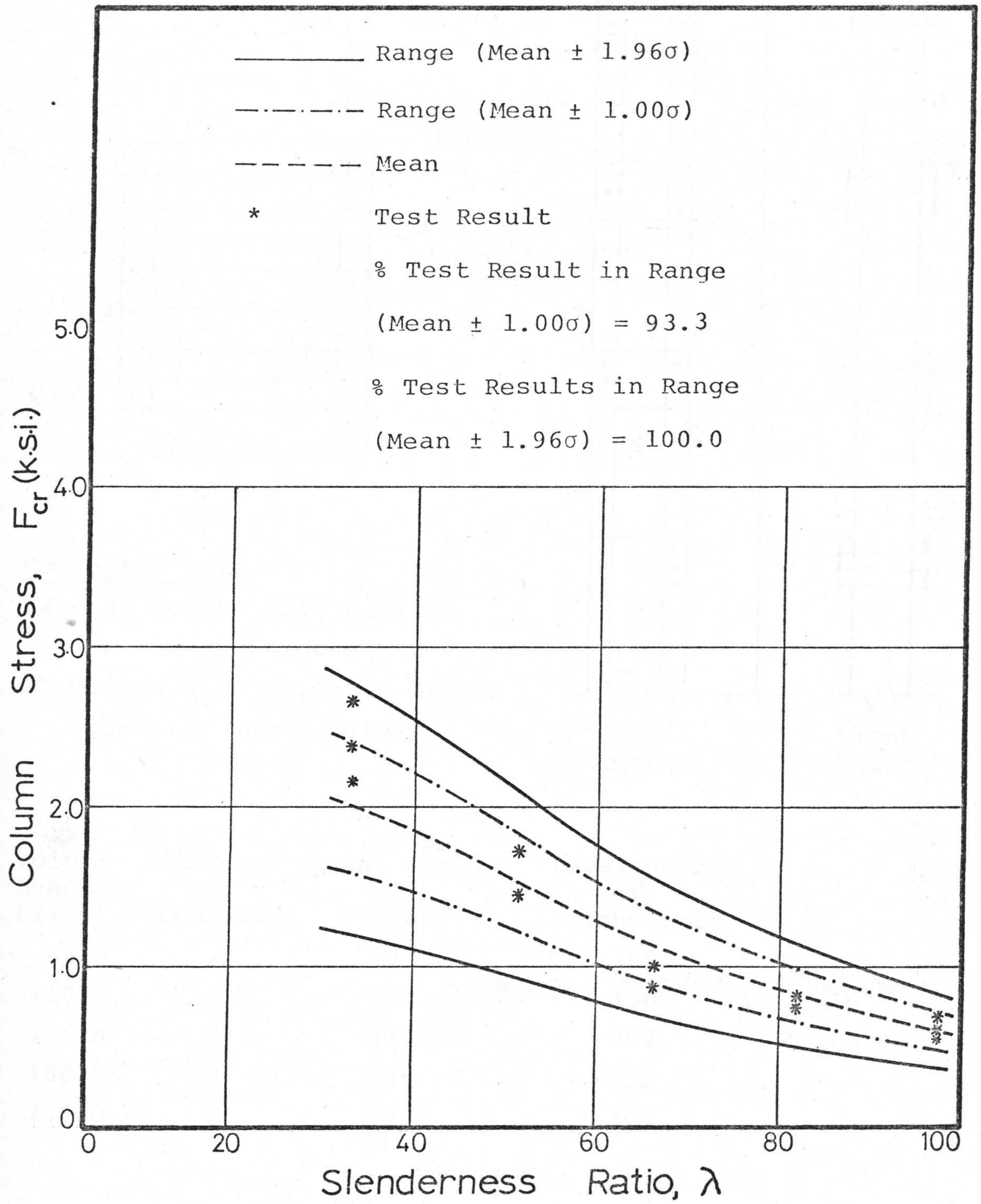
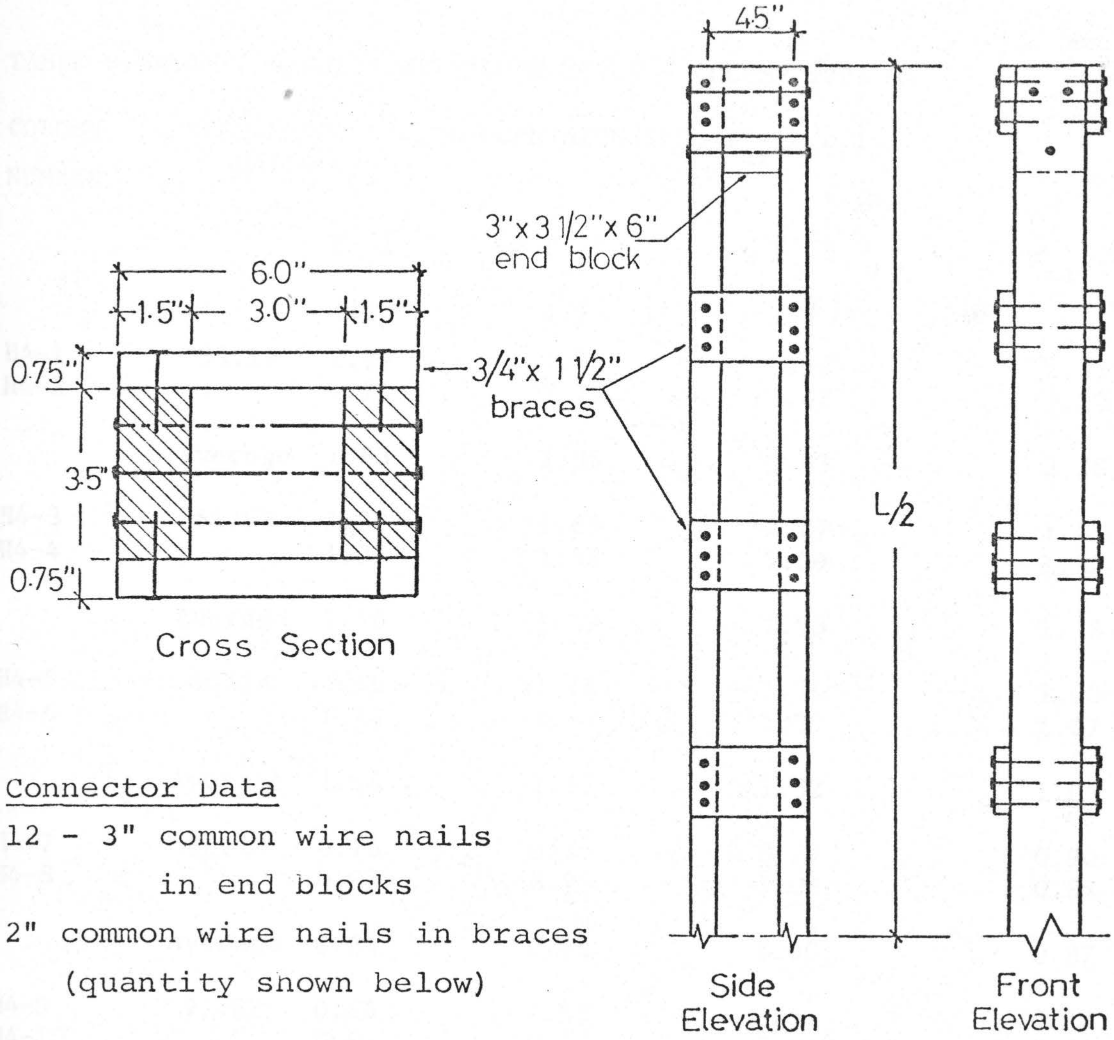


FIGURE 8.H3.3 Statistical Analysis Curves - Column Type H3



Connector Data

12 - 3" common wire nails
in end blocks

2" common wire nails in braces
(quantity shown below)

Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
68.0	2	16	96
110.0	2	26	156
144.0	2	34	204
180.0	2	42	252
216.0	2	50	300

FIGURE 8.H4.1 Details of Column Type H4

TABLE 8.H4.1 RESULTS OF COLUMN TESTS - COLUMN TYPE H4

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
H4-1	33.19	1.89	1.93	3.97	2.37
H4-2		1.75	1.77	3.68	2.20
	Average	1.83	1.85	3.83	2.28
H4-3	51.53	1.59	1.60	2.27	1.43
H4-4		1.37	1.38	1.59	1.32
	Average	1.48	1.49	1.93	1.38
H4-5	66.38	1.16	1.16	1.34	1.02
H4-6		1.14	1.14	1.30	1.07
	Average	1.15	1.15	1.32	1.05
H4-7	82.10	0.73	0.73	0.77	0.90
H4-8		0.95	0.95	1.03	0.85
	Average	0.84	0.84	0.90	0.87
H4-9	97.82	0.65	0.65	0.69	0.70
H4-10		0.61	0.61	0.63	0.64
	Average	0.63	0.63	0.66	0.67

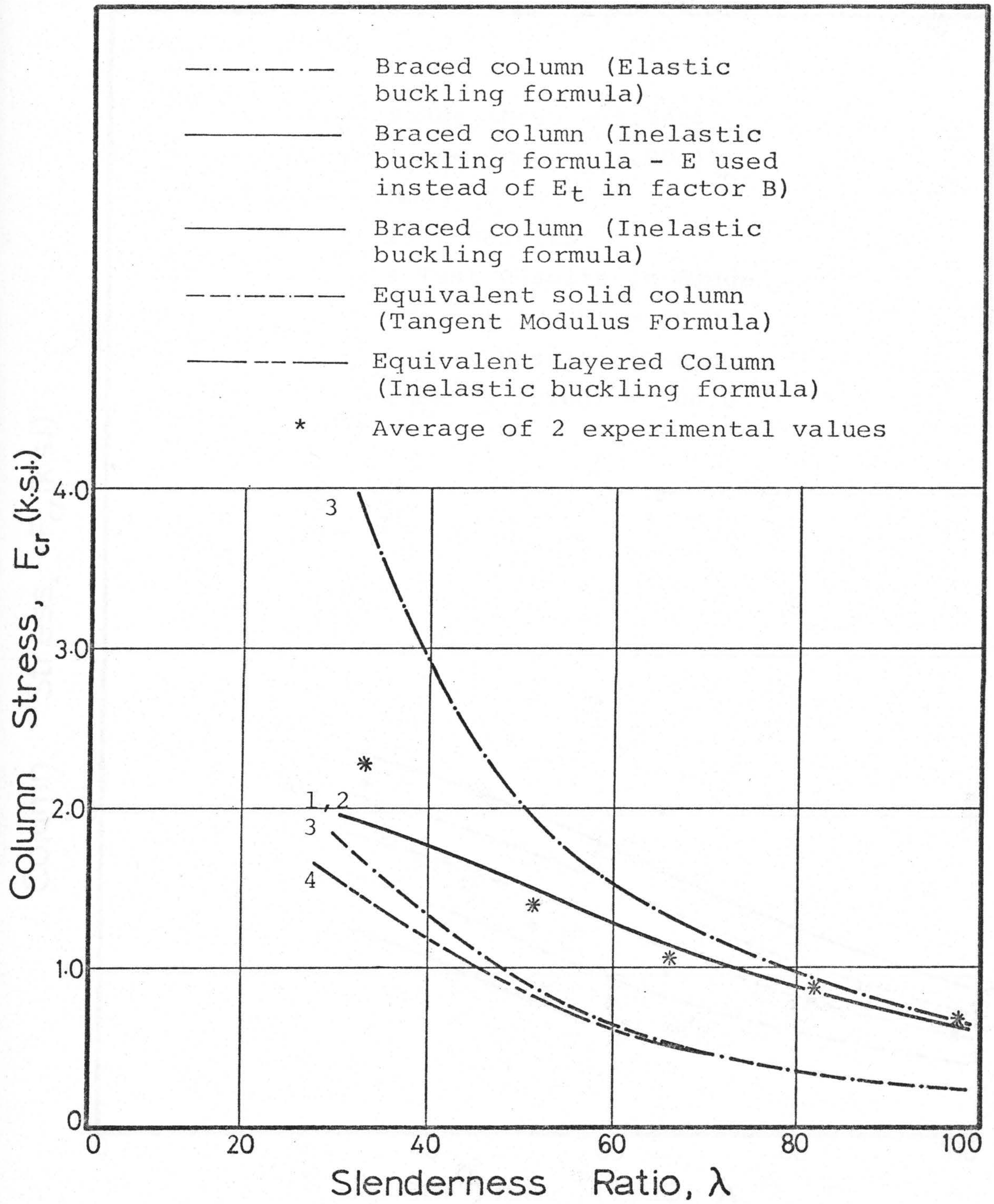


FIGURE 8.H4.2 Column Stress versus Slenderness Ratio Curves - Column Type H4

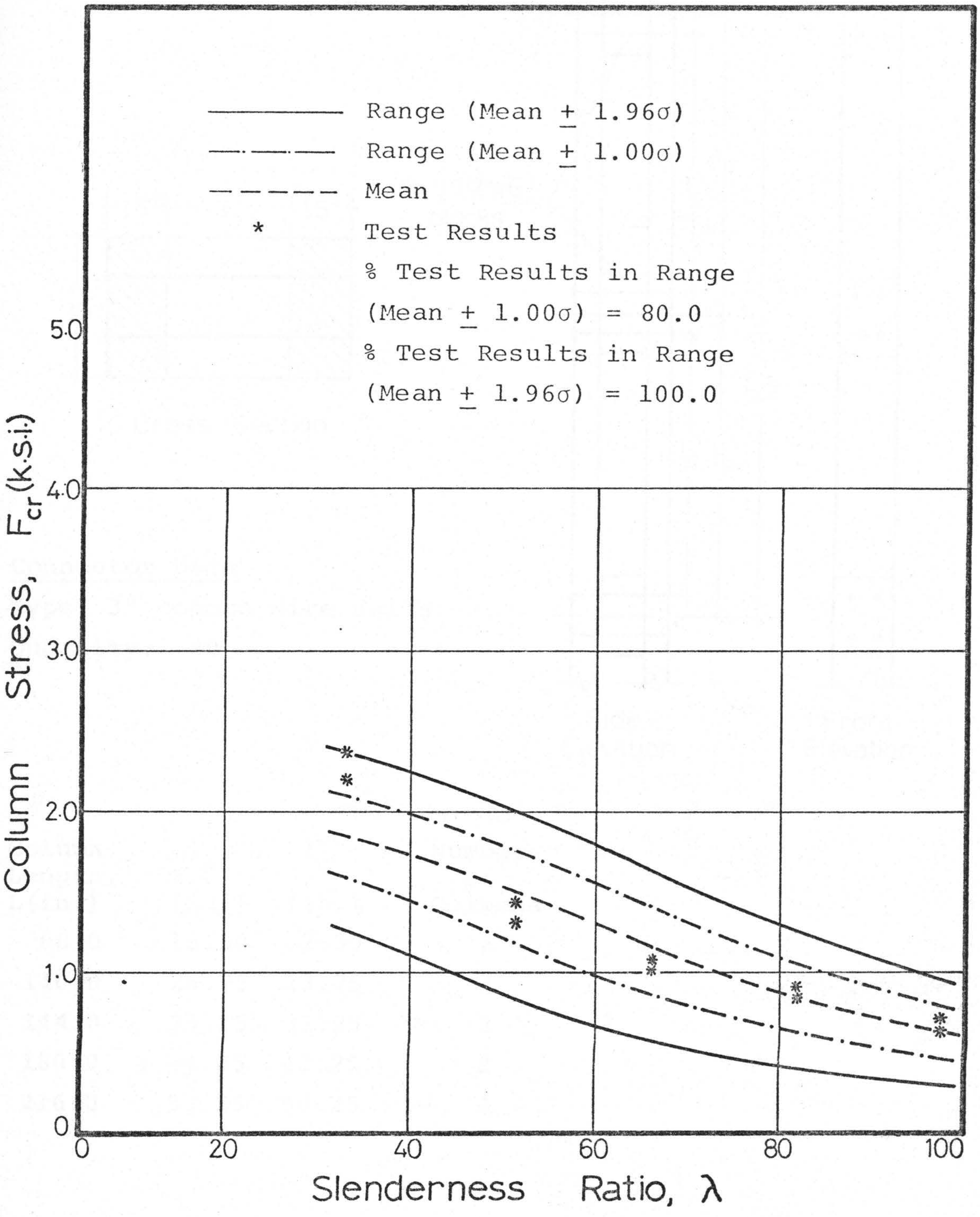
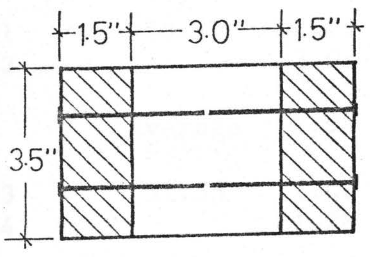
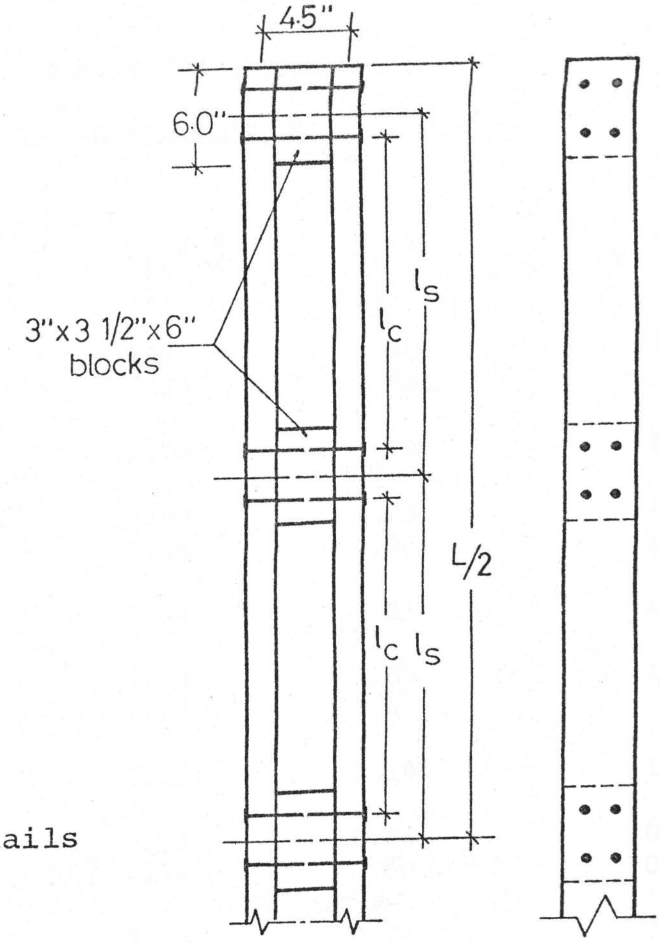


FIGURE 8.H4.3 Statistical Analysis Curves - Column Type H4



Cross Section



Side Elevation

Front Elevation

Connector Data

Type: 3" common wire nails

Quantity: 40

Column Length, L(in.)	l_s (in.)	l_c (in.)	Number of Columns
68.0	15.50	12.50	2
110.0	26.75	23.75	2
144.0	35.25	32.25	2
180.0	44.25	41.25	2
216.0	53.25	50.25	2

FIGURE 8.H5.1 Details of Column Type H5

TABLE 8.H5.1 RESULTS OF COLUMN TESTS - COLUMN TYPE H5

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr}(k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
H5-1	33.19	2.79	2.94	3.50	3.03
H5-2		3.08	3.25	3.98	2.94
	Average	2.94	3.09	3.74	2.99
H5-3	51.53	2.02	2.08	2.25	1.85
H5-4		1.28	1.30	1.37	1.65
	Average	1.65	1.68	1.81	1.75
H5-5	66.38	1.01	1.03	1.06	1.32
H5-6		1.25	1.27	1.32	1.24
	Average	1.13	1.15	1.19	1.28
H5-7	82.10	0.82	0.83	0.84	0.70
H5-8		0.58	0.59	0.60	0.75
	Average	0.70	0.71	0.72	0.73
H5-9	97.82	0.51	0.51	0.52	0.45
H5-10		0.51	0.51	0.52	0.51
	Average	0.51	0.51	0.52	0.48

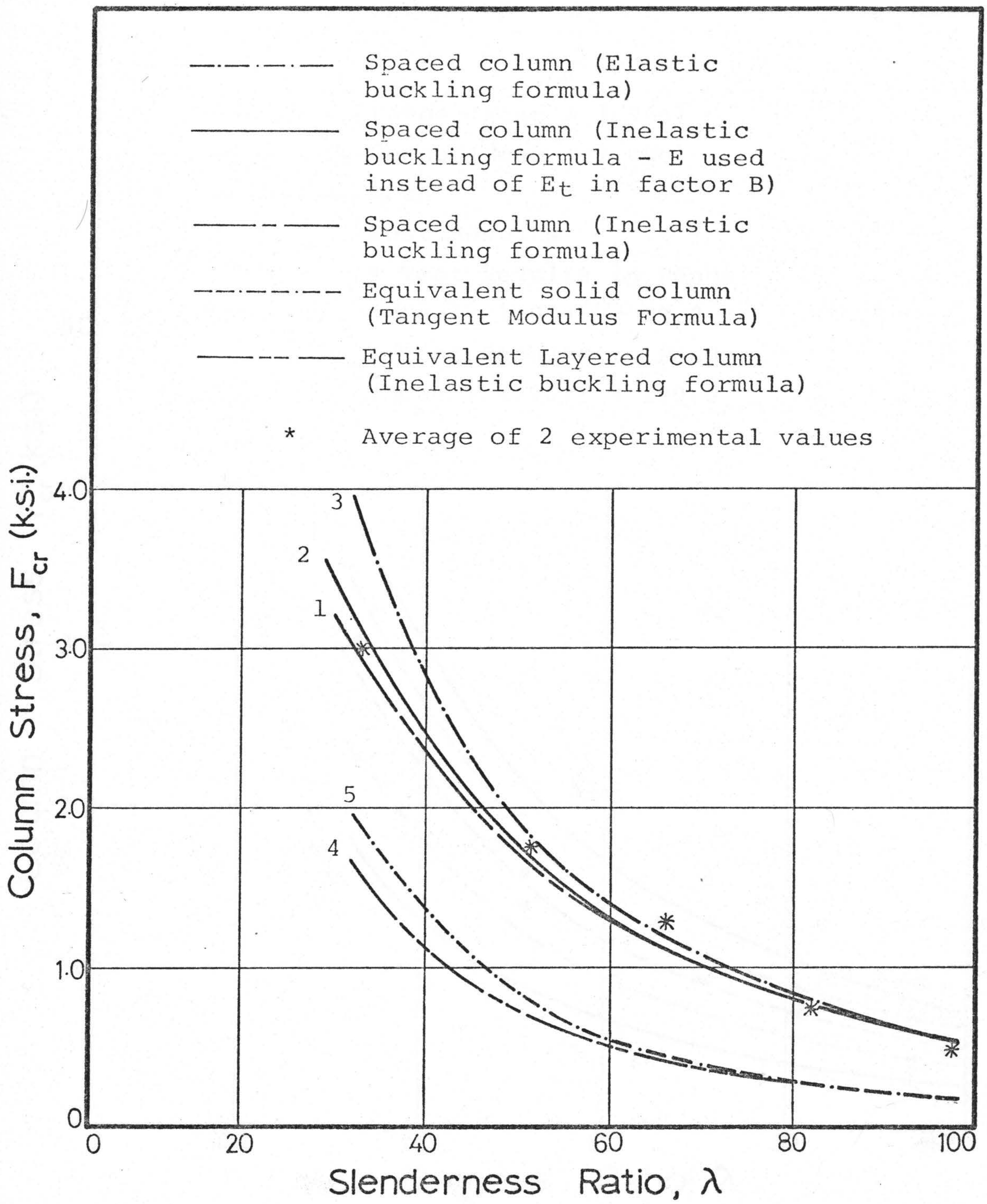


FIGURE 8.H5.2 Column Stress versus Slenderness Ratio Curves - Column Type H5

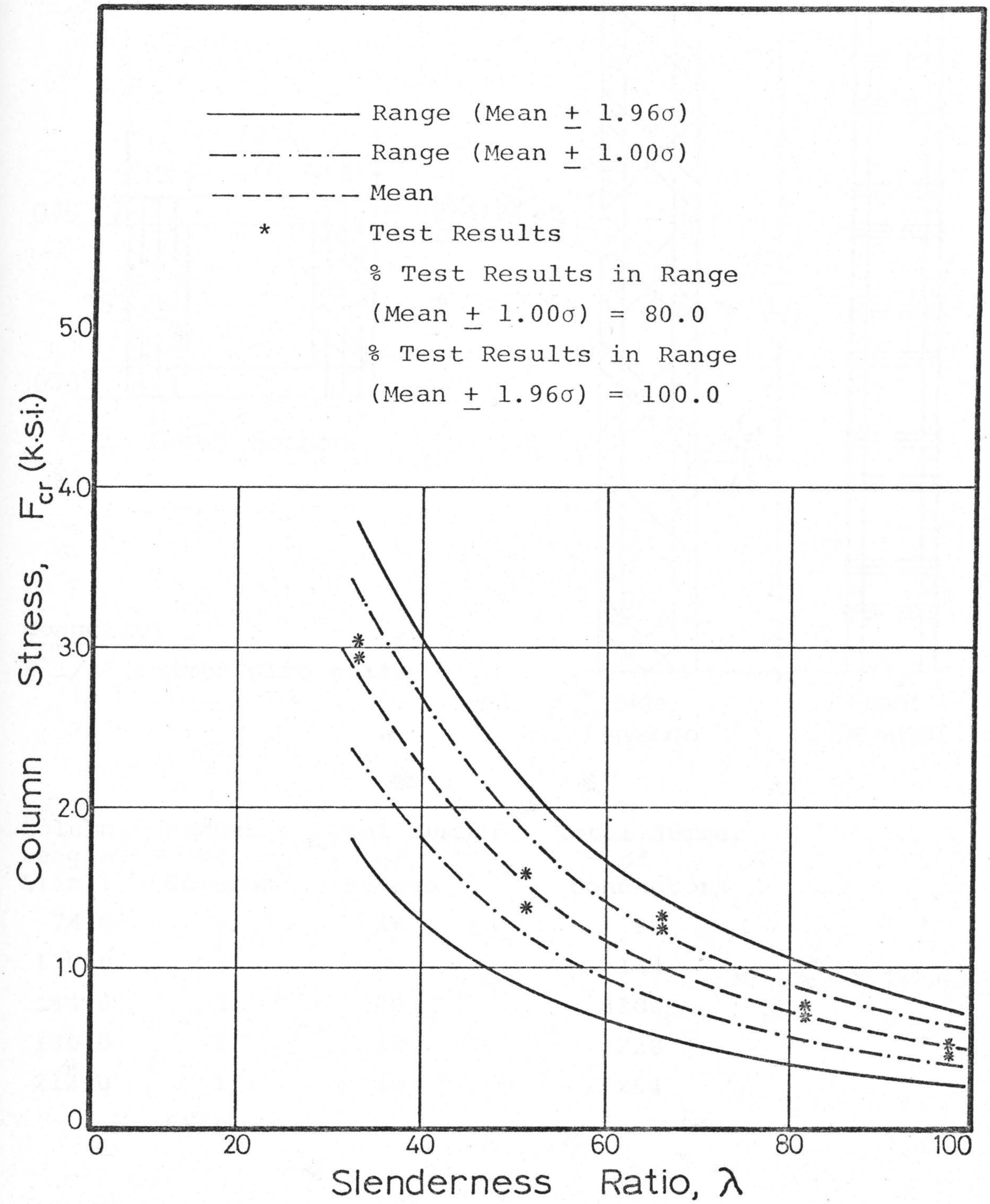
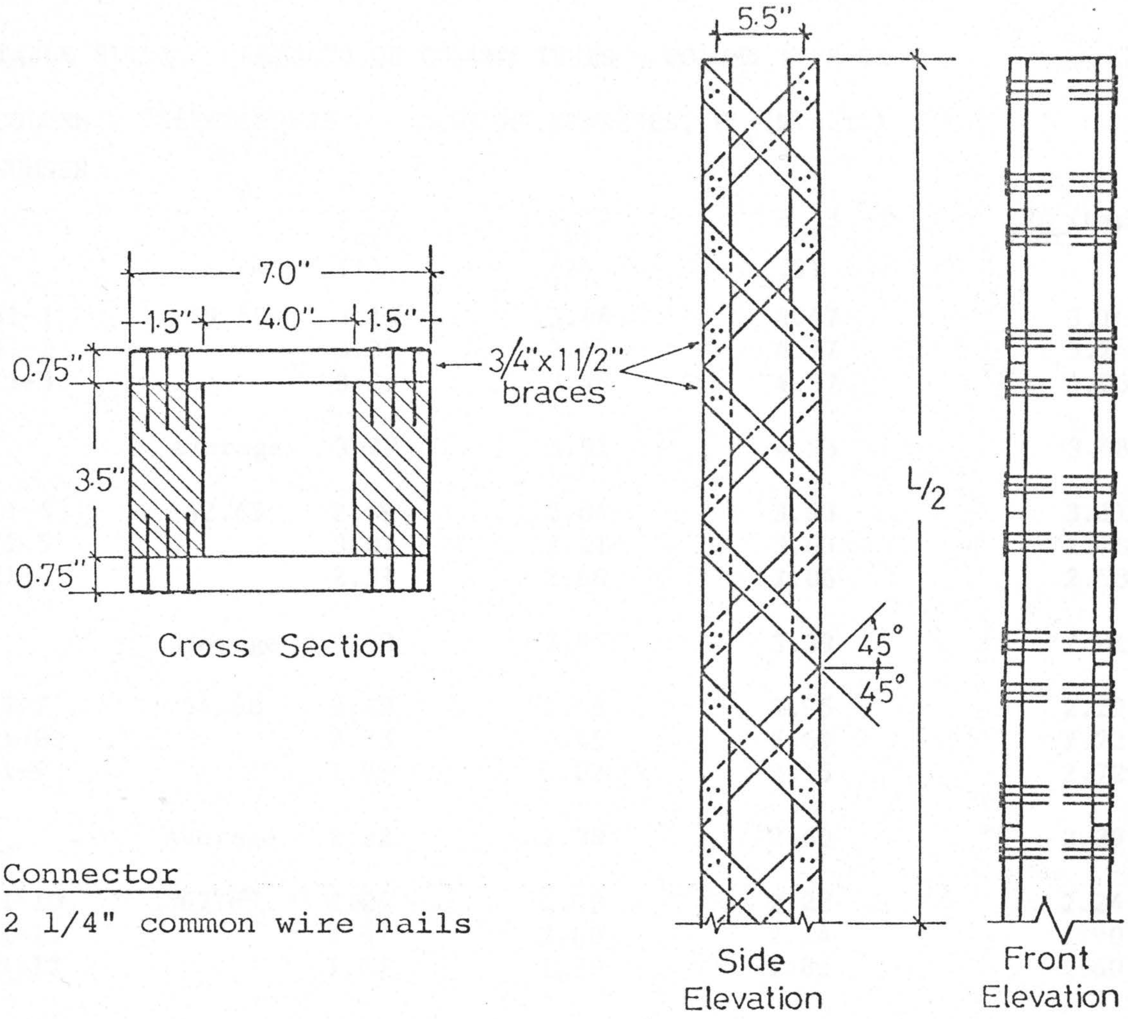


FIGURE 8.H5.3 Statistical Analysis Curves - Column Type H5



Connector

2 1/4" common wire nails

Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
74.0	3	16	96
110.0	3	24	144
144.0	3	30	180
180.0	3	38	228
212.0	3	44	264

FIGURE 8.I1.1 Details of Column Type I1

TABLE 8I1.1 RESULTS OF COLUMN TESTS - COLUMN TYPE I1

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)			
		$F_{cr}1$ (1)	$F_{cr}2$ (2)	$F_{cr}3$ (3)	F_{cr}(Exp.)
I1-1	29.50	3.19	3.44	4.47	3.16
I1-2		3.36	3.62	4.77	3.24
I1-3		3.25	3.48	4.37	4.03
	Average	3.27	3.51	4.53	3.48
I1-4	42.45	2.89	3.04	3.63	3.11
I1-5		3.07	3.21	3.69	2.78
I1-6		2.51	2.60	2.86	2.58
	Average	2.82	2.95	3.39	2.81
I1-7	54.68	2.60	2.65	2.95	2.22
I1-8		2.38	2.45	2.69	2.22
I1-9		1.98	2.02	2.15	2.72
	Average	2.32	2.38	2.60	2.39
I1-10	67.63	2.06	2.09	2.22	2.24
I1-11		1.97	2.00	2.14	1.90
I1-12		1.67	1.70	1.82	1.60
	Average	1.90	1.93	2.06	1.91
I1-13	79.14	1.49	1.49	1.57	1.45
I1-14		1.71	1.72	1.81	1.48
I1-15		1.55	1.56	1.64	1.39
	Average	1.58	1.59	1.67	1.44

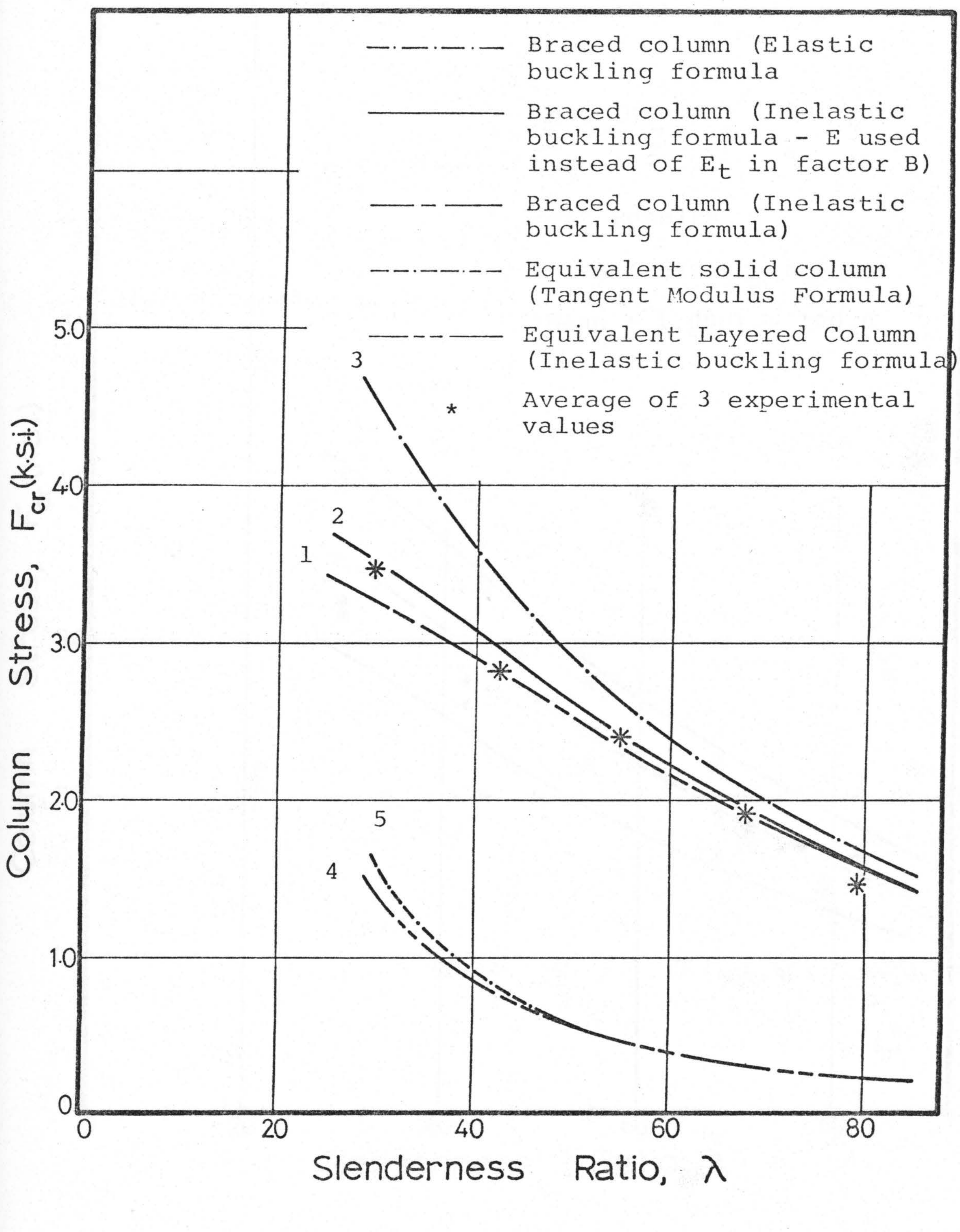


FIGURE 8.I1.2 Column Stress versus Slenderness Ratio Curves -

Column Type I1

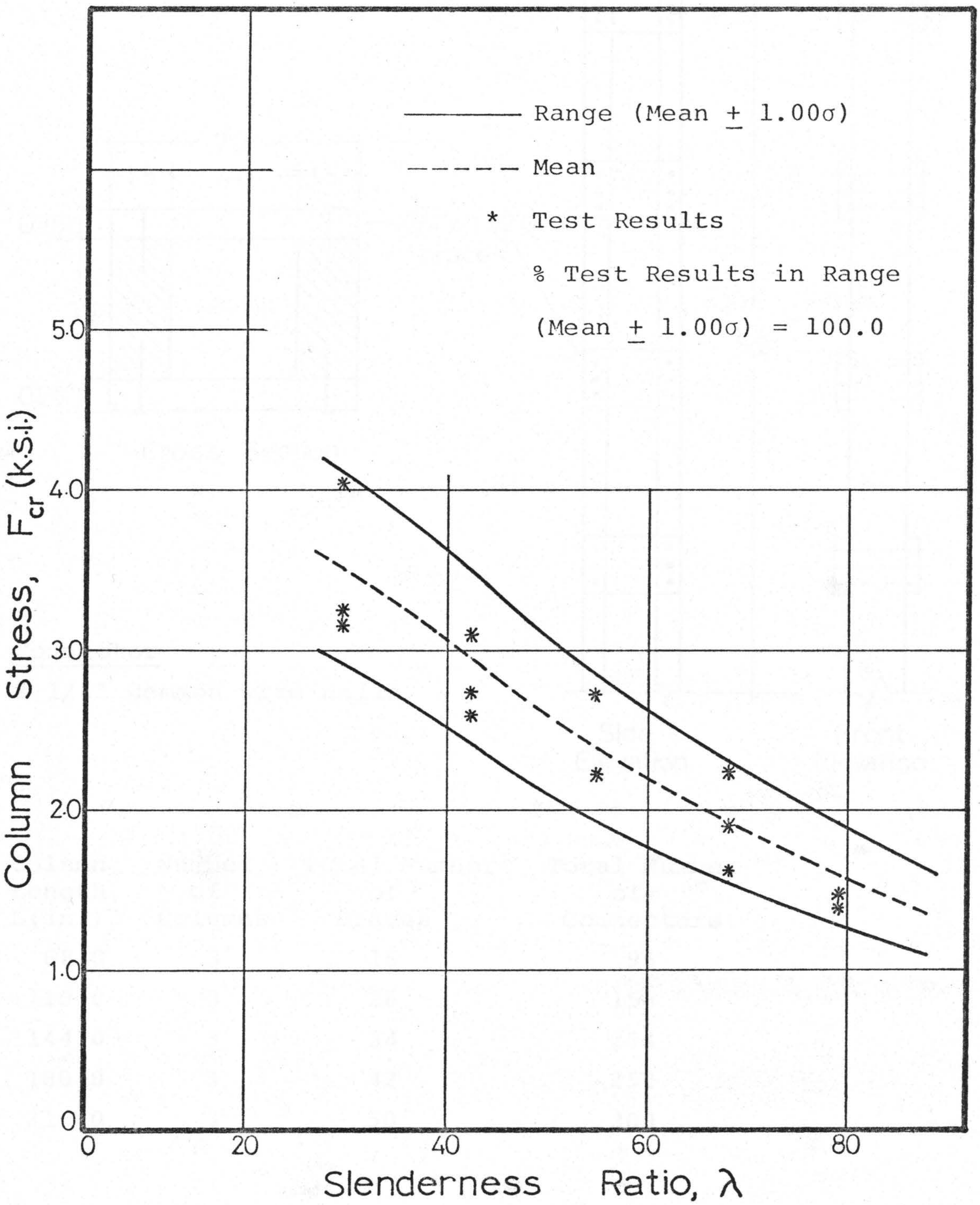
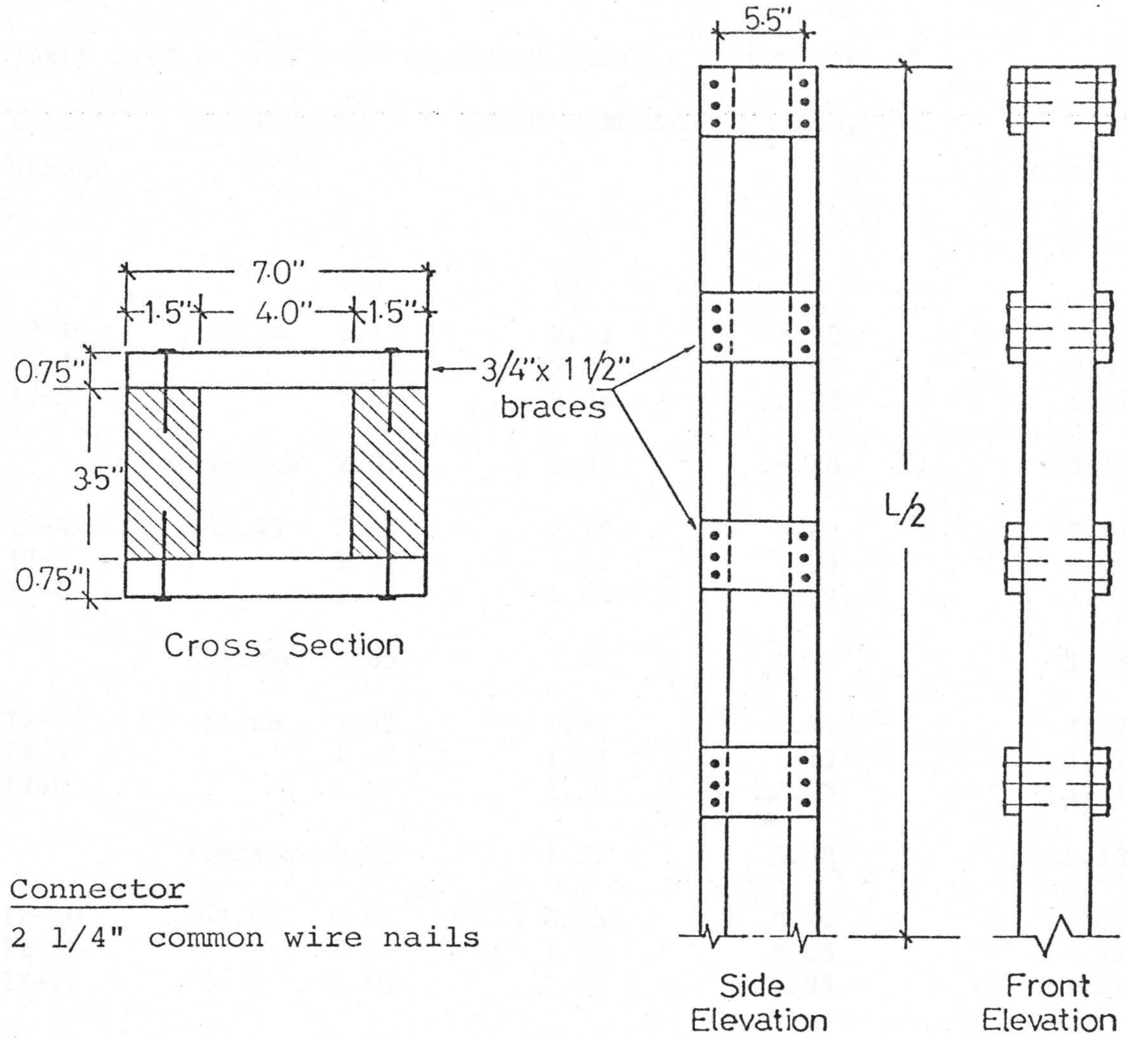


FIGURE 8.I1.3 Statistical Analysis Curves - Column Type I1



Column Length, L(in.)	Number of Columns	Total Number of Braces	Total Number of Connectors
68.0	3	16	96
110.0	3	26	156
144.0	3	34	204
180.0	3	42	252
216.0	3	50	300

FIGURE 8.I2.1 Details of Column Type I2

TABLE 8.I2.1 RESULTS OF COLUMN TESTS - COLUMN TYPE I2

COLUMN NUMBER	SLENDERNESS RATIO λ	COLUMN STRESSES, F_{cr} (k.s.i.)			
		$F_{cr}{}^1$ (1)	$F_{cr}{}^2$ (2)	$F_{cr}{}^3$ (3)	$F_{cr(Exp.)}$
I2-1	27.34	2.12	2.12	6.90	2.56
I2-2		2.05	2.05	5.93	2.45
I2-3		2.05	2.05	6.30	2.37
	Average	2.07	2.07	6.38	2.46
I2-4	42.45	2.15	2.15	3.42	2.01
I2-5		1.77	1.77	2.93	1.75
I2-6		1.62	1.62	2.29	1.88
	Average	1.85	1.85	2.88	1.88
I2-7	54.68	1.62	1.62	2.01	1.37
I2-8		1.47	1.47	1.75	1.37
I2-9		1.59	1.59	2.03	1.41
	Average	1.55	1.55	1.93	1.38
I2-10	67.63	0.85	0.85	0.94	1.02
I2-11		1.29	1.29	1.45	0.98
I2-12		1.40	1.40	1.55	1.07
	Average	1.18	1.18	1.32	1.02
I2-13	80.58	0.73	0.73	0.83	0.81
I2-14		0.73	0.73	0.83	0.79
I2-15		1.10	1.10	1.23	0.85
	Average	0.85	0.85	0.96	0.82

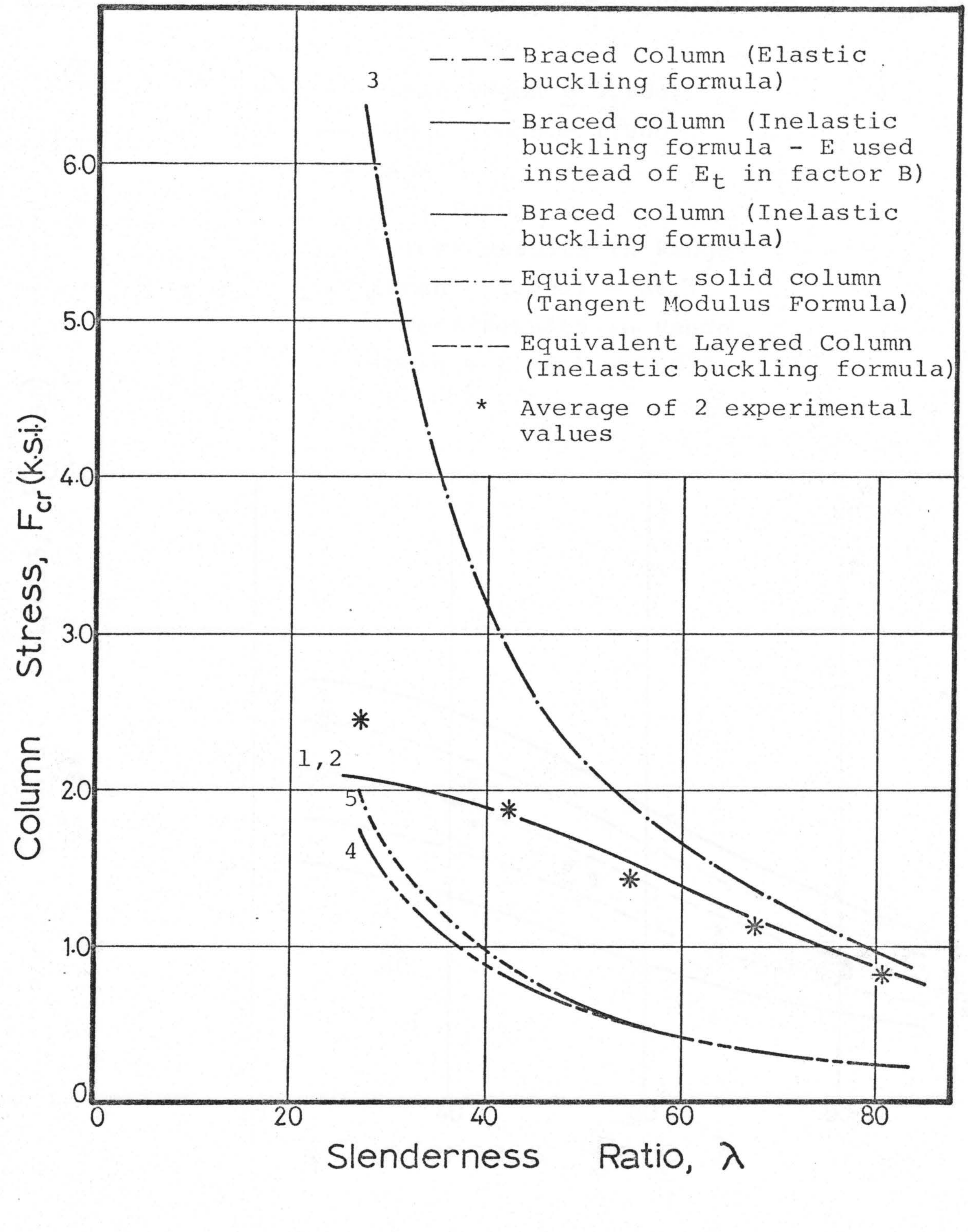


FIGURE 8.I2.2 Column Stress versus Slenderness Ratio Curve - Column Type I2

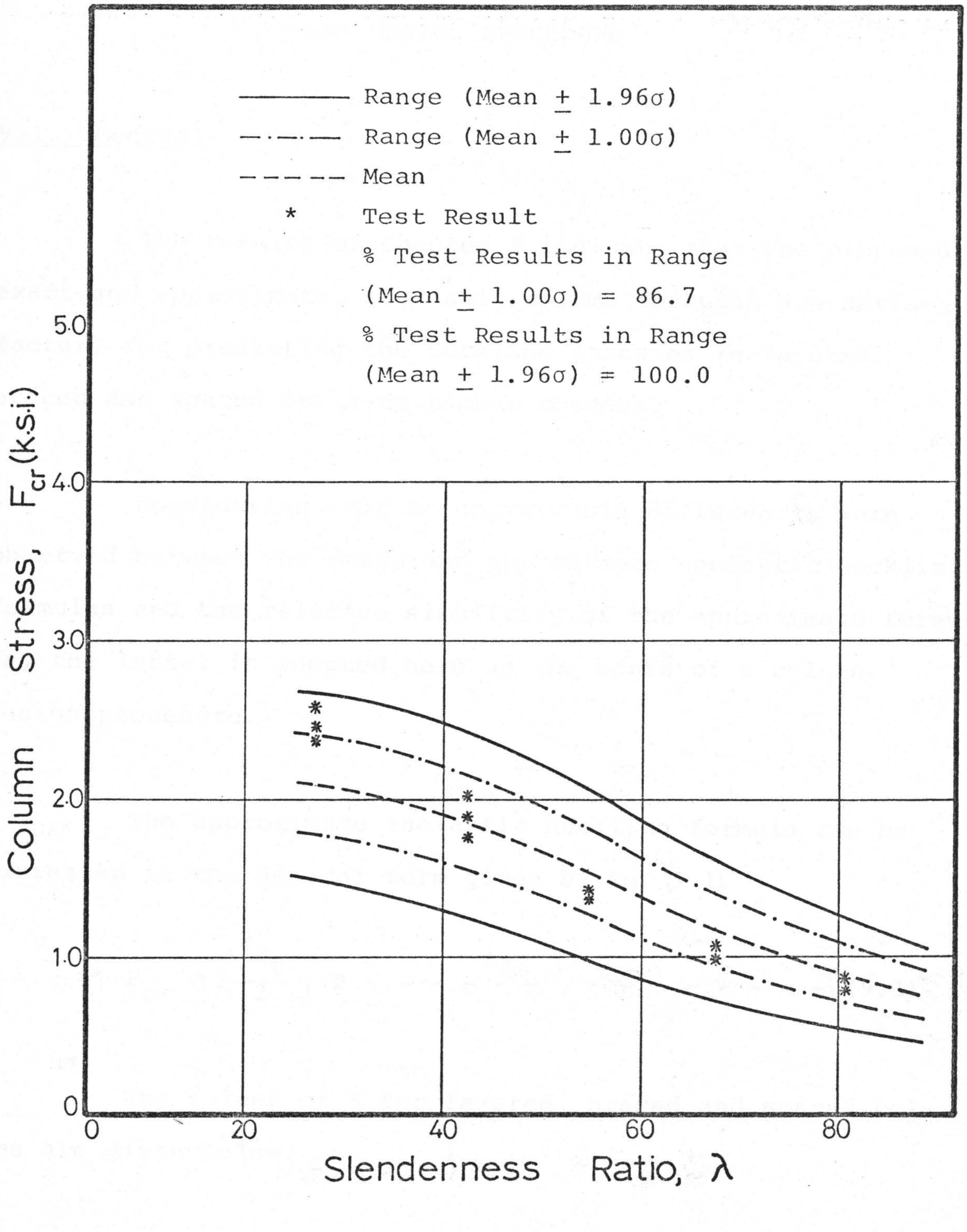


FIGURE 8.I2.3 STATISTICAL ANALYSIS CURVES - Column Type I2

CHAPTER 9

COLUMN DESIGN PROCEDURE

9.1. General

The results of Chapter 8 indicate that the proposed, exact and approximate, inelastic column formulas are satisfactory for predicting the buckling stresses in layered, braced and spaced built-up timber columns.

Considering that no appreciable differences were observed between the exact and approximate inelastic buckling formulas and the relative simplicity of the approximate formula, the latter is adopted here as the basis of a column design procedure.

The approximate inelastic buckling formula can be expressed in the general form given by Eq. (9.1).

$$F_{cr} = \frac{\pi^2 E_t}{\lambda^2} \cdot B \quad (9.1)$$

The values of B for layered, braced and spaced colums are given below.

LAYERED COLUMNS

$$B = \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2mkL^2}} \qquad (9.2)$$

SPACED COLUMNS AND BRACED COLUMNS WITH 45° BRACES

$$B = \frac{1 + \alpha \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12 l_s L^2}}{1 + \dfrac{\pi^2 E A_r a}{mkL^2} + \dfrac{\pi^2 l_c^3}{12\, l_s L^2}} \qquad (9.3)$$

BRACED COLUMNS WITH HORIZONTAL BRACES

$$B = \left(\frac{1 + \alpha \dfrac{\pi^2 E A_r a}{2\,mkL^2}}{1 + \dfrac{\pi^2 E A_r a}{2\,mkL^2}}\right) \frac{\Sigma h}{L_1} \qquad (9.4)$$

Equation (9.1) is a combination of Pleskov's buckling formula for elastic built-up timber columns and the tangent modulus theory. Equation (9.1), therefore, is applicable to inelastic as well as elastic built-up timber columns.

9.2. Proposed Buckling Coefficient Method for the Design of Built-up Timber columns

The following procedure is based on a study conducted by Malhotra (34, 44) and Malhotra and Mazur (45) for solid timber columns.

For $F = F_{cr}$, substituting the value of E_t from Eq. (3.7) into Eq. (9.1), it yields

$$F_{cr} = \frac{\pi^2 EB}{\lambda^2} \cdot \frac{F_u - F_{cr}}{F_u - cF_{cr}} \qquad (9.5)$$

Multiplying both sides of Eq. (9.5) by $\frac{F_u}{\pi^2 E}$ and rearranging the terms, the following dimensionless form is obtained

$$\frac{F_u \lambda^2}{\pi^2 E.B} = \frac{F_u}{F_{cr}} \; \frac{1 - \frac{F_{cr}}{F_u}}{1 - c\,\frac{F_{cr}}{F_u}} \qquad (9.6)$$

The right side of Eq. (9.6) is a function of the parameter c and the ratio $\frac{F_{cr}}{F_u}$. Using a value of c for eastern spruce lumber equal to 0.9, as recommended by Malhotra (44), a curve can be plotted between the two dimensionless quantities $\frac{F_{cr}}{F_u}$ and $\frac{\lambda}{\pi}\sqrt{\frac{F_u}{E.B}}$, as shown in Figure 9.1.

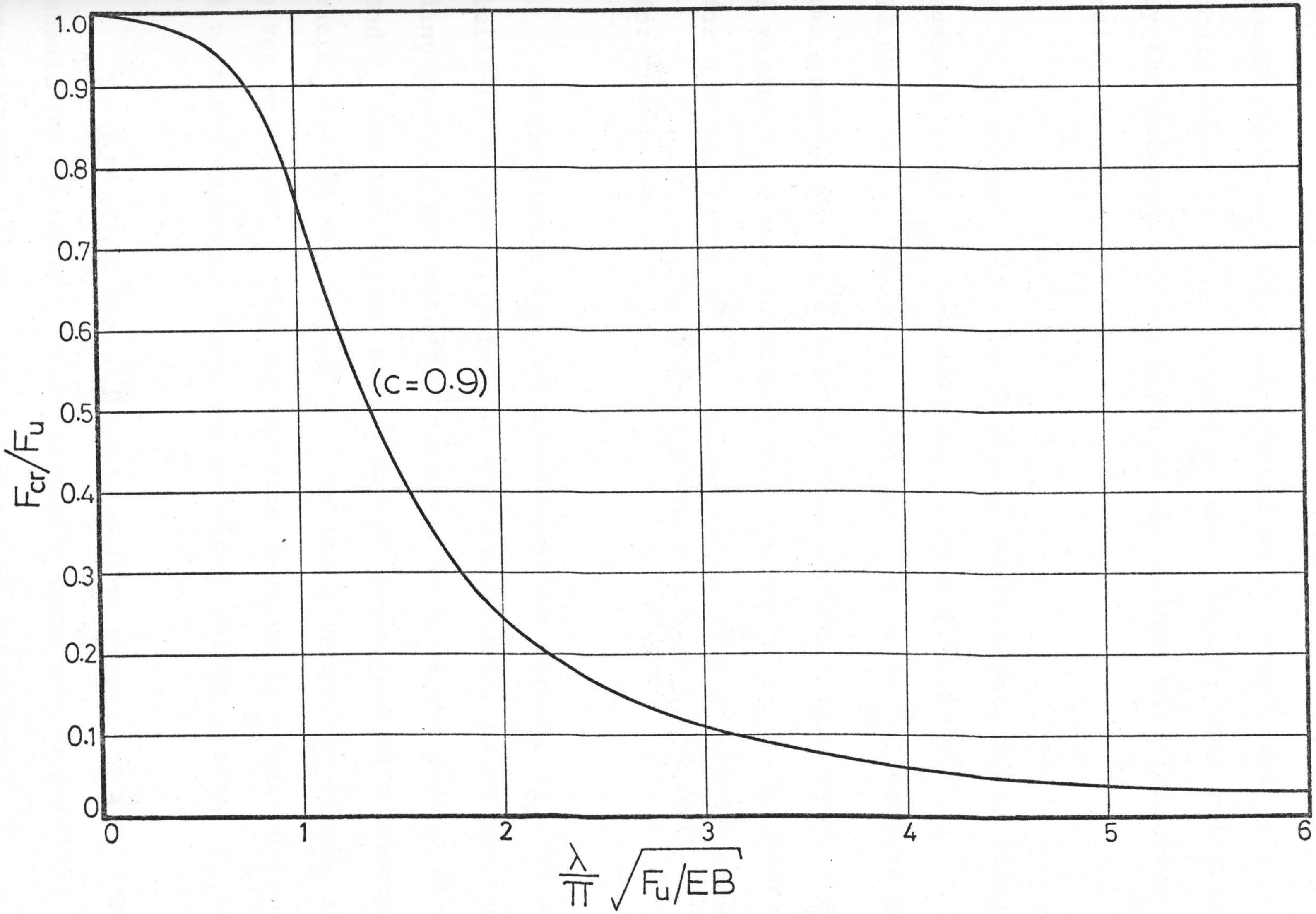


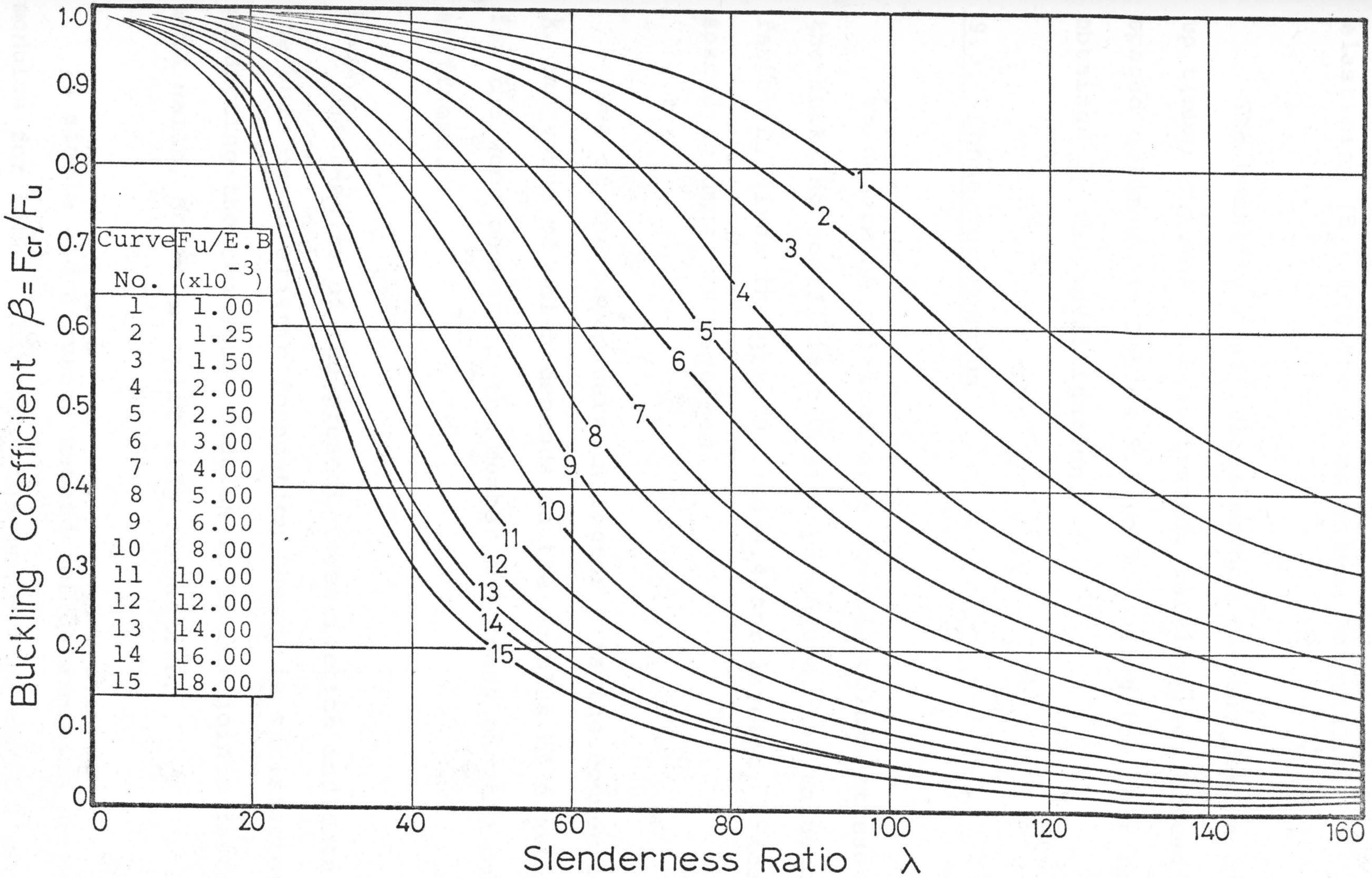
FIGURE 9.1 Buckling Stress Diagram in Dimensionless Form

The ratio $\frac{F_{cr}}{F_u}$ will be referred to as the "Buckling Coefficient" and will be denoted by β. The buckling coefficient is thus defined as the quantity when multiplied by the compressive strength of the column material gives the column buckling stress.

In Figure 9.2, the buckling coefficient, β, is plotted against λ for $\frac{F_u}{E.B.}$ values ranging from 1.00×10^{-3} to 18.00×10^{-3}. The test values of the ratio $\frac{F_u}{EB}$ encountered in the present investigation were well within this range. The β versus λ curves in Figure 9.2 can quite easily be plotted for a wider range of $\frac{F_u}{E.B.}$ values to provide a comprehensive aid for design.

To determine the critical buckling stress, F_{cr}, of any built-up column (layered, braced or spaced), using the buckling coefficient method, one needs to know the values of E and F_u for the column material and the values of slenderness ratio, λ, and the factor B. For the given ratio of $\frac{F_u}{E.B}$ and the given value of λ, the appropriate β-value is obtained from the β versus λ curves. Then, the critical column stress, $F_{cr} = \beta \times F_u$.

To determine the allowable column stress, f_c, F_u and E should be replaced by the allowable compressive parallel to grain stress (f) and the design value for modulus of



Curve No.	$F_u/E.B$ ($\times 10^{-3}$)
1	1.00
2	1.25
3	1.50
4	2.00
5	2.50
6	3.00
7	4.00
8	5.00
9	6.00
10	8.00
11	10.00
12	12.00
13	14.00
14	16.00
15	18.00

FIGURE 9.2 Buckling Coefficient versus Slenderness Ratio Curves for various $F_u/E.B$ Values

elasticity (E_a) for the given column material.

The buckling coefficient method for the design of built-up timber columns is illustrated, for layered, braced and spaced columns, in Tables 9.1 to 9.7 using the test results obtained in the investigation.

9.3. Connector Modulus

To determine critical or allowable column stresses using the buckling coefficient design procedure, the value of the factor B, given in Eqns. (9.2) to (9.4) for layered, braced and spaced columns, is required.

One of the parameters in factor B is the connector modulus, k, the value of which depends on the modulus of elasticity for the wood members, the member thickness and the connector stiffness.

The results of the present investigation indicate that the beam on an elastic foundation theory is satisfactory for predicting the connector modulus in timber joints fastened with nails, bolts or split ring connectors.

A simple and accurate method for determining connector modulus for timber joints having members of nominal one inch

TABLE 9.1 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE A1

λ	B	$(F_u/E.B)\mathrm{x}10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr}(Exp.) (ksi)
43.88	0.689	3.9	0.85	3.58	3.57	3.15
61.20	0.738	3.6	0.66	2.78	2.63	2.45
90.07	0.794	3.4	0.32	1.35	1.45	1.33
120.09	0.832	3.2	0.19	0.80	0.87	0.79
159.35	0.864	3.1	0.12	0.47	0.52	0.53

* Buckling coefficient method

** Approximate inelastic formula

$E = 1.577 \times 10^6$ psi.

$F_u = 4.208 \times 10^3$ psi.

TABLE 9.2 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE B2

λ	B	$(F_u/E.B)\times10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr} (Exp.) (ksi)
41.57	0.385	7.5	0.70	3.26	3.01	2.83
58.20	0.449	6.4	0.45	2.07	1.97	2.10
81.29	0.518	5.6	0.28	1.30	1.21	1.37
118.24	0.600	4.8	0.15	0.70	0.67	0.75
153.35	0.656	4.4	0.09	0.42	0.44	0.42

* Buckling coefficient method — $E = 1.616 \times 10^6$ psi.

** Approximate inelastic formula — $F_u = 4.668 \times 10^3$ psi.

TABLE 9.3 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE C3

λ	B	$(F_u/E.B)\times 10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr}(Exp.)
39.96	0.666	4.6	0.85	3.92	3.93	4.00
56.67	0.726	4.2	0.69	3.18	2.83	3.06
85.73	0.792	3.9	0.32	1.48	1.53	1.70
116.25	0.834	3.7	0.20	0.92	0.90	1.19
115.48	0.869	3.5	0.10	0.46	0.53	0.43

* Buckling coefficient method

** Approximate inelastic formula

$E = 1.506 \times 10^6$ psi.

$F_u = 4.612 \times 10^3$ psi.

TABLE 9.4 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE D

λ	B	$(F_u/E.B) \times 10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr}(Exp.) (ksi)
39.58	0.235	13.3	0.43	2.17	2.21	2.33
56.73	0.292	10.7	0.27	1.36	1.39	1.40
85.75	0.372	08.4	0.16	0.81	0.79	0.80
116.10	0.439	07.1	0.10	0.51	0.51	0.56
151.72	0.501	06.2	0.06	0.30	0.34	0.34

* Buckling coefficient method

** Approximate inelastic formula, m=4

$E = 1.615 \times 10^6$ psi.

$F_u = 5.043 \times 10^3$ psi.

TABLE 9.5 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE H1

λ	B	$(F_u/E.B) \times 10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr} (Exp.) (ksi)
33.19	0.250	12.7	0.63	3.04	2.94	2.93
51.53	0.450	07.4	0.50	2.42	2.24	2.13
66.38	0.551	05.8	0.40	1.93	1.77	1.70
82.10	0.647	04.9	0.30	1.45	1.38	1.29
97.82	0.720	04.4	0.22	1.06	1.09	1.00

* Buckling coefficient method

** Approximate inelastic formula

$E = 1.518 \times 10^6$ psi.

$F_u = 4.829 \times 10^3$ psi.

TABLE 9.6 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE H3

λ	B	$(F_u/E.B) x10^{-3}$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr}(exp.) (ksi)
33.19	0.201	16.1	0.46	2.32	1.98	2.39
51.53	0.320	10.1	0.34	1.71	1.51	1.54
66.38	0.357	09.1	0.24	1.20	1.12	1.00
82.10	0.371	08.7	0.16	0.81	0.81	0.78
97.82	0.385	08.4	0.12	0.61	0.59	0.61

* Buckling coefficient method

** Approximate inelastic formula

$E = 1.563 \times 10^6$ psi

$F_u = 5.055 \times 10^3$ psi.

TABLE 9.7 ILLUSTRATION OF BUCKLING COEFFICIENT DESIGN METHOD - COLUMN TYPE H5

λ	B	$(F_u/E.B) x 10^3$	$\beta = F_{cr}/F_u$	F_{cr} * (ksi)	F_{cr} ** (ksi)	F_{cr}(Exp.) (ksi)
33.19	0.273	11.3	0.67	2.87	2.94	2.99
51.53	0.12	10.0	0.36	1.54	1.65	1.75
66.38	0.332	09.3	0.24	1.03	1.13	1.28
82.10	0.348	08.9	0.16	0.69	0.70	0.73
97.82	0.359	08.6	0.11	0.47	0.50	0.48

* Buckling coefficient method

** Approximate inelastic formula

$E = 1.386 \times 10^6$ psi.

$F_u = 4.292 \times 10^3$ psi.

or two inch thickness has been presented in Chapter 7. This method required the use of the expression

$$k = \frac{tdE}{x} \qquad (9.7)$$

where

t = least member thickness, in.

d = connector diameter, in

E = average modulus of elasticity of joint members, lb/in^2.

The value of the factor $1/x$ is obtained from Figures 7.9 or 7.10 for nominal one inch or two inch thick wood, respectively.

As an alternative to this method, Kuenzi's single shear formula, (Eq. 3.1), for connector modulus is presented graphically in Figures 9.3 to 9.6. For nominal one inch and two inch thick wood, curves of connector modulus versus connector diameter are plotted for values of modulus of elasticity, E(wood), ranging from 1.0×10^6 psi to 3.0×10^6 psi. Similar curves can be plotted for other wood sizes.

The curves shown in Figures 9.3 to 9.6 were plotted for joints having members of equal thicknesses (an optimum condition) and for connectors having a modulus of elasticity, $E = 25.0 \times 10^6$ psi. This value of E was based on the average obtained for the connectors used in this investigation.

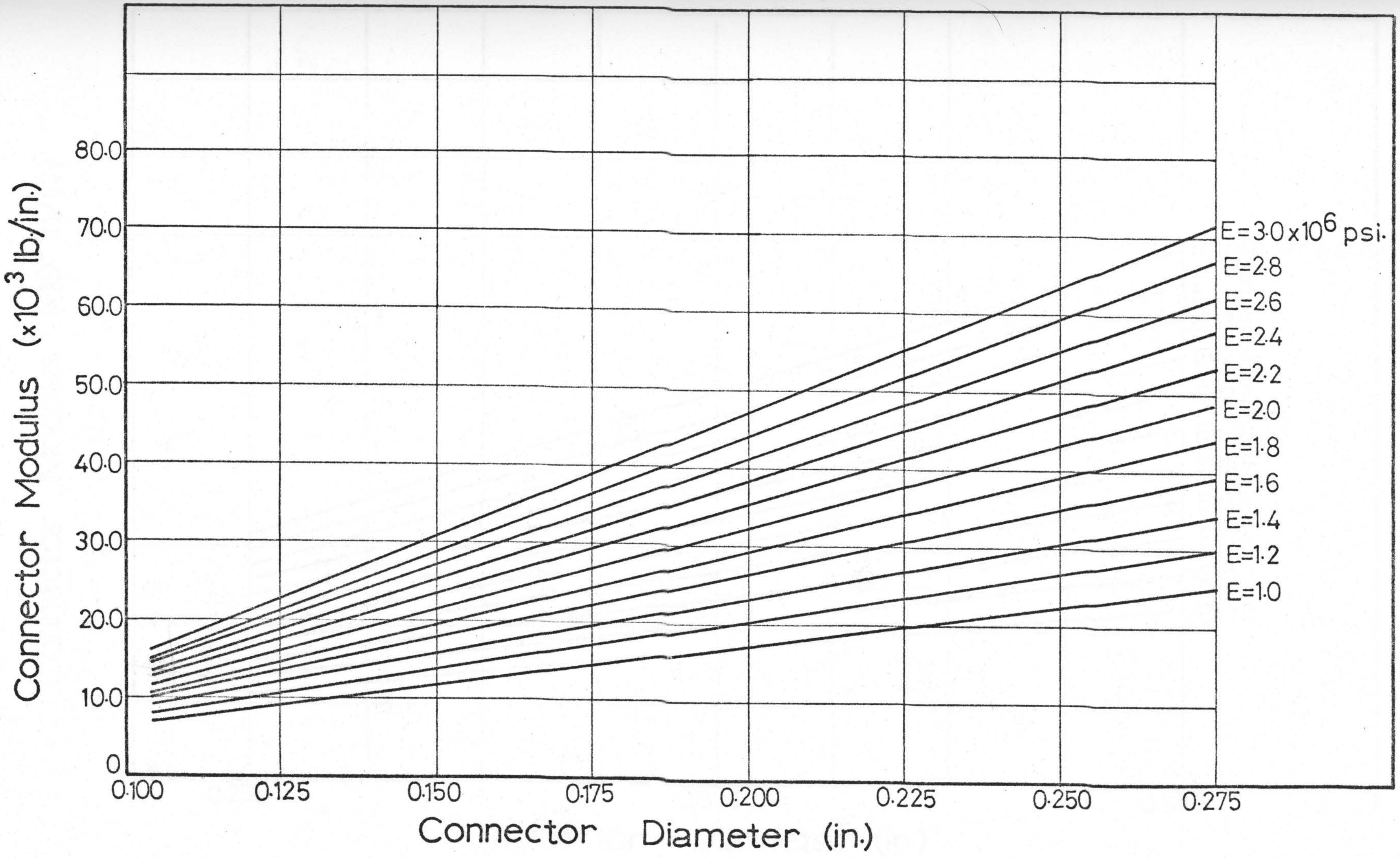


FIGURE 9.3 Connector Modulus versus Connector Diameter for Nominal 1-inch Thick Wood - Connector Diameter 0.100 to 0.250 inch

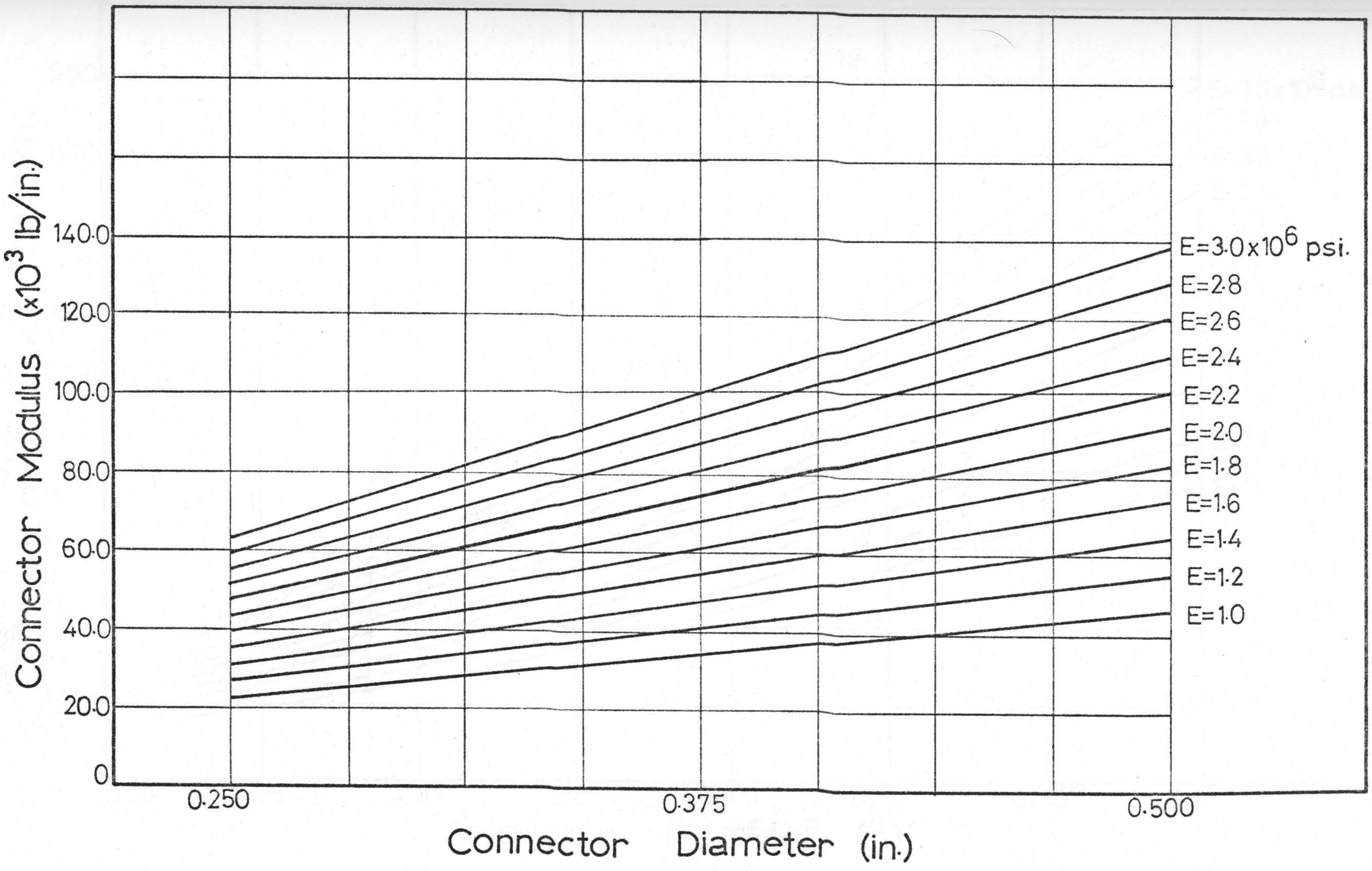


FIGURE 9.4 Connector Modulus versus Connector Diameter for Nominal 1-inch Thick Wood - Connector Diameter 0.250 to 0.500 inch

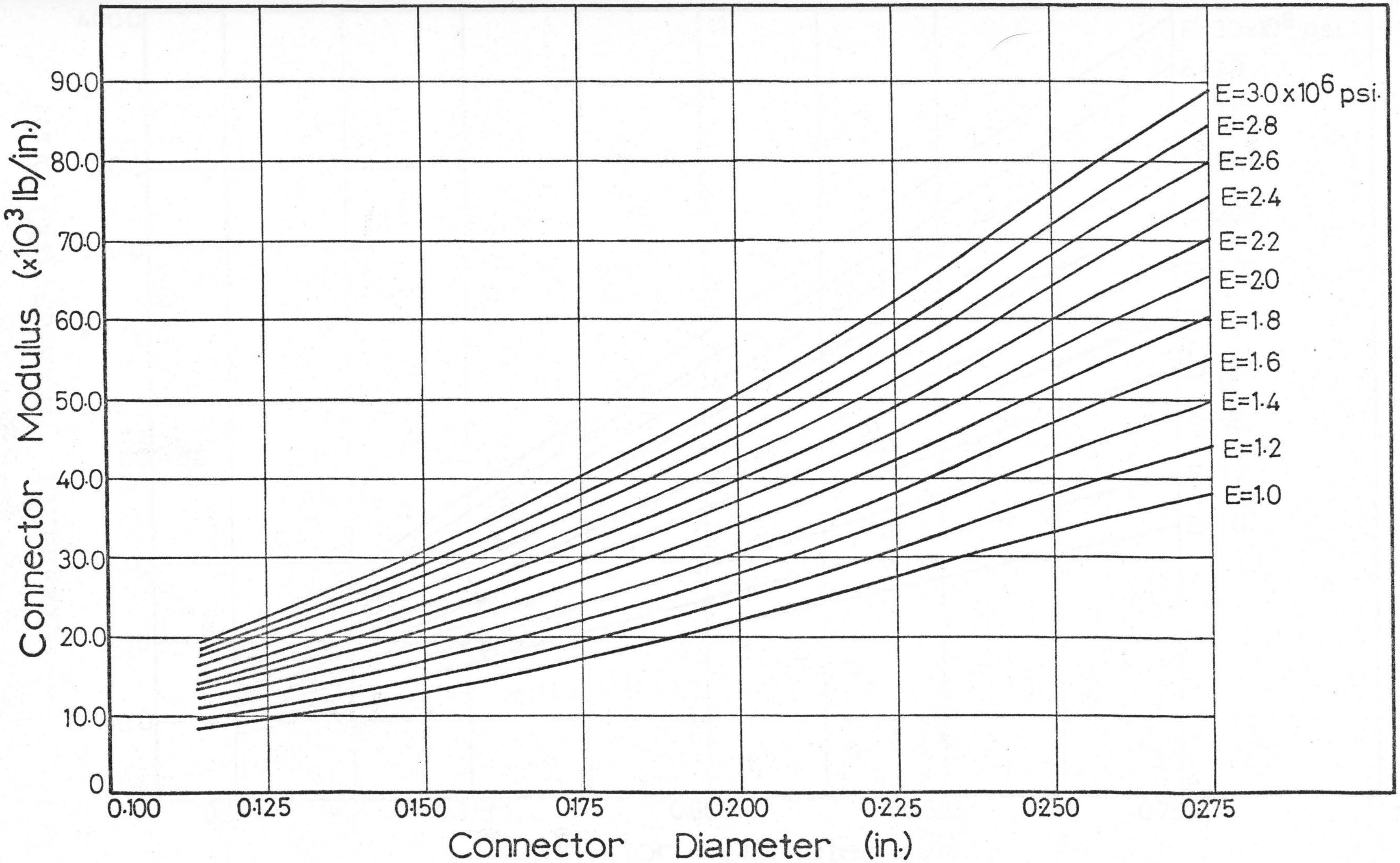


FIGURE 9.5 Connector Modulus versus Connector Diameter for Nominal 2-inch Thick Wood - Connector Diameter 0.100 to 0.275 inch

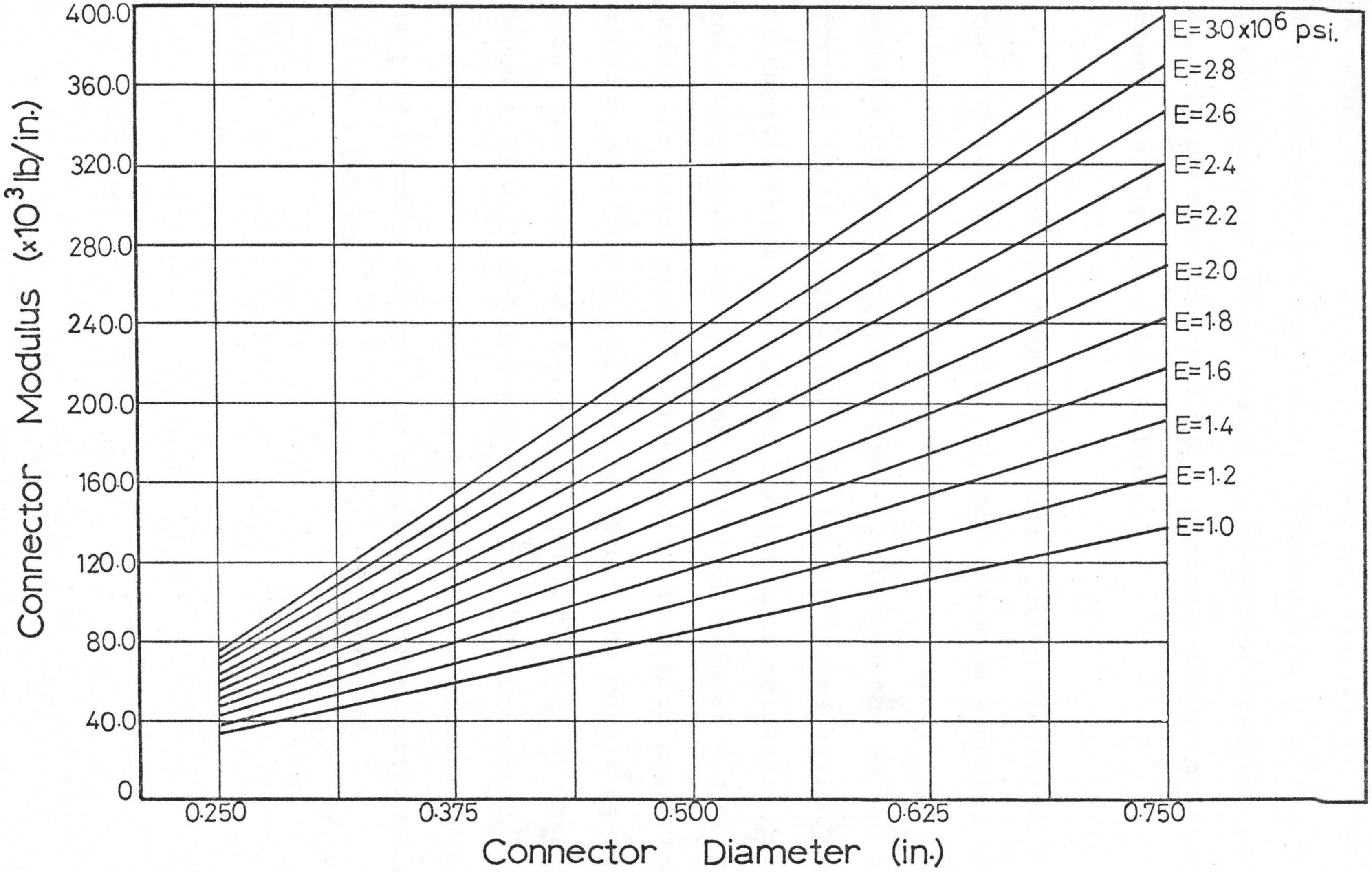


FIGURE 9.6 Connector Modulus versus Connector Diameter for Nominal 2-inch Thick Wood - Connector Diameter 0.250 to 0.500 inch

A change in this value of E does not significantly affect the connector modulus value. Varying E by ± 20 percent produces a negligible change in connector modulus and a change of ± 50 percent in the E value produces about ± 10 percent change in the connector modulus.

To determine connector modulus using Figures 9.3 to 9.6, the modulus of elasticity, E, for the wood members, member thickness and connector diameter are required. The value of modulus of elasticity, E(wood), is determined as the average for the joint considered. For timber joints having members of unequal thicknesses, the least member thickness determines the appropriate curve to be used from Figures 9.3 to 9.6. Using the least member thickness to determine connector modulus does not significantly affect the result as can be seen from Table 7.2, by comparing the results of Connection Types A-13 with B-2 and B-2 with C-1. Furthermore, it should be pointed out that varying the connector modulus by as much as ± 20 percent changes the predicted column stresses by only ± 2 to 3 percent.

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

The conclusions from various aspects of the study reported in this thesis have been summarized at the end of each chapter. Following are the general conclusions drawn with a comprehensive appraisal of the entire study:

1. The beam on elastic foundation concept is seen to be satisfactory for predicting the connector modulus in mechanically fastened timber joints. As an aid to the analysis and design of built-up timber columns, charts are presented for determining connector modulus.

2. Incorporating a stress-strain function for the wood material, formulas have been developed for the buckling stresses of layered, braced and spaced timber columns. These formulas are valid for all slenderness ratios, thus, applicable for elastic and inelastic columns.

3. To verify the validity of the formulas developed, an extensive testing program was undertaken. The predicted values were seen to be in good agreement with experimental results for layered, braced and spaced columns.

4. The statistical analysis of the experimental data indicates that for most of the column types the percentages of the test results compare reasonably well with the probability predictions. Thus, it can be said that the buckling formulas developed are satisfactory predictors of column stresses in various probability ranges.

5. Based on the present theoretical and experimental studies, a design procedure is developed. As an aid to the design of built-up timber columns, charts have been presented. The salient advantages of the proposed design method are as follows:

a. It is applicable to all slenderness ratios.
b. It is developed on a rational basis.
c. It can be used to determine critical or allowable column stresses.
d. It is simple and easy to apply.

10.2. Recommendations

The following aspects of analysis and design of built-up timber columns are recommended for future research.

1. Further experimental investigation should be carried out for layered columns built up from several small size

laminates fastened with small size nails.

2. The problem of built-up columns subjected to eccentric loads should be investigated. Because of the eccentricity of the applied load the determination of connector modulus has to be modified. Also, the effect of initial crookedness of column specimens should be studied.

3. The effect of long term loading on the buckling strength of built-up columns should be investigated.

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APPENDIX "A"

-Tables-

TABLE A.1 RESULTS OF COMPRESSIONS TESTS - CONNECTION SPECIMENS

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT (%)	MODULUS OF ELASTICITY, E ($x10^6$psi)
A-1-1	0.388	13.58	1.765
A-1-2	0.403	12.88	1.369
A-1-3	0.408	13.65	1.822
A-1-4	0.350	13.43	1.444
A-1-5	0.398	13.68	1.631
A-1-6	0.354	13.68	1.313
A-1-7	0.376	13.92	1.080
A-1-8	0.436	13.99	1.085
A-1-9	0.391	13.72	1.354
A-1-10	0.412	13.80	1.390
A-2-1	0.392	14.23	1.818
A-2-2	0.371	14.42	1.293
A-2-3	0.447	14.28	2.044
A-2-4	0.384	14.54	1.740
A-2-5	0.368	14.52	1.427
A-3-1	0.414	13.81	1.984
A-3-2	0.396	13.89	1.518
A-3-3	0.387	13.93	1.624
A-3-4	0.420	13.95	1.710
A-3-5	0.397	13.81	1.881
A-4-1	0.389	13.32	1.641
A-4-2	0.447	14.07	1.797
A-4-3	0.381	13.26	2.099
A-4-4	0.392	13.68	1.876
A-4-5	0.398	12.87	1.726
A-4-6	0.483	13.06	1.848
A-4-7	0.414	12.70	1.717
A-4-8	0.345	12.00	1.633
A-4-9	0.359	12.72	1.490
A-4-10	0.433	12.20	1.670
A-5-1	0.440	13.72	1.990
A-5-2	0.414	14.00	1.885
A-5-3	0.445	14.24	1.797
A-5-4	0.310	13.23	1.461
A-5-5	0.329	13.32	1.449
A-5-6	0.421	13.64	1.641
A-5-7	0.441	14.10	2.038
A-5-8	0.432	13.49	1.727
A-5-9	0.399	14.38	1.875
A-5-10	0.412	14.43	1.489

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY, E(x10^6psi)
A-6-1	0.424	14.24	1.770
A-6-2	0.343	14.21	1.212
A-6-3	0.413	14.30	1.939
A-6-4	0.418	14.17	1.817
A-6-5	0.359	13.92	1.284
A-7-1	0.366	12.59	1.404
A-7-2	0.427	14.23	1.436
A-7-3	0.420	13.24	1.776
A-7-4	0.432	14.78	0.909
A-7-5	0.421	13.48	1.832
A-8-1	0.425	10.53	1.826
A-8-2	0.394	13.13	1.391
A-8-3	0.456	12.93	2.300
A-8-4	0.358	13.43	1.414
A-8-5	0.455	13.29	2.369
A-9-1	0.440	14.35	1.907
A-9-2	0.429	14.21	1.995
A-9-3	0.433	14.29	1.938
A-9-4	0.454	14.29	2.065
A-9-5	0.413	14.39	1.131
A-10-1	0.365	13.79	1.647
A-10-2	0.388	14.14	1.876
A-10-3	0.415	13.79	1.879
A-10-4	0.391	14.04	1.909
A-10-5	0.428	13.88	2.387
A-11-1	0.442	13.65	1.422
A-11-2	0.377	13.49	1.329
A-11-3	0.379	13.15	1.430
A-11-4	0.413	13.18	1.589
A-11-5	0.359	12.91	1.778
A-12-1	0.364	13.97	1.636
A-12-2	0.437	14.10	1.621
A-12-3	0.432	14.34	1.652
A-12-4	0.361	13.86	1.572
A-12-5	0.408	14.06	1.970
A-13-1	0.331	13.70	1.491
A-13-2	0.337	13.73	1.695
A-13-3	0.421	14.53	1.467
A-13-4	0.397	13.98	2.025
A-13-5	0.361	13.74	1.467

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY E(x10^6psi)
A-14-1	0.343	13.60	2.016
A-14-2	0.351	13.30	1.560
A-14-3	0.429	14.17	1.317
A-14-4	0.403	14.13	1.904
A-14-5	0.368	13.88	1.524
A-14-6	0.469	12.46	1.216
A-14-7	0.411	11.78	2.001
A-14-8	0.514	11.98	1.411
A-14-9	0.510	11.94	1.301
A-14-10	0.398	12.28	1.589
A-15-1	0.371	13.41	1.738
A-15-2	0.396	13.60	1.749
A-15-3	0.372	13.85	1.698
A-15-4	0.399	13.93	1.827
A-15-5	0.371	13.85	1.682
A-16-1	0.403	12.41	1.853
A-16-2	0.402	12.55	1.561
A-16-3	0.401	12.73	1.465
A-16-4	0.420	12.58	1.552
A-16-5	0.408	12.66	1.661
A-17-1	0.468	12.61	1.742
A-17-2	0.420	12.15	1.811
A-17-3	0.423	12.55	1.773
A-17-4	0.449	12.63	1.601
A-17-5	0.413	12.49	1.932
A-18-1	0.485	12.01	1.528
A-18-2	0.476	12.21	2.399
A-18-3	0.479	12.26	1.918
A-18-4	*0.382*	*11.84*	*2.045*
A-18-5	0.394	12.06	1.492
B-1-1	0.414	12.65	1.690
B-1-2	0.399	12.40	1.691
B-1-3	0.345	12.02	1.636
B-1-4	0.382	12.27	1.499
B-1-5	0.376	12.05	1.656
B-2-1	0.483	13.05	1.848
B-2-2	0.448	12.99	2.059
B-2-3	0.419	12.96	1.772
B-2-4	0.386	12.60	1.707
B-2-5	0.419	12.82	1.839
B-2-6	0.442	12.86	1.965

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY E($\times10^6$psi)
B-2-7	0.400	13.36	1.687
B-2-8	0.403	13.27	1.689
B-2-9	0.424	13.57	1.723
B-2-10	0.413	12.76	1.812
C-1-1	0.516	13.19	2.175
C-1-2	0.442	12.91	2.140
C-1-3	0.381	12.81	1.633
C-1-4	0.447	12.92	2.052
C-1-5	0.459	13.35	1.660
C-2-1	0.384	14.26	1.702
C-2-2	0.390	14.07	1.718
C-2-3	0.365	13.96	1.819
C-2-4	0.391	14.23	1.656
C-2-5	0.404	12.34	1.644
C-2-6	0.397	14.51	1.491
C-2-7	0.422	14.23	1.703
C-2-8	0.412	14.26	1.701
C-2-9	0.384	14.48	1.625
C-2-10	0.384	13.94	1.584
C-3-1	0.378	14.24	1.639
C-3-2	0.388	14.24	1.671
C-3-3	0.374	14.19	1.510
C-3-4	0.365	13.81	1.905
C-3-5	0.359	13.85	1.625
C-4-1	0.404	14.93	1.391
C-4-2	0.376	14.48	1.570
C-4-3	0.368	14.07	1.840
C-4-4	0.372	14.36	1.701
C-4-5	0.372	14.07	1.790
C-5-1	0.379	14.38	1.887
C-5-2	0.351	14.06	1.403
C-5-3	0.412	14.65	1.548
C-5-4	0.380	14.14	1.626
C-5-5	0.376	14.30	1.808
C-6-1	0.435	13.05	1.678
C-6-2	0.393	13.05	1.793
C-6-3	0.397	13.24	1.610
C-6-4	0.433	13.24	1.666
C-6-5	0.344	12.81	1.622

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY $E(x10^6 psi)$
C-7-1	0.432	13.32	1.688
C-7-2	0.358	12.97	1.542
C-7-3	0.359	12.77	1.488
C-7-4	0.400	13.00	1.768
C-7-5	0.397	12.93	1.393
C-8-1	0.449	12.84	1.972
C-8-2	0.431	12.76	1.270
C-8-3	0.413	12.59	1.443
C-8-4	0.449	10.47	1.823
C-8-5	0.348	12.40	1.447
C-9-1	0.370	12.64	1.596
C-9-2	0.416	12.51	1.570
C-9-3	0.364	12.16	1.645
C-9-4	0.357	12.15	1.836
C-9-5	0.340	12.07	1.519
C-10-1	0.422	14.15	1.139
C-10-2	0.441	14.02	1.349
C-10-3	0.403	14.08	1.388
C-10-4	0.371	13.81	1.385
C-10-5	0.360	13.74	1.493
C-10-6	0.386	14.28	1.420
C-10-7	0.402	14.59	1.602
C-10-8	0.415	14.63	1.537
C-10-9	0.393	14.64	1.463
C-10-10	0.393	14.70	1.469
D-1-1	0.385	12.60	1.712
D-1-2	0.435	12.82	2.104
D-1-3	0.395	12.73	1.517
D-1-4	0.393	13.04	1.925
D-1-5	0.438	12.91	1.903
D-2-1	0.400	13.44	1.683
D-2-2	0.399	13.37	1.748
D-2-3	0.402	13.41	1.599
D-2-4	0.441	13.30	1.584
D-2-5	0.424	13.63	1.692
D-3-1	0.373	11.92	1.621
D-3-2	0.353	11.29	1.423
D-3-3	0.355	11.72	1.384
D-3-4	0.376	11.78	1.576
D-3-5	0.383	11.88	1.702

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY $E(x10^6 psi)$
D-4-1	0.416	12.85	1.777
D-4-2	0.412	12.98	1.863
D-4-3	0.411	12.89	1.915
D-4-4	0.420	13.02	1.452
D-4-5	0.407	12.82	1.882
E-1-1	0.416	13.18	1.897
E-1-2	0.404	13.14	1.812
E-1-3	0.438	13.00	1.965
E-1-4	0.388	13.17	1.785
E-1-5	0.392	12.77	1.433
E-2-1	0.377	12.21	1.894
E-2-2	0.406	12.29	2.315
E-2-3	0.373	12.27	1.971
E-2-4	0.399	12.27	1.456
E-2-5	0.353	12.14	1.397
E-3-1	0.380	13.00	1.841
E-3-2	0.399	13.03	1.313
E-3-3	0.433	12.91	1.311
E-3-4	0.360	13.00	1.647
E-3-5	0.430	12.71	1.511
F-1-1	0.446	13.11	2.232
F-1-2	0.441	12.85	2.115
F-1-3	0.376	13.34	1.737
F-1-4	0.368	13.00	1.547
F-1-5	0.386	13.37	1.633
F-1-6	0.392	12.65	1.440
F-1-7	0.386	14.10	1.420
F-1-8	0.449	12.50	1.601
F-1-9	0.423	12.55	1.811
F-1-10	0.407	11.73	1.650
F-2-1	0.373	12.13	1.819
F-2-2	0.382	12.26	1.743
F-2-3	0.382	12.07	1.524
F-2-4	0.393	11.67	1.799
F-2-5	0.369	11.56	1.358
F-2-6	0.416	10.28	1.550
F-2-7	0.376	12.21	1.875
F-2-8	0.389	13.80	1.625
F-2-9	0.381	13.20	2.079
F-2-10	0.440	13.63	1.991

TABLE A.1 (cont'd)

CONNECTION NUMBER	SPECIFIC GRAVITY	MOISTURE CONTENT(%)	MODULUS OF ELASTICITY $E(x10^6psi)$
G-1	0.390	11.57	1.808
G-2	0.416	10.29	1.545
G-3	0.438	11.69	1.642
G-4	0.407	11.74	1.630
G-5	0.423	12.25	1.839
G-6	0.371	14.32	1.313
G-7	0.408	13.53	1.823
G-8	0.391	13.63	1.356
G-9	0.310	13.12	1.527
G-10	0.343	14.15	1.337

TABLE A.2 RESULTS OF COMPRESSION TESTS - COLUMN SPECIMENS

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS Fu, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
Al-1	12.4	0.38	0.24	1.683	1.481	5.209	3.959
Al-2	12.0	0.37	0.19	1.871	1.684	5.341	4.326
Al-3	12.4	0.36	0.10	1.439	1.367	4.624	4.162
Al-4	12.3	0.37	0.17	2.056	1.882	4.812	3.993
Al-5	12.2	0.36	0.38	1.831	1.483	4.770	2.957
Al-6	12.3	0.37	0.26	1.747	1.520	5.523	4.087
Al-7	12.4	0.40	0.26	1.977	1.720	5.072	3.753
Al-8	11.7	0.34	0.24	1.653	1.454	4.535	3.446
Al-9	12.4	0.43	0.05	1.474	1.437	5.597	5.317
Al-10	11.5	0.36	0.24	1.594	1.402	5.329	4.050
Al-11	12.1	0.38	0.05	1.962	1.913	5.566	5.287
Al-12	11.9	0.37	0.20	1.750	1.575	5.134	4.107
Al-13	11.7	0.38	0.24	1.807	1.590	5.309	4.034
Al-14	11.5	0.38	0.26	1.770	1.557	4.980	3.685
Al-15	12.2	0.42	0.26	2.171	1.888	5.162	3.819
Al-16	12.3	0.39	0.05	1.838	1.792	5.080	4.826
Al-17	11.9	0.35	0.19	1.730	1.565	4.283	3.469
Al-18	12.4	0.37	0.20	1.765	1.589	4.758	3.806
Al-19	11.9	0.40	0.09	1.644	1.570	6.016	5.474
Al-20	12.4	0.41	0.26	1.503	1.307	5.423	4.013
Al-21	12.1	0.39	0.14	1.910	1.728	5.710	4.910
Al-22	11.8	0.35	0.19	1.526	1.381	4.966	4.022
Al-23	11.8	0.37	0.29	1.672	1.429	5.614	3.985
Al-24	12.3	0.40	0.21	1.595	1.428	5.750	4.543
Al-25	12.3	0.40	0.24	1.895	1.667	5.875	4.465
Al-26	12.0	0.37	0.29	1.652	1.412	5.416	3.845
Al-27	12.1	0.37	0.29	1.493	1.578	5.151	3.657
Al-28	12.3	0.41	0.29	1.899	1.623	6.033	4.283
Al-29	12.1	0.41	0.24	1.754	1.543	5.617	4.268
Al-30	11.7	0.38	0.34	1.870	1.552	5.675	3.764

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS Fu, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
A2-1	11.9	0.40	0.14	1.842	1.713	6.180	5.314
A2-2	12.0	0.39	0.08	1.976	1.896	6.027	5.544
A2-3	11.9	0.35	0.22	1.684	1.498	5.352	4.174
A2-4	12.3	0.42	0.12	2.072	1.947	6.004	5.283
A2-5	11.8	0.40	0.21	2.152	1.926	6.659	5.269
A2-6	12.2	0.38	0.07	1.753	1.692	5.560	5.171
A2-7	12.2	0.35	0.19	1.734	1.569	5.101	4.131
A2-8	13.1	0.37	0.13	1.553	1.452	4.962	4.316
A2-9	12.9	0.37	0.24	1.761	1.549	5.479	4.164
A2-10	13.4	0.37	0.23	1.797	1.590	5.164	3.976
A2-11	13.3	0.40	0.14	1.456	1.354	5.283	4.643
A2-12	12.4	0.38	0.16	1.567	1.442	4.975	4.179
A2-13	12.3	0.36	0.19	1.393	1.260	4.474	3.623
A2-14	12.4	0.38	0.14	1.533	1.425	5.006	4.305
A2-15	11.8	0.37	0.34	1.559	1.293	4.847	3.199
A2-16	12.2	0.41	0.19	1.785	1.615	5.771	4.674
A2-17	11.9	0.33	0.24	1.637	1.440	4.729	3.594
A2-18	12.1	0.36	0.20	1.625	1.463	5.130	4.104
A2-19	12.5	0.43	0.10	1.051	0.998	4.871	4.383
A2-20	11.9	0.35	0.26	1.752	1.524	4.662	3.449
A2-21	12.2	0.35	0.19	1.601	1.448	4.462	3.614
A2-22	12.7	0.40	0.14	1.693	1.574	5.323	4.577
A2-23	12.4	0.39	0.21	1.697	1.518	4.842	3.825
A2-24	12.0	0.38	0.24	1.570	1.382	4.752	3.612
A2-25	12.1	0.41	0.19	1.769	1.600	5.770	4.673
A2-26	12.2	0.36	0.19	1.540	1.393	4.687	3.796
A2-27	11.8	0.37	0.09	1.615	1.542	5.022	4.570
A2-28	12.4	0.40	0.29	1.553	1.327	4.960	3.521
A2-29	12.4	0.46	0.24	1.818	1.599	5.064	3.848
A2-30	12.3	0.41	0.33	1.827	1.526	5.215	3.494

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATION	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
B1-1	13.2	0.39	0.20	1.722	1.550	4.910	3.928
B1-2	13.3	0.38	0.18	1.814	1.650	5.094	4.177
B1-3	13.1	0.38	0.22	1.680	1.495	4.992	3.894
B1-4	13.1	0.37	0.20	1.693	1.523	4.776	3.823
B1-5	12.9	0.38	0.16	1.762	1.621	5.176	4.348
B1-6	12.8	0.43	0.24	1.492	1.312	5.659	4.300
B1-7	13.3	0.43	0.16	1.490	1.370	5.677	4.768
B1-8	13.3	0.47	0.16	1.840	1.692	6.495	5.425
B1-9	13.1	0.41	0.19	1.553	1.405	5.655	4.580
B1-10	12.9	0.41	0.19	1.645	1.488	5.846	4.735
B1-11	12.9	0.42	0.21	1.790	1.602	5.886	4.649
B1-12	13.2	0.42	0.24	1.941	1.708	5.778	4.391
B1-13	13.0	0.41	0.19	1.835	1.660	5.634	4.563
B1-14	13.1	0.43	0.13	1.672	1.563	6.009	5.227
B1-15	13.2	0.39	0.20	1.713	1.541	5.586	4.468
B1-16	11.1	0.42	0.23	1.992	1.762	6.044	4.653
B1-17	10.9	0.44	0.14	2.213	2.058	7.141	6.141
B1-18	10.9	0.38	0.24	1.820	1.601	5.862	4.455
B1-19	11.0	0.43	0.07	1.949	1.880	6.585	6.124
B1-20	11.0	0.43	0.17	1.980	1.811	6.853	5.687
B1-21	12.8	0.36	0.23	1.516	1.341	5.073	3.907
B1-22	13.2	0.45	0.29	1.452	1.241	5.910	4.196
B1-23	13.0	0.39	0.20	1.563	1.406	5.383	4.306
B1-24	13.1	0.38	0.27	1.420	1.157	4.754	3.470
B1-25	12.9	0.38	0.25	1.785	1.561	5.267	3.950

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
B2-1	13.5	0.39	0.22	1.859	1.655	5.351	4.174
B2-2	13.3	0.41	0.24	1.985	1.747	5.844	4.441
B2-3	12.9	0.41	0.20	1.989	1.790	6.088	4.870
B2-4	12.9	0.37	0.17	1.763	1.613	5.382	4.467
B2-5	13.0	0.38	0.19	1.830	1.656	5.596	4.533
B2-6	10.8	0.41	0.18	2.030	1.847	6.464	5.300
B2-7	10.7	0.42	0.33	2.021	1.687	6.386	4.278
B2-8	10.7	0.43	0.17	2.110	1.930	6.702	5.562
B2-9	10.8	0.42	0.24	1.934	1.701	6.568	4.991
B2-10	10.9	0.42	0.21	2.018	1.806	6.494	5.130
B2-11	10.8	0.41	0.20	1.840	1.656	5.943	4.754
B2-12	10.9	0.41	0.13	1.608	1.503	5.984	5.206
B2-13	11.0	0.43	0.24	1.709	1.503	5.854	4.449
B2-14	10.8	0.42	0.20	2.050	1.845	6.523	5.218
B2-15	10.8	0.41	0.19	1.947	1.762	6.283	5.089
B2-16	10.6	0.41	0.21	1.724	1.542	6.282	4.962
B2-17	10.7	0.41	0.21	1.973	1.765	6.318	4.991
B2-18	10.9	0.40	0.18	1.344	1.223	5.472	4.487
B2-19	10.9	0.39	0.19	1.673	1.514	5.661	4.585
B2-20	10.6	0.39	0.36	1.604	1.347	5.918	3.787
B2-21	13.2	0.40	0.24	1.720	1.513	5.194	3.947
B2-22	13.1	0.38	0.23	1.530	1.354	4.831	3.719
B2-23	12.9	0.38	0.24	1.606	1.413	4.863	3.695
B2-24	12.9	0.35	0.16	1.489	1.369	4.663	3.916
B2-25	12.7	0.38	0.10	1.539	1.462	5.198	4.678

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS Fu, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
B3-1	12.5	0.42	0.08	2.102	2.017	5.944	5.468
B3-2	12.7	0.40	0.05	1.790	1.745	5.722	5.435
B3-3	12.7	0.43	0.01	1.803	1.793	5.765	5.707
B3-4	12.4	0.41	0.02	2.000	1.980	5.765	5.649
B3-5	12.6	0.42	0.11	1.795	1.696	5.961	5.305
B3-6	12.7	0.39	0.11	1.926	1.820	5.273	4.692
B3-7	12.8	0.40	0.09	1.678	1.602	5.369	4.885
B3-8	12.7	0.41	0.06	2.102	2.038	5.689	5.347
B3-9	12.7	0.38	0.17	1.488	1.361	5.280	4.382
B3-10	12.7	0.39	0.04	1.836	1.799	5.280	4.963
B3-11	12.5	0.41	0.13	2.140	2.000	6.306	5.486
B3-12	12.8	0.44	0.13	2.009	1.878	6.436	5.599
B3-13	12.7	0.46	0.13	1.966	1.838	6.339	5.514
B3-14	12.7	0.39	0.16	1.573	1.447	5.252	4.411
B3-15	12.7	0.40	0.11	1.883	1.779	5.518	4.911
B3-16	12.6	0.40	0.19	2.106	1.905	5.896	4.775
B3-17	12.7	0.42	0.20	2.007	1.806	5.834	4.667
B3-18	12.8	0.42	0.21	2.062	1.845	5.716	4.515
B3-19	12.8	0.41	0.11	1.752	1.655	5.506	4.900
B3-20	12.6	0.39	0.20	1.666	1.499	5.295	4.236
B3-21	11.6	0.45	0.11	2.014	1.903	6.654	5.922
B3-22	11.3	0.44	0.14	2.087	1.940	6.549	5.632
B3-23	11.3	0.41	0.17	1.968	1.800	6.521	5.673
B3-24	11.2	0.43	0.19	1.962	1.775	6.995	5.665
B3-25	11.1	0.39	0.16	1.713	1.575	6.027	5.062

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
Cl-1	11.8	0.39	0.14	1.613	1.500	6.102	5.247
Cl-2	12.8	0.38	0.12	1.531	1.439	5.839	5.138
Cl-3	12.0	0.37	0.16	1.433	1.318	4.992	4.193
Cl-4	11.7	0.38	0.07	1.560	1.505	5.618	5.224
Cl-5	12.0	0.37	0.14	1.511	1.405	5.545	4.768
Cl-6	11.9	0.41	0.12	1.877	1.764	6.213	5.467
Cl-7	12.1	0.42	0.16	1.765	1.623	6.236	5.238
Cl-8	12.0	0.37	0.21	1.576	1.410	5.623	4.442
Cl-9	11.5	0.37	0.09	1.303	1.224	5.172	4.706
Cl-10	11.7	0.37	0.16	1.395	1.283	5.000	4.200
Cl-11	12.1	0.35	0.24	1.487	1.308	4.846	3.682
Cl-12	12.3	0.38	0.05	1.297	1.264	4.957	4.709
Cl-13	12.0	0.42	0.14	1.964	1.826	6.422	5.522
Cl-14	12.0	0.37	0.10	1.323	1.256	5.055	4.549
Cl-15	11.4	0.36	0.26	1.569	1.365	5.168	3.824
Cl-16	11.9	0.40	0.10	1.825	1.733	5.908	5.317
Cl-17	11.9	0.42	0.14	1.942	1.806	6.537	5.621
Cl-18	12.1	0.41	0.17	1.877	1.717	6.094	5.058
Cl-19	11.9	0.44	0.07	1.919	1.851	6.647	6.181
Cl-20	11.8	0.38	0.21	1.522	1.362	5.528	4.367
Cl-21	11.5	0.39	0.17	1.389	1.270	5.574	4.604
Cl-22	11.8	0.41	0.21	1.440	1.288	5.607	4.429
Cl-23	11.6	0.41	0.29	1.402	1.184	5.540	3.933
Cl-24	11.4	0.40	0.10	1.495	1.420	5.900	5.310
Cl-25	11.2	0.43	0.10	1.603	1.522	6.580	5.922

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
C2-1	11.9	0.43	0.19	2.014	1.812	6.630	5.370
C2-2	12.2	0.40	0.22	1.723	1.533	5.695	4.442
C2-3	12.1	0.39	0.17	1.481	1.347	5.394	4.477
C2-4	12.0	0.37	0.12	1.903	1.788	5.756	5.065
C2-5	11.6	0.35	0.09	1.207	1.146	4.493	4.088
C2-6	12.0	0.44	0.10	1.966	1.867	6.452	5.806
C2-7	12.3	0.41	0.21	1.449	1.289	5.348	4.224
C2-8	12.3	0.41	0.10	1.760	1.672	6.021	5.418
C2-9	12.1	0.38	0.16	1.525	1.403	5.279	4.434
C2-10	11.7	0.35	0.12	1.649	1.550	5.165	4.545
C2-11	12.2	0.42	0.16	1.616	1.486	6.272	5.268
C2-12	12.1	0.43	0.17	2.288	2.082	6.655	5.523
C2-13	12.4	0.44	0.14	1.406	1.307	5.991	5.152
C2-14	12.0	0.35	0.14	1.388	1.290	4.947	4.254
C2-15	11.8	0.35	0.22	1.649	1.467	5.313	4.144
C2-16	12.0	0.40	0.16	2.007	1.846	6.306	5.297
C2-17	12.0	0.42	0.09	1.782	1.692	6.228	5.667
C2-18	12.0	0.39	0.05	1.667	1.616	5.735	5.448
C2-19	12.0	0.41	0.17	1.748	1.591	5.864	4.867
C2-20	11.9	0.40	0.19	1.835	1.651	6.179	5.004
C2-21	12.0	0.39	0.16	1.424	1.402	5.486	4.608
C2-22	12.0	0.40	0.10	1.567	1.488	5.841	5.256
C2-23	12.2	0.43	0.14	1.549	1.440	5.791	4.980
C2-24	11.7	0.42	0.17	1.924	1.750	6.691	5.553
C2-25	11.8	0.43	0.12	1.624	1.526	6.126	5.390

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
C3-1	12.3	0.42	0.10	2.055	1.952	6.219	5.597
C3-2	12.6	0.38	0.05	1.673	1.622	5.499	5.224
C3-3	12.3	0.41	0.10	2.047	1.944	6.158	5.542
C3-4	12.3	0.37	0.07	1.644	1.578	5.378	5.001
C3-5	12.0	0.36	0.09	1.424	1.352	4.935	4.490
C3-6	12.5	0.38	0.16	1.406	1.293	5.097	4.281
C3-7	12.6	0.39	0.10	1.703	1.617	5.777	5.199
C3-8	12.5	0.40	0.12	1.675	1.574	5.612	4.938
C3-9	12.4	0.40	0.10	1.723	1.636	5.784	5.205
C3-10	12.7	0.40	0.12	1.552	1.458	5.471	4.814
C3-11	12.5	0.43	0.10	1.698	1.613	6.019	5.417
C3-12	12.3	0.42	0.16	1.943	1.786	6.050	5.082
C3-13	12.6	0.42	0.09	1.730	1.634	6.047	5.502
C3-14	12.3	0.37	0.10	1.267	1.203	5.000	4.500
C3-15	12.2	0.41	0.10	1.694	1.609	5.990	5.391
C3-16	12.4	0.43	0.09	1.823	1.731	6.029	5.486
C3-17	12.4	0.39	0.12	1.537	1.444	5.416	4.766
C3-18	12.4	0.40	0.09	1.742	1.654	5.946	5.410
C3-19	12.5	0.43	0.14	1.837	1.708	5.892	5.067
C3-20	12.4	0.41	0.03	1.834	1.797	6.174	5.988
C3-21	11.9	0.43	0.12	1.626	1.528	6.135	5.398
C3-22	12.2	0.45	0.10	1.776	1.687	6.479	5.831
C3-23	12.4	0.47	0.09	1.541	1.463	5.885	5.355
C3-24	12.4	0.44	0.10	1.387	1.317	5.496	4.946
C3-25	12.0	0.40	0.40	1.501	1.425	5.638	5.130

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
D-1	11.3	0.41	0.08	1.738	1.668	5.522	5.080
D-2	11.4	0.37	0.07	1.411	1.354	5.123	4.764
D-3	11.5	0.42	0.13	1.844	1.714	6.514	5.667
D-4	11.3	0.42	0.08	1.914	1.875	6.450	5.934
D-5	11.0	0.41	0.14	1.826	1.698	6.377	5.484
D-6	11.1	0.40	0.15	1.754	1.613	6.125	5.206
D-7	10.9	0.38	0.12	1.621	1.523	5.682	5.000
D-8	11.0	0.41	0.13	1.795	1.669	6.276	5.460
D-9	10.8	0.40	0.11	1.811	1.702	5.987	5.328
D-10	10.8	0.38	0.16	1.671	1.537	5.909	4.963
D-11	11.7	0.42	0.16	1.774	1.632	6.193	5.202
D-12	11.6	0.43	0.20	1.886	1.697	6.448	5.158
D-13	11.6	0.41	0.25	1.871	1.627	6.063	4.547
D-14	11.5	0.42	0.11	1.914	1.548	6.134	5.459
D-15	11.5	0.40	0.09	1.486	1.411	5.414	4.926
D-16	10.9	0.43	0.08	1.929	1.851	6.886	6.335
D-17	10.8	0.42	0.10	1.881	1.786	6.676	6.008
D-18	10.7	0.42	0.06	1.830	1.751	6.693	6.291
D-19	10.6	0.42	0.10	1.892	1.797	6.638	5.974
D-20	10.3	0.39	0.11	1.587	1.491	6.243	5.556
D-21	11.0	0.44	0.16	1.822	1.676	6.466	5.431
D-22	10.9	0.43	0.18	1.814	1.651	6.702	5.495
D-23	10.9	0.46	0.16	1.986	1.827	7.397	6.213
D-24	10.5	0.45	0.14	2.032	1.889	6.931	5.960
D-25	10.7	0.44	0.18	1.778	1.617	6.752	5.536

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
E1-1	12.3	0.37	0.09	1.807	1.717	5.331	4.851
E1-2	12.9	0.43	0.12	1.964	1.846	6.547	5.761
E1-3	12.9	0.41	0.17	1.935	1.761	6.244	5.183
E1-4	12.4	0.45	0.09	2.495	2.370	7.152	6.508
E1-5	12.9	0.43	0.14	1.654	1.538	5.899	5.073
E1-6	13.0	0.41	0.21	1.586	1.412	5.318	4.201
E1-7	12.0	0.42	0.12	2.269	2.133	6.605	5.812
E1-8	12.1	0.44	0.11	2.234	2.100	6.548	5.828
E1-9	12.1	0.45	0.14	2.201	2.047	6.796	5.845
E1-10	08.4	0.39	0.15	2.062	1.897	7.275	6.184
E1-11	08.6	0.41	0.21	2.049	1.824	7.089	5.600
E1-12	08.6	0.42	0.18	1.945	1.770	7.382	6.053
E1-13	08.7	0.47	0.09	2.197	2.087	7.768	7.069
E1-14	08.6	0.44	0.18	2.043	1.859	6.624	5.432
E1-15	08.7	0.41	0.18	1.342	1.221	6.207	5.090

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
E2-1	12.7	0.41	0.18	1.714	1.560	6.026	4.941
E2-2	12.7	0.40	0.05	1.660	1.610	5.843	5.551
E2-3	12.7	0.40	0.14	1.386	1.289	5.442	4.680
E2-4	13.0	0.43	0.05	1.577	1.530	5.700	5.415
E2-5	13.1	0.40	0.17	1.645	1.679	5.444	4.519
E2-6	12.7	0.34	0.09	1.380	1.311	4.706	4.282
E2-7	12.0	0.42	0.11	1.657	1.558	1.5893	5.245
E2-8	12.1	0.43	0.09	1.790	1.701	6.597	6.003
E2-9	12.0	0.43	0.09	1.828	1.737	6.581	5.989
E2-10	08.6	0.45	0.15	1.934	1.779	7.857	6.678
E2-11	08.6	0.41	0.17	1.888	1.718	6.971	5.786
E2-12	08.6	0.44	0.09	2.053	1.950	7.330	6.670
E2-13	08.5	0.42	0.17	2.216	2.017	7.223	5.995
E2-14	08.6	0.40	0.15	1.700	1.564	6.537	5.556
E2-15	08.3	0.45	0.12	2.147	2.018	7.791	6.856

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E,(x10^6 lb/in^2)		ULTIMATE STRESS Fu, (x10^3 lb/in^2)	
				Initial	Reduced	Initial	Reduced
F-1	14.7	0.35	0.14	1.444	1.342	4.260	3.663
F-2	14.8	0.36	0.18	1.558	1.417	4.872	3.995
F-3	14.8	0.45	0.05	1.892	1.835	6.425	6.103
F-4	14.4	0.40	0.14	1.805	1.678	5.339	8.591
F-5	13.4	0.38	0.09	1.976	1.877	5.596	5.092
F-6	14.1	0.41	0.18	1.873	1.704	5.526	4.531
F-7	13.5	0.38	0.09	1.724	1.637	5.368	4.884
F-8	13.4	0.31	0.14	1.195	1.111	4.011	3.449
F-9	13.0	0.36	0.11	1.521	1.429	5.048	4.492
F-10	13.1	0.38	0.09	1.998	1.848	6.129	5.577
F-11	14.1	0.41	0.14	1.713	1.593	5.289	4.548
F-12	13.8	0.35	0.07	1.473	1.414	4.664	4.337
F-13	13.8	0.38	0.14	1.391	1.293	4.697	4.039
F-14	13.9	0.44	0.09	1.819	1.728	6.134	5.581
F-15	14.5	0.42	0.18	1.015	0.923	4.610	3.780
F-16	13.4	0.41	0.27	1.626	1.398	5.538	4.042
F-17	13.4	0.41	0.11	1.748	1.643	5.662	5.039
F-18	12.8	0.38	0.14	1.664	1.547	5.507	4.736
F-19	14.5	0.45	0.05	2.150	2.085	6.273	5.959
F-20	14.4	0.39	0.25	1.592	1.385	4.838	3.628
F-21	14.4	0.39	0.20	1.468	1.321	4.349	3.479
F-22	14.2	0.37	0.18	1.508	1.372	4.658	3.819
F-23	14.0	0.39	0.18	1.689	1.536	5.059	4.148
F-24	14.5	0.46	0.11	1.845	1.734	5.474	4.871
F-25	14.0	0.38	0.14	1.549	1.440	5.062	4.353
F-26	14.4	0.38	0.18	1.312	1.193	4.362	3.576
F-27	14.7	0.42	0.14	1.745	1.623	5.264	4.475
F-28	14.4	0.44	0.18	1.294	1.177	4.948	4.057
F-29	14.0	0.38	0.14	1.517	1.410	4.626	3.978
F-30	14.0	0.43	0.14	1.782	1.657	5.625	4.837

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
G-1	13.1	0.39	0.10	1.656	1.573	5.183	4.665
G-2	12.9	0.30	0.12	1.437	1.351	5.210	4.584
G-3	12.9	0.43	0.13	1.960	1.823	6.210	5.402
G-4	12.9	0.39	0.08	1.533	1.472	5.452	5.015
G-5	12.9	0.37	0.11	1.348	1.267	4.676	4.162
G-6	12.9	0.42	0.07	1.647	1.581	5.712	5.312
G-7	12.7	0.40	0.14	1.322	1.229	4.923	4.233
G-8	12.9	0.36	0.12	1.214	1.141	4.381	3.855
G-9	12.8	0.45	0.09	1.402	1.332	5.167	4.701
G-10	12.5	0.40	0.16	1.663	1.529	5.091	4.276
G-11	12.7	0.41	0.13	1.839	1.710	5.688	4.949
G-12	12.7	0.39	0.10	1.342	1.275	4.996	4.496
G-13	12.6	0.40	0.10	1.664	1.581	5.183	4.665
G-14	12.6	0.39	0.06	1.583	1.536	5.253	4.937
G-15	12.4	0.41	0.04	1.834	1.797	5.653	5.427

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
H1-1	13.6	0.41	0.21	1.553	1.382	5.278	4.169
H1-2	13.9	0.40	0.21	1.296	1.241	4.635	3.661
H1-3	13.6	0.36	0.25	1.304	1.134	4.880	3.660
H1-4	13.4	0.40	0.11	1.309	1.230	5.137	4.571
H1-5	13.4	0.41	0.14	1.708	1.588	5.282	4.542
H1-6	13.4	0.37	0.07	1.332	1.278	4.836	4.497
H1-7	13.5	0.40	----	1.686	1.686	5.735	5.735
H1-8	13.9	0.43	----	1.752	1.752	6.216	6.216
H1-9	13.8	0.41	----	1.771	1.771	5.826	5.826
H1-10	08.6	0.47	----	2.042	2.042	7.993	7.993
H1-11	08.6	0.40	0.07	1.914	1.837	6.775	4.440
H1-12	08.7	0.44	----	1.771	1.771	6.820	6.820
H1-13	08.5	0.40	0.14	1.896	1.763	6.562	5.643
H1-14	08.7	0.46	0.14	1.977	1.838	7.602	6.537
H1-15	08.6	0.41	0.11	1.901	1.786	6.909	6.149

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
H2-1	12.3	0.38	0.18	1.315	1.196	5.063	4.152
H2-2	12.3	0.39	0.18	1.753	1.595	5.752	4.717
H2-3	13.0	0.42	0.20	1.402	1.261	5.263	4.210
H2-4	12.6	0.46	0.21	2.017	1.795	5.948	4.728
H2-5	12.4	0.38	0.16	2.088	1.920	5.320	4.469
H2-6	12.3	0.40	0.18	1.564	1.423	5.698	4.672
H2-7	12.3	0.41	0.22	1.984	1.766	6.969	5.435
H2-8	12.4	0.41	0.18	2.156	1.961	6.792	5.569
H2-9	12.6	0.41	0.17	1.457	1.325	5.314	4.410
H2-10	12.5	0.41	0.20	1.473	1.326	5.373	4.298

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E,(x10^6 lb/in^2)		ULTIMATE STRESS F_u, (x10^3 lb/in^2)	
				Initial	Reduced	Initial	Reduced
H3-1	13.8	0.43	0.21	1.507	1.341	5.023	3.968
H3-2	14.4	0.48	0.21	1.400	1.246	5.983	4.727
H3-3	13.7	0.38	0.25	1.240	1.078	4.634	3.476
H3-4	13.9	0.42	0.11	1.699	1.597	5.931	5.279
H3-5	13.9	0.43	0.25	1.873	1.816	5.808	4.356
H3-6	14.0	0.49	0.07	1.420	1.363	6.129	5.700
H3-7	13.6	0.37	----	1.850	1.850	5.513	5.513
H3-8	13.9	0.44	----	1.993	1.993	6.448	6.448
H3-9	13.8	0.44	----	1.223	1.223	4.741	4.741
H3-10	08.7	0.46	0.14	2.432	2.261	8.678	7.463
H3-11	08.7	0.49	0.07	2.275	2.184	8.843	8.224
H3-12	08.4	0.41	0.18	2.025	1.842	7.415	6.080
H3-13	08.6	0.40	0.14	1.892	1.759	6.768	5.820
H3-14	08.6	0.39	0.18	1.577	1.435	6.081	4.986
H3-15	08.7	0.41	0.11	1.786	1.678	6.785	6.174

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, (x10^6 lb/in^2)		ULTIMATE STRESS F_u, (x10^3 lb/in^2)	
				Intial	Reduced	Initial	Reduced
H4-1	12.8	0.44	0.22	1.590	1.415	5.977	4.662
H4-2	12.5	0.40	0.21	1.464	1.303	5.397	4.263
H4-3	12.7	0.39	0.18	2.735	2.489	5.795	4.751
H4-4	12.4	0.41	0.20	2.002	1.802	5.405	4.324
H4-5	12.4	0.40	0.17	1.683	1.531	5.596	4.644
H4-6	12.9	0.44	0.16	1.638	1.507	5.966	5.011
H4-7	12.7	0.40	0.10	1.855	1.669	6.058	5.452
H4-8	12.8	0.39	0.23	2.420	2.130	6.710	5.167
H4-9	12.9	0.41	0.22	1.791	1.594	5.684	4.433
H4-10	12.6	0.37	0.20	1.635	1.472	5.347	4.278

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($x10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($x10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
H5-1	12.1	0.35	0.19	1.648	1.483	5.571	4.513
H5-2	12.2	0.40	0.25	1.910	1.662	5.887	4.415
H5-3	12.6	0.39	0.22	1.386	1.234	5.338	4.164
H5-4	12.2	0.37	0.20	1.302	1.172	4.884	3.907
H5-5	12.5	0.41	0.18	1.666	1.516	5.354	4.390
H5-6	12.5	0.40	0.17	2.024	1.842	6.005	4.984
H5-7	12.5	0.41	0.21	1.703	1.516	4.776	3.773
H5-8	12.6	0.40	0.20	1.179	1.061	4.940	3.952
H5-9	12.6	0.39	0.12	1.069	1.005	5.369	4.724
H5-10	12.5	0.39	0.18	1.515	1.379	5.532	4.536

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E, ($\times 10^6$ lb/in^2)		ULTIMATE STRESS F_u, ($\times 10^3$ lb/in^2)	
				Initial	Reduced	Initial	Reduced
I1-1	13.2	0.39	0.25	1.619	1.408	5.456	4.092
I1-2	13.5	0.38	0.25	1.760	1.531	5.667	4.250
I1-3	13.8	0.45	0.21	1.572	1.399	5.826	4.602
I1-4	12.4	0.37	0.25	1.689	1.469	5.278	3.958
I1-5	13.1	0.43	0.11	1.724	1.620	6.227	5.542
I1-6	12.9	0.39	0.11	1.253	1.177	6.117	5.444
I1-7	12.9	0.42	0.18	1.885	1.715	5.866	4.810
I1-8	13.2	0.41	0.21	1.693	1.506	5.524	4.363
I1-9	13.2	0.42	0.21	1.303	1.159	5.251	4.148
I1-10	08.8	0.47	0.18	2.171	1.975	8.094	6.637
I1-11	08.8	0.41	0.18	2.095	1.906	7.371	6.044
I1-12	08.9	0.39	0.21	1.796	1.598	6.291	4.969
I1-13	08.3	0.41	0.18	1.688	1.536	6.921	5.675
I1-14	08.8	0.41	0.25	1.967	1.711	6.736	5.052
I1-15	08.9	0.42	0.14	1.208	1.123	6.961	5.986

TABLE A.2 (cont'd)

COLUMN NUMBER	MOISTURE CONTENT %	SPECIFIC GRAVITY	KNOT RATIO	MODULUS OF ELASTICITY E,(x10^6 lb/in^2)		ULTIMATE STRESS F_u,(x10^3 lb/in^2)	
				Initial	Reduced	Initial	Reduced
I2-1	13.6	0.44	0.18	2.058	1.872	6.121	5.019
I2-2	13.7	0.42	0.21	1.743	1.551	5.931	4.685
I2-3	13.2	0.37	0.14	1.866	1.735	5.915	5.086
I2-4	11.4	0.46	0.07	2.204	2.115	7.235	6.728
I2-5	12.6	0.38	0.11	1.870	1.757	5.871	5.225
I2-6	13.2	0.48	0.21	1.437	1.278	5.757	4.548
I2-7	13.1	0.44	0.14	2.098	1.951	6.524	5.610
I2-8	12.1	0.43	0.18	1.836	1.670	6.471	5.435
I2-9	12.9	0.43	0.29	2.115	1.797	6.437	4.570
I2-10	08.7	0.38	0.14	1.292	1.201	4.930	4.239
I2-11	08.8	0.41	0.18	2.010	1.829	6.774	5.554
I2-12	08.8	0.47	0.21	2.155	1.917	8.088	6.389
I2-13	08.9	0.38	0.25	1.331	1.157	5.823	4.367
I2-14	09.1	0.44	0.25	1.317	1.145	6.598	1.948
I2-15	08.7	0.41	0.07	2.096	2.012	6.928	6.443

TABLE A.3 RESULTS OF CONNECTION TESTS

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
A-1-1	11773	11727	12432	12217	12284
A-1-2	9803	9696	9648	10283	10363
A-1-3	11724	12010	12834	12300	12175
A-1-4	9827	10088	10171	10493	10237
A-1-5	11029	11057	11494	11541	11447
A-1-6	8959	9395	9250	9731	9947
A-1-7	7672	8116	7611	8498	8705
A-1-8	7593	8142	7643	8458	8994
A-1-9	9820	9615	9540	10197	10678
A-1-10	9812	9803	9790	10355	10654
A-2-1	11803	11989	12804	12340	12436
A-2-2	9050	9286	9108	9741	10253
A-2-3	13070	13092	14398	13482	14046
A-2-4	11391	11603	12258	11959	12436
A-2-5	10086	10002	10056	10575	11193
A-3-1	13131	12803	13976	13354	13775
A-3-2	10380	10472	10691	10950	11431
A-3-3	10554	11018	11440	11253	11722
A-3-4	11266	11454	12048	11832	12210
A-3-5	12343	12300	13249	12763	13103
A-4-1	12803	13025	12748	13727	13641
A-4-2	14120	13945	13963	14773	15545
A-4-3	15756	15669	16311	16409	16789
A-4-4	14361	14403	14578	15137	14390
A-4-5	13526	13532	13414	14314	14690
A-4-6	14267	14270	14357	15035	14400
A-4-7	13256	13259	13340	13969	14712
A-4-8	12608	12620	12687	13286	13816
A-4-9	11503	11506	11576	12122	12855
A-4-10	12893	12899	12974	13587	14700
A-5-1	14383	15052	15460	15402	16210
A-5-2	13856	14452	14644	14879	15670
A-5-3	14131	13944	13962	14774	15164
A-5-4	11976	11943	11356	12779	12436
A-5-5	11626	11864	11256	12600	12914
A-5-6	13236	13029	12754	13873	14411
A-5-7	14739	15323	15832	15711	16354
A-5-8	13836	13536	13419	14344	14923
A-5-9	14387	14398	14572	15146	15764
A-5-10	12173	12109	11567	12950	13705

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
A-6-1	13812	13786	13752	14572	15438
A-6-2	10377	10382	9422	11222	12210
A-6-3	14388	14766	15069	15306	15876
A-6-4	13931	14063	14121	14768	15176
A-6-5	10592	10838	9978	11597	11193
A-7-1	11444	11587	10907	12373	12436
A-7-2	11625	11786	11158	12559	13431
A-7-3	13911	13824	13801	14630	15263
A-7-4	8078	8369	7068	9124	9594
A-7-5	14250	14151	14239	14952	15989
A-8-1	14149	14161	14192	14893	14923
A-8-2	10970	11507	10807	12096	12210
A-8-3	16987	16780	17870	17527	16789
A-8-4	11366	11653	10990	12371	12210
A-8-5	17206	17154	18404	17813	17908
A-9-1	14752	14579	14816	15390	15438
A-9-2	15469	15082	15501	15833	15989
A-9-3	15021	14757	15058	15582	15670
A-9-4	15572	15480	16048	16229	15764
A-9-5	9519	9852	8786	10589	10660
A-10-1	13318	13061	12795	13890	13991
A-10-2	14537	14401	14574	15205	15332
A-10-3	14455	14422	14603	15188	14923
A-10-4	14855	14590	14831	15419	14923
A-10-5	16902	17256	18549	17679	17219
A-11-1	11591	11700	11049	12496	11598
A-11-2	10976	11112	10328	11917	10920
A-11-3	11392	11748	11110	12429	11479
A-11-4	12240	12715	12345	13294	12436
A-11-5	13226	13834	13814	14284	13103
A-12-1	15637	16340	14590	17480	16789
A-12-2	15839	16226	14455	17514	16789
A-12-3	16466	10461	14734	17841	17016
A-12-4	15474	15854	14015	17147	16353
A-12-5	18827	18782	17568	20175	19372

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
A-13-1	17853	18614	17114	20565	21029
A-13-2	19834	20485	19456	22544	22612
A-13-3	16920	18384	16839	20053	20189
A-13-4	22421	23412	23243	25421	22777
A-13-5	15399	18386	16838	18989	19412
A-14-1	28953	32365	29771	35276	34330
A-14-2	23482	26709	23037	29329	29689
A-14-3	21111	23524	19450	26190	27339
A-14-4	29037	31010	28118	34450	34648
A-14-5	22488	26249	22506	28477	33648
A-14-6	17776	20815	17957	22980	24496
A-14-7	28367	30236	29550	33605	30620
A-14-8	20007	23269	20837	25555	26626
A-14-9	19264	21894	19213	24347	24496
A-14-10	22786	25436	23466	28234	29161
A-15-1	30442	34730	30758	38572	37387
A-15-2	31392	34894	30952	38745	38824
A-15-3	29079	34138	30050	37527	35050
A-15-4	31101	36056	32333	39705	40376
A-15-5	30283	33890	29767	37657	37050
A-16-1	32120	40161	33675	43308	43276
A-16-2	28469	35312	28368	38766	40826
A-16-3	27371	33678	26624	37250	38274
A-16-4	28069	35158	28205	38409	40207
A-16-5	30484	37004	30185	40874	41464
A-17-1	35106	44725	34895	49072	46950
A-17-2	36789	45322	36277	50276	47971
A-17-3	36492	46048	35516	50624	47971
A-17-4	32467	41968	32071	46005	43888
A-17-5	39982	48332	38701	53630	51033
A-18-1	32877	46255	34077	48848	51033
A-18-2	53905	64871	53501	72023	65445
A-18-3	41935	57553	42775	59890	58323
A-18-4	44393	54853	45607	60627	61239
A-18-5	33641	45432	33274	49642	48031

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
B-1-1	9768	10668	10352	10643	11094
B-1-2	10421	10672	10358	11066	11598
B-1-3	10218	10412	10021	10852	11118
B-1-4	9246	9750	9182	10039	10419
B-1-5	10564	10507	10143	11072	11300
B-2-1	20009	20523	19525	23204	23059
B-2-2	22301	22262	21754	24896	24731
B-2-3	19194	19893	18721	22438	23059
B-2-4	18485	19340	18034	20577	21117
B-2-5	19913	20450	19430	23070	23553
B-2-6	21280	21826	20760	24322	26216
B-2-7	18269	18738	17823	20881	21115
B-2-8	18291	18760	17845	20906	19674
B-2-9	18659	19137	18204	21327	22105
B-2-10	19623	20126	19144	22429	22706
C-1-1	23197	23536	23363	23702	22484
C-1-2	22917	22962	22779	23320	23506
C-1-3	18710	18420	18264	17796	18469
C-1-4	22207	22389	22211	22361	22780
C-1-5	18941	18818	18654	18089	19153
C-2-1	37286	37683	37364	37864	35976
C-2-2	37916	37952	37716	38373	39743
C-2-3	40625	39608	39932	40647	37973
C-2-4	37918	36928	36354	38019	36308
C-2-5	35739	36720	36090	36536	30630
C-2-6	32854	34137	32732	33888	35976
C-2-7	38548	37699	37386	38702	38743
C-2-8	37716	37665	37342	38159	39503
C-2-9	35363	36408	35674	36196	37587
C-2-10	36198	35718	34774	36576	38743
C-3-1	43103	42732	41179	44176	42325
C-3-2	42600	43353	41983	44099	40865
C-3-3	41200	40178	37938	41946	38503
C-3-4	48756	47827	47862	49351	47663
C-3-5	41999	42448	40827	43434	43797
C-4-1	43224	43126	40571	45323	46883
C-4-2	47332	47205	45792	49244	51472
C-4-3	53404	53178	53667	54997	54564
C-4-4	51199	50127	49613	52422	54766
C-4-5	50652	52083	52209	52974	53128

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
C-5-1	65119	65841	64906	69243	67296
C-5-2	51349	52721	48258	56255	56080
C-5-3	53170	56749	53246	58918	59379
C-5-4	56921	58877	55929	61918	61178
C-5-5	61028	63765	62189	66154	64976
C-6-1	46916	48028	47014	49084	48991
C-6-2	49019	50489	50236	51178	53251
C-6-3	46944	46574	45109	48532	51033
C-6-4	46539	47778	46678	48769	49808
C-6-5	46861	46838	45445	48612	49331
C-7-1	93705	91723	93929	100039	96693
C-7-2	85564	85678	85805	92417	93859
C-7-3	82583	83430	82800	91608	94544
C-7-4	98124	94948	98381	102993	108069
C-7-5	77330	79417	77514	85635	88450
C-8-1	147565	151153	155073	166557	153098
C-8-2	102415	108658	99869	119911	119212
C-8-3	113578	119574	113474	132592	135408
C-8-4	133591	142494	143356	156592	137324
C-8-5	107391	119805	113788	130980	129578
C-9-1	130740	147091	132763	163632	157155
C-9-2	127205	145269	130600	161032	160304
C-9-3	134721	150411	136839	167428	152376
C-9-4	146868	163361	152728	181461	170614
C-9-5	124797	141691	126358	157522	150768
C-10-1	120407		127593		125223
C-10-2	150920		151117		132578
C-10-3	152156		155486		135977
C-10-4	155387		155150		147125
C-10-5	159477		167249		150023
C-10-6	156157		159071		147159
C-10-7	173874		179459		185789
C-10-8	163545		172178		171362
C-10-9	143028		163888		152176
C-10-10	156797		164560		178951

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
D-1-1	50733	52430	50377	49953	53074
D-1-2	54541	57352	58798	61390	55285
D-1-3	46759	48422	45999	44263	47387
D-1-4	54549	56355	54986	56168	55285
D-1-5	53781	55731	54527	55526	54173
D-2-1	93092	107159	94951	93160	106503
D-2-2	96692	109610	97692	96758	111344
D-2-3	88450	102506	91378	88511	106776
D-2-4	87629	98343	90741	87680	98285
D-2-5	93605	107536	95343	93659	106148
D-3-1	89675	100675	92325	89736	95261
D-3-2	78721	95020	83731	78776	94127
D-3-3	76573	93488	82011	76617	92396
D-3-4	87194	98395	90403	87245	96395
D-3-5	94147	103394	95757	94220	100892
D-4-1	154604	188711	159394	147201	181299
D-4-2	142302	182737	165130	154325	178752
D-4-3	153502	192874	168625	158632	180966
D-4-4	132104	164129	137025	120279	160491
D-4-5	150935	190228	166452	155899	186633
E-1-1	96995	97667	98154	107990	98000
E-1-2	92005	94356	93756	104008	97065
E-1-3	101294	100300	101672	111033	102437
E-1-4	94057	93326	92359	104062	95065
E-1-5	79361	79130	74146	89707	94048
E-2-1	101060	97535	97947	104906	106148
E-2-2	123549	113398	119719	123261	118395
E-2-3	105183	100504	101929	111129	110230
E-2-4	77709	80090	75296	89609	90724
E-2-5	74561	77645	72245	87846	85054
E-3-1	153092	153316	157346	177553	160108
E-3-2	118270	118980	112219	140344	136087
E-3-3	118478	118832	112049	138099	139987
E-3-4	135964	141002	140766	163522	150304
E-3-5	121238	132205	129142	150737	147207

TABLE A.3 (cont'd)

CONNECTION TYPE AND NUMBER	CONNECTOR MODULUS PER CONNECTOR, k (lb/in)				
	Single Shear			Double Shear	
	Kuenzi	Wilkinson	k=tdEx	Kuenzi	Experimental
F-1-1	31212	32823	33823	37101	36023
F-1-2	31984	31519	32049	37942	36274
F-1-3	23698	27186	26322	29433	27836
F-1-4	21687	24923	23442	27388	24626
F-1-5	20419	25954	24745	25990	27836
F-1-6	21612	23858	21821	26446	24732
F-1-7	21312	23527	21518	26079	23157
F-1-8	24028	26526	24261	29403	26897
F-1-9	27181	30005	27443	33260	30133
F-1-10	24764	27337	25003	30303	29544
F-2-1	58154	100644	59150	103621	98523
F-2-2	63314	77481	56679	110985	103460
F-2-3	55009	88125	49557	98241	93686
F-2-4	59153	99814	58499	103320	98065
F-2-5	46822	80833	44159	85233	88057
F-2-6	50438	83374	50403	89536	97714
F-2-7	61013	100856	60971	108309	96753
F-2-8	52878	87404	52842	93868	96106
F-2-9	67651	111828	67604	120093	99573
F-2-10	64789	107095	64743	115010	101247
G-1-1	36704	45974	36202	48595	45362
G-1-2	28252	40863	30937	41115	39692
G-1-3	27879	42785	32879	40675	39692
G-1-4	32474	42541	32639	45228	44228
G-1-5	37333	46569	36823	46043	44228
G-1-6	26174	36307	26291	36793	38766
G-1-7	36340	50410	36503	51084	41206
G-1-8	27031	37496	27152	37998	37351
G-1-9	30439	42225	30576	42790	37198
G-1-10	26652	36971	26772	37466	39222

APPENDIX "B"

BEAM ON AN ELASTIC FOUNDATION

The differential equation for the deflection curve of a beam supported on an elastic foundation is given by

$$EI \frac{d^4y}{dx^4} = -\bar{k}y$$

where

EI = stiffness of the beam in lb-in^2

E = modulus of elasticity in lb/in^2

I = moment of inertia in in.^{4}

y = deflection at point x in in.

$\bar{k}$ = foundation modulus in lb/in^2.

The beam on an elastic foundation solution, as applied to single shear and double shear frictionless nailed timber connections, has been theoretically solved by Kuenzi*. Kuenzi's relevant expressions are presented on the following pages.

Kuenzi, E.W., "Theoretical Design of a Nailed or Bolted Joint Under Lateral Load", United States Forest Products Laboratory Report No. D1951, 1953, Revised 1955, 31 pp.

SINGLE SHEAR CONNECTIONS

The following expressions are for deflection, moment and shear at any point along the nail. Figure B.1 indicates the notations used for this type of connection.

MEMBER 1

$$\lambda_1 = \sqrt[4]{\frac{k_1}{4EI}}$$

$$\Delta_1 = \text{Sinh}^2\lambda_1 a - \sin^2\lambda_1 a$$

Deflection

$$y_1 = -\frac{2P\lambda_1}{\Delta_1 k_1}\left(\text{Sinh}\lambda_1 a\ \text{Cosh}\lambda_1 x\cos\lambda_1(a-x) - \sin\lambda_1 a\cos\lambda_1 x\text{Cosh}\lambda_1(a-x)\right)$$

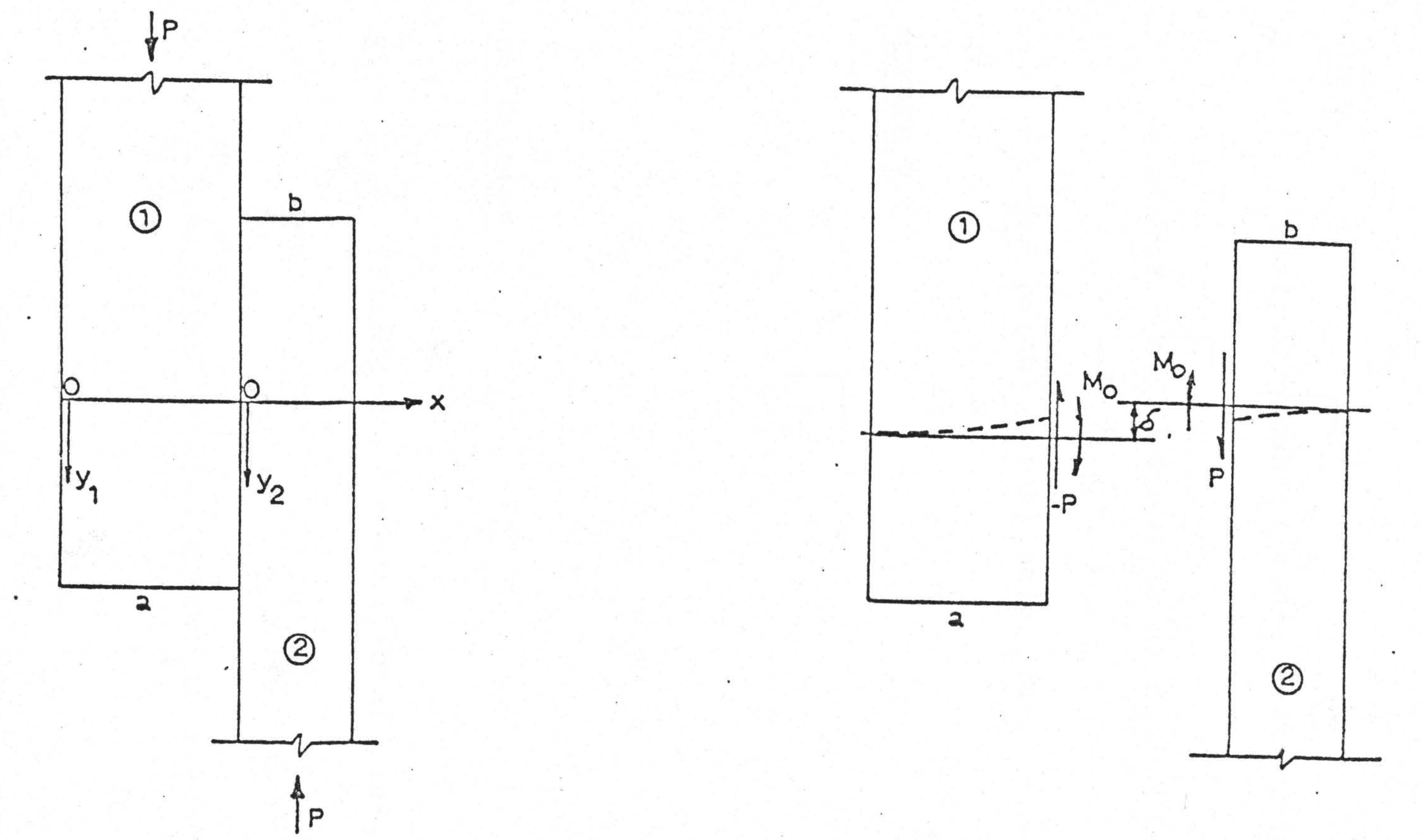
$$+\frac{2M_0\lambda_1{}^2}{\Delta_1 k_1}\left(\text{Sinh}\lambda_1 a\left[\text{Cosh}\lambda_1 x\sin\lambda_1(a-x) - \text{Sinh}\lambda_1 x\cos\lambda_1(a-x)\right]\right.$$

$$\left.+\sin\lambda_1 a\left[\text{Sinh}\lambda_1(a-x)\cos\lambda_1 x - \text{Cosh}\lambda_1(a-x)\sin\lambda_1 x\right]\right)$$

Moment

$$M_1 = \frac{P}{\Delta_1\lambda_1}\left(\text{Sinh}\lambda_1 a\text{Sinh}\lambda_1 x\sin\lambda_1(a-x) - \sin\lambda_1 a\sin\lambda_1 x\text{Sinh}\lambda_1(a-x)\right)$$

$$+\frac{M_0}{\Delta_1}\left(\text{Sinh}\lambda_1 a\left[\text{Sinh}\lambda_1 x\ \cos\lambda_1(a-x) + \text{Cosh}\lambda_1 x\sin\lambda_1(a-x)\right]\right.$$



A. CHOICE OF AXIS

B. FORCES AND DEFLECTIONS

NOTE

a or b is member thickness or amount of nail penetration

FIGURE B.1 Notation for Single Shear Connections

$$- \sin\lambda_1 a \left[\cos\lambda_1 x \mathrm{Sinh}\lambda_1(a-x) + \sin\lambda_1 x \,\mathrm{Cosh}\lambda_1(a-x)\right]\Bigg)$$

Shear

$$Q_1 = \frac{P}{\Delta_1}\left(\mathrm{Sinh}\lambda_1 a \left[\mathrm{Cosh}\lambda_1 x \sin\lambda_1(a-x) - \mathrm{Sinh}\lambda_1 x\cos\lambda_1(a-x)\right]\right.$$

$$\left. - \sin\lambda_1 a \left[\cos\lambda_1 x \mathrm{Sinh}\lambda_1(a-x) \; \sin\lambda_1 x \,\mathrm{Cosh}\lambda_1(a-x)\right]\right)$$

$$+ \frac{2M_0\lambda_1}{\Delta_1}\left(\mathrm{Sinh}\lambda_1 a \mathrm{Sinh}\lambda_1 x \sin\lambda_1(a-x) + \sin\lambda_1 a \sin\lambda_1 x \mathrm{Sinh}\lambda_1(a-x)\right)$$

MEMBER 2

$$\lambda_2 = \sqrt[4]{\frac{k_2}{4EI}}$$

$$\Delta_2 = \mathrm{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b$$

Deflection

$$y_2 = \frac{2P\lambda_2}{\Delta_2 k_2}\left(\mathrm{Sinh}\lambda_2 b\cos\lambda_2 x\mathrm{Cosh}\lambda_2(b-x) - \sin\lambda_2 b\mathrm{Cosh}\lambda_2 x\cos\lambda_2(b-x)\right)$$

$$+ \frac{2M_0\lambda_2{}^2}{\Delta_2 k_2}\left(\mathrm{Sinh}\lambda_2 b\left[\sin\lambda_2 x\mathrm{Cosh}\lambda_2(b-x) - \cos\lambda_2 x\mathrm{Sinh}\lambda_2(b-x)\right]\right.$$

$$\left. + \sin\lambda_2 b\left[\mathrm{Sinh}\lambda_2 x\cos\lambda_2(b-x) - \mathrm{Cosh}\lambda_2 x\sin\lambda_2(b-x)\right]\right)$$

Moment

$$M_2 = -\frac{P_2}{\Delta_2\lambda_2}\left(\text{Sinh}\lambda_2 b\sin\lambda_2 x\text{Sinh}\lambda_2(b-x) - \sin\lambda_2 b\text{Sinh}\lambda_2 x\sin\lambda_2(b-x)\right)$$

$$+\frac{M_0}{\Delta_2}\left(\text{Sinh}\lambda_2 b\left[\cos\lambda_2 x\text{Sinh}\lambda_2(b-x) + \sin\lambda_2 x\text{Cosh}\lambda_2(b-x)\right]\right.$$

$$\left.-\sin\lambda_2 b\left[\text{Sinh}\lambda_2 x\cos\lambda_2(b-x) + \text{Cosh}\lambda_2 x\sin\lambda_2(b-x)\right]\right)$$

Shear

$$Q_2 = -\frac{P}{\Delta_2}\left(\text{Sinh}\lambda_2 b\left[\cos\lambda_2 x\text{Sinh}\lambda_2(b-x) - \sin\lambda_2 x\text{Cosh}\lambda_2(b-x)\right]\right.$$

$$\left.-\sin\lambda_2 b\left[\text{Cosh}\lambda_2 x\sin\lambda_2(b-x) - \text{Sinh}\lambda_2 x\cos\lambda_2(b-x)\right]\right)$$

$$-\frac{2M_0\lambda_2}{\Delta_2}\left(\text{Sinh}\lambda_2 b\sin\lambda_2 x\text{Sinh}\lambda_2(b-x) + \sin\lambda_2 b\text{Sinh}\lambda_2 x\sin\lambda_2(b-x)\right)$$

The pressure under the nail is given by

$$p = \bar{k}y$$

Therefore the maximum pressure in member 1 will occur at $x = a$ and the maximum pressure in member 2 will occur at $x = 0$.

$$P_1 \text{ max.} = - Pk_1 \left[2L_1 - \frac{J_1(J_1 - J_2)}{K_1 + K_2} \right]$$

$$P_2 \text{ max.} = Pk_2 \left[2L_2 + \frac{J_1(J_1 - J_2)}{K_1 + K2} \right]$$

where

$$J_1 = \frac{\lambda_1{}^2}{k_1} \left[\frac{\text{Sinh}^2\lambda_1 a + \sin^2\lambda_1 a}{\text{Sinh}^2\lambda_1 a - \sin^2\lambda_1 a} \right]$$

$$J_2 = \frac{\lambda_2{}^2}{k_2} \left[\frac{\text{Sinh}^2\lambda_2 b + \sin^2\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b} \right]$$

$$K_1 = \frac{\lambda_1{}^3}{k_1} \left[\frac{\text{Sinh}\lambda_1 a \ \text{Cosh}\lambda_1 a + \sin\lambda_1 a \ \cos\lambda_1 a}{\text{Sinh}^2\lambda_1 a - \sin^2\lambda_1 a} \right]$$

$$K_2 = \frac{\lambda_2{}^3}{k_2} \left[\frac{\text{Sinh}\lambda_2 b \ \text{Cosh}\lambda_2 b + \sin\lambda_2 b \ \cos\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b} \right]$$

$$L_1 = \frac{\lambda_1}{k_1} \left[\frac{\text{Sinh}\lambda_1 a \ \text{Cosh}\lambda_1 a - \sin\lambda_1 a \ \cos\lambda_1 a}{\text{Sinh}^2\lambda_1 a - \sin^2\lambda_1 a} \right]$$

$$L_2 = \frac{\lambda_2}{k_2} \left[\frac{\text{Sinh}\lambda_2 b \ \text{Cosh}\lambda_2 b - \sin\lambda_2 b \ \cos\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b} \right]$$

The slip of the connection is given at the junction of member 1 and member 2.

$$\delta = -y_1\Big|_{x=a} + y_2\Big|_{x=0}$$

$$\delta = P\left[2(L_1 + L_2) - \frac{(J_1 - J_2)^2}{K_1 + K_2}\right]$$

DOUBLE SHEAR CONNECTIONS

The following expressions are deflection, moment, and shear at any point along the nail. FIGURE B.2 indicates the notations used for this type of connection.

MEMBER 1

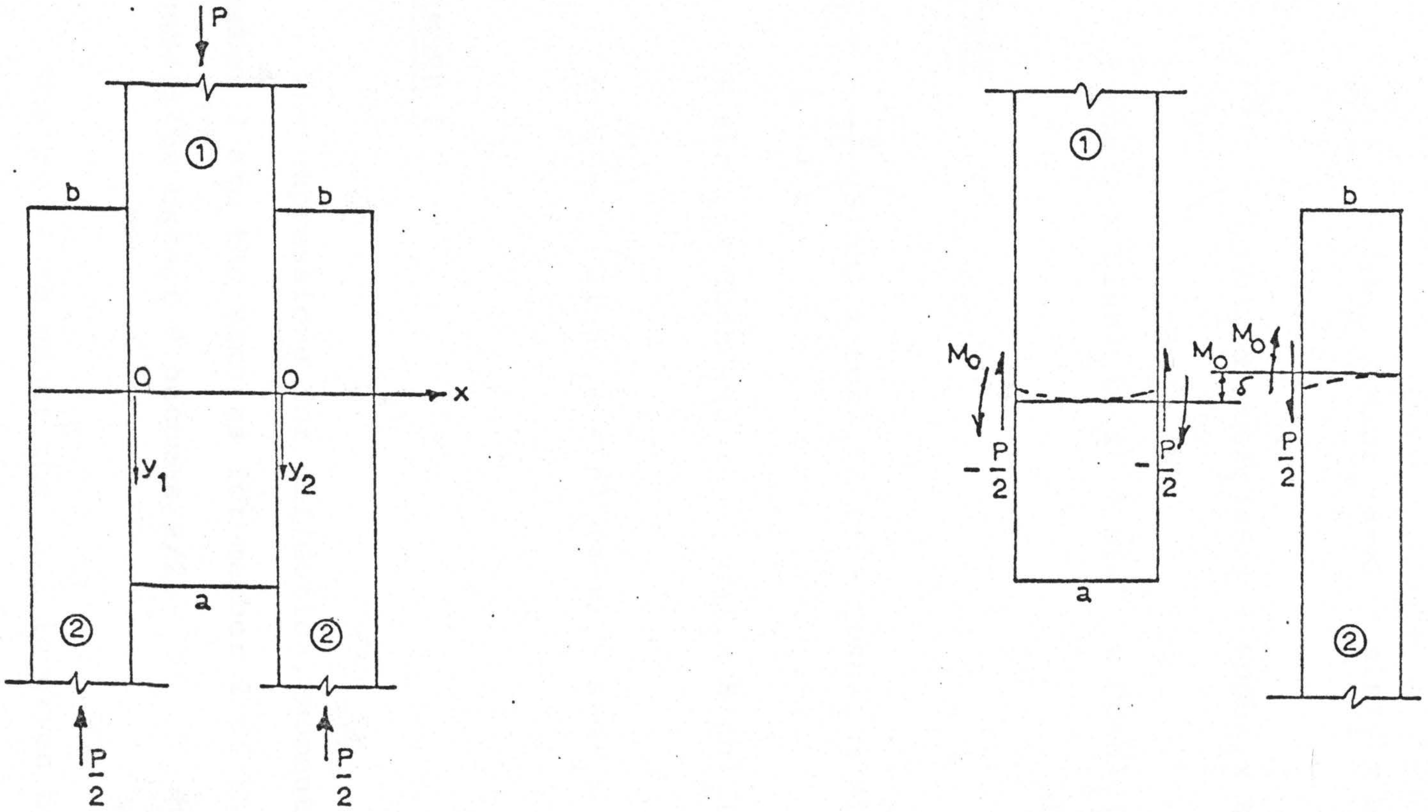
$$\lambda_1 = \sqrt[4]{\frac{k_1}{4EI}}$$

$$\Delta_1 = \mathrm{Sinh}\lambda_1 a + \sin\lambda_1 a$$

Deflection

$$y_1 = -\frac{P\lambda_1}{\Delta_1 k_1}\left(\mathrm{Cosh}\lambda_1 x\ \cos\lambda_1(a-x) + \cos\lambda_1 x\ \mathrm{Cosh}\lambda_1(a-x)\right)$$

$$-\frac{2M_0\lambda_1^2}{\Delta_1 k_1}\left(\mathrm{Sinh}\lambda_1 x\ \cos\lambda_1(a-x) - \mathrm{Cosh}\lambda_1 x \sin\lambda_1(a-x)\right.$$



A. CHOICE OF AXIS

B. FORCES AND DEFLECTIONS

FIGURE B.2 Notation for Double Shear Connections

$$+ \cos\lambda_1 x \mathrm{Sinh}\lambda_1 (a-x) - \sin\lambda_1 x \ \mathrm{Cosh}\lambda_1 (a-x)\Big)$$

Moment

$$M_1 = \frac{P}{2\Delta_1\lambda_1}\left(\mathrm{Sinh}\lambda_1 x \sin\lambda_1 (a-x) + \sin\lambda_1 x \ \mathrm{Sinh}\lambda_1 (a-x)\right)$$

$$+ \frac{M_0}{\Delta_1}\Big(\mathrm{Sinh}\lambda_1 x \cos\lambda_1 (a-x) + \mathrm{Cosh}\lambda_1 x \sin\lambda_1 (a-x)$$

$$+ \cos\lambda_1 x \ \mathrm{Sinh}\lambda_1 (a-x) + \sin\lambda_1 x \ \mathrm{Cosh}\lambda_1 (a-x)\Big)$$

Shear

$$Q_1 = -\frac{P}{2\Delta_1}\Big(\mathrm{Sinh}\lambda_1 x \cos\lambda_1 (a-x) - \mathrm{Cosh}\lambda_1 x \sin\lambda_1 (a-x)$$

$$+ \sin\lambda_1 x \ \mathrm{Cosh}\lambda_1 (a-x) - \cos\lambda_1 x \ \mathrm{Sinh}\lambda_1 (a-x)\Big)$$

$$+ \frac{2M_0\lambda_1}{\Delta_1}\left(\mathrm{Sinh}\lambda_1 x \sin\lambda_1 (a-x) - \sin\lambda_1 x \ \mathrm{Sinh}\lambda_1 (a-x)\right)$$

MEMBER 2

The expressions for deflection, moment, and shear in member 2 are the same as for member 2 of the single shear connection except P becomes P/2.

The pressure under the nail is given by

$$p = \overline{k}y$$

Therefore the maximum pressure in member 1 will occur at $x = 0$ or at $x = a$ and the maximum pressure in member 2 will occur at $x = 0$.

$$P_1 \text{ max.} = - Pk_1 \left[L_1 - \frac{J_1(J_1 + J_2)}{2(K_1 + K_2)} \right]$$

$$P_2 \text{ max.} = Pk_2 \left[L_2 + \frac{J_2(J_1 - J_2)}{2(K_1 + K_2)} \right]$$

where

$$J_1 = \frac{\lambda_1^{\,2}}{k_1} \left[\frac{\text{Sinh}\lambda_1 a - \sin\lambda_1 a}{\text{Sinh}\lambda_1 a + \sin\lambda_1 a} \right]$$

$$J_2 = \frac{\lambda_2^{\,2}}{k_2} \left[\frac{\text{Sinh}^2\lambda_2 b + \sin^2\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b} \right]$$

$$K_1 = \frac{\lambda_1^{\,3}}{k_1} \left[\frac{\text{Cosh}\lambda_1 a - \cos\lambda_1 a}{\text{Sinh}\lambda_1 a + \sin\lambda_1 a} \right]$$

$$K_2 = \frac{\lambda_2^{\,3}}{k_2} \left[\frac{\text{Sinh}\lambda_2 b \ \text{Cosh}\lambda_2 b + \sin\lambda_2 b \ \cos\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b} \right]$$

$$L_1 = \frac{\lambda_1}{k_1} \left[\frac{\text{Cosh}\lambda_1 a + \cos\lambda_1 a}{\text{Sinh}\lambda_1 a + \sin\lambda_1 a} \right]$$

$$L_2 = \frac{\lambda_2}{k_2}\left[\frac{\text{Sinh}\lambda_2 b\ \text{Cosh}\lambda_2 b - \sin\lambda_2 b\ \cos\lambda_2 b}{\text{Sinh}^2\lambda_2 b - \sin^2\lambda_2 b}\right]$$

The slip of the connection is given at the junction of member 1 and member 2.

$$\delta = -y_1\Big|_{x=a} + y_2\Big|_{x=0}$$

$$\delta = P\left[L_1 + L_2 - \frac{(J_1 - J_2)^2}{2(K_1 + K_2)}\right]$$

APPENDIX "C"

-Plates-

PLATE C.1   Connection Types

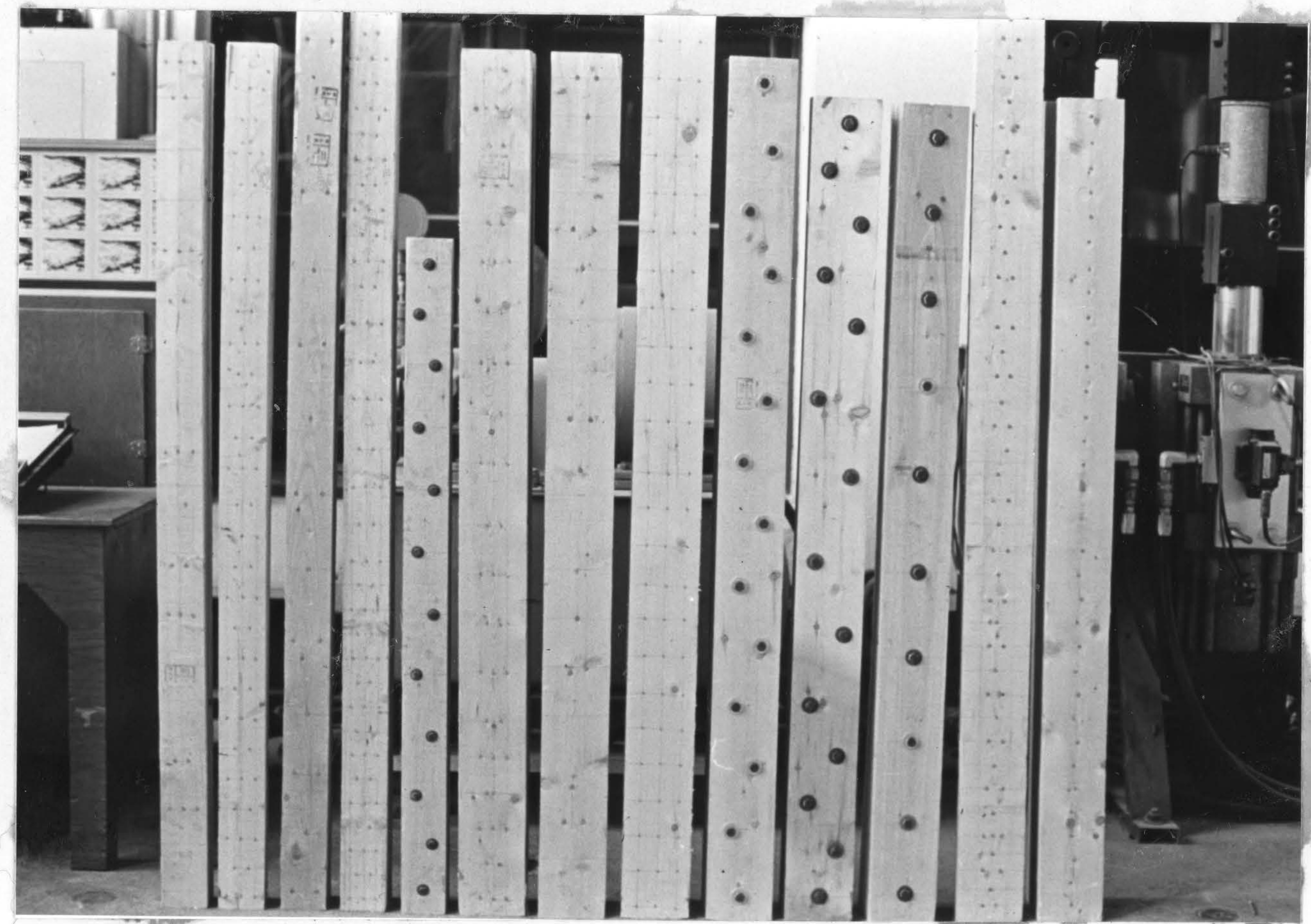


PLATE C.2 Layered Columns

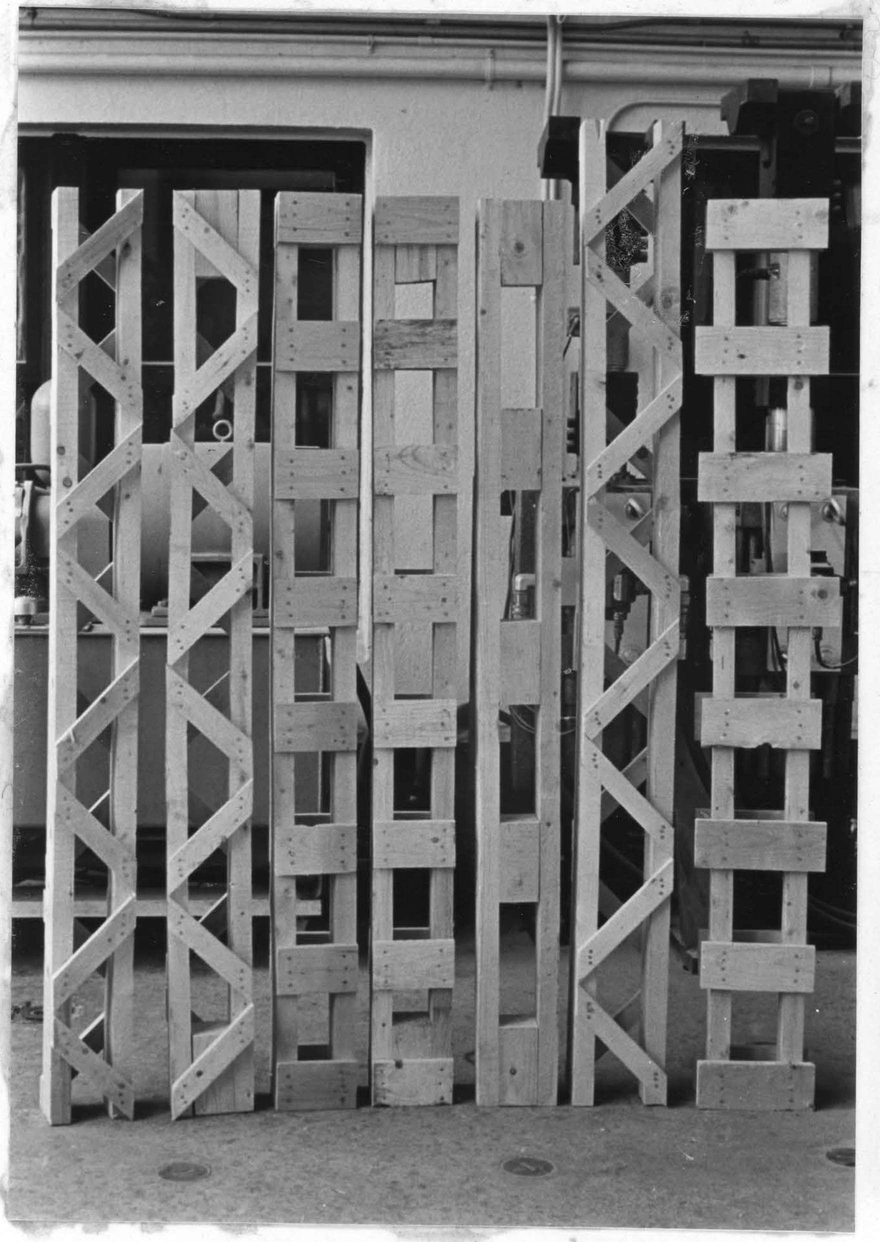


PLATE C.3 Braced and Spaced Columns