



Revision of ANSI/TPI 1-2002

American National Standard

**NATIONAL DESIGN
STANDARD
FOR
METAL PLATE CONNECTED
WOOD TRUSS CONSTRUCTION**

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Printed in the United States of America

FOREWORD

(This Foreword is not part of American National Standard ANSI/TPI 1-2007)

An association of manufacturers actively engaged in the production of metal connector plates for the wood truss industry, and individuals or firms engaged in related activities, the Truss Plate Institute was organized in 1961 for the purpose of maintaining the wood truss industry on a sound engineering basis. To accomplish its purpose, the Institute establishes methods of design and construction for wood trusses using metal connector plates, supports and disseminates test and research data, assists in the development of proper building code regulations, recommends quality control standards, and distributes information on the use of metal-plate-connected wood trusses in the interest of public safety.

Both Imperial (inches and pounds) and SI (millimeters and Newtons) have been included in this document to facilitate its use by a wider audience. The intent is not to require dual units to be shown on all drawings and designs; rather, the intent is to allow the designer to use whichever system of measure is most useful for a given project. The dual system also allows the designer to comply with the United States Federal Government mandate for use of metric units on all federal projects.

Structural components covered in this document are wood trusses using metal plate connectors at their joints. Metal-plate-connected/metal web wood trusses, steel pin connected pipe web trusses and other structural elements are expressly excluded from this Standard. Appendices to this Standard are non-mandatory and are not part of this Standard.

It is the sole responsibility of the user to apply the criteria in this Standard. The Truss Plate Institute and the metal-plate-connected wood truss industry at large expressly disclaim any liability arising from the use, application or reference to the present document.

TARGET AUDIENCE

This ANSI/TPI 1-2007, *National Design Standard for Metal Plate Connected Wood Truss Construction* has been developed primarily for use by professional engineers and architects involved in the design of metal-plate-connected wood trusses. This document will also serve the truss manufacturer, and aid building officials, approved quality assurance agencies, and building engineers or architects of record.

ACKNOWLEDGMENTS

TPI would like to acknowledge the efforts of its Technical Advisory Committee and its ANSI accredited Project Committee and it's contributing participants for their many hours of work in developing this Standard. Thanks also go out to those participating in ANSI's "Standards Action" call for comment on this ANSI/TPI 1-2007.

This Standard was processed and approved for submittal to ANSI by the Accredited Truss Plate Institute's Project Committee. Project Committee approval of this Standard does not necessarily imply that all committee members voted for its approval. At the time it approved this standard, TPI Project Committee had the following membership.

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Published by:

Truss Plate Institute

218 North Lee Street, Ste. 312
Alexandria, VA 22314
(703) 683-1010
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CHAPTER 1 GENERAL

1.1 INTRODUCTION

The Truss Plate Institute (TPI) has developed this *National Design Standard for Metal Plate Connected Wood Truss Construction* to provide state-of-the-art technical information and specifications on metal-plate-connected wood Truss design and manufacturing.

1.2 DEVELOPMENT

This Standard was prepared by a committee comprised of: Users - professional engineers; Producers - wood Truss Manufacturers, Metal Connector Plate manufacturers, lumber manufacturers, and structural hardware manufacturers; and General Interest - participants from universities, Building Code agencies, and others. It is based on the collective engineering knowledge and best data available upon attaining evidence of consensus.

1.3 GENERAL

1.3.1 Scope.

This Standard establishes minimum requirements for the design and construction of metal-plate-connected wood Trusses. This Standard describes the materials used in a Truss, both lumber and steel, and design procedures for Truss members and joints. Responsibilities, methods for evaluating the Metal Connector Plates, and manufacturing quality assurance for the Trusses are also contained in this Standard.

1.3.2 Alternate Provisions.

This Standard does not intend to preclude the use of materials, assemblies, structures, or designs not meeting the criteria herein, when they demonstrate equivalent performance for the intended use to those specified in this Standard. The divisions of Responsibilities between the Truss Designer, Truss Design Engineer, Building Designer and others as defined in Chapter 2 and elsewhere in this Standard are not intended to preclude alternate provisions as agreed upon by the parties involved.

1.4 REFERENCED STANDARDS

ACI530/ASCE5/TMS402, Building Code Requirements for Masonry Structures.

ANSI A108/ANSI A118/A136-2005, Specifications for the Installation of Ceramic Tile.

ANSI/AF&PA NDS-2005, National Design Specification for Wood Construction & Supplement.

ANSI/AITC A190.1-2002, Structural Glued Laminated Timber.

ANSI/ASCE 7-05, Minimum Design Loads for Buildings and other Structures.

American Softwood Lumber Standard, Voluntary Product Standard *PS 20-05*, National Institute of Standards and Technology, U.S. Department of Commerce.

ASTM A153/A153M-05, Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

ASTM A167-99 (2004), Standard Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip.

ASTM A240/A240M-06c, Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications.

ASTM A879-06, Standard Specification for Steel Sheet, Zinc Coated by the Electrolytic Process for Applications Requiring Designation of the Coating Mass on Each Surface.

ASTM A653/A653M-06a, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process.

ASTM A792/A792M-06a, Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process.

ASTM C840-07, Standard Specification for Application and Finishing of Gypsum Board.

ASTM D245-06, Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.

ASTM D2395-06, Standard Test Methods for Specific Gravity of Wood and Wood-Base Materials.

<i>ASTM D2555-06</i> , Standard Test Methods for Establishing Clear Wood Strength Values.	A_{gc}	Cross-sectional area of the solid metal control specimen
<i>ASTM D2559-04</i> , Standard Specification for Adhesives for Structural Laminated Wood Products for Use Under Exterior (Wet Use) Exposure Conditions.	A_{gp}	Cross-sectional area of the Metal Connector Plate
<i>ASTM D4442-92 (2003)</i> , Standard Test Methods for Direct Moisture Content Measurement of Wood and Wood-Base Materials.	A_p	Minimum required Metal Connector Plate contact area on Wood Members
<i>ASTM D4444-92 (2003)</i> , Standard Test Methods for Use and Calibration of Hand-Held Moisture Meters.	AA, AE, EA, EE	Designations for orientation of plates with respect to tooth holding (V_{LR}); see notations V_{LRAA} , V_{LRAE} , V_{LREA} , and V_{LREE}
<i>ASTM D5456-06</i> , Standard Specification for Evaluation of Structural Composite Lumber Products.	B	Width of bearing
<i>ASTM E4-07</i> , Standard Methods of Load Verification of Testing Machines.	B_R	Minimum required bearing width
<i>ASTM E8-04</i> , Standard Methods for Tension Testing of Metallic Materials.	C	Wood compression strength normal to joint line for design of plated joints subject to bending (see Section 8.7.1)
<i>ASTM E631-06</i> , Standard Terminology of Building Constructions.	C_b	Bearing area factor
<i>BCSI</i> , Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses.	C_D	Load duration factor
<i>BCSI-B1</i> , Guide for Handling, Installing, Restraining & Bracing of Trusses.	C_{fu}	Flat use factor
<i>BCSI-B2</i> , Truss Installation & Temporary Restraint/Bracing.	C_i	Incising factor
<i>BCSI-B3</i> , Permanent Restraint/Bracing of Chords & Web Members.	\mathcal{Q}	Centerline of Truss or member
<i>BCSI-B7</i> , Temporary & Permanent Restraint/Bracing for Parallel Chord Trusses.	C_M	Wet service factor
<i>BCSI-B10</i> , Post Frame Truss Installation & Temporary Restraint/Bracing.	C_m	Factor to account for P-delta effects on chord splices subject to moment (see Section 8.7.1)
	C_{plate}	Bearing plate increase factor
	C_q	Quality control factor (see Section 6.4.10)
	C_r	Repetitive member factor
	C_R	Reduction factor for the compression force component across the joint interface
	C_s	Compression force in steel across joint using a Truss plate subject to bending (see Section 8.7.1)
	C_T	Buckling stiffness factor
	C_t	Temperature factor

C_w	Compression force carried by wood-to-wood butting across joint using a Truss plate subject to bending (see Section 8.7.1)	$F_{c\perp}, F'_{c\perp}$ Reference and adjusted compression design stress perpendicular to grain
c	One-half the depth (d) of the Wood Member	F_{cE} Critical buckling design stress for compression members
COV_E	Coefficient of variation for modulus of elasticity	F_{sc} Theoretical ultimate shear strength of the solid metal control specimen ($0.577F_{tc}$)
D	(1) Overall depth of Truss, (2) depth of chord at splice joint or (3) diagonal of a rectangular area with area of A_{ef} (see Section 8.7.2)	F_{sp} Ultimate shear strength of the Metal Connector Plate
D_{PR}	Gross dimension of the Metal Connector Plate measured perpendicular to the Wood Member's grain	F_{st} Allowable tensile stress of the steel
d	(1) Dowel diameter or (2) critical dimension of the rectangular member in buckling	F_t, F'_t Reference and adjusted tension design stress parallel to grain
d_1, d_2	Cross-sectional dimension of the rectangular member in the plane of the Truss and perpendicular to the plane of the Truss, respectively	F_{tc} Ultimate tensile strength of solid metal control specimen
d_e	Effective depth of member for tension perpendicular to grain loads ($c + y$)	F_{tp} Ultimate tensile strength of Metal Connector Plate
d_{le}	Distance from the outer edge of the chord to the outer edge of the Metal Connector Plates joining the two chord members	F_u Ultimate tensile strength of steel
E, E'	Reference and adjusted modulus of elasticity	F_v, F'_v Reference and adjusted shear design stress parallel to grain (horizontal shear)
E_{min}, E'_{min}	Reference and adjusted modulus of elasticity for stability calculations	F_{vs} Allowable shear stress of the steel
F_θ	Allowable bearing design stress at an angle to grain	F_y Minimum yield strength of the steel
F_b, F'_b	Reference and adjusted design stress for bending	$F_{y,spec}$ Specified minimum steel yield strength
F_b^*	Design stress for bending adjusted by modification factors	$F_{y,test}$ Average measured steel yield strength of test plates
F_{bE}	Critical buckling design stress for bending members	f_b Actual bending stress
F_c, F'_c	Reference and adjusted design stress for compression parallel to grain	f_c Actual compression stress parallel to grain
F_c^*	Design stress for compression parallel to grain adjusted by modification factors	$f_{c\perp}$ Actual compression stress perpendicular to grain
		f_t Actual tension stress parallel to grain
		f_v Actual shear stress parallel to grain
		G (1) Specific gravity (oven-dry basis) of wood specified for design (also shown as $G_{specified}$) or (2) tooth embedment gap (see Section 3.7.7.2.4)

G_{test}	Average measured specific gravity (oven-dry basis) of wood used in test joints	L_w	Actual length of Web member
g	Transverse center-to-center spacing or gauge of any two consecutive holes (see Section 5.4.8.2)	M	Actual bending moment
H_R	Heel reduction factor	M_a	Maximum allowable bending moment
h	Height of equivalent rectangle (see Section 8.7.2)	N	Minimum required number of Teeth per face
JSI	Joint Stress Index, which is determined for each joint as the largest ratio of applied force to allowable design force	P	(1) Total concentrated load or axial load or (2) force in Wood Member, for design of Truss joints
K	Effective length factor	P'	Resultant compressive force used for determination of minimum required Metal Connector Plate contact area
K_b	Load and span effect constant	P_{\perp}	Net force component perpendicular to grain
K_{cr}	Creep factor	P_c	Axial compression force
K_f	Column stability coefficient for bolted and nailed built-up columns	P_{CR}	Axial force in compression parallel to the grain of the chord to the right of the panel point
K_m	Bending capacity modification factor	P_{CL}	Axial force in compression parallel to the grain of the chord to the left of the panel point
ℓ	(1) Calculated shear length of the Metal Connector Plate oriented at an angle (α) to the joint or (2) distance measured from the centerline of the girder cross-section to the midpoint ($\frac{1}{2}$ seat width) of the hanger where the point load is located, or the distance to the center of a structural member framing into the girder (i.e., jack)	P_{CW}	Axial force in compression parallel to the grain of the Web
ℓ_b	Length of bearing	P_{iN}	Compression force component of the Wood Member under investigation normal to the joint interface
l'	Plate dimension parallel to the loading direction for Test Specimens used to develop lateral resistance strength of Metal Connector Plate Teeth	P_{ip}	Compression force component of the Wood Member under investigation parallel to the joint interface
L	(1) Nominal panel length of chords used in computing bending moment and buckling length or (2) length of solid metal control specimen	P_s	Force parallel to the joint across the shear plane
L'	Effective buckling length of compression member	P_{sp}	Maximum shear force carried by the Metal Connector Plate
L_e	Effective span length for bending members	P_t	Axial tension force
L_p	(1) Center to center spacing of purlins or (2) Gross length of Metal Connector Plates	P_{tc}	Maximum tensile force carried by the solid metal control specimen
L_u	Unsupported length of bending member	P_{tp}	Maximum tensile force carried by the Metal Connector Plate
		P_{TW}	Axial force in tension parallel to the grain of the Web

Q_R	Strength ratio between matched sets of single pass full embedment roller pressed joints and hydraulic pressed joints, in accordance with Annex A.5.2	V_{LR}^*	Allowable lateral resistance value per Metal Connector Plate unit adjusted per all applicable factors except C_q
R_G	Adjustment factor for specific gravity of Test Specimens exceeding specified value (see Section 5.2.9.3)	V_{LRAA}	Allowable value for Metal Connector Plates loaded parallel to the grain with the plate axis (tooth slots) parallel to the load
R_s	Shear efficiency, F_{sp}/F_{sc}	V_{LRAE}	Allowable value for Metal Connector Plates loaded perpendicular to the grain with the plate axis (tooth slots) parallel to the load
R_T	Adjustment factor to account for steel thickness	$V_{LRA\theta}$	Allowable value for Metal Connector Plates loaded at an angle, θ , to the grain with the plate axis (tooth slots) parallel to the load
R_t	Tensile efficiency, F_{tp}/F_{tc}	V_{LREA}	Allowable value for Metal Connector Plates loaded parallel to the grain with the plate axis (tooth slots) perpendicular to the load
R_i	Reaction or load imposed by uniformly spaced members on a girder	V_{LREE}	Allowable value for Metal Connector Plates loaded perpendicular to the grain with the plate axis (tooth slots) perpendicular to the load
R_Y	Adjustment factor to account for steel yield	$V_{LRE\theta}$	Allowable value for Metal Connector Plates loaded at an angle, θ , to the grain with the plate axis (tooth slots) perpendicular to the load
S	Section modulus	V_s	Allowable design value in shear for a pair of metal connector plates
s	Longitudinal center-to-center spacing or pitch of any two consecutive holes (see Section 5.4.8.2)	$V_{s\parallel}$	Capacity of a pair of Metal Connector Plates to resist shear along the major axis
T_G	Applied torque to girder Truss	$V_{s\perp}$	Capacity of a pair of Metal Connector Plates to resist shear at 90° to the major axis
T_1, T_2	Components of tension force in steel of Truss plate across joint subject to bending (see Section 8.7.1)	V_t	Allowable design value in tension for a pair of Metal Connector Plates
t	Minimum specified steel thickness (also shown as t_{spec})	$V_{t\parallel}$	Tensile capacity of the Metal Connector Plate section where the load is applied parallel to the major axis
t_l	Base metal design thickness of the Metal Connector Plate, $(t - t_c) / 0.95$	$V_{t\perp}$	Tensile capacity of the Metal Connector Plate section where the load is applied at a 90° to the major axis
t_c	Coating thickness	W	Gross width of the solid metal control specimen
t_{net}	Net thickness, $(t - t_c)$	W_p	Gross Metal Connector Plate Width
V	Actual shear force		
V'	Maximum allowable shear force (see Section 7.3.7.3)		
V_{LR}	Lateral resistance value per Metal Connector Plate unit, based on a plate on each face		
V_{LR}'	Allowable lateral resistance design value per Metal Connector Plate unit adjusted per all applicable factors		

W_p'	Maximum effective width of Metal Connector Plates for all panel point tension chord splices with Metal Connector Plates that extend past the chord member
w	Uniformly distributed load (constant or varying)
X_{st}	Combined shear/tension value for the effective horizontal dimension of a pair of Metal Connector Plates
x	(1) Offset of centerline of rectangular opening from centerline of Truss or (2) distance between splice and nearest panel point (see Section 8.7.1)
x_{max}	Distance from the inner edge of the chord to the inner edge of the Metal Connector Plate joining the chord-splice and extended past the chord member
Y_{st}	Combined shear/tension value for the effective vertical dimension of a pair of Metal Connector Plates
y	(1) Distance to the centerline of the extreme fastener from the centerline of the Wood Member or (2) Distance to neutral axis from edge of Wood Member with moment-induced compression
z	Distance from compression edge of lumber to compression edge of plate (see Section 8.7.1)
Δ_T	Total deflection
Δ_{LT}	Immediate deflection due to the long-term component of the design load (deflection due to a sustained load, typically dead load)
Δ_{ST}	Deflection due to short term or normal component of the design load (deflection due to transient loads, typically live load)
α	Angle between the load and the Metal Connector Plate Length (slot) direction (see Section 5.3.2 and Figures 5.3-2 through 5.3-5)
θ	(1) Angle between the Top and Bottom Chords or (2) angle between the load and wood grain

1.6 DEFINITIONS

The following definitions apply in this Standard. See Chapter 2 of this Standard for additional definitions. For general definitions of terms used in the test methods in Chapter 5 of this Standard, and not given in Section 1.6, see *ASTM E631*.

Bottom Chord - Horizontal or inclined (e.g., scissors Truss) member that establishes the lower edge of a Truss, usually carrying combined tension and bending stresses. A member is defined as including all such chord members in that it may consist of shorter spliced pieces.

Cantilever – Extension of both chords of a Truss beyond its end support for a distance that is included in the defined span, exclusive of Overhang.

Critical Slip - When measured from the Metal Connector Plate to each active Wood Member, Critical Slip is 0.015 in. (0.38 mm); when measured from Wood Member to Wood Member, it is 0.030 in. (0.76 mm), except for AE and EE orientations, in which case it is 0.015 in. (0.38 mm).

Edge Distance - Distance from the edge of the Wood Member, measured perpendicular to the length of the Wood Member, in which Teeth are presumed to be ineffective for lateral resistance design purposes and in which Teeth are prohibited from being located for lateral resistance testing purposes. This distance shall be permitted to vary for different testing methods, but shall be consistent between the Test Specimens used to establish a lateral resistance design value and the design calculations in which that lateral resistance design value is used, except as permitted by Section 8.3.2.1.

Effectiveness Ratio - Ratio of the Metal Connector Plate ultimate strength (shear or tensile) to the matched solid metal control specimen ultimate strength.

End Distance - Distance from the end of the Wood Member, measured parallel to the length of the Wood Member, in which Teeth are presumed to be ineffective for lateral resistance design purposes and in which Teeth are prohibited from being located for lateral resistance testing purposes. This distance shall be permitted to vary for different testing methods, but shall be consistent between the Test Specimens used to establish a lateral resistance design value and the design calculations in which that lateral resistance design value is used, except as permitted by Section 8.3.2.1.

Gross Area Method - Lateral resistance testing and design calculation in which the End and Edge Distances are zero.

Joint QC Detail - Graphical detail of a Truss joint that shows positioning tolerances calculated by the Truss Designer for any particular joint of a Truss selected for Truss inspection per the requirements of Chapter 3 of this Standard.

Keeper Nails – Nails driven through the Metal Connector Plate, during Truss fabrication, to hold its location on the Wood Members before pressing.

Ladder Frame - Short wall fabricated in the factory containing a top plate, a series of vertical members spaced 24 in. (61 cm) on center or less, and a sole plate, also known as knee wall or cripple wall.

Metal Connector Plate - Metal plate used to connect co-planar Wood Members, so as to transmit forces from one Wood Member to one or more other Wood Members, and connected to such Wood Members using either integral Teeth formed from the metal plate and subsequently embedded into the wood, or separately applied driven fasteners such as nails. Also referred to as plate, Truss plate, metal plate, metal-plate connector, and nail plate.

Metal Connector Plate Length - Dimension of the Metal Connector Plate parallel to the longitudinal axis of the face coil from which the Teeth were stamped during Metal Connector Plate fabrication normally the dimension parallel to the length of the slots.

Metal Connector Plate Width - Dimension of the Metal Connector Plate perpendicular to the longitudinal axis of the face coil from which the Teeth were stamped during Metal Connector Plate fabrication normally the dimension perpendicular to the length of the slots.

Nail Hole - Round perforation in a Metal Connector Plate through which a nail can be driven to fasten a Metal Connector Plate to a Wood Member and to transmit shear loads; providing a predetermined location for appropriately locating the nail to be driven.

Narrow Face - Face or surface of a Wood Member with a dimension less than or equal to 2 in. (50.8 mm).

Overhang - Extension of the Top Chord beyond the Bottom Chord or the Bottom Chord beyond the Top Chord of a Truss, exclusive of Cantilever.

Sample Block - Sample cut from the Wood Member for the purpose of determining the specific gravity and/or moisture content of the Wood Member used in testing.

Net Area Method - Lateral resistance testing and design calculation in which the End Distance is $\frac{1}{2}$ in. (13 mm) and the Edge Distance is $\frac{1}{4}$ in. (6 mm). (Note: For design purposes using lateral resistance design values in units of force per unit area, these distances shall be adjusted to match the distances tributary to the Teeth that are removed in the given distances.)

Sidesway - Top of a column is relatively free to displace laterally with respect to the bottom of the column and its lateral displacement is resisted primarily by the flexural rigidity of the column. The opposite condition (without Sidesway) occurs when the ends of the column are prevented from moving relative to each other by a much stiffer restraint, such as a trussed frame (where openings between members consist solely of triangles) connected to both ends of the compression member, or other structure.

Solid Metal Control Sample - Solid plate sample of the same material as the Metal Connector Plate of dimensions large enough so as to fabricate solid metal control specimens in accordance with *ASTM E8*; without integral Teeth. It is used to determine the mechanical properties of the metal.

Species Combination - Grouping of several species into a single category (e.g., Spruce-Pine-Fir).

Structural Composite Lumber - See *ASTM D5456*.

Teeth - Integral metal projections of the Metal Connector Plate formed to be more or less perpendicular to the face of the Metal Connector Plate during the stamping process. Also called prongs, barbs, plugs, nails, etc., but henceforth they will be termed Teeth.

Test Specimen - Connection to be tested; lateral strength and tensile strength Test Specimens are fabricated by joining two Wood Members together with two Metal Connector Plates, and the shear strength Test Specimen is fabricated by joining three Wood Members together with four Metal Connector Plates.

Top Chord - Inclined or horizontal member that establishes the upper edge of a Truss. A member is defined as including all such chord members in that it may consist of shorter spliced pieces.

Webs - Wood Members that join the Top and Bottom Chords that form the triangular patterns that give Truss action, usually carrying tension or compression stresses.

Wood Member - Piece of lumber, from the Species Combination or Structural Composite Lumber evaluated, which is to be joined to a like piece of lumber, and which contains clear wood in the area in which the Metal Connector Plate is embedded (see Section 5.2.4.3).

1.7 CONVERSION FACTORS

1 in	=	25.40 mm
1 ft	=	0.3048 m
1 lb	=	0.4536 kg
1 lbf	=	4.448 N
1 ksi	=	6.894 MPa
1 psf	=	47.88 Pa
1 pli	=	0.1751 N/mm
1 plf	=	14.59 N/m

CHAPTER 2

STANDARD RESPONSIBILITIES IN THE DESIGN AND APPLICATION OF METAL-PLATE-CONNECTED WOOD TRUSSES

2.1 GENERAL PURPOSES

The purpose of this Chapter of the Standard is to define and draw attention to the Responsibilities of the Owner, Building Designer, Registered Design Professional for the Building, Truss Manufacturer, and Truss Designer or Truss Design Engineer, with respect to the application of Trusses in the construction of a Building.

2.2 DEFINITIONS

BCSI: *Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses* jointly produced by *WTCA – Representing the Structural Building Components Industry and the Truss Plate Institute*.

BCSI-B1: *Guide for Handling, Installing, Restraining & Bracing of Trusses* of the Building Component Safety Information (BCSI).

BCSI-B2: *Truss Installation & Temporary Restraint/Bracing* of the Building Component Safety Information (BCSI).

BCSI-B3: *Permanent Restraint/Bracing of Chords & Web Members* of the Building Component Safety Information (BCSI).

BCSI-B7: *Temporary & Permanent Restraint/Bracing for Parallel Chord Trusses* of the Building Component Safety Information (BCSI).

BCSI-B10: *Post Frame Truss Installation & Temporary Restraint/Bracing* of the Building Component Safety Information (BCSI).

Building: Structure used or intended for supporting or sheltering any use or occupancy.

Building Code: As it applies to a Building, any set of standards set forth and enforced by a Jurisdiction for the protection of public safety.

Building Designer: Owner of the Building or the Person that contracts with the Owner for the design of the Framing Structural System and/or who is responsible for the preparation of the Construction Documents. When

mandated by the Legal Requirements, the Building Designer shall be a Registered Design Professional.

Building Official: Officer or other designated authority charged with the administration and enforcement of the Building Code, or a duly authorized representative.

Building Permit: Certificate of permission issued by a Jurisdiction to an Owner to construct, enlarge, or alter a Building.

Construction Documents: Written, graphic and pictorial documents prepared or assembled for describing the design (including the Framing Structural System), location and physical characteristics of the elements of a Building necessary to obtain a Building Permit and construct a Building.

Contract: Legally recognized agreement between two parties.

Contractor: Owner of a Building, or the Person who contracts with the Owner, who constructs the Building in accordance with the Construction Documents and the Truss Submittal Package. The term “Contractor” shall include those subcontractors who have a direct Contract with the Contractor to construct all or a portion of the construction.

Cover/Truss Index Sheet: Sheet that is signed and sealed, where required by the Legal Requirements, by the Truss Design Engineer, and depending on the Legal Requirements shall be permitted to contain the following information: (1) Identification of the Building, including Building name and address, lot, block, subdivision, and city or county; (2) Identification of Construction Documents by drawing number(s) with revision date; (3) specified Building Code; (4) computer program used; (5) roof dead and live loads; (6) floor dead and live loads; (7) wind load criteria from a specifically defined code (e.g., *ASCE 7*) and any other design loads (such as ponding, mechanical loads, etc.); (8) name, address and license number of Registered Design Professional for the Building, if known; (9) a listing of the individual identification numbers and dates of each Truss Design Drawing referenced by the Cover/Truss Index Sheet; and (10) name, address, date of drawing and license number of Truss Design Engineer.

Deferred Submittal: Those portions of the design that are not completed at the time of the application for the Building Permit and that are to be submitted to the Building Official within a specified period in accordance with the Legal Requirements.

Diagonal Bracing: Structural member installed at an angle to a Truss chord or web member and intended to temporarily and/or permanently stabilize Truss member(s) and/or Truss(es) (See *BCSI-B1*, *BCSI-B2*, *BCSI-B3*, *BCSI-B7*, and *BCSI-B10*).

Framing Structural System: Completed combination of Structural Elements, Trusses, connections and other systems, which serve to support the Building's self-weight and the specified loads.

Jurisdiction: Governmental unit that is responsible for adopting and enforcing the Building Code.

Lateral Restraint: Also known as continuous lateral brace or CLB. A structural member installed at right angles to a chord or Web member of a Truss to reduce the laterally unsupported length of the Truss member (See *BCSI-B1*, *BCSI-B2*, *BCSI-B3*, *BCSI-B7*, and *BCSI-B10*).

Legal Requirements: Any applicable provisions of all statutes, laws, rules, regulations, ordinances, codes, or orders of the governing Jurisdiction.

Owner: Person having a legal or equitable interest in the property upon which a Building is to be constructed, and: (1) either prepares, or retains the Building Designer or Registered Design Professional to prepare the Construction Documents; and (2) either constructs, or retains the Contractor to construct the Building.

Permanent Building Stability Bracing: Lateral force resisting system for the Building that resists forces from gravity, wind, seismic and/or other loads.

Permanent Individual Truss Member Restraint: Restraint that is used to prevent local buckling of an individual Truss chord or Web member due to the axial forces in the individual Truss member (See *BCSI-B2* and *BCSI-B3*).

Person: Individual or organization that may exist in accordance with the Legal Requirements. (The term "Person" as used in this Chapter 2 may either appear as "Person" or "person.")

Registered Design Professional: Architect or engineer, who is licensed to practice their respective design profession as defined by the Legal Requirements of the Jurisdiction in which the Building is to be constructed.

Standard: *National Design Standard for Metal Plate Connected Wood Truss Construction (ANSI/TPI 1)*.

Structural Element: Single structural member (other than a Truss) that is specified in the Construction Documents.

Temporary Installation Restraint/Bracing: Lateral Restraint and Diagonal Bracing installed during construction for the purposes of holding Trusses in their proper location, plumb and in plane, until Permanent Individual Truss Member Restraint, Diagonal Bracing and Permanent Building Stability Bracing are completely installed (See *BCSI-B1*, *BCSI-B2*, *BCSI-B3*, *BCSI-B7*, and *BCSI-B10*).

Truss: Individual metal-plate-connected wood component manufactured for the construction of a Building.

Truss Design Drawing: Written, graphic and pictorial depiction of an individual Truss that includes the information required in Sections 2.3.5.5 and 2.4.5.4.

Truss Design Engineer: Person who is licensed to practice engineering as defined by the Legal Requirements of the Jurisdiction in which the Building is to be constructed and who supervises the preparation of the Truss Design Drawings.

Truss Designer: Person responsible for the preparation of the Truss Design Drawings.

Truss Manufacturer: Person engaged in the fabrication of Trusses.

Truss Placement Diagram: Illustration identifying the assumed location of each Truss.

Truss Submittal Package: Package consisting of each individual Truss Design Drawing, and, as applicable, the Truss Placement Diagram, the Cover/Truss Index Sheet, Lateral Restraint and Diagonal Bracing details designed in accordance with generally accepted engineering practice, applicable *BCSI*-defined Lateral Restraint and Diagonal Bracing details, and any other structural details germane to the Trusses.

2.3 RESPONSIBILITIES WHERE THE LEGAL REQUIREMENTS MANDATE A REGISTERED DESIGN PROFESSIONAL FOR BUILDINGS

2.3.1 Requirements of the Owner.

2.3.1.1 Building Permit.

Where required by Legal Requirements, including the Building Code, the Owner shall obtain a Building Permit.

If special inspections or structural observations related to Trusses are required as part of the Construction Documents and/or permitting process, these requirements shall be communicated in writing to the Contractor or Truss Manufacturer as appropriate.

2.3.1.2 Registered Design Professional Designation.

The Owner shall engage and designate on the Building Permit application the Registered Design Professional for the Building.

2.3.1.3 Engagement with the Registered Design Professional.

The Owner shall engage a Registered Design Professional to prepare the Construction Documents and review the Truss Submittal Package.

The Truss Manufacturer shall be notified in writing by either the Owner or Contractor if the Registered Design Professional for the Building is changed or is unable to continue to perform their duties.

2.3.1.4 Engagement with the Contractor.

The Owner shall engage a Contractor to store, handle and install the Trusses for the Building, in compliance with any and all Legal Requirements.

2.3.1.5 Review and Coordinate Submittal Packages.

The Owner or Owner's representative shall be responsible for ensuring that the requirement of Section 2.3.4.2 is accomplished.

2.3.1.6 Long Span Truss Requirements.

2.3.1.6.1 Restraint/Bracing Design.

In all cases where a Truss clear span is 60 ft. (18 m) or greater, the Owner shall contract with any Registered Design Professional for the design of the Temporary Installation Restraint/Bracing and the Permanent Individual Truss Member Restraint and Diagonal Bracing.

2.3.1.6.2 Special Inspection.

In all cases where a Truss clear span is 60 ft. (18 m) or greater, the Owner shall contract with any Registered

Design Professional to provide special inspections to assure that the Temporary Installation Restraint/Bracing and the Permanent Individual Truss Member Restraint and Diagonal Bracing are installed properly.

2.3.1.7 Responsibility Exemptions.

The Owner is responsible for items listed in Section 2.3.1, and is not responsible for the requirements of other parties specified outside of Section 2.3.1.

2.3.2 Requirements of the Registered Design Professional.

2.3.2.1 Construction Documents.

The Construction Documents shall be prepared by the Registered Design Professional for the Building and shall be of sufficient clarity to indicate the location, nature and extent of the work proposed, and show in detail that such documents conform to the Legal Requirements, including the Building Code.

2.3.2.2 Deferred Submittals.

The Registered Design Professional for the Building shall list the Deferred Submittals on the Construction Documents. The Registered Design Professional shall review Deferred Submittals in accordance with Section 2.3.2.3.

2.3.2.3 Review Submittal Packages.

The Registered Design Professional for the Building shall review the Truss Submittal Package for compatibility with the Building design. All such submittals shall include a notation indicating that they have been reviewed and whether or not they have been found to be in general conformance with the design of the Building.

2.3.2.4 Required Information in the Construction Documents.

The Registered Design Professional for the Building, through the Construction Documents, shall provide information sufficiently accurate and reliable to be used for facilitating the supply of the Structural Elements and other information for developing the design of the Trusses for the Building, and shall provide the following:

- (a) All Truss and Structural Element orientations and locations.
- (b) Information to fully determine all Truss profiles.
- (c) All Structural Element and Truss support locations and bearing conditions (including the allowable bearing stress).

- (d) The location, direction, and magnitude of all dead, live, and lateral loads applicable to each Truss including, but not limited to, loads attributable to: roof, floor, partition, mechanical, fire sprinkler, attic storage, rain and ponding, wind, snow (including snow drift and unbalanced snow), seismic; and any other loads on the Truss;
- (e) All anchorage designs required to resist uplift, gravity, and lateral loads.
- (f) Truss-to-Structural Element connections, but not Truss-to-Truss connections.
- (g) Permanent Building Stability Bracing; including Truss anchorage connections to the Permanent Building Stability Bracing.
- (h) Criteria related to serviceability issues including:
 - (1) Allowable vertical, horizontal or other required deflection criteria.
 - (2) Any dead load, live load, and in-service creep deflection criteria for flat roofs subject to ponding loads.
 - (3) Any Truss camber requirements.
 - (4) Any differential deflection criteria from Truss-to-Truss or Truss-to-adjacent Structural Element.
 - (5) Any deflection and vibration criteria for floor Trusses including:
 - (a) Any strongback bridging requirements.
 - (b) Any dead load, live load, and in-service creep deflection criteria for floor Trusses supporting stone or ceramic tile finishes.
 - (6) Moisture, temperature, corrosive chemicals and gases expected to result in:
 - (a) Wood moisture content exceeding 19 percent,
 - (b) Sustained temperatures exceeding 150 degrees F, and/or

- (c) Corrosion potential from wood preservatives or other sources that may be detrimental to Trusses.

2.3.2.5 Responsibility Exemptions.

The Registered Design Professional for the Building is responsible for items listed in Section 2.3.2, and is not responsible for the requirements of other parties specified outside of Section 2.3.2.

2.3.3 Requirements for the Permanent Member Restraint/Bracing of Truss Systems.

2.3.3.1 Method of Restraint.

The method of Permanent Individual Truss Member Restraint/Bracing and the method of anchoring or restraining to prevent lateral movement of all Truss members acting together as a system shall be accomplished by:

2.3.3.1.1 Standard Industry Details.

Standard industry Lateral Restraint and Diagonal Bracing details in accordance with *BCSI-B3: Permanent Restraint/Bracing of Chords & Web Members* and/or *BCSI-B7: Temporary & Permanent Restraint/Bracing for Parallel Chord Trusses* of the Building Component Safety Information (*BCSI*).

2.3.3.1.2 Substitution with Reinforcement.

Permanent Individual Truss Member Restraint shall be permitted to be replaced with reinforcement designed to prevent buckling (e.g., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.).

2.3.3.1.3 Project Specific Design.

A project specific Truss member permanent Lateral Restraint/bracing design for the roof or floor Framing Structural System shall be permitted to be specified by any Registered Design Professional.

2.3.3.2 Method Specified by any Registered Design Professional.

The method of Permanent Individual Truss Member Restraint and Diagonal Bracing for the Truss Top Chord, Bottom Chord, and Web members shall be permitted to be specified by any Registered Design Professional.

2.3.3.3 Absence of Truss Restraint/Bracing Method or Details.

If a specific Truss member permanent bracing design for the roof or floor Framing Structural System is not provided by the Owner or any Registered Design Professional, the method of Permanent Individual Truss Member Restraint and Diagonal Bracing for the Truss Top Chord,

Bottom Chord, and Web members shall be in accordance with *BCSI-B3* or *BCSI-B7*.

2.3.3.4 Trusses Spanning 60 Feet (18 m) or Greater.

For Trusses with clear spans 60 ft. (18 m) or greater, see Section 2.3.1.6.

2.3.4 Requirements of the Contractor.

2.3.4.1 Information Provided to the Truss Manufacturer.

The Contractor shall provide to the Truss Manufacturer a copy of all Construction Documents pertinent to the Framing Structural System and the design of the Trusses (i.e., framing plans, specifications, details, structural notes), and the name of the Registered Design Professional for the Building if not noted on the Construction Documents.

Amended Construction Documents upon approval through the plan review/permitting process shall be immediately communicated to the Truss Manufacturer.

2.3.4.2 Information Provided to the Registered Design Professional.

The Contractor, after reviewing and/or approving the Truss Submittal Package, shall forward the Truss Submittal Package to the Registered Design Professional for the Building for review.

2.3.4.3 Truss Submittal Package Review.

The Contractor shall not proceed with the Truss installation until the Truss Submittal Package has been reviewed by the Registered Design Professional for the Building.

2.3.4.4 Means and Methods.

The Contractor is responsible for the construction means, methods, techniques, sequences, procedures, programs, and safety in connection with the receipt, storage, handling, installation, restraining, and bracing of the Trusses.

2.3.4.5 Truss Installation.

The Contractor shall ensure that the Building support conditions are of sufficient strength and stability to accommodate the loads applied during the Truss installation process. Truss installation shall comply with installation tolerances shown in *BCSI-B1*. Temporary Installation Restraint/Bracing for the Truss system and the permanent Truss system Lateral Restraint and Diagonal Bracing for the completed Building and any other construction work related directly or indirectly to the Trusses shall be in-

stalled by the Contractor in accordance with:

- (a) The Construction Documents, and/or
- (b) The Truss Submittal Package.

For Trusses clear spanning 60 ft. (18 m) or greater, see Section 2.3.1.6.

2.3.4.6 Pre-Installation Check.

The Contractor shall examine the Trusses delivered to the job site for:

- (a) Dislodged or missing connectors,
- (b) Cracked, dislodged or broken members, or
- (c) Any other damage that may impair the structural integrity of the Truss.

2.3.4.7 Post-Installation Check.

The Contractor shall examine the Trusses after they are erected and installed for:

- (a) Dislodged or missing connectors,
- (b) Cracked, dislodged or broken members, or
- (c) Any other damage that may impair the structural integrity of the Truss.

2.3.4.8 Truss Damage Discovery.

In the event that damage to a Truss is discovered the Contractor shall:

- (a) Ensure that the Truss not be erected, or
- (b) That any area within the Building supported by any such Truss already erected shall be appropriately shored or supported to prevent further damage from occurring and shall remain clear and free of any load imposed by people, plumbing, electrical, mechanical, bridging, bracing, etc. until field repairs have been properly completed per Section 2.3.4.9.

2.3.4.9 Truss Damage Responsibilities.

In the event of damage, the Contractor shall:

- (a) Contact the Truss Manufacturer and Registered Design Professional for the Building to determine an adequate field repair, and

- (b) Construct the field repair in accordance with the written instructions and details provided by any Registered Design Professional.

2.3.4.10 Responsibility Exemptions.

The Contractor is responsible for items listed in Section 2.3.4, and is not responsible for the requirements of other parties specified outside of Section 2.3.4.

2.3.5 Requirements of the Truss Design Engineer.

2.3.5.1 Preparation of Truss Design Drawings.

The Truss Design Engineer shall supervise the preparation of the Truss Design Drawings based on the Truss design criteria and requirements set forth in the Construction Documents or as otherwise set forth in writing by the Registered Design Professional for the Building as supplied to the Truss Design Engineer by the Contractor through the Truss Manufacturer.

2.3.5.2 Single Truss Component Design.

The Truss Design Engineer shall be responsible for the single Truss component design depicted on the Truss Design Drawing.

2.3.5.3 Truss Design Drawing Seal and Signature.

Each individual Truss Design Drawing shall bear the seal and signature of the Truss Design Engineer.

Exception: When a Cover/Truss Index Sheet is used, it is the only document required to be signed and sealed by the Truss Design Engineer.

2.3.5.4 Truss Placement Diagram.

When the Truss Placement Diagram serves only as a guide for Truss installation, it does not require the seal of the Truss Design Engineer.

2.3.5.5 Information on Truss Design Drawings.

Truss Design Drawings shall include, at a minimum, the information specified below:

- (a) Building Code used for design, unless specified on Cover/Truss Index Sheet.
- (b) Slope or depth, span and spacing.
- (c) Location of all joints and support locations.
- (d) Number of plies if greater than one.
- (e) Required bearing widths.

- (f) Design loads as applicable, including:
 - (1) Top Chord live load (for roof Trusses, this shall be the controlling case of live load or snow load);
 - (2) Top Chord dead load;
 - (3) Bottom Chord live load;
 - (4) Bottom Chord dead load;
 - (5) Additional loads and locations;
 - (6) Environmental load design criteria (wind speed, snow, seismic, and all applicable factors as required to calculate the Truss loads); and
 - (7) Other lateral loads, including drag strut loads.
- (g) Adjustments to Wood Member and Metal Connector Plate design values for conditions of use.
- (h) Maximum reaction force and direction, including maximum uplift reaction forces where applicable.
- (i) Metal Connector Plate type, manufacturer, size, and thickness or gauge, and the dimensioned location of each Metal Connector Plate except where symmetrically located relative to the joint interface.
- (j) Size, species and grade for each Wood Member.
- (k) Truss-to-Truss connection and Truss field assembly requirements.
- (l) Calculated span to deflection ratio and/or maximum vertical and horizontal deflection for live and total load and K_{CR} as applicable.
- (m) Maximum axial tension and compression forces in the Truss members.
- (n) Fabrication tolerance per Section 6.4.10.
- (o) Required Permanent Individual Truss Member Restraint location and the method of Restraint/Bracing to be used per Section 2.3.3.

2.3.5.6 Responsibility Exemptions.

The Truss Design Engineer is responsible for items listed in Section 2.3.5, and is not responsible for the requirements of other parties specified outside of Section 2.3.5.

2.3.6 Requirements of the Truss Manufacturer.

2.3.6.1 Truss Design Criteria and Requirements.

The Truss Manufacturer shall obtain the Truss design criteria and requirements from the Construction Documents.

2.3.6.2 Communication to Truss Design Engineer.

The Truss Manufacturer shall communicate the Truss design criteria and requirements to the Truss Design Engineer.

2.3.6.3 Alternate Truss Designs.

If an alternative or partial set of Truss design(s) is proposed by either the Truss Manufacturer or the Truss Design Engineer, such alternative set of design(s) shall be sent to and reviewed by the Registered Design Professional for the Building prior to manufacturing. These alternative set of design(s) do not require the seal of the Truss Design Engineer until accepted by the Registered Design Professional for the Building, whereupon these alternative Truss Design Drawings shall be sealed by the Truss Design Engineer.

2.3.6.4 Truss Placement Diagram.

Where required by the Construction Documents or Contract, the Truss Manufacturer shall prepare the Truss Placement Diagram that identifies the assumed location for each individually designated Truss and references the corresponding Truss Design Drawing. The Truss Placement Diagram shall be permitted to include identifying marks for other products including Structural Elements, so that they may be more easily identified by the Contractor during field erection. When the Truss Placement Diagram serves only as a guide for Truss installation and requires no engineering input, it does not require the seal of any Truss Design Engineer or Registered Design Professional.

2.3.6.5 Required Documents.

The Truss Manufacturer shall supply to the Contractor the Truss Submittal Package, including the Truss Design Drawings sealed by a Truss Design Engineer, a Truss Placement Diagram, if required by the Construction Documents or Contract, and the required Permanent Individual Truss Member Restraint and the method to be used per Section 2.3.3.

2.3.6.6 Special Application Conditions.

The Truss Manufacturer shall be allowed to provide detail drawings to the Contractor to document special application conditions.

2.3.6.7 Truss Submittal Packages.

Where required by the Construction Documents or Contract, Legal Requirements or the Building Official, the Truss Manufacturer shall provide the appropriate Truss Submittal Package to one or more of the following: Building Official; Registered Design Professional for the Building and/or Contractor for review and/or approval per Section 2.3.4.2.

2.3.6.8 Reliance on Construction Documents.

The Truss Manufacturer shall be permitted to rely on the accuracy and completeness of information furnished in the Construction Documents or otherwise furnished in writing by the Registered Design Professional for the Building and/or Contractor.

2.3.6.9 Fabrication Tolerance.

The Truss Manufacturer shall determine the value for the fabrication tolerance to be used in the design of the Trusses (see Section 6.4.10).

2.3.6.10 Manufacturer Quality Criteria.

The Truss Manufacturer shall manufacture the Trusses in accordance with the final Truss Design Drawings, using the quality criteria required by this Standard unless more stringent quality criteria is provided by the Owner in writing or through the Construction Documents.

2.3.6.11 In-Plant Truss Inspections.

Truss inspections, as required by the Jurisdiction, shall be performed at the manufacturer's facility using the manufacturer's In-Plant Quality Assurance Program monitored by an inspection agency approved by the Jurisdiction, and shall satisfy any Quality Control/quality assurance requirements for the Trusses, and shall satisfy any designated in-plant special inspection requirements for the Trusses.

2.3.6.12 Responsibility Exemptions.

The Truss Manufacturer is responsible for items listed in Section 2.3.6, and is not responsible for the requirements of other parties specified outside of Section 2.3.6.

2.4 RESPONSIBILITIES WHERE THE LEGAL REQUIREMENTS DO NOT MANDATE A REGISTERED DESIGN PROFESSIONAL FOR BUILDINGS

2.4.1 Requirements of the Owner.

2.4.1.1 Building Permit.

Where required by Legal Requirements, including the Building Code, the Owner shall obtain a Building Permit.

If special inspections or structural observations related to Trusses are required as part of the Construction Documents and/or permitting process, these requirements shall be communicated in writing to the Contractor or Truss Manufacturer as appropriate.

2.4.1.2 Engagement with the Building Designer.

The Owner shall engage a Building Designer to prepare the Construction Documents.

In the absence of an independent Building Designer, the Owner shall assume the role of Building Designer.

2.4.1.3 Engagement with the Contractor.

The Owner shall engage a Contractor to store, handle and install the Trusses for the Building, in compliance with any and all Legal Requirements.

2.4.1.4 Review and Coordinate Submittal Packages.

The Owner or Owner's representative shall be responsible for ensuring that the requirement of Section 2.4.4.2 is accomplished.

2.4.1.5 Long Span Truss Requirements.

2.4.1.5.1 Restraint/Bracing Design.

In all cases where a Truss clear span is 60 ft. (18 m) or greater, the Owner shall contract with any Registered Design Professional for the design of the Temporary Installation Restraint/Bracing and the Permanent Individual Truss Member Restraint and Diagonal Bracing.

2.4.1.5.2 Special Inspection.

In all cases where a Truss clear span is 60 ft. (18 m) or greater, the Owner shall contract with any Registered Design Professional to provide special inspections to assure that the Temporary Installation Restraint/Bracing and the Permanent Individual Truss Member Restraint and Diagonal Bracing are installed properly.

2.4.1.6 Responsibility Exemptions.

The Owner is responsible for items listed in Section 2.4.1, and is not responsible for the requirements of other parties specified outside of Section 2.4.1.

2.4.2 Requirements of the Building Designer.

2.4.2.1 Construction Documents.

The Construction Documents shall be prepared by a Building Designer and shall be of sufficient clarity to indicate the location, nature and extent of the work proposed, and show in detail that such documents conform to the Legal Requirements, including the Building Code.

2.4.2.2 Deferred Submittals.

The Building Designer shall list the Deferred Submittals on the Construction Documents. The Building Designer shall review Deferred Submittals in accordance with Section 2.4.2.3.

2.4.2.3 Review Submittal Packages.

The Building Designer shall review the Truss Submittal Package for compatibility with the Building design. All such submittals shall include a notation indicating that they have been reviewed and whether or not they have been found to be in general conformance with the design of the Building.

2.4.2.4 Required Information in the Construction Documents.

The Building Designer, through the Construction Documents, shall provide information sufficiently accurate and reliable to be used for facilitating the supply of the Structural Elements and other information for developing the design of the Trusses for the Building, and shall provide the following:

- (a) All Truss and Structural Element orientations and locations.
- (b) Information to fully determine all Truss profiles.
- (c) All Structural Element and Truss support locations and bearing conditions (including the allowable bearing stress).
- (d) The location, direction, and magnitude of all dead, live, and lateral loads applicable to each Truss including, but not limited to, loads attributable to: roof, floor, partition, mechanical, fire sprinkler, attic storage, rain and ponding, wind, snow (including snow drift and unbalanced snow), seismic; and any other loads on the Truss.
- (e) All anchorage designs required to resist uplift, gravity, and lateral loads.

- (f) Adequate Truss-to-Structural Element connections, but not Truss-to-Truss connections.
- (g) Permanent Building Stability Bracing; including Truss anchorage connections to the Permanent Building Stability Bracing.
- (h) Criteria related to serviceability issues including:
 - (1) Allowable vertical, horizontal or other required deflection criteria.
 - (2) Any dead load, live load, and in-service creep deflection criteria for flat roofs subject to ponding loads.
 - (3) Any Truss camber requirements.
 - (4) Any differential deflection criteria from Truss-to-Truss or Truss-to- adjacent Structural Element.
- (5) Any deflection and vibration criteria for floor Trusses including:
 - (a) Any strongback bridging requirements.
 - (b) Any dead load, live load, and in-service creep deflection criteria for floor Trusses supporting stone or ceramic tile finishes.
- (6) Moisture, temperature, corrosive chemicals and gases expected to result in:
 - (a) Wood moisture content exceeding 19 percent,
 - (b) Sustained temperatures exceeding 150 degrees F, and/or
 - (c) Corrosion potential from wood preservatives or other sources that may be detrimental to Trusses.

2.4.2.5 Responsibility Exemptions.

The Building Designer is responsible for items listed in Section 2.4.2, and is not responsible for the requirements of other parties specified outside of Section 2.4.2.

2.4.3 Requirements for the Permanent Member Restraint/Bracing of Truss Systems.

2.4.3.1 Method of Restraint.

The method of Permanent Individual Truss Member Restraint/Bracing and the method of anchoring or restraining to prevent lateral movement of all Truss members acting together as a system shall be accomplished by:

2.4.3.1.1 Standard Industry Details.

Standard industry Lateral Restraint and Diagonal Bracing details in accordance with *BCSI-B3: Permanent Restraint/Bracing of Chords & Web Members* and/or *BCSI-B7: Temporary & Permanent Restraint/Bracing for Parallel Chord Trusses* of the Building Component Safety Information (*BCSI*).

2.4.3.1.2 Substitution with Reinforcement.

Permanent Individual Truss Member Restraint shall be permitted to be replaced with reinforcement designed to prevent buckling (e.g., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.).

2.4.3.1.3 Project Specific Design.

A project specific Truss member permanent Lateral Restraint/bracing design for the roof or floor Framing Structural System shall be permitted to be specified by any Building Designer.

2.4.3.2 Method Specified by any Building Designer.

The method of Permanent Individual Truss Member Restraint and Diagonal Bracing for the Truss Top Chord, Bottom Chord, and Web members shall be permitted to be specified by any Building Designer.

2.4.3.3 Absence of Truss Restraint/Bracing Method or Details.

If a specific Truss member permanent bracing design for the roof or floor Framing Structural System is not provided by the Owner or any Building Designer, the method of Permanent Individual Truss Member Restraint and Diagonal Bracing for the Truss Top Chord, Bottom Chord, and Web members shall be in accordance with *BCSI-B3* or *BCSI-B7*.

2.4.3.4 Trusses Spanning 60 Feet (18 m) or Greater.

For Trusses with clear spans 60 ft. (18 m) or greater, see Section 2.4.1.5.

2.4.4 Requirements of the Contractor.

2.4.4.1 Information Provided to the Truss Manufacturer.

The Contractor shall provide to the Truss Manufacturer a copy of all Construction Documents pertinent to the Framing Structural System and the design of the Truss-

es (i.e., framing plans, specifications, details, structural notes), and the name of the Building Designer if not noted on the Construction Documents.

Amended Construction Documents upon approval through the plan review/permitting process shall be immediately communicated to the Truss Manufacturer.

2.4.4.2 Information Provided to the Building Designer.

The Contractor, after reviewing and/or approving the Truss Submittal Package, shall forward the Truss Submittal Package to the Building Designer for review.

2.4.4.3 Truss Submittal Package Review.

The Contractor shall not proceed with the Truss installation until the Truss Submittal Package has been reviewed by the Building Designer.

2.4.4.4 Means and Methods.

The Contractor is responsible for the construction means, methods, techniques, sequences, procedures, programs, and safety in connection with the receipt, storage, handling, installation, restraining, and bracing of the Trusses.

2.4.4.5 Truss Installation.

The Contractor shall ensure that the Building support conditions are of sufficient strength and stability to accommodate the loads applied during the Truss installation process. Truss installation shall comply with installation tolerances shown in *BCSI-B1*. Temporary Installation Restraint/Bracing for the Truss system and the permanent Truss system Lateral Restraint and Diagonal Bracing for the completed Building and any other construction work related directly or indirectly to the Trusses shall be installed by the Contractor in accordance with:

- (a) The Construction Documents, and/or
- (b) The Truss Submittal Package.

For Trusses clear spanning 60 ft. (18 m) or greater, see Section 2.4.1.5.

2.4.4.6 Pre-Installation Check.

The Contractor shall examine the Trusses delivered to the jobsite for:

- (a) Dislodged or missing connectors,
- (b) Cracked, dislodged or broken members, or

- (c) Any other damage that may impair the structural integrity of the Truss.

2.4.4.7 Post-Installation Check.

The Contractor shall examine the Trusses after they are erected and installed for:

- (a) Dislodged or missing connectors,
- (b) Cracked, dislodged or broken members, or
- (c) Any other damage that may impair the structural integrity of the Truss.

2.4.4.8 Truss Damage Discovery.

In the event that damage to a Truss is discovered that would likely impair the structural integrity of the Truss, the Contractor shall:

- (a) Ensure that the Truss not be erected, or
- (b) That any area within the Building supported by any such Truss already erected shall be appropriately shored or supported to prevent further damage from occurring and shall remain clear and free of any load imposed by people, plumbing, electrical, mechanical, bridging, bracing, etc. until field repairs have been properly completed per Section 2.4.4.9.

2.4.4.9 Truss Damage Responsibilities.

In the event of damage, the Contractor shall:

- (a) Contact the Truss Manufacturer to determine an adequate field repair, and
- (b) Construct the field repair in accordance with the written instructions and details provided by any Registered Design Professional.

2.4.4.10 Responsibility Exemptions.

The Contractor is responsible for items listed in Section 2.4.4, and is not responsible for the requirements of other parties specified outside of Section 2.4.4.

2.4.5 Requirements of the Truss Designer.

2.4.5.1 Preparation of Truss Design Drawings.

The Truss Designer is responsible for the preparation of the Truss Design Drawings based on the Truss design criteria and requirements set forth in the Construction Documents or as otherwise set forth in writing by the Building Designer as supplied to the Truss Designer by the Truss Manufacturer.

2.4.5.2 Single Truss Component Design.

The Truss Designer shall be responsible for the single Truss component design depicted on the Truss Design Drawing.

2.4.5.3 Truss Placement Diagram.

When the Truss Placement Diagram serves only as a guide for Truss installation, it does not require the seal of the Truss Design Engineer.

2.4.5.4 Information on Truss Design Drawings.

Truss Design Drawings shall include, at a minimum, the information specified below:

- (a) Building Code used for Design, unless specified on Cover/Truss Index Sheet.
- (b) Slope or depth, span and spacing.
- (c) Location of all joints and support locations.
- (d) Number of plies if greater than one.
- (e) Required bearing widths.
- (f) Design loads as applicable, including:
 - (1) Top Chord live load (for roof Trusses, this shall be the controlling case of live load or snow load);
 - (2) Top Chord dead load;
 - (3) Bottom Chord live load;
 - (4) Bottom Chord dead load;
 - (5) Additional loads and locations;
 - (6) Environmental load design criteria (wind speed, snow, seismic, and all applicable factors as required to calculate the Truss loads); and
 - (7) Other lateral loads, including drag strut loads.
- (g) Adjustments to Wood Member and Metal Connector Plate design values for conditions of use.
- (h) Maximum reaction force and direction, including maximum uplift reaction forces where applicable.

- (i) Metal Connector Plate type, manufacturer, size, and thickness or gauge, and the dimensioned location of each Metal Connector Plate except where symmetrically located relative to the joint interface.
- (j) Size, species and grade for each Wood Member.
- (k) Truss-to-Truss connection and Truss field assembly requirements.
- (l) Calculated span to deflection ratio and/or maximum vertical and horizontal deflection for live and total load and K_{CR} as applicable.
- (m) Maximum axial tension and compression forces in the Truss members.
- (n) Fabrication tolerance per Section 6.4.10.
- (o) Required Permanent Individual Truss Member Restraint location and the method of Restraint/Bracing to be used per Section 2.4.3.

2.4.5.5 Responsibility Exemptions.

The Truss Designer is responsible for items listed in Section 2.4.5, and is not responsible for the requirements of other parties specified outside of Section 2.4.5.

2.4.6 Requirements of the Truss Manufacturer.

2.4.6.1 Truss Design Criteria and Requirements.

The Truss Manufacturer shall obtain the Truss design criteria and requirements from the Construction Documents.

2.4.6.2 Communication to Truss Designer.

The Truss Manufacturer shall communicate the Truss design criteria and requirements to the Truss Designer.

2.4.6.3 Alternate Truss Designs.

If an alternative or partial set of Truss design(s) is proposed by either the Truss Manufacturer or the Truss Designer, such alternative set of design(s) shall be sent to and reviewed by the Building Designer prior to manufacturing.

2.4.6.4 Truss Placement Diagram.

Where required by the Construction Documents or Contract, the Truss Manufacturer shall prepare the Truss Placement Diagram that identifies the assumed location for each individually designated Truss and references the corresponding Truss Design Drawing. The Truss

Placement Diagram shall be permitted to include identifying marks for other products including Structural Elements, so that they may be more easily identified by the Contractor during field erection.

2.4.6.5 Required Documents.

The Truss Manufacturer shall supply to the Contractor the Truss Submittal Package, including the Truss Design Drawings, a Truss Placement Diagram, if required by the Construction Documents or Contract, and the required Permanent Individual Truss Member Restraint and the method to be used per Section 2.4.3.

2.4.6.6 Special Application Conditions.

The Truss Manufacturer shall be allowed to provide detail drawings to the Contractor to document special application conditions.

2.4.6.7 Truss Submittal Packages.

Where required by the Construction Documents or Contract, Legal Requirements or the Building Official, the Truss Manufacturer shall provide the appropriate Truss Submittal Package to one or more of the following: Building Official; Building Designer and/or Contractor for review and/or approval per Section 2.4.4.2.

2.4.6.8 Reliance on Construction Documents.

The Truss Manufacturer shall be permitted to rely on the accuracy and completeness of information furnished in the Construction Documents or otherwise furnished in writing by the Building Designer and/or Contractor.

2.4.6.9 Fabrication Tolerance.

The Truss Manufacturer shall determine the value for the fabrication tolerance to be used in the design of the Trusses (see Section 6.4.10).

2.4.6.10 Manufacturer Quality Criteria.

The Truss Manufacturer shall manufacture the Trusses in accordance with the final Truss Design Drawings, using the quality criteria required by this Standard unless more stringent quality criteria is provided by the Owner in writing or through the Construction Documents.

2.4.6.11 In-Plant Truss Inspections.

Truss inspections, as required by the Jurisdiction, shall be performed at the manufacturer's facility using the manufacturer's In-Plant Quality Assurance Program (see Section 3.2) monitored by an inspection agency approved by the Jurisdiction, and shall satisfy any Quality Control/quality assurance requirements for the Trusses, and shall satisfy any designated in-plant special inspection require-

ments for the Trusses.

2.4.6.12 Responsibility Exemptions.

The Truss Manufacturer is responsible for items listed in Section 2.4.6, and is not responsible for the requirements of other parties specified outside of Section 2.4.6.

2.5 CONTRACTS

2.5.1 Defer to Construction Documents.

This Chapter of the Standard is not intended to take precedence over the Construction Documents, where a Contract between parties incorporates by reference the Construction Documents, and therefore the Construction Documents shall apply as between the parties to the Contract.

2.5.2 Defer to Contract.

This Chapter of the Standard is not intended to take precedence over a Contract as a Contract shall be permitted to contain provisions that take precedence over the Standard and/or the Construction Documents. A party shall not exclude in a Contract a responsibility established by this Standard or the Construction Documents unless that responsibility is assigned to a qualified party and that party agrees to that assignment.

Any changes made to the Construction Documents by Contract shall be submitted, reviewed and approved by the Building Official.

2.5.3 Incorporation into Contract.

A Contract shall be permitted to incorporate this Chapter of the Standard to establish the Responsibilities of the parties to such Contract.

CHAPTER 3

QUALITY CRITERIA FOR THE MANUFACTURE OF METAL-PLATE-CONNECTED WOOD TRUSSES

3.1 GENERAL

3.1.1 Scope.

Chapter 3 of this Standard is the quality standard for the manufacturing processes of metal-plate-connected wood Trusses, and shall be used in conjunction with a manufacturing quality assurance procedure and a Truss design. These provisions shall be included in the In-Plant Quality Assurance Program of each Truss Manufacturer.

3.1.2 Requirements.

Metal-plate-connected wood Trusses shall meet the minimum manufacturing quality requirements specified in Chapter 3 of this Standard, so that design assumptions are met.

3.1.3 Documentation.

Truss Manufacturers and inspection agencies shall establish methods that document the application of these quality assurance procedures throughout the manufacturing process. The Truss Manufacturer's methods shall be subject to periodic audit for compliance with the requirements of this Standard by an approved inspection agency per Section R109 Inspections of the *International Residential Code* / Section 109 Inspections of the *International Building Code*, where required by local authorities having Jurisdiction, or other means.

3.1.4 Non-Conforming Inspections.

Manufacturing inaccuracies exceeding the allowable tolerances described within Chapter 3 are acceptable upon approval and follow-up documentation by a Truss Designer as defined in Chapter 2 of this Standard. Any necessary repair authorization shall be prepared by a Truss Designer.

3.2 IN-PLANT QUALITY ASSURANCE PROGRAM

3.2.1 In-Plant Quality Control Manual.

An in-plant quality control manual shall be maintained for each Truss manufacturing facility, which will include the requirements for daily quality control and any audits that will be performed. At a minimum, the in-plant quality control manual shall contain:

- (a) Either a production flowchart or a description of the manufacturing process;

- (b) Manufacturer's organizational chart, a description of the duties and Responsibilities assigned to key positions in the quality program;
- (c) Quality control procedures, including sampling criteria and how manufacturing processes are monitored to ensure that the product is consistently manufactured within the allowable tolerances; and
- (d) A document retention policy.

3.2.2 Inspection Frequency.

At a minimum, three Trusses per week per operational set-up location per shift as outlined in the in-plant quality control manual shall be inspected and recorded for in-plant audits.

3.2.3 Inspection Sampling.

A random representative sampling of Trusses shall be chosen for inspection, either off the production line after all pressing operations are completed, or from finished goods storage.

3.2.4 Inspection Procedure.

For Trusses chosen for inspection, the joint inspection procedures of Section 3.7 shall be used.

3.3 TRUSS DESIGN

3.3.1 Truss Design Drawing.

A Truss Design Drawing shall be provided for every Truss, prior to manufacture and inspection. When the Truss Design Drawing specifies quality criteria that conflict with Chapter 3, the Truss Design Drawing shall prevail.

3.3.2 Fabrication Tolerance.

All Truss joints shall be designed using a fabrication tolerance specified by the Truss Manufacturer. The fabrication tolerance correlates to the quality control factor, C_q , as defined in Section 6.4.10.

3.3.3 Roller Press Value Reduction.

Joint QC Details for manufacturing lines utilizing single pass, full embedment rollers shall indicate the minimum permitted roller diameter resulting from Q_R per Section 5.2.5.

3.4 LUMBER

3.4.1 Lumber Specifications.

Truss lumber shall be the size, species and grade specified on the Truss Design Drawing.

3.4.2 Lumber Substitutions.

Truss lumber of a different grade shall be permitted if the substitute grade meets or exceeds the specified grade for each of the following engineering design properties:

- (a) Reference design value for bending (F_b);
- (b) Reference design value for tension (F_t);
- (c) Reference design value for compression parallel to grain (F_c);
- (d) Reference design value for compression perpendicular to grain ($F_{c\perp}$);
- (e) Reference design value for shear (F_v);
- (f) Specific gravity (G);
- (g) Reference modulus of elasticity (E); and
- (h) Reference modulus of elasticity for stability calculations (E_{min}).

Any substitution of a specified Lumber grade not meeting the above requirements, or any substitution of a specified lumber grade to Structural Composite Lumber products shall require the review and approval of a Truss Designer.

3.4.3 Lumber Identification.

Prior to cross-cutting, lumber shall be identified by the grade mark or the certificate of inspection issued by a lumber inspection agency accredited by the Board of Review of the American Lumber Standard Committee.

3.4.4 Preservative Treatment Identification.

Preservative treated lumber shall be identified by the quality mark of, or a certificate of inspection from, an approved inspection agency and shall be identified by a label affixed to the package (see also Section 6.4.9).

3.4.5 Fire Retardant Identification.

Lumber impregnated with fire retardant chemicals shall be identified by the quality mark of, or a certificate of inspection from, an approved inspection agency and shall

be identified by a label affixed to the package (see also Section 6.4.9).

3.4.6 Use of Finger-Jointed Lumber.

Structural finger-jointed lumber shall be permitted to be used interchangeably with solid-sawn members of the same grade and species if the finger joints are manufactured with an adhesive meeting the requirements of *ASTM D2559*. Structural finger-jointed lumber shall be identified by the grade mark of, or certificate of inspection from, a lumber grading or inspection agency that has been approved by an accreditation body that complies with U.S. Department of Commerce (DOC) *PS 20* or equivalent. The grade mark and certification of inspection for structural finger-jointed lumber shall indicate that joint integrity is subject to qualification and quality control. Finger-jointed lumber marked "STUD USE ONLY" or "VERTICAL USE ONLY" shall not be used in metal-plate-connected wood Trusses.

3.4.7 Lumber Splits.

Splits in any Wood Member caused by Metal Connector Plate Teeth or the manufacturing process shall not exceed those permitted in the grade and species of lumber used.

3.5 ASSEMBLY

3.5.1 General.

Trusses shall be manufactured using cutting, jigging, and pressing equipment. The location of Top and Bottom Chords, Webs, and joints shall be as specified on the Truss Design Drawing.

3.5.2 Wood Members.

Members shall be cut in accordance with the Truss Design Drawing.

3.5.3 Height and Length.

Truss height and length dimensions that vary from the Truss Design Drawing shall not exceed the tolerances shown in Table 3.5-1.

Table 3.5-1 In-Plant Manufacturing Tolerances for Finished Truss Units.

	Truss-to-Truss Variance Dimension of Identical Trusses	Variance from Design Dimensions
Length¹ of Finished Truss Unit	1/2 inch	3/4 inch
Height² of Finished Truss Unit	1/4 inch	1/2 inch

1. Length for manufacturing tolerance purposes, is the overall length of the Truss unit, excluding overhangs or extensions.
2. Height, for manufacturing tolerance purposes, is the overall height of the Truss unit measured at joints, excluding projections above the Top Chord and below the Bottom Chord, overhangs and extensions.

3.6 PLATING OF JOINTS

3.6.1 Storage and Care.

Metal Connector Plates shall be protected from damage during storage and shall be in an undamaged condition when used in the manufacture of wood Trusses.

3.6.2 Specifications.

Metal Connector Plates shall be of the gauge, type, manufacturer and size specified by the Truss Designer.

3.6.3 Substitutions.

A Metal Connector Plate with larger dimensions in one or both directions, but of the same type and gauge specified on the Truss Design Drawing, shall be an acceptable substitution provided the requirements of Section 3.6.5 are met. Overplating shall not be considered as a corrective measure for joints not meeting the gap tolerances of Section 3.7.6 unless approved by a Truss Designer.

3.6.4 Installation.

Unless otherwise specified by the Truss Designer, Metal Connector Plates shall be installed on both faces of the Truss at each joint, and positioned in accordance with the Truss Design Drawing. All Trusses

assembled using a single-pass full embedment roller press shall be designated on the Joint QC Detail as being for no less than the given diameter of roller press. Acceptable tolerances for plating are specified in Section 3.7.

3.6.5 Plate Position.

The placement of the Metal Connector Plate shall not interfere with other design aspects or the function of the Truss. Any alternate plate must have the same orientation and entirely cover the area footprint of the intended original plate size as shown on the Truss Design Drawing. Examples are shown in Figure 3.6-1.

3.7 JOINT INSPECTION

3.7.1 Critical Joint Inspection.

The inspection procedures shall apply to joints selected for inspection. The selection of joints for inspection shall be as defined in the Truss Manufacturer's In-Plant Quality Assurance Program. No less than one critical joint per Truss selected for inspection, on average across all operational set-up locations at the Truss manufacturing facility, shall be inspected. Critical joints have a joint stress index (JSI), as defined in Section 8.11.3, greater than or equal to 0.80.

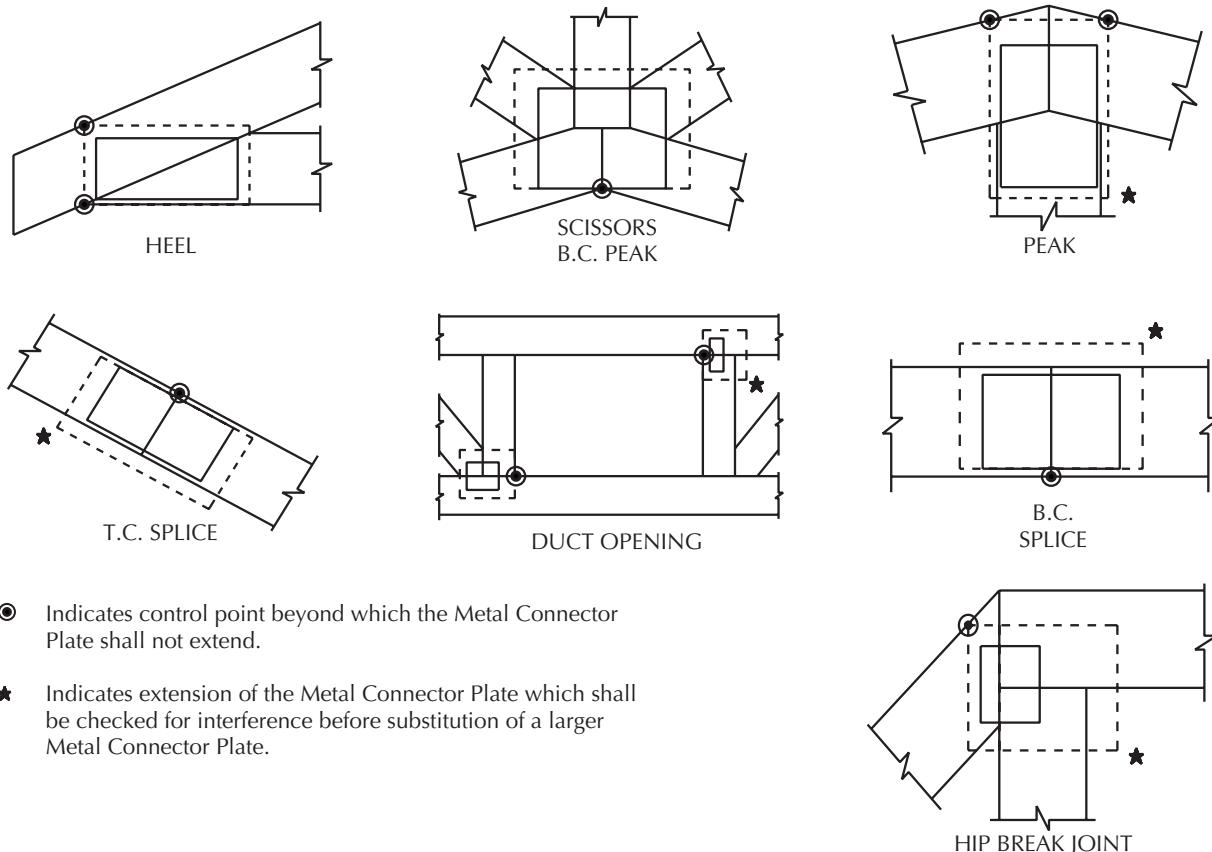


Figure 3.6-1 Examples of Metal Connector Plate Positionings that Affect the Function of a Truss.

3.7.2 Plate Positioning Procedures.

3.7.2.1 Plate Placement.

A Joint QC Detail (see Figure 3.7-1), illustrating the positioning tolerance, shall be obtained for any joint selected for inspection, except as permitted in Section 3.7.2.2. The actual midpoint of the Metal Connector Plate shall be within the selected fabrication tolerance polygon as calculated by the Truss Designer for each joint per Section 8.11. However, if the Joint QC Detail contains no polygons and the actual midpoint is within $\frac{1}{8}$ in. (3 mm) of the specified midpoint, the placement shall be considered acceptable. If all members of a joint selected for inspection are free from characteristics reducing the plate contact area, consisting of lumber characteristics and flattened teeth as outlined in Section 3.7.4, the 0 percent fabrication tolerance polygon shall be used to evaluate plate positioning. If any member contains characteristics reducing the plate contact area, the 0 percent fabrication tolerance polygon shall be permitted to be used provided that Teeth shall be counted and compared to the minimum required Teeth for that member. If fabrication tolerance polygons are smaller than $\frac{1}{2}$ in. (13 mm) and the actual midpoint falls outside of both polygons but within $\frac{1}{2}$ in. (13 mm) of the specified midpoint, it shall be permitted to evaluate the plate per Section 3.7.2.2. If the actual midpoint does not meet any of these requirements, the procedures set forth in Section 3.9.1 shall be followed.

3.7.2.2 Alternative Positioning Procedure.

It shall be permitted that the actual midpoint be within $\frac{1}{2}$ in. (13 mm) from the specified midpoint as shown on

the Truss Design Drawing and Teeth shall be counted for each member of that joint and compared to the minimum required Teeth.

3.7.3 Plate Rotation.

Unless otherwise specified in the Truss Manufacturer's In-Plant Quality Assurance Program or by the Truss Designer, plate orientation shall not vary by more than 10 degrees from the design position. If the plate rotation does not meet this requirement, the procedures set forth in Section 3.9.1 shall be followed.

Exception: If the procedures of Section 3.7.2.2 are followed, a 10 degrees tolerance shall always be allowed.

3.7.4 Plate Contact Area.

3.7.4.1 Lumber Characteristics and Tooth Flattening.

Any combination of lumber characteristics and tooth flattening shall not exceed the fabrication tolerance set by the Truss Manufacturer within the plate contact area except as permitted by Section 3.7.4.2 on any member. The fabrication tolerance shall be permitted to be set at any increment and shall be represented on the Joint QC Detail (see Figure 3.7-1) for the plate contact area of each member. For example, a 0 percent fabrication tolerance assumes that 100 percent of Teeth in the plate contact area are effective. A 20 percent fabrication tolerance assumes that 20 percent of Teeth are in a characteristic reducing the plate contact area (see Figure 3.7-2). Lumber characteristics include, but are not limited to, loose knots, decayed knots, unsound wood, bark, pitch

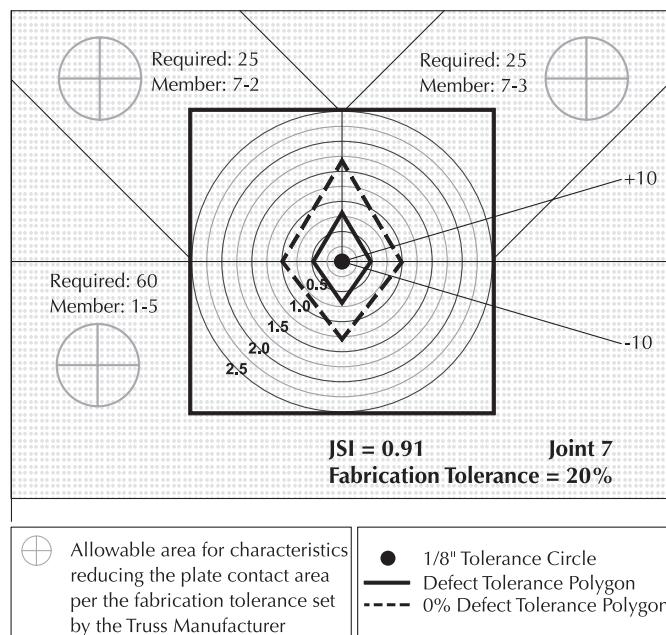


Figure 3.7-1 Example of a Joint QC Detail and Fabrication Tolerance Polygons.

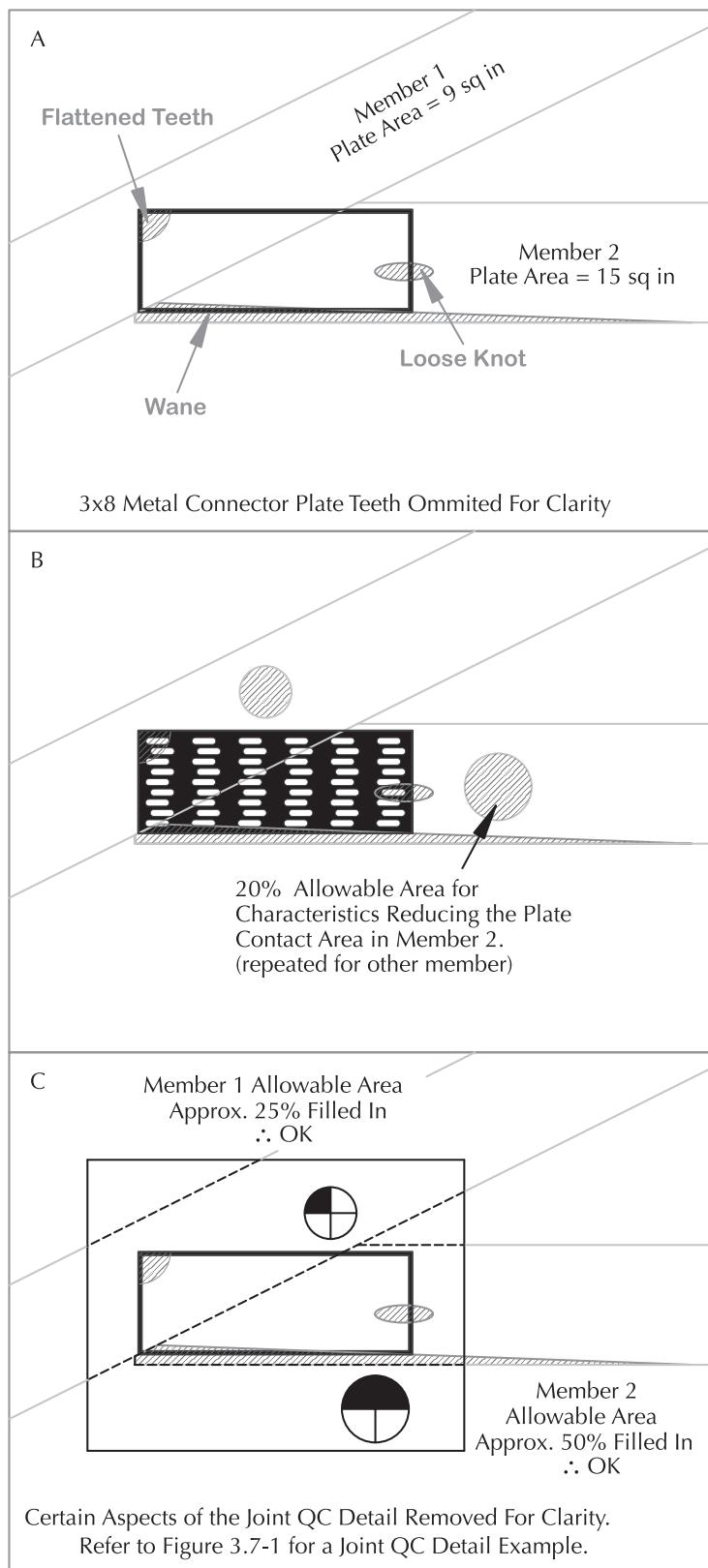


Figure 3.7-2 Lumber Characteristics and Flattened Teeth in Plate Contact Area.

Note: The maximum allowable area (shown as circles) for the characteristics reducing the plate contact area is equal to the fabrication tolerance set by the Truss Manufacturer and used in the design. In this example, a 20 percent fabrication tolerance was used. Truss Manufacturers have the ability to change this percentage based on their In-Plant Quality Assurance Program.

content, holes, and wane. A tooth shall be considered flattened if it meets criteria outlined in Section 3.7.7.2.3.

3.7.4.2 Fabrication Tolerance Exceeded Tooth Count.

A member tooth count in accordance with Section 3.7.7 shall be permitted to be done if characteristics reducing the plate contact area are greater than the member's fabrication tolerance.

3.7.5 Plate Embedment.

3.7.5.1 Plate Embedment Gap Tolerance.

The maximum allowable embedment gap shall be $\frac{1}{32}$ in. (1 mm) except as permitted by Section 3.7.5.2. The plate embedment gap shall be measured at the perimeter of the entire plate except the perimeter within a 1 in. (25 mm) distance from lumber ends and edges along joint lines (see Figure 3.7-3).

3.7.5.2 Excessive Embedment Gap Tooth Count.

A member tooth count in accordance with Section 3.7.7

shall be permitted to be done when the perimeter of the plate has more than a $\frac{1}{32}$ in. (1 mm) gap.

3.7.6 Wood Member-To-Wood Member Gaps.

3.7.6.1 Tolerance.

Except as indicated in Section 3.7.6.2 or as otherwise specified on the Truss Design Drawing, maximum gaps in all joints except floor Truss chord splices shall not exceed $\frac{1}{8}$ in. (3 mm), where the gap is measured at each edge of the Metal Connector Plate for joints in which the plate edge is within the scarf, and measured at the end of the scarf for joints in which the plate edge is outside the scarf. Scarf is the portion of the joint in which it is intended that there be wood-to-wood contact between two Wood Members. The maximum gap for floor Truss chord splices shall not exceed $\frac{1}{16}$ in. (1.5 mm) across the entire scarf. For joints designed with single points of contact between adjacent members as shown on the Truss Design Drawing, the maximum gap between all contact points shall not exceed $\frac{1}{8}$ in. (3 mm) (see Figure 3.7-4).

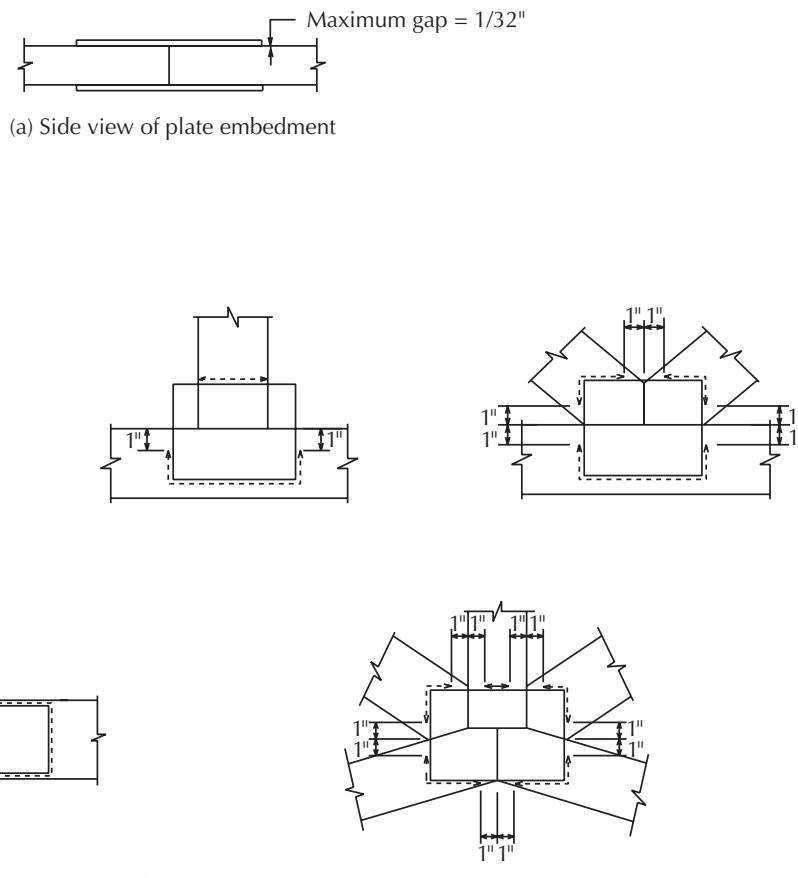


Figure 3.7-3 Plate Embedment Gap Measurement.

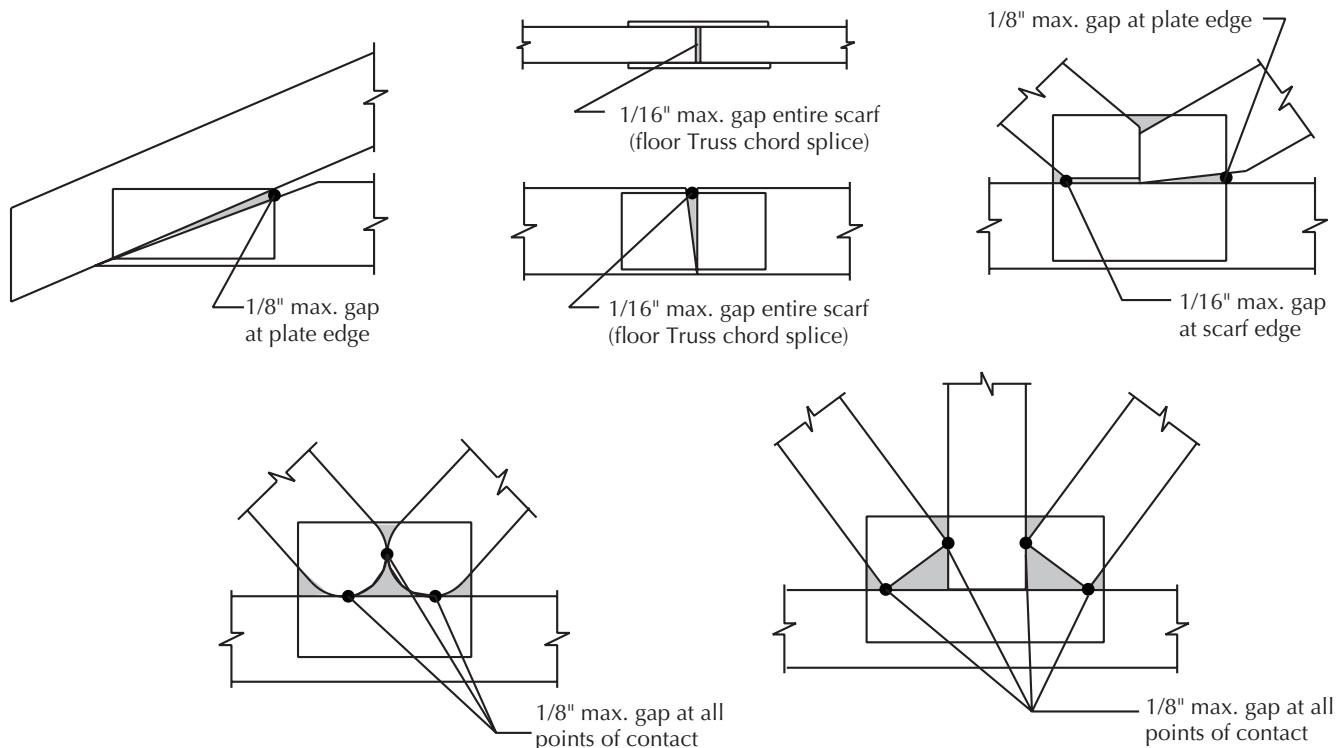


Figure 3.7-4 Wood Member-to-Wood Member Gaps.

3.7.6.2 Compression Load Tolerance.

Where a Metal Connector Plate is designed to carry all compression load at the joint without buckling of the plate steel section, the allowable gap shall be that amount of gap used in sizing the Metal Connector Plate as specified on the Truss Design Drawing by the Truss Designer.

3.7.6.3 Correction Procedure.

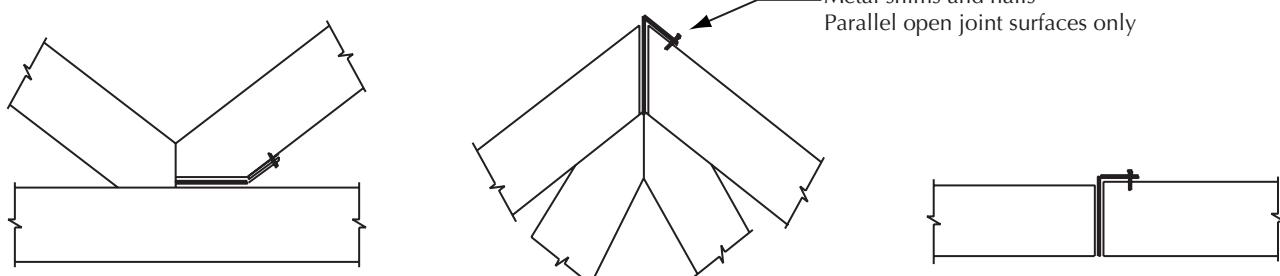
Correction procedures for joints with gaps exceeding these tolerances shall require shimming, unless otherwise specified by a Truss Designer. Shims shall be of galvanized metal, or alternatives approved by a Truss Designer, to obtain firm bearing between members. Metal shims shall be at least $\frac{3}{4}$ in. (19 mm) wide and long enough to bend over at least 1 in. (25 mm) along the member being shimmed. The metal shim shall be fixed in position with a deformed-shank (i.e., ring- or screw-shank) 6d nail (0.120 in. (3 mm)

diameter and 2 in. (5 mm) long), or other fastener capable of resisting withdrawal, to prevent loss or accidental removal (see Figure 3.7-5).

3.7.7 Effective Tooth Count.

3.7.7.1 Total Effective Teeth.

The combined number of effective Teeth for both faces of the Truss at each joint in each Metal Connector Plate contact area shall meet or exceed two times the minimum number specified for a single face by the Truss Designer per Section 6.1.4. The number of effective Teeth for a single plated face in a contact area shall be permitted to be up to 15% less than the number specified for a single face per Section 6.1.4, provided the sum of the number of effective Teeth on both faces meets or exceeds the total required number for both faces for that contact area.



Note: shims are driven to tight contact after Metal Connector Plates are in place.

Figure 3.7-5 Shimming Gaps.

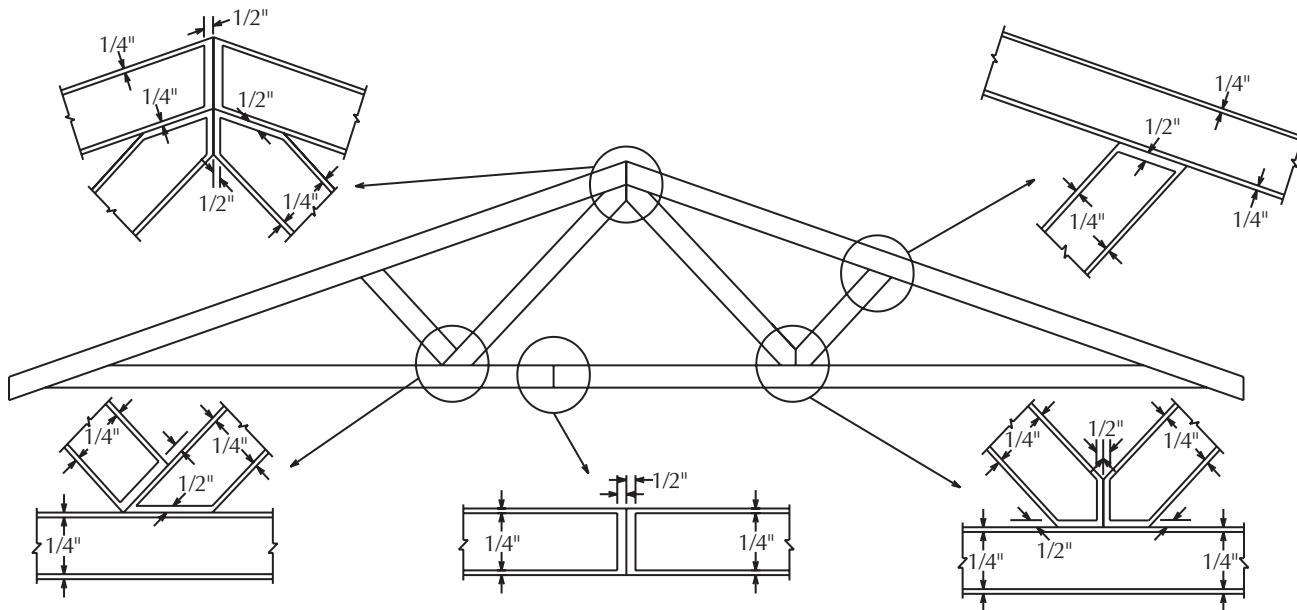


Figure 3.7-6 End and Edge Distances.

3.7.7.2 Number of Effective Teeth.

The number of effective Teeth at each plate contact area shall be determined using the requirements in Sections 3.7.7.2.1 through 3.7.7.2.4.

3.7.7.2.1 Ineffective Teeth – Lumber Characteristics.

Teeth placed in loose knots, decayed knots, unsound wood, bark, pitch content, holes, wane, and joint gaps shall be considered ineffective. Teeth placed in tight knots shall be considered effective.

3.7.7.2.2 Ineffective Teeth – End and Edge Distances.

If the Metal Connector Plate Lateral Resistance design values are based on the Net Area Method, then all Teeth located within the End and Edge Distances according to Figure 3.7-6 shall be considered ineffective.

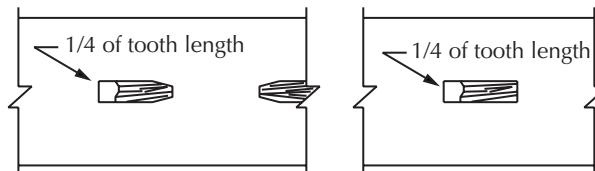
3.7.7.2.3 Ineffective Teeth – Tooth Flattening.

The number of ineffective Teeth due to tooth flattening shall be calculated as twice the number of visibly flattened teeth. A tooth shall be considered flattened if one quarter ($\frac{1}{4}$) or greater of the tooth length is visible within the tooth-slot opening (see Figure 3.7-7). A tooth shall also be considered flattened if the surface of the wood has raised (i.e., wood lifted up beyond its normal surface plane) within the tooth-slot opening of the Metal Connector Plate.

3.7.7.2.4 Ineffective Teeth – Embedment Gap.

The total number of effective Teeth, after excluding Teeth that are considered ineffective per Sections 3.7.7.2.1 through 3.7.7.2.3, shall be determined based on the tooth

effectiveness versus tooth embedment gap ratios shown in Table 3.7-1. Tooth embedment gap is defined as the distance between the underside of the embedded Metal Connector Plate and the surface of the Wood Member. Tooth embedment gap shall be measured through the slot opening of the Metal Connector Plate, with an accuracy of 0.01 in. (0.25 mm).



(a) Top view of flattened teeth - 1/4 of more of the tooth length is visible through the tooth slot.



(b) Cross-section showing flattened teeth.

Figure 3.7-7 Tooth Flattening.

Tooth Embedment Gap, G	Tooth Effectiveness
$G = 0"$	119%
$0" < G \leq 1/32" (0.03")$	100%
$1/32" < G \leq 1/16" (0.06")$	60%
$1/16" < G \leq 3/32" (0.09")$	40%
$G > 3/32" (0.09")$	0%

Table 3.7-1 Tooth Effectiveness.

3.7.8 Re-evaluation.

Trusses that fail to meet the criteria in Section 3.7 shall be re-evaluated and approved, or repaired as specified by a Truss Designer.

3.8 REPRESSING

The repressing of embedded Metal Connector Plates to improve plate embedment shall be permitted.

3.9 REPAIR

3.9.1 Repair Specifications.

When any installed (i.e., embedded) Metal Connector Plate does not meet the requirements of Sections 3.6 and 3.7, a Truss Designer shall do one of the following:

- (a) Specify the repair removing the plate,
- (b) Specify the repair leaving the plate in place, or
- (c) Review and approve the plate “as is”.

3.9.2 Plate Removal.

Metal Connector Plate Teeth installed into lumber which has been damaged (i.e., wood removed, or in violation of Section 3.4.7) by the installation/removal of a previous Metal Connector Plate shall be considered ineffective in the damaged areas.

3.9.3 Lumber Condition.

When a Metal Connector Plate is installed in the connection area of lumber that contains tooth holes from a previously installed plate and where the wood is otherwise undamaged, Metal Connector Plate Teeth shall be considered 50% effective. Metal Connector Plate Teeth in areas that do not overlap those of a previously installed plate (i.e., no tooth holes) are not subject to the reduction.

3.10 GIRDER TRUSS PLY-TO-PLY CONNECTIONS

Multi-ply girders that are fastened together with nails by the Truss Manufacturer at the manufacturing plant shall fasten each ply to the next ply per the requirements of the Truss Design Drawing. The manufacturing plant shall be permitted to place the second ply onto the first ply and nail the two plies together and then place the third ply onto the two ply Truss and nail the third ply into the two ply Truss below it, etc, without turning the multi-ply Truss over. The In-Plant Quality Control Assurance Program (see Section 3.2) shall monitor the ply-to-ply nailing process.

CHAPTER 4

METAL CONNECTOR PLATE MANUFACTURING

4.1 GENERAL

4.1.1 Scope.

Metal Connector Plates used in the manufacturing of wood Trusses shall be manufactured to the requirements of Chapter 4 and to other applicable sections of this Standard. This chapter is designed to provide the Metal Connector Plate manufacturer with procedures and production tolerances for Metal Connector Plates that are used in wood Truss manufacturing.

4.1.2 Requirements.

The provisions of Chapter 4 of this Standard shall be included in the Quality Assurance Program (QAP) of each Metal Connector Plate manufacturer.

4.2 QUALITY ASSURANCE PROGRAM (QAP)

4.2.1 Documentation.

The Metal Connector Plate manufacturer shall have a written QAP approved by executive management.

4.2.2 QAP Manager.

Executive management shall designate a QAP manager who shall report directly to executive management.

4.2.2.1 Selection.

The QAP manager shall not be in the Metal Connector Plate production department.

4.2.2.2 Delegation.

The QAP manager shall be permitted to delegate QAP functions to others, provided all specified data, reports, and noncompliance to this section are submitted to the manager.

4.2.2.3 Authority.

The QAP manager shall have the authority to, and shall, reject any Metal Connector Plates that do not meet the specifications of Chapter 4. Metal Connector Plates so rejected shall be disposed of in such a way that they will not be used in the manufacturing of wood Trusses.

4.2.2.4 Responsibility.

The QAP manager shall inform each Person directly responsible for the procurement of materials and the production of Metal Connector Plates of the requirements of Chapter 4 of this Standard that are applicable to that

Person's function. Each Person shall be responsible for monitoring of the particular Specifications that are assigned to that Person's job function.

4.2.2.5 Record Retention.

The QAP manager shall maintain records and reports for a minimum period of three years.

4.3 PRODUCTION STEEL

4.3.1 Specifications.

All steel used in the manufacturing of Metal Connector Plates shall meet specifications of the Metal Connector Plate manufacturer including minimum yield strength, minimum ultimate tensile strength, minimum thickness, and any other parameters required to assure adequate plate performance.

4.3.2 Master Coil.

4.3.2.1 Certification.

Each master coil of steel shall be marked and shall have a certified report from the producing steel mill that includes the grade of steel, the mechanical properties of the steel, and the chemical properties of the steel.

4.3.2.2 Identification.

Each production coil of steel processed from the master coil shall be marked to indicate the master coil from which it was slit.

4.3.3 Steel Sheet.

4.3.3.1 Requirements.

Metal Connector Plates shall be of galvanized steel conforming to the requirements of Section 4.3.3.2 or Section 4.3.3.3, aluminum-zinc alloy coated steel conforming to the requirements of Section 4.3.3.4, or stainless steel conforming to the requirements of Section 4.3.3.5.

4.3.3.2 Hot-Dip Galvanized Steel.

Hot-dip galvanized steel shall meet or exceed yield and ultimate tensile strengths of *ASTM A653/A653M*, Structural Grade 33, and galvanized coating shall meet or exceed coating designation G60.

4.3.3.3 Electrolytic Galvanized Steel.

Electrolytic galvanized steel shall meet or exceed *ASTM A879* coating designation 30Z30Z. Structural properties shall meet or exceed those specified in Section 4.3.3.2.

4.3.3.4 Aluminum-Zinc Alloy Coated Steel.

Aluminum-zinc alloy coated steel shall meet or exceed *ASTM A792/A792M, AZ50* coating weight. Structural properties shall meet or exceed those specified in Section 4.3.3.2.

4.3.3.5 Stainless Steel.

Stainless steel shall meet or exceed *ASTM A167* or *A240/A240M*. Structural properties shall meet or exceed those specified in Section 4.3.3.2.

4.3.4 Steel Thickness.

Minimum thickness in inches (coated thickness, if galvanized or aluminum-zinc alloy coated), shall be specified for each type of Metal Connector Plate. The following gauge designations shall be permitted to be used as descriptive terms only when the minimum Metal Connector Plate thickness equals or exceeds the thickness listed:

- 20 gauge - 0.0356 in. (0.904 mm)
- 18 gauge - 0.0466 in. (1.184 mm)
- 16 gauge - 0.0575 in. (1.461 mm)
- 14 gauge - 0.0705 in. (1.791 mm)

4.3.5 Records.

The Metal Connector Plate manufacturer shall maintain records that include the following information for each master coil of steel used in Metal Connector Plate production:

- (a) Name of steel producer;
- (b) Material description and specification;
- (c) Heat number;
- (d) Yield point;
- (e) Tensile strength;
- (f) Elongation; and
- (g) Chemical analysis.

4.3.6 Nail-on Plates.

Metal Connector Plates without integral Teeth (nail-on plates), shall provide some means, such as holes, dimples, bosses, or marked pattern, to indicate the location of any separately applied nails for fasteners.

4.3.7 Tooth Tolerances.

Metal Connector Plates shall be manufactured with all

holes, plugs, Teeth, or prongs properly spaced and properly formed per the requirements of the Metal Connector Plate manufacturer.

4.3.8 Marking.

All Metal Connector Plates 3 in. (76 mm) in width or wider, and 25 percent of Metal Connector Plates less than 3 in. (76 mm) in width, shall be individually marked with the name or symbol of the manufacturer.

4.3.9 Regalvanizing.

Regalvanizing the Metal Connector Plates in accordance with *ASTM A153* after the stamping operation shall not be necessary unless specified by the Truss Designer or the Building Designer.

4.4 PRODUCTION

4.4.1 Measurement.

Prior to and during production, each production coil of steel shall be measured to the closest 0.001 in. (0.02 mm) for conformance to minimum thickness specification. Steel with a thickness measurement less than specification shall not be used.

4.4.2 Visual Observations.

During the stamping of Metal Connector Plates, the machine operator shall visually observe production Metal Connector Plates to assure proper forming as follows:

- (a) Metal Connector Plate primary surface shall be flat or as specified;
- (b) Metal Connector Plate Teeth shall be uniform, with no Teeth malformed;
- (c) Roots of Teeth where they join the flat plane of the Metal Connector Plate shall show no abnormal fracture; and
- (d) Teeth shall be formed as specified and shall have the appropriate angle to the plane of the Metal Connector Plate.

4.5 IDENTIFICATION

4.5.1 Plate Markings.

Metal Connector Plates shall be marked as specified in Section 4.3.8.

4.5.2 Package Markings.

Each package or individual shipping unit shall be marked to indicate the production run and master coil of steel in

order to provide recall ability should quality problems be identified at a later date.

4.6 INSPECTION

4.6.1 Frequency.

The QAP manager shall obtain a sample from each production line at a minimum of five times per week, on a random, unannounced schedule. Time, date, production line, and master coil identification code shall be recorded for each sample.

4.6.2 Requirements.

The QAP manager shall inspect each Metal Connector Plate with all data recorded in a journal for the following characteristics:

- (a) Time and date of production, production line, master coil identification code and series, gauge, and size of Metal Connector Plate shall be recorded.
- (b) Metal Connector Plates shall be visually inspected for misforming, flatness, and steel fracture. All Teeth and cut outs shall be uniformly consistent.
- (c) Steel thickness shall be measured to the closest 0.001 in. (0.02 mm).
- (d) Steel hardness shall be measured on the Rockwell B scale to the closest +/- 3. Optionally, the hardness of the production coil shall be measured at time of sampling.
- (e) Width and length of the Metal Connector Plate shall be measured to the closest $\frac{1}{64}$ in. (0.5 mm).
- (f) Several Teeth in each Metal Connector Plate shall be bent at the root line to determine number of bends to fracture. Grip and bend procedure is to be defined in the QAP.

4.7 TOLERANCES

Inspected Metal Connector Plates shall be in accordance with the following:

- (a) Visual observations shall be a qualitative decision by the QAP manager as to acceptability;

- (b) Steel thickness shall be equal to or greater than the minimum specification;
- (c) Steel hardness shall be in a range specified by the QAP manager;
- (d) Metal Connector Plate Width shall not be more than $\frac{1}{32}$ in. (1.0 mm) undersized;
- (e) Metal Connector Plate Length shall not be more than $\frac{1}{64}$ in. (0.5 mm) less than the total length for each 1 in. (25 mm) of specified length; and
- (f) Metal Connector Plate root bend test requirements shall be determined by the QAP manager.

4.8 ACCEPTANCE

Metal Connector Plates meeting the specifications of Chapter 4 of this Standard are acceptable for use in the manufacturing of metal-plate-connected wood Trusses.

4.9 REJECTION

4.9.1 Nonconforming Metal Connector Plates.

If the press operator or supervisor visually observes non-conforming Metal Connector Plates, operation of that production line shall cease until corrections are made, and the QAP manager shall be notified.

4.9.2 Sampling Previous Production.

The QAP manager shall sample previous production and shall reject Metal Connector Plates that do not meet these specifications.

CHAPTER 5

PERFORMANCE EVALUATION OF METAL CONNECTOR PLATED CONNECTIONS

5.1 GENERAL

5.1.1 Scope.

Chapter 5 of this Standard includes three test procedures to determine the performance of metal connector plated connections:

- (a) Determination of lateral resistance of Metal Connector Plate Teeth (see Section 5.2).
- (b) Determination of shear strength of Metal Connector Plates (see Section 5.3).
- (c) Determination of Tensile Strength of Metal Connector Plates (see Section 5.4).

The results of these tests shall be used in designing metal connector plated connections.

5.1.2 Testing Apparatus Requirements.

5.1.2.1 General.

The testing apparatus used for each of the Standard methods of test specified in Chapter 5 of this Standard shall be in accordance with this section.

5.1.2.2 Testing Machine.

A testing machine shall be used which is capable of applying tensile and compressive loads at a constant rate of crosshead movement, and which is calibrated in accordance with *ASTM E4*.

5.1.2.3 Hydraulically Driven Testing Machine.

Hydraulically driven testing machines shall be controlled by a valve allowing a constant rate of flow of hydraulic fluid. Hydraulically driven testing machines controlled by a pressure valve shall not be acceptable.

5.1.2.4 Gripping Devices.

Gripping devices shall be used which are capable of carrying the test joints to failure and allowing for uniform axial loading of the Test Specimen without introducing bending in the joint.

5.1.2.5 Measuring Devices.

Measuring devices with an accuracy of 0.001 in. (0.02 mm) shall be used to measure the separation of, or slip

between, the Wood Members at the joint being tested, or the slip between the Metal Connector Plate and Wood Member.

5.2 STANDARD METHOD OF TEST FOR DETERMINING LATERAL RESISTANCE OF METAL CONNECTOR PLATE TEETH

5.2.1 Calculation Permitted For Nail-on Plates.

Lateral resistance design values for nail-on plates, meaning Metal Connector Plates without integral Teeth that are connected to the wood using only separately applied nails or other fasteners, shall be permitted to be established solely using recognized design criteria for the separately applied nails or other fasteners such as those criteria given in the *ANSI/AF&PA NDS*.

5.2.2 Metal Connector Plates.

5.2.2.1 Test Specimen Selection.

Metal Connector Plates selected for Test Specimen fabrication shall be typical of production. Test coil metal shall be sampled from the production inventories of the Metal Connector Plate manufacturers that are procured with a specified minimum yield or grade. Where such samples are found to exceed the specified minimum yield by more than 7 ksi (48.26 MPa), the lateral resistance shall be multiplied by the adjustment factor (R_Y) shown in Equation E5.2-1. Where the thickness of the test coil steel exceeds the minimum specified thickness by more than 5 percent, the lateral resistance shall be multiplied by the adjustment factor (R_T) shown in Equation E5.2-2. If both yield and thickness exceed the above specified limits, both adjustment factors R_Y and R_T shall be applied to the lateral resistance simultaneously.

$$R_Y = (F_{y,spec} / F_{y,test})^{(1.2 \times G_{test} - 0.4)} \leq 1.0 \quad (\text{E5.2-1})$$

where:

R_Y = Adjustment factor to account for steel yield

$F_{y,spec}$ = Specified minimum steel yield strength

$F_{y,test}$ = Average measured steel yield strength of test plates

G_{test} = Average measured specific gravity (oven-dry basis) of wood used in test joints

$$R_T = (t_{\text{spec}}/t_{\text{test}})^{0.7} \leq 1.0 \quad (\text{E5.2-2})$$

where:

R_T = Adjustment factor to account for steel thickness

t_{spec} = Specified minimum steel thickness

t_{test} = Average measured steel thickness of test plates

5.2.2.2 Cleaning Plates.

The Metal Connector Plates shall be washed in a solvent so that they are free of oil and any substance which will alter the Metal Connector Plate performance.

5.2.2.3 Test Specimen Design.

The Test Specimens shall be designed to produce the type of failure intended for each test. The plate dimension parallel to the loading direction, l' , shall be the maximum which consistently produces withdrawal failure of the Teeth without inducing net section steel failure. The plate dimension perpendicular to the loading direction shall comply with Section 5.2.6.

5.2.2.4 Number of Samples.

The number of plates selected shall be sufficient to fabricate five joints for each combination of plate type, plate/wood orientation, wood face width, Species Combination, and fabrication method tested.

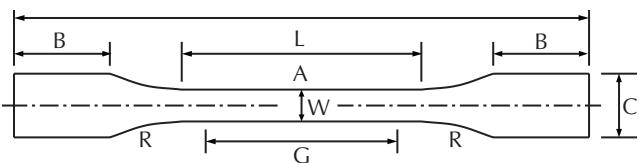
5.2.3 Solid Metal Control Specimens.

5.2.3.1 Number of Samples.

A Solid Metal Control Sample shall be taken at each end of the section of each slit coil used to manufacture the Metal Connector Plates used in Section 5.2.2. A minimum of three solid metal control specimens shall be machined from each Solid Metal Control Sample.

5.2.3.2 Control Specimen.

The solid metal control specimens shall be machined into standard rectangular Test Specimens with a reduced cross-section (see Figure 5.2-1).



For dimensions, see ASTM E8.

Figure 5.2-1 Solid Metal Control Specimen.

5.2.4 Wood Members.

5.2.4.1 General.

The Sample Block for determining oven-dry specific gravity shall be taken within 12 in. (305 mm) of where the Metal Connector Plates will be embedded. Each Sample Block shall be stored in an individual impermeable package to prevent any change in moisture content before weighing. The Wood Member shall be used for assembly at moisture content of 11 percent or greater for solid-sawn lumber, and 7 percent or greater for Structural Composite Lumber. The Sample Block for determining moisture content shall be taken at the time of Test Specimen fabrication, within 12 in. (305 mm) from where the Metal Connector Plates will be embedded.

5.2.4.2 Test Specimen Characteristics.

The specific gravity, moisture content, and moisture content adjustments of the Wood Members shall be determined by *ASTM D2395* for specific gravity, and *ASTM D4442* or *ASTM D4444* for moisture content testing, except that the parallel to load member in joints at the AE and EE orientations shall not be required to have these properties determined.

5.2.4.3 Clear Wood Under Plates.

Wood Members shall have clear wood in the area in which the Metal Connector Plates will be embedded.

5.2.4.4 Test Specimen Length.

The length of the Wood Members shall be determined according to the type of gripping apparatus used. In no case shall the gripping apparatus interfere with the connection at the joint or the measuring device. The width of the Wood Members shall comply with Section 5.2.6.2.

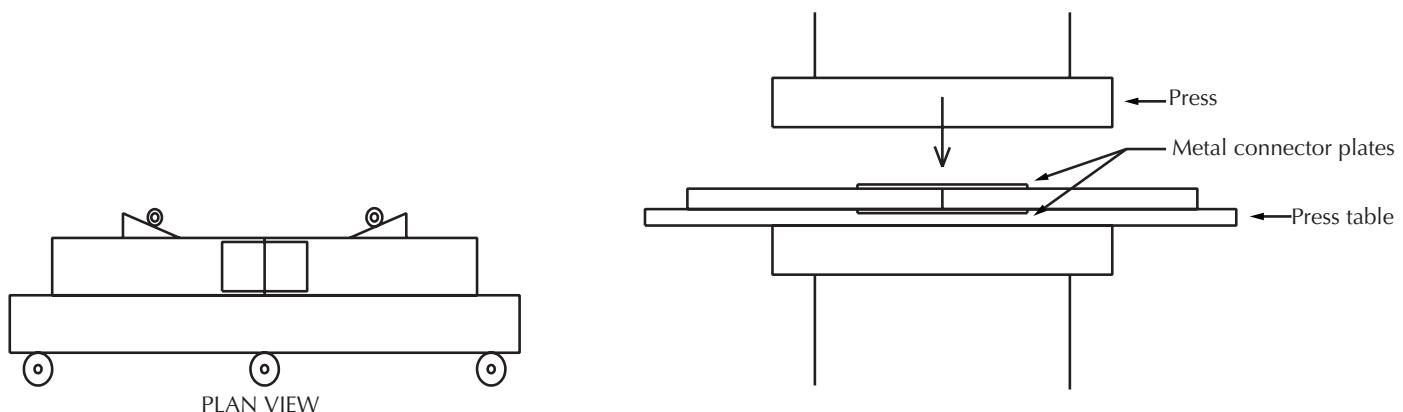
5.2.4.5 Number of Samples.

The number of Wood Members selected shall be sufficient to fabricate five joints for each combination of plate type, plate/wood orientation, wood face width, Species Combination, and fabrication method tested.

5.2.5 Embedment Methods.

5.2.5.1 Design Values.

Design values intended for Metal Connector Plates pressed hydraulically shall be obtained by testing hydraulically embedded Test Specimens as shown in Figure 5.2-2. Design values intended for Metal Connector Plates pressed with a single pass roller shall be obtained by testing a Metal Connector Plates Test Specimen embedded with a single pass roller press as shown in Figure A5.2-3. Design values determined for a specific roller diameter shall be applicable to Metal Connector Plates



STEP 1. Jig pieces on Press table and position Metal Connector Plate. Clamps and straight edge assure wood members in joint are colinear.

STEP 2. Press Metal Connector Plate.

Figure 5.2-2 Suggested Joint Fabrication Setup.

pressed with the same diameter roller or greater. Design values determined for single pass roller presses are not prohibited from being used for double pass roller presses and hydraulic pressing equipment.

5.2.5.2 Reduction Value.

In lieu of testing Metal Connector Plates for use with a single pass, full embedment roller press as specified in Section 5.2.5.1, a reduction value, Q_R , determined in accordance with Annex A5.2, shall be permitted. This reduction does not apply to plates embedded using full embedment hydraulic platen presses, multiple roller systems which utilize partial embedment followed by full embedment rollers, and combinations of partial embedment roller/hydraulic presses that feed Trusses into a stationary finish roller.

5.2.6 Test Specimen Fabrication.

5.2.6.1 Plate Thickness.

The Metal Connector Plate thickness shall be measured to the nearest 0.001 in. (0.02 mm) before the Test Specimen is assembled.

5.2.6.2 Assembly.

The Test Specimens shall be assembled as shown in Figure 5.2-3 for the Gross Area Method and Figure 5.2-4 for the Net Area Method.

5.2.6.2.1 Gross Area Method.

End and Edge Distances shall be zero for the Gross Area Method. For AA and EA orientations, the actual width of the Wood Members shall be reduced to the Metal Connector Plate dimension perpendicular to the grain before embedment.

5.2.6.2.2 Net Area Method.

For AA and EA orientations, the Metal Connector Plate dimension perpendicular to the grain shall be no more than $\frac{1}{2}$ in. (12.7 mm) less than the Wood Member width.

Metal Connector Plate Teeth at the wood edges and at the member interface within the applicable End or Edge Distance shall be ground off as shown in Figure 5.2-4. Edge Distance shall be $\frac{1}{4}$ in. (6 mm) measured perpendicular to wood grain. End Distance shall be $\frac{1}{2}$ in. (13 mm) measured parallel to wood grain and applies to joints loaded parallel to grain (AA and EA orientations). Alternatives to these $\frac{1}{4}$ in. (6mm) and $\frac{1}{2}$ in. (13 mm) standard values shall be permitted provided the alternative values are used in the design process.

5.2.6.3 Full Contact of Wood Members.

The Test Specimens shall be assembled such that the two Wood Members are held tightly together, in full contact against each other, before the Metal Connector Plates are attached and during the embedment of the plates.

5.2.6.4 Joint Fabrication Setup.

The Metal Connector Plates shall be embedded on both sides of the Test Specimens in the same manner as typically used in the manufacture of Trusses. Any presetting techniques used in embedding the plates, such as tapping plates in place prior to roller pressing, shall be typical of those used in the manufacture of Trusses and fully described in the test report. A suggested joint fabrication setup is shown in Figure 5.2-2 for hydraulically pressed manufacturing methods.

In typical Truss Manufacture applications where Keeper Nails, or any supplemental fasteners, are not an integral part of the joint design method, they shall not be used in

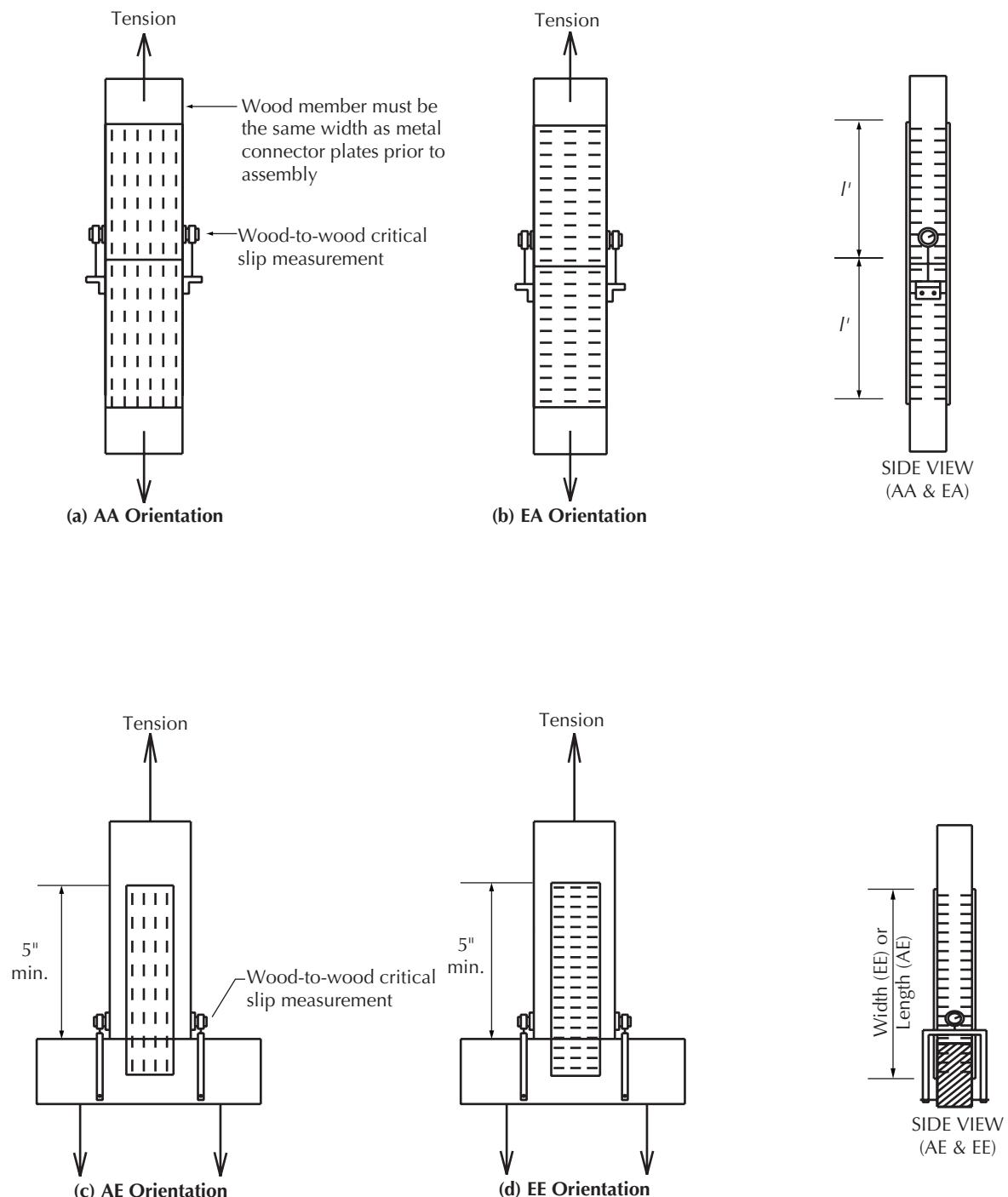


Figure 5.2-3 Test Specimen Fabrication for Gross Area Method.

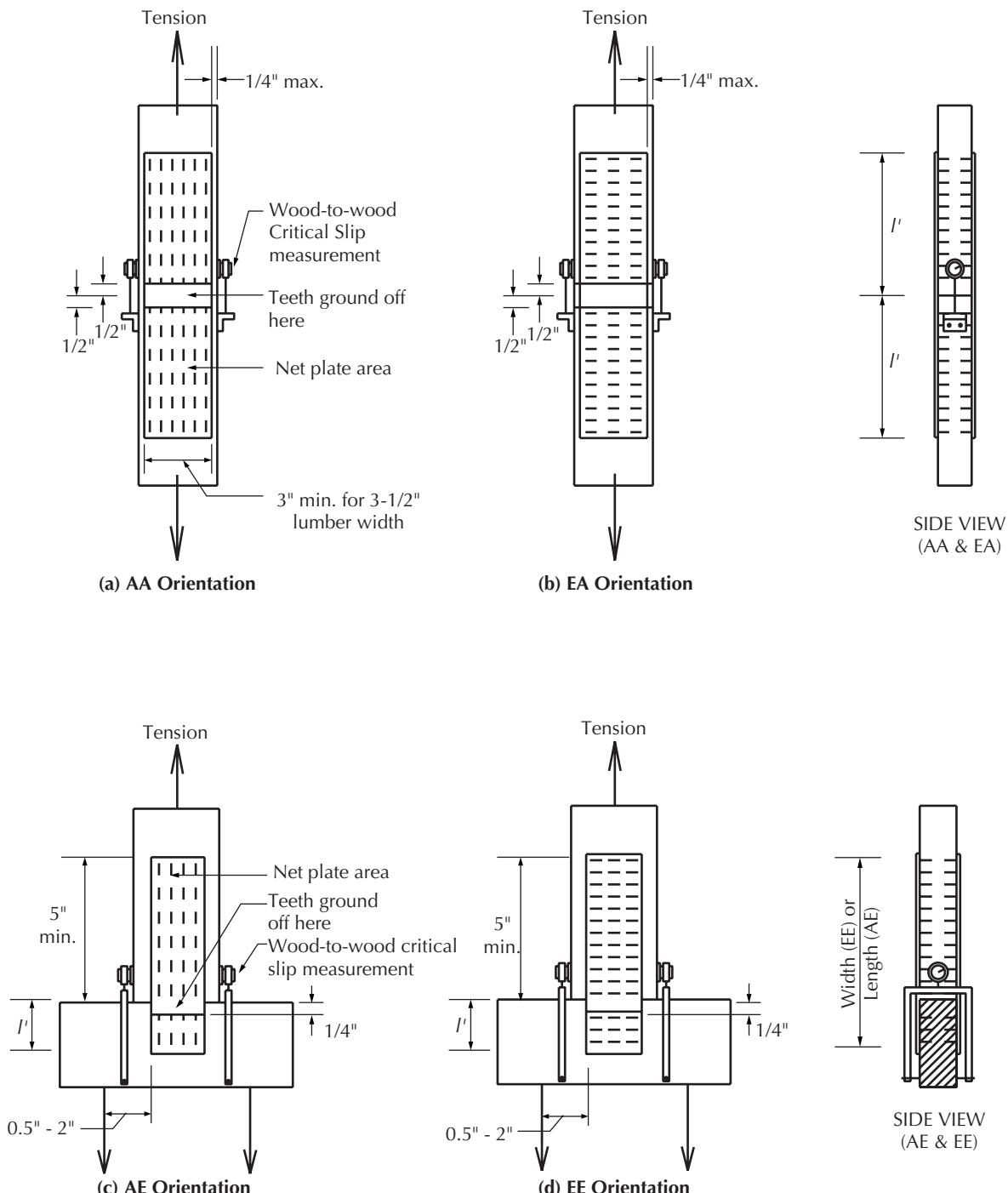


Figure 5.2-4 Test Specimen Fabrication for Net Area Method.

the Metal Connector Plates lateral resistance evaluation testing. Where Keeper Nails are an integral part of the joint design method, are used in the manufacturing process, and are intended to be used in production Trusses, they shall be installed in the tooth holding in the same proportions and with the same distribution as those intended to be used in production.

5.2.6.5 Plate Embedment.

Metal Connector Plates shall be embedded in clear wood, and shall be installed so that the Teeth are fully embedded in the Wood Member and no gaps remain between the Metal Connector Plate and the Wood Member. Overpressing shall be avoided, so that the Metal Connector Plates do not embed into the Wood Member more than half the steel thickness.

5.2.6.6 Wood-to-Plate Slip.

Test Specimens assembled for evaluating Metal Connector Plates perpendicular to the grain of the Wood Member shall be fabricated by extending the Metal Connector Plate a minimum of 5 in. on the vertical Wood Member. To obtain the wood-to-plate slip, either measure movement wood-to-wood, or measure movement wood-to-plate as shown in Figure 5.2-5. When slip is measured wood-to-wood, the plate is permitted to be glued to the vertical Wood Member to minimize plate slip on this member.

5.2.6.7 Structural Composite Lumber.

When Metal Connector Plates are tested with Structural Composite Lumber (SCL), the test plates shall be embedded into the same surface of the SCL as anticipated in service. If plates will be applied in service to both the pressed face and the non-pressed face of the SCL, sepa-

rate series of tests shall be required for plates on each of these surfaces.

5.2.7 Test Specimens Required.

5.2.7.1 Wide Face Lateral Resistance Values.

5.2.7.1.1 Replicates and Orientations.

For each Species Combination and embedment method selected for testing, a minimum of five Test Specimens shall be tested for each of the following connector plate/wood orientations:

- (a) Load parallel to grain, Metal Connector Plate Length parallel to load (AA orientation, Figures 5.2-3(a) and 5.2-4(a)).
- (b) Load parallel to grain, Metal Connector Plate Length perpendicular to load [EA orientation, see Figures 5.2-3(b) and 5.2-4(b)].
- (c) Load perpendicular to grain, Metal Connector Plate Length parallel to load [AE orientation, see Figures 5.2-3(c) and 5.2-4(c)].
- (d) Load perpendicular to grain, Metal Connector Plate Length perpendicular to load [EE orientation, see Figures 5.2-3(d) and 5.2-4(d)].

5.2.7.1.2 Symmetrical Plate Teeth.

When Metal Connector Plate Teeth, or groups of Teeth, are symmetrical across both the slot (or plug) length and slot (or plug) width, the EA and EE orientations need not be tested and shall be taken as equal to the AA and AE orientations, respectively.

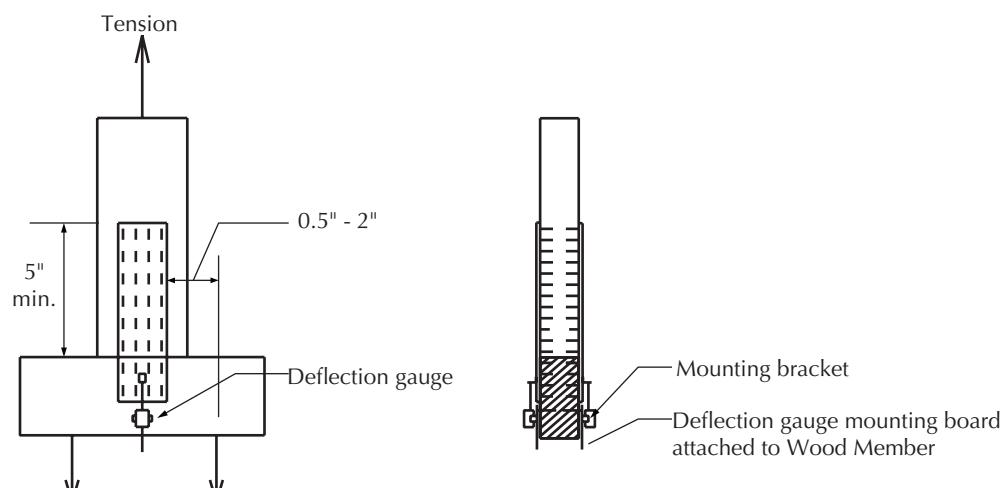


Figure 5.2-5 Fabrication of Test Specimen for Measuring Wood-To-Plate Slip.

5.2.7.2 Narrow Face Lateral Resistance Values.

5.2.7.2.1 Gross Area Tests.

Narrow Face lateral resistance design values for Metal Connector Plates shall be obtained for plates tested by the Gross Area Method by testing a minimum of five Test Specimens for a Species Combination for each orientation specified in Section 5.2.7.1.

5.2.7.2.2 Alternative.

In lieu of testing per Section 5.2.7.2.1, Narrow Face lateral resistance design values for Metal Connector Plates using the Gross Area Method shall be determined by reducing design values determined for Metal Connector Plates pressed in the wide face of the Wood Member, as determined in Section 5.2.7.1, by 15 percent.

5.2.8 Test Procedure to Determine Lateral Strength Capacities.

5.2.8.1 Measurements.

After assembly, measure the Metal Connector Plate Length and Width to the nearest 0.03 in. (0.10 mm). Count the number of Teeth on each side of the joint.

5.2.8.2 Time Period.

A minimum period of seven days shall elapse between assembly and testing of the Test Specimens.

5.2.8.3 Loading Procedure.

Conduct tests on the Metal Connector Plate Test Specimens at a constant movable crosshead speed to attain ultimate load in not less than one minute. Record the rate of loading used. Take readings of both the applied load and the amount of corresponding slip indicated by each measuring device at intervals not exceeding 400 lbs. (1780 N) to permit plotting of an accurate load-deformation curve. Obtain at least three readings before Critical Slip is reached. Continue the test until the ultimate failure load is reached. Load at Critical Slip shall be determined by linear interpolation between points in the load-deformation curve.

5.2.8.4 Testing Procedure for Solid Metal Control Specimens.

Conduct tests on the solid metal Control Specimens in accordance with *ASTM E8* procedures. Thickness shall be measured to the nearest 0.0001 in. (0.003 mm) and width to the nearest 0.001 in. (0.02 mm). Thickness of galvanized (or other) coating, if present, shall be measured or the coating shall be removed prior to thickness measurement. In lieu of coating measurement, the thicknesses given in Section 6.3.4.1.3 shall be permitted to be used for coating thickness.

5.2.9 Calculations.

5.2.9.1 General.

The calculations specified herein shall establish the basic allowable lateral resistance design values, for the four specified orientations, on a load per unit area basis: V_{LRAA} , V_{LRAE} , V_{LREA} , and V_{LREE} [psi/pair (N/mm²/pair)].

5.2.9.2 Lateral Resistance Design Values.

Design values for lateral resistance, V_{LR} , shall be the lesser of the following values, which shall be further adjusted in accordance with Section 5.2.2.1 when applicable, and expressed on a unit plate area basis:

- (a) At Critical Slip, divide the load by 1.3 for each Test Specimen. Average the test values for each Metal Connector Plate orientation for all Test Specimens and multiply the resulting average value by R_G , given in Section 5.2.9.3, and R_Y and R_T given in Section 5.2.2.1.
- (b) At ultimate failure, divide the load by 3.2 for each Test Specimen. Average the test values for each Metal Connector Plate orientation for all Test Specimens and multiply the resulting average value by R_G , given in Section 5.2.9.3, and R_Y and R_T given in Section 5.2.2.1.

5.2.9.3 Adjustment Factor for Specific Gravity of Test Specimens Exceeding Specified Value (R_G).

The value of R_G shall be the lesser of $G_{specified}/G_{test}$ or 1.0; where $G_{specified}$ is the published average specific gravity of the Species Combination listed in an approved design standard or *ASTM D2555*, and G_{test} is the average specific gravity of the wood in the five or more Test Specimens producing the loads used in the calculations shown in Section 5.2.9. Both $G_{specified}$ and G_{test} shall be based on volume at oven-dry moisture content. $G_{specified}$ shall be for the species to which the design values will apply. This species is permitted to be different from the species tested.

5.2.10 Report.

The report shall include the following information:

- (a) Date of fabrication, date of test, and date of report.
- (b) Test sponsor and test agency.
- (c) Complete description of test method and loading procedure used if there are any deviations from the prescriptive methods in this Standard.

- (d) Description of pressing equipment including roller diameter, roller press description, jigging apparatus, and any plate setting or pre-pressing techniques, if used.
- (e) The number of Teeth in the failure zone.
- (f) Rate of testing (crosshead speed or initial rate of load application).
- (g) Elapsed time of test.
- (h) Load deformation curve, or a minimum of three load, and corresponding deformation, readings prior to achieving Critical Slip.
- (i) Load at Critical Slip.
- (j) Maximum load obtained before failure and maximum load per plate unit values.
- (k) Description of type and path of failure.
- (l) Wood Member sizes and species.
- (m) Either a detailed drawing of the Metal Connector Plate showing type, model, size, thickness, material and manufacturer, or a written description of the plate noting size, thickness, tooth spacing, material and manufacturer along with a photograph showing both faces.
- (n) Moisture content of Wood Members at time of fabrication.
- (o) Oven-dry specific gravity of Wood Members.
- (p) The number of Test Specimens tested.
- (q) Certification of calibration of the testing machine.
- (r) Mill certification data for the test steel coil heat number, or the results of *ASTM E8* tests of solid metal control specimens.

ANNEX A5.2: DESIGN VALUE ADJUSTMENTS FOR SINGLE PASS ROLLER PRESSES (MANDATORY INFORMATION)

A5.2.1 Scope.

The test procedures contained in Annex A5.2 provide a basis to assign Metal Connector Plate design values to joints assembled with roller presses by adjustment of

design values determined with joints assembled with hydraulic presses.

A5.2.2 Sampling.

A5.2.2.1 Number of Samples.

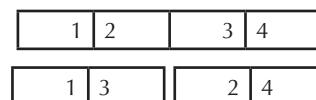
Five matched specimen pairs shall be tested for each plate type, property (wood/plate orientation), roller type and diameter, and Species Combination evaluated.

A5.2.2.2 Matched Test Specimen Pairs.

Each specimen pair shall consist of one joint with plates embedded with a hydraulic press and one matching joint with plates embedded with a roller press. Each 2x4 (38 x 89 mm) Wood Member selected for matched specimen sampling shall be coded (identified) as to member number, and whether it is used with roller pressed plates or hydraulically pressed plates.

A5.2.2.3 Materials.

All materials shall be matched between joints within each specimen pair. Plates used within a specimen pair shall be typical of production, shall be of identical sizes produced from the same solid steel coil or sheet stock, and shall comply with Section 5.2.2. Lumber within a specimen pair shall be cross-matched between joints in accordance with Figure A5.2-1 and shall satisfy the requirements of Section 5.2.4.



Cut Test Specimen in 4 pieces, sequentially number 1-4, and rejoin as shown for the test: 1-3 & 2-4.

Figure A5.2-1

A5.2.3 Test Specimens Required.

A5.2.3.1 Property Evaluated.

Tests shall be performed for lateral withdrawal resistance for V_{LRAA} (load parallel to grain, Metal Connector Plate Length parallel to load) (see Figure A5.2-2). Additional plate/wood orientations are not prohibited from being tested.

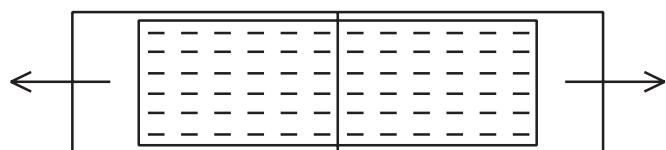


Figure A5.2-2 Load parallel to grain, length parallel to load (VLRAA).

A5.2.3.2 Species.

If results will be used to adjust hydraulic pressed test values determined with Species Combinations having

published average oven-dry specific gravities that range both at or above 0.50, and at or below 0.49, then tests shall be conducted with two Species Combinations: one species with a published average oven-dry specific gravity of 0.50 or greater, and one species with a published average oven-dry specific gravity of 0.49 or less. If results will be used to adjust hydraulic pressed test values determined with Species Combinations having published average oven-dry specific gravities that range only at or above 0.50, or only at or below 0.49, then tests shall be conducted with one Species Combination within the relevant specific gravity range. Published average specific gravities are from *ANSI/AF&PA NDS*. Additional Species Combinations are not prohibited from being tested.

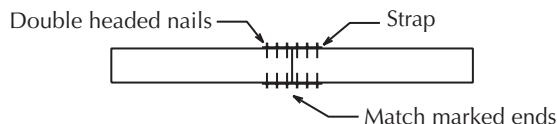
A5.2.3.3 Plate Type.

Each plate type, for which roller pressed design values are to be determined, using a Q_R ratio, shall be tested in accordance with Annex A5.2.

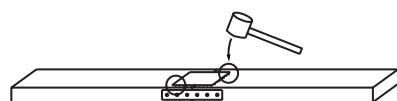
A5.2.3.4 Roller Presses.

Roller embedment equipment used for fabricating test joints shall be typical of equipment used in manufacturing Trusses. The smallest roller diameter used in manufacturing Trusses shall be used in fabricating test joints. Additional roller diameters are not prohibited from being tested.

STEP 1. Attach 1 x 5 inch temporary restraining straps with 6 double headed nails to each side of the Wood Members at the joint to hold the Wood Members together and straight with matching marked ends.



STEP 2. Set the Metal Connector Plate on the first face by striking opposite corners of the Metal Connector Plate once each with a hammer, turn the Test Specimen over and set the Metal Connector Plate on the opposite face in the same manner.



Note: This is one suggested method for fabricating roller pressed test specimens. Alternative methods, which will ensure that the Wood Members butt tightly against each other, and remain in line, are not prohibited from being used.

A5.2.4 Test Specimen Fabrication.

A5.2.4.1 General.

Test Specimen fabrication shall satisfy the requirements of Section 5.2.6. Any jigging required to maintain full wood contact across the joint shall be fully described in the test report. Any presetting techniques used to affix plates to the wood prior to roller pressing shall be typical of those used in the manufacture of Trusses and fully described in the test report.

A5.2.4.2 Roller Test Specimens.

All roller Test Specimens shall pass through the roller press with the length of the Test Specimen perpendicular to the length of the rollers (see Figure A5.2-3).

A5.2.5 Test Procedure.

A5.2.5.1 General.

Test procedures shall comply with the requirements of Section 5.2.8, except that tests of solid metal control specimens shall not be required.

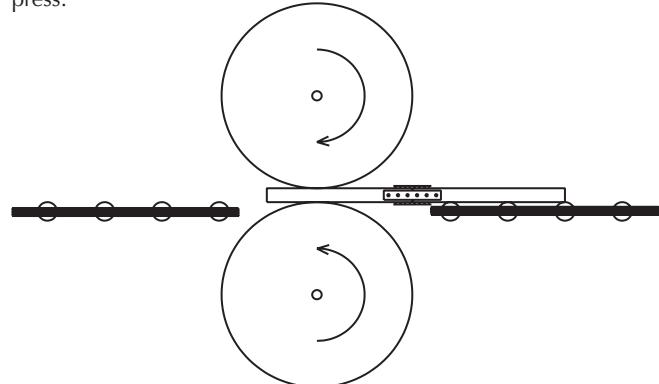
A5.2.5.2 Test Specimen Pairs.

Both joints within a matched pair shall be tested at the identical number of days from their respective dates of fabrication.

A5.2.5.3 Tooth Withdrawal Failure.

All hydraulic pressed specimens shall fail by tooth withdrawal.

STEP 3. Feed the assembled test specimens through the roller press.



STEP 4. Carefully remove the temporary restraining straps.

Figure A5.2-3 Roller Plating.

A5.2.6 Calculations.

A5.2.6.1 Strength Adjustment Ratios, (Q_R).

Strength adjustment ratios for roller presses (Q_R), per Section 5.2.5.2, shall be determined at Critical Slip levels and at ultimate levels, for each combination of plate type, roller press type and diameter, plate/wood orientation, and Species Combination selected for matched specimen testing, as follows:

- (a) $Q_{R@Slip}$ shall be the average V_{LR} based on Critical Slip of the five roller pressed joints divided by the average V_{LR} based on Critical Slip of the five matched hydraulically pressed joints. $Q_{R@Slip}$ shall be less than or equal to 1.0.
- (b) $Q_{R@ultimate}$ shall be the average V_{LR} based on ultimate load of the five roller pressed joints divided by the average V_{LR} based on ultimate load of the five hydraulically pressed joints. $Q_{R@ultimate}$ shall be less than or equal to 1.0.

A5.2.6.2 Plate Design Values.

Basic plate design values for plates embedded by roller presses shall be limited to the lower of the following two quantities:

$$(Q_{R@Slip})(V_{LRXX@Slip}) \quad (\text{EA5.2-1})$$

$$(Q_{R@ultimate})(V_{LRXX@ultimate}) \quad (\text{EA5.2-2})$$

where the V_{LRXX} terms above shall be from hydraulic pressed joint tests in accordance with Section 5.2.8 for the applicable Species Combination and plate/wood orientations to which the design values will be applied.

A5.2.6.3 Q_R Application.

Where matched pair testing is performed at only the AA plate/wood orientation, resulting Q_R values shall be applied to all plate/wood orientations. Where only two Species Combinations of lumber in accordance with Section A5.2.3.2 are tested with matched pairs, the Q_R value resulting from the lower specific gravity Species Combination shall apply to all Species Combinations with average published specific gravities of 0.49 or lower, and the Q_R value resulting from the higher specific gravity Species Combinations shall apply to all Species Combinations with average published specific gravities of 0.50 or higher. Published average specific gravities are from ANSI/AF&PA NDS.

A5.2.7 Report.

The report on tests conducted in accordance with Annex A5.2 shall comply with Section 5.2.10, with the exception that the items in Section 5.2.10(r) shall not be required.

5.3 STANDARD METHOD OF TEST FOR STRENGTH PROPERTIES OF METAL CONNECTOR PLATES UNDER SHEAR FORCE

5.3.1 Scope.

Nail-on plates, meaning Metal Connector Plates without integral Teeth that are connected to the wood using only separately applied nails or other fasteners, shall be tested in accordance with this section using the same density of fasteners and presence (or lack thereof) of fasteners in Edge Distance zones as used in application of such plates on Trusses.

5.3.2 Metal Connector Plates.

5.3.2.1 Test Specimen Selection.

Metal Connector Plates selected for Test Specimen fabrication shall be typical of production, and shall be manufactured in accordance with materials specified by the Metal Connector Plate manufacturer.

5.3.2.2 Test Coil Steel.

The mechanical properties (yield, tensile, and elongation) of the test coil steel shall meet the requirements for the specified grade of steel for plate manufacture.

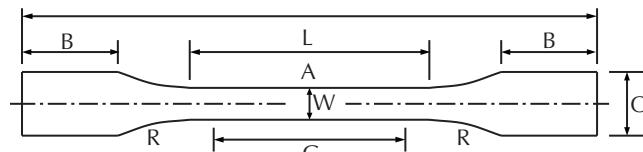
5.3.3 Solid Metal Control Specimens.

5.3.3.1 Number of Samples.

A Solid Metal Control Sample shall be taken at each end of the section of each slit coil used to manufacture the Metal Connector Plates used in Section 5.3.4. A minimum of three solid metal control specimens shall be machined from each Solid Metal Control Sample.

5.3.3.2 Control Specimen.

The solid metal control specimens shall be machined into standard rectangular Test Specimens with a reduced cross-section (see Figure 5.3-1).



For dimensions, see ASTM E8.

Figure 5.3-1 Solid Metal Control Specimen.

5.3.4 Test Specimen Fabrication.

5.3.4.1 General.

The Test Specimens shall be assembled with three Wood Members and four equal size Metal Connector Plates, one on each side of each joint interface, with the Metal Connector Plate Length (L_p) inclined at an angle, α , to the wood shear plane.

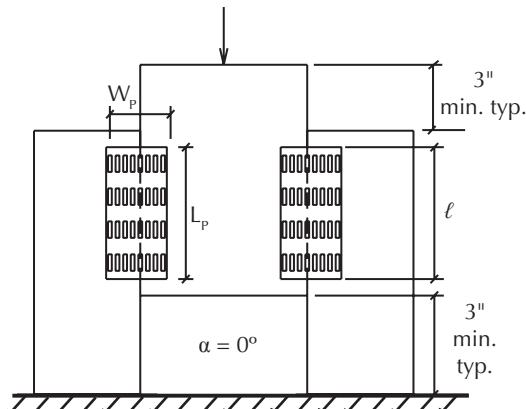


Figure 5.3-2 Zero Degree Orientation.

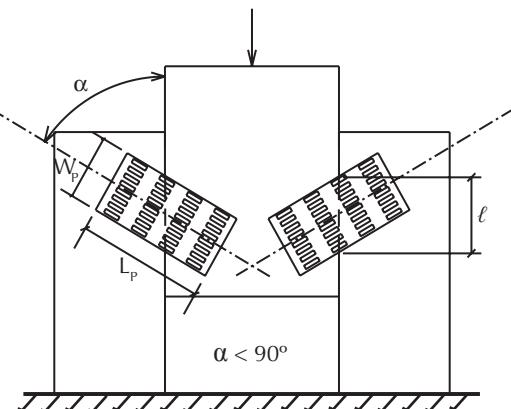


Figure 5.3-3 Acute Angle Orientation.

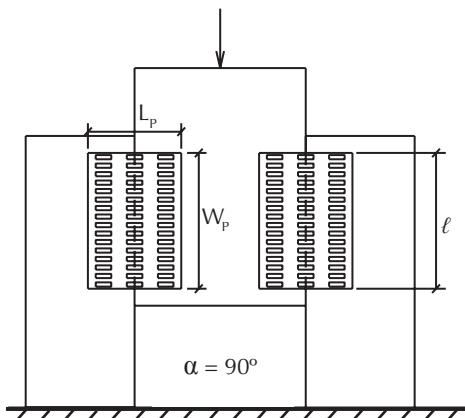


Figure 5.3-4 Ninety Degree Orientation.

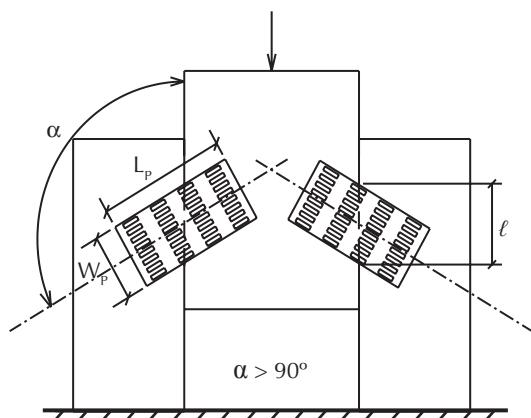


Figure 5.3-5 Obtuse Angle Orientation.

A zero degree angle is defined when the Metal Connector Plate Length (L_p) is parallel to the wood joint (see Figure 5.3-2). Values for α greater than zero are defined when the Metal Connector Plate Length is rotated counter-clockwise from the vertical (zero) position (see Figures 5.3-3 through 5.3-5).

5.3.4.2 General Orientation.

The three Wood Members shall be placed so that the joint and lumber grain are in the same direction. The ends of the center Wood Member shall be 3 in. (76 mm) minimum above the corresponding ends of the outer Wood Members (see Figure 5.3-2), or as otherwise required to prevent any contact between the testing machine and the base of the center member or between the testing machine and the top of the exterior members.

5.3.4.3 Less Than 90 Degree Orientation.

For orientations with α less than 90 degrees, the centroid

of the Metal Connector Plate contact area on the outer Wood Members shall be above the centroid of the Metal Connector Plate contact area on the center Wood Member to ensure tension shear.

5.3.4.4 Greater Than 90 Degree Orientation.

For orientations with α greater than 90 degrees, the centroid of the Metal Connector Plate contact area on the outer Wood Members shall be below the centroid of the Metal Connector Plate contact area on the center Wood Member to ensure compression shear.

5.3.4.5 Metal Connector Plate Length.

The Metal Connector Plates shall be of sufficient length to induce failure in the plate steel, rather than failure by tooth withdrawal. Where necessary, it shall be permitted to clamp the Metal Connector Plates, or otherwise firmly fasten them, a minimum of 2 in. (50 mm) from the joint to prevent withdrawal.

5.3.4.6 Plate Embedment.

The Metal Connector Plates shall be firmly embedded, without removal of any Teeth, and shall be positioned with the minimum net section of the Metal Connector Plate directly over the shear plane.

5.3.4.7 Embedment Procedure.

The embedment procedure shall be consistent with the method for embedding the Metal Connector Plates in the manufacturing process (i.e., by pressing or rolling). Over-pressing shall be avoided, so that the Metal Connector Plates do not embed into the Wood Member more than half the steel thickness.

5.3.4.8 Joints.

The joints in the assembly shall be a close fit but not compressed.

5.3.5 Test Specimens Required.

A minimum of three Test Specimens shall be tested at each of six specific Metal Connector Plate orientations, α : 0°, 30°, 60°, 90°, 120°, and 150°.

5.3.6 Test Procedure.

5.3.6.1 General Measurements.

Before testing, measure all Metal Connector Plates to determine their gross width (W_p) and length (L_p) to the nearest 0.01 in. (0.3 mm), and minimum thickness (t) to the nearest 0.0001 in. (0.002 mm). Take measurements at a minimum of three different locations on each Metal Connector Plate, using the average of three readings for the record.

5.3.6.2 Angles.

For Metal Connector Plates to be tested at any orientation, angle α shall be accurately measured for use in calculating the Metal Connector Plate Length along the shear line, ℓ . When the angle of placement equals 0° and 90°, ℓ equals L_p and W_p , respectively (see Figures 5.3-2 and 5.3-4).

5.3.6.3 Set-Up.

Conduct tests on the Metal Connector Plate Test Specimens by placing the Test Specimen between and perpendicular to the testing machine platens and loading the Test Specimen in compression as shown in Figures 5.3-2 through 5.3-5. Apply the load concentrically throughout the tests at a uniform rate of movement of the platens of the testing machine so that the maximum load is reached in not less than 60 seconds.

5.3.6.4 Testing Procedure for Solid Metal Control Specimens.

Conduct tests on the solid metal control specimens in accordance with *ASTM E8* procedures. Thickness shall be measured to the nearest 0.0001 in. (0.003 mm) and width to the nearest 0.001 in. (0.02 mm). Thickness of galvanized (or other) coating, if present, shall be measured or the coating shall be removed prior to thickness measurement. In lieu of coating measurement, the thicknesses given in Section 6.3.4.1.3 shall be permitted to be used for coating thickness.

5.3.6.5 Maximum Loads.

For the Metal Connector Plates and solid metal control specimens, observe the maximum loads in pounds-force (or Newton).

5.3.7 Calculations.

5.3.7.1 Ultimate Tensile Strength - Control Specimen.

Calculate the ultimate tensile strength of the solid metal control specimen (F_{tc}) by dividing the maximum tensile loads of each solid metal control specimen (P_{tc}) by the cross-sectional area of the respective solid metal control specimen (A_{gc}):

$$F_{tc} = \frac{P_{tc}}{A_{gc}} \quad (\text{E5.3-1})$$

The cross-sectional area of each solid metal control specimen is determined by multiplying the minimum thickness (t_{net}) of the solid metal control specimen by the width of the solid metal control specimen (W):

$$A_{gc} = t_{net} W \quad (\text{E5.3-2})$$

The F_{tc} values for all six, or more, solid metal control specimens from an individual coil of steel shall be averaged together, and the average value shall be used in Section 5.3.7.2.

5.3.7.2 Theoretical Ultimate Shear Stress - Control Specimen.

Determine the theoretical ultimate shear stress of the solid metal control specimen (F_{sc}) by multiplying the average ultimate tensile stress (F_{tc}) by 0.577:

$$F_{sc} = 0.577 F_{tc} \quad (\text{E5.3-3})$$

5.3.7.3 Ultimate Shear Strength - Test Specimen.

For each Test Specimen, calculate the Metal Connector Plate ultimate shear strength (F_{sp}) by dividing one-fourth of the maximum shear load carried by the Test Specimen

(P_{sp}) by the average gross cross-sectional area (A_{gp}) of all four plates on the Test Specimen:

$$F_{sp} = \frac{P_{sp}}{4A_{gp}} \quad (\text{E5.3-4})$$

The gross cross-sectional area of each Metal Connector Plate (A_{gp}) is obtained by multiplying the minimum thickness of the Metal Connector Plate (t_{net}) by the calculated shear length of the Metal Connector Plate, ℓ :

$$A_{gp} = t_{net}(\ell) \quad (\text{E5.3-5})$$

The three, or more, F_{sp} values calculated for each orientation shall be averaged together, and the average value shall be used in Section 5.3.7.4.

5.3.7.4 Shear Strength Effectiveness Ratio.

Calculate the shear Effectiveness Ratio (R_s), for each orientation of the Metal Connector Plate, by dividing the average Metal Connector Plate ultimate shear stress (F_{sp}) for each orientation by the theoretical ultimate shear stress of the matched solid metal control specimen (F_{sc}):

$$R_s = \left(\frac{F_{sp}}{F_{sc}} \right) = \frac{F_{sp}}{0.577 \times F_{tc}} \quad (\text{E5.3-6})$$

5.3.8 Report.

The report shall include the following information:

- (a) Date of test and date of report.
- (b) Test sponsor and test agency.
- (c) Identification of Metal Connector Plates: manufacturer, model, type, material, finish, shape, dimensions, and other pertinent information. Metal Connector Plate material specifications shall include allowable tensile stress (F_{st}) and allowable shear stress (F_{vs}).
- (d) Complete description of test method and loading procedures used, if there are any deviations from the prescribed methods in this Standard.
- (e) Number of Test Specimens tested.
- (f) Rate of testing (crosshead speed or initial rate of load application).
- (g) Elapsed time of test.
- (h) All test data, including extrema and averages.

- (i) Shear Effectiveness Ratio for each individual Test Specimen and averages for all identical Test Specimens.
- (j) Description of type and path of failure.
- (k) Summary of findings.
- (l) Certification of calibration of the testing machine.
- (m) Results of the solid metal control specimen tests conducted per *ASTM E8*.

5.4 STANDARD METHOD OF TEST FOR STRENGTH PROPERTIES OF METAL CONNECTOR PLATES UNDER TENSION FORCES

5.4.1 Scope.

Nail-on plates, meaning Metal Connector Plates without integral Teeth that are connected to the wood using only separately applied nails or other fasteners, shall be permitted to have tensile efficiency determined solely by geometric efficiency (see Section 5.4.1.1) if the nail holes, and any other perforations in the Metal Connector Plate, have smooth, rounded edges.

5.4.1.1 Geometric Efficiency.

Geometric efficiency is the ratio of the net steel width at a cross-section to the gross steel width of the cross-section. Net steel width shall be the gross steel width minus the width removed by any holes in the cross-section.

5.4.1.2 Chain of Holes Extending in Diagonal or Zigzag Line.

For a chain of holes extending in a diagonal or zigzag line, the net width of the cross-section shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions of all holes in the chain and adding, for each space between holes in the chain, the quantity: $s^2/4g$.

where:

s = longitudinal center-to-center spacing or pitch of any two consecutive holes

g = transverse center-to-center spacing or gauge of any two consecutive holes

5.4.2 Metal Connector Plates.

5.4.2.1 Test Specimen Selection.

Metal Connector Plates selected for Test Specimen fabrication shall be typical of production and shall be

manufactured in accordance with materials specified by the Metal Connector Plate manufacturer.

5.4.2.2 Test Coil Steel.

The mechanical properties (yield, tensile, and elongation) of the test coil steel shall meet the requirements for the specified grade of steel for plate manufacture.

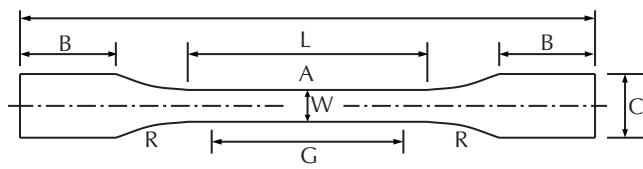
5.4.3 Solid Metal Control Specimens.

5.4.3.1 Number of Samples.

A Solid Metal Control Sample shall be taken at each end of the section of each slit coil used to manufacture the Metal Connector Plates used in Section 5.4.4. A minimum of three solid metal control specimens shall be machined from each Solid Metal Control Sample.

5.4.3.2 Control Specimens.

The solid metal control specimens shall be machined into standard rectangular Test Specimens with a reduced cross-section (see Figure 5.4-1).



For dimensions, see ASTM E8.

Figure 5.4-1 Solid Metal Control Specimen.

5.4.4 Test Specimen Fabrication.

5.4.4.1 Minimum Thickness.

The Metal Connector Plate minimum thickness (t) shall be measured to the nearest 0.0001 in. (0.003 mm) before the Test Specimen is assembled.

5.4.4.2 Test Specimen Assembly.

The Test Specimens shall be assembled as shown in Figures 5.4-2 and 5.4-3. The Metal Connector Plates shall be firmly embedded on both sides of the Test Specimen in the same manner as typically used in the manufacture of Trusses, and shall be positioned with the minimum net section of the Metal Connector Plate directly over the joint, even if this requires that the plate be positioned unsymmetrically.

5.4.4.3 Metal Connector Plate Length.

The Metal Connector Plates shall be of sufficient length to induce a tensile or tearing failure of the net section steel, rather than lateral withdrawal failure in the Teeth. The Metal Connector Plates shall be permitted to be clamped a minimum of 2 in. (50 mm) from the joint to prevent lateral withdrawal of the Teeth, provided such clamping

or fastening does not affect the tensile resistance of the Metal Connector Plate.

5.4.5 Test Specimens Required.

5.4.5.1 Across Plate Width.

For tests across the Metal Connector Plate Width, a minimum of three Test Specimens shall be assembled. The Metal Connector Plate shall be embedded in the Wood Members such that the Metal Connector Plate Width is perpendicular to the grain of the Wood Members (see Figure 5.4-2).

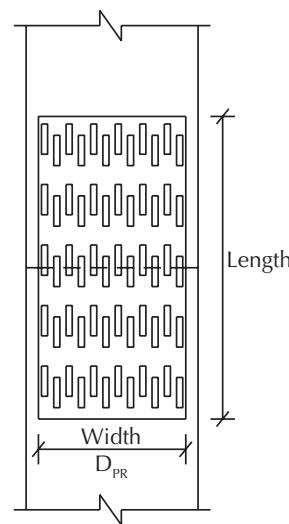


Figure 5.4-2 Metal Connector Plate Width Perpendicular to Grain of Wood Members.

5.4.5.2 Across Plate Length.

For tests across the Metal Connector Plate Length, a minimum of three Test Specimens of a single Metal Connector Plate Length shall be assembled. The Metal Connector Plate shall be embedded in the Wood Members such that the Metal Connector Plate Length is perpendicular to the grain of the Wood Member (see Figure 5.4-3).

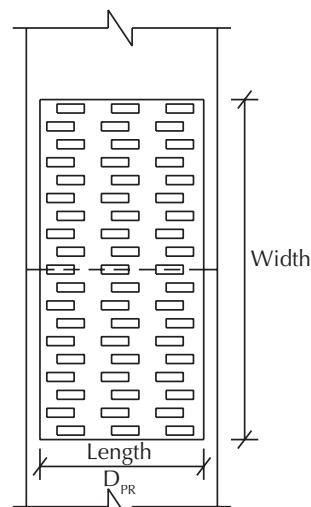


Figure 5.4-3 Metal Connector Plate Length Perpendicular to Grain of Wood Members.

5.4.6 Test Procedure.

5.4.6.1 Measurements.

After assembly, measure the Metal Connector Plate gross dimension perpendicular to the Wood Member's grain (D_{PR}) to the nearest 0.03 in. (0.08 mm). Take measurements at a minimum of three different locations on each Metal Connector Plate.

5.4.6.2 Loading Procedure.

Conduct tests on the Metal Connector Plate Test Specimens by concentrically loading the Test Specimen in tension applied normal to the joint (parallel to the grain of the Wood Members) at a uniform rate of movement of the movable crosshead of the testing machine so that maximum load is reached in not less than 60 seconds.

5.4.6.3 Testing Procedure for Solid Metal Control Specimens.

Conduct tests on the solid metal control specimens in accordance with *ASTM E8* procedures. Thickness shall be measured to the nearest 0.0001 in. (0.003 mm) and width to the nearest 0.001 in. (0.03 mm). Thickness of galvanized (or other) coating, if present, shall be measured or the coating shall be removed prior to thickness measurement. In lieu of coating measurement, the thicknesses given in Section 6.3.4.1.3 shall be permitted to be used for coating thickness.

5.4.6.4 Maximum Loads.

For the Metal Connector Plates and solid metal control specimens, observe the maximum loads in pounds-force (or Newton).

5.4.7 Calculations.

5.4.7.1 Ultimate Tensile Strength - Test Specimen.

For each Test Specimen, calculate the Metal Connector Plate ultimate tensile strength (F_{tp}) by dividing one-half of the maximum tensile load carried by the Test Specimen (P_{tp}) by an average gross cross-sectional area (A_{gp}) of the two Metal Connector Plates on the Test Specimen:

$$F_{tp} = \frac{P_{tp}}{2 \times A_{gp}} \quad (E5.4-1)$$

The gross cross-sectional area of each Metal Connector Plate (A_{gp}) is determined by multiplying the minimum thickness (t_{net}) of the Metal Connector Plate by the gross dimension of the Metal Connector Plate perpendicular to the Wood Member's grain (D_{PR}):

$$A_{gp} = t_{net} D_{PR} \quad (E5.4-2)$$

The three, or more, F_{tp} values calculated for each test (across the Metal Connector Plate Width and across the Metal Connector Plate Length) shall be averaged together, and the average value for each test shall be used in Section 5.4.7.3.

5.4.7.2 Ultimate Tensile Strength - Control Specimen.

Calculate the ultimate tensile strength of the solid metal control specimen (F_{tc}) by dividing the maximum tensile loads of each solid metal control specimen (P_{tc}) by the cross-sectional area of the respective solid metal control specimen (A_{gc}):

$$F_{tc} = \frac{P_{tc}}{A_{gc}} \quad (E5.4-3)$$

The cross-sectional area of each solid metal control specimen is determined by multiplying the minimum thickness (t_{net}) of the solid metal control specimen by the width of the solid metal control specimen (W):

$$A_{gc} = t_{net} W \quad (E5.4-4)$$

The F_{tc} values for all six or more solid metal control specimens from an individual coil of steel shall be averaged together, and the average value shall be used in Section 5.4.7.3.

5.4.7.3 Tensile Effectiveness Ratio.

Calculate the tensile Effectiveness Ratio (R_t), for both Metal Connector Plate Orientations - length perpendicular to grain and width perpendicular to grain - by dividing the average Metal Connector Plate ultimate tensile strength (F_{tp}) for each orientation by the average ultimate tensile strength of the matched solid metal control specimen (F_{tc}):

$$R_t = \frac{F_{tp}}{F_{tc}} \quad (E5.4-5)$$

5.4.8 Geometric Efficiency.

5.4.8.1 Definition.

Geometric efficiency is defined in Section 5.4.1.1.

5.4.8.2 Chain of Holes Extending in Diagonal or Zigzag Line.

For a chain of holes extending in a diagonal or zigzag line, the net width of the cross-section shall be obtained per Section 5.4.1.2.

5.4.9 Report.

The report shall include the following information:

- (a) Date of test and date of report.
- (b) Test sponsor and test agency.
- (c) Identification of Metal Connector Plates: manufacturer, model, type, material, finish, shape, dimensions, and other pertinent information. Metal Connector Plate material specifications shall include allowable tensile stress; also, identification of fasteners, such as type, size, quantity, and quality as well as the method of installing the Metal Connector Plates and their fasteners, used for load transfer in the case of nail-on plates, including the Nail Hole description.
- (d) Detailed drawings or photographs of test specimens before testing, if not fully described otherwise.
- (e) Complete description of test method and loading procedure used, if there are any deviations from the methods in this Standard.
- (f) Number of Test Specimens tested.
- (g) Rate of testing (crosshead speed or initial rate of load application).
- (h) Elapsed time of test.
- (i) All test data, including extrema and averages.
- (j) Tensile Effectiveness Ratios for each individual Test Specimen, and average values for all identical Test Specimens.
- (k) Description of type and path of failure.
- (l) Summary of findings.
- (m) Results of the solid metal control specimen test conducted per *ASTM E8*.
- (n) Certification of calibration of the testing machine.

CHAPTER 6

MATERIALS AND GENERAL DESIGN CONSIDERATIONS

6.1 GENERAL

6.1.1 Structural Analysis.

6.1.1.1 Mathematical Model.

Truss member axial forces, bending moments, and effective buckling lengths shall be based on a mathematical model of the Truss that closely approximates the geometry and properties of the Truss members and connections.

6.1.1.2 Structural Analysis Method.

An accepted structural analysis method for analyzing statically indeterminate structures, such as the matrix stiffness method, shall be used to determine the design moments and axial forces for each Truss member.

6.1.2 Truss Design Information.

Each Truss Design Drawing shall set forth, as a minimum, the information outlined in Sections 2.3.5.5 and 2.4.5.4 and any other requirements specified by the Building Designer. In addition, the following Truss design data shall be available:

- (a) Comprehensive design calculations, including the load combinations and conditions considered, along with the axial forces and moments resulting from these conditions;
- (b) The required number of effective Teeth for lateral resistance in each joint member contact area as determined in accordance with Section 8.3 using lateral strength design values derived per Section 5.2.9.2; and
- (c) The JSI for each joint, as calculated per Section 8.11.3.

6.2 LOADS

6.2.1 General Loading Requirements.

In the absence of a governing Building Code, loads, forces, and combinations of loads shall be in accordance with accepted engineering practice for the geographical area under consideration and the appropriate sections of the most recent implemented ASCE 7.

6.2.2 Loading Requirements for Metal-Plate-Connected Wood Trusses.

The following loading conditions shall apply to the design of metal-plate-connected wood Trusses.

6.2.2.1 Non-Bearing Partitions.

The weight of non-bearing partitions shall be permitted to be ignored for Truss design purposes given the following conditions:

- (a) Trusses are spaced less than or equal to 24 in. (610 mm) on center;
- (b) All Top Chord panel lengths of supporting Trusses are less than or equal to 30 in. (760 mm) when the lumber is oriented in the flat direction;
- (c) Design live load of supporting Trusses results from a residential occupancy and is not less than 40 psf (1920 Pa); and
- (d) Partition weight is less than or equal to 60 pounds per linear foot (875 N/m).

6.2.2.1.1 Non-Bearing Partition Weight Not Permitted to be Ignored.

If the conditions listed above do not exist, the Building Designer shall specify in the structural design documents the non-bearing partition loads that need to be applied to the Trusses.

6.2.2.1.2 Non-Load Bearing Partitions Parallel to Supporting Trusses.

When non-load bearing partitions parallel to supporting Trusses are not located on or immediately adjacent to a Truss, the sub-floor shall be of adequate strength and stiffness to support the non-load bearing partition load, or other provisions shall be made by the Building Designer to distribute the non-load bearing partition weight to the supporting Trusses.

6.2.2.2 Attic Live Loads.

Attic live loads, other than floor live loads, shall comply with the governing Building Code.

6.2.2.3 Effect of Pitch.

Dead loads specified on a projected horizontal area basis

shall have taken into account the effect of the pitch.

6.2.2.4 Dead Loads for Determining Wind Uplift.

The dead load used in determining wind uplift shall not exceed the minimum expected actual weight of the materials, or 0.6 times the nominal design dead load if the minimum expected actual weight of the materials is not known.

6.2.2.5 Full- and Partial-Length Live Loading.

Live load on Trusses shall be considered for both cases of full-length loading and partial-length loading, where partial-length loading shall exclude the live load over the Truss in the area between a non-triangulated panel (i.e., open or mechanical chases in floor or roof Trusses) and the nearest bearing.

6.3 DESIGN VALUES

6.3.1 Design Values for Solid Sawn Lumber.

Design values (E , E_{\min} , F_b , F_c , $F_{c\perp}$, F_t , and F_v) for solid-sawn lumber and approved, grade stamped, finger-jointed lumber shall be as defined by the grade stamp prior to cross cutting and in accordance with the published values of lumber rules writing agencies approved by the Board of Review of the American Lumber Standards Committee.

Design of lumber chord and web members shall be based on dressed sizes as set forth by the U.S. Department of Commerce, *PS-20*. If other sizes or materials are used, the net dressed size shall be stated in the design and used in the design calculations.

6.3.2 Design Values for Structural Composite Lumber.

Design values for Structural Composite Lumber shall be approved by the authorities having Jurisdiction. The allowable tension stress value, F_t , for Structural Composite Lumber shall account for the length of lumber exposed to tensile stress. An allowable tension stress value established on the basis of a 20 ft. (6.096 m) length fully exposed to the maximum tensile stress shall be permitted to be used without further reduction for length effects for Structural Composite Lumber used as members subject to combined bending and tensile stresses.

6.3.3 Design Values for Fasteners Other Than Metal Connector Plates.

Design values for fasteners other than Metal Connector Plates shall be in accordance with the *ANSI/AF&PA NDS*. Other fasteners shall be permitted when approved by the authorities having Jurisdiction.

6.3.4 Design Values for Metal Connector Plates.

6.3.4.1 Allowable Steel Stresses.

Allowable stresses in steel shall be applied to Effectiveness Ratios for Metal Connector Plates as determined per Chapter 5 of this Standard.

6.3.4.1.1 Tensile Stress.

The allowable tensile stress, F_{st} , shall not exceed $0.60F_y$ or $0.50F_u$.

6.3.4.1.2 Shear Stress.

The allowable shear stress, F_{vs} , shall not exceed $0.40F_y$ or $0.30F_u$.

6.3.4.1.3 Design Thickness.

The design thickness (t_1) of Metal Connector Plates shall be the minimum steel thickness specified (t), less any specified coating thickness (t_c), divided by 0.95, as shown in Equation E6.3-1.

$$t_1 = \frac{t - t_c}{0.95} \quad (\text{E6.3-1})$$

For galvanized steel, coating thickness deductions (t_c) shall be:

G185	0.0031 in. (0.078 mm)
G90	0.0015 in. (0.038 mm)
G60	0.0010 in. (0.025 mm)
Electrolytic	0.0003 in. (0.008 mm)

For aluminum-zinc alloy coated steel, coating thickness deduction (t_c) for designation AZ50 shall be 0.002 in. (0.05 mm). These deductions are the total for both sides.

6.3.4.2 Allowable Lateral Resistance Stresses.

Allowable stresses for lateral resistance shall be determined in accordance with the procedures of Chapter 5 of this Standard.

6.4 ADJUSTMENTS TO DESIGN VALUES

6.4.1 Load Duration Factor (C_D).

6.4.1.1 Applicability.

Design values shall be permitted to be adjusted for load duration conditions in accordance with this section, unless otherwise specified by the authorities having Jurisdiction. The Building Designer shall be permitted to reduce the load duration factor when the expected load

durations are greater than the assumed durations in Section 6.4.1.3.

6.4.1.2 Design Values Affected.

Adjustments for load duration apply to all lumber and plate lateral resistance (tooth holding) Design Values, with the exception of modulus of elasticity (E) and compression perpendicular to grain ($F_{c\perp}$).

6.4.1.3 Method.

The adjustment for load duration shall be accomplished by multiplying the design value by the appropriate C_D factor as shown in Table 6.4-1.

Table 6.4-1 Load Duration Factor (C_D) for Load Durations.

Load Duration	C_D
Permanent	0.90
Normal - 10 Years duration	1.00
Snow - 2 Months duration	1.15
Construction - 7 Days duration	1.25
Wind & Earthquake - 5-10 minutes	1.60
Impact*	2.0

* For FRT and pressure-preserved lumber and all connections subject to an impact load, the duration of load factor shall not exceed 1.6.

6.4.1.4 Load Combinations.

For combinations of loads with different durations, the load duration factor, C_D , for the shortest duration load that is part of that load combination shall apply for that entire load combination.

6.4.2 Repetitive Member Increase (C_r).

6.4.2.1 Definitions and Conditions for Use.

Repetitive member design values apply to all Truss chord members where three or more Trusses are positioned side by side, are in contact, or are spaced no more than 24 in. (610 mm) on center and are joined by roof sheathing, flooring, gypsum, or other load distributing elements attached directly to the chords, as follows:

- (a) For solid sawn lumber members to which structural wood sheathing is mechanically attached: use the repetitive member design value listed in the recognized lumber grading rules, or a 15 percent increase to F_b and 10 percent increase to F_c , F_t and E_{min} .
- (b) For solid sawn lumber members to which struc-

tural wood sheathing is not attached: use the repetitive member design value listed in the recognized lumber grading rules, or a 10 percent repetitive member design value increase to F_b , F_c , F_t and E_{min} .

- (c) For Structural Composite Lumber: repetitive member design values shall be limited to no more than a 4 percent increase to F_b and no (zero) increase to other allowable design values.

6.4.2.2 Limitations.

Single-ply and two-ply girder Trusses are not permitted to use the repetitive member increases outlined in Section 6.4.2.1.

6.4.3 Bending Capacity Modification Factor (K_m).

The bending capacity modification factor, K_m , as stated in Table 6.4-2, shall be permitted to be applied to the bending design value, F_b , for solid sawn lumber. K_m must be applied with respect to the entire Truss.

Note: K_m shall equal 1 for chord material other than solid sawn lumber (e.g., structural composite lumber). It shall not be permitted to use K_m per Table 6.4-2 for a portion of a Truss and to not use K_m (effective $K_m = 1$) for other portions of a Truss.

6.4.4 Flat Use Factor (C_{fu}).

Flat use factor for solid sawn lumber, C_{fu} , as stated in Table 6.4-3, shall be permitted to be applied to the bending design value, F_b , of solid sawn lumber members when the member is subjected to bending about its weak axis. Otherwise, C_{fu} shall be taken as unity.

Table 6.4-3 Flat Use Factor (C_{fu}) for Lumber 2" Thick.

Width (in.)	C_{fu}
2 & 3	1.0
4	1.1
5	1.1
6	1.15
8	1.15
10 & wider	1.2

6.4.5 Buckling Stiffness Factor (C_T).

6.4.5.1 Conditions for Use.

The buckling stiffness factor, C_T , shall be applied to E_{min} , and shall be determined using Section 6.4.5.2 when the following conditions in (a) through (d) are met:

Table 6.4-2 Bending Capacity Modification Factors.

K_m	Applicable Condition
$\left[1.31 + 0.12 \times \left(\frac{M_1}{M_2} \right) \right] \times \left(\frac{L_p}{d} \right)^{\frac{1}{6}} \leq 1.3$	Compression chord members continuous over one or more panel points, and where $1.0 \leq \frac{M_1}{M_2} \leq 3.0$
$\left[2.20 - 0.53 \times \left(\frac{M_1}{M_2} \right) - 0.64 \times \left(\frac{M_1}{M_2} \right)^2 + 0.41 \times \left(\frac{M_1}{M_2} \right)^3 \right] \times \left(\frac{L_p}{d} \right)^{\frac{1}{6}} \leq 1.3$	Compression chord members continuous over one or more panel points, and where $-1.0 \leq \frac{M_1}{M_2} \leq 1.0$
$1.67 \times \left(\frac{L_p}{d} \right)^{\frac{1}{6}} \leq 1.3$	All other compression chord members

L_p = Actual length of the member between adjacent panel points, in.

d = Depth of the member between adjacent panel points, in.

M_1 = Maximum bending moment between panel points (not including panel points)

M_2 = Maximum of the two bending moments at the adjacent panel points

Note: The sign of the bending moments, M_1 and M_2 , are retained in determining K_m . When M_1 is of the same sign as M_2 , then one of either M_1 or M_2 will actually be a minimum bending moment for that panel (rather than a maximum).

- (a) The member size is 2x4 (38 x 89 mm) or smaller;
- (b) Continuous $\frac{3}{8}$ in. (9.5 mm) or thicker wood structural panel sheathing is attached to the chord with fasteners of the type, size, and spacing as required;
- (c) The member is subjected to combined bending and axial compression; and
- (d) The Trusses are used under dry service conditions.

If these conditions are not met, C_T shall be taken as unity.

6.4.5.2 Method.

When permitted in accordance with Section 6.4.5.1, the buckling stiffness factor (C_T) shall be determined as follows:

$$C_T = 1 + \frac{2300 \times L'}{k \times E} \quad (\text{E6.4-1})$$

for wood seasoned to a moisture content of 19 percent or less at the time the sheathing is applied to the chord, or as:

$$C_T = 1 + \frac{1200 \times L'}{k \times E} \quad (\text{E6.4-2})$$

for wood that is unseasoned or partially seasoned at the time of sheathing attachment, where:

L' = Effective buckling length in inches, but not greater than 96 in. (2440 mm)
and

$k = 0.82$ for $\text{COV}_E \leq 0.11$ for machine stress rated or structural composite lumber

= 0.75 for $\text{COV}_E \leq 0.15$ for machine evaluated lumber

= 0.59 for $\text{COV}_E \approx 0.25$ for visually graded lumber

6.4.6 Wet Service Factor (C_M).

6.4.6.1 Conditions for Use and Values.

When dimension lumber is used where moisture content

will exceed 19 percent for an extended time period, design values shall be multiplied by the appropriate wet service factors in Table 6.4-4, except as specified in Section 6.4.6.2.

6.4.6.2 Exceptions.

C_M shall be taken as unity for F_b or F_c if the following conditions are met:

If $(F_b)(C_{fu}) \leq 1150$ psi, $C_M = 1.0$ for F_b

If $(F_c) \leq 750$ psi, $C_M = 1.0$ for F_c

6.4.6.3 Moisture Content > 19% at the Time of Fabrication.

Metal Connector Plates installed in lumber having a moisture content greater than 19 percent at the time of Truss fabrication shall have the lateral resistance value (V_{LR}) multiplied by C_M .

6.4.7 Temperature Factor (C_t).

For structural members that will experience sustained exposure to elevated temperatures up to 150 degrees Fahrenheit, the tabulated design values shall be multiplied by the temperature factors in Table 6.4-5.

6.4.8 Incising Factor (C_i).

6.4.8.1 Conditions for Use.

Tabulated design values shall be multiplied by the incising factor, C_i , per the current edition of *ANSI/AF&PA NDS*, when structural sawn lumber is incised to increase penetration of preservatives with incisions cut parallel to grain, a maximum depth of 0.4 in. (10 mm), a maximum

length of $\frac{3}{8}$ in. (9.5 mm), and a maximum density of incisions of $1100/\text{ft}^2$ ($11800/\text{m}^2$). Incising factors shall be determined by test or by calculation using reduced section properties for incising patterns exceeding these limits.

6.4.8.2 Reductions for the Design of Metal Connector Plates.

Reduction factors shall be used for design of Metal Connector Plates installed in incised lumber.

6.4.9 Chemically Treated Lumber.

6.4.9.1 Fire Retardant Treated (FRT) Lumber.

All FRT lumber used in Trusses shall be re-dried after treatment to 19 percent maximum moisture content at temperatures not to exceed 160°F (71°C). FRT lumber design values shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including effects of elevated temperature and moisture. Design values shall be approved by the authorities having Jurisdiction.

6.4.9.2 Metal Connector Plates Installed in FRT.

Metal Connector Plates installed in lumber pressure-impregnated with fire retardant chemicals shall have the reductions for lateral resistance values specified by the FRT chemical manufacturer. The quality mark shall indicate that the design value adjustments are in accordance with either the FRT manufacturer's specifications or based upon an approved method of investigation which takes into consideration the effects of the anticipated temperature and humidity of which the FRT will be subjected.

Table 6.4-4 Wet Service Factor (C_M).

F_b	F_t	F_v	$F_{c\perp}$	F_c	$E & E_{min}$	V_{LR}
0.85	1.0	0.97	0.67	0.8	0.9	0.8

Table 6.4-5 Temperature Factor, C_t .

Design Values	In Service Moisture Conditions	$T \leq 100^\circ\text{F}$	C_t $100^\circ\text{F} \leq T \leq 125^\circ\text{F}$	$125^\circ\text{F} < T \leq 150^\circ\text{F}$
F_t, E, E_{min}	Wet or Dry	1.0	0.9	0.9
$F_b, F_v, F_c, F_{c\perp}$	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

Table 6.4-6 Quality Control Factor (C_q).

Fabrication Tolerance	C_q
0%	1.00
5%	0.95
10%	0.90
15%	0.85
20%	0.80
25%	0.75
30%	0.70

Note: These are example fabrication tolerances for a given C_q factor. The actual C_q factor shall be based on the fabrication tolerance set by the Truss Manufacturer.

6.4.9.3 Preservative Treated Lumber.

All preservative treated lumber used in Trusses re-dried after treatment shall be re-dried to 19 percent maximum moisture content at temperatures not to exceed 160°F (71°C). Design values for preservative treated lumber used in Trusses shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including incising marks. Design values shall be approved by the authorities having jurisdiction.

6.4.10 Quality Control Factor (C_q).

6.4.10.1 Applicability to Design Values.

The quality control factor (C_q) shall only apply to plate lateral resistance design values (V_{LR}).

6.4.10.2 Conditions for Use.

The C_q factor shall be based on the fabrication tolerance selected by the Truss Manufacturer outlined in their In-Plant Quality Assurance Program per Section 3.2. Fabrication tolerance and resulting C_q factor are shown in Table 6.4-6. The C_q factor shall not exceed 1.00 in any case.

6.4.10.3 Modified Fabrication Tolerance.

The Truss Designer shall be permitted to increase the C_q factor based on the fabrication tolerance selected by the Truss Manufacturer, on a joint-by-joint basis, where necessary to evaluate the design requirements for that joint. In such cases, the Truss Design Drawing shall indicate that the joint was designed using a modified fabrication tolerance.

6.4.11 Other Adjustment Factors.

Wood design stresses for dry lumber shall be permitted to be used for green lumber when the following three conditions are met:

- (a) Trusses shall be stored after fabrication and installed in an exposure with equilibrium moisture content conditions of 19 percent or less;
- (b) Appropriate reduction factors (C_M) shall be used for design of fasteners installed prior to the drying of the lumber, including Truss plates, nails, joist hangers, and similar fasteners; and
- (c) Typical conditions in that geographical area permit drying of the lumber to 19% moisture content or less prior to the closing in of the structure.

6.5 CORROSIVE ENVIRONMENTS

6.5.1 Recognized Coatings.

The following coatings are recognized as providing increased corrosion protection to Metal Connector Plates:

- (a) Epoxy-Polyamide Primer (SSPC-Paint 22).
- (b) Coal-Tar Epoxy-Polyamide Black or Dark Red Paint (SSPC-Paint 16).
- (c) Basic Zinc Chromate-Vinyl Butyral Wash Primer (SSPC-Paint 27) and cold applied Asphaltic Mastic (Extra Thick Film) Paint (SSPC-Paint 12).

- (d) Post-plate-manufacture hot dip galvanizing per *ASTM A153*.

6.5.2 Application of Coating.

Embedded Metal Connector Plates shall be free of dirt and oil prior to coating application. If the coating is damaged prior to, or during Truss installation, such damage shall be alleviated before accessibility is impeded.

6.5.3 Stress Corrosion Cracking.

Metal Connector Plates, including Types 304 and 316 stainless steel plates, shall not be exposed to swimming pool environments unless adequate provision is made to prevent stress corrosion cracking. In lieu of use of a stainless steel that is not susceptible to stress corrosion cracking, Trusses shall be separated from the pool environment by a vapor barrier and shall be separately ventilated from the pool environment.

6.6 USE OF METAL CONNECTOR PLATES IN CONVENTIONAL WALLS (LADDER FRAME)

6.6.1 Conditions for Use.

Metal Connector Plates not less than 1 in. x 3 in. (19 x 64 mm) on each face shall be permitted in lieu of nails or staples to fasten the wood studs to the wood plates in conventional walls. The bottom plate of the wall shall be continuously supported along its length and the maximum wall height (Ladder Frame depth) shall not exceed 24 in. (610 mm).

6.6.2 Cutting and Notching.

The cutting and notching of members shall be permitted within the requirements detailed in the Building Code for any wall member.

CHAPTER 7

MEMBER DESIGN PROCEDURES

7.1 SCOPE

Each member shall be designed in accordance with Chapter 7 of this Standard to resist all forces and bending moments determined in accordance with Section 6.1.1.

7.2 EFFECTIVE BUCKLING LENGTHS

7.2.1 Effective Buckling Lengths for Chord Members.

The effective buckling lengths (L') for chord members (see Figures 7.2-1 and 7.2-3) shall be determined by an accepted structural analysis method, such as multiplying the unbraced lengths by an effective length factor as described in Sections 7.2.1.1 and 7.2.1.2 for buckling in the plane of the Truss, and by an effective length factor of 1.0 for buckling out of the plane of the Truss.

7.2.1.1 Chord Member Panels Not Subject to Sidesway.

Chord panels between panel points not subject to Sidesway, as shown in Figure 7.2-2, shall be permitted to use effective buckling lengths, L' , equal to $K \times L$, where K is calculated per Equation E7.2-1 and shall not be less than 0.65, except for interior panels of continuous chord members spanning at least five panels, for which K shall be no less than 0.55. Panel points not subject to Sidesway shall include only those panel points that are prevented from moving relative to one another by adequate restraint and bracing, as shown in Figure 7.2-2.

$$K = \frac{\{(\pi^2 + 2 \times N_a) \times (\pi^2 + 2 \times N_b)\} / [(\pi^2 + 4 \times N_a) \times (\pi^2 + 4 \times N_b)]}{\}^{0.5} \quad (E7.2-1)$$

where:

$$\pi = 3.1416$$

$$N_a = (4 \times E_a \times I_a / L_a) / (E_t \times I_t / L_t)$$

$$N_b = (4 \times E_b \times I_b / L_b) / (E_t \times I_t / L_t)$$

E_t = MOE of chord member for which K is being determined

I_t = Moment of inertia of chord member for which K is being determined

L_t = Length of member for which K is being determined

E_a = MOE of chord member adjacent to member for which K is being determined

I_a = Moment of inertia of chord member adjacent to member for which K is being determined

L_a = Length of chord member adjacent to member for which K is being determined

E_b, I_b, L_b = The same as E_a, I_a, L_a except for the adjacent chord member to the opposed side of the member for which K is being determined. If there is no such adjacent chord member, N_b shall equal zero.

7.2.1.2 Chord Member Panels Subject to Sidesway.

For the effective buckling length, L' , equal to $K \times L$, for chord member panels subject to Sidesway, such as Overhangs including those Overhangs where the Truss bearing is at the end of the Overhang (also known as a tray condition), K shall be no less than 2.1 for panels with deflected shapes having only a single direction of curvature and no less than 1.0 for members in panels with deflected shapes having two directions of curvature.

7.2.2 Effective Buckling Lengths for Web Members.

7.2.2.1 Buckling in the Plane of the Truss.

The effective buckling length of Web members for buckling in the plane of the Truss shall be 0.8 times the unbraced length with respect to the Web width, d_1 , for Webs not subject to Sidesway. The Effective Buckling Length shall be 2.1 times the unbraced length with respect to the Web width, d_1 , for Webs subject to Sidesway, where the unbraced length is the length of the Web between Truss joints. For Webs crossing more than two joints, each panel shall be considered and checked separately where a panel is the length of the Web between adjacent Truss joints. Webs subject to Sidesway include those Web panels that extend outside of triangulated portions of the Truss, such as Webs extended to bearings below the Bottom Chord, Webs extended above the Top Chord to form a parapet wall without attachment to another Web or chord at the upper end of the extended Web, and similar conditions (see Figures 7.2-1 and 7.2-3).

Member	Effective Dimension (d)	Effective Buckling Length (L')
Top Chord & Bottom Chord	d_1	see Section 7.2.2
	d_2	L_u
Web	d_1	$0.8L_w$
	d_2	$0.8L_u$ or $0.8L_w$

Note: Where sheathing or ceiling is nailed to a compression chord (either Top or Bottom), lateral support of the supported axis shall be assumed to be continuous and $L'/d = L_u/d_2$ is neglected.

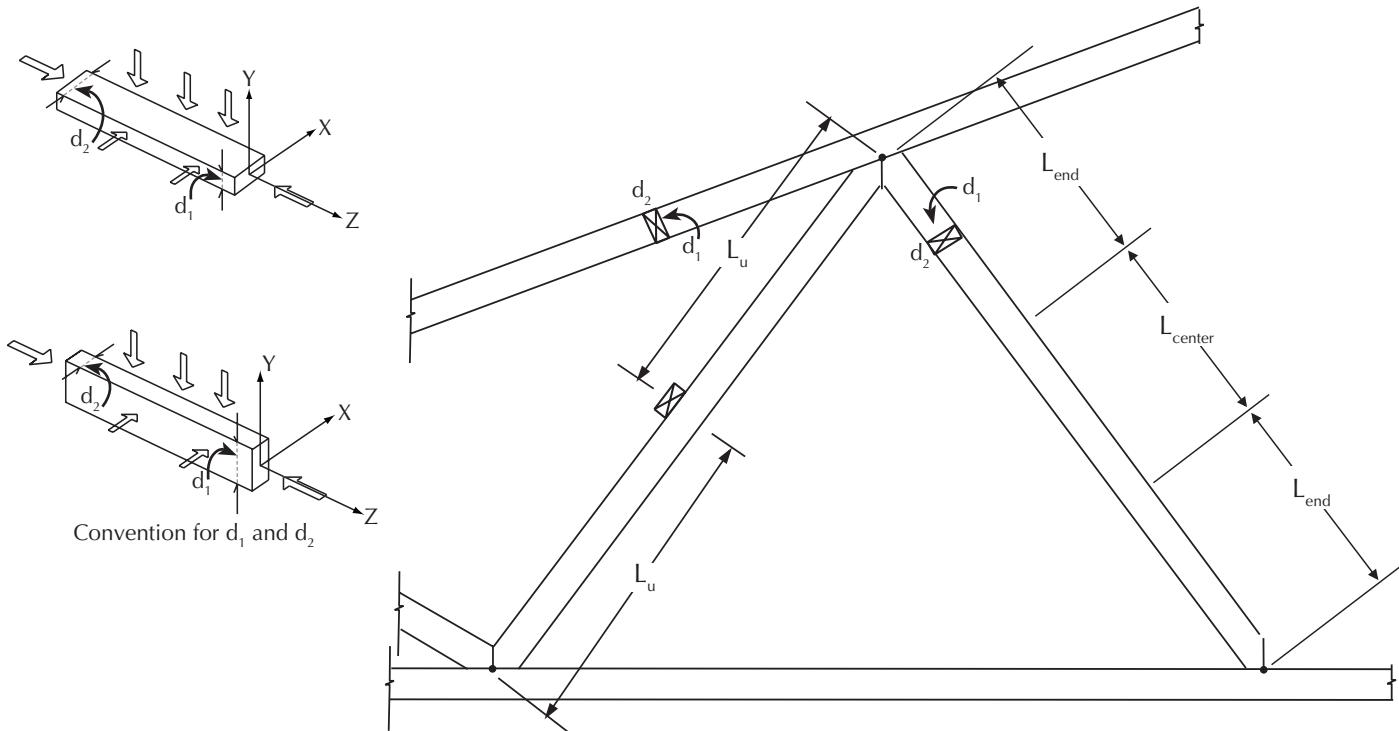


Figure 7.2-1 Unbraced Lengths for Out-of-Plane Buckling, Webs with 1 or 2 Lateral Restraint Locations.

7.2.2.2 Buckling out of the Plane of the Truss.

The effective buckling length for buckling out of the plane, meaning with respect to the Web thickness, d_2 , shall be as follows (see Figures 7.2-1 and 7.2-3):

For Webs that do not require any intermediate Lateral Restraint: $L' = 0.8 \times L_w$, where L_w is the Web length.

For Webs requiring one intermediate point of Lateral Restraint: $L' = 0.8 \times L_u$, where L_u is the length between the

lateral point of Lateral Restraint and the farthest Web end from that restraint point.

For Webs requiring two intermediate points of Lateral Restraint: $L' = \text{greater of } L_{\text{center}} \text{ or } 0.8 \times L_{\text{end}}$, where L_{end} is the larger of the lengths between the point of Lateral Restraint and the adjacent Web end, and L_{center} is the length between the two points of Lateral Restraint.

Webs requiring three or more intermediate points of Lateral Restraint shall not be permitted.

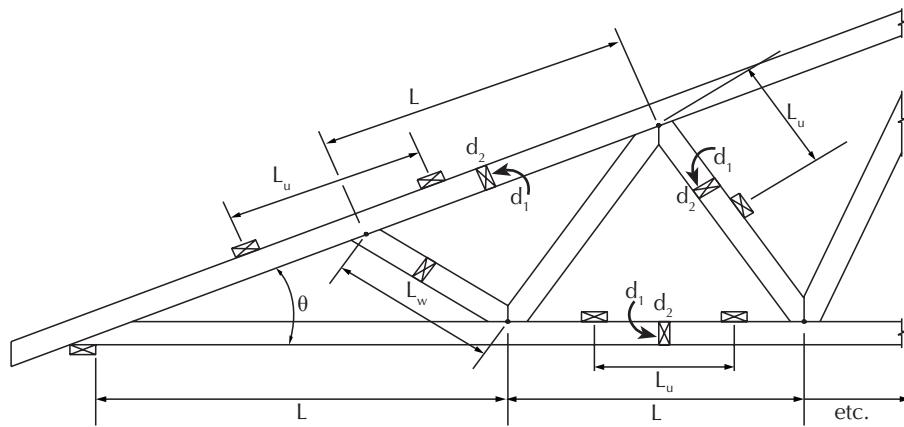


Figure 7.2-2 Unbraced Lengths for Out-of-Plane (L_u , L_w) and In-Plane (L , L_w) Buckling, Members Not Subject to Sidesway.

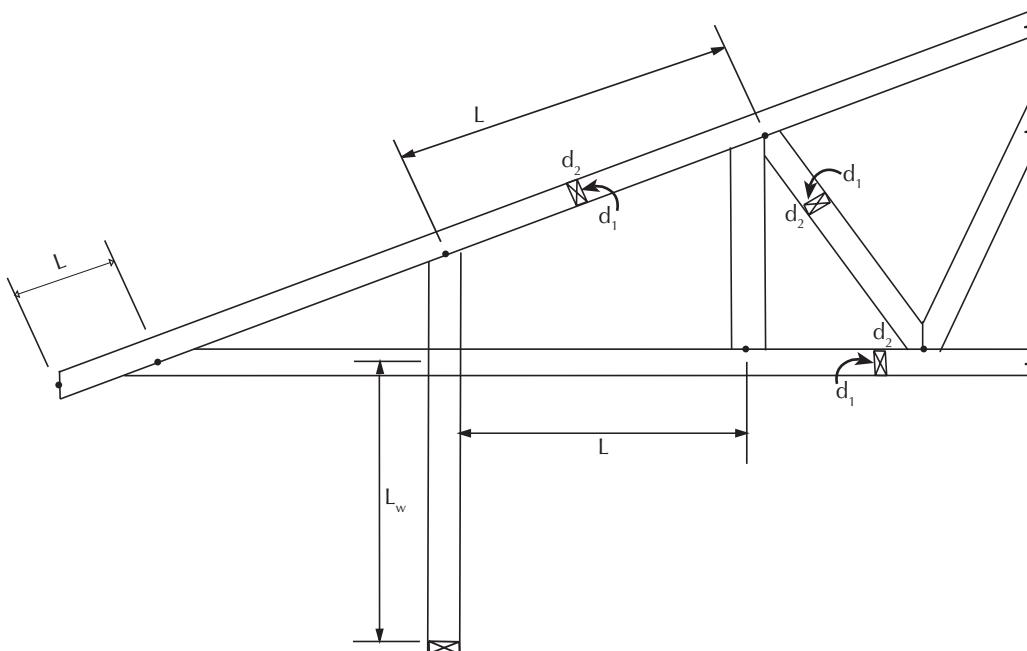


Figure 7.2-3 Unbraced Lengths for In-Plane (L , L_w) Buckling, Members Subject to Sidesway.

7.3 MEMBER DESIGN

$$f_c \leq F_c' \quad (\text{E7.3-2})$$

7.3.1 Tension Members.

Members subject to axial tension only shall be so proportioned that:

$$f_t \leq F_t' \quad (\text{E7.3-1})$$

where:

$$f_t = P_t / A$$

$$F_t' = F_t^* \times (K_t) \times$$

where:

$$f_t = P_t / A$$

$$F_t' = F_t^* \times (K_t) \times \left[\left(1 + \frac{F_{cE}}{F_t^*} \right) \times \sqrt{\left(1 + \frac{F_{cE}}{F_t^*} \right)^2 - \left(\frac{F_{cE}}{(2c)} \right)^2} \right]$$

7.3.2 Compression Members.

Members subject to axial compression only shall be so proportioned that:

$$\begin{aligned} c &= 0.8 \text{ for sawn lumber} \\ &= 0.9 \text{ for Structural Composite Lumber} \end{aligned}$$

$K_f = 1.0$ for single ply members and determined per ANSI/AF&PA NDS for multiple ply members

$$F_c^* = F_c(C_D)(C_M)(C_t)(C_r)(C_i)$$

Except that F_c' shall be permitted to equal F_c^* at panel point checks for strong axis bending.

And the buckling design value, F_{cE} , is determined as:

The smaller of the following:

$$F_{cEx} = \frac{(0.822) \times E'_{min}}{\left(\frac{L'}{d_1} \right)^2}$$

$$F_{cEy} = \frac{(0.822) \times E'_{min}}{\left(\frac{L'}{d_2} \right)^2}$$

where:

$$E'_{min} = E_{min}(C_T)(C_t)(C_M)(C_i)(C_r)$$

7.3.3 Bending Members.

7.3.3.1 Design Requirements.

Members subject to bending stresses in the plane of the Truss only shall be so proportioned that:

$$f_b \leq F_b' \quad (\text{E7.3-4})$$

where:

$$f_b = M / S$$

Note: Reductions in section modulus (S) for tapered bending members, such as shown in Figures 7.3-2 or 7.3-3 shall be considered.

$$F_b' = F_b^* \times \left[\left(1 + \frac{F_{bE}}{F_b^*} \right) - \sqrt{\left(1 + \frac{F_{bE}}{F_b^*} \right)^2 - \left(\frac{F_{bE}}{F_b^*} \right)} \right]$$

$$F_b^* = F_b(C_D)(C_M)(C_t)(C_r)(C_{fu})(C_i)(K_m)$$

And the buckling design value, F_{bE} , is determined, when d_1 is greater than d_2 :

$$F_{bE} = \frac{(1.20) \times E'_{min}}{\left[\frac{L_e \times d_1}{d_2^2} \right]}$$

where:

$$E'_{min} = E_{min}(C_T)(C_t)(C_M)(C_i)(C_r)$$

$$\sqrt{\frac{L_e \times d_1}{d_2^2}} < 50$$

And the effective span length, L_e , for bending members is determined as follows:

$$L_e = 2.06L_u \quad \text{when } L_u / d_1 < 7$$

$$L_e = 1.63L_u + 3d_1 \quad \text{when } 7 \leq L_u / d_1 \leq 14.3$$

$$L_e = 1.84L_u \quad \text{when } L_u / d_1 > 14.3$$

7.3.3.2 L_u Value When Depth Exceeds Breadth.

When the depth of a bending member, d_1 , exceeds its breadth, d_2 , lateral support shall be provided at points of bearing to prevent rotation or lateral displacement at those points. When intermediate support is provided by purlins or bracing, connected so that they prevent lateral displacement of the loaded edge of the bending member, the unsupported length, L_u , shall be the maximum spacing between purlins or bracing at the top (loaded) edge of the chord member (see Figure 7.2-2).

7.3.3.3 L_u Not Required When Depth Does Not Exceed Breadth.

For members where depth, d_1 , does not exceed breadth, d_2 , F_b' shall be equal to F_b^* .

7.3.3.4 Fully Supported Bending Members.

When the loaded edge of a bending member is supported throughout its length by continuous sheathing to prevent its lateral displacement, and the ends at points of bearing have lateral support to prevent out-of-plane rotation, the member shall be considered fully supported, and F_b' shall be equal to F_b^* .

7.3.3.5 Bottom Chord Bending Members.

When the Bottom Chord member of a Truss has a depth-to-thickness ratio (i.e., d_1/d_2 , based on nominal

dimensions) not exceeding 5 to 1, is spaced no more than 2 ft. (61 cm) on center, is braced throughout its length by an approved sheathing material, such as gypsum board or wood structural panels installed directly to the Bottom Chord of the Truss and fastened in accordance with *ASTM C840* or governing Building Codes or standards, F_b' shall be equal to F_b^* .

7.3.3.6 Panel Point Moment Region at the Heel.

For the panel point moment region at the heel of a Truss, when the bearing is under the Bottom Chord and within the scarf of the heel joint, the allowable design value for bending shall be permitted to be increased 30 percent (see Figure 7.3-1) for solid sawn lumber and 10 percent for Structural Composite Lumber, provided that K_m is set equal to 1.0 and not calculated per the equations in Table 6.4-2. This region shall be limited to no more than 2 times the chord depth as measured along the Top and Bottom Chords from the point of maximum moment.

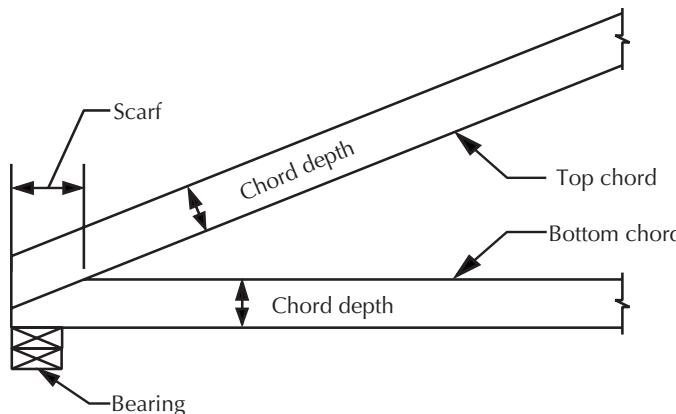


Figure 7.3-1 Heel Detail.

7.3.3.7 Composite Action of Multiple Layers.

Bending members consisting of multiple layers that are not glued-laminated or otherwise connected to assure composite action shall be designed assuming the layers are separate with no composite action other than resulting from the discrete connections connecting the layers. For bending members consisting of two layers that are connected with Metal Connector Plates at intervals not exceeding 30 in. (76 cm), moment of inertia and section modulus shall be permitted to be determined as 60 percent and 70 percent, respectively, of the moment of inertia and section modulus for the fully composite member.

7.3.4 Combined Bending & Tension.

Members subject to both bending and axial tension shall be so proportioned that:

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.00 \quad (\text{E7.3-5})$$

and

$$f_b - f_t \leq F_b' \quad (\text{E7.3-6})$$

7.3.5 Combined Bending & Compression.

7.3.5.1 Design Requirements.

Members subjected to a combination of bending about one or both principal axes and axial compression shall be so proportioned that:

$$\left(\frac{f_c}{F_c'} \right)^2 + \left[\frac{f_{bx}}{F_{bx}' \times \left(1 - \frac{f_c}{F_{cEx}} \right)} \right] + \frac{f_{by}}{F_{by}' \times \left[1 - \frac{f_c}{F_{cEy}} - \left(\frac{f_{bx}}{F_{bE}} \right)^2 \right]} \leq 1.00 \quad (\text{E7.3-7})$$

where:

$$f_c < F_{cEx}$$

$$f_c < F_{cEy}$$

$$f_{bx} < F_{bE}$$

F_c' , F_{cEx} , and F_{cEy} shall be based on the controlling buckling design value determined from Section 7.3.2, and F_{bE} shall be calculated as shown in Section 7.3.3.

F_{bx}' and F_{by}' are the allowable Design Values for bending in the plane of the Truss and out of the plane of the Truss, respectively. F_{bx}' shall be calculated as F_b' as shown in Section 7.3.3, and F_{by}' shall be calculated as F_b^* in Section 7.3.3 except with the following modifications: d_2 and d_1 shall be reversed in all occurrences, L_u shall be the distance between adjacent panel points, and Sections 7.3.3.4 and 7.3.3.5 shall not apply.

At a panel point, the quantity $[1 - f_c/F_{cEx}]$ shall be replaced by 1.

7.3.5.2 Chord Members Continuously Braced.

Chord members that are braced throughout their length by continuous sheathing need only be checked per Section 7.3.5 for buckling within the plane of the Truss. Hence F_c' shall be based upon L'/d , where d is equal to d_1 in Figure 7.2-1.

7.3.5.3 Chord Members Not Continuously Braced.

Chord members that are not continuously braced throughout their length shall be checked per Section 7.3.5 for both buckling within the plane of the Truss and buckling within a plane perpendicular to the plane of the Truss. For buckling in the plane perpendicular to the Truss, F_c' shall be determined based upon L'/d .

where:

$$L' = L_u$$

$$d = d_2 \text{ (as shown in Figure 7.2-1)}$$

7.3.6 L'/d Ratios for Compression & Tension Members.

Unless design calculations are performed to account for the interaction of axial compression with initial deformation of compression members due to warp or other causes, the maximum L'/d for long-term compression members shall not exceed 50, and the maximum L'/d for tension members subject to reversal of stress due to short term loads other than gravity loads, shall not exceed 80. For Chords and Webs, the effective buckling length shall be as shown in Section 7.2.

7.3.7 Shear.

7.3.7.1 Design Requirements.

Members subject to shear stress shall be so proportioned that:

$$f_v \leq F_v' \quad (\text{E7.3-8})$$

where:

$$f_v = \frac{3V}{2A}$$

$$F_v' = F_v(C_D)(C_t)(C_M)(C_i)$$

7.3.7.2 Tapered Bottom Chord Heel.

For tapered Bottom Chord members at heel joints, a shear check is required when the projection point is closer to the end of the Truss than the Truss point, as illustrated in Figure 7.3-2. The actual shear stress along-the-grain (horizontal) shall be calculated using the depth, d_n , at the projection point, a point at which a line initiating at the inside edge of the bearing and extending upward at an angle of 45 degrees to the length of the Chord intersects with the Bottom Chord centerline, as illustrated in Figure 7.3-2. If the Bottom Chord center line intersects the scarf cut before intersecting with the 45 degree line, d_n , is determined from where the 45 degree line intersects the scarf cut.

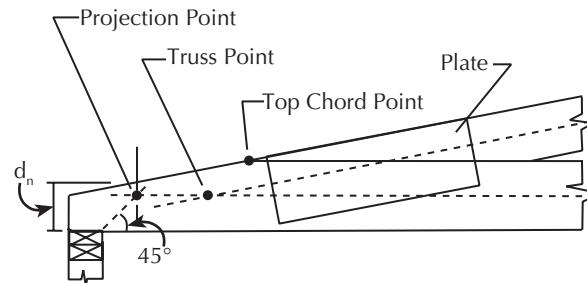


Figure 7.3-2 Depth for Shear Check of Tapered Bottom Chord Heel.

7.3.7.3 Scarf Cut Bearings.

The maximum shear load imposed from bearings shall not exceed the allowable shear load shown below for Wood Members that bear at an angle other than parallel to grain and with the bearing cut on the member extending beyond the inside face of the bearing, such as shown in Figure 7.3-3.

$$V' = (\frac{2}{3}) \times F_v' \times d_2 \times d_n \times (d_n/d_1)^2 \quad (\text{E7.3-9})$$

where:

d_n = Depth of the member measured perpendicular to the slope at the inside edge of the bearing

Other variables are as defined in previous sections.

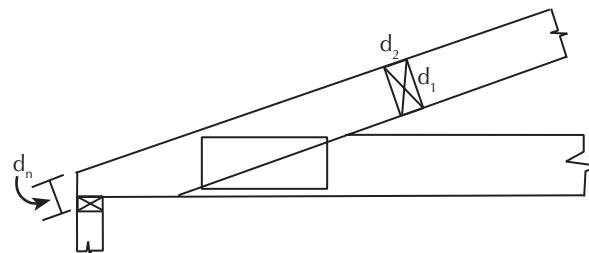


Figure 7.3-3 Depth for Shear Check of Scarf Cut Bearing.

7.3.7.4 Use of Metal Connector Plates to Resist Shear.

Metal connector plates are permitted to be used to resist the horizontal shear stress induced in chord members subject to large shear forces (e.g., heel joints).

7.3.8 Bearing Perpendicular to Grain.

7.3.8.1 Design Requirements.

The stress induced in compression perpendicular to grain ($f_{c\perp}$) at Reactions, joints, or from loads applied to members, shall be based on the net bearing area and shall not exceed the stresses derived using Equations E7.3-10 and E7.3-11:

$$f_{c\perp} \leq F_{c\perp}' \quad (\text{E7.3-10})$$

$$f_{c\perp} \leq E' \times (d_2/d_1)^2/20 \quad (\text{E7.3-11})$$

where:

$$f_{c\perp} = R/A_b$$

$$F_{c\perp}' = F_{c\perp}(C_t)(C_M)(C_i)(C_b)(C_{\text{plate}})$$

$F_{c\perp}$ = Reference compression perpendicular to grain design value, psi

$$E' = E(C_t)(C_M)(C_i)$$

E = Reference modulus of elasticity, psi
(Note: C_T is not applicable for E' use in Equation E7.3-11)

C_b = Bearing area factor per Section 7.3.8.2

C_{plate} = Bearing plate increase factor per Section 7.3.8.3

A_b = Net bearing area, in.² = $\ell_b B$

R = Force transferred through bearing area, lbs.

ℓ_b = Length of bearing area parallel to span of Truss, in.

B = Width of bearing area perpendicular to span of Truss, in.
(= d_2 for Truss member unless bearing area is narrower than Truss)

d_1, d_2 = Cross-sectional in-plane and out-of-plane dimensions, respectively, of the Truss member being checked, modified as permitted below (see Figures 7.3-4 through 7.3-6)

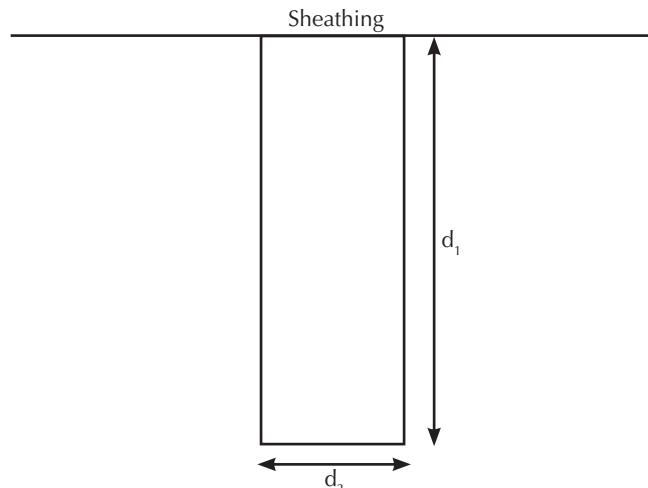


Figure 7.3-4 Cross-Sectional In-Plane and Out-of-Plane Dimensions.

7.3.8.1.1 d_2

The value of d_2 in Equation 7.3-11 shall be permitted to equal the total thickness of a multiple-ply Truss provided the individual Truss plies are fastened to each other di-

rectly over the bearing with nails, screws or other fasteners at a spacing perpendicular to grain no greater than 4 in. (102 mm) and a spacing parallel to grain no greater than 4 in. (102 mm) for bearings exceeding 4 in. (102 mm) in length.

7.3.8.1.2 d_1

The value of d_1 for use in Equation E7.3-11 shall equal the greatest value of the member depth that occurs perpendicular to the bearing surface when checking the member for the component of the bearing force that is imposed from the opposite edge of the member from the bearing surface (see Figure 7.3-5).

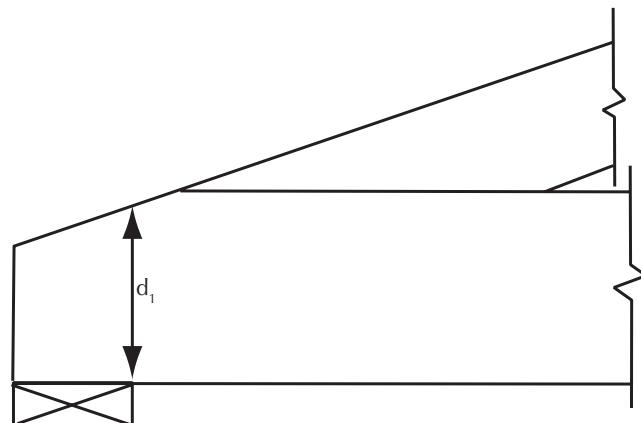


Figure 7.3-5

When such forces are imposed across multiple members, such as multiple chord members that are stacked or otherwise vertically laminated, such as at heels of Trusses, d_1 shall be set equal to the sum of the dimensions of the stacked members (see Figure 7.3-6).

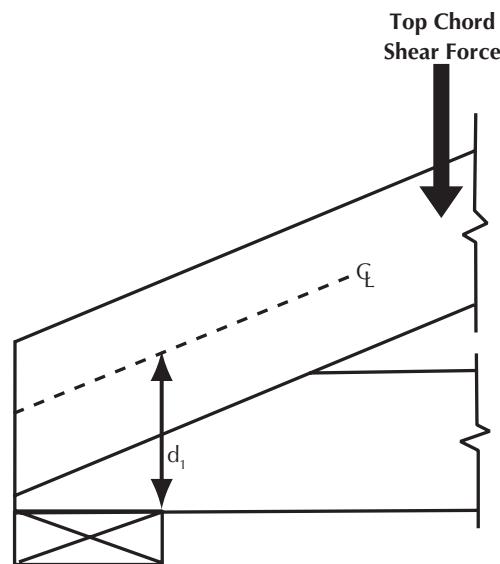


Figure 7.3-6

The value of d_1 for use in Equation E7.3-11 shall equal one half of the greatest value of the member depth that occurs perpendicular to the bearing surface when checking the member for the component of the bearing force that is imposed from loads other than those from the opposite edge of the member from the bearing surface, such as loads carried as shear forces within the member being checked from lengths of that member adjacent to the bearing surface. These values of d_1 shall be permitted to be reduced to the distance between out-of-plane bracing when such bracing is present, including the following situations:

- (a) When the member subject to the bearing stress is fastened to a reinforcing member designed to prevent the bearing member from buckling, d_1 shall be permitted to be reduced to the greatest perpendicular to grain spacing between fasteners or between the edge of the member and the nearest fastener.
- (b) When the member subject to the bearing stress is in full contact across its entire depth by confining members, such as by solid blocking, on both sides, then Equation E7.3-11 may be disregarded.

7.3.8.2 Bearing Area Factor.

For bearings less than 6 in. (152 mm) in length ($\ell_b < 6$ in.) and not nearer than 3 in. (76 mm) to the end of the Truss member, the tabular design values in compression perpendicular to grain shall be permitted to be multiplied by the bearing area factor, C_b , in addition to any other applicable modification factors.

where:

$$C_b = \frac{\ell_b + 0.375}{\ell_b}$$

This equation gives the following bearing area factors, C_b , for the indicated bearing lengths:

Length of Bearing, ℓ_b	3.5 in.	5.5 in.	6 in.+
Bearing Area Factor, C_b	1.11	1.07	1.00

7.3.8.3 Bearing Plate Increase Factor.

For compression perpendicular to grain loads bearing on a 1.5 in. (38 mm) wide face of a Truss member with a Truss plate on each adjacent normal face positioned with the nearest edge of the Truss plate no far-

ther than $\frac{1}{4}$ in. (6.5 mm) from the lumber edge common to both the 1.5 in. (38 mm) wide bearing face and the plated face, the tabular design value in compression perpendicular to grain shall be permitted to be multiplied by the bearing plate increase factor, C_{plate} , of 1.18 in addition to any other applicable modification factors.

7.3.9 Bearing Parallel to Grain.

7.3.9.1 Design Requirements.

The actual compressive stress parallel to grain, f_c , shall be based on the net bearing area and shall not exceed the allowable bearing design value parallel to grain, F_c^* . Values for F_c^* apply to end-to-end bearing of compression members provided there is adequate lateral support and the end cuts are accurately squared and parallel.

$$f_c \leq F_c^* \quad (\text{E.7.3-12})$$

where:

$$F_c^* = F_c(C_D)(C_M)(C_t)(C_i)$$

7.3.9.2 Additional Requirements.

When $f_c > 0.75 F_c^*$, bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material of sufficient stiffness to distribute the applied load. When a rigid insert is required for end-to-end bearing of compression members, it shall be equivalent to a 20 gauge metal plate or better, inserted with a snug fit between abutting ends. The rigid insert shall also be illustrated or specified on the Truss Design Drawing.

7.3.10 Bearing at an Angle to Grain.

The adjusted bearing design value at an angle, θ , to grain, as shown in Figure 7.3-7, shall be calculated as follows:

$$F_\theta' = \frac{F_c^* \times F_{c\perp}'}{(F_c^* \times \sin^2 \theta) + (F_{c\perp}' \times \cos^2 \theta)} \quad (\text{E7.3-13})$$

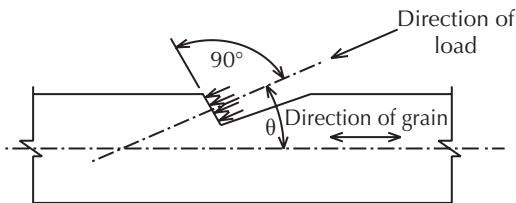


Figure 7.3-7 Load at an Angle, θ , to the Grain.

Table 7.4-1 Top Chord and Intermediate-Height Bearing Limits.
 (nominal $2 \times 3 = 38 \times 63$ mm, $2 \times 4 = 38 \times 89$ mm, $2 \times 6 = 38 \times 140$ mm)

bearing Detail Figure	Number of Top Chords	End Vertical Web	Top Chords	Maximum Allowable R (lbs)	A (in)	B (in)	C (in)
7.4-1 (a)	1	1	4x2	600	$\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$
7.4-1 (b)	2	1	4x2	1600	$\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$
7.4-1 (b)	2	1	3x2	1150	$\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$
7.4-1 (c)	2	0	4x2	1600	$\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$
7.4-1 (c)	2	0	3x2	1150	$\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$
7.4-1 (d)	2	0	4x2	1600	$\frac{1}{2}$	----	$\frac{1}{8}$
7.4-1 (d)	2	0	3x2	1150	$\frac{1}{2}$	----	$\frac{1}{8}$
7.4-1 (e)	1	0-1	4x2	1600	----	$\frac{1}{8}$	$\frac{1}{2}$
7.4-1 (f)	2	0-1	4x2	4000	----	$\frac{1}{8}$	$\frac{1}{2}$
7.4-2 (a)	1	1	2x4	1700	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{8}$
7.4-2 (a)	1	1	2x6*	2500	$\frac{1}{2}$	2	$\frac{1}{8}$
7.4-2 (b)	1	0	2x4	1700	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{8}$
7.4-2 (b)	1	0	2x6*	2500	$\frac{1}{2}$	2	$\frac{1}{8}$
7.4-2 (c)	1	0	2x4	2400	$\frac{1}{2}$	----	$\frac{1}{4}$
7.4-2 (c)	1	0	2x6*	4000	$\frac{1}{2}$	----	$\frac{1}{4}$
7.4-2 (d)	1	0	2x4	3200	----	$\frac{1}{2}$	$\frac{1}{2}$
7.4-2 (d)	1	0	2x6*	4000	----	2	$\frac{1}{2}$

* or greater

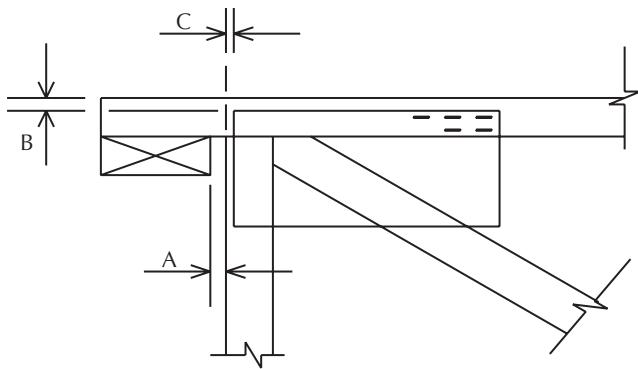


Figure 7.4-1(a)

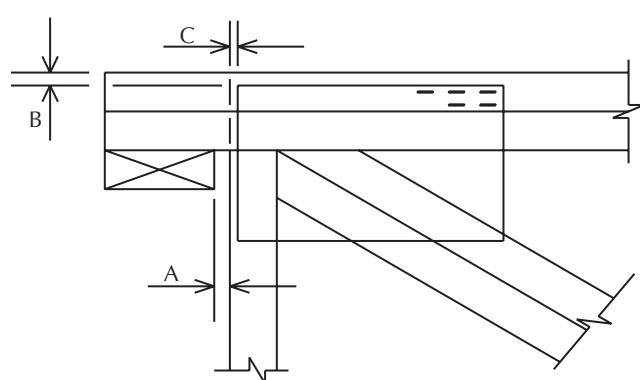


Figure 7.4-1(b)

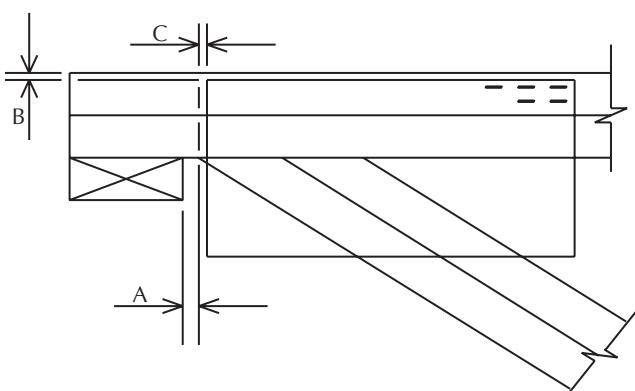


Figure 7.4-1(c)

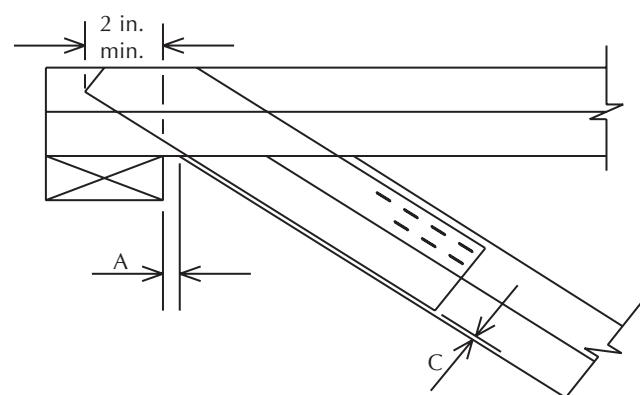


Figure 7.4-1(d)

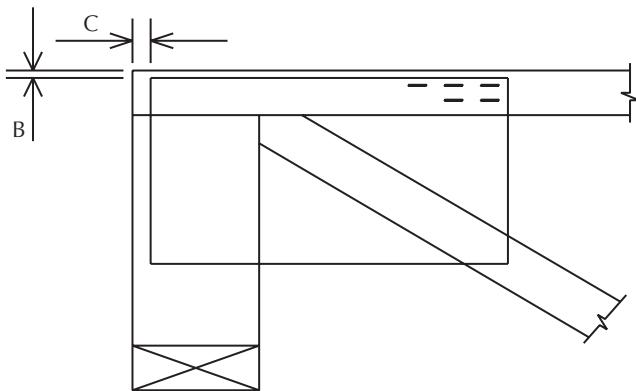


Figure 7.4-1(e)

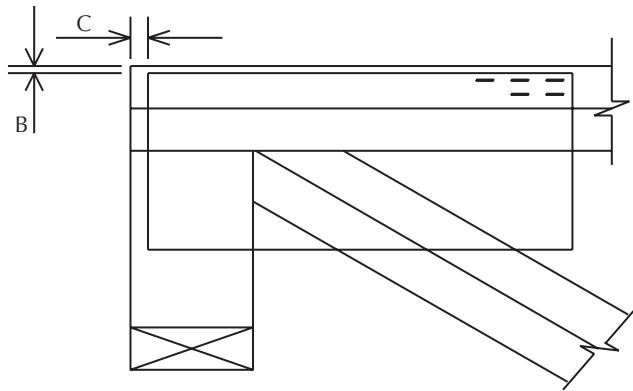


Figure 7.4-1(f)

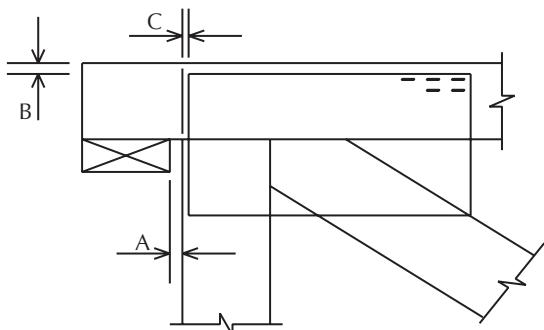


Figure 7.4-2(a)

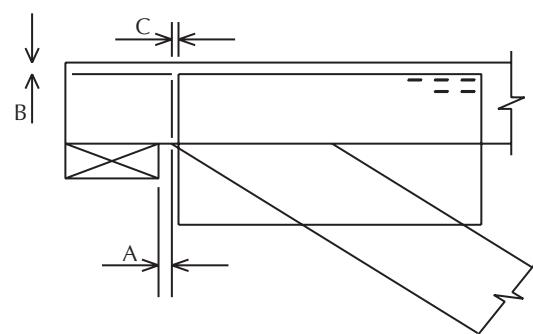


Figure 7.4-2(b)

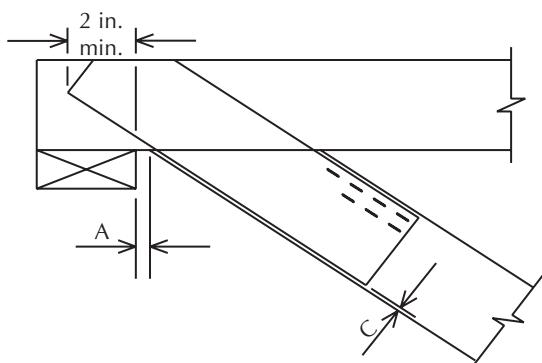


Figure 7.4-2(c)

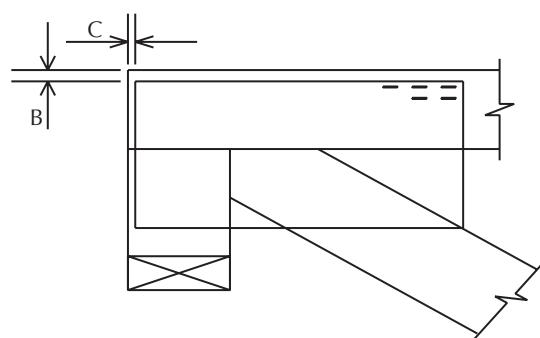


Figure 7.4-2(d)

7.4 ALLOWABLE TRUSS REACTIONS

7.4.1 Minimum Length of Bearing.

Except where supported by mechanical fasteners such as nails or screws, or framing hardware such as hangers, all bearing supports that are at the end of a Truss shall provide no less than 1.5 in. (38 mm) of bearing length on wood or metal and not less than 3 in. (76 mm) of bearing length on masonry or concrete unless approved by any Registered Design Professional.

The minimum length of the bearing shall not be less than that required by the allowable bearing stress.

7.4.2 Top Chord Bearing Parallel Chord Trusses.

Top Chord bearing parallel chord Trusses with a gap between the inside of the bearing and the first diagonal or vertical web exceeding $\frac{1}{2}$ in. (13 mm) shall be designed considering effects of shear and bending on the extended chord. In all cases involving gaps that are equal to or less than $\frac{1}{2}$ in. (13 mm) on Top Chord bearing Trusses and for intermediate-height bearing Trusses, reaction at the bearings shall not exceed the limits shown in Table 7.4-1 for the configurations shown, unless otherwise established by test or alternate analysis method.

7.5 GIRDER TRUSS DESIGN

7.5.1 General.

A girder Truss is a Truss that carries concentrated loads imposed by another Truss or other structural framing, and shall be designed in accordance with Section 7.5.

7.5.2 Girder Loading.

7.5.2.1 Application of Reactions onto Girder Trusses.

Reactions, R_i , imposed by uniformly spaced members spaced at more than 34 in. (86 cm) on center shall be applied as concentrated loads. Conversion of Reactions imposed by uniformly spaced members spaced less than or equal to 34 in. (86 cm) on center to an equivalent uniform load is not prohibited.

7.5.2.2 Applied Loads Based on the Most Critical Reaction.

Concentrated loads, P , for determination of critical stresses, shall be based on the most critical calculated reaction imposed by a structural member and/or other designated point load.

7.5.2.3 Load Continuous Across All Plies.

If a structural member imposing a load on a girder is continuous across all plies, the load shall be considered to be equally distributed to all plies.

7.5.2.4 Maximum Plies.

The maximum number of plies shall be five, if the structural members imposing a load are attached to one side of the girder, or six, if the structural members imposing a load are attached to both sides of the girder.

7.5.2.5 Location of the Applied Load.

The design load shall be applied on the structural member that the imposing load on the girder is framed into.

7.5.3 Member Design.

7.5.3.1 General.

Design of girder Truss members shall be in accordance with Section 7.3, except as modified herein.

7.5.3.2 Design for Tension Perpendicular to Grain Forces.

In addition to meeting the requirements of Section 6.3.3, any connection to a girder Truss that induces tension perpendicular to the grain shall be designed and detailed to limit the cross grain tension forces in accordance with the following subsections.

7.5.3.2.1 Extent of Connection.

Where a connection is made with nails or bolts, or other approved fasteners, the connection shall extend past the centerline of the carrying member a minimum distance, y (see Figure 7.5-1), when it meets the following conditions:

For connections greater than or equal to a distance of $5 \times d_i$ from the end of the member when P_{\perp} , exceeds 800 lbs.:

$$y > c [1 - (F_v' 2A_e) / (3P_{\perp})]^{0.5} \quad (\text{E7.5-1})$$

For connections within a distance of $5 \times d_i$ from the end of the member when P_{\perp} , exceeds 400 lbs.:

$$y > c [1 - (F_v' 2A_e) / (3P_{\perp}) \times (d_e/d)^2]^{0.5} \quad (\text{E7.5-2})$$

If Equations E7.5-1 or E7.5-2 result in an imaginary number (quantity in brackets is negative), then $y = 0$.

where:

$$A_e = d_2 \times d_e$$

d_e = Effective depth of chord member

= Dimension from loaded edge of chord to opposite edge of Metal Connector Plate

For connections with nails, bolts, or other approved fasteners to a multiple ply girder Truss, the maximum number of plies that shall be used in this area calculation is two.

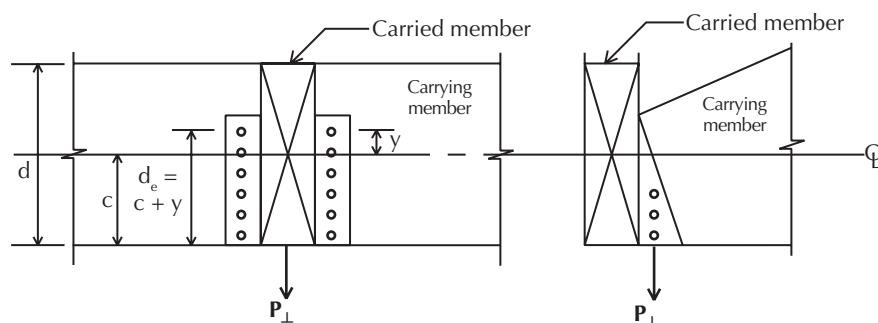


Figure 7.5-1 Distance (y) to Extend Connection Past Centerline to Enable Carrying Member to Resist Tension Perpendicular-to-Grain Forces.

7.5.3.2.2 Alternate Extent of Connection.

Equation E7.5-1 shall be permitted to be solved for an alternate distance, y' , as shown in Figures 7.5-2 (a) through (c), if any of the following cases are met:

- (a) The girder connection occurs at a Truss joint, and the top most fastener of the connection is located within the Metal Connector Plate Area [see Figure 7.5-2(a)];

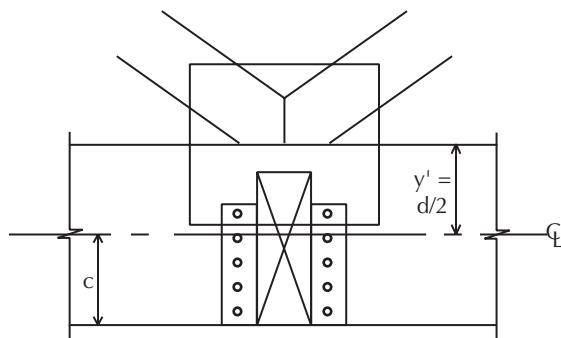


Figure 7.5-2(a)

- (b) Two Metal Connector Plates, one on each side of the girder connection, are placed no more than 12 in. (305 mm) on either side of the connection, and extend below the centerline of the member and top most fastener of the connection [see Figure 7.5-2(b)]; or

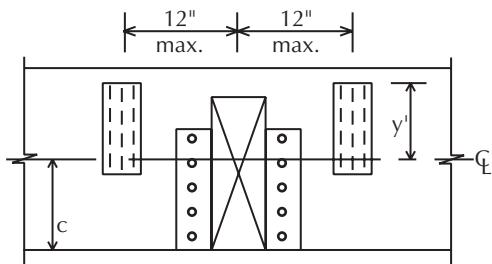


Figure 7.5-2(b)

- (c) A Metal Connector Plate is placed directly under the girder connection, extends below the top most fastener of the connection, and is wider than the connection width [see Figure 7.5-2(c)].

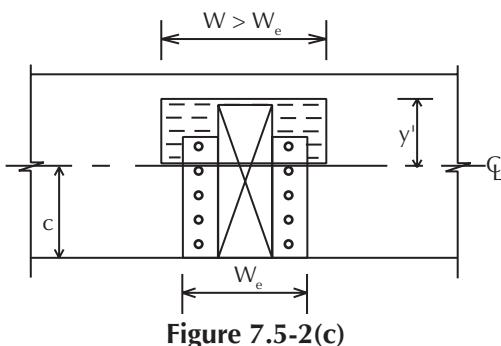


Figure 7.5-2(c)

7.5.3.2.3 Effective Depth Greater Than 85% of Member Depth.

Any connection where d_e (see Figure 7.5-1) is at least 85% of the member depth (i.e., $d_e = c + y \geq 0.85d$) shall be considered to meet the requirements of Section 7.5.3.2.1.

7.5.3.3 Design for Tension Perpendicular to Grain.

Any joint with connector plates in which the net force component perpendicular to the member induces tension perpendicular to the grain, shall require a Metal Connector Plate that extends past the centerline of the member a minimum distance, y (see Figure 7.5-3), when it meets the following conditions:

For connections greater than or equal to a distance of $5 \times d_l$ from the end of the member when P_{\perp} exceeds 800 lbs. tension:

$$y > c [1 - (F_v^2 A_e) / (3P_{\perp})]^{0.5} \quad (\text{E7.5-3})$$

For connections within a distance of $5 \times d_l$ from the end of the member when P_{\perp} exceeds 400 lbs. tension:

$$y > c [1 - (F_v^2 A_e) / (3P_{\perp}) \times (d_e/d)^2]^{0.5} \quad (\text{E7.5-4})$$

If Equations E7.5-3 or E7.5-4 result in an imaginary number (quantity in brackets is negative), then $y = 0$

where:

$$A_e = d_2 \times d_e$$

d_e = Effective depth of chord member

= Dimension from loaded edge of chord to opposite edge of Metal Connector Plate

7.5.3.3.1 Effective Depth Greater Than 85% of Member Depth.

Any connection where d_e (see Figure 7.5-3) is at least 85% of the member depth (i.e., $d_e = c + y \geq 0.85d$) shall be considered to meet the requirements of Section 7.5.3.3.

7.5.3.4 Reduced Section.

The design net section of a member shall be calculated to account for a reduction in the gross cross sectional area of a member due to, but not limited to, drilling or notching.

7.5.3.5 Torsion.

Girders subject to Truss torsion, induced by structural members framing into them, shall be adequately laterally braced to prevent excessive displacement due to the applied torque, T_G (see Figure 7.5-4).

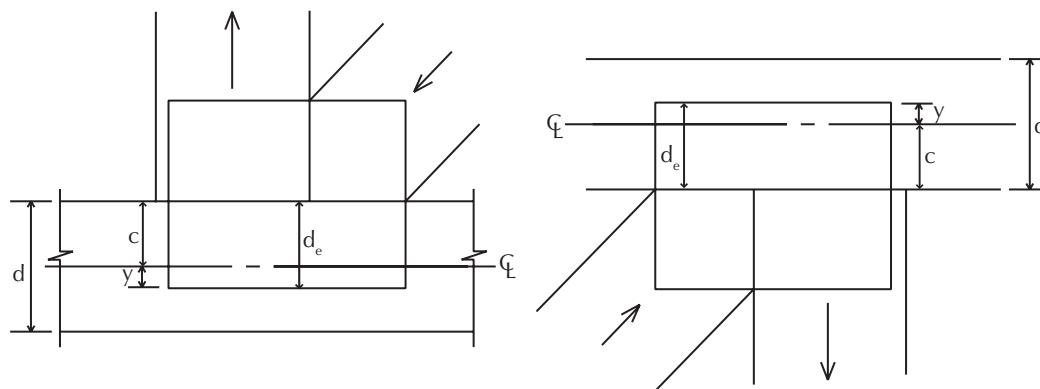


Figure 7.5-3 Distance to Extend Metal Connector Plate Past Centerline to Resist Tension Perpendicular-To-Grain Forces.

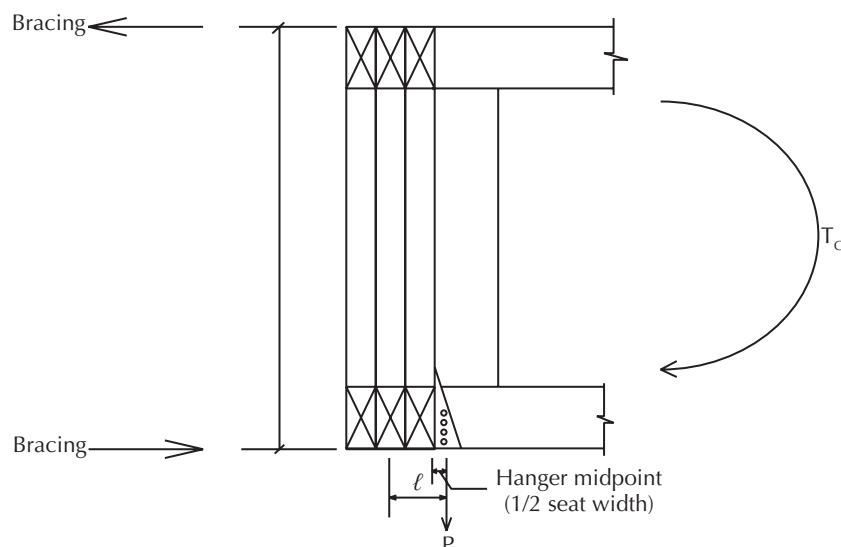


Figure 7.5-4 Secondary Torsional Stresses, T_G , Induced by Structural Members Framing into Girders.

7.5.3.6 Structural Framing Connections.

Members or structural framing connections shall be so proportioned to resist compression perpendicular to grain stresses induced at the framing connection.

7.5.4 Truss-to-Truss Girder Connections.

The connection between a metal-plate-connected wood Truss and a metal-plate-connected girder Truss shall either be a specified commercially available structural framing connection or a specially designed structural framing connection. The connection shall meet the applicable requirements of Section 7.5.3.

7.5.5 Ply-to-Ply Connections.

7.5.5.1 Connection of Members.

Girders with up to three plies shall be connected by nailing, bolting, or other approved fasteners in accordance with an approved design criteria. Girders with four or more plies, and having structural members imposing a

load on one side of the girder, shall be connected by bolting, a combination of nailing and bolting, or by other approved fasteners. Either nails, bolts, or other approved fasteners shall be designed to transmit 100 percent of the imposed load from one side; the values for more than one type of approved fastener in the same connection shall not be combined. Webs in girders of any number of plies shall be permitted to be joined with nails.

7.5.5.2 Design Load.

Connections shall be designed to transmit load from ply to ply in accordance with the ply-to-ply load distribution assumed in the design of the girder. Connections shall be adequate to carry the cumulative load of the remaining plies.

7.5.5.3 Design for Withdrawal Load.

Connections between the individual plies of a member shall be designed for withdrawal loads equal to two per-

cent of the axial compression force in each ply so connected, for each unbraced length of the member, or these connections shall comply with the provisions of the *ANSI/AF&PA NDS* for use of K_f when used per Section 7.3.2. For the purposes of this section, for members braced by sheathing, the unbraced length over which the fasteners carrying this withdrawal load are distributed shall be permitted to be 10 times the cross-section dimension parallel to the dimension in which the sheathing prevents buckling.

7.5.5.4 Nail Spacing.

Nail spacing shall be the smaller of the two determined from Sections 7.5.5.2 and 7.5.5.3, but in no case shall the spacing exceed 12 in. (305 mm) on center. Nailing patterns shall be specified on the Truss Design Drawing.

7.5.5.5 Bolt Spacing.

Bolt spacing shall be the smaller of the two determined from Sections 7.5.5.2 and 7.5.5.3, but in no case shall the spacing exceed 24 in. (610 mm) on center unless the bolts are used solely for reasons other than to carry loads addressed by Sections 7.5.5.2 and 7.5.5.3. bolts shall have a diameter no less than $\frac{1}{2}$ in. (13 mm) and no greater than 1 in. (25 mm).

7.6 DEFLECTION

7.6.1 Method of Calculation.

Live load deflection shall be based on full live load. Time dependent deformation under long term loading shall be accounted for in total deflection calculations, as follows.

$$\Delta_T = K_{cr} \times \Delta_{LT} + \Delta_{ST}$$

where:

K_{cr} = Creep factor
 ≥ 1.5 for seasoned lumber used in dry service conditions
 ≥ 2.0 for green lumber or for wet service conditions

Δ_T = Total deflection

Δ_{LT} = Immediate deflection due to the long term component of the design load (deflection due to a sustained load, typically dead load)

Δ_{ST} = deflection due to short term or normal component of the design load (deflection due to transient loads, typically live load)

Table 7.6-1 Deflection Limits for Non-Cantilevered Portions of Trusses.⁴

Values given in the table are divisors that are applied to the clear span length, L_s , to establish a deflection limit (limit = $L_s / \text{specified value}$).

Member	Deflection due to LL only	Deflection due to Total Load (LL + DL)
Roof Truss supporting plaster ¹	360	240
Roof Truss supporting drywall ¹	240	180
Roof Truss not supporting ceilings ¹	180	120
Floor Trusses ^{2,4} (see footnotes for Trusses supporting ceramic tile)	360 or 480 ³	240
Top Chord panel ⁵	180	120 (600 ⁶)
Habitable spaces in Trusses ⁷	360	

¹ Roofs not having sufficient slope or camber to assure adequate drainage shall be investigated for ponding.

² Certain floor coverings require more restrictive deflection criteria. For ceramic tile, Truss spacing and appropriate dead load for the installation method, and other aspects of design per *ANSI/AI08/AI18/A136* shall be such that the system passes the requirements of the Building Designer per Chapter 2 of this Standard.

³ Floor Trusses with ceilings attached that meet L/480 criteria shall not require strongbacks to meet deflection criteria.

⁴ Cantilevered and overhang portions of Trusses are subject to deflection limits using the values shown above applied to twice the length of the cantilever, L_k .

⁵ Span length for Top Chord panel limits shall be the panel length.

⁶ Where required by *ACI530/ASCE5/TMS402* for Trusses used as a beam or lintel providing support of vertical masonry veneer.

⁷ Limit is for panel deflection of the loaded panel when loaded with 30 psf (14.4 kPa) or greater of live load.

7.6.2 Vertical Deflection Limits.

7.6.2.1 Designated Limits.

Truss vertical deflection, as determined by structural analysis, shall be limited to the proportions to span length as shown in Table 7.6-1, unless otherwise limited by regulations in the local Jurisdiction.

7.6.2.2 Deflection Using Beam Formulas.

Vertical deflection shall be permitted to be determined using beam formulas as shown in Section 7.6.2.3. When chord lumber having different E values is used, the deflection calculations shall be based on the average E value.

7.6.2.3 Deflection Calculation for Parallel Chord Trusses.

For uniformly loaded, simply supported parallel chord Trusses, deflection shall be permitted to be calculated as follows:

$$D = \frac{1.33K_b}{EI_e} \times (1 + 0.015x) \quad (\text{E7.6-1})$$

where:

D = deflection at centerline of Truss (in. or mm)

K_b = Load and span effect constant

$$= \frac{(5)(w)(L_s)^4}{384}$$

w = Uniform load (lb/in. or N/mm)

L_s = Clear span (in. or mm)

I_e = Moment of inertia of the cross sectional areas of the Top and Bottom Chords about the neutral axis (N.A.) of the Truss (see Figure 7.6-1)

E = Average modulus of elasticity of Chord lumber (psi or N/mm²)

x = Offset of centerline of opening from centerline of Truss (in. or mm) (see Figure 7.6-2); Not to exceed 15 in. (381 mm) for this deflection formula

7.6.2.4 Strongbacking.

When specified, strongbacks shall comply with the following installation criteria:

- (a) Strongbacks shall, as a minimum, be 2x6 (nominal), and shall be attached to each Truss with

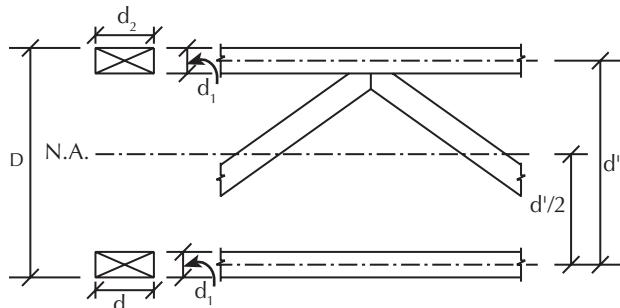


Figure 7.6-1 Dimensions Used to Determine I_e .

a minimum of three 10d (0.131 in. (3.33 mm) diameter x 3 in. (76 mm) long) nails each to a vertical Web member, or shall be similarly attached to a vertical scab secured at Top and Bottom Chords of each Truss with a minimum of two 10d (0.131 in. (3.33 mm) diameter x 3 in. (76 mm) long) nails into each Chord.

- (b) Strongback cross-section shall be oriented vertically and shall be continuous. When required to be cut, removed, or modified to allow the installation of mechanical and/or plumbing lines, the continuity at adjoining floor sections shall be maintained, and the methods of maintaining continuity shall be specified by the designer specifying the strongbacks.
- (c) Cross bridging shall be permitted as an alternative to strongbacking, as determined by the designer specifying the strongbacks.
- (d) Spacing between multiple strongbacks shall not exceed 10 ft. (3 m) for floor assemblies.
 - (1) When strongbacking is specified to control vibration or protect brittle floor surfaces (such as ceramic tile and natural stone), the Contractor shall locate strongbacking as stipulated on the Truss Design Drawing and as required by the floor surfacing specifications, unless otherwise specified.
 - (2) Strongbacking required to control deflections shall be in accordance with the following criteria unless otherwise specified. When deflection due to live load exceeds 0.67 in. (17 mm), one strongback shall be placed near the centerline of the Truss clear span. When live load deflection exceeds 0.85 in. (22 mm), two strongbacks shall be placed near the centerline of the Truss clear span, or near the third points of the Truss clear span.

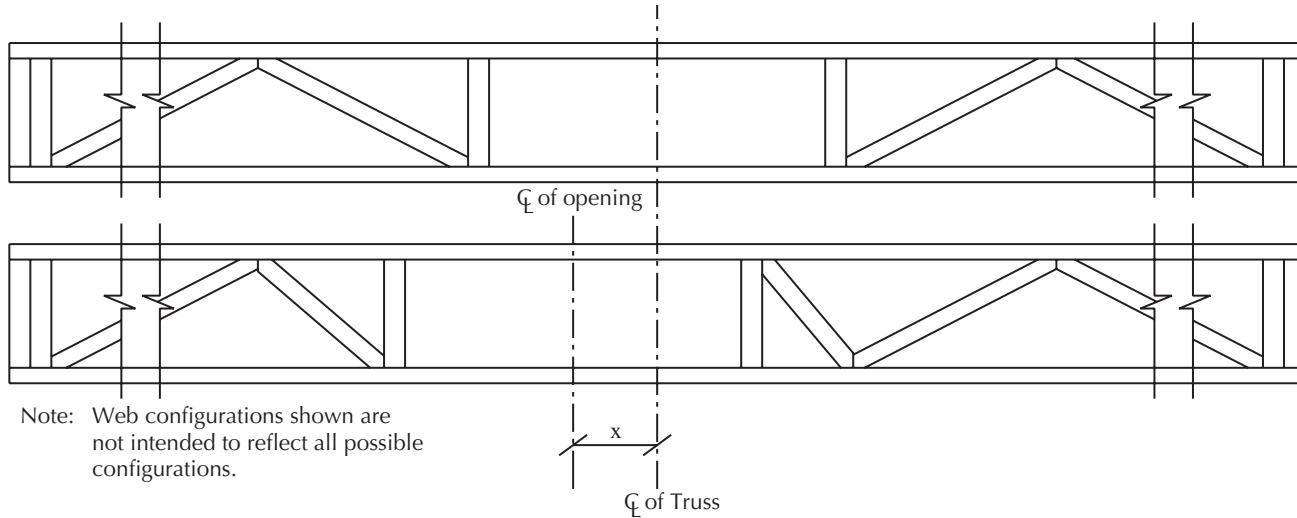


Figure 7.6-2 Measurement of Opening Offset.

7.6.3 Horizontal Deflection Limits.

In lieu of specific provisions for lateral movement of Trusses and supports, total horizontal deflection at the Reactions for the design of Trusses shall be limited to 1.25 in. (32 mm) due to total load, and 0.75 in. (19 mm) due to live load. The supporting structure and Truss-to-wall connection shall be designed accordingly.

CHAPTER 8

METAL CONNECTOR PLATE JOINT DESIGN

8.1 SCOPE

Each Metal Connector Plate shall be designed in accordance with Chapter 8 of this Standard, to resist all forces and bending moments determined in accordance with Section 6.2.

8.2 MINIMUM AXIAL DESIGN FORCES

The minimum axial design force for any Wood Member to be used when designing Metal Connector Plates on Trusses with overall lengths exceeding 16 ft. (5 m) shall not be less than 375 lbs. (1670 N).

8.3 LATERAL RESISTANCE

8.3.1 General.

Each Metal Connector Plate shall be designed to transfer the required load without exceeding the allowable load per tooth, or per unit area, based on the species, the orientation of Teeth relative to the load, and the direction of load relative to grain.

8.3.2 Adjustments.

The allowable lateral resistance value, V_{LR}' , determined in Sections 5.2 and 8.3.3.2 for a given lumber type, plate type, plate/wood orientation, and embedment method, shall be multiplied by all applicable adjustment factors specified in Section 6.4. In addition, heel joints shall be multiplied by the reduction factor specified in Section 8.3.2.2.

8.3.2.1 Exclusion of Area under the Net Area Method.

For Metal Connector Plates rated by the Net Area Method (see Section 5.2.6.2), the plate area tributary to those Teeth within $\frac{1}{2}$ in. (13 mm) of the ends of members, measured parallel to the grain, and within $\frac{1}{4}$ in. (6 mm) of the edges of members, measured perpendicular to grain, shall be excluded when determining the Metal Connector Plate coverage for each member of a joint (see Figure 8.3-1). Alternately, if distances greater than those shown in Figure 8.3-1 are used to establish the allowable Net Area design values in Section 5.2.6.2, those End and Edge Distances used shall be excluded from the Metal Connector Plate coverage area.

8.3.2.2 Additional Consideration at Heel Joints.

To allow for moment effects, Metal Connector Plates at heel joints (see Figure 8.3-2) shall be designed to have sufficient capability to withstand the direct axial force of the Top and Bottom Chords using lateral resistance design values multiplied by the following reduction factor, H_R :

$$H_R = 0.85 - 0.05(12 \tan \theta - 2.0) \quad (\text{E8.3-1})$$

$$0.65 \leq H_R \leq 0.85$$

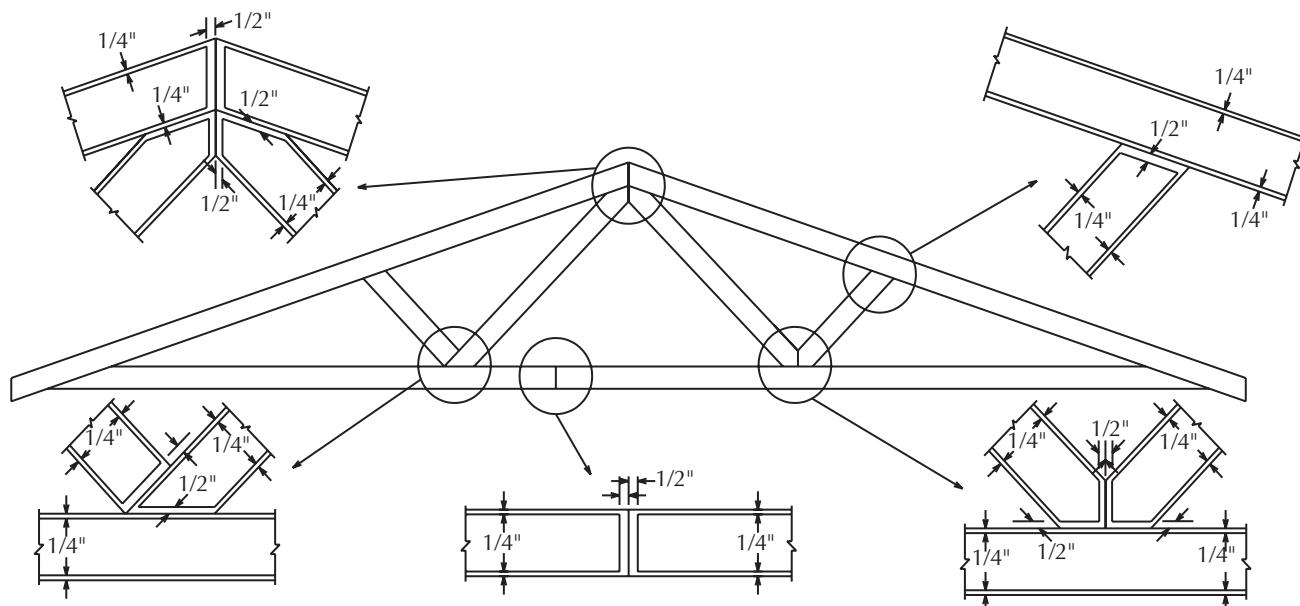


Figure 8.3-1 End & Edge Distance Requirements for the Net Area Method.

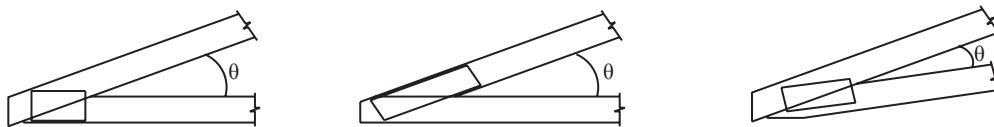


Figure 8.3-2 Heel Joints for which the Reduction Factor, H_R , Applies.

For conditions with Top Chord slopes of 12/12 or greater and all non-heel joints,

$$H_R = 1.0$$

The End and Edge Distance requirements of Section 8.3.2.1 shall not apply to the design of Metal Connector Plates at the heel joints.

8.3.3 Design.

Design of Truss joints for lateral resistance of Metal Connector Plates shall be as follows:

$$A_p = P/V_{LR}' \quad (E8.3-2)$$

where:

$$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) \text{ [psi/pair (N/mm}^2/\text{pair)]}$$

A_p = Minimum required Metal Connector Plate contact area for each member, total area for one face (in.² or mm²)

P = Force in Wood Member (lbs. or N)

V_{LR} = Lateral resistance design value per Metal Connector Plate unit, based on a plate on each face [psi/pair (N/mm²/pair)]

8.3.3.1 Number of Teeth Reported for Inspection.

The required number of Teeth, N, that shall be reported for inspection of plate areas, shall be determined as follows:

$$N = MP/V_{LR}^* \quad (E8.3-3)$$

where:

$$V_{LR}^* = V_{LR}(C_D)(C_M)(H_R) \text{ [psi/pair (N/mm}^2/\text{pair)]}$$

N = Required number of Teeth per face

M = Total tooth density based on a plate on each face (Teeth/sq. in. or Teeth/sq. mm)

8.3.3.2 Load Applied at an Angle.

When load is applied at an angle other than 0 or 90 degrees with respect to lumber grain, the allowable lateral resistance design values shall be as follows:

$V_{LRA\theta}$ = allowable value for Metal Connector Plates loaded at an angle, θ , to the grain with the plate axis (tooth slots) parallel to the load [see Figure 8.3-3(a)],

$$= \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2 \theta) + (V_{LRAE} \times \cos^2 \theta)} \quad (E8.3-4)$$

$V_{LRE\theta}$ = allowable value for Metal Connector Plates loaded at an angle, θ , to the grain with the plate axis (tooth slots) perpendicular to the load [see Figure 8.3-3(b)],

$$= \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2 \theta) + (V_{LREE} \times \cos^2 \theta)} \quad (E8.3-5)$$

When the plate axis is oriented at an angle, α , from the load direction [see Figure 8.3-3(c)], allowable Design Values shall be determined by linear interpolation between the values $V_{LRA\theta}$ and $V_{LRE\theta}$ as follows:

V_{LR}' = allowable value for Metal Connector Plates adjusted for plate and grain orientation,

$$= \frac{((90 - \alpha) \times V_{LRA\theta}) + ((\alpha) \times V_{LRE\theta})}{90} \quad (E8.3-6)$$

8.3.3.3 Metal Connector Plates Resisting Member Compressive Forces.

Metal Connector Plates resisting member compressive forces shall be sized to provide lateral resistance equal to the vectorial sum of the reduced component force(s) normal to the Wood Member interface and 100 percent of the component force(s) parallel to the Wood Member interface. Additionally, Wood Members shall be designed to transmit 100 percent of the remaining component force across the interface in wood-to-wood bearing.

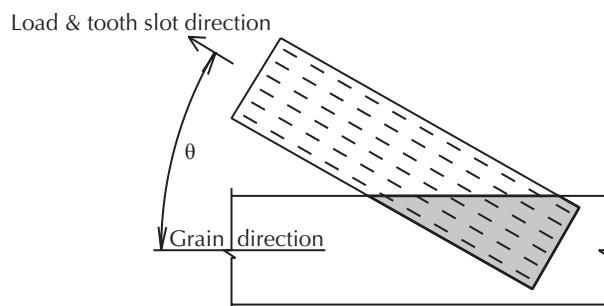


Figure 8.3-3 (a)

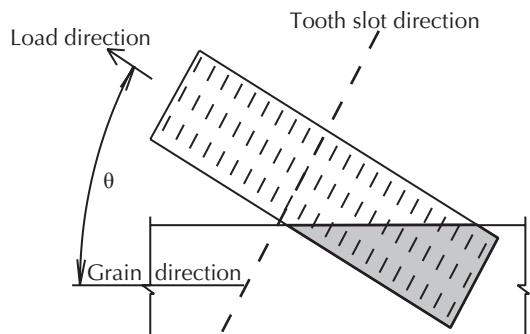


Figure 8.3-3 (b)

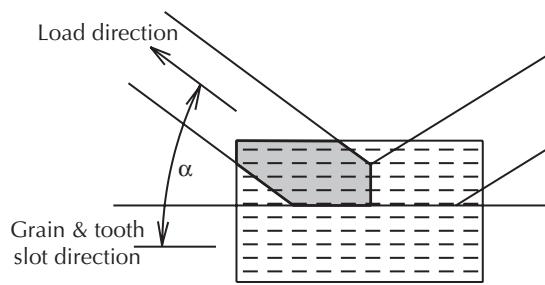


Figure 8.3-3 (c)

$$A_p = P' / V_{LR}' \quad (\text{E8.3-7})$$

where:

$$P' = \sqrt{(P_{ip})^2 + (P_{in} \times C_R)^2}$$

A_p = Minimum required Metal Connector Plate contact area for each member (in.^2 or mm^2)

P' = Resultant compressive force used for determination of minimum required Metal Connector Plate contact area (lbs. or N)

V_{LR} = Allowable lateral resistance value of Metal Connector Plate [psi/pair (N/mm^2 /pair)]

C_R = Reduction factor for compression force component across the joint interface for Metal Connector Plate design: $0 \leq C_R \leq 1.0$

P_{in} = Compression force component of the Wood Member under investigation normal to the joint interface (lbs. or N)

P_{ip} = Compression force component of the Wood Member under investigation parallel to the Wood Member interface (lbs. or N)

8.3.3.3.1 No Wood Member Present.

If there is no Wood Member present to provide force transfer through wood-to-wood bearing, the Metal Connector

Plate shall be designed for the full 100 percent component.

8.3.3.3.2 Adjacent Members.

Metal Connector Plate areas of adjacent members shall be designed to resist this transfer of forces.

8.4 TENSION

8.4.1 Account for Orientation.

Each Metal Connector Plate shall be designed for tension based on the orientation of the Metal Connector Plate relative to the direction of the load.

8.4.2 Method.

The net section of Metal Connector Plates for all tension joints shall be designed using the allowable tensile stress of the metal, adjusted by the Metal Connector Plate tensile Effectiveness Ratio as determined in Section 5.4. Allowable design values in tension (pounds per unit length), for a pair of Metal Connector Plates, one on each face of the joint, shall be determined by the following formula:

$$V_t = 2(R_t \times F_{st} \times t_l) \quad (\text{E8.4-1})$$

8.4.3 Required Cross Section.

The required cross section of the Metal Connector Plate for tension shall be determined as shown in Equation E8.4-2. For chord splices with plates that extend past the chord member, W_p shall not exceed W'_p as specified in Sections 8.4.3.1 and 8.4.3.2.

$$W_p = P_t / V_t \quad (\text{E8.4-2})$$

where:

W_p = Gross width of Metal Connector Plate measured parallel to the joint line (in. or mm)

P_t = Axial tensile force in the Wood Member (lbs. or N)

V_t = Allowable design value in tension for a pair of Metal Connector Plates, one on each face of the joint (pli or N/mm)

8.4.3.1 Maximum Effective Width at Mid-Panel Tension Splices.

The maximum effective width of Metal Connector Plates for all mid-panel tension chord splices with Metal Connector Plates that extend past the chord member (see Figure 8.4-1) shall be:

$$W'_p = d - d_{le} + x_{max} \quad (\text{E8.4-3})$$

where:

$$x_{max} = 0.25L_{pc}$$

d = Depth of the tension chord (in. or mm)

d_{le} = Distance from the outer edge of the chord to the outer edge of the Metal Connector Plate joining the chord-splice (in. or mm)

L_{pc} = Smaller length of Metal Connector Plate in chord on either side of the splice (in. or mm)

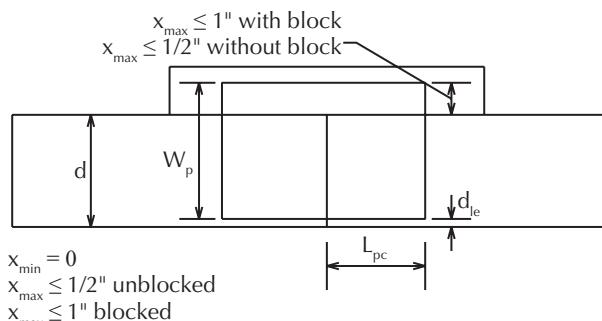


Figure 8.4-1

8.4.3.2 Maximum Effective Width at Panel Point Tension Splices.

The maximum effective width of Metal Connector Plates for all panel point tension chord splices with Metal Connector Plates that extend past the chord member shall be:

$$W'_p = d - d_{le} + 1.5 \quad (\text{E8.4-4})$$

where d and d_{le} are as defined in Section 8.4.3.1 and are in units of inches.

8.4.3.3 Connector Plate Designed for Moment.

When the design of a Truss chord assumes the chord is connected to a joint with a pinned connection, so that no moment is transferred to the joint, and when the Metal Connector Plate at the joint is located so that the center of the steel cross-section on the chord is not on the centerline of the chord, the joint shall be designed in accordance with Section 8.7 for a moment applied simultaneously to the axial force. The moment shall be equal to the axial force normal to the joint times the eccentricity where the eccentricity shall be equal to the distance measured along the joint between the center of the steel cross-section on the chord and the chord centerline (see Figure 8.4-2).

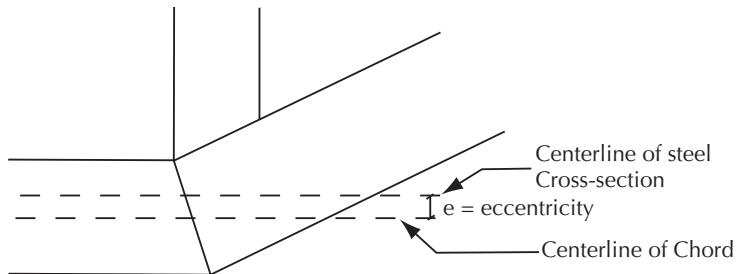


Figure 8.4-2

8.5 SHEAR

8.5.1 General.

Each Metal Connector Plate shall have sufficient shear capacity based on orientation of the Metal Connector Plate relative to all possible lines of shear.

8.5.2 Method.

The net shear section of Metal Connector Plates, for all heel joints and other joints involving shear, shall be designed using the Metal Connector Plate shear design value, adjusted by the Metal Connector Plate shear efficiency as determined in Section 5.3. Allowable Design Values in shear (pounds per unit length or kPa per unit length), for a pair of Metal Connector Plates, one on each face of the joint, shall be determined by the following formula:

$$V_s = 2(R_s \times F_{vs} \times t_l) \quad (\text{E8.5-1})$$

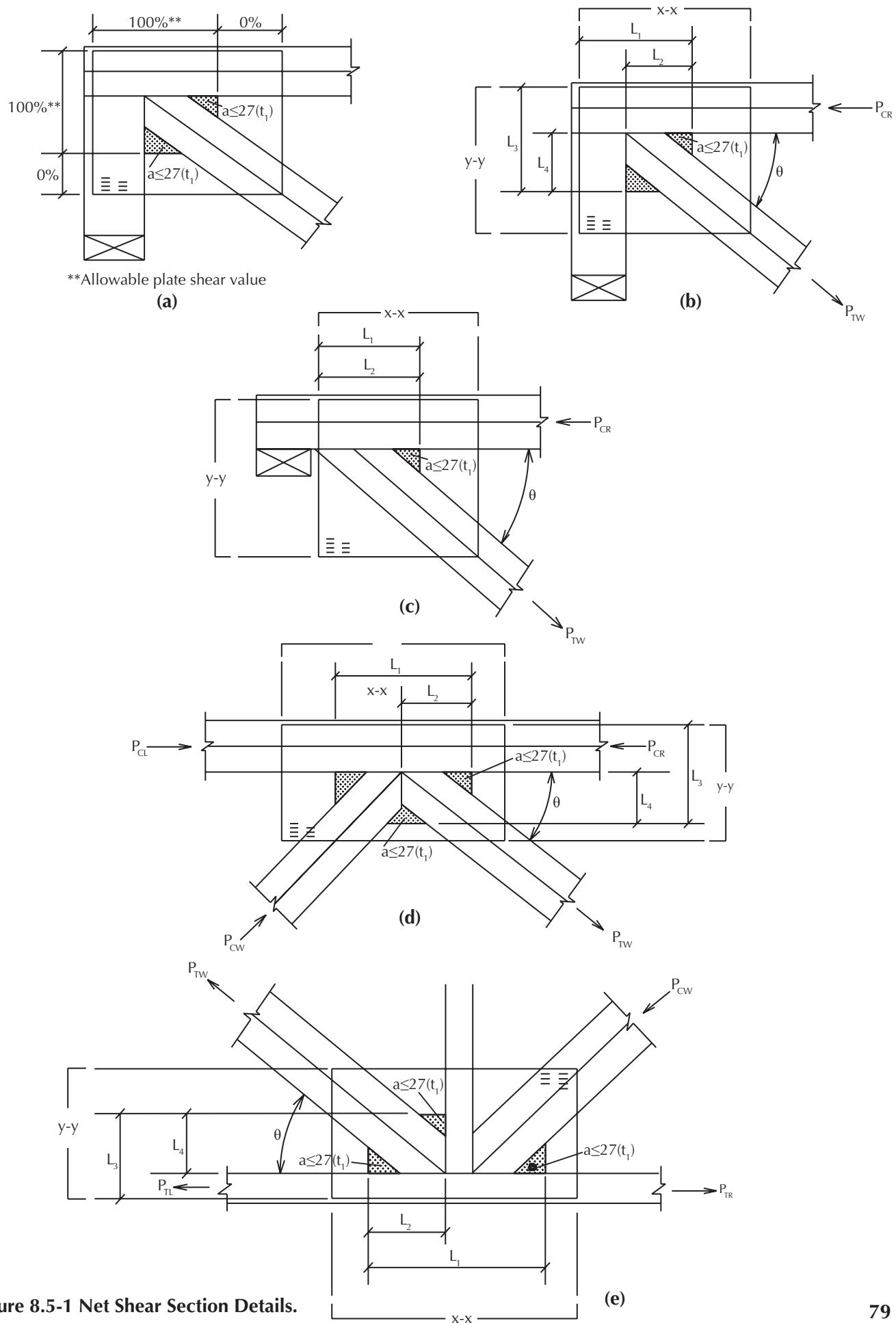


Figure 8.5-1 Net Shear Section Details.

8.5.3 Required Cross Section.

The required cross section of the Metal Connector Plate for shear at each joint shall be determined as follows:

$$\ell = P_s / V_s \quad (\text{E8.5-2})$$

where:

ℓ = Gross plate dimension parallel to the joint line as limited by Section 8.5.6 (in. or mm)

P_s = Force parallel to the joint across the shear plane (lbs. or N)

V_s = Allowable design value in shear for a pair of Metal Connector Plates, one on each face of the joint (pli or N/mm)

8.5.4 Shear Values Parallel and Perpendicular to the Major Axis.

The net shear section for Metal Connector Plates in joints subject to shear shall be designed using Metal Connector Plate shear values parallel and perpendicular to the major axis. The major axis is parallel to the chord. For Metal Connector Plate shear, Equations E8.5-3 and E8.5-4 shall apply for connections detailed in Figures 8.5-1(a)-(e). Alternate design calculations confirmed by test shall be permitted to be used.

At x-x:

$$L_1(V_{s\parallel}) \geq |P_L - P_R| \quad (\text{E8.5-3})$$

At y-y:

$$L_3(V_{s\perp}) \geq P_{TW} \sin\theta \quad (\text{E8.5-4})$$

where:

P_L = Axial force parallel to the grain of the chord to the left of the panel point (lbs. or N)

P_R = Axial force parallel to the grain of the chord to the right of the panel point (lbs. or N)

P_{TW} = Axial force in tension parallel to the grain of the Web (lbs. or N)

$V_{s\parallel}$ = Capacity of a pair of Metal Connector Plates to resist shear along the major axis (pli or N/mm)

$V_{s\perp}$ = Capacity of a pair of Metal Connector Plates to resist shear at 90 degrees to the major axis (pli or N/mm)

8.5.5 Combination of Shear Plates and Tension Plates.

Plates that transfer load primarily in shear shall not be used in combination with separate plates that transfer load primarily in tension on the same joint.

8.5.6 Prevention of Shear Buckling.

To prevent the occurrence of shear buckling of unsupported plate areas, the plate cross-section of unsupported plate areas shall be considered to have a shear efficiency of zero, except for those portions of the cross-section bordering a triangular area, a , between two Wood Members (see Figure 8.5-1), not to exceed $27 \times t_l$, when ' a ' is in sq. in. and t_l is in in., or 686 times t_l , when ' a ' is in sq. mm, and t_l is in mm.

8.6 COMBINED SHEAR-TENSION

For combined shear and tension in the Metal Connector Plate contact area of Webs, Equation E8.6-1 shall apply for connections detailed in Figures 8.5-1(a)-(e). Alternate design calculations that are confirmed by test shall be permitted to be used.

$$(X_{st} \times L_2) + (Y_{st} \times L_4) > P_{TW} \quad (\text{E8.6-1})$$

where:

X_{st} = Combined shear/tension value for the horizontal projection of a pair of Metal Connector Plates: $V_{s\parallel} + (\theta/90)(V_{t\perp} - V_{s\parallel})$

Y_{st} = Combined shear/tension value for the vertical projection of a pair of Metal Connector Plates: $V_{t\parallel} + (\theta/90)(V_{t\perp} - V_{t\parallel})$

$V_{t\parallel}$ = Tensile capacity of the Metal Connector Plate section where the load is applied parallel to the major axis

$V_{t\perp}$ = Tensile capacity of Metal Connector Plate section where the load is applied at 90 degrees to the major axis

8.7 COMBINED FLEXURE AND AXIAL LOADING

8.7.1 Design of Steel Section for Effect of Moment.

The moment applied to a Metal Connector Plate used in chord splices (including perimeter joints with changes in slope, i.e., peaks, hips, and scissors centerline joints) shall not exceed the moment capacity defined as follows:

$$M_a = C_m \{T_1(W_p + y + z - d_1') + T_2(4W_p + 2y + 4z - 3d_1') / 3 + C_s(d_1' - z - y) + C_w(d_1' - y)\} / 5 \quad (\text{E8.7-1})$$

where:

C_m = For splices designed using member forces resulting from a structural analysis that produces moment without consideration of interaction of axial compression and transverse deflection on moment (P - δ effect) and when the joint carrying moment is simultaneously subject to a compression force: $1 - (x/L)(f_c/F_{cEx})$. For all other situations: 1.0

x = Distance between splice and nearest panel point (in.)

L = Length of panel in which splice is located (in.)

f_c, F_{cEx} = Stresses per Section 7.3.5.1 for Wood Members adjacent to splice (psi)

y = Distance to neutral axis from wood edge with moment-induced compression stress (in.):

$$\{t_l R_t \{F_y(1.8z + W_p) + F_u(W_p + z)\} - 2P_t\} / (d_2 C + t_l R_t (1.8F_y + F_u))$$

M_a = Maximum allowable moment in plane parallel to Metal Connector Plate surface (in. lbs.) acting in direction causing compression on wood edge used to reference plate location (see variable z below). M_a shall be considered to equal zero when this equation produces a value less than zero.

T_1 = $2t_l R_t F_y (W_p - y + z)$

T_2 = $t_l R_t (F_u - F_y) (W_p - y + z)$

C_s = $0.8t_l R_t F_y (y - z)$

C_w = $yd_2 C$

R_t = Plate tensile Effectiveness Ratio for direction perpendicular to joint

C = $F_{c\perp}(1.7F_c^*)/(F_{c\perp}\sin^2\theta + 1.7F_c^*\cos^2\theta)$, where F_c^* and $F_{c\perp}$ are the allowable wood compression stresses parallel and perpendicular to grain for the Wood Members adjacent to the joint line, and θ is the angle between the joint line and the length of the Wood Member; use lower C for either of the Wood Members (psi)

W_p = Plate dimension parallel to joint line, not to exceed W_p' as specified in Sections 8.4.3.1 and 8.4.3.2 (in.)

z = Distance from compression edge of lumber to compression edge of plate (in.). For members subject to axial tension force such that there is

no compression, the compression edge is defined as that edge with the least tension

F_y = Plate steel tensile yield strength (psi)

F_u = Plate steel tensile ultimate strength (psi)

P_t = Axial force applied to joint: positive if tension, negative if compression (lbs.)

t_l = Plate steel design thickness (in.)

d_1' = Wood cross-section dimensions in the plane of the Truss, measure along the joint cut-line. For differing chord sizes, d_1' is the dimension of the joint cut-line for the smaller chord (in.)

d_2 = Wood cross-section dimensions perpendicular to the plane of the Truss (in.)

8.7.2 Design of Plate Lateral Resistance for Effect of Moment.

The moment applied to a Metal Connector Plate used in chord splices shall not exceed the moment capacity in lateral resistance in any orientation, or the combined capacity for moment and non-moment loads, defined as follows:

$$V_M \leq V_{LRmin}' \quad (E8.7-2)$$

$$V_M + V_p \leq V_{LR} \quad (E8.7-3)$$

where:

V_M = Tooth holding stress due to moment (psi/pair)

V_p = Tooth holding stress resultant of shear/axial loads in wood (psi/pair), equal to the vector addition of shear + axial loads in wood, divided by A_{ef} : $4M_A / (A_{ef}D)$

V_{LR}' = Allowable tooth holding stress for orientation of V_p , per Section 8.3 [psi/pair (N/mm²/pair)]

V_{LRmin}' = Minimum allowable tooth holding value for any angle of load at the joint [psi/pair (N/mm²/pair)]

M_A = Design moment load applied to joint in the plane parallel to the Metal Connector Plate surface (lb-in.)

A_{ef} = Effective plate area on one face of each Wood Member at splice joint (in.²)

D = Diagonal of a rectangle equivalent to A_{ef} (in.)

$$= \sqrt{\left(\frac{A_{ef}}{h}\right)^2 + h^2}$$

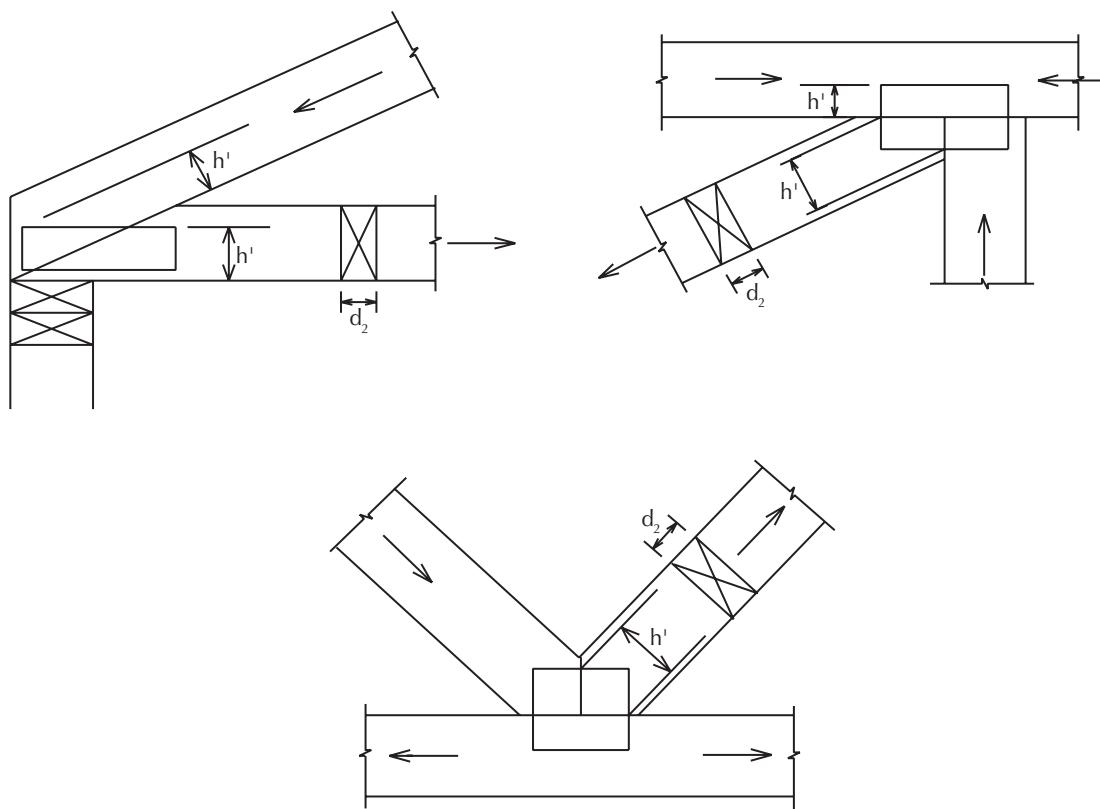


Figure 8.8-1 Reduced Net Section Checks.

$h =$ Height of equivalent rectangle, equal to the greatest dimension across A_{ef} perpendicular to the longest side of A_{ef}

8.8 NET SECTION LUMBER CHECK (H')

8.8.1 Reduced Net Section Checks.

At all joints, members shall have Metal Connector Plates sized or positioned so that the allowable axial tension stress, F_t' , of any Wood Member, or the allowable axial compressive stress, F_c^* , of any Wood Member at any joint without wood-to-wood bearing in the direction of the axial force, is not exceeded on the reduced net section, h' times d_2 (see Figure 8.8-1).

8.8.2 Limit on Tension Introduced into a Wood Member.

For wood thickness greater than 2 in. (38 mm) with plates embedded only on the surface normal to the thickness, the tension, T , introduced by a single joint into a Wood Member shall not exceed the limits defined by the Truss Design Engineer in pounds per inch (N/mm) of wood width, where wood thickness is the wood cross-section dimension perpendicular to the plane of the Truss and wood width is the wood cross-section dimension in the plane of the Truss, and this tension limit shall be adjusted per Section 6.4, including the application of the repetitive axial stress to Truss chords but not to Truss Webs. This

section shall apply only to Wood Members with a cut end under, or within 1 ft. (30 cm) of the edge of, a Metal Connector Plate.

User (non-mandatory) note: Unless otherwise justified, TPI's Technical Advisory Committee (TAC) recommends the use of 2300 lbs./in. (403 N/mm) for T .

8.9 INTERACTION BETWEEN PLATES AND OTHER FASTENERS

8.9.1 Lumber Dowel Bearing Strength Increase.

Metal Connector Plates with integral Teeth shall be permitted to be used to increase the dowel bearing strength of lumber, for dowel-type fasteners embedded into the plated lumber, subject to the limits specified in Sections 8.9.1(a) through 8.9.1(g). Except as mentioned in Sections 8.9.1(a) through 8.9.1(g), fastener design shall otherwise be in accordance with the latest edition of *ANSI / AF&PA NDS* or other approved design specification.

- (a) Plates shall extend no less than two times the dowel diameter in all directions from the center of the dowel.
- (b) Plates shall extend 2.5 times the dowel diameter in the direction of loading.

- (c) If the fastener load is not parallel to the grain, the plate shall extend across the entire wood depth to within $\frac{3}{8}$ in. (10 mm) of each edge.
- (d) For bolts loaded at least 40 degrees from the grain direction, the End Distance shall be permitted to be reduced to the greater of four bolt diameters or 2.2 in. (56 mm).
- (e) Fasteners shall be driven through solid steel sections of the plate, or shall be of a greater diameter than the slot dimension parallel to the direction of loading. holes to permit fastener installation shall be permitted to be drilled in the Metal Connector Plate to a diameter no greater than $\frac{1}{32}$ in. (0.8 mm) greater than the fastener diameter. When holes are drilled through plated lumber, a backer board shall be used, or other steps shall be taken, to assure that the drilling procedure does not result in the plate on the back face of the lumber being forced out of the wood.
- (f) The dowel embedment strength of the plated cross-section, F_{section} , shall be determined using a weighted average based on thickness, as follows:

(E8.9-1)

$$F_{\text{section}} = \frac{(F_{\text{wood}} \times t_{\text{wood}}) + (2 \times f_{\text{steel}} \times t_1)}{(t_{\text{wood}} + 2 \times t_1)}$$

where:

F_{wood} = wood dowel embedment strength as specified by ANSI/AF&PA NDS, or other approved design specification

t_{wood} = wood thickness penetrated by the dowel

$$f_{\text{steel}} = (1.95 - J \times d) F_y \quad (\text{E8.9-2})$$

where:

F_y = Yield strength of the steel used to produce the Metal Connector Plate

d = Dowel diameter

J = 1.04 when 'd' is in units of in.
= 0.041 when 'd' is in units of mm

- (g) The portion of dowel load, P , that shall be considered to be introduced into the plate, shall be determined as follows:

$$P = f_{\text{steel}} \times d \times t_1 \quad (\text{E8.9-3})$$

Lateral resistance of the plate for this force shall be limited in accordance with Section 8.3 and in accordance with the following limitation: no Teeth shall be considered effective unless they are within 85 times t_1 of the dowel center.

8.9.2 Bolts and Other Fasteners.

Bolts and other fasteners used at a joint, which are not included in the design of the Metal Connector Plate at that joint, shall not penetrate Metal Connector Plates at critical locations. Critical locations are defined as the cross-section of the Metal Connector Plate at the joint line, and within 1 in. (25 mm) of this joint line, and as otherwise identified by the Truss Designer. holes for such fasteners that are not at critical locations shall be permitted to be made in the Metal Connector Plate, provided the remaining plate area satisfies Section 8.3.

8.10 TENSION PERPENDICULAR TO GRAIN

Any joint in which the net force component that is perpendicular to the chord and will cause separation along the grain, shall be checked for plate positioning per Section 7.5.3.3.

8.11 PLATE POSITIONING TOLERANCE

8.11.1 General.

A tolerance for Metal Connector Plate positioning shall be calculated for each joint selected for inspection per Chapter 3 of this Standard. This tolerance shall be based on calculations as defined in Chapter 8 and Sections 7.3.7.4 and 7.3.8. This tolerance shall be depicted in a Joint QC Detail, which shall be provided for any joint being inspected per Section 3.7.

8.11.2 Joint QC Detail Information.

The tolerances for plate positioning shall be shown for the fabrication tolerance set by the Truss Manufacturer, as well as for a 0 percent fabrication tolerance ($C_q = 1.00$). For each fabrication tolerance, the Joint QC Detail will include a polygon consisting of no less than four points at the maximum allowable positive and negative placement tolerances in two perpendicular directions, where the x-axis and y-axis are parallel and perpendicular to the plate length and/or joint line, respectively. Maximum allowable distances shall be based off of the center point of the Truss plate and relative to the Wood Members at that joint. The Joint QC Detail shall also include the minimum number of required effective Teeth for each plate contact area and a representation of the allowable area for characteristics reducing the plate contact area per

the fabrication tolerance set by the Truss Manufacturer. An example Joint QC Detail is shown in Figure 3.7-1.

8.11.3 Joint Stress Index.

The JSI reported on the Joint QC Detail shall be the JSI determined for the plate located at the intended position and orientation using the selected C_q value.

User (non-mandatory) note: TPI's Technical Advisory Committee (TAC) recommends using a selected C_q value equal to 1.0.

8.11.4 Minimum Plate Positioning Tolerance.

The plate positioning tolerance shall be $\frac{1}{2}$ in. (13 mm) minimum in any direction for all design stress checks other than lateral resistance, and shall be $\frac{1}{8}$ in. (3mm) minimum in any direction for lateral resistance design stress checks.

8.11.5 Plate Rotation.

All design checks in Chapter 8 shall account for plate rotations of plus and minus 10 degrees, or as specified in the Truss Manufacturer's In-Plant Quality Assurance Program, about the plate center at its design position, including the determination of the plate positioning tolerance polygon in Section 8.11. For conditions where the Truss Design used a plate rotation tolerance less than 10 degrees, the maximum allowable plate rotation shall be specified on the Truss Design Drawing.



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**ANSI/TPI 1-2007
COMMENTARY & APPENDICES**

Commentary & Appendices to

**NATIONAL DESIGN
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FOR
METAL PLATE CONNECTED
WOOD TRUSS CONSTRUCTION**

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Printed in the United States of America

COMMENTARY & APPENDICES TO THE NATIONAL DESIGN STANDARD FOR METAL PLATE CONNECTED WOOD TRUSS CONSTRUCTION

FOREWORD

The Commentary portion of this document is intended to provide background and supplementary information to mandatory provisions of *ANSI/TPI 1-2007*. Paragraph numbers in the Commentary marked with a "§" symbol refer to the corresponding section numbers in *TPI 1*. Commentary is only provided for those sections of the Standard whose application by the user can benefit from supplementary information or interpretive discussion. Additionally, the non-mandatory Appendices in this book provide useful information to the user of the *TPI 1* Standard.

Information contained in this book should not be construed to be a compulsory part of *TPI 1*. It is the sole responsibility of the user to apply the information provided. The Truss Plate Institute and the metal-plate-connected wood truss industry at large expressly disclaim any liability arising from the use, application or reference to the present document.

The Commentary & Appendices is a fluid document and is subject to periodic revision. As the opportunity for additional supplementary information and interpretive discussion of the mandatory Standard arises, the Commentary & Appendices provides a suitable means for the dissemination of that information and discussion inside the normal review cycle for *TPI 1*.

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§CHAPTER 1 GENERAL

§1.4 GENERAL PURPOSE

The list of referenced standards includes those standards specifically cited within the *TPI 1* Standard, with the exception of the *ANSI/AITC A190.1* standard, which was included as glued-laminated lumber is discussed in Section 7.3.3.7.

§1.6 DEFINITIONS

Further definitions and terminology are provided in *TPI 1* Chapter 2 and in Appendix D.

§CHAPTER 2

RESPONSIBILITIES IN THE DESIGN PROCESS INVOLVING METAL-PLATE-CONNECTED WOOD TRUSSES

Overview of (non-editorial) Changes

New to TPI 1-2007

- 1) The revised language of Chapter 2 used the language from Appendix A of *TPI 1-2002* and language in the *IBC/IRC* as its basis. (All Sections)
- 2) Chapter 2 now contains two distinct sections; one covering responsibilities for Buildings when a Registered Design Professional (RDP) as the Building Designer is required and one for Buildings when a Registered Design Professional (RDP) as the Building Designer is not required. (Sections 2.3 and 2.4)
- 3) Includes added Owner responsibilities regarding Trusses with a clear span of 60 ft. or greater to contract with a Registered Design Professional to provide inspections to assure bracing is installed properly. (Sections 2.3.1.6 and 2.4.1.5)
- 4) Explicitly identifies allowable bearing stress as a bearing condition that is the responsibility of the Registered Design Professional for the Building (Section 2.3.2.4(c)) or the Building Designer (Section 2.4.2.4(c)) to specify for the Truss Design Engineer or Truss Designer to use.
- 5) Clarifies that lateral, rain and ponding loads must be specified by the Registered Design Professional for the Building (Section 2.3.2.4(d)) or the Building Designer. (Section 2.4.2.4(d))
- 6) Serviceability criteria that the Registered Design Professional for the Building (Section 2.3.2.4(h)) or the Building Designer (Section 2.4.2.4(h)) requires the Truss Design Engineer or the Truss Designer to use is more completely identified, including criteria for creep, deflection, vibration, and differential deflection between adjacent Trusses or another element.
- 7) Replacement of "Continuous Lateral Bracing" and "CLB" terms with "Continuous Lateral Restraint" and "CLR" although the prior terms are recognized as alternative names. (All Sections)
- 8) Clarifies that specification of Continuous Lateral Restraint (CLR) and Diagonal Bracing (other than showing location on the Truss) can be addressed using *BCSI* document. (Sections 2.3.3.1.1, 2.3.3.3, 2.4.3.1.1, and 2.4.3.3)
- 9) Clarification of contractor requirements, including communication, adherence to *BCSI* installation tolerances, inspections of Trusses after delivery and after installation, and damage/repair responsibilities. (Sections 2.3.4 and 2.4.4)

- 10) Uses language intended to be consistent with *IBC* language regarding Truss design contents, including sealing Cover/Index Sheets in lieu of individual Truss designs. (Sections 2.3.5 and 2.4.5)
- 11) Truss Design Engineer is now required to sign and seal Truss Design Drawings when a Building legally requires a Registered Design Professional as the Building Designer. (Section 2.3.5.3)
- 12) The Building Code used must be shown on the Truss Design Drawing (or Cover/Truss Index Sheet). (Sections 2.3.5.5(a) and 2.4.5.4(a))
- 13) Number of Truss plies (if more than 1) must be shown on the Truss Design Drawing. (Sections 2.3.5.5(d) and 2.4.5.4(d))
- 14) Truss design requirement to show wind and earthquake loads in units of force/area replaced by requirement to show environmental load design criteria and lateral loads. (Sections 2.3.5.5(f)(6) and 2.4.5.4(f)(6))
- 15) Clarification that reaction forces that must be shown on the Truss Design Drawing are the maximum such forces. (Sections 2.3.5.5(h) and 2.4.5.4(h))
- 16) K_{cr} (creep factor) to be shown on Truss Design Drawing as applicable. (Sections 2.3.5.5(l) and 2.4.5.4(l))
- 17) The Truss Design Drawing must show the fabrication tolerance instead of C_q factor. Fabrication tolerance is $1 - C_q'$, expressed as a percentage. (Sections 2.3.5.5(n) and 2.4.5.4(n))
- 18) The method of Restraint/Bracing must be shown on the Truss Design Drawing (Sections 2.3.3.1.1 and 2.4.3.1.1 state that reference to *BCSI-B3* or *BCSI-B7* satisfies this, and also note that according to Sections 2.3.3.3 and 2.4.3.3 these apply if no such method is shown). (Sections 2.3.5.5(o) and 2.4.5.4(o))
- 19) Supplemental information to be available from the Truss Designer was moved from Chapter 2 to Chapter 6 Section 6.1.2. (*TPI 1-2002* Section 2.1.3)
- 20) Provision for Truss Manufacturer not providing a Truss Design Drawing (if Truss is marked for bearing and field-installed bracing) was removed. (*TPI 1-2002* Section 2.4.4)
- 21) Clarification of Truss Manufacturer requirements, including communication, content of Truss submittal packages, determination of fabrication tolerance, and Truss plant quality control. (Sections 2.3.6 and 2.4.6)

§2.1 GENERAL PURPOSE

The 2007 edition of the Standard used the language from Appendix A of *ANSI/TPI 1-2002* and language in the *International Building Code (IBC)* and/or *International Residential Code (IRC)* as its basis. Chapter 2 now contains two distinct sections: one for Buildings when a Registered Design Professional (RDP) as the Building Designer is required and one when a RDP as the Building Designer is not required. Responsibilities under these two conditions are different and it was important to clarify them in the standard. As a result of having these two sections, the term Truss Design Engineer (one licensed to practice engineering) is now used to define the one responsible for the design of the Trusses when a RDP is required and the term Truss Designer is used to define the one responsible for the design of the Trusses when a RDP is not required.

§2.2 DEFINITIONS

Building Design: The term Building Designer, while it may include an architect or an engineer, is not restricted to architects or engineers, as many state and local laws do not require the involvement of a registered or licensed professional in the design of certain types of Buildings or structures. In these instances, responsibility for compliance of the Building or structure with all Legal Requirements rests with the Owner (see definition of Owner in Section 2.2).

Construction Documents: Construction Documents are drawings and specifications prepared for the design of the Building and its structural system (see *2006 IBC*, Section 106.1 for related information).

Framing Structural System: The Framing Structural System of a Building is the combination of Structural Elements that support the Building's self weight, applicable occupancy live loads, and environmental loads (e.g., snow, wind, seismic, etc.). These elements may include beams, columns, structural members, and prefabricated structural components, including metal-plate-connected wood Trusses (see definition of Truss in Section 2.2).

Lateral Restraint: The 2007 edition of the Standard replaced the term "Continuous Lateral Bracing (CLB)" with the terms "Lateral Restraint" or "Continuous Lateral Restraint (CLR)" although the prior terms are recognized as alternative names.

Truss Design Drawing: The Trusses designed for use in the Framing Structural System are individually depicted in the Truss Design Drawings.

Truss Placement Diagram: The Truss Placement Diagram, if required, is prepared based on the Truss Manufacturer's interpretation of the Construction Documents and is meant to assist the Contractor (see definition of Contractor in Section 2.2) in correctly locating individual Trusses in the structure. The Truss Placement Diagram will reflect a Truss identifying mark and perhaps other products supplied by the Truss Manufacturer so these products can be more easily identified by the Contractor during field erection. The Truss Placement Diagram is not an engineered drawing and is not intended to replace the Construction Documents; it is only a guide for installation and requires no engineering input. Nevertheless, if the Truss Placement Diagram is required, it must be reviewed and approved by the Owner, either directly or by Contract with the Contractor and the Building Designer.

§2.3.1.6 and §2.4.1.5 Long Span Requirements.

New to the 2007 edition of the Standard is the Owner's responsibilities regarding on-site inspections to assure that the temporary and permanent bracing are installed properly in the case of long span Trusses. This requirement was added due to the increased hazard of failure during construction of long span Trusses.

§2.3.2 and §2.4.2 Requirements of the Registered Design Professional/Building Designer.

For commercial construction projects, the "Building Designer" as used in the Standard is the Registered Design Professional who has responsibility for the overall Building design in accordance with the applicable regulations and the Construction Documents.

The Building Designer is also the person who reviews and coordinates all the Construction Documents delegated to or prepared by others to make sure they do not conflict (see *2006, IBC* Section 106.3.4). On typical commercial construction projects, plans for fire alarm systems, manufactured Trusses and sprinkler systems may not be completed at the time that the Construction Documents are submitted for approval. These become known as "deferred submittals" and the Building Designer is responsible for reviewing these for general conformance to the design of the Building (see Section 2.3.2.3). As such, the Building Designer is responsible for the review of the Truss Design Drawing submittals to verify that all the Structural Building Components and their placement comply with the Construction Documents. The Building Designer needs to provide the Truss Design Engineer/Truss Designer with the information necessary to properly design the structural building components for the Building on the Construction Documents. The Building Designer is responsible for the design of a Permanent Building Stability Bracing system that resists the Truss member brac-

ing loads, including specifying lateral restraint members, diagonal braces and diaphragm sheathing as applicable. This bracing resists forces out of the plane of the individual Truss and is generally part of the seismic, wind or other external lateral force resisting system.

§2.3.2.4 and §2.4.2.4 Required Information in the Construction Documents.

In addition to information required in the Construction Documents in past editions of the Standard, the 2007 edition of the Standard specifically identifies information which must be specified by the Registered Design Professional (RDP)/Building Designer if applicable to the design of the Trusses. This includes allowable bearing stresses; lateral, rain and ponding loads; and serviceability criteria including criteria for creep, deflection, vibration, and differential deflection between adjacent Trusses or other elements.

§2.3.3.1.1 and §2.4.3.1.1 Standard Industry Details.

The use of standard industry details has been added as one of the methods for depicting permanent restraint/bracing of Truss systems. *BCSI-B3: Permanent Restraint/Bracing of Chords & Web Members* and *BCSI-B7: Temporary & Permanent Restraint/Bracing for Parallel Chord Trusses* of the *Building Component Safety Information (BCSI)* are documents explicitly referenced that have industry standard details and other information applicable to the bracing of Trusses and Truss members.

Permanent bracing must provide sufficient support at right angles to the plane of the Truss to hold every Truss member in the position assumed for it in the design. *BCSI-B3* shows the various planes of the Truss that typically must be restrained/braced and provides installation guidelines for gable end frame restraint/bracing, individual Chord and Web member permanent restraint/bracing, Web member reinforcement and permanent restraint/bracing for special conditions.

BCSI-B7 shows the bracing requirements, the strong-backing recommendations and the allowable construction loading for floor Trusses.

§2.3.3.3 and §2.4.3.3 Absence of Truss Restraint/Bracing Method or Details.

This section clarifies that if the Owner or RDP does not specify a method of Permanent Individual Truss Member Restraint and Diagonal Bracing, then bracing/restraint shall be in accordance with the *BCSI-B3* and *BCSI-B7* documents.

§2.3.4 and §2.4.4 Requirements of the Contractor.

The 2007 edition of the Standard provides clarification of Contractor requirements, including communication (see

Sections 2.3.4.1 – 2.3.4.3 and 2.4.4.1 – 2.4.4.3), adherence to *BCSI* installation tolerances (see Sections 2.3.4.5 and 2.4.4.5), examination of Trusses after delivery (see Sections 2.3.4.6 and 2.4.4.6), examination of Trusses after installation (see Sections 2.3.4.7 and 2.4.4.7), and damage/repair responsibilities (see Sections 2.3.4.8 – 2.3.4.9 and 2.4.4.8 – 2.4.4.9).

As defined in *TPI 1*, the Contractor is responsible for the construction methods and safety in connection with the handling, storing, installation, restraint and bracing of the Trusses.

Sections 2.3.4.6 – 2.3.4.9 and 2.4.4.6 – 2.4.4.9 outline that the Contractor is responsible for the examination of Trusses for damage and the steps to take if damage is discovered.

The Contractor is responsible for the safe installation of the Trusses. The Contractor is also responsible for installing the Permanent Individual Truss Member Restraint at locations specified on the Truss Design Drawings and the Permanent Building Stability Bracing as specified by the RDP/Building Designer.

§2.3.4.8 and §2.4.4.8 Truss Damage Discovery.

As Trusses are engineered structural components, their structural integrity can be substantially altered by damaging, cutting, or modifying any of their members. Truss members may break if improperly handled during the unloading, storage, installation and erection phases of the project. If the Trusses are cut or modified by the construction trades to avoid interference with other building components, the Contractor is responsible for securing the documentation required for repairing the Truss. The Truss Manufacturer should be informed immediately of any damaged, cut, or field modified Trusses and may be able to assist in providing the required documentation.

§2.3.4.9 and §2.4.4.9 Truss Damage Responsibilities.

The Contractor is required to report to the Truss Manufacturer (and RDP when applicable) any Truss damage so they can determine adequate field repairs. *BCSI-B5 Truss Damage, Jobsite Modifications and Installation Errors* is a valuable resource and describes necessary steps and common repair techniques. The *BCSI-B5* document also contains information required from the field crew in order to secure an accurate repair detail from the Truss Manufacturer or RDP.

§2.3.5 and §2.4.5 Requirements of the Truss Design Engineer/Truss Designer.

The Building Designer typically delegates the design of

all the individual Trusses to the Truss Design Engineer (Section 2.3.5) or Truss Designer (Section 2.4.5).

§2.3.5.2 and §2.4.5.2 Single Truss Component Design.

It is the responsibility of the Truss Design Engineer/Truss Designer to properly design each Truss according to the information provided in the Construction Documents, and to depict each such Truss as detailed in a Truss Design Drawing.

§2.3.5.3 Truss Design Drawing Seal and Signature.

Section 2.3.5.3 is included for consistency with the 2006 IBC.

§2.3.5.4 and §2.4.5.3 Truss Placement Diagram.

The Truss Placement Diagram prepared by the Truss Manufacturer is not an engineering document and should never be considered as a replacement for a structural framing plan prepared by the Building Designer. The preparation of the Truss Placement Diagram does not require the special education, training and experience that define the practice of engineering (as found in state engineering laws; further information specific to individual state laws is available at: www.sbcindustry.com/technotes.php).

Since the Truss Placement Diagram prepared by the Truss Manufacturer is not an engineering document, it should not be sealed. When a sealed structural framing plan is required, it should be prepared by the RDP for the Building/Building Designer responsible for the overall Building design to ensure the adequacy and safety of the entire structure. The Truss Placement Diagram prepared by the Truss Manufacturer should ordinarily be reviewed and accepted for conformance with the overall Building design by the RDP for the Building/Building Designer.

Applying a professional engineer's seal to a Truss Placement Diagram may confer more responsibility for the Building design than was intended by the sealing engineer since the Truss Placement Diagram may depict materials and portions of the structure other than considered by the engineer. When the reason for requesting a professional engineer's seal on a Truss Placement Diagram is to assure or attribute engineering responsibility for certain materials not shown on Truss Design Drawings, such as Truss to Truss connections, the information can be better provided in a specification sheet with a separate table of such connections and relevant details, which can then be signed and sealed by the Truss Design Engineer as appropriate. An exception to this general concept is when the seal on the Truss Placement Diagram is made by the

engineer who has design responsibility for the structural framing layout and bearing conditions of the entire Building.

§2.3.5.5 and §2.4.5.4 Information on Truss Design Drawings.

The Trusses designed for use in a Framing Structural System are individually depicted in the Truss Design Drawings. Truss Design Drawings are not typical construction shop drawings, as they do not set forth sufficient fabrication, assembly or installation details on their own, but they include sufficient information regarding the Truss to identify its purpose and suitability for that purpose. They are referred to as Truss Design Drawings within *TPI 1* as that is considered the most accurate term for their content, but it is noted here that others within the construction process may refer to Truss Design Drawings as shop drawings. Other drawings that may be produced by (or for) the Truss Manufacturer to aid in the fabrication of individual Trusses include cutting lists, materials lists, and individual joint depictions. An additional resource of information on Truss Design Drawings can be found in WTCA's *Truss Technology in Building (TTB)* brochure *How to Read a Truss Design Drawing*: www.sbcindustry.com/ttbdrawing.

The additional design data (e.g., design calculations, effective Teeth, JSI, etc.) previously in Chapter 2 of *ANSI/TPI 1-2002* which is required to be available from the Truss Designer has been relocated to Chapter 6 Section 6.1.2 in the 2007 edition of the Standard.

§2.3.5.5(a) and §2.4.5.4(a) Building Code.

New to the 2007 edition of the Standard, the applicable Building Code must now be called out on the Truss Design Drawing (or Cover/Truss Index Sheet).

§2.3.5.5(d) and §2.4.5.4(d) Multi-Ply.

The 2007 edition of the Standard now standardizes the requirement that the Truss Design Drawing must now list the number of Truss plies (if more than one).

§2.3.5.5(f)(1) and §2.4.5.4(f)(1) Top Chord Live Load (including snow loads).

For snow loads, there must be a common understanding on the part of the Truss Designer and Building Designer as to which snow load value is to be used as the uniform design load for snow: ground snow or a factored ground snow load. There must also be a common understanding regarding how ground snow is factored, as well as if and when snow conditions, other than snow as a uniform

load, are to be considered. *ASCE 7-05*, Section 7 and *2006 IBC*, Section 1608 outline a procedure to determine design snow loads for flat and/or sloped roofs.

§2.3.5.5(f)(6) and §2.4.5.4(f)(6) Environmental Load Design Criteria.

The 2007 edition of the Standard replaced the requirement to show wind and earthquake loads in units of force per unit area on the Truss Design Drawing with the requirement to show environmental load design criteria and lateral loads.

The interpretation of wind loading provisions and the methodology used to perform the calculations offer great potential for divergence. Regardless of the given wind speed or whether the structure is designed using prescriptive code or engineered design, wind loading on Trusses is usually considered using *ASCE 7*, Section 6.5, Method 2 – Analytical Procedure. Neither the Simplified Method of Section 1609.6 in *IBC* editions prior to 2006 or *ASCE 7* (current edition) Section 6.4 nor the C&C tabular design values of the *IRC* are generally applicable for Truss design nor are they commonly used. *IRC* allows the use of engineered design of Structural Elements per the *IBC* (which directly references *ASCE 7* for wind design) for either structures or parts of structures that otherwise fall within the scope of the *IRC*. Because of the complexity of wind design, wind design parameters rather than loads are required as the wind pressures acting on the Truss and Truss members are normally calculated within the Truss design software. All other load cases typically provide the design information in load units (psf or similar).

§2.3.5.5(h) and §2.4.5.4(h) Maximum Reaction Force and Direction.

This section clarifies reaction forces which must be shown on the Truss Design Drawing are to be the maximum of such forces. The reaction uplift and downward force is the maximum amount to which the Truss is expected to be subjected. The reaction shown on a Truss Design Drawing is not intended to supersede values determined by a licensed architect or engineer as appropriate for design of the supporting structure for the Truss(es).

§2.3.5.5(l) and §2.4.5.4(l) Deflection.

The 2007 edition of the Standard requires that the creep factor (K_{cr}) be shown on Truss Design Drawing as applicable.

§2.3.5.5(n) and §2.4.5.4(n) Fabrication Tolerance.

New to the 2007 edition of the Standard, the Truss Design Drawing must show the fabrication tolerance instead of the C_q factor. Fabrication tolerance is $1 - C_q$, expressed as a percentage (see Section §3.3.2).

§2.3.5.5(o) and §2.4.5.4(o) Permanent Individual Truss Member Restraint.

When Lateral Restraint (lateral bracing) is necessary, the locations for such restraint or the maximum interval (spacing) between such restraint locations along the member is required to be specified on the Truss Design Drawing. The Truss Design Drawing includes items (m) (maximum axial forces in Truss members) and (o) so that the Building Designer has the necessary information to generate the appropriate engineering calculations for the Permanent Building Stability Bracing.

New to the 2007 edition of the Standard, the method of providing this Restraint/Bracing must now be shown on the Truss Design Drawing (other sections state that reference to *BCSI-B3* or *BCSI-B7* satisfies this, and also note that these apply if no such method is shown- see Sections §2.3.3.3 and §2.4.3.3). The intent of this change, coupled with the changes in the 2006 edition of *BCSI*, is to provide sufficient prescriptive information so that adequate restraint/bracing can be installed by the Contractor.

§2.3.6 and §2.4.6 Requirements of the Truss Manufacturer.

The 2007 edition of the Standard provides clarification of the Truss Manufacturer requirements, including communication (see Sections 2.3.6.1 – 2.3.6.8 and 2.4.6.1 – 2.4.6.8), content of the Truss Submittal Packages (see Sections 2.3.6.7 and 2.4.6.7), determination of the fabrication tolerance (see Sections 2.3.6.9 and 2.4.6.9) and in-plant quality control (see Sections 2.3.6.10 – 2.3.6.11 and 2.4.6.10 – 2.4.6.11).

The Truss Manufacturer's responsibility is to manufacture the Trusses in conformance with the Standard. *TPI 1* provides manufacturing quality requirements to ensure the Trusses perform as expected.

It is advisable that Truss Manufacturers include important safety information in their jobsite packages as a guide to aid the Contractor in the proper handling, installing, restraining and bracing of metal-plate-connected wood Trusses. *BCSI-B1* is an example of this type of information and is currently the best known source. Pre-assembled jobsite packages are also available through WTCA and TPI (more information can be found at: www.sbcindustry.com/bcsi.php).

§2.3.6.3 Alternate Truss Designs.

This section clarifies that alternate Truss designs do not require a professional engineer's seal until accepted by the RDP.

§2.3.6.4 and §2.4.6.4 Truss Placement Diagram.

Additional information on how to read a Truss Placement Diagram can be found in the WTCA *Truss Technology in Building (TTB)* brochure *How to Read a Truss Placement Diagram*: www.sbcindustry.com/ttbplace.

§2.3.6.7 and §2.4.6.7 Truss Submittal Packages.

TPI 1-2002 Sections 2.4.4 – 2.4.4.3 permitting the marking of Trusses for bearing locations, spacing, etc., in lieu of supplying Truss Design Drawings, were removed in the 2007 edition as this practice is no longer common nor considered desirable. Marking Trusses in addition to supplying Truss Design Drawings is acceptable and desirable.

§CHAPTER 3**QUALITY CRITERIA FOR THE MANUFACTURE OF METAL-PLATE-CONNECTED WOOD TRUSSES****Overview of (non-editorial) Changes****New to TPI 1-2007**

- 22) Code Sections 109/R109 (*IBC/IRC*) mentioned with respect to Truss Manufacturer QC audits. (Section 3.1.3)
- 23) Further details specified for content of Truss Manufacturer's In-Plant Quality Control Manual including manufacturer's flowchart or description, organizational chart, key personnel, QC procedures, and document retention policy. (Section 3.2.1)
- 24) Joint QC Details now available in one form [no longer separate Plate Placement Method (PPM) and Tooth Count Method (TCM) types] showing multiple plate placement polygons, so there are no separate inspection procedures that are dependent upon the C_q value. (Section 3.7.2.1)
- 25) Substitution of larger plate clarified to also require same plate orientation. (Section 3.1.3)
- 26) Joint QC Detail to show minimum permitted roller diameter for fabrication for Trusses designed to be built with single pass roller presses. (Sections 3.3.3 and 3.6.4)
- 27) Bearing parallel to grain (F_g) value deleted and COV for MOE replaced with E for stability (E_{min}) for consideration of lumber substitution. (Section 3.4.2)
- 28) Lumber grades identified by package labels (versus grade stamps or inspection certificates) no longer recognized for short lumber. (Section 3.4.3)
- 29) C_q factor basis changed from a maximum value of 1.25 to a maximum value of 1.00 (so 20 percent reduction in plate lateral resistance values with *TPI 1-2002* suggested $C_q=1.00$ for 2x_ Trusses is now obtained if $C_q=0.80$). (Section 3.3.2)
- 30) The tolerance for lumber characteristics permitted in the plate contact area is now whatever tolerance is selected by the Truss Manufacturer, rather than always 20 percent or ten percent for 2x_ and 4x2/3x2 orientations, respectively. (Section 3.3.2)
- 31) Trusses selected for QC inspection no longer require detailed joint inspection of all critical joints, but only those critical joints as defined in the In-Plant Quality Control Manual provided no less than one critical joint/Truss, on average for the manufacturing plant, is inspected. This permits the Truss Manufacturer the ability to adjust inspections by production line, time or as otherwise needed, if permitted by their In-Plant Quality Control Manual, so as to better focus on critical Trusses and better meet production schedules. (Section 3.7.1)
- 32) Critical joints are defined as those with JSI over 0.80 (Set at 0.65 except for 0.80 only if C_q value was 20 percent below the maximum C_q value in *TPI 1-2002*). (Section 3.7.1)
- 33) Larger polygon (formerly the TCM polygon) on Joint QC Detail now incorporates max. C_q factor rather than ignoring tooth holding limits and referred to as 0 percent fabrication tolerance polygon. The PPM polygon is now referred to as fabrication tolerance set by the Truss Manufacturer. The 0 percent fabrication tolerance polygon is permitted to be used for inspection of any joint when either the wood contains no characteristics reducing the plate contact area or the Teeth in the member are counted and exceed the minimum required Teeth/member. (Section 3.7.2.1 and Figure 3.7-1)
- 34) If no Joint QC Detail is available, a $\frac{1}{8}$ in. radius circular positioning tolerance is permitted to be used for plate placement inspection. (Section 3.7.2.1 and Figure 3.7-1)
- 35) Tooth counts, just as in the prior TCM inspection, remain permissible to check for adequacy of plates with excessive embedment gaps or excessive wood characteristics or flattened Teeth that reduce the effective plate area, but are now also permitted to be used to accept plates that exceed limited (less than ten degree) rotational tolerances provided the plates are within ten degrees of the intended orientation. (Section 3.7.7)
- 36) Girder ply-to-ply nailing from only one face (without flipping the girder) is recognized as permitted when done at the Truss manufacturing plant. The In-Plant Quality Assurance Program is to monitor this action. (Section 3.10)

§3.1.1 Scope.

Chapter 3 is the structural quality standard for manufactured metal-plate-connected wood Trusses. It also provides the necessary interpretive rules to inspect metal-plate-connected wood Trusses for quality compliance with respect to a given Truss design. Each manufacturer should have the rules outlined in Chapter 3 as part of their quality program. Generally, there are two levels of quality control (QC) processes within a Truss manufacturing plant:

- 1) Manufacturing quality processes that apply to every Truss and must be followed by all Truss fabrication personnel. This is handled primarily through education and management within a Truss manufacturing plant so that all personnel understand what is required to properly manufacture a Truss in accordance with its design.
- 2) A quality assurance audit process internal to the Truss Manufacturer, which is carried out periodically to verify that the first level processes are operating correctly. The in-house quality assurance audit is performed by one or more of a Truss plant's personnel (e.g., an appointed QC manager), and provides the Truss Manufacturer with the necessary monitoring and management tools for ensuring that proper quality is being met, and for maintaining proper quality as set forth by the Standard.

In addition to these levels, a separate external audit process involving an independent agency may exist to further assure that the Truss Manufacturer's internal processes are operating correctly. The independent (also known as third party) inspection/auditing agency will conduct inspections and audits of the manufactured Trusses and/or the in-house quality inspection records to verify and ensure that the quality of those Trusses inspected meets or exceeds the requirements of the Standard and any appropriate provisions of applicable Building Code(s). The requirement to have periodic, unannounced inspections by an independent inspection agency may be set forth by the applicable Building Code, other requirements of the local Jurisdiction, or may be voluntary by the Truss Manufacturer.

§3.1.2 Requirements.

Structural performance of any manufactured product depends on the finished (manufactured) product being consistent with what was intended in the design. In the design of metal-plate-connected wood Trusses, the Truss Designer must account for some degree of manufacturing inaccuracies (i.e., plate misplacement, partial plate embedment between joined Wood Members) and nat-

urally occurring lumber characteristics (i.e., knots, wane, knot holes, etc.), in order to allow for practical tolerances during the manufacturing process without compromising the structural integrity of the Truss. Therefore, integral to the design of metal-plate-connected wood Trusses is the assumption that the Trusses, and Metal Connector Plates (see Chapter 4), will be manufactured to the minimum requirements for manufacturing quality set forth in the Standard. Given the inherent linkage between manufacturing, design and performance, quality criteria for metal-plate-connected wood Trusses focuses on the following main issues:

- a) Lumber grade/size as specified;
- b) Plate size/type/thickness as specified;
- c) Plate positioned as specified or within acceptable tolerance (including maximum translation tolerances and minimum plate area or Teeth at each member);
- d) Truss dimensions as specified; and
- e) Tight-fitting joints, or gaps between members within acceptable tolerance.

§3.1.3 Documentation.

An effective QC procedure is a continuous process, and the satisfactory manufacture of metal-plate-connected wood Trusses can be assured through:

- a) Proper QC of materials and of the manufacturing process, and
- b) Frequent visual inspection of the manufactured product.

Thus, continuous documentation of some form of in-house product control procedure is critical to maintaining quality in the manufacturing process. Some of the documentation requirements are defined in Section 3.2.1 but much is left up to the Truss Manufacturer and may be in the form of a quality checklist(s), log sheets documenting crews and production levels, or it may involve a process control procedure that tracks quality performance and trends over time. Any necessary corrective actions taken as a result of monitoring the quality of the manufactured Trusses should be included in the documentation.

As the Standard does indicate that the Truss Manufacturer's methods shall be "...*subject to periodic audit for requirements...*" of the Standard, it is important that the Truss Manufacturer maintain documentation to justify that its quality assurance procedure is achieving an ac-

ceptable level of quality. A periodic inspection and audit requires review of a Truss Manufacturer's in-house QC documentation in addition to the optional supplemental inspections of some additional Trusses. It is advisable that the Truss Manufacturer clarify with its third party inspection agency or other entity engaged to assess in-plant quality assurance what the specifics of the audit process are and what actions will be taken in the event of non-conformances.

A Truss Manufacturer is not required by the *TPI 1* Standard to use a third party quality auditor and may find some other means to audit its methods for ensured compliance with the Standard, unless otherwise required by the local authority having Jurisdiction. This requirement for third party quality audits is a requirement in the *International Building Code (IBC)* and is therefore typical for Trusses provided for Buildings constructed in compliance with the *IBC*. The same is not true of the *International Residential Code (IRC)*, but may be required by local authorities for Buildings covered by their local residential code.

The direct reference to Section 109 of the *IBC* and Section R109 of the *IRC* is new to the 2007 edition of the Standard. The language states the Building Official can accept inspections or reports of approved inspection agencies. The criteria in Chapter 3 of *TPI 1*, which is referenced in the 2006 *IBC* (see Section 2304) and 2006 *IRC* (see Sections R502 and R802) codes, serves as a suitable inspection standard for wood Trusses and Truss Manufacturers that is recognized by typical Building Codes.

The Truss Plate Institute, as well as several commercial inspection agencies, operates quality assurance inspection programs for the wood Truss industry. Further information is available from TPI at: www.tpininst.org/quality.html. Additional information for Truss plants regarding the operation of in-plant quality assurance programs, including software and forms to aid in the process, is available from WTCA – Representing the Structural Building Components Industry at: www.sbcindustry.com/wtcaqc.php.

§3.1.4 Non-Conforming Inspections.

Non-conformance with any of the quality criteria in the Standard signifies that one or more design considerations or assumptions have not been met. This does not necessarily mean that the Truss will not perform adequately as built; however, the Truss Designer must confirm acceptance of the Truss, most often through re-analysis of the Truss with the appropriate change or changes in design assumptions. An example of a manufacturing inaccuracy that would be in violation of the Standard, and would thus require follow-up approval or repair (if necessary) by the

Truss Designer upon re-analysis, is a rotation of a Metal Connector Plate by 90 degrees. The Standard requires the plate to be positioned in the orientation specified on the Truss Design Drawing, which is important because plate lateral resistance and steel strength vary based on tooth slot orientation. The deviation in design values between one plate orientation and its orthogonal orientation might be significant enough to affect the Truss design capacity. Thus, only a re-evaluation by the Truss Designer will determine whether a plate rotated by 90 degrees is acceptable, and this requires approval and documentation by the Truss Designer.

§3.2 IN-PLANT QUALITY ASSURANCE PROGRAM

Criteria specific to the Truss Manufacturer's in-plant quality assurance program documentation and inspection process has existed in the Standard since the 2002 edition. These criteria were added to better standardize the QC process within Truss plants and elevate the overall quality of the entire industry.

§3.2.1 In-Plant Quality Control Manual.

The in-plant QC manual sets the foundation of the Truss Manufacturer's quality assurance procedures. As discussed in Section §3.1.1, the first level of a QC process includes proper manufacturing practices that apply to every Truss and are carried out by all personnel on a daily basis. These practices should be included in the QC manual and might include, for example, instructions on the proper use of pressing equipment, management directives on what types of lumber and/or plate substitutions are acceptable without prior notification of management, guidelines for reading the Truss Design Drawing (e.g., symmetrical versus unsymmetrical plating), and other standardized rules. In addition to proper daily manufacturing practices, the QC manual should also include the plant's methods for the next level of the QC process, i.e., the periodic in-house audits of the manufacturing process to ensure conformance with the Standard. The QC manual should include the requirements for these in-house audits, whether through a checklist system, process control procedure, or some other means.

New to the 2007 edition of the Standard are specific requirements for a production flowchart or description of the manufacturing process (Figure C3.2-1), an organizational chart or description of key duties and responsibilities (Figure C3.2-2), a sampling criterion sufficient to ensure consistent product quality, and a document retention policy. The goal of these items is to make a plant's QC manual a more comprehensive document and to document the relevant aspects of the QC process. The flowchart and

organizational chart should be included in the manual and updated as changes occur. For more information on the inspection sampling criteria, refer to Section §3.2.3. Sampling criterion refers to the method a plant uses to select Trusses to inspect. The requirement is to inspect all lines, shifts, and crews, and to make sure selection is random. It can be helpful to include a plant layout in this section (see Figure C3.2-3). *TPI 1* does not specify a minimum required document retention period, but some in the

industry have suggested retention of inspection summary documents for a minimum of two years, and retention of the most recent two weeks of detailed inspection documents such as Joint QC Details, Truss Design Drawings, tooth reports, and inspection forms. The most recent two weeks of inspection documentation is useful information for periodic audits and can also provide more detailed information that can be useful to the Truss Manufacturer.

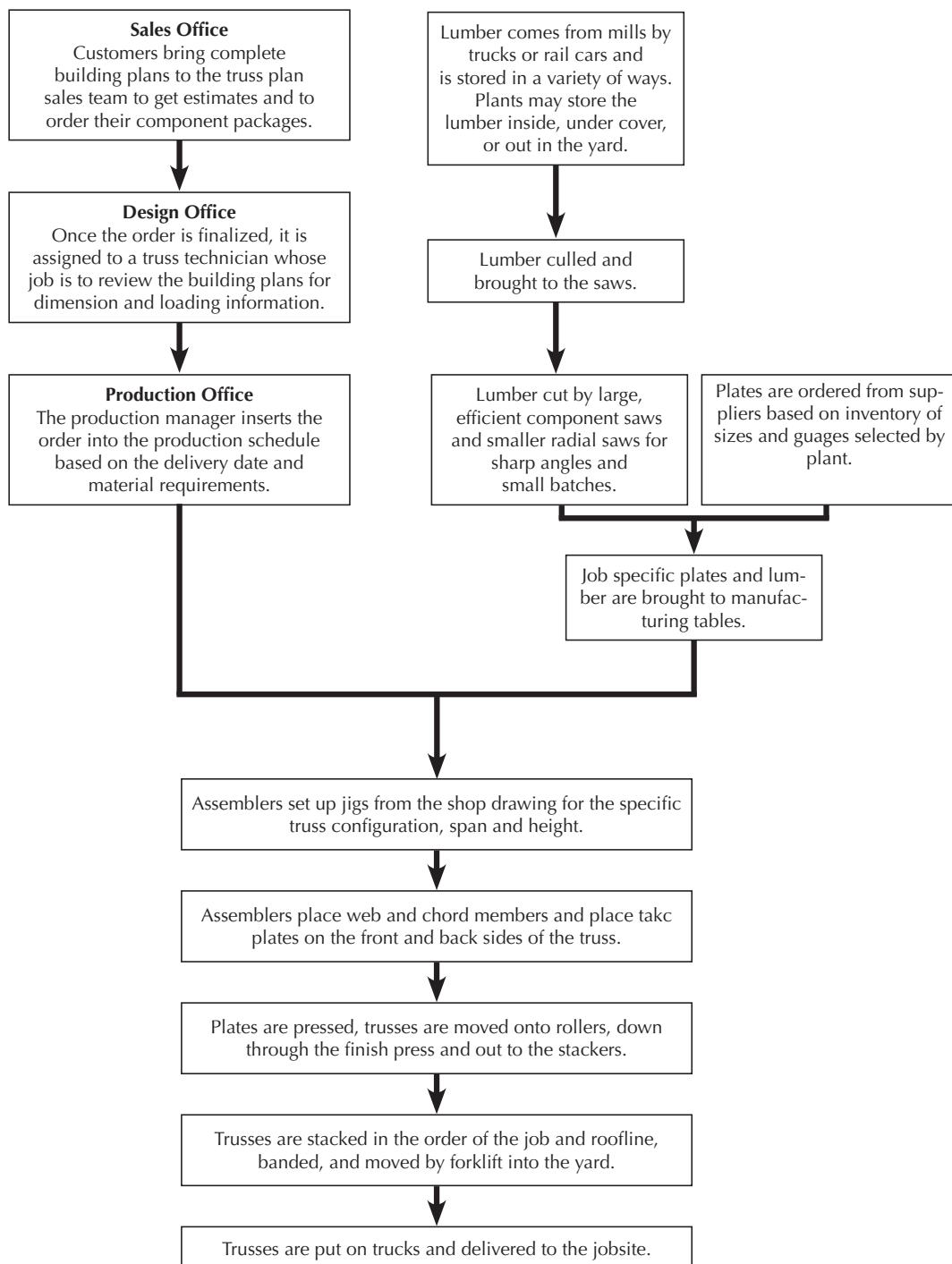


Figure C3.2-1. Truss Design & Manufacturing Flowchart.

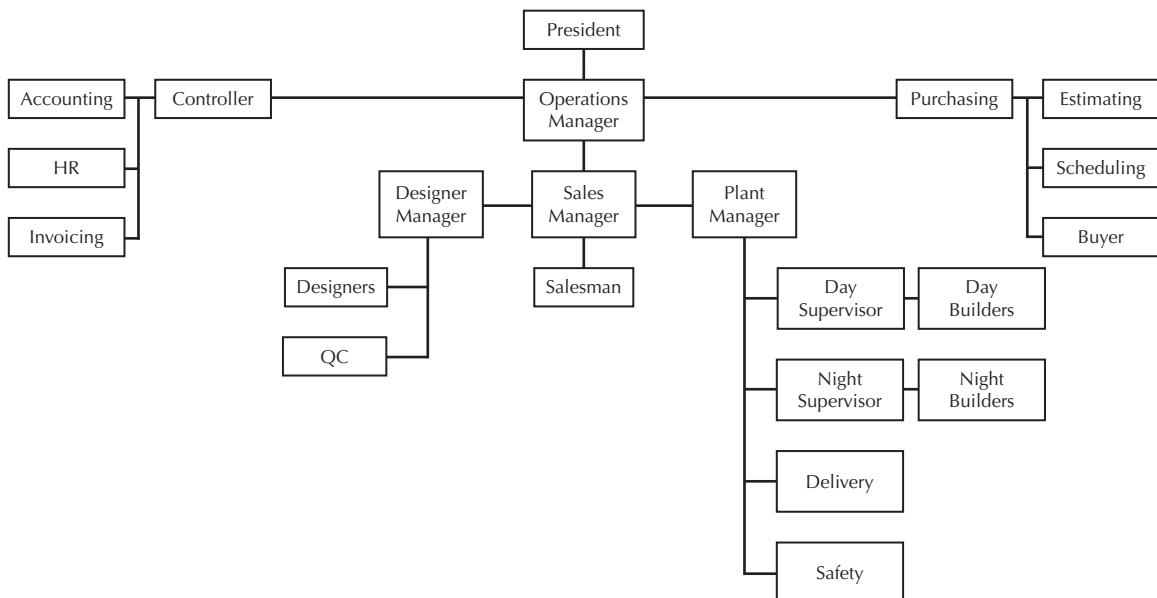


Figure C3.2-2. Organizational Chart Example.

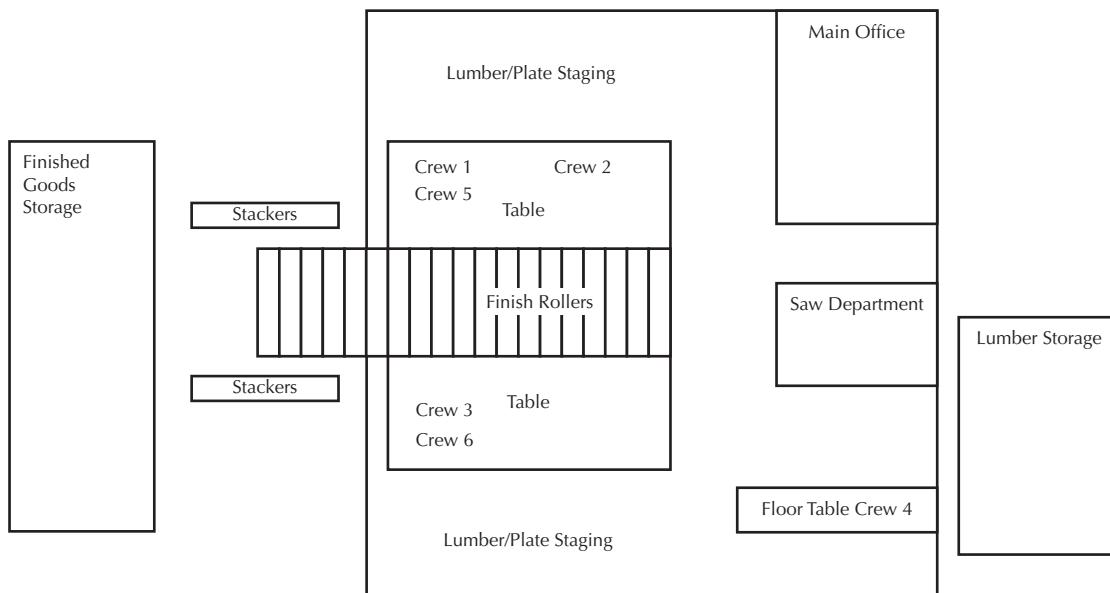


Figure C3.2-3. Plant Layout Example.

§3.2.2 Inspection Frequency.

The minimum inspection frequency set forth is in recognition of the need to frequently monitor for the potential of any consistent inaccuracies or quality non-conformances in the manufacturing process. Because manufacturing inaccuracies can be related to both manufacturing equipment and personnel, an effective inspection frequency must provide monitoring of all areas where a potential problem could arise, including manageable sized groups that together comprise the entire personnel

and which will provide enough indication of any needs for additional training of personnel; repairing, recalibrating, or replacing equipment; developing new or different assembly practices; or any other modifications to the in-house quality assurance program.

Although “operational set-up location” in the inspection frequency is not strictly defined, it is intended that each Truss manufacturing plant will establish reasonable, manageable sized groups in each work shift from which a minimum of three Trusses per week will be inspected

and recorded for the in-plant audit. For example, an “operational set-up location” might be defined as the location on a manufacturing table where a crew, or group of personnel within a defined area work building Trusses five days per week. If so defined, then each “operational set-up location” during each shift will have a minimum of three Trusses inspected per week. Set-up locations should be defined by each Truss Manufacturer depending on the structure and management of the plant. If the crew remains in a static location all week then it is clear that the plant will be required to inspect three Trusses at that set-up location. If the plant has more set-up locations available than are being used five days a week, then the plant must do inspections proportionately to how often that set-up location is being used. For example, take a case where a plant has two set-up locations on one manufacturing line or table. In this specific case, only one crew is building Trusses while another employee sets up the Truss configuration for the next run so the one crew can continuously build. In this situation the plant could very well choose to inspect three Trusses for each location. However, since neither set-up location is operating continuously, it is also reasonable for the plant to split those three inspections each week between the two locations - perhaps doing two on one set-up location and one on the other.

It should be recognized that there may be instances that would warrant additional inspection beyond the minimum established inspection frequency. Examples include the introduction of any new personnel, new fabrication equipment, or any other changes in the production line or assembly practices. It is recommended that the first Truss manufactured after the introduction of any change in the production line be inspected. Additionally, the plant should ensure that compliance with the quality Standard is achieved consistently (e.g., for at least three Trusses in a row) following such changes.

It is recommended that some documentation be maintained to show the production fluctuations (i.e., Trusses produced and crews/personnel working) that can occur in a plant over time. This information can be in the form of a log or any form deemed appropriate. By documenting this type of information, the Truss Manufacturer can monitor the production fluctuations and adjust the inspection frequency accordingly (e.g., low production levels and fewer crews/personnel can equate to a lower frequency of inspections while high production levels and more crews/personnel can equate to a greater frequency of inspections). This is also valuable documentation for periodic audits by others, such as by a third party inspection agency.

§3.2.3 Inspection Sampling.

A random sampling implies that the Truss type should vary in span, pitch, and height. A representative sampling implies that the proportion of any one Truss type inspected will be consistent with its share of the total Truss production at the plant. For example, if a Truss plant produces 20 percent parallel chord Trusses of its total production, then approximately 20 percent of the Trusses inspected should be parallel chord Trusses. It is also advisable that a plant regularly documents the type of Trusses and Truss spans it produces since this can vary over time too. By tracking and documenting this information, a Truss Manufacturer can see if they are in fact inspecting a representative sampling, and if not make the necessary adjustments. As with production levels mentioned above, this is also valuable documentation for periodic audits by others.

§3.2.4 Inspection Procedure.

The joint inspection procedures referred to and set forth in Section 3.7 were specifically developed for incorporation into the 2002 edition of the Standard and have been retained with some minor revisions in the 2007 edition of the Standard. The inspection procedures in both the 2007 and 2002 editions differ significantly from earlier procedures because they center quality assessment on Metal Connector Plate placement, whereas the 1995 edition of the Standard required an assessment of the plate Teeth in each plate contact area in addition to assessing plate placement on a joint. The inspection procedures in Section 3.7 are presented as the primary quality assessment for use in a Truss plant because it is easily evaluated by plant personnel. Metal Connector Plate placement has been shown to have a direct correlation to structural performance (see Section §3.7 for more information).¹

§3.3.2 Fabrication Tolerance.

The C_q factor was introduced in the 2002 edition of the Standard. Truss Manufacturers set this C_q factor, which is used to reduce the tooth holding values to make an allowance for lumber characteristics and flattened Teeth. The C_q factor was originally set up to range as high as a value of 1.25 (i.e., tooth holding value was adjusted by multiplication by $C_q / 1.25$). In the 2007 edition of the Standard, the C_q factor was modified to range only up to a maximum of 1.00 (i.e., tooth holding value was adjusted by multiplication by C_q alone) and was also correlated to its inverse quantity which is now called the fabrication tolerance, as the fabrication tolerance is more easily understood. Truss Manufacturer will set a fabrication tolerance rather than the C_q factor. The fabrication tolerance can be set anywhere from 0 to 30 percent or more.

¹ S. M. Cramer, *Quality control impacts on metal plate connected truss performance*, Report to Truss Plate Institute, 2001.

The most commonly used fabrication tolerance has been 20 percent (which correlates to a C_q factor of 1.00 for Trusses designed under the 2002 edition of the Standard or a C_q factor of 0.80 for Trusses designed under the 2007 edition of the Standard).

The 2007 edition of the Standard has added flexibility for plants to increase the ability to control the tolerance for plate placement, using the fabrication tolerance as the means to do so, in addition to maintaining the use of the fabrication tolerance to allow for lumber characteristics and flattened Teeth. The 2007 edition also removed a limitation set in the 2002 edition, but which could not be justified, and which required more time-consuming formal tooth counts be done (labeled Tooth Count Method), rather than use plate placement polygons only to evaluate Teeth capacity (labeled Plate Placement Method) when fabrication tolerances were set below 20 percent (or ten percent for 4x2 Trusses). In the 2007 edition of the Standard, all plate placement polygons are established considering Teeth capacity, thus formal tooth counts are never specifically required although they may be used when they are advantageous. The 2007 edition now mandates that placement polygons be established, and shown on the Joint QC Detail, for the 0 percent fabrication tolerance in addition to whatever is specified by the Fabricator. This permits the inspector to make use of the increased positioning tolerance with the 0 percent fabrication tolerance when members are free of lumber characteristics and flattened Teeth. For a Wood Member's plate contact area, any combination of lumber characteristics and tooth flattening are not to exceed the fabrication tolerance set by the Truss Manufacturer. Since the 2002 edition was published, plants have typically used the 20 percent tolerance (or ten percent for 4x2 Trusses) required to make use of the Plate Placement Method, or a lesser tolerance (e.g., 0 percent) and made use of the Tooth Count Method. A greater fabrication tolerance results in larger plate sizes, but did not typically result in a greater defined tolerance for plate positioning since the 2002 edition assigned the fabrication tolerance entirely to account for characteristics in the lumber reducing the effectiveness of Teeth, such as wane, holes or loose knots. To identify the increased positioning tolerance possible with the plate, the fabricator had to make use of the more time-consuming Tooth Count Method of inspection. Since the 2002 edition of the Standard was released, TPI has learned that a majority of plants have more problems with placement rather than the quality of lumber or flattened Teeth. Therefore, plants require more flexibility with regard to how they consider lumber characteristics and flattened Teeth in the design process. They need to evaluate if they want to make more of an allowance for lumber characteristics and flattened

Teeth or for misplacement. To set their fabrication tolerance, each plant should analyze how big a problem misplacement and lumber characteristics are at their plant. It is suggested to use a fabrication tolerance set to 10 percent to allow for some amount of lumber characteristics and flattened Teeth, and to adjust the fabrication tolerance larger if subsequent inspections show inadequate results, or possibly to decrease the fabrication tolerance if subsequent inspections show adequate results. Some fabrication tolerance greater than demonstrated by past inspections as necessary should be used to provide some margin against unexpected errors but the degree to which this is used is dependent upon the variation in the plant's manufacturing quality and the confidence or safeguards in place against any loss in manufacturing quality.

There are recognized times, where there might be reason to reduce, or remove, the additional conservatism intended for manufacturing defects, and this can be accomplished by setting the fabrication tolerance to 0 percent. One reason may be for practical design purposes, if it is necessary for the Truss Designer to utilize more available strength (i.e., up to the limit permitted by design Standards presuming completely accurate placement of the Metal Connector Plate) in order to arrive at a successful design. This type of decrease is acceptable from a design standpoint, but notification of this change should be provided to the Truss Manufacturer to assure that special attention may be paid in the fabrication process to these critical joints, since the standard practices may not result in adequate placement as evidenced by the Manufacturer's specified fabrication tolerance being greater than 0 percent. There may also be cases where the added conservatism provided through the use of a higher fabrication tolerance is not as advantageous from a manufacturing standpoint. This might be the case if a Truss Manufacturer has procedures specifically in place to avoid plate misplacement and loss of effective Teeth in the plate area due to Teeth flattening or lumber characteristics; for example, trimming lumber to ensure no knots or wane are contained within the ends of the lumber where plates can be placed, and closely monitoring roller press operations such that Teeth are not flattened. If such procedures are in place and can be shown to be effective, then a plant may elect to reduce their fabrication tolerance.

§3.4.1 Lumber Specifications and §3.4.2 Lumber Substitutions.

The selection of Wood Members in a metal-plate-connected wood Truss affects not only the Truss members but also the Truss joints. The ability of the Metal Connector Plate to transfer forces from one Wood Member to another depends on, in addition to its own metal plate

properties, the species, density, and moisture content of the Wood Members. Thus, it is important that each property of the lumber is considered when making a substitution for the specified lumber.

The requirement in the 2007 edition of the Standard similar to the 2002 edition for the substitute Wood Member species to have an equal or greater specific gravity than that of the specified Wood Member species, in addition to having equal or higher strength and stiffness properties, is considered to sufficiently address variation in plate lateral strength among species. A substantial amount of research has correlated a relationship between specific gravity and tooth holding strength (i.e., lateral resistance strength), such that as specific gravity increases, plate lateral resistance increases.^{2, 3, 4} The correlation with specific gravity is applicable across species, although some variation does occur that is considered to be minimal. Furthermore, the permitted substitution of denser species for less dense species is consistent with typical application to all other types of wood fasteners.

§3.4.3 Lumber Identification.

For typical 8 ft. to 20 ft. lengths, most lumber manufacturers provide grade stamps, whereas shorter lengths (i.e., 6 ft., 4 ft., 2 ft.) often are not individually grade stamped, but rather are identified by a label on the outside of the package, as well as a line item on the invoice. To be consistent with the way lumber is commercially provided, the provision regarding marked lumber in the 2002 and later editions was revised to permit use of certificates of inspection.

Grade marks and other identification should be preserved whenever possible or practical during the manufacturing process for verification in the completed Truss unit.

§3.4.4 Preservative Treatment & §3.4.5 Fire Retardant Identification.

The marking by an approved inspection agency is specified to assure that lumber processing during the treatment process, and subsequent processes such as kiln-drying, are conducted in accordance with the processes employed during the evaluation of the treatment's effects. Approval

of an inspection agency for treated lumber is typically the authority of the local Building Official. Inspection agencies may be accredited for inspection of pressure treated wood products by the American Lumber Standard Committee, Inc. (ALSC, see www.alsc.org), the International Accreditation Service, Inc. (IAS, see www.iasonline.org), or other accrediting services.

§3.4.6 Use of Finger-Jointed Lumber.

The 1995 edition of the Standard and its predecessors did not contain provisions for finger-jointed lumber. Although no restrictions have existed in the Standard that would exclude the use of approved finger-jointed lumber in metal-plate-connected wood Trusses, there was a recognized need to better accommodate the increasing availability and use of structural finger-jointed lumber and provisions were added to the 2002 edition.

Structural-glued (finger-jointed) lumber is permitted by typical Building Codes to be used interchangeably with solid-sawn lumber members of the same species and grade. Because structural finger-jointed lumber is graded using the same rules applied to solid-sawn dimension lumber and adhesive joints are evaluated with respect to the design values for the given grade, the design values for a grade and species of structural finger-jointed lumber are identical to the same grade and species of conventional solid-sawn lumber. The reported suitability of finger-jointed lumber, and its use in metal-plate-connected wood Trusses, long precedes the Standard's explicit recognition of it. There has been no substantive evidence that finger-joints adversely affect the performance of Truss plated joints⁵ and their use in Trusses dates to the 1980s, or earlier. However, another consideration is how the finger-joints might affect the proper embedment of the plate, particularly due to the hardened glue on the surface and offset that is permitted to exceed $\frac{1}{32}$ in. (i.e., adjoining surfaces are out of plane from each other by more than $\frac{1}{32}$ in.). These characteristics may cause plate embedment problems when plates are embedded at the location of the finger-joints. To help avoid such problems, Truss Manufacturers purchasing finger-jointed lumber may want to specify that this lumber have no hardened glue on the surface exceeding $\frac{1}{32}$ in. thick and that lumber offset at such joints be limited to no more than $\frac{1}{32}$ in.

The requirement for finger-joints to be in conformance with all the requirements of *ASTM D2559* is included to ensure that the finger-jointed product used in a Truss is suitable for all structural applications. Another type

2 S. K. Suddarth, D. H. Percival and Q. B. Comus, *Testing and analysis of 4x2 parallel chord metal plate connected trusses*, Small Homes Council-Building Research Council, Research Report 81-1, Urbana-Champaign, IL, May 1981.

3 L. C. Palka, *Effect of wood density on truss plate joint strength*, Prepared for Science Council of British Columbia, Forintek Canada Corp., Western Laboratory, Vancouver, BC, 1984.

4 B. K. Via, A. Zink-Sharp, F. E. Woeste and J.D. Dolan, *Relationship between tooth withdrawal strength and specific gravity for metal plate truss connections*, Forest Products Journal, Vol. 49, No. 7/8, 56-63, 1999.

5 R. J-M. Fouquet, R.J-M., J. D. Barrett and D. M. Ainsworth, *Suitability of fingerjoined lumber for truss manufacture*, Report to National Research Council of Canada, Industrial Research Assistance Program, Council of Forest Industries of British Columbia, Project No. 6-1223-19s. Vancouver, B.C., 1986.

of finger-jointed product is intended for carrying only compression loads as a vertical stud member. Unlike the structural finger-jointed lumber, this type of finger-jointed lumber (i.e., “VERTICAL USE ONLY”) is not required to meet certain requirements of *ASTM D2559*, and is therefore not suitable for Truss construction.

Finger-jointed lumber must be identified by the quality mark of an inspection or lumber grading agency approved in accordance with US DOC PS20, which is the American Software Lumber Standard. Agencies that are approved to certify structural-glued products by the ALSC, under the Department of Commerce, will qualify as approved agencies (see listing at: www.alsc.org). These agencies must comply with criteria established by the ALSC’s Glued Lumber Policy, and their quality marks signify that the structural-glued lumber is subjected to extensive QC and testing during production.

§3.5.3 Height and Length.

The allowable variances in Truss length and height in Table 3.5-1 are set forth as the maximum allowable tolerances for both structural purposes and practical purposes. Although these allowable tolerances will not affect the Trusses structurally, there may be further non-structural considerations that warrant tighter variances. One important consideration is the ability to achieve, during field installation, a very flat nailing surface, for drywall, plywood, or any other directly applied diaphragm material. The smaller the variance between adjacent Trusses, the flatter the ceiling or floor plane. Another consideration is the goal to have all Building lines and dimensions as accurate as possible, especially where inaccuracy of one component’s dimension might affect another component. These non-structural considerations are more significant for floor production than roof production, and the maximum tolerances for floor Trusses as shown in Table C3.5-1 are recommended for installation purposes.

§3.6.2 Specifications.

It is not acceptable to substitute two plates, which together equal the size of the specified plate, for a single plate of the same size without Truss Design Engineer/Truss Designer approval (e.g., if the Truss Design Drawing specifies a 5x16 plate at a heel joint, placing two 5x8 plates side by side is not acceptable).

§3.6.3 Substitutions.

The significance of this provision is that both specified plate dimensions must be equaled or exceeded for a substitution to be acceptable. Also, the connector plate gauge must stay the same. Consequently, a connector plate that has a larger dimension than specified in one direction, but a smaller dimension than specified in the other direction would not be an acceptable substitution, even if it provides a larger area [e.g., a 5x8 plate (40 in.²) is not an acceptable substitution for a 6x6 plate (36 in.²) without Truss Design Engineer/ Truss Designer approval]. Also, the orientation of the substituted plate must be the same orientation as the specified plate as indicated on the Truss Design Drawing, even if a larger plate is used. Therefore, if the Truss Design Drawing specifies a 5x6 plate oriented with the tooth slots vertical, it would not be acceptable to use a 6x10 plate with its tooth slots horizontal, unless approved and documented by the Truss Design Engineer/ Truss Designer.

A final consideration to be made when substituting a plate with a larger dimension than specified in one or more directions is its effect on the plate’s intended position on the Truss. In some instances, such as the substitution of a longer plate at a Bottom Chord splice (e.g., a 5x6 plate substituted for a 5x5 plate) the position of the plate is not affected by the substitution. However, in a joint such as a peak plate, a larger plate might cause the upper corners of the plate to extend beyond the top edge of the Top Chord. Because this would interfere with the application of roof sheathing and would be unacceptable interference, the tendency to make a larger plate fit in this case would be to shift the plate down until the corners of

	Truss-to-truss variance dimension of identical trusses	Variance from design dimensions
Length of finished truss unit	1/4 inch	1/4 inch
Height of finished truss unit	1/8 inch	1/8 inch

Table C3.5-1. Recommended Quality Manufacturing Tolerances for Finished Floor Truss Units.

the plate no longer extend beyond the top edge of the Top Chord. However, this would require a quality check to ensure that the substituted plate, as shifted, is within its positioning tolerance (see Section 3.6.4).

§3.6.4 Installation.

Positioning a plate in accordance with the Truss Design Drawing involves orienting the plate slots as shown on the drawing (e.g., horizontally or vertically, or angled with the Teeth slots oriented either parallel or perpendicular to one of the Wood Members at the joint) and positioning the plate in its design position relative to the joint. The Truss Design Drawing may have a note stating that the Truss plates are to be symmetric about the joint unless otherwise shown. Unsymmetrical joints will have dimensions indicating the location of the plate with respect to some reference point within the joint. In many cases, an expanded detail of the joint can be obtained to better determine the plate's design position at the joint. With the increasing complexity of Trusses, use of these expanded details at a Truss plant becomes increasingly necessary to accurately position the plates according to their design positions.

Although the goal for a manufacturing plant is perfect plate placement per the design position, acceptable quality of a joint, even with some deviation from the plate's design position, is determined with the use of allowable tolerances for plate positioning and embedment. The 2002 and later editions replaced the standardized plate location tolerances found in earlier editions of the Standard and its predecessors with a more customized approach in which positioning tolerances are calculated for each joint by the Truss Design Engineer/Truss Designer and available on a Joint QC Detail. Thus, a plate that is located within the specified positioning tolerance shown on the Joint QC Detail will meet the design requirements for that joint. Refer to Section §3.7 for additional discussion on the plate positioning tolerances.

The 2007 edition of the Standard added the requirement that the Joint QC Detail show that the Truss was designed for manufacture with an appropriate diameter of roller press, when applicable. This was added as there was no prior requirement to report this and the use of single-pass roller presses usually requires the use of lower design values than other types of fabrication equipment. As stated in Section 3.3.3, the Joint QC Detail is required to show the minimum diameter of single-pass roller press that is provided for in the Truss Design Drawing. The intent of the wording in Section 3.6.4 regarding roller press diameter is that Trusses fabricated using single-pass roller pressing processes use equipment with the designated minimum diameter of rollers or larger.

§3.6.5 Plate Position.

Figure 3.6-1 illustrates situations where the location of a plate, if the plate edge extends beyond one or more Wood Members, can adversely affect the function of the Truss when installed. The most critical situation is where an extended plate would interfere with the application of sheathing material along the top edge of the Top Chord, or bottom edge of the Bottom Chord. This type of interference is prohibited by the provision in Section 3.6.5. Another situation in which the extension of a plate beyond a Wood Member may be problematic to the function of the Truss is where a plate extends into the open space of an attic Truss or into the duct chase of a floor Truss.

§3.7 JOINT INSPECTION

Section 3.7 at its core is the visual evaluation of Metal Connector Plate placement and has two major advantages over earlier methods employed by the Standard: it provides a faster method for Truss Manufacturers to assess quality, and it provides for more tolerance in the plating area, which provides more assurance that deviations from the intended position and the occasional occurrences of flattened Teeth or lumber characteristics in the plating area will not adversely affect the lateral resistance of the plate.

§3.7.1 Critical Joint inspection.

There have been several changes to Section 3.7.1 in the 2007 edition of this Standard. One of the changes is that all critical joints are defined as having a Joint Stress Index (JSI) of 0.80 or greater for all types of Trusses. Formerly, a lower value was specified if a fabrication tolerance of less than 20 percent was selected. Two other significant changes in this edition of the Standard are that the joints selected for inspection are required to be defined in the Truss Manufacturer's In-Plant Quality Assurance Program and that no less than one critical joint (as defined in the Standard) per Truss selected for inspection, on average, throughout the plant's operating set-up locations for each shift. These revisions to the Standard were made to allow plants more flexibility in the selection of critical joints being inspected to meet the operational needs of the plant while still maintaining a minimum number of critical joint inspections. See also Section §8.11.3 regarding the C_q value to be used for calculation of the JSI, as that also affects the determination of the number of critical joints.

It is still a requirement that all Trusses selected for inspection have preliminary inspections performed, on the front and back side of the entire Truss (i.e., all joints), regardless of whether or not joints are defined as "critical joints" per this section of the Standard.

Additionally, while not stated in the Standard, it is recommended that plants inspect all critical joints on at least some Trusses selected for inspection, or otherwise ensure that all types of joints are included in joint inspections, in order to identify deficiencies that may occur but which may not be present on joints defined as critical. This is especially important when the C_q value selected for determining the JSI (see Section 8.11.3) is set lower (corresponding to a higher fabrication tolerance) than otherwise is possible. Critical joints are those joints that have been identified as the most stressed and are joints that have the least amount of room for error in the manufacturing process. Inspecting all of the critical joints will provide a plant with a better sense of their ability to accurately assemble the joints according to the original design and will help improve the overall quality of the manufacturing process, but it is also recognized that the costs of detailed inspections of joints may not be warranted when there are more efficient ways to ensure adequate joint quality. For example, plants may find that more frequent, but less detailed, inspection of joints is a more efficient approach than the highly detailed inspections that were formerly mandated by the 2002 edition of the Standard on all critical joints of Trusses selected by inspection.

As indicated earlier, at a minimum, a plant can choose to inspect an average of one critical joint per Truss selected for inspection. However, many plants choose to inspect more than the minimum, which is recommended for purposes of generating more complete data to document their quality levels and to provide some margin to allow for tolerances in scheduling which may periodically reduce the sampling rate. This flexibility means that plants looking to inspect a majority of the critical joints could find instances when trying to inspect all the critical joints could present a production work flow problem, especially when inspecting complicated or longer span Trusses. In that instance, capping the number of critical joints, as defined in their in-house QC program and manual, on certain Trusses could provide a sensible option. The cap could be any figure that the plant feels is reasonable as long as the minimum number of critical joint inspections as defined in *TPI 1* are met. Another option may be to increase the JSI level at which inspections of critical joints are required. For example, a JSI level of 0.90 may be selected for purposes of inspecting if the plant wants to focus only on the most critical joints and it is found that this level results in attainment of the required average sampling rate of one critical joint inspection per inspected Truss.

Additionally, suppose a plant has specific critical joint types (e.g., heel joints, etc.) that it feels require greater QC scrutiny. The plant could elect to focus its inspection

on those types of critical joints as long as they 1) meet or exceed the minimum per *TPI 1* of an average of one critical joint per Truss selected for inspection, and 2) the plant still performs preliminary inspections for the overall inspection frequency of three Trusses per week per operational set-up location per shift.

These examples of inspecting critical joints are just a few of the possibilities. Whatever in-house inspection method is used, it must be outlined in the Truss Manufacturer's In-Plant Quality Assurance Program's QC manual. The manual must indicate how the plant's method was derived, by showing a clear relationship to its unique population of Trusses produced. The number of critical joints chosen by a Truss Manufacturer's design program (0.8 JSI and greater) can be influenced by voluntary Truss Manufacturer-driven design protocols affecting plate selection such as handling (on/off), selection of the C_q value to be used for calculation of JSI, selection of a design limit on JSI of less than 1.0, plates available in inventory, Truss type, and Truss span, all of which can influence an alternative joint sampling regimen. In addition, it is important to realize that one critical joint per Truss selected for inspection per operational set-up per shift on average is the minimum requirement as per the Standard and that it is recommended that as many critical joints be inspected as possible in order to improve the QC process, subject to the other constraints of the Truss Manufacturer.

§3.7.2 Plate Positioning Procedures.

The purpose of any quality assurance protocol is to consistently manufacture a product that will perform structurally as intended by its design. Using a Joint QC Detail provides a quick means to visually inspect a plate's position, rotation, embedment, and member-to-member joint gaps, while ensuring that any joint within these criteria sufficiently meets the Truss design requirements. Further, the use of the fabrication tolerance permits plate sizing with a customized degree of conservatism built into the design phase to account for increased quality imperfections in the plate contact area when needed. Specifically, this fabrication tolerance affects tooth holding values to account for a loss of effective Teeth in the plate contact area due to wood or joint characteristics or plate misplacement. Since this potential loss of effective Teeth is accounted for in the design, it reduces the time consuming task of determining the actual number of effective Teeth available at each plate contact area for assurance that the required number of effective Teeth has been met.

A prior investigation of the influence of QC on Truss performance demonstrated that more highly stressed joints

have the greatest influence on Truss structural capacity.⁶ This finding implies that it is not necessary to conduct detailed quality inspections on less highly stressed joints that are unlikely to influence Truss failure. More highly stressed joints that are critical to the Truss performance are defined in the Standard as those joints having a JSI of 0.80 or higher, which indicates the plate is stressed to 80 percent or greater of its design capacity. Thus inspections are focused on joints with a JSI of 0.80 or greater, per the procedures of Section 3.7. For joints with a JSI of less than 0.80 or which are otherwise not inspected in detail, it is still necessary to ensure that all plates conform to the gauge, type, and size indicated on the Truss Design Drawing, and that no gross manufacturing inaccuracy is evident. Thus, a preliminary assessment of the overall Truss would first be required with regards to the general plating, lumber, and assembly requirements per Sections 3.4 through 3.6.

§3.7.2.1 Plate Placement.

A Joint QC Detail is a visual aid for determining acceptance of a Metal Connector Plate's position at a joint relative to its design position and the allowable plate positioning tolerance as calculated by the Truss Design Engineer/Truss Designer. The Joint QC Detail graphi-

cally depicts the allowable plate positioning tolerance by illustrating the zone within which the center of the actual plate on the Truss must be located in relation to the Wood Members at the joint. Simply stated, the tolerance polygon shows where the plate's midpoint can and cannot be placed. At a minimum, the allowable placement tolerances will be provided in two perpendicular directions with positive and negative distances based off of the center point of the Truss plate. The significance of showing only the area within which the center of the plate must be located, rather than the entire plate, is that it enables a full-scale drawing to be printed on an 8-1/2 x 11 in. piece of translucent paper or vellum that can then be placed over the actual Truss joint in accordance with the reference lines of the Wood Members. This provides a simple visual aid to determine whether the plate is located within design tolerance. To better understand the updated placement procedures in the 2007 Standard, review Figure C3.7-1.

First, if no Joint QC Detail exists, then follow Section 3.7.2.2 to check if the actual plate midpoint is within 1/2 in. of the specified location and that each member in the joint has enough effective Teeth. If there is a Joint QC Detail but the Joint QC Detail contains no polygons and

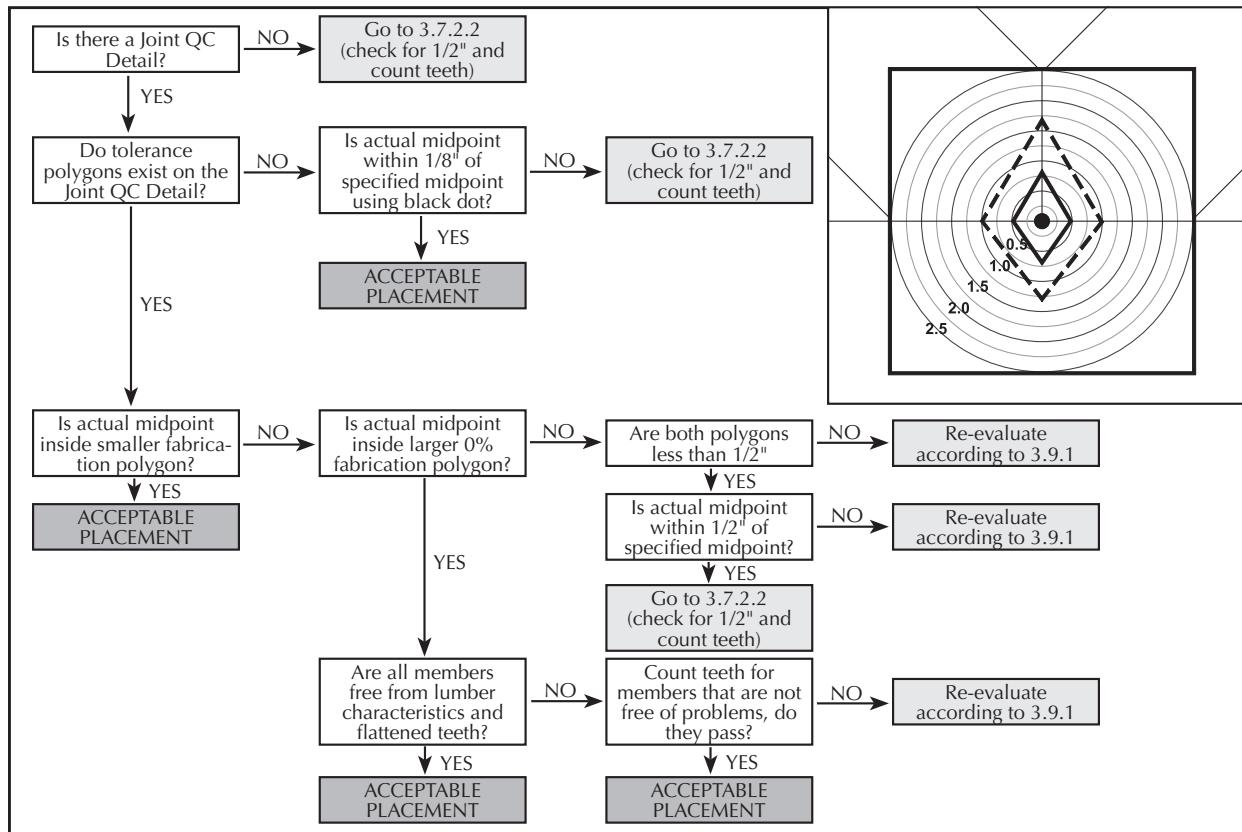


Figure C3.7-1. Updated Placement Procedures.

the actual midpoint is within $\frac{1}{8}$ in. (3 mm) of the specified midpoint, the placement shall be considered acceptable. If placement is not within $\frac{1}{8}$ in., then follow Section 3.7.2.2. If placement is within the smaller tolerance polygon (generated by the manufacturer-defined fabrication tolerance, typically greater than 0 percent), then placement is acceptable. If all members of a joint selected for inspection are free from characteristics reducing the plate contact area, consisting of lumber characteristics and flattened Teeth as outlined in Section 3.7.4, the 0 percent fabrication tolerance polygon can be used to evaluate plate positioning. If any member contains characteristics reducing the plate contact area, the 0 percent fabrication tolerance polygon shall be permitted to be used provided that Teeth are counted and equal or exceed the minimum required number of Teeth for that member. If position tolerance polygons are smaller than $\frac{1}{2}$ in. (13 mm) as measured from the designed center of the joint and the actual plate midpoint falls outside of both polygons but within $\frac{1}{2}$ in. (13 mm) of the specified midpoint, it shall be permitted to evaluate the plate per Section 3.7.2.2. If the actual midpoint does not meet any of these requirements, the procedures set forth in Section 3.9.1 shall be followed.

See also the discussion in Section §3.7.2.2 pertaining to when slots must be prohibited from occurring over the joint line between Wood Members.

§3.7.2.2 Alternative Positioning Procedure.

This alternative procedure is new to the Standard, although it is similar to the method formerly known as the Tooth Count Method in the 2002 edition of the Standard, except that it is presumed the plate placement polygon is the minimum size possible. The option permits plants to not print the Joint QC Detail, and simply check that the specified midpoint is within $\frac{1}{2}$ in. of the specified midpoint and that each member of the joint has enough effective Teeth.

This $\frac{1}{2}$ in. value assigned for the maximum tolerance was established by *TPI 1* based on connection failure modes other than lateral resistance (tooth holding), thus a separate evaluation of lateral resistance (tooth count) is also needed. These other failure modes are steel fracture and wood fracture at the connection. The use of this value is dependent upon the use of design values that comply with the provisions of Chapter 5, including steel design values based on the minimum cross-section of the steel. Steel design values for some Metal Connector Plates have been established from non-standard tests that locate the solid steel cross-section of the connector plate at the joint. This non-standard positioning can increase the steel capacity by 20 percent, or more in some cases,

and is only appropriate for plates that are so positioned in use. However, typical plates have the minimum steel cross-section located very close, usually within $\frac{1}{4}$ in. or less of any solid steel location, thus the use of a $\frac{1}{2}$ in. tolerance is not acceptable for such plates as it can result in the minimum steel cross-section at the joint with the corresponding loss in steel strength. In situations where non-standard steel design values are used in design, the Truss Designer should specify that Section 3.7.2.2 not be applied and plate placement polygons on Joint QC Details should be limited to avoid permitting plate slots over a joint line.

§3.7.3 Plate Rotation.

The separate maximum plate rotation specified in Section 3.7.3 is a form of the quality criterion that was new to the 2002 edition of the Standard and carries on in the 2007 edition. The plate rotation limit was added as part of the new provision for plate positioning in Section 3.7.2 since plate translation only pertains to the plate center, whereas previous plate placement tolerances in earlier editions of the Standard and its predecessors applied to all locations on a plate and thus addressed both plate translation and rotation together. The ten-degree allowable plate rotation specified in Section 3.7.3 is explicitly considered in the Metal Connector Plate joint design, per Section 8.11.5. When inspecting a plate for rotation, the ten-degree maximum limit can be approximated by deviating a maximum of $\frac{1}{2}$ in. from a given reference line (e.g., Chord edge) along any 3 in. plate length. Hence, all points along an edge of a 3-in. long plate intended to be oriented horizontally should not deviate more than $\frac{1}{2}$ in. from a horizontal line through any point along that edge, and a 6-in. plate should not deviate more than 1 in. Alternatively, the Joint QC Detail that is used to inspect the plate positioning tolerance may also be used to check plate rotation with the inclusion of \pm ten-degree angle reference lines. The 2007 edition of the Standard added that if the joint is outside the rotation tolerance, it will need to be re-evaluated or repaired. Furthermore, if the plant decides to check for the $\frac{1}{2}$ in. placement tolerance and count Teeth, they can use the ten-degree tolerance for every joint regardless of specific plating or loading conditions of that joint, unless otherwise indicated on the Truss Design Drawing or specified in their Quality Control Manual. For example, if the standard rotational tolerance specified by the fabricator for design purposes is five degrees, then only a five-degree tolerance is permitted.

§3.7.4.1 Lumber Characteristics and Tooth Flattening.

If the Truss Manufacturer uses a fabrication tolerance greater than 0 percent, it will be allowed to evaluate each

member's plate contact area to make sure the combination of lumber characteristics and tooth flattening does not exceed the tolerance for each member, as shown on the Joint QC Detail. This is accomplished by displaying a circle for each Truss member contact area where the area of the circle is equal to the tolerance percentage of the contact area. Lumber characteristics include, but are not limited to, loose knots, decayed knots, unsound wood, bark, pitch pockets, holes, and wane. For example, a 0 percent fabrication tolerance assumes that 100 percent of Teeth in the plate contact area are effective. A 20 percent fabrication tolerance assumes that 20 percent of Teeth are in a characteristic reducing the plate contact area. See Figure 3.7-2 in the Standard for an example (see Section §3.3.2 for more information on the fabrication tolerance).

§3.7.5 Plate Embedment.

The effectiveness of a Metal Connector Plate to provide lateral resistance at a joint depends on the amount of embedment of the plate Teeth into the Wood Members. The greater the gap there is between the underside of the plate and the surface of the Wood Member, the less the resistance provided by the plate Teeth. Predecessor standards to *TPI 1* reduced lateral resistance strength by ten percent to account for this and other quality characteristics at joints, based on provisions limiting embedment gaps to $\frac{1}{16}$ in. maximum values and occurring over only one-third of the area of a plate. Due to a change to permit $\frac{1}{32}$ in. uniform embedment gap over the entire plate area, and as a result of a study that found a 16 percent reduction in lateral resistance strength of Metal Connector Plates with an average uniform embedment gap of $\frac{1}{32}$ in., the 1995 edition incorporated a 0.84 strength reduction factor into the allowable plate lateral resistance design values.⁷ Thus, all plate lateral resistance design values include a built-in reduction that accounts specifically for an anticipated plate embedment gap up to $\frac{1}{32}$ in., which is a reasonable gap size to expect from typical Truss production. This strength reduction factor remains included in the 2007 edition, which means that a plate embedment gap of $\frac{1}{32}$ in. or less will not adversely affect the plate's lateral resistance design strength.

One type of observed embedment gap results from a certain lumber phenomenon rather than from a manufacturing inaccuracy. This phenomenon is often termed lumber "cushioning," "cupping," or "cavitation," and is characterized by the indentation of the lumber in the center of the plate area due to the compressive force exerted by the Teeth during pressing. This is more common with

the denser woods, due to their greater resistance to tooth embedment. Because the gap resulting from this lumber effect occurs predominately in the center area of the plate and cannot be practically controlled by the manufacturing process, the maximum plate embedment gap provision in Section 3.7.5 is limited to the plate perimeter only. The effect of this particular type of embedment gap on the interior Teeth is considered to be offset by the expected increase in grip strength due to the high density of the lumber. Furthermore, it is expected that any manufacturing inaccuracies resulting in insufficient plate embedment will be revealed at the perimeter of the plate, and thus an inspection of the plate embedment at the plate perimeter provides adequate assurance of the overall plate embedment.

Another possible lumber-related cause of embedment gaps under the plate is the occasional variation in lumber thickness of different Wood Members at a Truss joint. The uneven joint line created by different lumber thickness can prevent full embedment of at least some of the plate Teeth near the joint line. In recognition of this, and to allow for the occurrence of small amounts of unevenness in Wood Member thickness at the joint while still permitting acceptance of a properly embedded plate, the maximum plate embedment gap provision is excluded from plate perimeters within 1 in. of any Wood Member-to-Wood Member joint line.

§3.7.5.2 Excessive Embedment Gap Tooth Count.

If a plate exhibits a perimeter embedment gap that exceeds $\frac{1}{32}$ in., which is the gap size that is accounted for in the design process, then an additional assessment is necessary in order to determine whether the plate contact area has enough extra Teeth to offset the reduction in tooth effectiveness and still provide sufficient lateral resistance strength. This must be accomplished by performing tooth counts for members in question.

§3.7.6.1 Tolerance.

The gap criteria beginning in the 2002 edition of the Standard was simplified from the gap criteria in earlier editions, which previously included criteria for average gaps as well as maximum gaps for different joint types subject to tension or compression. The change in the 2002 edition, also incorporated into the 2007 edition, was made for ease of measurement, which is important to the effectiveness of the criteria to be properly implemented. Also, most joints transfer both tension and compression forces, depending on the load case being considered, and it is not practical for the Truss Manufacturer or quality inspector to distinguish "tension joints" from "compression joints." The average and maximum allowable gaps have been combined into a single specified maximum allowable

⁷ M. H. Triche, *Partial plate embedment study*, Report to Truss Plate Institute, University of Alabama, February 2, 1994.

gap, which is more explicitly defined as the gap present at the edge of the plate, when the gap is within the scarf, or the gap present at the end of the scarf, when the plate edge is outside of the scarf. In addition, the maximum allowable gap criterion now only distinguishes floor Truss Chord splices from all other joint types, and a tighter gap allowance is specified for floor Truss Chord splices due to the greater significance of Chord splice gaps on floor Truss deflection and serviceability.

§3.7.6.2 Compression Load Tolerance.

Since Metal Connector Plates are not generally designed to carry compression load at a joint, the amount of gap between Wood Members must be limited so that the plate buckling or slip necessary to close the gap is small and will not cause joint failure. If the Truss plates are designed to carry the load, however, then gaps consistent with those used to establish the Truss plate compression capacity should be permitted. The Standard does not currently address how to establish compression capacity of Truss plates for given gap sizes, however, the Truss Designer may still elect to design the Metal Connector Plate for compression by testing and the use of plate-buckling analysis methods. Another consideration that the Truss Designer must account for when gaps are present is the effect of the gap on the shear capacity of the plate, since shear tests are determined using zero gap between Wood Members per Section 5.3 and this capacity may be reduced when gaps are present due to the propensity for buckling of the steel plate when gaps are present (e.g., see limits on web shear for cold-formed structural steel beams⁸).

§3.7.6.3 Correction Procedure.

The use of galvanized metal shims, or other acceptable alternates, to obtain firm bearing between members with gaps has been permitted in published quality criteria for metal-plate-connected wood Trusses since 1986. Since the exposed metal connector Teeth at the gap can inhibit the installation of the shim, the shim is permitted to be narrower than the width of the gap, with the minimum shim width of $\frac{3}{4}$ in. selected so that, in 1-1/2 in. thick lumber, it can be driven in without contact with the plate Teeth.

§3.7.7 Effective Tooth Count.

If the critical joint being inspected has an area of lumber characteristics or flattened Teeth greater than the area of the fabrication tolerance circle, or if the plate has more than $\frac{1}{32}$ in. perimeter embedment gap, or if the 0 percent

fabrication tolerance polygon is used, the inspector can count Teeth. This is a more precise examination of the joint. The method is to determine the actual number of effective Teeth in each plate contact area, which must meet or exceed the minimum number of required effective Teeth specified for the Truss design. The determination of the number of effective Teeth in a plate contact area requires not only a tally of the Teeth found in the contact area, but also an assessment of the effectiveness of those Teeth, which depends on the amount of embedment gap, if any, and the elimination of any Teeth from the tally that are flattened or are located in a lumber quality characteristic.

Metal Connector Plate joint design includes a calculation of the minimum required plate area (for each member), or the equivalent number of effective Teeth, necessary for the lateral resistance force transfer. Quality assurance of a joint requires confirmation that the plate, as pressed, provides enough effective Teeth in each plate contact area as required per the design, after consideration has been given to any manufacturing and/or lumber quality characteristics in the plating area and to the effect of any plate embedment gaps between the underside of the plate and the wood surface.

Similar to the 2002 edition of the Standard, the 2007 edition contains a provision that is revised from the 1995 edition, which simply required both plated faces at each plate contact area to meet or exceed the required number of effective Teeth specified by the Truss Design Engineer/Truss Designer for a single plated face. The provisions in the 2002 and 2007 editions provide explicit tolerance for variation between the two faces, recognizing that the forces acting at a Metal Connector Plated joint are resisted by the combined effort of two opposing plated faces. Thus, excess Teeth on one plated face will make up for a shortage of Teeth on the opposite plated face. The maximum 15 percent variation permitted with this provision is based on engineering judgment to assure that the load transferred on each face of a member does not become too unbalanced.

§3.7.7.2.1 Ineffective Teeth – Lumber Characteristics.

The 2007 edition of the Standard, similar to the 2002 edition, distinguishes between loose knots, in which Teeth are considered to be ineffective, and tight knots, in which Teeth are considered to be effective. The 1995 edition considered Teeth in all knots to be ineffective. A loose knot is one that is not held firmly in place by growth or position and cannot be relied upon to remain in place; a tight knot is a sound knot so fixed by growth or position that it will retain its place in the piece. This distinction is

8 North American Specification for the Design of Cold-Formed Steel Structural Members, AISI/COS/NASPEC 2001, American Iron and Steel Institute, 2001.

supported by testing in which Teeth embedded in tight knots have shown adequate tooth holding strength. In addition, there are examples of this allowance for tight (or live) knots in other countries, provided that the Teeth or nails can be satisfactorily embedded without distortion or splitting of the timber.

§3.7.7.2.3 Ineffective Teeth – Tooth Flattening.

A flattened tooth occurs when the tooth is bent under the plate as the plate is pressed into the wood. Since a flattened tooth is not properly embedded into the wood, it is considered not effective in contributing to the lateral resistance of the plate. It is assumed that if one tooth of a tooth slot is flattened, as visible through the tooth slot opening, the other tooth will likewise be flattened in the same direction but will not be visible through the tooth slot opening. Thus, the number of visible flattened Teeth must be doubled when determining the number of ineffective Teeth. A “flattened” tooth is defined as a tooth that has a visible length inside the tooth slot of one-fourth or greater of the total length of the tooth, as shown in Figure 3.7-7. Thus, if the actual tooth length is $\frac{3}{8}$ in., any visible tooth length of $\frac{3}{32}$ in. or greater is considered flattened. Any single flattened tooth in this member (i.e., where only one tooth in a slot is actually in this member and the other tooth may be on the other side of the joint or in a gap) shall be so designated.

§3.7.7.2.4 Ineffective Teeth – Embedment Gap.

It is recognized that certain lumber and manufacturing characteristics can impede the full embedment of a Metal Connector Plate into the Wood Members. Wood with higher density is more prone to partial plate embedment, and studies at several manufacturing facilities have shown that embedment gaps are almost non-existent in lower density Spruce-Pine-Fir lumber, as compared to higher density Southern Pine lumber.

Tooth Effectiveness Ratios, as a function of plate embedment gap sizes, were first included in the 1995 edition of the Standard to enable partial plate embedment to be evaluated based on the embedment of individual Teeth. These Effectiveness Ratios improved upon earlier plate embedment criteria, which permitted only one-third or less of the required plate contact area to exhibit a gap of up to a maximum $\frac{1}{16}$ in. Moreover, if a plate exhibited the maximum $\frac{1}{16}$ in. allowable embedment gap, none of the other allowable quality tolerances, such as plate location or tooth flattening, were permitted to occur at the same joint.

During the development of the 1995 edition of the Standard, a research project reported a strength ratio of 0.84 for Metal Connector Plates with uniform embedment gaps

of $\frac{1}{32}$ in. to Metal Connector Plates without embedment gaps, based on the Southern Pine data.⁹ A $\frac{1}{32}$ in. gap was considered to be a reasonable quality tolerance attainable for a wide variety of Truss manufacturing facilities, and thus a strength ratio of 0.84 was incorporated into the determination of basic allowable lateral resistance values in order to account for this amount of gap in all joints. The 0.84 strength ratio replaced an earlier strength ratio of 0.90 intended to account for manufacturing characteristics. This shifted the divisor used to convert average ultimate (test) strengths to allowable lateral resistance strength from 3.0 in earlier editions of the Standard to 3.2 in the 1995 edition ($3.0 \times 0.90/0.84$), to account for the maximum embedment gap of $\frac{1}{32}$ in. as the baseline for plate strength, or for plate Teeth having full (100 percent) lateral resistance design strength.

A single allowable maximum embedment gap does not take into account over-plating (i.e., specifying a plate that provides more Teeth than the minimum number required per design) or the effectiveness of the remainder of the Teeth if only a portion of the Teeth have an embedment gap exceeding the maximum allowable gap. Plate strength versus plate gap data from a variety of sources has shown that plates with $\frac{1}{16}$ in. embedment gaps have 50-60 percent of the strength of a fully embedded plate, and suggests an exponential curve that, of course, approaches zero as the embedment gaps increase.^{10, 11, 12, 13} These additional studies were used to establish the strength ratings of 60, 40, and 0 percent for incrementally increasing gaps greater than $\frac{1}{32}$ in., relative to the $\frac{1}{32}$ -in. baseline that already incorporates a 16 percent strength reduction. Hence, an average reported strength reduction of 52 percent for a $\frac{1}{16}$ -in. embedment gap converts to a strength reduction of 0.52 / 0.84, or 60 percent conservatively, relative to the 100 percent baseline. With the incorporation of the tooth Effectiveness Ratios for varying gap sizes, the 1995 edition of the Standard introduced the method of calculating adjustments to the tooth count in a plate contact area based on embedment gaps.

A change in the 2002 and later editions of the Standard was made to recognize an effectiveness rating of 119 per-

9 Triche.

10 G. J. Klein, *Effect of partial embedment on the strength of plate-connected joints*, Presentation to the TPI Technical Advisory Committee, Wiss, Janney, Elstner Associates, Inc., May 1985.

11 G. J. Klein and R. J. Kristie, *Inspection and repair of plate-connected wood trusses*, Proceedings of the 1988 International Conference on Timber Engineering, Seattle, WA. Vol. 1, 449-459, 1988.

12 J. Ljorring and J. Adelholj, *Fastening of punched metal plate fasteners*, Technological Institute, Taastrup, Denmark, February 1989.

13 MiTek Industries, Inc. Communication to Truss Plate Institute: 1994.

cent for an embedment gap of 0 in. This change recognizes that a joint designed with plate design values that incorporate a 0.84 strength ratio for an assumed gap of $\frac{1}{32}$ in. will have 1/0.84, or 1.19 times more lateral strength than assumed in the design process if the plate is fully embedded and exhibits no embedment gaps. This recognition can be useful from a quality assurance standpoint, if a fully embedded plate is shifted and, consequently, is found to have less than the required number of effective Teeth, per the original design in one of the plate contact areas. If, for example, the required number of effective Teeth for the plate contact area is 50, assuming the 16 percent strength reduction resulting from a $\frac{1}{32}$ -in. gap, then a fully embedded plate contact area with 42 Teeth (50×0.84) will provide the required strength, and thus is acceptable from a quality assurance standpoint.

To measure tooth embedment gap for a determination of tooth effectiveness, a depth gauge with $\frac{1}{64}$ in. increments may be used as follows. First, set the depth gauge to $\frac{1}{32}$ in., plus the thickness of the plate (obtain thickness by measuring a free plate). Count the number of slots where the depth gauge does not touch the lumber, or where the surface of the wood cannot be felt when the gauge is slid back and forth on the Truss plate. For plates with two Teeth per slot, multiply the number of slots by two to get the number of Teeth, and record the total number of Teeth found exceeding the measured gap size. Repeat this for $\frac{1}{16}$ in. and $\frac{3}{32}$ in. The number of Teeth counted in the larger gaps will include those Teeth recorded for the smaller gaps, as shown in Figure C3.7-2.

When calculating total tooth ineffectiveness using the formula above, subtract from the number of Teeth found for each gap size the number of Teeth counted for the next larger gap size increment. For the example shown in Figure C3.7-2, the number of Teeth with gaps $> \frac{1}{32}$ in. but $< \frac{1}{16}$ in. is $8 - 4 = 4$, the number of Teeth with

gaps $> \frac{1}{16}$ in. but $< \frac{3}{32}$ in. is $4 - 2 = 2$, and the number of Teeth remaining with gaps $> \frac{3}{32}$ in. is 2. Once a tooth is considered ineffective, it is not counted again for another quality imperfection.

For example, if a tooth is located in a lumber quality characteristic that would render it ineffective, shows evidence of tooth flattening, and exhibits a $\frac{1}{16}$ -in. plate embedment gap, it would only be counted as an ineffective tooth once due to being located in the characteristic. If necessary, the increased effectiveness of fully embedded plate Teeth exhibiting no embedment gap could also be taken into account. Thus, after calculating total effective Teeth as previously shown, the number of Teeth with full embedment (i.e., no embedment gap) can be subtracted from the total effective Teeth, multiplied by 1.19, and added back to get a new (i.e., increased) total effective Teeth.

3.8 REPRESSING

As discussed in Section §3.7.5, the ability of Metal Connector Plate Teeth to provide lateral resistance, or grip, at a joint depends on the Teeth having adequate embedment into the Wood Members. An embedment gap of up to $\frac{1}{32}$ in. is accounted for in plate lateral resistance design values through the incorporation of a built-in strength reduction factor, but gaps larger than $\frac{1}{32}$ in. reduce the lateral resistance strength of the Teeth, which can result in a plate contact area not having enough grip strength as required by design if there are not enough extra Teeth to compensate for the reduced strength per tooth.

Because the goal is full embedment, it is logical to permit a plate with an embedment gap due to a manufacturing inaccuracy to be repressed in order to achieve full tooth penetration, thereby ensuring that the joint has or exceeds the tooth holding strength assumed in the design. The repressing allowance in this section is intended to address

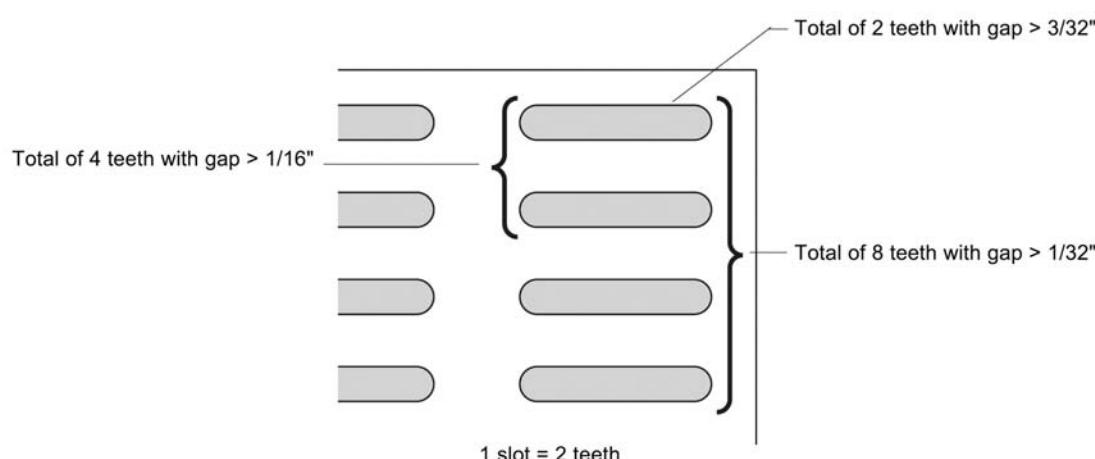


Figure C3.7-2. Methodology for Counting Teeth with Embedment Gaps.

quality deficiencies resulting from the manufacturing process; however, additional pressing of a plate after manufacturing, but before the Truss is in service, is also reasonable to increase a joint's lateral resistance strength, provided the wood is sound and not visibly discolored by weathering.

It should be noted, however, that there may be a difference between plates that were not fully embedded during Truss fabrication and plates that were fully embedded but have "backed out" or withdrawn from the wood for some reason such as weathering, lumber drying, lumber swelling and shrinking resulting from moisture cycling, or lateral resistance failure. There is data showing reduced plate strength for plates reinstalled into lumber that had previously been loaded and had plates withdraw from the lumber (see Section 3.9.3). Based on this data, it is not considered reasonable to repress a Truss that has had plates withdraw for any reason, including those that may have been temporarily stored and then exhibits plates that have "backed out" or withdrawn from the wood.

§3.10 GIRDER TRUSS PLY-TO-PLY CONNECTION

This section was added in the 2007 edition of the Standard to address concerns of overly conservative field inspection practices. It had been common practice of some inspectors to presume that ply-to-ply nailing was inadequate if nail heads were not visible on both sides of the multiple-ply girder, but this is not appropriate for multiple-ply girders that are nailed together while laying on a table without turning the multiple-ply Truss over. In the latter case, nail heads will only be visible from one face of the multiple-ply Truss, making complete inspection after the nailing process difficult. However, this nailing practice is desirable for both labor efficiency (economy) and construction safety, and can be assured as adequate by appropriate monitoring through an in-plant inspection process. This provision requires consideration by the Truss Manufacturer's In-Plant Quality Assurance Program's QC manual in order to address the inspection concerns.

§CHAPTER 4

METAL CONNECTOR PLATE MANUFACTURING

§4.3 PRODUCTION STEEL

As part of its mission to maintain the Truss industry on a sound engineering basis, the Truss Plate Institute provides a voluntary steel verification program for Metal Connector Plate manufacturers to assure that Metal Connector Plates used in wood Truss construction are manufactured exclusively from structural grades of steel. Further information on this program is available at: www.tpinst.org/Steel_Verification.html.

§4.3.3.2 Hot Dip Galvanized Steel.

The referenced ASTM specification of *ASTM A653*, like those cited elsewhere in the Standard, is the applicable specification that was current at the time of the development of this edition of the Standard. Construction specifications for wood Trusses with Metal Connector Plates are sometimes encountered that reference withdrawn ASTM standards for hot-dipped galvanized steel, namely *ASTM A446* [standard specification for steel sheet, zinc-coated (galvanized) by the hot-dip process, structural (physical) quality] and *ASTM A525* [standard specification for general requirements for steel sheet, zinc-coated (galvanized) by the hot-dip process]. These standards were withdrawn in 1994 and replaced by *ASTM A653* on information specific to grade and galvanizing level and *ASTM A924* on information pertaining to general requirements. The structural grades formerly specified in *ASTM A446* and the galvanizing levels formerly specified in *ASTM A525* are now included in *ASTM A653*. Old specifications containing a reference to *ASTM A446* and/or *A525* should be updated to cite *ASTM A653* if the intent of the specification is to obtain new Trusses using hot dip galvanized steel. Specifications for grades characterized by a letter (e.g., Grade A) are also archaic as the letter designations have since been replaced by numeric designations corresponding to the material yield strength. For example, Grade A, which had a yield strength of 33 ksi (or 230 MPa for metric units) specified within *ASTM A446*, is now specified as Grade 33 (or Grade 230) within *ASTM A653* (termed *A653M* for metric).

§4.3.4 Steel Thickness.

Steel thickness is specified within this section on a coated basis for galvanized or aluminum-zinc alloy coated steel. There are other systems for designating gauges, and it must be recognized that the steel industry, including the Metal Connector Plate industry, does not make use of gauge designations for purposes other than descrip-

tive labels for products. Thickness must be specified by decimal number in units of inches, millimeters or similar units for purposes of ordering and engineering design. The thicknesses given in this section as corresponding to the given gauge designations of 20, 18, 16 and 14 gauge have been in use within the plated wood Truss industry for many years, and are therefore included within *TPI 1* to serve as a standard reference.

Measurement of coated thickness is acceptable for purposes of assuring adequate thickness of steel for structural purposes. Base steel thickness, exclusive of coating, is determined as the coated thickness minus the nominal thickness of the specified coating designation (see Section 6.3.4.1.3, or the appropriate reference specification for the coating type). Variations in actual coating thickness from the nominal coating thickness are small enough to not significantly impact the assurance of adequate performance. For example, the resolution to which production steel is required to be measured (0.001 in. per Section 4.6.2(c), which corresponds to just under three percent, at most, for typical thicknesses for Truss plates) is equal to the total thickness from both faces of a typical *ASTM A653 G60* galvanized coating thickness (nominal thickness of 0.0010 in.). In other words, a doubling (or 100 percent increase) of the coating thickness from the nominal thickness would result in a drop in base metal thickness equal to only the least resolution which could be identified by measurement, which in turn would cause an error of less than three percent. Variations in coated thicknesses are typically well under even this level.

§CHAPTER 5

PERFORMANCE EVALUATION OF METAL CONNECTOR PLATED CONNECTIONS

Overview of (non-editorial) Changes

New to TPI 1-2007

- 37) Wood properties (specific gravity, moisture content, moisture adjustments, etc.) for parallel to load members are no longer required to be measured in joints at the AE and EE orientations. (Section 5.2.4.2)
- 38) Alternatives to given values of End and Edge Distances now explicitly recognized (formerly only recognized in the *TPI 1 Commentary*). (Section 5.2.6.2.2)
- 39) Specified (versus measured) coating thickness is permitted to be used to determine uncoated steel thickness from measurements of coated steel thickness. (Sections 5.2.8.4 and 5.4.6.3)
- 40) Dimensional measurements of steel coupon thickness specified to 0.0001 in. (Sections 5.2.8.4 and 5.4.6.3)
- 41) Slight changes to steel shear tests to remove unnecessary limits (3 in. offset now permitted to be less, lumber can be ripped). (Section 5.3.4.2)
- 42) Test machine calibration certificate is now required to be reported for steel tensile testing. (Section 5.4.9(n))

§5.1.2.2 Testing Machine.

A constant rate of crosshead movement is important, as variation in the testing speed can increase the measured load. Commercial testing machines typically can satisfy this requirement as they are either screw-driven, which results in a constant crosshead speed, or hydraulic driven with a valve that controls the hydraulic flow rate, which can result in a constant speed.

§5.1.2.3 Hydraulically Driven Testing Machine.

Hydraulic machines that use valves for controlling the hydraulic pressure, which must be turned manually by the operator during the test, will only result in a constant speed if the operator increases the pressure linearly and if the Test Specimen behavior is linear. As tooth holding joints are typically non-linear with a great reduction in stiffness as the failure load is approached, use of machines with loading rate controlled by a pressure valve will typically result in a very fast rate of loading as stiffness decreases. This results in overestimates of the ultimate load of the Test Specimen due to this fast rate of loading as the joint approaches its ultimate capacity. For this reason, hydraulically driven testing machines controlled by a pressure valve are not permitted by Section 5.1.2.3.

§5.1.2.5 Measuring Devices.

The measuring devices are intended to measure slip between the plates and the Wood Members. The gauge length used between the attachment points for the measuring devices should be kept as small as possible to avoid excessive influence from the axial strain of the Wood Members.

§5.2 STANDARD METHOD OF TEST FOR DETERMINING LATERAL RESISTANCE STRENGTH OF METAL CONNECTOR PLATE TEETH

Integral to the establishment of the lateral resistance strength of Metal Connector Plate Teeth is the quality criteria for the manufacture of metal-plate-connected wood Trusses (see Chapter 3). The evaluation criteria for lateral resistance strength provides for a composite strength divisor of 3.2, which adjusts for quality tolerances pertaining to partially penetrated Teeth, strength variability, duration of load, and a safety factor (see Section §5.2.9.2).

§5.2.2.1 Test Specimen Selection.

Earlier upper limits on steel sampling, of 7 ksi above the minimum specified yield strength and five percent above the minimum specified thickness, were established due to the belief that these variables may affect lateral resistance (tooth holding) strength. Steel thickness affects the stiffness of Truss plate Teeth, which may affect tooth holding strength and slip parameters. Steel yield strength affects the level at which a plastic hinge may form in the Teeth, which may affect tooth holding strength, and joint slip parameters at high loads. There has been little study of the effect of these steel properties on plate tooth holding properties, but available information indicates these properties can affect tooth-holding strength, with higher values being obtained for greater thicknesses and yield strengths. For this reason, limits were selected to minimize variation due to greater-than-minimum-specified steel properties, while still permitting "typical" steel to be used for test samples. It should be noted, that while the limits of 7 ksi on yield strength and five percent on thickness were considered to be a suitable balance of these two objectives, the resulting variation in tooth holding properties permitted within these limits is not known.

The 2002 and later editions of the Standard include adjustments for when these limits are exceeded, because it is recognized that steel used for the production of Truss plates often does not fall within this range. Typical grades

of steel, such as *ASTM A653 SS* Grades 33, 37 and 40 (previously known as *ASTM A446* Grades A, B and C), often vary up to 20 ksi above the minimum specified yield strength. Similarly, steel thickness tolerances may vary up to 20 percent above minimum.¹ Without the use of an adjustment, the steel tolerances of 7 ksi on yield strength and 5 percent on thickness would exclude a large proportion of the steel received for production of Truss plates from being used to manufacture plates for testing in accordance with Section 5.2. For example, one Truss plate manufacturer reported only 23 percent of the *ASTM A653 SS* Grade 37 steel coils received in one year were within 7 ksi of the specified yield strength. This percentage is likely to vary, both upward and downward, depending on the specified grade of steel and the steel manufacturer. A similar percentage of typical steel supplied for plate manufacture may be expected to meet the steel thickness tolerance of five percent. The interaction between these two restrictions would leave very few steel coils acceptable for production of test plates. For this reason, the adjustments for plate lateral resistance strength, when either or both of these limits are exceeded, were added to the 2002 edition and maintained in the 2007 edition of the Standard.

The adjustment given in Equation E5.2-1 to account for steel yield strength is based on two sets of test data using 20 gauge plates, one in Southern Pine ($G = 0.58$) and one in Norway Spruce ($G = 0.41$). Each set of tests consisted of five matched pairs of joints, with one joint of each matched pair tested with plates made from *ASTM A653 SS* Grade 37 steel ($F_y = 37$ ksi), and the other joint of each matched pair tested with identical plates made from *ASTM A653 HSLAS* Grade 60 (previously known as *ASTM A816* Grade 60) steel ($F_y = 60$ ksi). A regression equation was fit to the test data, resulting in the adjustment specified. Although the steel thickness of the Grade 60 plates was five percent larger than that of the Grade 37 plates in both sets of tests, the increase in tooth holding strength was attributed solely to the change in yield strength. This is conservative, which is appropriate given the limited number of tests.

$$R_Y = (F_{y,spec} / F_{y,test})^{(1.2*G_{test} - 0.4)} \leq 1.0 \quad (\text{E5.2-1})$$

Note that Equation E5.2-1 includes the effect of specific gravity, G . The variation in the equation due to specific gravity matches expectations, i.e., plates tested in low density wood are more likely to yield due to wood fiber yielding, which is unrelated to steel yield strength,

while plates tested in high density wood are more likely to slip due to inelastic bending of the steel tooth, which is a function of steel yield strength.

The adjustment given in Equation E5.2-2 to account for steel thickness was empirically derived from a comparison of data from published model Building Code evaluation reports of 18 gauge and 20 gauge plates that were otherwise identical. This comparison showed tooth holding design strength increases for an 18 gauge plate, relative to a 20 gauge plate, between 0 and 31 percent. The adjustment is based on an empirical fit towards the conservative end of this range, i.e., assuming a 20 percent increase in strength for 18 gauge plates, relative to 20 gauge plates.

$$R_T = (t_{spec} / t_{test})^{(0.7)} \leq 1.0 \quad (\text{E5.2-2})$$

The adjustments in Equations E5.2-1 and E5.2-2 were originally provided in the Commentary to the 1995 edition of the Standard. While they are based on limited data, they are considered to be reasonable, i.e., conservative to accurate. Note that the Standard provides no limitation on the application of design values established from tests of plates of a given thickness and yield strength to plates of greater thickness and/or greater yield strength that are otherwise identical. For this reason, it is permitted to apply design strengths resulting from tests of 20 gauge plates of a given grade of steel to plates made of 18 gauge steel or plates made of a higher yield strength of steel using the same tooling (presuming the tooling is capable of accommodating the increased thickness while still meeting all manufacturing requirements of Chapter 4).

§5.2.2.2 Cleaning Plates.

The provision to degrease plates serves to avoid variability that may occur due to solvents used during manufacturing, which are typically volatile and therefore evaporate prior to use. Testing of creosote-treated wood has shown that this oil-based substance increases tooth holding strength,² so it is unlikely that removing such solvents from plates increases tooth holding strength.

§5.2.2.3 Test Specimen Design.

The intent of this section is to require the plate length to be such as to produce tooth withdrawal failure, and for the AA and EA orientations, the tooth withdrawal failure should occur possibly from either Wood Member. Thus, the plate length should be symmetrically located on the Wood Members on the AA and EA orientation specimens, as shown in the figures for Test Specimen fabrica-

¹ T. D. Skaggs, F. E. Woeste and S. L. Lewis, *Steel Properties used to manufacture wood truss metal connector plates*, Transactions of the ASAE, American Society of Agricultural Engineers, Vol. 38(1), 187-195, 1995.

² M. G. Oliva, L. Krahn, M. McCarthy and M. Ritter, *Behavior of metal-plate connected joints in creosote treated wood: a pilot study*, Forest Products Journal, Vol. 38(7/8), 76-80, 1988.

tion, Figures 5.2-3(a) and (b) and 5.2-4(a) and (b). Plates should not be offset on these specimens, which would force failure in one Wood Member and prevent failure in the other Wood Member.

§5.2.4.1 General.

There is no upper limit on the maximum moisture content since high moisture content (or changes in moisture content) will result in lower strength, which is more conservative. The moisture content requirement that is appropriate for Structural Composite Lumber (SCL) was added in the 2002 edition of the Standard, because the limit of 11 percent is not consistent with the moisture content of this type of product as compared to solid sawn lumber. SCL is typically produced to moisture contents of seven to nine percent and the expected service range for SCL is seven to 15 percent.

§5.2.4.2 Test Specimen Characteristics.

New to the 2007 edition of the Standard, wood properties (specific gravity, moisture content, moisture adjustments, etc.) for parallel to load members are not required to be measured in joints at the AE and EE orientations [see Figures 5.2-3(c) and (d) and 5.2-4(c) and (d)]. This is because the AE and EE joints are tested only to establish the strength of the plate Teeth within the member perpendicular to the load.

§5.2.6.2 Assembly.

The effects of End Distance and Edge Distance are two separate effects that are unrelated to each other. End Distance effects occur when Teeth are close to the joint along the grain and tear through the wood parallel to grain at a lower load (per tooth) than is required to withdraw these Teeth from the wood. Edge Distance effects refer to the decreased friction between the tooth and the surrounding wood fibers that may occur when the tooth is driven near the edge of the lumber, relative to being driven away from the edge of the lumber. The Edge Distance effect is due to wood's tendency to split along the grain.

New to the 2007 edition of the Standard, alternatives to given values of End and Edge Distances are now explicitly recognized (formerly only recognized in the Commentary).

The proportion of Teeth affected by End and Edge Distance effects in tooth holding test joints can be quite large compared to typical Truss joints, since precluding steel fracture requires tooth holding tests be done with small plate sizes. For this reason, the small plate sizes can result in unrealistically low values for tooth holding strength due to the effects of End Distance and Edge Distance. This occurs with the Gross Area Method, in which

Teeth must be present in End and Edge Distance zones. Given the small size of plates that must be used due to steel strength constraints, design values established by the Gross Area Method are conservative when applied to larger plate areas than that tested. This is recognized by permitting tests to be done with Teeth removed from End and Edge Distance areas so that the tooth holding tests result in the strength required to actually withdraw the Teeth from solid wood. This method is called the Net Area Method, which requires that no Teeth be in the End and Edge Distance zones when the plate is tested. The Net Area Method results in higher design values on a unit area basis than the Gross Area Method, due to the removal of Teeth from the tested area that may be affected by End or Edge Distances. However, these higher design values cannot be applied for design purposes to any Teeth within the given End or Edge Distances on a joint (see Section 8.3.2.1).

The Standard defines that End Distance be $\frac{1}{2}$ in. and Edge Distance be $\frac{1}{4}$ in. for the Net Area Method. The particular distances defined for End and Edge Distance in the tests must be used for joint design when using design values established by the Net Area Method. However, there is no particular interest in the specific values given by the Standard, other than they were considered to be the End and Edge Distances beyond which there is no change in tooth holding values due to End and Edge Distance effects. From the standpoint of joint performance, any other values may be used provided that they are used both in the testing to establish design values and in the joint designs using the resulting design values. Thus, it is possible to use different End Distances and Edge Distances. For example, a different End Distance should be used if tooth spacing is other than $\frac{1}{2}$ in. because End Distance should be a multiple of tooth spacing for those Teeth.

Virtually all methods for rating Truss plate tooth values are conservative. Gross Area values are only accurate for the same size plate tested and are conservative for all larger plate sizes due to the high proportion of end- and edge-distance-affected Teeth in Gross Area Method tests. Net Area design values cannot be applied to Teeth in End and Edge Distance zones, even though those Teeth exist in all joints and provide some additional joint strength, so this method is conservative for designing small plate areas, but approaches accurate designs for larger plate areas where the End and Edge Distance areas are small relative to the total plate area.

For nail-on plates, which use separately applied nails, End and Edge Distances should be specified to prevent splitting of the lumber and should be used in the fabrication of the Trusses, as well as for plate testing and joint

design. Recommended End and Edge Distances for nails can be found in the *NDS* Commentary.³

§5.2.6.6 Wood-to-Plate Slip.

This section specifies a 5-in. minimum lap of the plate onto the vertical members for Test Specimens for loads perpendicular to the wood member (AE and EE orientations) in order to assure that failure occurs on the test member (horizontal member) and to minimize the amount of slip between the plate and the non-test member (vertical member) when slip is measured wood-to-wood. Tests done with smaller amounts of plate lap are suitable for use in determining safe design values, but may underestimate results.

§5.2.7 Test Specimens Required.

The number of replicates specified for lateral resistance tests (five) is relatively low, but not atypical for fasteners. For example, a minimum number of replicates as low as five has been recognized in other fastener test standards, although a minimum requirement of ten is more generally specified.⁴ Section §5.2.9.2 provides some discussion on justification for this due to control of the density of the Test Specimens and the purpose of the testing to establish a mean strength, with fifth percentile strength levels then being based on a calculation presuming a statistical distribution from a larger data set, as opposed to direct empirical estimation of fifth percentile strength levels. Further discussion is provided here regarding the suitability of five replicates, given the intent of the testing is to establish a mean value for lateral resistance from testing. It is possible to estimate whether the number of replicates is suitable using the following formula from Section 3.4.2 of *ASTM D2915* which is based on estimating the mean to five percent precision.⁵

$$n = (CV \times t / 0.05)^2$$

Where n is the number of replicates, CV is the coefficient of variation (standard deviation/mean) and t is a variable taken from Table 1 of *ASTM D2915* that is dependent upon the number of replicates and the desired confidence. For 75 percent confidence and CV of 14 percent, a value for n of 12 (for which t = 1.214) is found by this equation, suggesting that larger number of replicates be used, similar to the minimum level of ten replicates recommended

by *ASTM D1761* note 2. However, this CV of 14 percent is from an uncontrolled sampling of lumber, including random sampling without restriction on specific gravity, while the *TPI 1* Standard limits density from exceeding average (or requires adjustment which, in turn, results in those following the Standard typically avoiding the use of lumber with a density exceeding the species average by any significant amount). In addition, since the divisor accounts for the variation in density, it is inappropriate to include variability due to density variation lower than average as this would result in underestimates of the average lateral resistance strength for the species. This latter concern (to use lumber with density not significantly less than the average for the species) is not imposed by the Standard, as it would result in a safe value for engineering design to use below-average density lumber for testing. However, it is normally imposed by users of the Standard, as the intent is typically to identify an appropriate, rather than an overly conservative, design value. Following such a practice has been found to limit the CV of test samples to about seven percent, on average. For 75 percent confidence and CV of seven percent, a value for n of 4 (for which t = 1.423) is found by this equation. In other words, it can be stated that 75 percent of the times this testing is performed, the resulting mean value from the testing is expected to be within five percent of the true mean. This is not the only uncertainty involved, since assumptions have been made regarding the sample CV and the sampling being representative of average due to control of density, and there is also uncertainty associated with the further use of this estimate of the mean value to determine a design value, but this is a reasonable and simple approach to estimating the uncertainty associated with the direct result of the lateral resistance tests. Modifying the equation so as to identify the precision estimated to result for 95 percent confidence, which is perhaps a more typical confidence level in general, while retaining the sample CV of seven percent, the denominator of 0.05 can be varied to estimate the precision for which the equation will result in five replicates. For 95 percent confidence and five replicates, t = 2.776. Solving for the precision level gives a precision of $CV \times t / (n^{0.5})$, or $0.07 \times 2.776 / (5^{0.5})$, which equals 8.7 percent. In other words, it can be stated that 95 percent of the times this testing is performed, the resulting mean value from the testing is expected to be within nine percent of the mean for the population. While these are only estimates, and additional sampling is always desirable, the level of precision resulting from five replicates is considered reasonable.

While not contemplated by the *TPI 1* Standard, a practice in use for most fastener design capacities is to correlate density and fastener capacity to establish an equation pre-

³ American Forest & Paper Association, *Commentary on the national design specification for wood construction*, 2005 edition, Washington, DC, 2005.

⁴ American Society for Testing and Materials, *Standard Test Methods for Mechanical Fasteners in Wood*, ASTM D1761-88(2000), West Conshohocken, PA, 2000.

⁵ American Society for Testing and Materials, *Standard Practice for Evaluating Allowable Properties for Grades of Structural Lumber*, ASTM D2915-03, Sec. 3.4.2, West Conshohocken, PA, 2000.

dicting lateral capacity given a specific gravity. For example, this practice was used by the U.S. Forest Products Laboratory⁶ and the American Forest & Paper Association to establish the dowel embedment strengths provided for various species in the *NDS*. This practice can also be applied to Metal Connector Plate lateral resistance capacity. The benefits of doing this is to increase confidence in the resulting value due to the pooling of data (increase in number of replicates) and minimizing deviations (potential errors) in data sets for particular specific gravities tested. For example, it is common for lateral resistance data reported for typical Truss plates to vary with density of species but it is not uncommon for exceptions to occur that indicate variability due to sampling error. Exceptions to this correlation may be expected when strength does not increase with an increase in density, as this may indicate lateral resistance is limited by steel properties (such as steel yielding of the Teeth) rather than wood properties at that density, but there is no such explanation when strength increases with decreasing density. This does not occur frequently, but is not uncommon. For example, strength in a lower density species such as SPF (0.42 specific gravity) in some test series may exceed the strength from a similar or higher density species such as Hem-Fir (0.43 specific gravity). When values illustrating this inconsistency are reported, they are typically less than ten percent or so, and illustrate the deviation that may be expected due to the limited test sampling. Such deviations can be minimized by pooling data, establishing a regression line correlating the given lateral resistance property to specific gravity, and then using the equation for the regression line to establish design values for given specific gravities. This would be done for each of the various types of lateral resistance values (AA, EA, AE, EE) since each may vary due to a different extent with variation in density. When this approach is used, resulting values are more consistent with expectations based on specific gravity and may therefore be viewed as more accurate. This type of predictive equation should not be used to increase lateral resistance design values beyond those demonstrated by testing. In other words, such an equation should not be used to assign a higher design value for S. Pine (0.55 specific gravity) based on data only from testing of DF-L (0.50 specific gravity) and SPF (0.42 specific gravity), since it is possible that the higher value predicted by an equation for untested 0.55 specific gravity cannot be obtained in reality due to the occurrence of a failure mode not present for the tested specimens, such as yielding of the steel in the Teeth. In such cases, the value established by the equation for the maximum density of material that

was tested should not be exceeded. This type of predictive equation is suitable for assigning design values to lower density species than tested and would be expected to be more accurate than the general adjustment specified in Section 5.2.9.3 of the Standard.

§5.2.8.2 Time Period.

There is no need to limit the maximum age of a specimen, as additional aging will result in more conservative results.

§5.2.8.3 Loading Procedure.

This section is consistent with *ASTM D5764* (dowel bearing) and *D5456*, Annex 2 (SCL connections) on wood fastener testing, as well as *ASTM D4761* on structural wood bending, tension and compression tests which permit one-minute minimum test durations to maximum load.^{7,8,9} A maximum time limit is not necessary, as longer test times will give conservative results.

§5.2.8.4 Test Procedure for Solid Metal Control Specimens.

New to the 2007 edition of the Standard, specified (versus measured) coating thickness is now permitted to be used to determine uncoated steel thickness from measurements of coated steel thickness. This is permitted for convenience and because the associated error is expected to be less than one percent (see Section §4.3.4).

Also new to this edition, the dimensional measurements of the solid metal control specimens' thickness are specified to 0.0001 in. This requirement also exists in *ASTM E8* for certain steel thicknesses, including those typically used for Truss plates, but resolution as great as 0.001 in. is permitted for greater thicknesses. As it has been reported that measurements to only 0.001 in. resolution have occurred at some labs, the 0.0001 in. resolution is included here to bring this to the attention of those doing testing per the *TPI 1* specification.

§5.2.9.2 Lateral Resistance Design Values.

The lateral resistance value for Metal Connector Plates in wood is largely a wood strength property, and thus the adjustments to the test data to arrive at a design value follow well-known and established principles for eval-

7 American Society for Testing and Materials, *Standard test methods for evaluation dowel-bearing strength of wood and woodbased products*, ASTM D 5764-97a. West Conshohocken, PA, 1997.

8 American Society for Testing and Materials, *Standard specification for evaluation of structural composite lumber products*, ASTM D 5456-99a. West Conshohocken, PA, 1999.

9 American Society for Testing and Materials, *Standard test methods for mechanical properties of lumber and wood-base structural material*, ASTM D 4761-02a. West Conshohocken, PA, 2002.

6 T. L. Wilkinson, *Dowel bearing strength*, Research Paper FPL-RP-505, U.S. Dept. of Agric., Forest Service, Forest Products Laboratory, 1991. (www.fpl.fs.fed.us/documents/fplrp/fplrp505.pdf)

ating lumber strength properties.¹⁰ The composite divisor of 3.2 in subparagraph (b) of this section as explained below has been formulated to incorporate necessary adjustments that consider wood-plate element variability, quality characteristics affecting strength, duration of load, and a safety factor. The allowable value for lateral resistance is expressed as:

$$V_{LR} = \frac{T_{LR} (1 - 1.645(COV))(0.84)}{(DOL)(SF)}$$

where:

V_{LR} = Metal Connector Plate allowable lateral resistance value, psi (or kPa). It includes adjustments for variation, quality, duration of load and safety.

T_{LR} = Average ultimate clear wood Metal Connector Plate lateral resistance value based on tests, psi (or kPa).

(1-1.645(COV)) = Adjustment of the normal mean to the five percent exclusion limit, using COV = 0.14.

(0.84) = Strength ratio to adjust for up to a $\frac{1}{32}$ in. embedment gap between the underside of the plate and the Wood Member.

DOL = Duration of load factor that adjusts a 10 minute test to a standard ten-year load duration (DOL = 1.60).

SF = Safety factor for unknowns such as overload conditions, stress reversals, environmental effects, handling, etc. (SF = 1.30).

Therefore:

$$V_{LR} = \frac{T_{LR} (1 - 1.645(0.14))(0.84)}{(1.6)(1.3)} = \frac{T_{LR}}{3.2}$$

Adjustment of Mean to Five Percent Exclusion Limit (1-1.645(0.14)) - The statistical analysis for lateral resistance value evaluation reduces the clear wood statistical average at ultimate based on tests to a clear wood lower five percent exclusion limit. The decision to use the five percent exclusion limit is based on the consequences of failure for a wood frame structure incorporating allowable stress design principles. It is also considered necessary to calculate this five percent limit from the empirical

average strength, rather than to permit it to be determined solely from empirical data, due to difficulties in mandating and controlling (or limiting the control of) densities of Test Specimens. Specific gravity, which significantly influences the lateral resistance values of Metal Connector Plate Teeth, generally exhibits a normal distribution and exhibits some degree of correlation with the ultimate lateral withdrawal property ($r = 0.563$ or higher, typically - see Section §5.2.9.3). Therefore, the assumption of normality for ultimate is reasonable.¹¹ This also permits the use of fewer replicates than are typical when determining a wood strength property, since the objective of the testing is to quantify only the average strength empirically (and to then adjust it based on variability of the population to identify the five percent limit), rather than to explicitly identify the five percent limit solely by testing.

To arrive at a representative statistical coefficient of variation derived from a test database of five observations would be erroneous due to the small sample size and typical control over variability of test sample lumber. Therefore, a global coefficient of variation for ultimate lateral resistance properties for evaluation purposes must be determined. The coefficient of variation for ultimate load of 0.14 is based on the largest known load test database for Metal Connector Plate lateral resistance.¹² A COV on ultimate load of 0.136 was obtained from a southern pine sample of 241 observations from five sources, which consisted of assorted lumber grades, a variety of plate types, and a number of Truss manufacturing locations. For Metal Connector Plate evaluation purposes, this value of 0.136 has been rounded to 0.14, and serves as a conservative global best estimate for coefficient of variation.

A 0.14 COV is viewed as a conservative estimate for lateral resistance values, since the lumber used for the study included a large variety of Southern Pine grades (including grades marked dense) and randomly sampled lumber from numerous metal-plate-connected wood Truss fabrication locations (estimate for global sampling) across several U.S. states. It is reasonable to expect lumber and Metal Connector Plates for any one particular metal-plate-connected wood Truss design to exhibit a lower coefficient of variation for lateral withdrawal than 0.14 since the lumber would be selected from a limited number of sources (lot sampling) and the Metal Connector Plates from one source. Assuming a high COV of 0.14 assures a built-in measure of conservatism.

Strength Ratio (0.84) - A 0.84 strength reduction factor

11 S. K. Suddarth, D. H. Percival and Q. B. Comus, *Testing and analysis of 4x2 parallel chord metal plate connected trusses*, Small Homes Council-Building Research Council, Research Report 81-1, Urbana-Champaign, IL, May 1981.

12 Suddarth, Percival and Comus.

10 American Society for Testing and Materials, *Establishing structural grades and related allowable properties for visually graded lumber*, ASTM D 245-99. West Conshohocken, PA, 1999.

is included to account for a $\frac{1}{32}$ -in. uniform partial plate embedment gap between the wood surface and the underside of the plate. The 0.84 strength ratio replaces an earlier strength ratio of 0.90 that was intended to account for any combination of manufacturing characteristics that would reduce the effective plate area. A $\frac{1}{32}$ -in. embedment gap is considered to be a reasonable gap size to expect from typical Truss production and the manufacturing process, and the 0.84 strength reduction is based on a study that found a 16 percent reduction in lateral resistance strength of Metal Connector Plates with this amount of average uniform embedment gap.¹³

With the incorporation of a 0.84 strength ratio into the establishment of lateral resistance design values, which are used in the plate design methodology for sizing the Metal Connector Plate (see Chapter 8), the quality criteria in Chapter 3 permit plates to exhibit up to a $\frac{1}{32}$ -in. embedment gap (see Section 3.7.5.1). Plates with embedment gaps larger than $\frac{1}{32}$ in. require assessment of the gap's effect (tooth effectiveness versus tooth embedment gap) on the total number of effective Teeth contributing to the lateral resistance strength of the plate, to ensure adequate strength as required by the design.

DOL (1.60) - Metal Connector Plate lateral resistance tests are conducted to reach ultimate load in one to ten minutes, typically, just as for other lumber resistance tests. The short-term test value is adjusted to an assumed ten-year loading period (called normal loading). The 1.6 adjustment is consistent with the duration of load (DOL) adjustment for structural lumber.

Safety Factor (1.30) - The 1.3 safety factor is suitable for engineering uses and is consistent with established safety factors for structural lumber strength design values.¹⁴ Variable effects, which could occur during the life cycle of the structure and which are difficult to assess, include (but are not limited to) unanticipated overload conditions, stress reversals, and handling and installation errors. For the Critical Slip check, previous editions of the Standard specified a 1.6 divisor on the Critical Slip load, which was based on the value once used for nails. The 2007 edition maintains consistency with current nail methodology, which uses a 1.3 divisor.

§5.2.9.3 Adjustment Factor for Specific Gravity of Test Specimens Exceeding Specified Value (R_G).

Prior to the 2002 edition, the Standard included wood sampling parameters pertaining to specific gravity, which stipulated that the average specific gravity of the Wood

Members used in the five or more Test Specimens (for each lateral resistance allowable value) could not exceed the published average specific gravity of the species or Species Combination. In addition, the specific gravity of any one Wood Member was limited to be within ten percent above the published average. The limit on the average specific gravity was due to the recognized correlation of lateral resistance strength with wood density, in which increased wood density generally results in increased tooth holding strength. Thus, the average species group limit was intended to prevent variation in tooth holding strength above that which would result from use of average density wood.

Despite these limits, it was acknowledged within the Commentary that it may not be possible to obtain wood with average or lower density from some lumber samples, due to species variation within a species group, or wood variation due to local forest conditions. In particular, limited samples of wood, such as a bundle of several hundred 2x4s obtained from a single lumber grade from a single manufacturer, may not include the typical range of density for the species. For this case, where the requirements for specific gravity could not be satisfied, but the tests were otherwise in accordance with the procedures, the Commentary provided an adjustment for specific gravity, which adjusted test results downward to the average listed specific gravity for the species group.

It is important that the test-based design value be based on the average density for the species, but because there has been a substantial amount of research conducted that shows that it is conservative to adjust the tested results downward linearly in proportion to the ratio of the target density to the average tested density, this adjustment is included in the Standard.

Test data in Southern Pine, with a range of 24 specific gravities between 0.49 and 0.69, for more than 100 otherwise identical joints using one type of 20 gauge plate, showed tooth holding to be roughly linearly correlated (with R^2 approximately 0.7) with specific gravity, with the percentage change in tooth holding value being similar to the percentage change in specific gravity. This means that the intercept term in a straight-line equation for tooth holding strength as a function of specific gravity is relatively close to 0. The plot shown in Figure C5.2-1 depicts the correlation between tooth holding strength and specific gravity obtained from this data.

Later studies also indicated that a linear equation shows moderate to good correlation of tooth holding to specific

13 M. H. Triche, *Partial plate embedment study*, Report to Truss Plate Institute, University of Alabama, February 2, 1994.

14 Wilkinson.

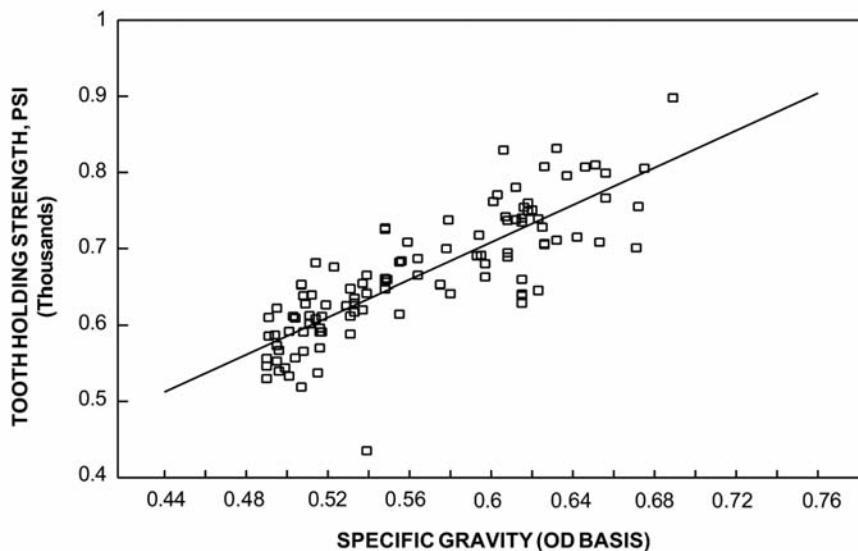


Figure C5.2-1. Tooth holding strength versus wood specific gravity.

gravity.^{15, 16, 17} This work also shows a meaningful intercept term in the equation, which means that the adjustment given in the Standard is conservative, since the intercept is not included. In other words, an adjustment of $G_{\text{specified}} / G_{\text{tested}}$ is lower than a full linear equation with a non-zero intercept term such as $(M \times G_{\text{specified}} + B) / (M \times G_{\text{tested}} + B)$. Conservatism is further assured with the condition that this adjustment is only permitted to be used to adjust test results downward to lower specific gravities, and not upward. In addition to the inclusion of an adjustment for the average specific gravity, the Standard has also eliminated the upper limit on an individual specimen's specific gravity of ten percent above average, because a linear relationship results in no difference if the range of specific gravity tested exceeds this limit, as supported by the research. Furthermore, at higher specific gravities, the effect of a change in specific gravity results in a lesser amount of change in tooth holding than occurs at lower specific gravities, which results in a quadratic correlation between tooth holding and density.¹⁸ Consequently, the average tooth holding result from specimens with a wide range of density would give a lower value than the average tooth holding result from specimens with a narrow range of density, for both groups having the same average density.

See also Section §5.2.7 for discussion of use of an alternate approach for accounting for specific gravity variation to that noted above.

§A5.2 ANNEX: DESIGN VALUE ADJUSTMENTS FOR SINGLE-PASS ROLLER PRESSES (MANDATORY INFORMATION)

The annex to Section 5.2 was initially established in the 1995 edition of the Standard. Predecessor Standards to the 1995 edition had not recognized the variation in strength that may occur with different types of fabrication equipment. This annex was initially developed as a test protocol within TPI's Technical Advisory Committee (TAC), circa 1990, in order to establish data regarding the effect of single-pass full-embedment roller presses on lateral resistance strength for purposes of considering how to best standardize assignment of roller press values. No data nor recognition of any trends suitable for incorporation into the Standard were identified by TPI, but it was included as an option in lieu of the more costly complete repetition of tests for both standard (vertically acting hydraulic or pneumatic) pressing equipment as well as single-pass, full-embedment roller pressing equipment.

§5.3 STANDARD METHOD OF TEST FOR STRENGTH PROPERTIES OF METAL CONNECTOR PLATES UNDER SHEAR FORCE

The test method in Section 5.3 is to be used for the determination of unit design values for pairs of Metal Connector Plates (one on each side of the connection) subject to a shear force through their cross-section. This resistance to shear is a measure of the ability of the Metal Connector Plate to transmit forces within the Metal Connector Plate itself.

15 Triche.

16 L. C. Palka, *Effect of wood density on truss plate strength*, Forintek Canada Corp. Western Laboratory, Prepared for Science Council of British Columbia, Vancouver, BC, 1984.

17 J. D. Moura, C. Bastian and P. Triboulot, *The influence of wood density on metal plate connector mechanical behavior under cyclic loading*, Forest Products Journal, Vol. 45, No. 11/12, 74-82, 1995.

18 B. K. Via, A. Zink-Sharp, F. E. Woeste and J.D. Dolan, *Relationship between tooth withdrawal strength and specific gravity for metal plate truss connections*, Forest Products Journal, Vol. 49, No. 7/8, 56-63, 1999.

Due to stress concentrations and the difficulty of predicting an accurate path of failure, shear design values for Metal Connector Plates should be based on tests rather than analytical methods. The shear resistance of the Metal Connector Plate is compared to the theoretical shear resistance of a solid metal control specimen to arrive at the shear Effectiveness Ratio (R_s) of the Metal Connector Plate. This permits the application of this Effectiveness Ratio to any thickness or grade (yield and ultimate tensile strengths) of steel, since adjustments in plate strength due to thickness, or steel yield and strengths, are accounted for by explicit use of the thickness, steel yield strength and steel ultimate strengths in accordance with Chapter 8. However, shear efficiencies may decrease slightly for substantially thinner steel or substantially higher grades of steel relative to thicker steel or lower grades of steel, since steel buckling strength may occur at lower levels for thinner steel and may not increase in proportion to yield or tensile strengths. It is therefore suitable for design values resulting from 20 gauge plates of a given grade of steel, for example, to be applied to otherwise identical plates made of 18 gauge steel and/or made of a lower yield strength of steel using the same tooling (presuming the tooling is capable of accommodating the increased thickness while still meeting all manufacturing requirements of Chapter 4). However, it is suggested that shear Effectiveness Ratios from 18 gauge plates or plates from lower grades of steel not be applied to otherwise identical plates made of 20 gauge steel or higher grades of steel without limiting the resulting shear design values appropriately (e.g., for higher grades of steel, the shear values may be limited to the same absolute force limits, in units of lbf, as the lower grade of steel).

See also Section §5.4 as much of that discussion also applies to Section 5.3 and is not included separately here.

§5.3.4 Test Specimen Fabrication.

The 2007 edition of the Standard provides a slight change to the steel shear tests in that the arbitrary limits of 3 in. on the offset between the center member and the exterior members was replaced with the more specific intent of assuring the load path through the steel plates, and removing a limit on ripping lumber.

While the longitudinal axis of a slot can only vary between 0 and 90 degrees from a joint line, the Standard defines angles for shear capacity ranging from 0 to 180 degrees. This is in recognition that the shear loading on a plate typically induces forces normal to the joint as well, and these normal forces result in either tension or compression being carried by the Metal Connector Plate. The direction of these normal forces is generally governed by the gross dimensions of the plates and the convention

prescribed by Sections 5.3.4.3 and 5.3.4.4 based on centroids of plate area. The intent is that shear load angles between 0 and 90 degrees to the plate produce shear with some tension on the plate steel section, while shear load angles between 90 and 180 degrees to the plate produce shear with some compression on the plate steel section. The Truss Designer must consider this intent when evaluating the shear angle for any given joint being designed.

Section 5.3.4.8 notes that joints in shear Test Specimens are to be closely fitting but not compressed. This is because the intent of the design portions of the *TPI 1* Standard are that all joints be tight-fitting. However, as Section 3.7.6.2 recognizes that there are occasions where gaps between Wood Members may be considered, it is recommended that shear design values for such joints be established making use of the maximum permitted gap present in such joints.

§5.3.5 Test Specimens Required.

The selection of shear angles at intervals of 30 degree increments was thought to be a reasonable balance of testing cost and accuracy. There is variation in shear strength across the 30 degree intervals, with some of this variation becoming quite large. For example, it is not unusual to have shear strengths increase by more than 50 percent from adjacent 30 degree increments, especially for angles at or adjacent to 90 degrees. For this reason, it is recommended that Truss Designers give due consideration to appropriately limiting shear design values to the lower adjacent tested shear angle or, if interpolation is used between tested shear angles, take other action to assure that shear design strengths are not overestimated. The Standard does not address interpolation between tested shear orientations, neither permitting it nor prohibiting it, leaving this matter to the judgment of the Truss Designer. Further testing of plates at intervals more frequent than 30 degrees is one means of further addressing this variation.

§5.3.6.4 Test Procedure for Solid Metal Control Specimens.

See Section §5.2.8.4.

§5.4 STANDARD METHOD OF TEST FOR STRENGTH PROPERTIES OF METAL CONNECTOR PLATES UNDER TENSION FORCES

Due to stress concentrations and the difficulty in predicting an accurate path of failure, especially in plates with fully staggered slots, tension design values for Metal Connector Plates should be based on the test method in this Section, with exceptions as noted in Section 5.4.1 for plates complying with typical steel design practices.

The tension resistance of the Metal Connector Plate is compared to the tension resistance of a solid metal control specimen to arrive at the tensile Effectiveness Ratio (R_t) of the Metal Connector Plate. This permits the application of this Effectiveness Ratio to any thickness or grade (yield and ultimate tensile strengths) of steel, since adjustments in plate strength due to thickness, or steel yield and strengths, are accounted for by explicit use of the thickness, steel yield strength and steel ultimate strengths in accordance with Chapter 8. For this reason, it is suitable for tensile Effectiveness Ratios resulting from a given thickness and grade of steel to be applied to any identical plates made of other thicknesses or grades of steel using the same tooling (presuming the tooling is capable of accommodating the increased thickness while still meeting all manufacturing requirements of Chapter 4).

§5.4.1.2 Chain of Holes Extending in Diagonal or Zigzag Line.

The basis for the diagonal or zigzag failure paths is typical steel design practice.¹⁹

§5.4.3 Solid Metal Control Specimens.

Metal thickness may vary from one end of a slit coil to the other end. Requiring three solid steel metal control specimens from each end of each slit coil will account for this variability. If a solid metal control sample is taken from the coil immediately adjacent to the portion of the coil used to produce the test plates (as opposed to being taken from the ends of the coil as described in Section 5.4.3), it is appropriate to require only the three solid steel metal control specimens from that sample.

§5.4.4.2 Test Specimens Assembly.

The requirement that the minimum net steel section of the Metal Connector Plate be positioned directly over the joint is due to the difference in strength that may result between this position and if the maximum net steel section is located directly over the joint. Testing has shown that some plates at some orientations may exhibit differences in strength on the order of 30 percent. Values resulting from tests with the maximum net steel section on the joint can be appropriate if the plate is positioned in practice as intended by the design, but normally the difference in location between these two positions is quite small, often well under $\frac{1}{4}$ in. Coupled with the requirement in Chapters 3 and 8 to produce Joint QC Details and standard prescriptions for plate misplacement of $\frac{1}{2}$ in., this deviation from the standard is not acceptable without quality control provisions assuring plate placement is

as intended (i.e., without allowance for the typical misplacement provisions prescribed in Chapter 3).

§5.4.5 Test Specimens Required.

The selection of angles for tension testing only at orthogonal orientations (0 and 90 degrees to the plate length) was thought to be a reasonable balance of testing cost and accuracy. However, the variation in values can be large, with tensile strength parallel to slots being as high as nearly three times the strength when loaded perpendicular to the slots for some plates. For this reason, it is recommended that Truss Designers give due consideration to appropriately limiting tensile design values to the lower adjacent tested orientation or, if interpolation is used between tested shear angles, take other action to assure that tensile design strengths are not overestimated. The Standard does not address interpolation between tested orientations, neither permitting it nor prohibiting it, leaving this matter to the judgment of the Truss Designer. Further testing of plates at intervals between the 0 and 90 degree angles specified is one means of further addressing this variation.

If a Metal Connector Plate has a steel cross-section that is identical in both directions, such as plates with circular plots at the same spacing along both the width and the length of the plate, only testing across its width should be necessary and the resulting Effectiveness Ratio is applicable to both orientations of the Metal Connector Plate.

§5.4.6.3 Testing Procedure for Solid Metal Control Specimens.

See Section §5.2.8.4.

§5.4.7.1 Ultimate Tensile Strength - Test Specimen.

The ultimate strength of the Metal Connector Plate is determined using the gross cross-sectional area of the metal instead of the net cross-sectional area. An Effectiveness Ratio is determined and will account for the difference between gross and net cross-sectional area.

§5.4.9(n) Report.

New to the 2007 edition of the Standard, a testing machine calibration certificate is now required to be reported for steel tensile testing, making this section consistent with the other sections in this Chapter.

¹⁹ American Institute of Steel Construction, *Specification for structural steel buildings*, Chicago, IL, 2005.

CHAPTER 6

MATERIALS AND GENERAL DESIGN CONSIDERATIONS

Overview of (non-editorial) Changes

New to TPI 1-2007

- 43)** Statements on Truss installation tolerance were removed from *TPI 1-2007 (BCSI now referenced in Chapter 2)*. (*TPI 1-2002 Section 6.1*)
- 44)** Non-bearing partition loads permitted to be ignored (formerly in *TPI 1-2002 Chapter 2*). (Section 6.2.2.1)
- 45)** Provision for attic Bottom Chord live loads applied to full length of Bottom Chord being non-concurrent replaced by reference to Building Code. (Section 6.2.2.2)
- 46)** Partial-length (unbalanced) live loads now specified to include for roof Trusses (previously only mentioned for floor Trusses). (Section 6.2.2.5)
- 47)** E_{min} values referenced and reference to F_g values deleted. (Section 6.3.1)
- 48)** Structural Composite Lumber F_t value now permitted to be based on 20 ft. length basis for member subject to combined bending and tension. (Section 6.3.2)
- 49)** Truss plate steel design values relocated to Chapter 6 from Chapter 4 and thickness of G185 galvanized coating explicitly recognized. (Section 6.3.4.1.3)
- 50)** Repetitive axial stress increase of ten percent is permitted to be applied to E_{min} for solid sawn lumber. (Section 6.4.2.1(b))
- 51)** Limitations and provisions for structural composite are now recognized, including repetitive increases are limited to four percent for bending stress and zero for other stresses. (Section 6.4.2.1(c))
- 52)** Bending capacity modification factor, K_M , introduced for solid sawn lumber (to recognize up to 30 percent increases for smaller, or up to about ten percent decreases for larger, volumes of compression Chords subject to maximum stress levels). (Section 6.4.3)
- 53)** Shear stress factor, C_H , was deleted in *TPI 1-2007*. (*TPI 1-2002 Section 6.4.7*)
- 54)** The flat use factors now limits to solid sawn lumber only. (Section 6.4.4)
- 55)** The C_T factor calculation for Structural Composite Lumber (SCL) to use same k factor as for machine stress rated (MSR) lumber. (Section 6.4.5.2)
- 56)** Wind/seismic 1.33 increase for steel strength deleted. (Section 6.4.3)

57) C_q factor basis changed from a maximum value of 1.25 to a maximum value of 1.00 (so 20 percent reduction in plate lateral resistance values with *TPI 1-2002* suggested $C_q=1.00$ for 2x_ Trusses is now obtained if $C_q=0.80$). (Section 6.4.10)

58) Specification of less-than-max values of C_q as default values were deleted in *TPI 1-2007*. (*TPI 1-2002 Section 6.4.11.12*)

59) Provisions for connector plates in conventional walls up to 24 in. high (i.e., ladder Frame or cripple walls) provided including 1x3 connector plates for stud-to-wall-plate connections instead of nails or staples, and cutting/notching of wall members is permitted per Building Code provisions for conventional walls. (Section 6.6)

§6.1 GENERAL

The 2007 edition of the Standard removed sections on Truss installation tolerances, which had been stated as design assumptions in the prior edition, as *BCSI-B1* is now referenced in Chapter 2 Sections 2.3.4.5 and 2.4.4.5 and contains these tolerances.

§6.1.1 Structural Analysis.

The provisions for structural analysis are not addressed within the *TPI 1* Standard as this is not considered unique to wood Trusses. Prior provisions specific to modeling joints at Chord breakjoints and between Webs and Chords using pins was removed in the 2002 edition of the Standard, in recognition that computer and engineering technology to accommodate more accurate modeling of joint stiffness had become possible. It is generally recognized by engineers designing Trusses that single-dimensional analog lines (if used) should follow member centerlines, joint stiffness should not be overestimated, bearing stiffness should not be overestimated, and similar conventionally followed practices.

§6.2.1 General Loading Requirements.

As the Registered Design Professional for the Building is responsible for specifying all dead and live design loads applicable to each Structural Element in a Building or other structure (see Section 2.2), and because load information is the domain of the Building Codes, information pertaining to load development is considered to fall outside the scope of this Standard.

The phrase “accepted engineering practice for the geographical area” implies the engineering practice pre-

scribed by the national or local area codes that are applicable in that particular area.

The loading requirements shall be as provided by the latest edition of *ASCE 7* in the absence of a governing Building Code specifying otherwise. The *ASCE 7* standard should be used after securing competent advice with respect to its suitability for a given application. Presently, the most recent implemented edition of *ASCE 7* is *ASCE 7-2005*. With the adoption of newer versions of *ASCE 7*, the loading requirements shall be permitted to follow the provisions of the latest implemented editions. The term “implemented” is intended to recognize the lag time that occurs between date of publication and the ability to make use of the latest published edition. It is not the intent of this Standard to mandate the use of a more recent edition than others may require (such as that enforced by the authority having Jurisdiction) or prefer (Truss Designer preferences or limitations of available design software).

Another document that can be used for the calculation of loading for the design of Trusses and other structural building components is *The Load Guide*. *The Load Guide* is subtitled “Guide to Good Practice for Specifying and Applying Loads to Structural Building Components” and is a spreadsheet produced by WTCA’s Engineering & Technology Committee in cooperation TPI’s Technical Advisory Committee (TAC). The positions, interpretations, comparisons and commentary included in this document are intended to assist with specifying and applying loads on Trusses and other structural building components. They are intended to aid in the consistent interpretation and application of loads, yet are not intended to supersede an architect’s or engineer’s judgment and design specification for the loads that should be applied to a specific Building (for more information on *The Load Guide* see: www.sbcindustry.com/loads.php).

§6.2.2 Loading Requirements for Metal-Plate-Connected Wood Trusses.

This section provides for specific loading requirements for the design of metal-plate-connected wood Trusses that may not otherwise be recognized. It includes the load from the non-load bearing partition, the attic live load, the effect of pitch on the loading, the full and partial length live loading and the value for wind uplift.

§6.2.2.1 Non-Bearing Partitions.

The information on non-bearing partition loads permitted to be ignored was moved from Chapter 2 Section 2.2.5 to Chapter 6 Section 6.2.2.1 in the 2007 edition of the Standard.

§6.2.2.2 Attic Live Loads.

The *International Building Code* states that attic live loads, other than floor live loads, that are applied to the entire length of the Bottom Chord shall not be required to be applied concurrently with other live loads. *TPI 1-2002* explicitly stated this in Section 6.2.1.1. The provision for attic Bottom Chord live loads applied to the full length of the Bottom Chord being non-concurrent was replaced by reference to the governing Building Code in the 2007 edition of the Standard.

It should be recognized that Truss Webs, attic insulation, and limited access to the attic space prevent the placement of significant storage loads in a Truss attic that is served only by a scuttle or pull-down stair. Also, short-term loads (i.e., for inspection and maintenance purposes) should not be considered to act concurrently with other design live loads, such as snow, wind and roof live load. Such short-term loads should also be designed using the appropriate load duration factor (typically $C_D = 1.25$).

§6.2.2.3 Effect of Pitch.

Applying loads on a projected horizontal area basis is common practice and has long been considered reasonable, given that specified dead loads are often nominal and/or conservative. However, the issue of applying dead loads to horizontal projected areas versus sloped areas becomes critical for steep slopes, where the actual dead loads can exceed the amount determined by applying the specified dead loads per unit area to a horizontal surface and not to the larger sloped surface. With steeper pitches being more common than in the past, the requirement for the effect of pitch to be taken into account is necessary to ensure that the actual dead load does not exceed the specified dead load. This also is intended to prevent the application of dead loads based on minimum material densities on the horizontal projected area without adjusting for pitch.

§6.2.2.4 Dead Loads for Determining Wind Uplift.

While “actual” dead loads should be used for design, allowance for variations in thickness of materials, substitution of similar but different weight materials, additional layers of materials, etc., implies that the dead loads used for gravity load design should often not be the absolute minimum.

Good design practice uses dead loads that account for these possibilities. However, using overstated dead load for wind uplift calculations also results in an overstated uplift resistance. *ASCE 7* addresses this issue by requiring design dead loads to be reduced by 40 percent for this load case, to ensure that they are not overstated when determining wind uplift and to assure an adequate margin

of safety for cases where the resistance to overturning of the structure is accomplished solely by the dead weight of the Building.¹ This latter case requires some safety margin to be provided in the load combination factors, and this is the source of the 0.6 wind load factor specified by ASCE 7. TPI 1 recognizes that it is also acceptable to use a dead load that is known to be the minimum expected actual weight of materials when there is no such need to guard against overturning, because this recognizes the actual gravity load that will counteract the uplift forces and a safety factor is applied when the tie-down device is specified.

§6.2.2.5 Full- and Partial-Length Live Loading.

New to the 2007 edition of the Standard, partial-length (unbalanced) live loading is now specified for both roof and floor Trusses (previous editions only mentioned unbalanced loading for floor Trusses).

For a floor Truss with a chase located at or near mid-span, the unbalanced load case with live load only to one side of the chase will result in the maximum amount of vertical load (beam shear) being transferred across the chase, which may result in a more critical design case for some Wood Members or plates.

The same situation may occur with roof Trusses; however, a different set of conditions has typically limited the need to run any unbalanced loads in the same manner as is done for floor Trusses. These conditions include the following:

- Snow and wind loading vary in response to the external shape of the roof and typically include unbalanced loading conditions that vary (meaning increase or decrease of the uniform loading) at points associated with ridges, valleys, or similar geometrically defined points.
- Roof Trusses carrying floor loadings, such as Trusses containing habitable rooms, are normally checked for unbalanced conditions due to the magnitude of the loads and size of the room opening.

The change made in the 2007 edition of this Standard to mention roof Trusses specifically was intended to address the situation of roof Trusses with smaller openings than typically occur with habitable rooms and with roof live loads due to loads other than snow or wind, such as construction live loads. This was recognized as a situation where Truss members may potentially be susceptible to a more critical load case from a partial loading, but

which were not typically being designed for such a partial loading. No guidance is provided with regard to what magnitude of unbalanced loading is appropriate for roof live load cases.

A similar need for considering partial-length loadings occurs with multiple span Trusses. This has not been considered necessary to point out specifically within TPI 1 as consideration of such unbalanced loadings is normally recognized as a requirement for all such structures and is not unique to Trusses.

§6.3.1 Design Values for Solid Sawn Lumber.

For consistency with the latest edition of the ANSI/AF&PA NDS, the 2007 edition of the Standard removed any references to F_g and added reference to E_{min} .

The use of E_{min} is simply a formatting change. The following comparison should help provide a better understanding of the E_{min} value which was added to the 2005 edition of the NDS. The differences between the versions are minimal due to the equations changing as shown below in Table C6.2-1 (approximately 99.4 percent for compression and 99.5 percent for bending) and are due to the effect of rounding.

Note Regarding Visually Graded Solid-Sawn Lumber: The TPI Technical Advisory Committee (TAC) recommends that the minimum grade of visually stress-rated lumber used for Truss Chords should be No. 2 grade. This recommendation (TPI Technical Report No. 2, TPI, available at: www.tpininst.org) does not apply to small Trusses up to 8 ft. in span (such as jacks and mansard frames), Trusses and Truss frames that have support at intervals of 4 ft. or less such as valley fills and gable end frames or Trusses used in manufactured housing (where the Truss structure is sheathed prior to site erection). This is a minimum recommendation and it may be necessary to supply higher grade material on any type of Truss to meet the Truss structural requirements or the end user's quality requirements.

§6.3.2 Design Values for Structural Composite Lumber.

New to the 2007 edition of the Standard, the Structural Composite Lumber (SCL) design values are specifically addressed, including that the F_t value is now permitted to be based on a 20 ft. length basis for members subject to combined bending and tension. The intent was to more accurately consider volume adjustments for establishing allowable tension values. Often allowable tension values are expressed on a shorter length basis than appropriate

¹ American Society of Civil Engineers, *Minimum design loads for buildings and other structures*, ASCE 7-05, New York, NY, 2005.

2001 NDS/TPI 1-2002	2005 NDS/TPI 1-2007
For Compression Members:	For Compression Members:
$F_{cE} = \frac{(0.418)E'}{\left(\frac{L'}{d}\right)^2}$	$F_{cE} = \frac{(0.822)E'_{min}}{\left(\frac{L'}{d}\right)^2}$
$F_{cE} = [0.418 (1,800,000)] / [(L'/d)^2]$	$F_{cE} = [0.822 (910,000)] / [(L'/d)^2]$
$F_{cE} = 752,400 / [(L'/d)^2]$	$F_{cE} = 748,020 / [(L'/d)^2]$
For Bending Members:	For Bending Members:
$F_{bE} = \frac{(0.610)E'}{\left(\frac{L_e d_1}{d_2^2}\right)}$	$F_{bE} = \frac{(1.20) E'_{min}}{\left(\frac{L_e d_1}{d_2^2}\right)}$
$F_{bE} = [0.610 (1,800,000)] / [(L_e d_1)/d_2^2]$	$F_{bE} = [0.822 (910,000)] / [(L_e d_1)/d_2^2]$
$F_{bE} = 1,098,000 / [(L_e d_1)/d_2^2]$	$F_{bE} = 1,092,000 / [(L_e d_1)/d_2^2]$

Table C6.2-1. NDS comparison for products with $COV_E \leq 0.11$. Example assumes 2400f-1.8E MSR Lumber ($E = 1,800,000$ psi / $E_{min} = 910,000$ psi).

for a Truss. The allowable tension stress for SCL is affected by the length of lumber exposed to tensile stress. SCL design literature provides the basis length and an adjustment factor. The base allowable tensile stress is multiplied by the adjustment factor to adjust for any length of lumber subject to tension stress. However, these factors are established from tests where the entire SCL length is subject to the same tensile stress. Given that tensile stresses typically vary within each panel of a Truss and that maximum stresses are usually due to a combination of tension and moment stresses and occur at a point, rather than the entire length of the SCL in a Truss, it is overly conservative to establish a factor based on the actual SCL length in a Truss. However, there is no guidance available elsewhere and design regulators unfamiliar with this issue may inappropriately conclude that the full SCL length should be used. *TPI 1* now specifies that in no case shall it be required to exceed a 20 ft. length basis for members subject to a combination of moment and bending stresses (it may also be reasonable to recognize that a length basis equal to the panel lengths subject to the maximum tensile stress be used).

For Truss designs with SCL, the designer should investigate the potential effect of Truss plate Teeth on SCL, including any recommendations put forth by the SCL manufacturer. While solid-sawn lumber does not require a reduction for the effect of Truss plate Teeth, a reduction

on SCL capacity for the effect of Truss plate Teeth may be justified considering the following. Solid sawn lumber values are based on wood characteristics like knots, which greatly reduce the strength of one cross-section compared to another cross-section. Hence, the likelihood for reduced strength due to interaction between the Teeth and a grade-controlling attribute for solid sawn lumber is quite low and does not reduce the fifth percentile strength upon which lumber design values are based. In contrast, SCL has such characteristics dispersed through the cross-section; with much less variability in wood strength along the board length, so loss in section capacity due to plate Teeth penetration is more likely to be observed, however small. In Europe, where plate Teeth are typically wider or longer, as well as often thicker due to heavier gauge plates, a reduction in SCL tension and bending capacities due to the nail plates is taken into account by using a reduced SCL cross section.² There has been some research in the U.S. that suggests reductions in wood bending and/or tension strength of Structural Composite Lumber due to penetration of the Teeth on conventional U.S. Metal Connector Plates does not occur or is inconsequential.³

2 A. Kevarinmaki and J. Kangas, *Nail plate reinforced bolt joints of Kerto-LVL structures*, Proceedings of the 1996 International Wood Engineering Conference, New Orleans, LA, Vol.3, 271-278, 1996.

3 R. Gupta and B. Wagner, *Effect of metal connector plates on the bending strength of solid sawn lumber and LVL: A pilot study*, Forest Products Journal, 52(10), 71-74, 2002.

However, designers using plates with longer Teeth than the approximately $\frac{3}{8}$ in. long Teeth typical for North American Truss plates may want to consider following an approach suggested by Finnish researchers of reducing the wood cross sectional width by $\frac{7}{64}$ in. to $\frac{13}{64}$ in. (3 to 5 mm) for design purposes to account for the effect of Metal Connector Plate Teeth on LVL strength.

§6.3.4.1 Allowable Steel Stresses.

In the 2007 edition of the Standard, Truss plate steel design values were relocated to Chapter 6 (previously in Chapter 4) and a thickness of G185 galvanized coating is now explicitly recognized. The limiting stresses based on steel yield strength were taken from standard steel industry practice, and generally resulted in safety factors on ultimate failure exceeding 2.0. Limiting stresses based on steel ultimate tensile strength were added in the 1995 edition of the Standard. This addition was made due to the availability at that time of Metal Connector Plates from Grade 60 steel with a relatively low value of 1.17 for the ratio of ultimate to yield strength. Safety factors for this grade (and possibly others) can fall below the nominal level of 2.0 intended by *TPI 1* if plate design values are not limited based on ultimate tensile strength as well.

§6.3.4.1.3 Design Thickness.

The division of the base metal thickness by 0.95 was the means used by the light gauge steel industry to convert steel ordering practices from a nominal thickness basis (where tolerance below that ordered were permitted) to a minimum thickness basis (where no tolerance is permitted below the minimum thickness specified). For this reason, a corresponding portion of the factor of safety for steel design may be considered to cover this allowance, which was originally assigned to minor negative thickness tolerances.

§6.4 ADJUSTMENTS TO DESIGN VALUES

For consistency with the *NDS-2005*, the shear stress factor, C_H , was deleted in the 2007 edition of the Standard (formerly *TPI 1-2002* Section 6.4.7).

§6.4.2 Repetitive Member Increase (C_r).

Repetitive member factors have long been utilized in wood design to account for the load-sharing effects of assemblies that strengthen them beyond the strength assumed in designing the single members within them. A load sharing factor of 1.15 for bending stresses in multi-member systems has been recommended in *ASTM D245* since 1970, and a 1.15 “repetitive member factor” for

bending design values of dimension lumber has been specified in the *NDS* since the 1968 edition.^{4,5}

The 1.15 factor for bending was originally developed from considerations of repetitive, parallel systems consisting of single, solid sawn joists.⁶ Consequently, and in the absence of other substantiating data, the use of the repetitive member factor in wood Truss applications has been limited in the past to bending resistance only, despite the fact that wood Truss members resist substantial compressive and tensile forces in addition to being bending members as a whole. Thus, the restricted application of the repetitive member increase factor to bending resistance, and not to tension and compression, offered an opportunity for a study that could better quantify load sharing effects specific to metal-plate-connected wood Truss systems and validate the load sharing benefit to these additional modes of resistance. Several such studies provided the basis for the repetitive member factors.

The repetitive member provision includes three possible increases for repetitive member assemblies. Permitted repetitive member design value increases are:

- (a) Those listed in the recognized lumber grading rules and consisting of a 15 percent increase to F_b for solid sawn lumber;
- (b) A 15 percent increase to F_b and ten percent increase to F_c and F_t for solid sawn lumber members to which structural wood sheathing is mechanically attached (new to the 2007 edition of the Standard, the repetitive axial stress increase of ten percent is now permitted to be applied to E_{min} for solid sawn lumber); or
- (c) A ten percent increase to F_b , F_c and F_t for solid sawn lumber members to which structural wood sheathing is not mechanically attached. These increases apply to Chord members where three or more Trusses are positioned side by side, are in contact, or are spaced no more than 24 in. on center and are joined by roof sheathing, flooring, gypsum, or other load distributing elements attached directly to the Chords.

4 American Forest & Paper Association, *Commentary on the national design specification for wood construction*, 2005 edition, Washington, DC, 2005.

5 American Society for Testing and Materials, *Standard practice for establishing structural grades and related allowable properties for visually-graded lumber*, ASTM D 245-99, West Conshohocken, PA, 1999.

6 S. M. Cramer, J. M. Drodzak and R. W. Wolfe, *Load sharing effects in light-frame wood-truss assemblies*, Forest Products Journal, Vol. 126, No. 12, 1388-1394, 2000.

The repetitive increases of ten percent for axial stresses and 15 percent for bending stresses were determined for solid-sawn lumber. SCL is usually permitted a four percent increase for bending stresses. New to the 2007 edition of the Standard, limitations and provisions for SCL are now recognized, including repetitive increases being limited to four percent for bending stress and zero for other stresses.

The increase permitted in (a) does not reflect any change to the long-standing 1.15 factor for allowable bending. The increases in (b) and (c) are based on studies of load-sharing effects in light-frame wood Truss assemblies. To directly quantify the load-sharing effects in the system, a structural analysis model and statistical characterizations of lumber stiffness and strength properties were used in one study to analyze Truss assemblies with and without attached sheathing. The study involved 200 simulations of six different Truss configurations, including fink Trusses, hip Trusses, and parallel chord Trusses, where the Trusses in each system were geometrically identical, but had unique, individual sets of member stiffness and strength properties. The mean load-sharing factors from the assemblies tested ranged from 1.06 to 1.17 at design load (up to 1.24 at two times design load), and were found to be applicable to wood Truss members subject to tensile or compressive forces in addition to bending forces.⁷ These findings supported the ten percent repetitive increase in F_b , F_c and F_t for assemblies meeting the basic load sharing requirements. In addition, since the study did not consider partial composite action, an additional five percent, or 15 percent total increase for F_b , was considered justified for members with structural wood sheathing attached, due to the additional strengthening effects of the composite action. Other studies, including those testing wood Truss assemblies, have typically found greater magnitude increases in strength of the repetitive assemblies relative to individual Trusses. The use of the lower bound values were based on the need to assure their applicability to all systems meeting the Standard's definition for repetitive systems.

As mentioned above, the 2007 edition of the Standard has added E_{min} to the repetitive increase. The repetitive increases for axial stresses recognize the variation in loading due to Truss stiffness results in stiffer and stronger Trusses carrying more load, with the benefit only being applied to design stresses for failure modes that are correlated to the member stiffness. No such failure mode is better correlated to member stiffness than buckling, which is directly correlated to modulus of elasticity, yet

previous editions of the Standard had not applied these increases to modulus of elasticity for buckling design.

§6.4.3 Bending Capacity Modification Factor (K_m).

The bending capacity modification factor, K_m , was introduced in the 2007 edition of the Standard for solid sawn lumber. This factor is used to recognize up to 30 percent increases for smaller, or up to about ten percent decreases for larger, volumes of compression Chords subject to maximum stress levels in bending. This issue is applicable to Trusses as load cases for Truss members typically cause bending stress distributions that are similar to that of continuous span bending members (closer to fixed-end beams), rather than the pin-end beams contemplated for typical solid-sawn dimension lumber applications. The volume of lumber subject to the maximum stress is substantially different in these two stress distributions, with the fixed-ended case showing substantially higher strengths due to lesser volume of lumber exposed to the highest stresses. This effect was first recognized in the 2002 edition of the Standard through the use of a 1.3 heel zone increase permitted for F_b , but the use of the K_m value incorporates adjustments for length effects as well as the loading distribution, which is more consistent with how bending design values for lumber are established. The former approach is still permitted in the 2007 edition if the K_m value is not used elsewhere in the Truss. The 2007 edition of the Standard also limits the heel zone increase factor to 1.1 for use with Structural Composite Lumber, based upon recommendations from SCL manufacturers. The values given for the K_m factor are based upon data for visually graded lumber. Lower variability in strength of lumber corresponds to lower values for K_m , thus K_m is limited from application to SCL.

The K_m factor is based upon research in Canada that was incorporated in 1995 to the TPIC (Truss Plate Institute of Canada) and Canadian Standards Association CSA O86 design standards.⁸

The Canadian standards limit the application of the K_m factor to cases when the following conditions are met:

- (a) The members form part of a fully triangulated, metal plate connected Truss, and;
- (b) The spacing of the Truss does not exceed 610 mm (24 in.) or the Truss does not support more than 610 mm (24 in.) of uniform loading, and;

⁷ S. M. Cramer, J. M. Drozdek R. W. Wolfe, *Load sharing effects in light-frame wood-truss assemblies*, Forest Products Journal, Vol. 126, No. 12, 1388-1394, 2000.

⁸ The background for this proposed change was based on material prepared for the November 16-18, 1995, meeting of the Canadian Standards Association Technical Committee on Engineering Design in Wood (CSA O86) when they adopted this factor.

- (c) Clear spans between bearings does not exceed 12.20 m (40 ft.) and, the design span or overall length of the Truss, not including Overhangs, does not exceed 18.3 m (60 ft.), and;
- (d) The Top Chord slope is not less than $\frac{1}{6}$, which is meant to exclude flat roof Trusses but not flat top Trusses forming part of hip roof systems.

Note: This clause is not for use with girder, bow string, semi-circular, attic, flat roof, or floor Trusses.

These limitations were not included in the *TPI 1* Standard because they are inappropriate. The limits were applied in Canada due to a unique set of circumstances tied to preexisting code provisions and the scope of the research and changes presented to the *CSA O86* committee. There is no other logical reason for these limits.

§6.4.4 Flat Use Factor (C_{fu}).

New to the 2007 edition of the Standard, the flat use factors are now limited to only solid sawn lumber. The flat use factor should not be applied to SCL. F_b values for SCL differ substantially with orientation and this must make use of F_b determined from tests at that orientation.

§6.4.5 Buckling Stiffness Factor (C_T).

The buckling stiffness factor has traditionally included the stipulation that the wood structural panel sheathing must be attached to the Narrow Face of the member in order for the member to utilize the C_T factor.⁹

Although the C_T factor was originally justified for this case, it is valid (conservative) to apply it to any other case for which the contribution of the sheathing to the strength of the composite sheathed Truss Chord is at least as great as it is when applied to the Narrow Face of a 2x4. Since the effect of a wood structural panel on the bending stiffness of a flat-wise oriented Chord is greater than on an edgewise oriented Chord, the provisions are not limited to cases with sheathing attached to the Narrow Face of the Chord since the 2002 and later editions of the Standard.

In general, Trusses with lumber oriented flat-wise (e.g., 4x2 Trusses) would typically derive no benefit from an increased E value, since common use of 4x2 lumber is for short panels in floor Trusses where buckling does not control Chord capacity. However, the increase is relevant for 4x2 Trusses with longer panels, as may be used in roof systems.

New to the 2007 edition of the Standard, the C_T factor calculation for SCL uses the same k factor (0.82) as for machine stress rated lumber (MSR). SCL normally has COV less than that of MSR (SCL manufacturers report values less than ten percent are appropriate) and this is now reflected in the Standard by at least stating it has values no greater than that of MSR lumber which is 11 percent (or glulam which is ten percent).

§6.4.7 Temperature Factor (C_t).

Regular cyclical temperature fluctuations in a roof need not be considered as “sustained,” but temperatures over 150 degrees can cause permanent damage to lumber; see *NDS* Appendix C.⁹ It is accepted practice to use tabulated design values without inclusion of temperature factors for Trusses used to support roofs where the roof cavity is ventilated in accordance with good practice.

§6.4.8 Incising Factor (C_i).

The incision limits were updated in the 2007 edition to match changes that occurred after the 1997 edition of the *NDS* (max. incision depth and density changed to 0.4 in. and 1100/in.² from $\frac{3}{4}$ in. and 357/in², respectively).⁹

§6.4.9 Chemically Treated Lumber.

Further information on Preservative Treated Lumber can be found at: www.tpininst.org/publications.html, and through information from the proprietary preservative treatment and fire retardant treatment manufacturers. Allowable lumber design stresses shall be adjusted, when needed, as recommended by these manufacturers.

§6.4.9.2 Metal Connector Plates Installed in FRTW.

Metal Connector Plate design values shall be adjusted per recommendations by the chemical manufacturer.

§6.4.10 Quality Control Factor (C_q).

The quality control factor (C_q) was a new adjustment factor that was added to accommodate changes in the 2002 edition of the Standard and maintained in this edition pertaining to quality in the manufacture of metal-plate-connected wood Trusses. It applies to plate lateral resistance design values to effectively calibrate the amount of built-in quality tolerance used in the design of plates for lateral resistance. Plate lateral resistance design incorporates this reduction, through the addition of a factor (typically 0.8), on plate lateral resistance values in order to account for quality imperfections in the plating area that are considered to be inherent to the manufacturing process. The quality control factor may, under certain cases, also be increased to avoid this reduction and potentially utilize the full lateral resistance design capacity of the plates. The conditions of use on the quality control factor are

⁹ American Forest & Paper Association, *National design specification for wood construction*, ANSI/AF&PA NDS-2005, Washington, DC, 2005.

specified in Sections 6.4.10.2 and 6.4.10.3 (see Section §3.3.2 for more information).

New to the 2007 edition of the Standard, the quality control factor basis changed from a maximum value of 1.25 to a maximum value of 1.00 (i.e., the 20 percent reduction in plate lateral resistance values, or fabrication tolerance, suggested for 2x_q Trusses required a value of $C_q = 1.00$ per the 2002 edition of this Standard but is now obtained using a value of $C_q = 0.80$).

§6.4.11 Other Adjustment Factors.

This section, which permits the use of dry lumber design stresses for green lumber under special conditions, is included to recognize this typical practice in parts of the western United States. In certain locales, it is common practice to fabricate Trusses with lumber that was surfaced at a moisture content exceeding 19 percent (SGRN), while the Wood Member design was based on dry design properties for all load cases, including construction loads. This is an accepted practice in these locales, due to the fact that the 2x lumber reportedly dries in a very short time, prior to application of any governing design loads. For example, the air-drying time for Douglas Fir 2x dimension lumber in Redding, CA is seven days in the summer.¹⁰ This short drying time is less than the time required to fabricate, ship, install and enclose the Trusses.

The appropriateness of this provision requires conditions in the geographical area to permit air-drying prior to the closing in of the structure, which is addressed by the conditions specified in Section 6.4.11. Thus, this provision will only apply in desert or low-humidity areas where air-drying is rapid enough to assure the wood will be at a sufficiently low moisture content prior to closing in of the structure.

§6.5 CORROSIVE ENVIRONMENTS

If the environment for the intended end use of the Trusses is expected to result in wood moisture content exceeding 19 percent, unusually high temperatures for typical wood Building construction, or unusual corrosion potential, the Registered Design Professional for the Building or Building Designer is responsible for specifying these conditions on the Construction Documents (see Sections 2.3.2.4 or 2.4.2.4). In these cases, a means for providing increased Metal Connector Plate resistance to the corrosive environment must be specified by the Truss Design Engineer or Truss Designer, unless specified by the Registered Design Professional for the Building or

Building Designer depending on the type of project. The decision of which option to use for increasing corrosion resistance, whether using one of the coatings recognized in Section 6.5.1 or another acceptable coating or means as determined by the Registered Design Professional for the Building or Building Designer, should be made with respect to the particular conditions, i.e., the suitability for the specified environment, given the advantages and drawbacks of the various options.

§6.5.1 Recognized Coatings.

The dual system of G60 hot dip galvanizing and proven barrier-coat painting options affords a synergistic improvement in protection of steel against corrosion, typically 50 percent greater than the simple addition of the protection provided by zinc or paint separately.¹¹

In one independent study of corrosion resistance effectiveness, a number of painting options (treatments) were applied to standard galvanized G60 connector plates embedded into a splice joint.¹² The combinations of various paints and G60 zinc were compared to control samples of Truss plates that were re-galvanized in accordance with ASTM A153 requirements, by subjecting all the plates to the same accelerated freeze/thaw and salt fog tests. Of the eight coating/wash primer combinations studied, those indicating the highest probability of lasting 70 or more years in an enclosed marine environment were epoxy, coal tar epoxy, and asphalt mastic with wash primer, and these coatings have been included in the Standard since 1985.

These coating systems, as specified in (a)-(c), should be considered as an alternative corrosion protection option to the code criteria that allow double dipped galvanized or stainless steel Metal Connector Plates used in Coastal High Hazard and Ocean Hazard Areas (see *FEMA Technical Bulletin 8-96, Corrosion Protection for Metal Connectors in Coastal Areas for Structures Located in Special Flood Hazard Areas in accordance with the National Flood Insurance Program*).

Prior to application of the paint coatings referenced in Section 6.5.1, the surface of the galvanized G60 Metal Connector Plates shall be solvent cleaned in accordance with SSPC-SP 1 "Solvent Cleaning."¹² Also, the paints shall be applied in accordance with SSPC-PA 1 "Shop,

11 American Hot Dipped Galvanizers Association, Inc. and Zinc Institute, Inc., *Painting galvanized structural steel*, Washington, DC, 1986.

12 J. A. Bruno, R. E. Weaver and B. R. Appleman, *Evaluation of coatings for metal connector plates*, SSPC Report 87-08, Steel Structures Painting Council, Pittsburgh, PA, 1989. www.sspc.org

Field, and Maintenance Painting.”¹³ The study found that spray-applied coatings were not practical for Metal Connector Plate joint protection compared to brush-applied coatings, which provided unbroken coverage over the entire joint.¹² Thus, the SSPC paints shall be brush applied, taking care to work the coatings into the slot openings and exposed underlying wood. Specifications for SSPC paints are found in the SSPC Painting Manual;¹³ the suitability and appropriate dry film thicknesses of the SSPC paints in (a)-(c) are as follows:

(a) SSPC PAINT 22 - Epoxy Polyamide Paint is suitable for exposures in SSPA rated environmental zones 2A (frequently wet by fresh water), 2B (frequently wet by salt water), 3A (chemical - acidic), 3B (chemical - neutral), and 3C (chemical alkaline).

SSPC Paint 22 shall be brush applied to a dry film thickness of 2.5 mils at its thinnest point.

(b) SSPC PAINT 16 - Coal Tar Epoxy-Polyamide Black (or Dark Red) Paint is suitable for exposures in SSPA rated environmental zones: 2A (frequently wet by fresh water), 2B (frequently wet by salt water), 2C (fresh water immersion), 2D (salt water immersion), 3A (chemical - acidic), 3B (chemical - neutral), and 3C (chemical - alkaline). SSPC Paint 16 shall be brush applied to a dry film thickness of 8 mils at its thinnest point. SSPC PAINT 27 - Basic Zinc Chromate Vinyl Butyral Wash Primer and SSPC PAINT 12 - Cold Applied Asphalt Mastic (Extra Thick Film). SSPC Paint 12 is suitable for exposures in SSPA rated environmental zones 2A (frequently wet by fresh water), 2B (frequently wet by salt water), 3B (chemical - neutral), and 3C (chemical - alkaline). One primer coat of SSPC Paint 27 shall be brush applied to a dry film thickness of 0.3 mils. SSPC Paint 12 shall then be brush applied as a topcoat to a dry mil film thickness of 5 mils at its thinnest point. Unless otherwise specified, it is advisable that the purchaser or Building Official reserve the right to inspect the applied SSPC paint system in accordance with SSPC-PA 2 “Measurement of Dry Paint Thickness with Magnetic Gages (Non-Destructive).”¹³

The study concluded that the paint coating systems over standard galvanized connector plates would be expected to out-perform the double galvanized Metal Connector Plates in field use. However, other options may be more appropriate when ocean salts or corrosives are not present in the air, or when plates extend off of the wood, and re-galvanizing is another option that is recognized for increasing corrosion protection. Although ASTM A153 regulates proper application of the re-galvanizing (i.e.,

excess zinc is required to be spun off by centrifuge) and thus addresses such concerns relating to excess zinc, some reduction in tooth holding or other warnings may be appropriate with re-galvanized plates due to the environments where re-galvanized plates are likely to be specified, such as where there is the potential for increased moisture content, in which case a C_M factor may be necessary.¹⁴

§6.5.3 Stress Corrosion Cracking.

Stress corrosion cracking (SCC) is an issue for components in swimming pool Building atmospheres that are safety-critical and load-bearing, but are not washed or cleaned frequently. Types 304 and 316 stainless steel have been found to fail due to SCC in highly aggressive chloride environments and have been affected by SCC in some swimming pools.¹⁵

More highly alloyed grades of austenitic stainless steel have a much greater degree of SCC resistance. Two grades have been tested and found to be resistant to SCC under laboratory conditions, namely 317 LMN and 904 L.¹⁵

§6.6 USE OF METAL CONNECTOR PLATES IN CONVENTIONAL WALLS (LADDER FRAME)

The 2007 edition of the Standard added provisions for connector plates in conventional walls up to 24 in. high (i.e., ladder Frame or cripple walls) using 1x3 connector plates for stud-to-wall-plate connections instead of nails or staples, and cutting/notching of wall members is permitted per governing Building Code provisions for conventional walls.

Building Code Officials have been reported to require certification for Ladder Frames because of the metal plates used to fasten the vertical “stud” members to the Chords. Additionally, repair drawings have been reported to be required for member cutting and notching of Ladder Frames whereas standard Building Code language is accepted for conventional wall cutting and notching. Section 6.6 is intended to aid in permitting the use of Metal Connector Plates as an alternate to other types of fasteners in walls that are otherwise prescriptively designed per typical Building Codes, and to clarify that provisions within the Building Code for cutting or notching of studs in such walls are permitted to be applied to such Ladder Frame design. It is hoped that this will eliminate the

¹⁴ American Society for Testing and Materials, *Standard specification for zinc coating (hot-dip) on iron and steel hardware*, ASTM A 153/A 153M-00, West Conshohocken, PA, 2000.

¹⁵ Nickel Development Institute, *Stainless steel in swimming pool buildings*, Toronto, Ontario, 1995.

need for Truss repair drawings for such Ladder Frames when such drawings are requested solely because they are modified in accordance with standard notching provisions permitted by the Building Code.

CHAPTER 7

MEMBER DESIGN PROCEDURES

Overview of (non-editorial) Changes

New to TPI 1-2007

- 60)** Example of accepted structural analysis method for determining effective buckling length for Chord members changed from a reference to PPSA to equations. (Section 7.2.1.1)
- 61)** Effective buckling length factors for members subject to Sidesway, such as Chord Overhangs or members extended beyond Truss joints to bearings (or Chord segments in un-triangulated panels possibly as shown in figure but not mentioned in text), specified to be 2.1. (Section 7.2.1.2)
- 62)** Clarification that effective length factor of 0.8 only applies to end segments of Webs with two Lateral Restraints with that factor being 1.0 for the center segment between the CLRs. (Section 7.2.2.1)
- 63)** Compression Webs restricted to a maximum of two CLRs. (Section 7.2.2.2)
- 64)** Compression design equation denominators reverted back to *NDS* format with "c" since Structural Composite Lumber uses a different value for "c." (Section 7.3.2)
- 65)** Changes in format to F_{ce} (Section 7.3.2) and F_{be} equations (Section 7.3.3.1).
- 66)** Heel bending factor increase to allowable bending stress of 30 percent now limited to solid sawn lumber, and changed to ten percent for application to Structural Composite Lumber. (Section 7.3.3.6)
- 67)** K_m factor permitted to modify solid sawn lumber bending stresses, provided heel bending factor increase is not used. (Section 7.3.3.6)
- 68)** Stacked bending members not glulaminated or otherwise connected to assure composite action is specified to be designed assuming no composite action, except as justified by the actual connections. When connections are Truss plates at 30 in. on center or less, design may presume a moment of inertia and section modulus equal to 60 and 70 percent, respectively, of a fully composite member. (Section 7.3.3.7)
- 69)** An exception to the maximum L'/d ratios for compression and tension members was provided to permit exceeding the given ratios when design calculations are performed to account for the interaction of axial compression with initial deformation of compression members due to warp or other causes. (Section 7.3.6)

- 70)** The maximum L'/d ratio of 50 for compression members was modified to apply only to "long-term" compression members and the max. L'/d ratio of 80 for tension members was modified to apply only to tension members subject to reversal of stress due to short-term loads other than gravity loads. (This means *TPI 1* does not explicitly assign a maximum L'/d ratio limit for tension members not subject to reversal of stress, or tension members subject to reversal of stress from short-term gravity loads. (Section 7.3.6)
- 71)** C_H deleted from use to adjust F_v . (Section 7.3.7.1)
- 72)** Wood shear design provisions added for tapered Bottom Chord heels (girder-cut heel, similar to TPIC provision). (Section 7.3.7.2)
- 73)** Scarf-cut bearings supported at outside end of scarf cut must be checked for tension perpendicular to grain fracture at inside edge of bearing using *NDS* shear in joints check. (Section 7.3.7.3)
- 74)** Bearing perpendicular to grain allowable stress modified to recognize appropriate adjustment factors, including a new bearing plate increase factor recognizing an 18 percent increase to Narrow Face compression perpendicular to grain capacity per the *NDS* limit when plates on the wide faces extend to within $\frac{1}{4}$ in. of the bearing surface. (Section 7.3.8.3)
- 75)** Bearing perpendicular to grain limit added for ultimate limit state of elastic buckling limit in the perpendicular to grain direction (may limit deeper members, like 2x8+, not supported at intermediate points along their depth). (Section 7.3.8.3)
- 76)** Increased compression perpendicular to grain bearing permitted for serviceability concerns only or, for heels, if heel plate sufficiently close to bearing surface. (Section 7.3.8.3)
- 77)** Allowable bearing stress parallel to grain now defined as F_c^* instead of F_g' , and subject to all relevant adjustment factors. (Section 7.3.9)
- 78)** Rigid inserts, like 20g metal inserts, for bearing surfaces with parallel to grain compression bearing stress exceeding 75 percent of allowable must now be illustrated or specified on the Truss Design Drawing. (Section 7.3.9.2)
- 79)** Truss bearings now specified to be 1.5 in. minimum length if wood or metal supports and 3 in. if concrete or masonry supports except with mechanical fasteners or as approved by a Registered Design Professional. (Section 7.4.1)

- 80)** Truss bearings also must satisfy that required by allowable bearing stress, which is a reference to a value specified by the Building Designer as an applicable limit to be used for design of the Truss bearing. (Section 7.4.1)
- 81)** Equation for limiting tension perpendicular to grain in girder Truss connections modified as follows (Section 7.5.3.2):
- o The cross-sectional area (used for calculating applied shear stress) is now restricted to that corresponding to the effective depth of the member (was entire depth of the member).
 - o For connections other than Truss plates, such as joist hangers, the cross-sectional area (used for calculating applied shear stress) is limited to the area in two plies regardless of the number of plies in the girder. (Section 7.5.3.2.1)
 - o For locations within a distance equal to five times the depth of the member from its end, the application of the more limiting $(d_e/d)^2$ term is required as prescribed by *NDS*, and the equation is required to be checked when the applied force exceeds 400 lbs. (versus 800 lbs.). (Equations E7.5-2 and E7.5-4)
 - o For Truss plate connections, the exception to the equation when the connection extends across 85 percent of the cross-section depth is now permitted. (Sections 7.5.3.2.3 and 7.5.3.3.1)
- 82)** Ply-to-ply connection design for withdrawal load can be avoided if the connections complies with the *NDS* for use of the K_f value used for compression design, and clarification is given that withdrawal design load for such connections in sheathed members may consider the unbraced length to be ten times the cross-section dimension in the buckling direction (i.e., 15 in. typically). (Section 7.5.5.3)
- 83)** Bolt spacing in ply-to-ply connections is permitted to exceed 24 in. if other fasteners are designed to carry the ply-to-ply loads. (Section 7.5.5.5)
- 84)** Total deflection calculation now explicitly specifies a component due to creep of no less than 50 or 100 percent of the initial deflection (meaning creep factors of 1.5 or 2) for long-term loads for dry and green (or wet service) use, respectively. (This was previously in the *TPI 1-2002 Commentary*.) (Section 7.6.1)
- 85)** Floor Truss deflection limit for total load added ($L/240$ to match *IBC*) and prior total load deflection limit of $L/360$ specified for floor Trusses supporting ceramic tile deleted and replaced with footnote referencing other criteria per *ANSI A108/A118/A136* and Building Designer requirements. (Table 7.6-1)
- 86)** Top Chord panel deflection limit of $L/600$ noted for beam or lintel supporting vertical masonry veneer per *ACI530/ASCE5/TMS402*. (Table 7.6-1)
- 87)** Live load deflection limit of $L/360$ noted as applicable for panel deflection of habitable spaces in Trusses. (Table 7.6-1)
- 88)** Deflection limits for Cantilevered portions of Trusses is also noted as applicable to Overhang portions of Trusses. (Table 7.6-1)
- 89)** Nails for attachment of strongbacks to Trusses now specified as 10d (0.131 in. x 3 in.) instead of 16d common. (Section 7.6.2.4)

§7.1 SCOPE

Member design provisions in the Standard prior to the 2002 edition included an empirical analysis approach for determining critical bending moments and buckling lengths. The empirical analysis approach that was included in the Standard from 1978 through 1995 used Q-factors, derived from extensive investigation using a structural analysis program (PPSA)¹ on standard Truss configurations, as a means to calculate effective panel lengths for a determination of panel point and mid-panel moments, based simply on Truss geometry and empirically derived constants.

The approximate method served an important purpose in the past because a more exact analysis method, using finite element or matrix analysis in combination with accurate analog assumptions, used to be beyond the computational capabilities of many engineers. However, this is no longer the case, and matrix methods are now preferred as the more accurate way of assessing the stresses in a Truss structure. This preference towards a more exact analysis has also been magnified by the increased complexity of Truss profiles and configurations over the years, whereas the approximations inherent in the empirical analysis require limitations upon its use. In recognition of its limited and increasingly outdated applications, the empirical method has been removed from both the Standard and the non-mandatory Appendix of the Standard. The structural analysis approach recognized by the 2007 edition is one in which the Truss members are designed to resist forces and bending moments that are determined using a matrix analysis or other accepted structural analysis method that accurately assesses the stresses in the Truss.

§7.2.1 Effective Buckling Lengths for Chord Members.

An acceptable structural analysis method for determin-

¹ S. K. Suddarth, *A computerized wood engineering system: Purdue plan structures analyzer*, Research Paper FPL 168, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI, 1972.

ing effective buckling lengths of Chord members must consider member end fixity. An example of an accepted structural analysis method for determining effective buckling length for Chord members changed in the 2007 edition of the Standard from a reference to the PPSA computer program to an equation. The determination of effective Chord lengths within the PPSA software has been modified (since the time this section was originally included in the Standard) to make use of a published equation instead of the numerical solution included in the original PPSA versions. This equation was added to the Standard in place of the indirect reference through PPSA, and also to provide a more consistent approach for all those involved with wood Truss design by informing all users, including engineers with oversight over Truss design but who may not be as familiar with this equation, as effective length factors for Truss Chords are sometimes conservatively presumed to be 1.0 in the absence of any other specifications.²

The equation presented for K is appropriate to apply to joints where two members are attached to each other with full rotational rigidity. If more than two members are rigidly attached to each other, it is theoretically appropriate to incorporate the rotational stiffness of all of the adjacent members. However, since *TPI 1* states that the effective length factor for Webs shall be 0.8 and since Webs are not anticipated as being modeled with rigid or partially rigid connections at present, it is suggested that the above discussion of effective length factors apply only to Chord members. It is appropriate for adjustments to be made in the Truss analyses for joint rotational stiffness when the end fixity is provided through a plated joint, rather than by the continuation of the member through the joint.

§7.2.1.1 Chord Member Panels Not Subject to Sidesway.

The lateral movement of a structure under the action of lateral loads or unsymmetrical vertical loads is defined as Sidesway.

As shown in Figure 7.2-3, the Overhang of the Truss is an example of unbraced Chord member.

§7.2.1.2 Chord Member Panels Subject to Sidesway.

Effective buckling length factors for members subject to Sidesway, such as Chord Overhangs or members extended beyond Truss joints to bearings (or Chord segments in un-triangulated panels possibly as shown in Figure 7.2-3 but not mentioned in text), are specified to be 2.1.

² Kavanaugh's equation is taken from Equation 9 of the article titled "Effective Length of Framed Columns" in the February, 1960, Journal of the Structural Division, ASCE.

§7.2.2 Effective Buckling Lengths for Web Members.

The use of 0.8L in determining the Web buckling length has been specified in the Standard since 1965 based on the degree of member end fixity and departure of the member end conditions from the pin joint condition contemplated by the Euler Formula. Research tests have verified this long-standing design assumption for compression Web members in Trusses.³

The 2007 edition of the Standard provides clarification that effective length factor of 0.8 only applies to end segments of Webs with two intermediate Lateral Restraints with that factor being 1.0 for the center segment between the intermediate Lateral Restraints.

Compression Webs that require three intermediate Lateral Restraints are not permitted in the 2007 edition. The concern is that Webs requiring three intermediate Lateral Restraints could potentially fail during Truss installation before the bracing is installed but with construction loads applied.

§7.3 MEMBER DESIGN

The member design equations in Section 7.3 of the Standard are recognized Allowable Stress Design (ASD) procedures for wood design. Thus, the basic design equations and accompanying criteria for tension members, compression members, bending members, etc., are the same as those found in the *NDS*, appended as necessary with additional provisions for their use in wood Truss design applications.⁴ This Commentary section is limited to a discussion on those provisions specific to this Standard. Refer to the *NDS Commentary* for additional discussion on the basic member design provisions.⁵

§7.3.1 Tension Members.

The determination of the allowable tension design value, F_t' , includes the repetitive member factor, C_r , which has previously been limited in its application to allowable bending design values only. Refer to Section §6.4.2 for discussion on the applicability of the repetitive member factor to tension design values.

The effect of a reduced cross-section when subject to normal stresses, such as those referenced in Section 7.3.7.2,

³ D. Grant, F. J. Keenan and J. E. Korhonen, *Effective length of compression web members in light wood trusses*, Forest Products Journal, Vol. 36, No. 5, 57-60, 1986.

⁴ American Forest & Paper Association, *National design specification for wood construction*, ANSI/AF&PA NDS-2005, Washington, DC, 2005.

⁵ American Forest & Paper Association, *Commentary on the national design specification for wood construction*, 2005 edition, Washington, DC, 2005.

should also be considered when evaluating normal stresses on the cross-section.

§7.3.2 Compression Members.

The determination of the adjusted design value for compression term, F_c^* , includes the repetitive member factor, C_r , which has previously been limited in its application to allowable bending design values only. Refer to Section §6.4.2 for discussion on the applicability of the repetitive member factor to compression design values.

Equation E7.3-3 was modified in the 2007 edition of the Standard. The compression design equation denominators reverted back to the *NDS* format with the parameters “c” since Structural Composite Lumber uses a different value for “c.” Refer to Equation 3.7-1 in *NDS-05* Chapter 3.7.1

§7.3.3.5 Bottom Chord Bending Members.

Gypsum board that is adequately attached directly to Bottom Chords of Trusses has proven to provide adequate lateral support for the Bottom Chords of Trusses in residential structures for many years. In related testing of gypsum sheathing, the test results demonstrated that gypsum wallboard contributes significantly to the racking strength of light-framed walls.⁶

§7.3.3.6 Panel Point Moment Region at the Heel.

This provision addresses a length effect in the Top and Bottom Chords when only a short fraction of the panel length is subject to high moment stresses in the heel region. These high moment stresses, in which the maximum moment occurs at a single point and drops off rapidly from this point, are caused by the eccentricity of the Chord forces, particularly in Trusses with deep (e.g., 2x12) members, such as Trusses used in post-frame construction. The increase in allowable bending capacity of the Top and Bottom Chords in the heel region of the Truss, as permitted by this new provision, only applies when the full eccentricity of the Chord force action lines is accurately modeled in the structural analysis, resulting in the prediction of additional moment. Consequently, the provision is incompatible with an empirical type analysis that models the heel joint as a pinned, concentric connection.

The rationale behind this length effect adjustment is that the localized moment at the heel, which occurs typically over less than one-third of the panel length, should not necessarily control the size and grade of the entire Chord member. In general, the length effect concept acknowl-

edges that the strength of a beam decreases with an increase in length. The specific application of the length effect to the heel region of the Truss is additionally supported by the adequate performance of Trusses in agricultural, post-frame structures that have formerly been designed using empirical analysis methods employed by earlier editions of the Standard. In earlier editions, the Standard incorporated an empirical method to provide a satisfactory, yet simplified means to determine panel point and mid-panel bending moments. However, because it assumed a pin connection at the heel joint, the empirical analysis neglected moment at the heel, and thus the heel joint did not ever control the overall design. While this approach has resulted in adequate designs, it did so without consideration of the eccentric heel moment and the associated strength increase of the short region subject to this bending. Recognition of this length effect validates the empirical designs in the past, and accommodates the use of a more exact analysis method that accurately reflects the heel eccentricity and accounts for the heel moment accordingly.

A length effect factor is a standard adjustment used in establishing lumber design values and to correct for one bending load test method versus another, i.e., 2-point concentrated loading versus a single mid-span concentrated load. Methods of adjusting for length effects vary slightly in different standards. *ASTM D1990* provides a formula for adjusting bending strength, where the length effect is accounted for by raising the ratio of lengths, L_1 and L_2 , to a 0.14 power.⁷ The 30 percent increase specified in this Standard is a simplified adjustment that can be obtained using conservative assumed lengths in the *ASTM* formula. Assuming a characteristic length of 12 ft., and assuming that the length subject to high moment stress is, conservatively, 2 ft., the resulting *ASTM D1990* adjustment is 1.29.

Section 7.3.3.6 is limited to the heel region of the Truss and when the bearing is under the Bottom Chord and within the scarf of the heel joint. These limits exclude Cantilever or tail-bearing conditions from this provision. While a Top Chord Overhang condition is not excluded from this provision, the increase in allowable bending should not be applied to the Overhang portion of the Top Chord, or up to the node of the Overhang member. The limit on the application of this provision to a length of two times the depth of the Chord is intended to keep the applicable region small relative to the length of the Chord members.

⁶ R. W. Wolfe, *Contribution of gypsum wallboard to racking resistance of light frame walls*, FPL-RP-439, U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI, 1983.

⁷ American Society for Testing and Materials, *Standard practice for establishing allowable properties for visually-graded dimension lumber from in-grade tests of full-size specimens*, ASTM D 1990-97, West Conshohocken, PA, 1997.

New to the 2007 edition of the Standard, a heel bending factor of 1.10 was added for Structural Composite Lumber (SCL). The heel area increase in bending strength in previous editions of 1.30 was based on solid sawn lumber. The value of 1.10 for SCL is based on recommendations from TrusJoist for a value of 1.11 for laminated strand lumber (LSL). The K_m factor is now also permitted to modify solid sawn lumber bending stresses, provided heel bending factor increase is not used.

§7.3.3.7 Composite Action of Multiple Layers.

Section 7.3.3 was revised in the 2007 edition to regard stacked members as individual members between connections, unless the connections are designed to assure fully composite action, or should provide guidance from available data.⁸ The addition is based on this article, summarized below:

The article investigated effectiveness of Truss plates connecting two 4x2 Chords. Beams 3 in. (76 mm) deep x 3.5 in. (89 mm) wide were tested in bending using a mid-span concentrated load over a 7.5 ft. (2286 mm) simply supported span. The beams were fabricated with 3x4.4 in. (76x111 mm) plates at 30 in. (762 mm) spacing or 3x8.8 in. (76x222 mm) plates at 24 to 30 in. (610 to 762 mm) spacing along the Chord. Beams were tested for stiffness and comparative fully effective section properties were then found by disassembling half of the beams, removing plates, rebuilding them by gluing to form a fully effective stacked double 4x2 Chord, and re-testing. The plated beams gave stiffness (moments of inertia) averaging 56 and 75 percent of the fully effective section stiffness for the smaller and larger Truss plates, respectively. This data was then used to calculate an effective depth and effective section modulus of the stacked Chord cross-section, which were found to average 68 and 83 percent of the fully effective section modulus (which would correlate to ultimate strength) for the smaller and larger Truss plates, respectively. These ratios of effective section moduli were approximately confirmed by the beam failure loads, which showed average strength of plated beams were 69 and 90 percent for the smaller and larger Truss plates, respectively, of the corresponding glued (fully effective) beam strength. If the beams consisted of completely unconnected 4x2s, one would expect stiffness of only 25 percent and strength of only 50 percent of the fully effective section, so the plated beams performed much better

than unconnected 4x2s but not as good as rigidly connected 4x2s.

§7.3.6 L'/d Ratios for Compression & Tension Members.

Limits on L'/d have been specified in the Standard and its predecessor Standards since the original *TPI 1* Standard was published. The original Standard in 1960 included the L'/d limit of 80 for tension members subject to temporary reversal of loads, the 1965 edition added a limit of 50 for compression members, and the 1970 limit extended the limit of 80 to all tension members regardless of load reversal. The L'/d limit of 50 for wood compression members has been long recognized in wood design standards, including since 1944 in the *NDS* but higher limits for wood under temporary compression loads were not recognized in the *NDS* until 1991, when a limit of 75 was introduced. The L'/d limit of 80 specified by the Standard for wood tension members accounts for short-term stress reversals and accordingly reflects a short-term increase on the compression L'/d limit of 50, for members stressed primarily in tension. The origin of these limits is believed to be similar limits applied to the ratio of length to radius of gyration (r) of 200 for compression and 300 for tension. For a rectangular member of depth d, these L/r limits convert to L/d limits of 57.7 and 86.6, and it is presumed that these were rounded downward to the next lowest increment of ten in recognition of the approximate nature of these limits. The *NDS* limit for temporary compression limits differs, and this is attributed to application of a 1.5 multiple taken from the steel design limits of 200 and 300 on L/r, rather than the approach used in this Standard of direct conversion and rounding down. These limits are prescriptive and intended to prevent interaction of creep on compression buckling capacity in lieu of more explicit calculation to account for creep effects.

The 2007 edition of the Standard added an exception to the maximum L'/d ratios for compression and tension members. This was provided to permit exceeding the given ratios when design calculations are performed to account for the interaction of axial compression with initial deformation of compression members due to warp or other causes.

The maximum L'/d ratio of 50 for compression members was modified in the 2007 edition of the Standard to apply only to “long-term” compression members and the maximum L'/d ratio of 80 for tension members was modified to apply only to tension members subject to reversal of stress due to short-term loads other than gravity loads. This latter limit is intended to address members subject to compression only from wind or seismic load cases. (This means *TPI 1* does not explicitly assign a maximum L'/d

⁸ J. H. Emanuel, C. M. Newhouse and D. B. Brakeman, *Investigation of metal-plate-connected 4 by 2 beams*, Forest Products Journal, Vol. 37, No. 11/12, 1987.

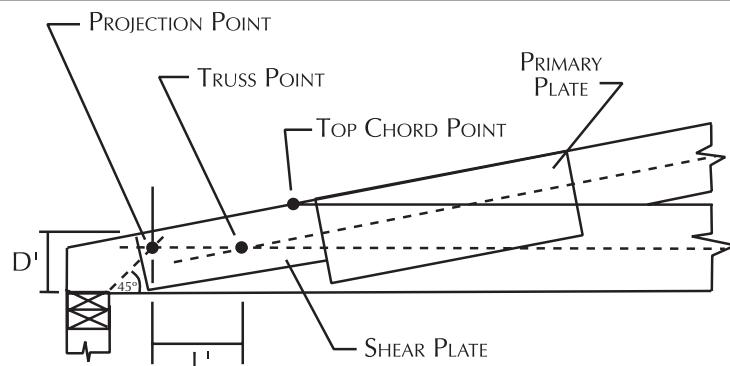
ratio limit for tension members not subject to reversal of stress, or tension members subject to reversal of stress from short-term gravity loads.)

§7.3.7.2 Tapered Bottom Chord Heel.

New to the 2007 edition of the Standard, the need for shear analysis at shallow heels is now recognized. For example, some Trusses may be designed to require a 1 in. heel height with plates 8 in. to 12 in. from bearing.

The Standard addresses the wood shear design provisions similar to the method the Truss Plate Institute-Canada (TPIC) uses.

According to TPIC, horizontal shear is checked at a distance away from the inside face of the bearing based on the intersection point of the centerline of the Bottom Chord member and a line 45 degrees from vertical extending from the inside edge of the bearing upward and



*Appendix F of TPIC-2007 Standard.
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Truss Point - the intersection of the centreline of top chord and the centreline of bottom chord.

Projection Point - the intersection of the forty five degree line from the inside edge of the bearing and the centreline of bottom chord.

Top Chord Point - the beginning point of the top chord scarf.

Shear Plate Requirements:

A shear check is required when the projection point is closer to the end of the truss than the truss point.

If $P_A > P_w$, then shear plates are required.

Where

P_A = The factored shear load at the girder heel, N (lbs)

$$P_A = \frac{1.5RL'}{D'}$$

R = Reaction N (lbs)

L' = Distance between projection point and truss point, mm (in)

D' = Lumber depth at projection point, mm (in)

P_w = The factored shear resistance of the lumber without plate, N (lbs)

$$P_w = \phi F_v t L'$$

ϕ = 0.9

$$F_v = f_v K_{Dv} K_{Hv} K_{Sv} K_{Zv}$$

t = Lumber thickness per ply, mm (in)

f_v = specified strength in shear, MPa (Psi)

n = Number of plies

K_D = Load duration factor

K_{Hv} = Load sharing factor for shear

K_{Sv} = Service condition factor for shear

K_T = Treatment factor

K_{Zv} = Size factor for shear

Shear Plate Design:

The shear plate must be sized and placed about the centreline of bottom chord to cover the distance L' such that:

- i) Area of the shear plate above or below the centreline of bottom chord must be capable of resisting the net shear force at the girder heel.

$$\text{Net Shear Force} = \frac{P_A - P_w}{n}$$

- ii) The length of the shear plate along the centreline of bottom chord must be such that the shear capacity of the plate, along the centreline of bottom chord, is greater than or equal to the net shear force at the girder heel. The shear length as calculated must not be less than L' .

Notes:

1. Where the primary plate interferes with the placement of the secondary shear plate then the primary plate must be specified long enough to provide the required grip and metal shear capacity due to the net shear force.
2. The tapered depth of the bottom chord at the inside edge of bearing should not be less than half the bottom chord size of 100mm (4in), whichever is greater.
3. An additional moment check should be carried out due to extension of the bottom chord past the top chord. The moment to be used for this check is the overall span reaction times the distance from the inside edge of the bearing to the top chord point.

Figure C7.3-1. Shear Plate Design for Girder Type Heel Joint.

inward. The procedure that is used to determine whether a shear plate is required is based on Canadian Limited States Design (see Figure C7.3-1).

§7.3.7.3 Scarf Cut Bearings.

New to the 2007 edition of the Standard, the scarf cut bearings supported at outside end of scarf cut must be checked for $F_{c\perp}$ fracture at inside edge of bearing using the NDS 2005 shear in joints check.

§7.3.7.4 Use of Metal Connector Plates to Resist Shear.

Lumber members may be reinforced with Metal Connector Plates on each lumber face. The primary purpose of the Metal Connector Plate is to prevent horizontal shear stress failure in the reduced lumber cross section. The Metal Connector Plate must be of sufficient size and location to ensure the shear load is transferred from the lumber into the Metal Connector Plate; sufficient gauge to resist the anticipated shear stress; and sufficient length to ensure the uncovered wood at the Metal Connector Plate edge can resist calculated shear stresses. The following guidelines are recommended unless alternate provisions are documented by appropriate testing in meeting these parameters.

- a) Lumber reinforced with Metal Connector Plates stress shall be limited to either the shear capacity of the Metal Connector Plate pair in the direction parallel to the wood length or the shear stress capacity of the wood section. To convert the metal plate steel capacity to an equivalent lumber allowable shear stress (F_v'), divide the allowable steel shear value for the pair of plates (at the angle corresponding to the intended plate orientation relative to the wood member length) by the lumber thickness. This limitation is specified as the brittle nature of lumber in shear is expected to prevent the attainment of the lumber shear strength and the plate shear strength simultaneously (the lumber fractures prior to attainment of the steel shear capacity, leaving only the steel capacity).
- b) Metal Connector Plates shall be positioned to cover no less of the member depth on each side of the overstressed portion of the cross-section (centered on the neutral axis) than that required to transfer the expected shear load in lateral resistance from the lumber into the metal plate. To convert the metal plate lateral resistance capacity for plates symmetrically located about the lumber neutral axis to an equivalent normal lumber allowable shear stress (F_v), multiply the plate lateral resistance value for a pair of plates (at the orientation corresponding to

the intended plate orientation relative to the wood member, i.e., V_{LRAA} for plate length parallel to grain, V_{LREA} for plate width parallel to grain) by half the plate dimension perpendicular to the wood member length and then divide by the lumber thickness.

- c) The Metal Connector Plate shall cover the entire length of wood which is overstressed in shear including, at reaction locations, a sufficient distance across the bearing support to prevent wood shear fracture.

It has been reported that 3x_ or wider Truss plates oriented parallel to grain and centered on both faces of 1.5 in. thick lumber so as to cover 70 percent or more of the cross-section typically permit doubling allowable lumber shear capacity.

§7.3.8 Bearing Perpendicular to Grain.

For checking bearing perpendicular to grain, the actual stress induced in compression perpendicular to grain ($F_{c\perp}$) at reactions, joints, or from loads applied to members, shall be based on the net bearing area.

New to the 2007 edition of the Standard, the ultimate $F_{c\perp}$ limit state was added based on an elastic buckling limit in the perpendicular to grain direction. This new $F_{c\perp}$ limit is a function of the reference modulus of elasticity and the cross-sectional in-plane and out-of-plane dimensions of the Truss member(s) being checked, as described in Equation E7.3-11. This new limit may affect deeper members, like 2x8 and larger depths that are not supported at intermediate points along their depth. The 2007 edition also provided for an increased bearing value, including an 18 percent increase for bearing surfaces if Metal Connector Plates are within $\frac{1}{4}$ in. of the edge on adjoining surfaces of the member. These increases are permitted for serviceability concerns only, meaning they are applicable only to the NDS tabulated $F_{c\perp}$ values only, not the new ultimate limit state equation for $F_{c\perp}$. The 18 percent increase mentioned is based on testing with Metal Connector Plates within $\frac{1}{4}$ in. of the bearing surface,⁹ summarized below, and is therefore only applicable to lumber sections within $\frac{1}{4}$ in. of a Metal Connector Plate. For example, to get the 18 percent increase over the full 3.5 in. dimension of a Truss bearing on a steel framed 3.5 in. wide wall, a Truss plate must be present on both faces of the 2x_ member over the bearing and must be within $\frac{1}{4}$ in. of the bearing edge of each face over the full 3.5 in. length of the bearing.

⁹ N. S. Bulmanis, H. A. Latos and F. J. Keenan, *Improving the bearing strength of supports of light wood trusses*, Canadian Journal of Civil Engineering, Vol. 10, No. 2, 306-312, 1983.

Summary of Canadian Journal of Civil Engineering article:¹⁰

Compression perpendicular-to-grain tests with SPF lumber were done on an edgewise 2x6 (i.e., Truss Bottom Chord) over a flat-wise 2x4 (i.e., sill plate). Results show: 1) embedding Truss plates 1/4 in. from the bearing surface on the sides of the 2x6 increased the average prop. limit by 18 percent; 2) using a bearing bracket to spread the load over the sill plate increased the average prop. limit by 5 percent; and 3) using both Truss plates and a bearing bracket increased the average prop. limit by 30 percent. All increases are relative to control specimens without Truss plates or bearing brackets. Increases were less at the low end of the distribution (i.e., at the 10th percentile), the Truss plates alone gave a 9 percent increase, the bearing bracket alone gave a 3 percent decrease and the plates and bracket together gave an 11 percent increase. Tests were stopped at 0.28 in. (7 mm) total deflection.

The new ultimate limit state for $F_{c\perp}$ design values was based on research showing that wood compression perpendicular to grain failures can cause catastrophic (as opposed to serviceability) failures at lower than expected loads. There have been reports of Truss failures due to compression perpendicular to grain failure at loads as low as 1.33 times design load,¹¹ and a report by Basta¹² confirmed failures may occur at even lower loads. The new ultimate strength-based design limit equation was based specifically on an elastic buckling basis, although it was recognized this was not entirely accurate as other failure modes initiated by compression perpendicular to grain stresses can also trigger failure; however it was found to provide a reasonable correlation to the reported ultimate failure loads.

While this new strength-based limit for compression perpendicular ($F_{c\perp}$) to grain stresses are specified only within the *TPI 1* Standard at this time, they are equally applicable to other applications of lumber. It is recognized that the occurrence of a catastrophic type of failure is dependent upon the application, with Trusses perhaps more susceptible than many other applications. For example, typical conventional lumber applications are often limited by bending and shear design limits, thus preventing the occurrence of high $F_{c\perp}$ stresses, or are used in applications with an aspect ratio (height/width) of 1.0 or less, such as wall plates and railroad cross ties, where no catastrophic (ultimate) limit state is ever reached, just as in the standard *ASTM D143*¹³ Test Specimen used to establish the $F_{c\perp}$ values tabulated by the *NDS*. Trusses typically minimize bending stresses in lumber, permitting the application of larger compression perpendicular to grain stresses than may be typical in conventional lumber applications, and make use of much larger aspect ratios (e.g., ranging from 2.33 to 7.5) than the 1.0 aspect ratio present in standardized Test Specimens. This provides more opportunity for instability and, where connections are typically located close to the bearing joint, requires the wood section to maintain cohesiveness in order to retain the fastener-to-wood bond through friction. Failure modes can occur which initiate with perpendicular-to-grain wood crushing and eventually result in an ultimate limit state due to subsequent wood splitting/cracking which can lead to reduced section capacity, perpendicular-to-grain wood buckling or other instability across the height of the Wood Member, or fastener disengagement.

§7.3.9 Bearing Parallel to Grain.

New to the 2007 edition of the Standard, the allowable bearing stress parallel to grain value is now defined as F_c^* instead of F_g' , and is subject to all relevant adjustment factors.

§7.4.1 Minimum Length of Bearing.

Section 7.4.1 was created in this edition of the Standard to address Truss bearing items often requested by code officials. It clarifies the minimum bearing length required and makes the Standard consistent with the *2006 IRC*, Section R502.6.

It is not clear if the *IRC*, Section R502.6 should apply to Trusses where the bearing is engineered, and the need for reflecting this provision in the Standard should perhaps be better defined. The *IRC* language has some problems as it apparently presumes only end supported members or is only enforced when an end bearing actually exists as Cantilevers, Overhangs, and ridge boards conflict with this wording. (The *2006 IRC*, Section R502.6 wording requires ends of joists, beams or girders to have not less than 1.5 in. of bearing on wood or metal and not less than 3 in. on masonry or concrete except where supported on a 1 in. x 4 in. ribbon strip and nailed to the adjacent stud or by the use of approved joist hangers. A subsection also requires joists framing into the side of a wood girder

10 Bulmanis, Latos, and Keenan, 306-312.

11 K. C. McMartin, A. T. Quaile and F. J. Keenan, *Strength and structural safety of long-span light wood roof trusses*, Can. J. Civil Eng., 11, 978-1007, 1984

12 C. T. Basta, *Characterizing Perpendicular-to-Grain Compression in Wood Construction Applications*, MS thesis, Dept. of Wood Science & Engineering, Oregon State University, Corvallis, OR, 2005.

13 American Society for Testing and Materials, *Standard methods of testing small clear specimens of timber*, ASTM D 143-94(2007), West Conshohocken, PA, 2007.

be supported by approved framing anchors or on ledger strips not less than nominal 2 in. x 2 in.) Also note that the *IRC* permits shorter ($\frac{3}{4}$ in.) bearings for balloon-style framing, but no wording was proposed to reflect this (i.e., for bearings not at ends of members) as it was not considered necessary.

New to the 2007 edition of the Standard, Truss bearings also must satisfy the required allowable bearing stress, which is a reference to a value specified by the Registered Design Professional for the Building as an applicable limit to be used for design of the bearing surface supporting the Truss.

§7.4.2 Top Chord Bearing Parallel Chord Trusses.

Allowable bearing reactions for Top Chord and intermediate height bearing details [see Figures 7.4-1(a)-(f) for flat-wise lumber and Figure 7.4-2(b) for edgewise lumber] were first specified in 1986 to ensure adequate safety factors. The maximum allowable guidelines were based primarily on results of 73 tests of Top Chord bearing, parallel chord, metal-plate-connected wood Trusses using various wood species, Metal Connector Plate sizes, and both flat-wise and edgewise lumber orientations.¹⁴ The reaction limits are based on gross reaction and ten-year load duration. Thus, the values should be adjusted downward for long-term loading and may be adjusted upward for short-term loading using appropriate duration of load factors.

One Truss member design concern associated with Top Chord bearing parallel chord Trusses is the possibility of shear failure in the extended Top Chord section. Early information gained from test results indicated that for Truss designs with gap distances of $\frac{1}{2}$ to 1 in. between the end vertical and the support, the shear stresses are not critical.¹⁵ However, since the empirical allowable bearing reactions in Table 7.4-1 are based on a maximum gap size of $\frac{1}{2}$ in., gaps exceeding $\frac{1}{2}$ in. cannot use these allowable bearing reactions without taking into account the effects of shear and bending on the extended Chord.

§7.5.2.1 Application of Reactions onto Girder Trusses.

Limiting the spacing of reactions imposed on a girder to 34 in. is based on structural members spaced at 2 ft. on center framing into a girder at 45 degrees.

¹⁴ D. H. Percival, et.al, *Test results from an investigation of parallel-chord, top-chord bearing wood trusses*, Research Report 85-1, Small Homes Council-Building Research Council, Urbana-Champaign, Illinois, 1985.

¹⁵ W. R. McAlpine and O. A. Grossthanner, *Proposed design methods for three typical truss details: Top Chord bearing of floor trusses*, Proceedings of the 1979 Metal Plate Wood Truss Conference, P-79-28, Forest Products Research Society, Madison, WI, 229-231, 1979.

§7.5.2.2 Applied Loads Based on the Most Critical Reaction.

In determining the most critical loading condition, consideration should also be given to unbalanced loads, wind loads, interior supports, Cantilevers, and continuous Trusses over girders (i.e., flat girders carrying Trusses on top of the Top Chord).

It should be noted that the reactions from non-concurrent Bottom Chord live loadings need not be simultaneously applied to supporting Trusses. These loadings are non-concurrent because they are intended to address isolated personnel loads and are not expected to occur on multiple Trusses simultaneously.

§7.5.2.3 Load Continuous Across All Plies.

For face-mounted hangers, the Truss Designer must assume a load distribution into the various plies of multiple-ply girder Trusses. The most simple, idealized load distribution is to assume each ply carries the same proportion of load. This idealized load distribution ignores any effect of eccentricity and presumes no out-of-plane movement or deformation, including no torsional rotation of the Bottom Chord, no moment transfer through the hanger connection, and completely rigid connections between plies. However, there are other mitigating factors that may be considered to offset some of the simplified assumptions inherent in an idealized load distribution.

Several factors may affect the load transfer from ply to ply, such as the type of fastener used to connect the plies together (e.g., nails, bolts, or other), the tightness of the bolts when bolts are used, the fastener spacing, and the characteristics of the individual plies. A notable observation from unpublished test results is that the stiffness of individual plies has a large effect on the percentage of load carried by the plies, depending on the relative stiffness of the various plies. This effect of stiffness is important to recognize because the repetitive system effects (i.e., stiffer Trusses, which are stronger, carry more load) should help to counter any loss in strength due to unequal loadings of each ply caused by loading only on one face of the girder. Another factor to consider is the effect of bracing, particularly if there is bracing of both the top and bottom of the loaded Truss Chord, which would mitigate the effect of eccentricity of load from face-mounted hangers. As an example, the carried member may provide sufficient bracing to the top edge of the loaded Bottom Chord, assuming the bottom edge of the Bottom Chord is sufficiently braced by a ceiling diaphragm.

Finally, it is important to recognize that the issue of stress concentration for face-mounted loads, i.e., the problem of cross-grain tension and shear in the plies that see the

load first, is addressed by the girder Truss member design procedures (see Section §7.5.3) with an empirical-based limit on tension perpendicular to grain forces induced by a face-mounted hanger.

§7.5.3.2 Design for Tension Perpendicular to Grain Forces.

Equations for limiting tension perpendicular to grain forces in girder Truss connections were modified in the 2007 edition of the Standard as follows:

- The cross-sectional area (used for calculating applied shear stress) is now restricted to that corresponding to the effective depth of the member (the entire depth of the member was used in the 2002 edition).
- For connections other than Truss plates, such as joist hangers, the cross-sectional area (used for calculating applied shear stress) is limited to the area in two plies regardless of the number of plies in the girder (see Section 7.5.3.2.1).
- For locations within a distance equal to five times the depth of the member from its end, the application of the more limiting $(d_e/d)^2$ term is required as prescribed by *NDS*, and the equation is required to be checked when the applied force exceeds 400 lbs. (versus 800 lbs.) (see Equations E7.5-2 and E7.5-4).
- For Truss plate connections, the exception to the equation when the connection extends across 85 percent of the cross-section depth is now permitted (see Sections 7.5.3.2.3 and 7.5.3.3.1).

§7.5.3.2.1 Extent of Connection.

This provision addresses a Wood Member design issue relating to the relatively low strengths of wood in tension perpendicular to grain and horizontal shear, which must be considered in side loaded members such as girder Trusses.

Based on information gained by tests on 2x8 lumber loaded in tension perpendicular to grain by Truss plates with varying plate bites, it was recommended that the safe concentrated design load for a regular Bottom Chord joint be limited to 800 lbs., beyond which the joint would require a Metal Connector Plate that extends past the centerline of the member.¹⁶ This 800-lb limit incorporates a factor of safety of 3 for the lowest observed failure load in the testing. The Standard adopted this limit into the 1995 edition for both connector plates inducing tension

forces perpendicular to grain and face-mounted hanger connections that impose cross grain tension forces.

The failure loads observed in the testing were attributed to a combination of horizontal shear and tension across the grain. Since little information is available regarding tension perpendicular to grain stresses, this loading condition has been studied approximately by analyzing the distribution of horizontal shear stresses over the member cross section.

The distribution of horizontal shear stress, f_v , in a rectangular Wood Member is parabolic, with the maximum shear stress occurring at the centerline. The equation for determining shear stress at any point along the parabolic curve is:

$$f_{vy} = \frac{3V}{2A} \left(1 - \frac{y^2}{c^2} \right) \quad (\text{E C7.5-1})$$

Where c is the distance to the member centerline, or one-half the member depth, and y is the distance measured from the centerline. The maximum stress occurs at the centerline, where $y = 0$.

Substituting F'_v for f_v in Equation E C7.5-1, where $F'_v > f_v$ as required for design, and solving for y gives:

$$y \geq c \sqrt{1 - \frac{F'_v 2 A}{3 V}} \quad (\text{E C7.5-2})$$

This equation gives a series of curves varying with lumber size and shear force as shown in Figure C7.5-2(a). This figure assumes shear values for #2 SPF with a DOL = 1.15; other curves can be developed for other species and load durations. Since the equation for determining y is non-linear, the curves are asymptotic to a line representing half the depth of the Wood Member, as shown in Figure C7.5-2(b). From Equation E C7.5-2, it can be seen that as V becomes infinitely large, y approaches c , or half the member depth ($d/2$). This means that the plate or hanger would extend ($d/2$) past the centerline, or to the edge of the Wood Member. Since V can be infinitely large in this design consideration, it is assumed that plate lateral resistance, tension stress or nail holding capacities will eventually control.

The empirical 800-lb. limit on tension perpendicular to grain forces for fasteners with bites not extending at least to the member centerline or beyond, along with an equation based on shear stress that specifies how much past

¹⁶ McAlpine and Grossthanner.

the Wood Member centerline the fastener must extend for F_{\perp} forces exceeding 800 lbs., has proven to be sufficient in preventing tension to perpendicular failures or in-service problems in girder Trusses subject to these forces.

The 800-lb. limit was established from interior joints, meaning that the force was equally distributed to the lumber to either side of the connector plate, and not for a joint at the end of a member where it is all distributed in one direction. A second equation is necessary to account for connections close to the ends of members.

The same issue is also relevant for any connection, not just girder Trusses, so this should be applicable to other Trusses with such connections, such as any Truss with an uplift connector even if not a girder Truss. Section 7.5.3.3 is now referenced by Section 8.10 so it does apply to all Trusses.

The change to Section 7.5.3.3 in the 2007 edition of the Standard to explicitly address any member, rather than simply Chords, is to be consistent with Section 8.10 (which does not limit it to Chord members) and recognize that this situation may apply to Webs as well. Two such examples are illustrated below with a vertical Web run past the Bottom Chord and with a raised Bottom Chord without any Webs on the opposite face. Both of these joints subject to these provisions (see Figure C7.5-1).

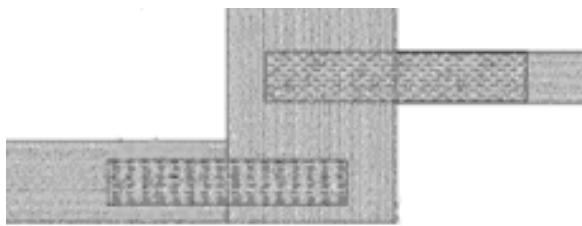


Figure C7.5-1

The change made in the *NDS 2005* only affects joints located less than five times the depth of the member from its end. In this case the design shear force, V' , is limited to:

$$V' = (2F_v'A_e/3) \times (d_e/d)^2$$

The *NDS* still gives the original formula (above formula except without the $(d_e/d)^2$ term) for other locations. For this reason, the existing sections are separated to address both situations with the given formulas. The formulas are adjusted as shown, which for the new *NDS* equation results in the following:

$$NDS \text{ Equation: } V = (2F_v'A_e/3) \times (d_e/d)^2$$

Solve for F_v' rather than V : $F_v' = [(3V/(2A_e)] \times (d/d_e)^2$

The formula above is for the maximum allowable shear stress. The $(1 - y^2/c^2)$ term is included to account for the connection depth on a member:

$$F_v' = [3V/(2A_e)] \times (d/d_e)^2 \times (1 - y^2/c^2)$$

where:

y = distance from centerline

c = half of member depth

Since $F_v' \geq f_v$ for design and solving for y gives:

$$y \geq c [1 - F_v' 2A_e / (3V) \times (d_e/d)^2]^{0.5}$$

which matches the existing equation with the addition of the $(d/d_e)^2$ term to reflect the change in the *NDS*.

§7.5.3.2.2 Alternate Extent of Connection.

The provision relating to the details in Figures 7.5-2(a)-(c) was a provision added to the 2002 Standard, which recognizes the contribution of Metal Connector Plates in resisting cross-grain tension forces when located in the area of the hanger connection inducing the cross-grain forces. Since the amount of bite onto the Chord member is the critical factor, the addition of Metal Connector Plates, provided the Metal Connector Plates overlap with the hanger connection, can be considered as adding to the amount of bite provided by the overall connection. Thus, if a minimum distance past the member centerline is determined for a tension perpendicular to grain force exceeding 800 lbs., either the hanger connection must extend to that distance, or an overlapping Metal Connector Plate in accordance with any of the details in Figures 7.5.2(a)-(c) must extend to that distance.

§7.5.3.2.3 Effective Depth Greater Than 85% of Member Depth.

85 percent of the member depth is a practical limit on how close the center of the topmost fastener can be located to the edge of a member, in which case it is considered as effectively developing the full depth of the member. Any hanger that is tall enough, or placed high enough on the carrying member, to reach 85 percent of the member depth will not be subject to the other subsections of Section 7.5.3.2.

§7.5.3.5 Torsion.

Structural members framing into the side of girders could create torsion on the entire girder. This would create a tendency for the girder to rotate without proper restraint. The proper restraint to resist Truss torsion would be in

the form of permanent lateral, diagonal or cross bracing.

§7.5.5.1 Connection of Members.

Three-ply Trusses may be connected by nailing since the exterior plies can be attached directly to the center ply. Four or more plies require bolts or other approved fasteners other than nails to develop continuity across the plies; in no case should a girder with four or more plies be connected only with nails. The requirement to have only one type of fastener (i.e., either nails, bolts, or other approved fastener) transmit 100 percent of the force between plies is in accordance with the *NDS*, which states that design

values for joints made with more than one type of fastener have not been developed and should be based on tests or other analysis.¹⁷

§7.5.5.2 Design Load.

Since the first ply of three or more plies generally carries a smaller percentage of the load than the sum of the remaining plies, the connections must be adequate to transfer the sum of the loads on the remaining plies. For example, if the designer assumes each ply of a three-ply Truss carries the same proportion of load (i.e., 33 percent per ply), the connections between the first and second ply

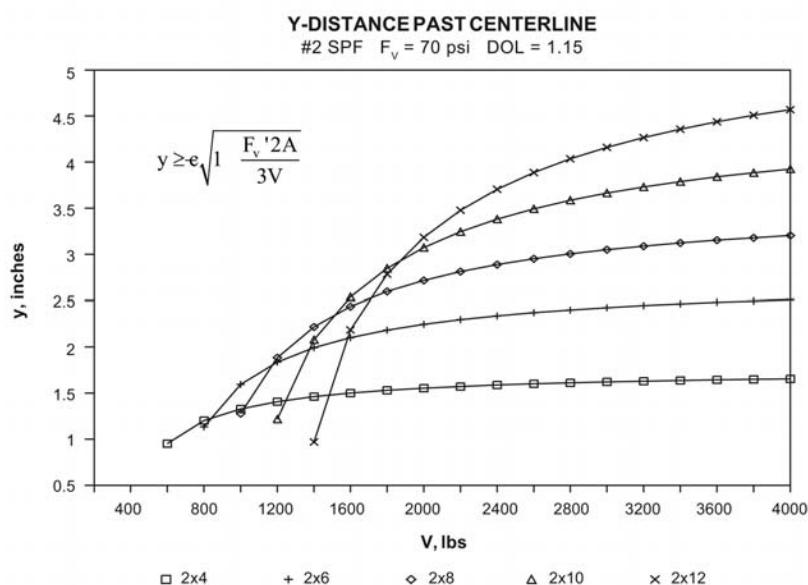


Figure C7.5-2(a). Graphical Representation of Equation E C7.5-2 for Determining the Distance (y) to Extend a Metal Connector Plate Past the Centerline of a Wood Member to Resist Tension Perpendicular-to-Grain Stresses.

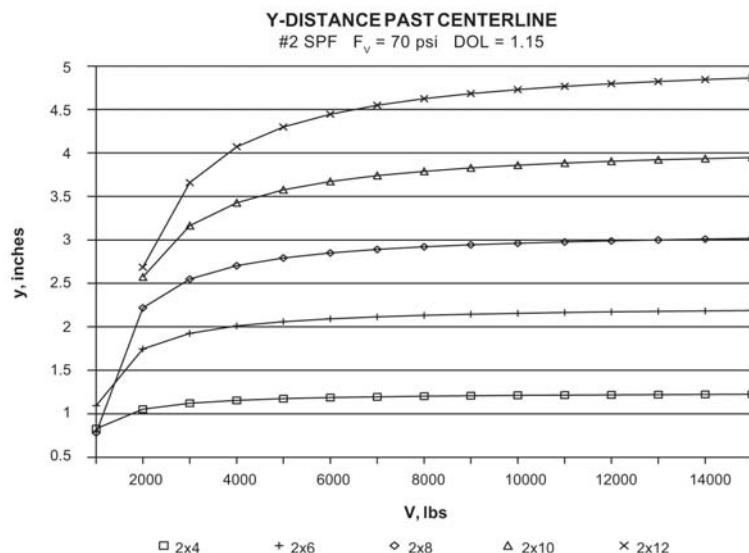


Figure C7.5-2(b). Graphical Representation of Equation E C7.5-2 Showing Asymptotic Relationship of Curves to a Line Representing one-half of the Wood Member Depth, c.

17 D. Grant, F. J. Keenan and J. E. Korhonen, *Effective length of compression web members in light wood trusses*, Forest Products Journal, Vol. 36, No. 5, 57-60, 1986.

would have to be designed to transfer the sum of the second and third ply loads, or 66 percent of the load. The design of the ply-to-ply connections will depend on the load distribution assumed in the design of the girder Truss (see Sections 7.5.2.3 and §7.5.2.3).

§7.5.5.3 Design for Withdrawal Load.

Compression members in Trusses may buckle laterally unless restrained. Sheathing, properly attached to all plies of compression Chord members, will provide this Lateral Restraint. At least one ply of the multi-ply member must be braced against lateral movement by proper attachment to the sheathing or other specified bracing. The remaining plies are attached to this braced ply with a nailed connection, where the nails are loaded in withdrawal. Designing for two percent of the axial compression force in each ply, including the cumulative effects, is in accordance with long recognized lateral bracing force assumptions.¹⁸

Section 7.5.5.3 established withdrawal requirements for ply-to-ply connections for each unbraced length of the member. However, this section has raised problems for some engineers who presume an unbraced length of zero for members that are attached to sheathing. The presumption of zero unbraced length for such members is reasonable; however, this presumption may either result in the engineer concluding there are no unbraced lengths, and thus no withdrawal requirements on fasteners, or the engineer may use Section 7.5.5.3 to establish a withdrawal capacity for every 0 in., which is an infinite number. For example, a 2x4 carrying 10,000 lbs. of compression would be required to have ply-to-ply connections sufficient to carry 200 lbs. (Section 7.5.5.3 states two percent of compression force) every 0 in. To resolve this, the 2007 edition of the Standard was revised to provide guidance to users that the unbraced length of members braced by sheathing in accordance with typical practice shall be permitted to be presumed to have an unbraced length of ten times their thickness. For example, this would give an unbraced length of 15 in. for a 2x₄ member. At this unbraced length or less, C_p is typically very close to 1 and thus the variation caused by this assumption is very small.

§7.5.5 Bolt Spacing.

New to the 2007 edition of the Standard, bolt spacing in ply-to-ply connections is now permitted to exceed 24 in. if other fasteners are designed to carry the ply-to-ply transfer loads.

18 W. Zuk, *Lateral bracing forces on beams and column*, Journal of the Engineering Mechanics Division, Proceedings of ASCE, Paper 1032, Vol. 82, No. EM-3, July 1956.

§7.6.1 Method of Calculation.

Creep is an important design consideration where dead loads or sustained live loads represent a relatively high percentage of the total design load. The requirement to account for creep in total deflection calculations was added to the 2002 Standard to address Trusses with these types of characteristics, such as floor Trusses with lightweight concrete toppings, which have appreciable long-term deflection that may produce deflection problems. Consideration for creep can result in stiffer floor Trusses under these conditions, whereas typical residential floor Trusses will generally not be governed by this provision and will instead be governed by the more stringent live load deflection criteria.

Per the *NDS*, total deflection is calculated as follows; see the *NDS Commentary* for additional general discussion on creep:^{19, 20}

$$\begin{aligned}\Delta_T &= K_{cr} \Delta_{LT} + \Delta_{ST} \\ K_{cr} &= \text{creep factor} \\ &= 1.5 \text{ for seasoned lumber or Structural Composite Lumber used in dry service conditions;} \\ &= 2.0 \text{ for unseasoned lumber or for seasoned lumber used in wet service conditions} \\ \Delta_{LT} &= \text{immediate deflection due to the long-term component of the design load (deflection due to dead load)} \\ \Delta_{ST} &= \text{deflection due to the short-term or normal component deflection (deflection due to live load)}\end{aligned}$$

According to the *NDS*, use of twice the initial deflection as a basis for accounting for deformation associated with long-term loads has long been recognized, and it wasn't until the 1977 edition of the *NDS* that the provision was revised to reflect creep factors of 1.5 for seasoned and 2.0 for unseasoned lumber.²¹ Factors that will cause an increase in creep include increased stress level, moisture content and temperature, as well as variable relative humidity conditions.

Testing on Trusses has been reported that suggests Trusses exhibit more creep than simple beams of solid sawn lumber. In a research study that measured deflection of four pairs of Trusses and one pair of joists over a ten-

19 Grant, Keenan, and Korhonen, 57-60.

20 American Forest & Paper Association, National design specification for wood construction.

21 American Forest & Paper Association. National design specification for wood construction.

year period, the joists showed better creep performance, which was attributed to two factors.²² First, the joist span was limited by deflection, and thus the bending stress in the joists was only at 73 percent of the allowable value, while the Trusses were stressed to the full allowable stress value in the Bottom Chords and 90 percent of allowable in the Top Chords. The second factor stated as causing the difference in creep performance was the creep due to the connections, namely the non-recoverable slip in the plated Truss connections. Other research has been noted that found long-term relative creep values for Trusses at 2.5 or higher.²³

In considering the magnitude of the creep factor, which will vary by application, it is also helpful to recognize that full dead load is not normally on the Truss permanently, which could be used to justify use of lower K values. As an example, a creep factor of 2.0 may be a selected as an appropriate creep factor for Trusses in general (for seasoned lumber and dry service conditions), and then reduced in cases where the dead load is conservative or similar. Thus, with this example, if only 75 percent of the design dead load is on the Truss, the creep factor would drop from 2.0 to 1.5.

For floor Truss applications with a total dead load of 25 psf or greater where the Registered Design Professional for the Building or the Building Designer does not specify adjustment factors for serviceability issues (per Section 2.3.2.4(h)(5) or Section 2.4.2.4(h)(5), respectively), a creep factor, K_{cr} , of 2.0 is recommended as a minimum adjustment in lieu of the 1.5 factor for seasoned lumber used in dry service conditions

New to the 2007 edition of the Standard, the total deflection calculation now explicitly specifies a component due to creep of no less than 50 or 100 percent of the initial deflection (meaning $K_{cr} = 1.5$ or 2.0) for long-term loads for dry and green (or wet service) use, respectively. Prior to the 2007 edition, this information was only presented in the Commentary.

§7.6.2.1 Designated Limits.

A common accepted practice for determining deflection at any point, conducive to use with a structural analysis program, is the following vir-

tual work method:²⁴

$$\Delta = \frac{P_u \times \ell}{AE}$$

where:

P = axial force load in a Truss member caused by design loads (lbs.)

u = force in a Truss member caused by a unit (virtual) load (dimensionless)

ℓ = length of the Truss member (in.)

A = cross-sectional area of the Truss member (in.²)

E = modulus of elasticity (psi)

This equation considers the effects of axial force in each member, as well as end slip, if the member elongation, $P\ell/AE$, is modified by adding or subtracting an amount, S, to cover slip of the end connections. Although this method assumes a pin-jointed Truss loaded only at the joints, it has proved to be sufficiently accurate for other Trusses because bending of the individual Truss members contributes little to overall Truss deflection.

The deflection limits in Table 7.6-1 are consistent with typical minimum Building Code requirements where specified. Where deflection requirements in any local Jurisdiction differ from those in Table 7.6-1, the local Jurisdiction regulations will apply. In addition, certain floor coverings require more restrictive deflection criteria than the typical limit on floor Trusses of L/360 in order to prevent cracking of the flooring materials. While Table 7.6-1 includes one common floor covering requiring more restrictive deflection criteria, namely ceramic tile, the Truss Designer should be aware of possible other deflection criteria associated with specific floor coverings (e.g., L/720 total load deflection for marble tile). Maximum on-center spacing of floor joists supporting certain floor materials may also be required by some material standards, such as the 16-in. on-center Truss spacing limit indicated in Footnote 4 for floor Trusses supporting ceramic tile. Although the 16-in. requirement is specified to provide enough stiffness in the sub-floor system for the tile, and is not a requirement for Truss deflection, it is important for the Truss Designer to be aware of this type of spacing requirement unrelated to Truss performance.

22 D. H. Percival and S. K. Suddarth, *Long-term tests of 4x2 parallel-chord metal plate connected wood trusses*, Addendum to research report 81-1, Research Report 89-2, Small Homes Council-Building Research Council, Urbana-Champaign, Illinois, 1989.

23 M. Leivo, *On the stiffness changes in nail plate trusses*, Department of Civil Engineering, Tampere University of Technology, Research Centre of Finland, Publications 80, Espoo, Finland, 1991.

24 American Forest & Paper Association, National design specification for wood construction.

The 16-in. on-center spacing limit that was explicitly referenced in the 2002 edition of the Standard per the referenced standard in the footnotes was revised in the 2007 edition to just reference *ANSI A108/A118/A136* for Truss spacing as there are a number of available systems that allow 19.2- and 24-in. on-center spacing of supporting joists. The Truss Designer should ascertain what deflection and on-center spacing requirements apply to any particular floor covering at the time of design.

The 2007 edition of the Standard made the following revisions to Table 7.6-1:

- Footnote 2: Floor Truss deflection limit for total load added ($L/240$ to match *IBC*) and prior total load deflection limit of $L/360$ specified for floor Trusses supporting ceramic tile deleted and replaced with footnote referencing other criteria per *ANSI A108/A118/A136* and Building Designer requirements per Chapter 2.
- Footnote 4: Deflection limits for Cantilevered portions of Trusses is also noted as applicable to Overhang portions of Trusses.
- Footnote 6: Top Chord panel deflection limit of $L/600$ noted for beam or lintel supporting vertical masonry veneer per *ACI530/ASCE5/TMS402*. Trusses are sometime placed in service where brick veneer must be supported and Section 1.10.1 in *ACI530/ASCE5/TMS402* provides deflection criteria for the lesser of $L/600$ or 0.3 in. as shown below (from page C-10 of *ACI 530-99/ASCE 5-99/TMS 402-99*):

1.10 – Deflection

1.10.1 Deflection of beams and lintels

Deflection of beams and lintels due to dead plus live loads shall not exceed $L/600$ or 0.3 in. (7.6 mm) when providing vertical support to masonry designed in accordance with Section 2.2 or Chapter 5.

- Footnote 7: Live load deflection limit of $L/360$ noted as applicable for panel deflection of habitable spaces in Trusses. This was added to the 2007 edition of the Standard because it should cover more than just attic “Trusses” since other Trusses can wind up with habitable spaces in them as defined by the governing Building Code. Total load deflection is omitted because it is not specified for floor Trusses and is not a requirement in the *IRC* (see Table 301.7). Local Building Codes can define habitable space differently so there is a need to make sure this is applied only when the live load reaches a living

space load. This approach allows two exceptions to this limit: 1) if the space cannot be a habitable space but has a large live load for storage or something else; or 2) if the space could become habitable but the local Building Code does not require it to be designed for that in advance.

More restrictive deflection requirements for increased stiffness and improved serviceability, particularly of floor Trusses, may be desired and may be specified as such by the Building Designer for any particular Building. If vibration in a floor is a concern, and since the probability of vibration problems increases as floor spans increase, using a more restrictive deflection limit, $L/480$ or even $L/600$, will help in preventing floor vibration. It may also be desirable to limit the Truss deflection to a finite amount (e.g., maximum number of in.) depending on Truss span or load or on Building usage. For example, the maximum acceptable amount of deflection for a particular roof Truss application may be 2 in., whereas a 60-ft. Truss subject to $L/240$ would otherwise allow a 3-in. maximum deflection. In addition to the overall deflection criteria, the maximum horizontally projected Bottom Chord panel lengths in Table C7.6-1 are recommended as a minimum standard.

Chord Size	Bottom Chord Uniformly Distributed Load, w (plf)		
	$0 < w < 6$	$6 < w < 14$	$w > 14$
2x4	15'-0"	13'-6"	12'-0"
2x6	18'-0"	16'-6"	15'-0"
2x8 & larger	20'-0"	19'-0"	18'-0"

Table C7.6-1 Maximum Recommended Bottom Chord Lengths

When parallel chord Trusses are used in a roof system, consideration must be given to the potential for roof ponding (i.e., overloading from the retention of water) due to the deflection of the Trusses following the application of roofing and ceiling materials. Per Footnote 1 of Table 7.6-1, ponding must be investigated for roofs not having sufficient slope or camber to assure adequate drainage, in order to ensure that ponding instability is precluded. A widely recognized minimum slope for adequate roof drainage is $\frac{1}{4}$ in. per foot. Sufficient camber to counter the deflection due to dead load may also be specified to prevent ponding, such as an amount equal to a deflection corresponding to 1.5 times dead load.

In the footnotes for Table 7.6-1, it is noted that for ceramic tile, Truss spacing and appropriate dead load for the installation method, and other aspects of design per *ANSI A108/A118/A136* shall be such that the system passes the requirements of the Building Designer per Chapter 2 of

the Standard. It is also noted that floor Trusses with ceilings attached that meet L/480 criteria shall not require strongbacks (meaning per *TPI 1*, although they may still be required for other purposes if specified by others, such as for a specific fire rated assembly or as otherwise specified in the contract documents).

§7.6.2.2 Deflection Using Beam Formulas.

As indicated in this section, vertical deflection shall be permitted to be determined using beam formulas as shown in Section 7.6.2.3. Another useful resource is *The Load Guide* produced by WTCA. Information on *The Load Guide* can be found at: www.sbcindustry.com/loads.php.

§7.6.2.3 Deflection Calculation for Parallel Chord Trusses.

The beam formula in Equation E7.6-1 is an approximate method for estimating deflection of parallel chord Trusses, which was first introduced in the 1980 Standard for the design of parallel chord wood Trusses. The equation incorporates a 1.33 factor for joint slippage, shear deformation and creep, and deflection estimates using this formula were shown to perform well when compared against both experimental results and a more thorough analysis.²⁵ Use of this method is still permitted in the 2007 edition; however, less emphasis is given to the approximate beam formula because it is recognized that the use of typical structural analysis using matrix methods will be more accurate.

§7.6.2.4 Strongbacking.

Strongbacking is recognized for serving two purposes: reducing floor vibrations and limiting differential deflection. Strongbacking does not, however, contribute to or enhance the strength or structural integrity of the system. Strongbacks are typically used to control potential vibration problems, as the addition of strongbacks has proven to stiffen the Trusses and increase the dampening of transient oscillations.^{26, 27} Vibration in a floor joist due to normal human activity (e.g., walking) includes vibration movements from side to side, and while floor sheathing prevents lateral vibration of the Top Chord, the bottom can still vibrate back and forth. Thus, placing a strongback at the bottom of the floor Truss helps control the side-to-side movement at the bottom and improves the

overall perceptible performance of the floor. Even when there is a ceiling on the bottom of the Trusses, in which case the drywall will reduce lateral movement, the addition of strongbacks can still help to further restrict vibration. It should be recognized that, while it will not affect the structural integrity of the system, cutting, removing or failing to provide such strongback bracing can result in degradation of the floor system's ability to dampen vibration. Another purpose of strongbacking is to limit differential deflection between adjacent floor Trusses by developing supplemental two-way action in the floor framing in addition to the floor sheathing. Floor Trusses with design live load deflections less than 0.67 in. are unlikely to have differential deflections large enough to develop two-way action from strongbacking.

New to the 2007 edition of the Standard, nails for attachment of strongbacks to Trusses are now specified as 10d (0.131 in. x 3 in.) to reflect common and accepted construction practices.

25 S. K. Suddarth, D. H. Percival and Q. B. Comus, *Testing and analysis of 4x2 parallel chord metal plate connected wood trusses*, Research Report 81-1, Small Homes Council-Building Research Council, Urbana-Champaign, Illinois, 1981.

26 J. W. Stark, *The effect of lateral bracing on the dynamic response of wood floor systems*, Department of Wood Science and Forest Products, Virginia Polytechnic Institute and State University, May, 1993.

27 X. Li, *The effect of stiffness and mass on the dynamic response of wood floors*. Department of Wood Science and Forest Products, Virginia Polytechnic Institute and State University, May, 1993.

§ CHAPTER 8

METAL CONNECTOR PLATE JOINT DESIGN

Overview of Changes (non-editorial)

New to TPI 1-2007

- 90) The 0.8 factor on the allowable lateral resistance stress was removed to match the change in the C_q factor basis from a maximum of 1.25 to a maximum of 1.00. (Equation E8.3-2)
- 91) Maximum reduction of 50 percent for plating forces perpendicular to the joint to account for force transferred in direct wood-to-wood bearing (butting) removed (C_R value calculation based on P_{aN} removed but the C_R parameter is just limited to vary between 0 and 1 with no further guidance given), and corrections made to clarify that force parallel to the joint must be included in the plating force and the allowable lateral resistance stress is adjusted by applicable factors. It is also required that the vector sum of these adjusted components be computed and that the angle of this vector be used to determine the allowable tooth holding value. The previous edition of the Standard used the member angle instead of this resultant angle. (Section 8.3.3.3)
- 92) Effective portion for steel tension design of plates extending off the Chord was clarified to be dependent upon the plate lap onto each Chord member parallel to grain, was adjusted to not be reduced by a 0.5 unit amount previously specified, and is no longer limited to plates positioned that cover nearly the entire joint line. (Figure 8.4-1)
- 93) A provision was added requiring moment design for joints modeled as pinned if the plate cross-section is eccentric to the Chord centerline. (Section 8.4.3.3)
- 94) The moment design equation of the plate steel cross-section were updated per TPI Technical Advisory Committee (TAC) research recognizing the effects of plate position, compression force on the joint and for mid-panel splices the P-delta effects. (Equation E8.7-1)
- 95) The net section limit of 1600 lbs./in. (2400 lbs. for a 4x2) was removed from mandatory language and replaced by non-mandatory note recommending a limit of 2300 lbs./in. (3450 lbs. for a 4x2). This limit was also modified to not apply at locations other than where the Wood Member ends under or within 1 ft. of the edge of the Truss plate. (Section 8.8.2)
- 96) The modifications to the tension perpendicular to grain limits mentioned in Chapter 7 for girder Truss plated connections also apply to all Trusses. (Section 8.10)

97) The plate placement limit need be no less than $\frac{1}{8}$ in. for lateral resistance limits. (Section 8.11.4)

98) Plate rotation to be considered for design now permitted to be less than 10 degrees if so specified in the Truss Manufacturer's quality control procedures. The plate placement polygon is now to be determined with consideration for this rotation as well, and limitations for plate rotation that causes the plate to extend outside the Truss profile have been eliminated. (Section 8.11.5)

§8.1 SCOPE

Chapter 8 contains procedures that are necessary for designing a Metal Connector Plate to adequately resist all applicable design forces and moments acting at a joint, including a determination of the minimum required Metal Connector Plate contact area and the minimum net sections of the plate based on the plate lateral resistance, tensile and shear strengths, and the allowable axial tension and compressive stress of the wood over a reduced net section at the joint. All of the joint design procedures presented in Chapter 8 were developed for Trusses with Web members that are cut to bear on a surface. The Standard does not include explicit joint design criteria for plating round-ended or square-ended Webs, which have only single points of contact between adjacent members, and thus require some additional joint design considerations relating to lateral resistance, member-to-member compression force transfer, and plate shear buckling. Until the Standard adopts specific joint design provisions to address these design issues, the Truss Designer should account for the effect of gaps on plate strength when designing joints for round-ended or square-ended Webs based on engineering judgment or testing (see Sections §8.3.2, §8.3.3.3, and §8.5.6 for additional discussion).

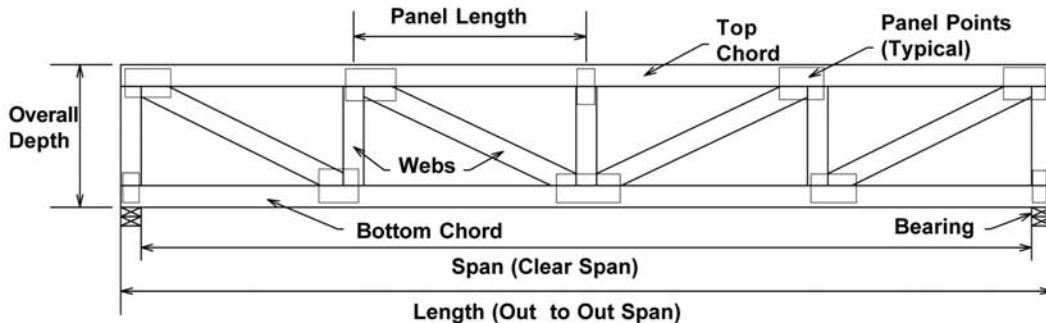
§8.2 MINIMUM AXIAL DESIGN FORCES

The requirement for Metal Connector Plate joints to be designed for a minimum of 375 lbs. axial force is to provide adequate resistance to the forces involved in the handling and erection of metal-plate-connected wood Trusses. With this minimum joint design load, all Metal Connector Plates in Trusses with spans exceeding 16 ft. will have sufficient Teeth, nails or plug groups to develop a minimum of 375 lbs. axial force in all members at the lateral resistance value. The 375-lb. requirement has been specified in the Standard since the 1968 edition, when it was required to design all joints to this minimum capac-

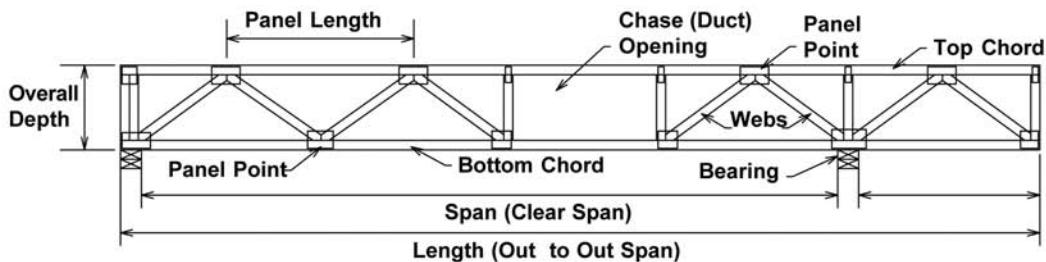
ity "as a rough handling precaution." Because the 375-lb. load is overly conservative for small light Trusses typical of very short spans, this provision is explicitly limited to Trusses over 16 ft. in length since the 2002 edition.

As stated above, Section 8.2 specifies a minimum axial design force of 375 lbs. (1670 N) for Truss with lengths exceeding 16 ft. The length of a Truss in this case is the overall length and is not the same as the span of the Truss.

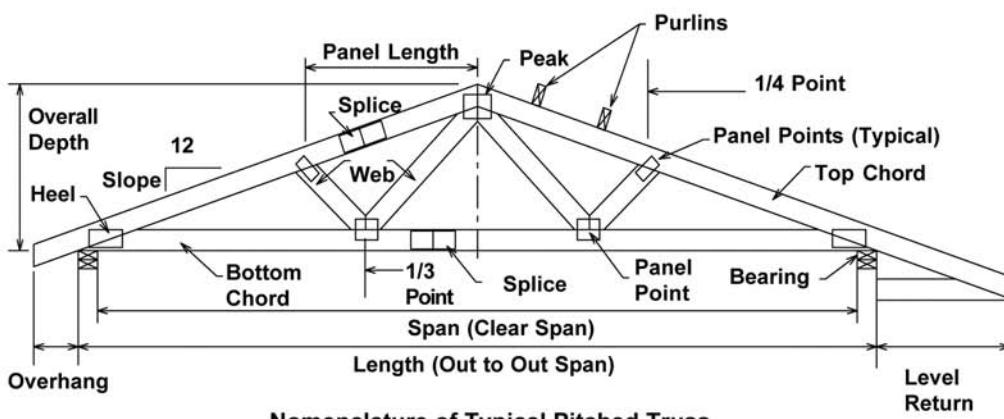
The "span" of a Truss or the "clear span" is defined as the centre-to-centre distance between the bearing supports of the Truss. The "length" of a Truss or the "Out to out span" on the other hand is the end-to-end Distance between the extreme members of the Truss. This could be the outer end of the Overhang of the Truss or the outer end of the bearing of the Truss depending on the situation. Figure C8.2-1 illustrates the definition of "length" of the Truss as used in Section 8.2.



Nomenclature of Typical Parallel Chord Truss With Lumber Oriented Vertically.



Nomenclature of Typical Parallel Chord Truss With Lumber Oriented Horizontally.



Nomenclature of Typical Pitched Truss

Figure C8.2-1. Illustration of Truss Nomenclature – Note Span and Length.

§8.3 LATERAL RESISTANCE

The lateral resistance of Metal Connector Plate Teeth is one measure of its ability to fasten Wood Members together where the axial forces must be transferred through the Metal Connector Plate from one Wood Member to another. The width of a plate at a joint must be adequate to resist the axial and shear forces at the joint, while the plate-to-wood contact area must be adequate to resist the withdrawal of the plate from the connecting members. Many factors have long been recognized as affecting the lateral resistance capacity of a Metal Connector Plate, which include not only properties of the plate itself (e.g., steel strength, tooth characteristics, etc.), but also wood-related properties, such as specific gravity, moisture content, and grain angle of the Wood Member, as well as the orientation of both the tooth slot direction and lumber grain relative to the applied load, and method of plate installation.

Lateral resistance design values for each type of plate are tested and reported for different species for each of four mandatory plate/wood grain orientations, AA, EA, AE, and EE, as specified in Section 5.2.7.1.1. For the design of other plate/wood grain orientations than those above, Section 8.3.3.2 addresses how to account for loads that are applied at an angle other than 0 or 90 degrees to the wood grain, or a plate axis that is oriented at an angle other than 0 or 90 degrees to the load direction.

§8.3.2 Adjustments.

Adjustment factors in Section 6.4 that are applicable to lateral resistance design values and might be applied to the allowable lateral resistance value include: Load duration factor (C_D) per Section 6.4.1, wet service factor (C_M) per Section 6.4.6, quality control factor (C_q) per Section 6.4.10, and any reductions for plates installed in chemically treated lumber per Section 6.4.9.

§8.3.2.1 Exclusion of Area under the Net Area Method.

In the establishment of lateral resistance design values, different methods can be employed to address the effects of End and Edge Distances on tooth strength relative to the tooth's resistance to withdrawal from solid wood outside of these end and edge zones (see Section §5.2.6.2). Because the Net Area Method removes Teeth from the tested area that may be affected by end and edge influences, the Net Area Method results in higher design values on a unit area basis than the Gross Area Method, in which Teeth must be present in End and Edge Distance zones. Thus, these higher design values cannot be applied for design purposes to any Teeth within the specified Net Area End and Edge Distances on a joint. The same ap-

plies when End or Edge Distances other than those specified for the Net Area Method, 0.5 in. and 0.25 in. respectively, are used in testing.

§8.3.2.2 Additional Consideration at Heel Joints.

Prior to 1978, the Standard incorporated specific reductions on lateral resistance values of heel plates depending on pitch, based on experience gained on thousands of applications of Truss designs. These reductions on allowable grip ranged from 85 percent of the allowable design value for lower pitches (under 3:12 slope) to 65 percent of the allowable design value for steeper pitches (over 5.5:12). The reductions accounted for eccentricity at the heel joint where the top and Bottom Chords were in contact but could not transfer force through wood-to-wood butting. These factors were eventually incorporated into an empirical formula, which gives reductions that closely approximate the original reductions specified for different pitches, as shown in Figure C8.3-1. The method of heel analysis using a reduction factor as specified in the Standard was shown to have a much closer correlation to test results than an alternate method using the torsion formula for those heel plate sizes studied.¹

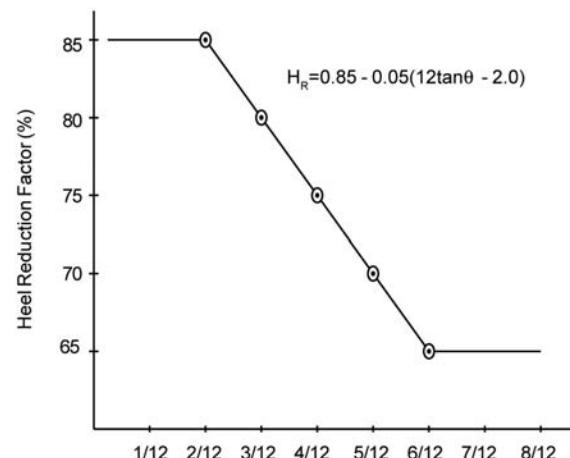


Figure C8.3-1. Heel Reduction Factor, H_R , Versus Pitch.

These factors were not developed to apply to heels where the top and Bottom Chords are not actually in contact, as in the case of raised heel or "energy" Trusses as shown in Figure C8.3-2.

Adjustments for Round or Square-Ended Members:

The effect of member-to-member gaps created by round-ended or square-ended Webs that have single points of

1 I. Sheppard and M.L. Esmay, *Contact area stresses and buckling resistance of light gauge metal heel plates for wood trusses*, Presented at the 1970 Annual Meeting ASAE, Paper 70-410, 1970.

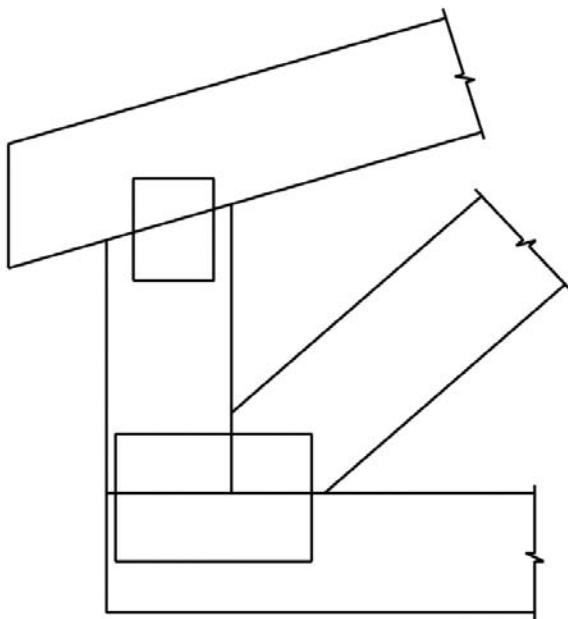


Figure C8.3-2. The Heel Reduction Factor, H_R , is Not Intended for Raised Heel Conditions Such as This.

contact with adjacent members should be accounted for in the design of plates for lateral resistance by excluding the area of plate located in these gaps from the Metal Connector Plate coverage area. Thus, round-ended and square-ended Web members may require larger plates than standard Webs (Web members that are cut to bear on a surface) to result in the same amount of effective Teeth for lateral resistance.

§8.3.3 Design.

In the 2002 edition, a 0.8 factor on the lateral resistance design value, V_{LR}' , was incorporated into Equation E8.3-2. This reduction was intended to account for locations in the plate contact area that have naturally occurring lumber characteristics, such as knots, wane, etc., which reduce the effectiveness of the plate Teeth (lateral resistance strength), as well as plate Teeth that are flattened (as described in Section 3.7.4) during the manufacturing process. The incorporation of the 0.8 factor into the 2002 edition of the Standard was due to a research program that examined, through full-scale Truss testing, the relationship between quality imperfections and structural performance.²

The 2007 edition of the Standard no longer has the 0.8 factor. The 0.8 factor on the allowable lateral resistance

stress was removed to match the change in the quality control factor (C_q) basis from a maximum of 1.25 to a maximum of 1.00. For more discussion on the quality control factor per Section 6.4.10, and the Quality Standard set forth in Chapter 3, see the corresponding Commentary sections.

§8.3.3.1 Number of Teeth Reported for Inspection.

The effective tooth count procedure is outlined within the quality criteria in Chapter 3. One of the criteria for the acceptance of a joint is described in Section 3.7.7.1: *The combined number of effective Teeth for both faces of the Truss at each joint in each Metal Connector Plate contact area shall meet or exceed two times the minimum number specified for a single face by the Truss Designer...* The calculation to express the required number of Teeth in each joint member contact area, in order to provide the Truss Manufacturer with a means to assess the quality of a joint in terms of its effective Teeth, has been specified since 1995 and remains in the 2007 edition.

§8.3.3.2 Load Applied at an Angle.

The Hankinson formula is the basis of Section 8.3.3.2 for deriving allowable lateral resistance values for Metal Connector Plates loaded at an angle, θ , to the grain. This is accomplished by an interpolation between the allowable values where the Teeth are either parallel or perpendicular to the load. Use of this formula in this application was validated by the work of R.O. Foschi.³ Additional commentary regarding the use of the Hankinson formula is found in Appendix J of *ANSI/AF&PA NDS*.⁴

§8.3.3.3 Metal Connector Plates Resisting Member Compressive Forces.

Prior to the 1995 edition, the Standard had a long history of permitting up to 50 percent of the compressive forces at a joint to be transferred through wood-to-wood contact in compression. This ensured that the plate would be sized for at least 50 percent of the compressive force, thus limiting the minimum size of a plate that could be specified at joints resisting compressive forces. Sizing the plate for more than 50 percent of the compressive force was left up to engineering judgment.

This long-standing provision was replaced in the 1995 edition with a procedure that utilizes a compression reduction factor (C_R). The compression reduction factor

3 R. O. Foschi, *Analysis of wood diaphragms and trusses, Part II: Truss plate connections*, Canadian Journal of Civil Engineering, Vol. 4, 1977.

4 American Forest & Paper Association, *National design specification for wood construction*, ANSI/AF&PA NDS-2005, Washington, DC, 2005.

2 S. M. Cramer, *Quality control impacts on metal plate connected truss performance*, Report to Truss Plate Institute, 2001.

was developed to address joints that have unequal compressive forces on either side of the joint, which produce unbalanced horizontal components, such as a panel point compression splice or Top Chord peak joint. In these cases, the amount of compression force that can be transferred from one member to the other through wood-to-wood compression depends on the magnitude of resistance of the adjacent opposing compression member. Thus, the compression reduction factor determines the magnitude of compression force to be plated for in cases with unequal compressive components, where an otherwise assumed transfer of 50 percent of the axial forces through wood-to-wood contact may be too high. The method still requires plates to be sized for a minimum of 50 percent of the compressive force by having a lower limit of 0.5 on the compression reduction factor. It should be noted that the assumptions made with regards to the stiffness of the wood-plate connection will impact how the forces are distributed. Thus, the approach presented in the Standard is considered to be one rational approach to allocate this wood-to-wood load transfer, while it is also recognized that there may be other acceptable approaches.

The maximum reduction of 50 percent for plating forces perpendicular to the joint to account for force transferred in direct wood-to-wood bearing (butting) was removed in the 2007 edition of the Standard (the C_R value calculation based on P_{aN} was removed but the C_R parameter is just limited to vary between 0 and 1 with no further guidance given). Clarifications were also made in this edition to recognize that the force parallel to the joint must be included in the plating force and the allowable lateral resistance stress is adjusted by all applicable factors. It is also required that the vector sum of these adjusted components be computed and that the angle of this vector be used to determine the allowable tooth holding value. The previous edition of the Standard used the member angle instead of this resultant angle.

The application of this method requires that a Wood Member is present to resist the remainder of the component forces so that the vectorial sum of all forces in the joint equals zero. Accordingly, this method would not apply to the splice shown in Figure C8.3-3 (see also the discussion under "Compression Joint Design for Round or Square-ended Members").

In the case of a compression web framing into a Chord member, the Chord member is capable of providing adequate resistance to the compressive force via wood-to-wood contact. While the transfer of shear forces is also a consideration, this is a separate design check. In the case of a compression web framing into another Web, as shown in Figure C8.3-4, this procedure is adequate pro-

vided proper consideration is made for restraint of the Web being framed into (i.e., there must be sufficient shear capacity along the Web-Chord interface and enough lateral resistance to develop this capacity).

In the case of a compression Web and a tension Web framing into a Chord as shown in Figure C8.3-5, this procedure may not apply in the same manner as previously described. Since the tension member will act in the same direction as the compression member, there will tend to be little resistance to the compressive force other than

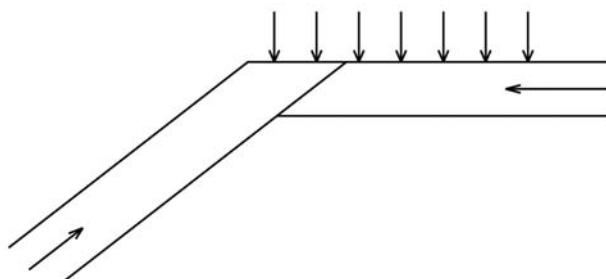


Figure C8.3-3. Example of a Splice for Which the Compression Criteria in Section 8.3.3.3 is Not Applicable.

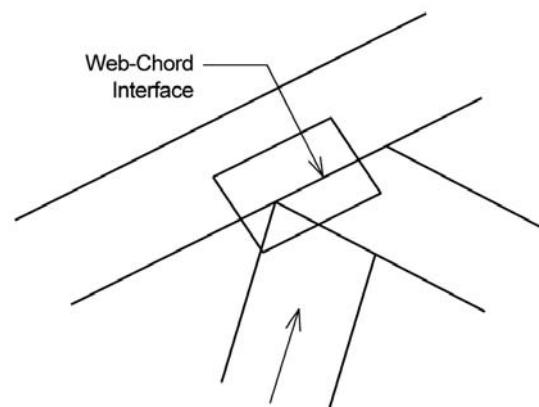


Figure C8.3-4. Compression Web Framing into Another Web.

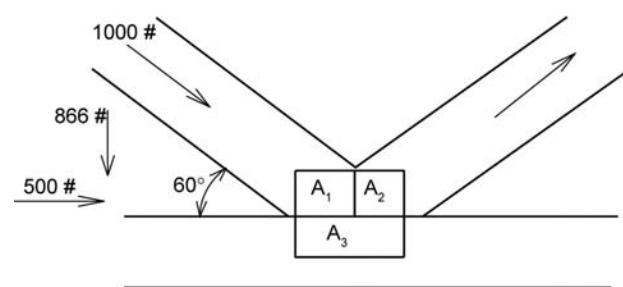


Figure C8.3-5. Compression Web and Tension Web Framing into a Chord.

that offered by the Chord member. This will vary depending on the particular application, and engineering judgment should be applied with this type of condition. In lieu of alternate methods determined by an engineer through testing or judgment, assuming that there is no force transfer at the compression and tension Web interface should provide a reasonable approach.

Compression Joint Design for Round or Square-Ended Members:

In the absence of empirical data, designing Metal Connector Plates for the full 100 percent compression component is recommended for round-ended or square-ended Web members, due to the single point of contact between these Web members and the Chord, through which the compression force must otherwise be transferred over a reduced area. Even though there is wood-to-wood bearing at the point of contacts, this approach conservatively assumes that there is no Wood Member present, or that the wood-to-wood interface is not stiff enough, to resist transfer load through wood-to-wood bearing, which requires designing the Metal Connector Plates for the full compression force per Section 8.3.3.3.1.

Effect of Gaps:

Another consideration in compression joint design is the existence of gaps between the compression members. Although it would be ideal to have full wood-to-wood contact at compression joints, this is not always achievable during manufacturing. In early versions of the Standard's quality criteria for the manufacture of metal-plate-connected wood Trusses, the requirement for compression splice joints was "good bearing."⁵ This requirement was eventually replaced by quantified maximum allowable Wood Member-to-Wood Member gaps. The current quality criteria in Chapter 3 specifies a maximum allowable gap between members of $\frac{1}{8}$ in. except for floor Truss Chord splices, for which a maximum gap of $\frac{1}{16}$ in. is allowed.

When a gap between the Wood Members exists at a compression joint, the connector plate will carry 100 percent of the compressive force until there is enough strain in the joint to close the gap and full wood-to-wood contact has been established. Research has shown that Metal Connector Plates will yield, either in localized plate buckling or in slip, and permit existing gaps to close. For the smaller tested gap size of $\frac{1}{16}$ in., the gap often closed primarily due to tooth-slip, whereas local plate buckling

was more prevalent in joints tested with a $\frac{1}{8}$ -in. gap.⁶ In any case, the buckling or slip necessary to close the allowable gap sizes of $\frac{1}{8}$ in. and $\frac{1}{16}$ in. is small and will not cause joint failure. Thus, although buckling of the Metal Connector Plate is undesirable, a large number of full-scale tests have shown that compression joints perform well and transfer the forces with a substantial margin of safety even after local buckling or joint slip occurs.

From a design standpoint, localized plate buckling at gaps does not affect the strength of the Truss; however, deflection due to deformation of the compression joint may be important from a serviceability standpoint for parallel chord floor Trusses, which are often limited by deflection criteria.

Consideration of gap closure in the Top Chord member of a parallel chord wood Truss shows its effect on deflection. For example, consider a Top Chord compression splice, with a uniform gap, at the centerline of the Truss (see Figure C8.3-6).

Assuming that each half of the Truss will rotate about the bearing until the Wood Members bear at the centerline of the gap, the deflection of the Truss can be calculated as follows:

$$\tan\theta = \frac{\text{GAP}}{2D} \quad (\text{E C8.3-1})$$

$$\Delta = \tan\theta(L/2) \quad (\text{E C8.3-2})$$

$$\Delta = \frac{\text{GAP}(L)}{4D} \quad (\text{E C8.3-3})$$

The graphical representation of compression splice gaps versus Truss deflection is shown in Figure C8.3-7.

The tighter gap criterion for floor Truss Chord splices of $\frac{1}{16}$ in., versus $\frac{1}{8}$ in. allowable gap for all other joints, is consistent with the serviceability considerations related to floor Trusses. Another measure the Truss Designer can consider for improved serviceability is the use of heavier gauge Metal Connector Plates on compression Chord splices.⁷

6 L. S. Kirk, T. E. McLain and F. E. Woeste, *Effect of gap size on performance of metalplated joints in compression*, Society of Wood Science and Technology, Wood and Fiber, Vol.21, No.3, 274-288, 1989.

7 Kirk, McLain and Woeste, 274-288.

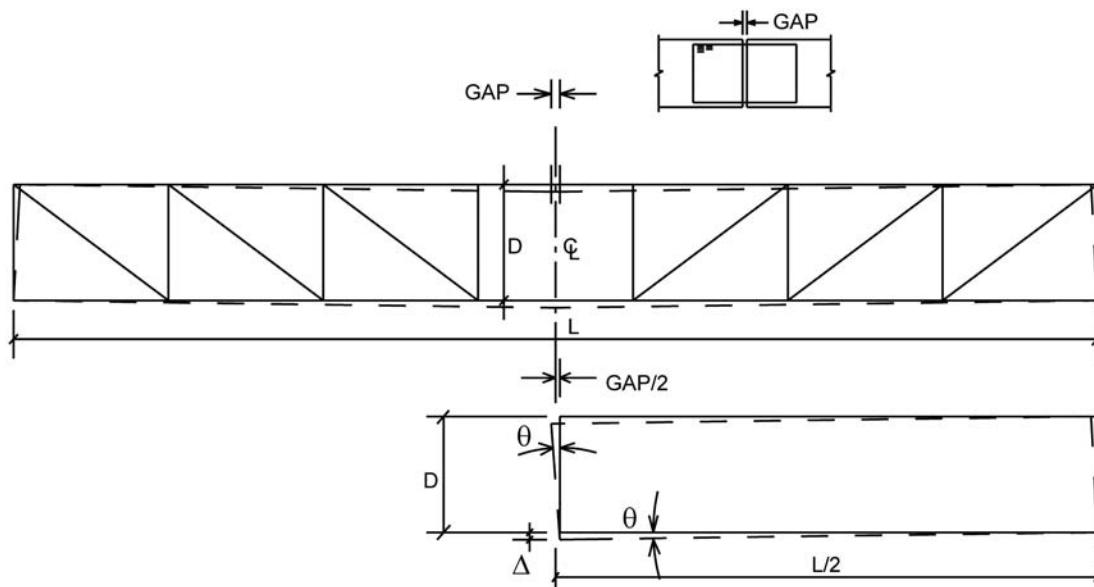


Figure C8.3-6. Theoretical Representation of Compression Splice Gap Closure Causing Truss Deflection.

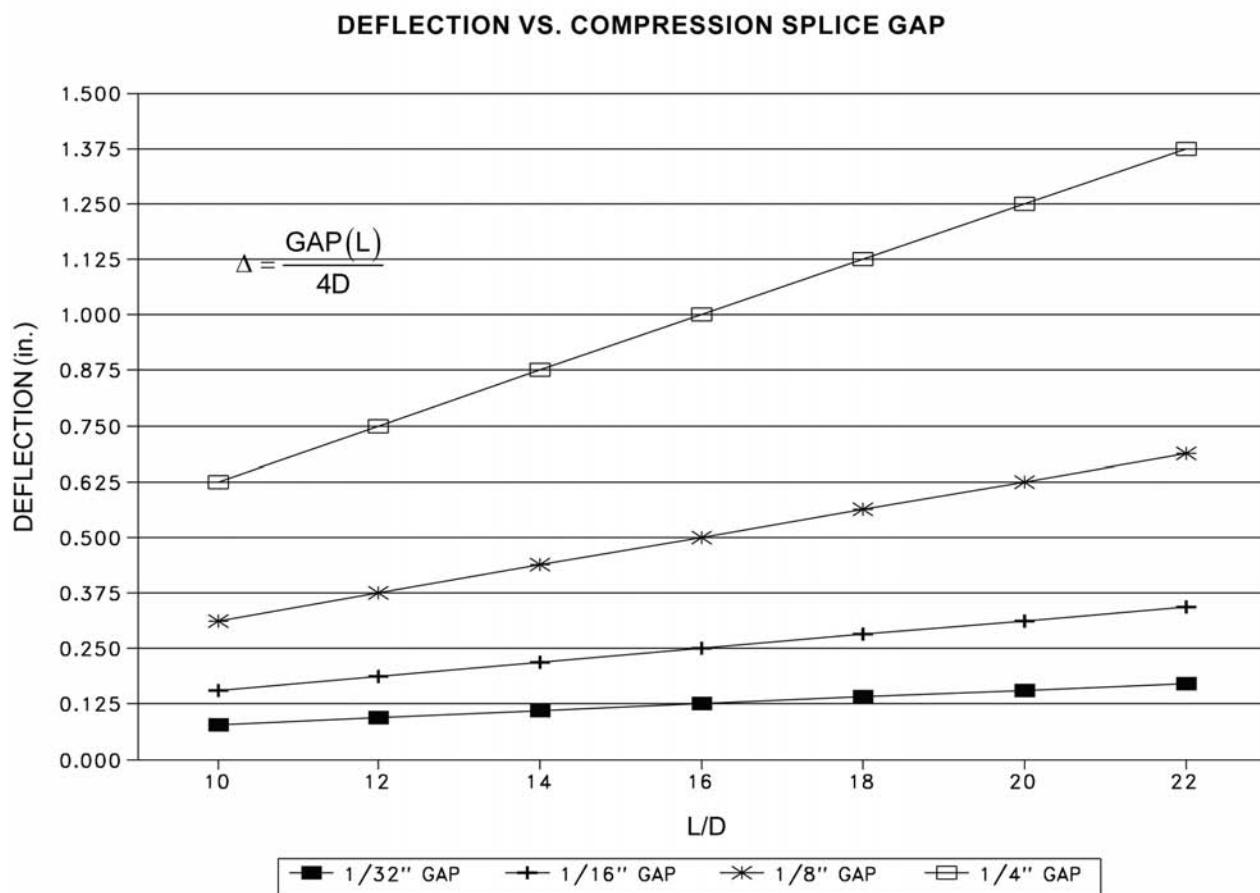


Figure C8.3-7. Graphical Representation of Compression Splice Gap Closure Versus Truss Deflection.

§8.4 TENSION

The resistance of a Metal Connector Plate to tensile forces is one measure of its ability to fasten Wood Members together where the tensile forces must be transferred through the Metal Connector Plate from one Wood Member to another.

§8.4.1 Account for Orientation.

Factors affecting the performance of the Metal Connector Plate, neglecting the effects of the Metal Connector Plate Teeth and of the Wood Members, include: length, width, and thickness of the Metal Connector Plate; location, spacing, orientation, size, and shape of holes in the Metal Connector Plate; Edge Distance of the holes in the Metal Connector Plate; and properties of the metal.

In the case of nail-on plates, their performance is also influenced by the type, size, quantity, and quality of the nails used for load transfer, as well as the method of installing the Metal Connector Plates and their fasteners.

Plates are tested for tension design values based on both orientations of the plate length with respect to direction of load (i.e., parallel and perpendicular per Section 5.4.5).

§8.4.2 Method.

Because the tension capacity of a Metal Connector Plate depends on its tooth pattern and configuration, a tensile Effectiveness Ratio is used to indicate the punched steel-to-unpunched steel strength in tension, where the unpunched steel is a solid metal control specimen.

§8.4.3 Required Cross Section.

Generally, the required tension capacity at a joint can be provided by using connector plates equal in width to the Chord material. The capacity of this type of joint is limited by the net section of the connector plates. When the capacity of this net section is exceeded, a connector plate might be used that is wider than the Chord, sometimes in combination with a wood splice block. However, in these cases, the actual Metal Connector Plate Width that is effective in transferring the tension across the joint interface is limited by a maximum allowable effective width for both mid-panel and panel point splices per Sections 8.4.3.1 and 8.4.3.2 respectively.

§8.4.3.1 Maximum Effective Width at Mid-Panel Tension Splices.

The arrangement shown in Figure 8.4-1, when tested, usually exhibits an initial tensile failure, beginning at the lower edge of the Chord-splice interface, followed by a shear failure along the interface formed by the Chord and splice block. The initial tensile failure at the lower edge

of the Chord-splice interface is caused by the eccentricity in this type of joint. This type of failure and the need to limit the maximum effective width of Metal Connector Plates that extend beyond the Chord edge at mid-panel tension Chord splices has been shown by a variety of studies.^{8, 9, 10, 11}

New to the 2007 edition of the Standard, the effective portion of plates extending off the Chord for steel tension design was clarified to be dependent upon the plate lap onto each Chord member measured parallel to grain. It was also adjusted to not be reduced by a 0.5-unit amount as previously specified in earlier editions of the Standard, and is no longer limited to plates positioned that cover nearly the entire joint line.

Early work in this area found that for a specific plate length, the plates' effectiveness for transferring tension forces decreased as the extension of the plate width increased. This initial work concluded that as the extension becomes wider, less stresses are transferred into the upper section of the extension.¹² Consequently, the Truss Plate Institute of Canada established criteria to modify the total Metal Connector Plate effectiveness for blocked tension splices using an empirically derived equation to compute a reduction factor, K.¹³ The factor, K, is used to reduce the tensile strength of the Metal Connector Plate based on the width of the Wood Member being spliced (h), and the extension of the Metal Connector Plate above the upper edge of the Wood Member (x). Figure C8.4-1 shows the Canadian equation graphically.

Later test data showed that the effective width of the Metal Connector Plate across a blocked tension splice is less than or equal to the width of the Metal Connector Plate on the Chord member plus one inch.¹⁴ This data, as summarized in Tables C8.4-1(a) and (b), is the basis for the maximum 1-inch limit past the Chord edge for splices with a block as shown in Figure 8.4-1. Note that the shear buckling and combined shear and tension provisions apply to both 2x_ and 4x_ joints.

- 8 I. Njoto and I. Salim, *Tensile strength of eccentric roof truss tension splices*, Department of Civil Engineering and Applied Mechanics, McGill University, December, 1978.
- 9 Weyerhaeuser Corporation, Communication of March 10, 1989 to Truss Plate Institute Technical Advisory Committee, 1989.
- 10 L. R. Heard, S. G. Winistorfer and O. A. Grossthanner, *Analysis of metal plate connected high capacity tension splices for wood trusses*, Proceedings of the 1988 International Conference on Timber Engineering, Seattle, WA, Vol. 2, 149-162, 1988.
- 11 Truss Plate Institute, Interim design methodology for PCT-CII 4x2 wood trusses, PCT-80 Supplement, 1986.
- 12 Njoto and Salim.
- 13 Truss Plate Institute of Canada, *Truss design procedures and specifications for light metal plate connected wood trusses*, TPIC-88 (1-88), 1988.
- 14 Weyerhaeuser Corporation.

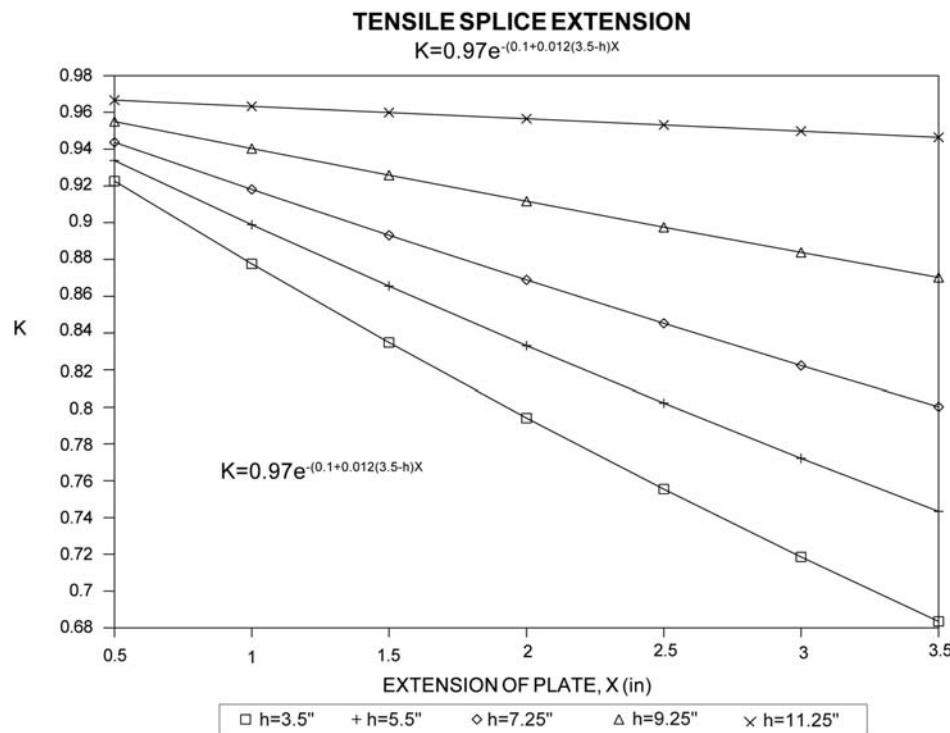


Figure C8.4-1. Approach for Modifying the Total Metal Connector Plate Effectiveness for Blocked Tension Splices as Used by the Truss Plate Institute of Canada (TPIC).

PLATE	ULTIMATE LOAD	FAILURE MODE
3x5.3	7,520 lbs	steel tension
3x5.3	6,760 lbs	steel tension
3x5.3	6,570 lbs	steel tension
3x5.3	6,890 lbs	steel tension
AVERAGE	6,935 lbs. 6935/3 = 2312 lbs/in/pair	
6x8.8	10,650 lbs	steel tension/shear
6x8.8	10,300 lbs	steel tension/shear
6x8.8	10,770 lbs	steel tension/shear
6x8.8	10,430 lbs	steel tension/shear
AVERAGE	10,538 lbs 10538/2312 = 4.56 in. effective width	

Table C8.4-1(a)
2x4 Chord Test Results.

When both blocked and unblocked tension splices are tested, it can be seen that the tensile strength can be increased more by blocking the extended plate connectors. The increased capacity of the blocked splice is due to two effects of the block at the extended section; first, the block prevents buckling of the plates under compression, and second, stresses are distributed not only into the ex-

PLATE	ULTIMATE LOAD	FAILURE MODE
5x15	22,630 lbs	steel tension
5x15	22,680 lbs	steel tension
AVERAGE	22,655 lbs. 22655/5 = 4531 lbs/in/pair	
9x17.5	30,420 lbs	steel tension
9x17.5	30,130 lbs	steel tension/shear
9x17.5	29,600 lbs	steel tension
9x17.5	30,150 lbs	steel tension/shear
AVERAGE	30,075 lbs 30075/4531 = 6.64 in. effective width	

Table C8.4-1(b)
2x6 Chord Test Results.

tended plates, but also into the wood blocking.¹⁵

Other research found that the length of the Metal Connector Plate on the tension splice had a significant effect on the ultimate tension capacity of the splices.¹⁶ Analysis of this test data, based on testing of 101 tension splice joints, produced an empirically derived equation (Equa-

15 Njoto and Salim.

16 Heard, Winistorfer and Grossthanner.

tion E C8.4-1) to determine effective width based on connector plate length as follows:

$$W_p = d - d_{le} - \frac{1}{2} + 0.12L_p \quad (\text{E C8.4-1})$$

The determination for the effective width of plates at mid-panel tension Chord splices specified in the Standard since the 1995 edition combines this effect of Metal Connector Plate Length in developing the tensile capacity of the joint with upper limits for the maximum effective distance the Metal Connector Plate can extend past the Chord member, x_{\max} ($\frac{1}{2}$ in. and 1 in. for unblocked and blocked, respectively).

The following example shows the application of the criteria outlined in Section 8.4.3.1:

Given:

$$V_t = 1000 \text{ pli/pair}$$

Bottom Chord axial tensile force = 4000 lbs.

Bottom Chord size = 2x4

Edge Distance = $\frac{1}{4}$ in.

$$W_p = P_{\max} / V_t = 4000/1000 = 4 \text{ in.}$$

1) Try unblocked, then:

$$x = \frac{1}{2} \text{ in. max}$$

$$W_p = d + x - d_{le} = 3.5 + 0.5 - 0.25 = 3.75 \text{ in.} < 4 \text{ in. NG}$$

2) Try blocked, then:

$$x = 1 \text{ in. max}$$

$$W_p = d + x - d_{le} = 3.5 + 1 - 0.25 = 4.25 \text{ in.} > 4 \text{ in. OK}$$

∴ Use a 4 in. wide Metal Connector Plate with block. Lateral resistance should also be checked to determine the required length of the Metal Connector Plate.

§8.4.3.2 Maximum Effective Width at Panel Point Tension Splices.

A Bottom Chord pitch change in a scissors Truss is an example of a joint addressed by this section that might require reduction of the actual Metal Connector Plate Width to an effective tensile width.

Earlier editions of the Standard did not specify how to compute the maximum effective width of this type of joint (i.e., panel point tension splices).

The 1995 edition required that reductions of the actual plate width to an effective tensile width be made, which was to be as specified by the Truss Designer, and the Commentary did not provide any guidance other than

to state, *Uneven force distributions through a joint of this nature make it difficult to assess the effective tensile width of the Metal Connector Plate. Other variables such as tooth patterns and configuration add to this difficulty. Misra and Esmay (1966) provided some insight regarding the effect of tooth configuration on tensile strength of Metal Connector Plates.*

The limit incorporated since the 2002 edition of the Standard of 1.5 in. past the inside edge of the Chord is based on empirical data.

§8.4.3.3 Connector Plate Designed for Moment.

This section is new to the 2007 edition of the Standard and is intended to provide for the transfer of moment that occurs due to eccentricity between the wood section and the plate section, which is recognized within the moment design equation in Section 8.7, but which would not necessarily be considered if the analysis is made presuming a Chord-to-Chord splice is a pin-connected joint. In such a case the joint shall be designed to carry axial force and the moment created due to this eccentricity. The centerline of the Truss Chord member is the geometric center of member. The concept of the centerline of the steel plate is described here. When the Metal Connector Plate is used at the joint, it creates an overlap with the edge of the Chord member. The geometric center of this overlap distance is assumed to be the centerline of the steel cross-section. Figure C8.4-2 explains this concept. The distance between the centerline of the Chord member and the steel cross-section is defined as the eccentricity, which when multiplied by the transferred axial force through the member results in the moment requirement for the joint design.

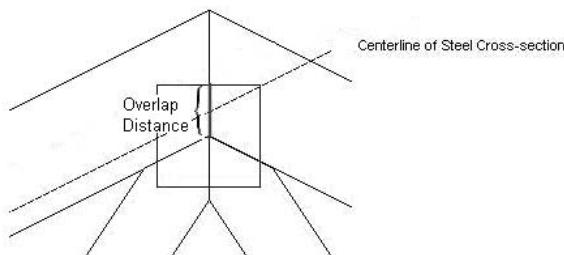


Figure C8.4-2. Geometric Center of Overlap Distance.

§8.5 SHEAR

The resistance of a Metal Connector Plate to shear forces is one measure of its ability to transmit loads from Wood Member to Wood Member where the forces must be transferred through the Metal Connector Plate.

§8.5.1 General.

Factors affecting the performance of the Metal Connector Plate, neglecting the effects of the Metal Connector Plate Teeth and of the Wood Members, include: length, width, and thickness of the Metal Connector Plate; location, spacing, orientation, size, and shape of holes in the Metal Connector Plate; Edge Distance of the holes in the Metal Connector Plate; and properties of the metal.

Plates are tested for shear design values based on six orientations of the plate length with respect to the wood shear plane as specified in Section 5.3.5.

§8.5.2 Method.

Like tension capacity, the shear capacity of a Metal Connector Plate depends on tooth pattern, and thus a shear resistance Effectiveness Ratio, indicating the punched steel-to-unpunched steel strength in shear, is used to determine allowable shear capacity.

§8.5.3 Required Cross Section.

Equation E8.5-2 gives a required plate dimension with respect to shear based on shear capacity per unit plate length. However, as a plate size increases, the ultimate unit shear strength decreases when the length of any unsupported plate edge increases as a result.¹⁷ Thus, if a plate has unsupported plate contact areas, this design check alone is not sufficient to determine if a plate has enough capacity to carry the shear load, and so Sections 8.5.4 and 8.5.6 must be met as well.

§8.5.4 Shear Values Parallel and Perpendicular to the Major Axis.

The use of a limited effective area (“triangle”) within unsupported plate areas [see Figures 8.5-1(a)-(e)] for determining shear capacity was first introduced as part of an interim design methodology for 4x2 and 2x4/2x6 parallel Chord Trusses in supplements to the 1980/1985 editions of the Standard.^{18, 19} These interim design methodologies were developed in order to address the then-new observances, in the early 1980s, of shear buckling in unsupported plate areas, as longer spans of 4x2 parallel chord Trusses were increasingly being used in commercial and industrial Buildings. Thus, the combined effect of larger member forces, due to longer spans, and limited surface area of wood available for plating, based on the 4x2 orientation, resulted in observed occurrences of shear

buckling in the unsupported areas of the plate (i.e., areas of plate not in direct contact with the wood) that had not been previously observed.

At the time that the interim design methodologies were issued, there was no published information on the subject of plate buckling in the unsupported areas of the plate. The provision in the interim design methodologies limiting the effective area of the plate to a fixed triangular area was based on unpublished, proprietary testing results and the collective experience of several companies and leading technical personnel in the industry. A research study later confirmed that plate buckling does occur at Truss joints having high shear stresses and unsupported plate areas, which can cause premature failure of the Truss joint when the design is based on full shear strength.¹³ Specifically, the research found that as the length of the unsupported plate edges increased, the ultimate unit shear strength decreased.

The methodology set forth in the Standard utilizing a fixed effective “triangle” area is a conservative approach to defining shear strength of the Metal Connector Plates, based on the correlation of shear strength to unsupported plate height rather than to unsupported plate area. In other words, the fixed triangle area method does not consider any of the strength contributed by the plate length outside of the fixed plate area.

§8.5.5 Combination of Shear Plates and Tension Plates.

An example of a joint addressed by Section 8.5.5 is a blocked Bottom Chord tension splice joint, with a single plate on each face acting in tension as described in Section 8.4.3.1 and two plates on each face acting in shear, one on either side of the tension plate (see Figure C8.5-1).

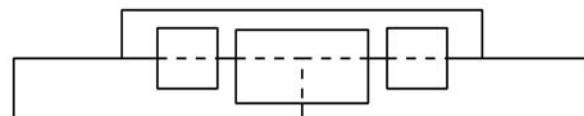


Figure C8.5-1. Blocked Bottom Chord Tension Splice Joint with Side Shear Plates.

This provision prohibiting the use of side shear plates at tension splice joints was added in the 2002 edition and maintained in this edition because side shear plates have been recognized to be largely ineffective, due to a much lower level of stiffness in plate shear relative to plate tension. Testing has shown that less than 20 percent of

17 G. L. Kocher, *An experimental investigation of buckling in the unsupported regions of metal connector plates as used in parallel-chord wood truss joints*, Department of Civil and Environmental Engineering, Marquette University, July, 1986.

18 Truss Plate Institute, *Interim design methodology for PCT-CII 4x2 wood trusses*, PCT-80 Supplement, 1986.

19 Truss Plate Institute.

the shear capacity of the side plates may be realized at the time of joint failure. Therefore, a splice joint utilizing side shear plates will fail prematurely if the design capacity of such a joint includes the full shear capacity of the side plates.

Much earlier splice tests indicated that attempts to increase the capacity of the tension splice joint by adding shear plates on either side of the tension plate did not significantly add to the strength of the joint and, consequently, resulted in a lower factor of safety than a simple, in-line tension splice.²⁰ A conclusion from this finding suggests that the use of heavier gauge plates, less critical splice joint locations, closer Truss spacing, or larger Bottom Chords all provide simpler, better alternatives to a complex blocked splice joint that utilizes shear plates.

§8.5.6 Prevention of Shear Buckling.

The provision in Section 8.5.4 that limits shear capacity in the unsupported areas of the plate to a triangular area only applies to the joints detailed in Figures 8.5-1(a)-(e). Since these details do not cover all joints subject to shear forces and having unsupported plate areas (e.g., a hip shoulder joint), the provision in Section 8.5.6 was added in order to require consideration for shear buckling in any shear-carrying joint, and not just those for which the phenomenon of shear buckling was initially discovered. It should be noted that this provision should be applied to those unsupported plate areas due in particular to round-ended or square-ended Web members, where the potential for a reduced shear strength should be considered.

§8.6 COMBINED SHEAR-TENSION

Similar to the provision in Section 8.5.4 for shear only, this provision for combined shear-tension, using a fixed limited effective area (triangle) for the unsupported region of the plate, has been specified since the interim supplements to the 1980/1985 editions of the Standard.^{21, 22} Refer to Section §8.5.4 and see also Example C8.6-1, which shows how the criteria for shear and combined shear-tension apply to a joint such as on an intermediate-height bearing parallel chord Truss.

§8.7 COMBINED FLEXURE AND AXIAL LOADING

Prior to the 2002 edition, the Standard did not include a design method to account for moment in Metal Connector Plates. The 1995 edition addressed combined flexure and axial loading with the following statement, *Design*

of inter-panel splices in the top or Bottom Chord not located within 12 inches of the calculated point of zero moment shall include the additional stress caused by flexure. The only guidance provided for this situation was an approximate approach presented in the Commentary. This approach, as applied to a Bottom Chord splice for example, was to convert the applied moment, M, into an equivalent tension force, T_{eq} , which could be added to the applied axial tension, T_{app} , determined from analysis, to get the resulting design tension force, T ($T = T_{app} + T_{eq}$). Equation E C8.7-1 is provided to calculate the equivalent tension force:

$$T_{eq} = 6M/d \quad (\text{E C8.7-1})$$

This is based on T_{eq} being set equal to a force corresponding to that produced presuming a uniform tensile stress equal in magnitude to the maximum bending stress on the wood cross-section. In other words, $T_{eq} = f_b \times d \times 2 \times t$, where $f_b = M/S$ and $S = 2 \times t \times d^2 / 6$, thus $T_{eq} = M \times d \times 2 \times t / (2 \times t \times d^2 / 6) = 6 \times M / d$. The plates would then be designed for this modified tension force.

This method is accurate in some situations but has been found to be generally conservative. It is provided here only as a simpler approach than the more accurate equations given in Section 8.7.

Published research findings following the 1995 edition of the Standard provided the basis for the new provisions for designing plate steel section for effect of moment (see Section 8.7.1) and designing plate lateral resistance for effect of moment (see Section 8.7.2) in the 2007 edition.²³

§8.7.1 Design of Steel Section for Effect of Moment.

The basis for the equations in this section is testing and theory. In one study on splice joints, the joints were tested in combined tension and bending loading, all of which failed in the steel net-section of the Metal Connector Plates.²⁴ The results of the tested specimens were compared against three theoretical models used to predict the ultimate moment capacity of the steel net-section of the splice joints. The equations in Section 8.7.1, originally included in the 2002 specification in a slightly different form, are developed from the most accurate model from this research as validated by testing. Subsequent use and further research showed the need for modification of this

23 S. L. Lewis, N. R. Mason, S. M. Cramer, D. C. Wert, P. J. O'Regan, G. Petrov and D. Goclan, *Design of Metal Plate Connected Wood Truss Joints for Moment*, Proceedings of 2006 World Conference on Timber Engineering, Portland, OR.

24 P. J. O'Regan, F. E. Woeste and S. L. Lewis, *Design procedure for the steel net section of tension splice joints in MPC wood trusses*, Forest Products Journal, Vol. 48, No. 5, 35-42, 1998.

20 Sheppard Engineering, Communication of September 18, 1979 to Truss Plate Institute, 1979.

21 Kocher.

22 Truss Plate Institute.

Example C8.6-1. Intermediate-Height Bearing Parallel Chord Truss.

Given: 20 gauge plates

$$\begin{aligned} V_{s\parallel} &= 600 \text{ pli/pair} & V_{s\perp} &= 650 \text{ pli/pair} \\ V_{t\parallel} &= 1000 \text{ pli/pair} & V_{t\perp} &= 700 \text{ pli/pair} \\ R &= 4000 \text{ lbs} \end{aligned}$$

Calculated data:

$$P_{TW} = 5656.8 \text{ lbs (assuming web at } 45^\circ)$$

$$P_R = 4000 \text{ lbs}$$

$$L_1 = 6.53"$$

$$L_2 = 3.53"$$

$$L_3 = 6.41"$$

$$L_4 = 3.53"$$

1) Shear check:

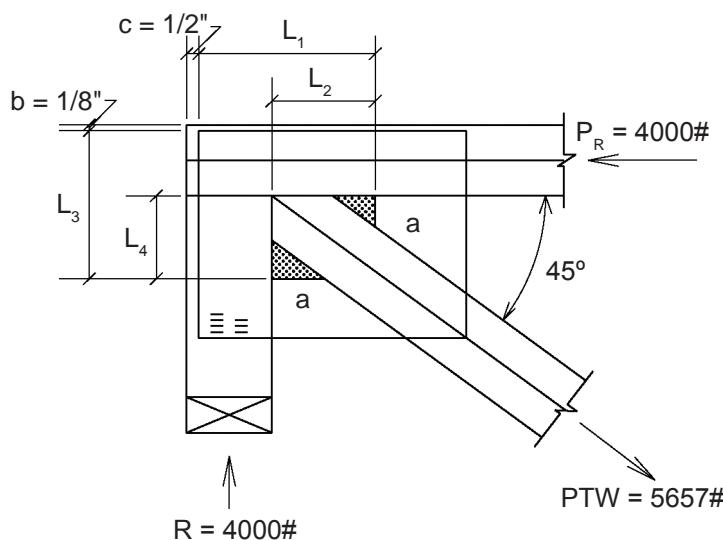
$$\text{a) } L_1(V_{s\parallel}) \geq |P_L - P_R| \quad (\text{E8.5-3})$$

$$6.53(600) = 3918 < 4000 \text{ NG}$$

Try $c = 1/4"$, then $L_1 = 6.78"$
 $6.78(600) = 4068 > 4000 \text{ OK}$

$$\text{b) } L_3(V_{s\perp}) \geq P_{TW} \sin \theta \quad (\text{E8.5-4})$$

$$6.41(650) = 4167 > 4000 \text{ OK}$$



2) Combined shear-tension check:

$$(X_{st} L_2) + (Y_{st} L_4) > P_{TW} \quad (\text{E8.6-1})$$

$$X_{st} = V_{s\parallel} + \frac{(V_{t\perp} - V_{s\parallel})\theta}{90^\circ}$$

$$X_{st} = 600 + 0.5(700 - 600) = 650$$

$$Y_{st} = V_{t\parallel} + \frac{(V_{s\perp} - V_{t\parallel})\theta}{90^\circ}$$

$$Y_{st} = 1000 + 0.5(650 - 1000) = 825$$

$$650(3.53) + 825(3.53) = 5206.8 < 5656.8 \text{ NG}$$

3) Try 16 gauge plates:

$$\begin{aligned} V_{s\parallel} &= 900 \text{ pli/pair} & V_{s\perp} &= 925 \text{ pli/pair} \\ V_{t\parallel} &= 1600 \text{ pli/pair} & V_{t\perp} &= 950 \text{ pli/pair} \end{aligned}$$

$$\begin{aligned} L_1 &= 6.85" & L_2 &= 3.85" \\ L_3 &= 6.73" & L_4 &= 3.85" \end{aligned}$$

Check shear:

$$6.85(900) = 6165 > 4000 \text{ OK}$$

$$6.73(925) = 6225 > 4000 \text{ OK}$$

Check combined shear-tension:

$$X_{st} = 900 + 0.5(950 - 900) = 925$$

$$Y_{st} = 1600 + 0.5(925 - 1600) = 1262.5$$

$$(925 + 1262.5)3.85 = 8421.9 > 5656.9 \text{ OK}$$

Use 16-gauge plates

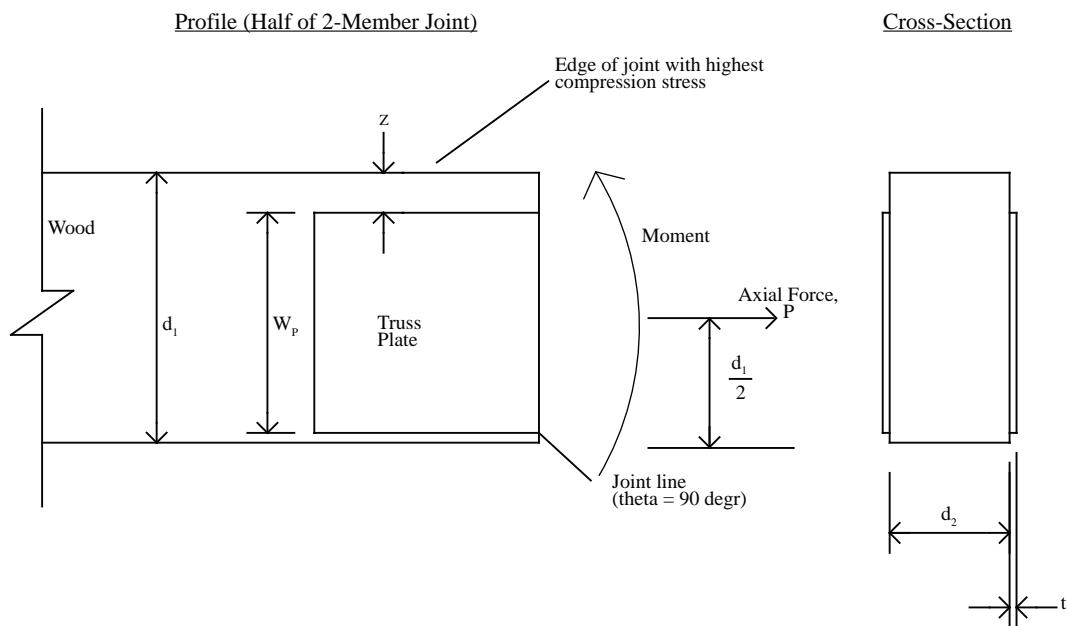


Figure C8.7-1. Illustration of Some Variables Referenced in Section 8.7.1.

method to recognize the interaction between axial compression and moment stresses and to recognize the effect of plates located off center. The moment design equation of the plate steel cross-section was updated in the 2007 edition of the Standard per recent TPI Technical Advisory Committee (TAC) research recognizing the effects of plate position, compression force on the joint and for mid-panel splices the P-delta effects. Further information on this research is available at: www.ewpa.com/Archive/2006/aug/Paper_322.pdf. Figure C8.7-1 illustrates some of the variables used in this equation.

§8.7.2 Design of Plate Lateral Resistance for Effect of Moment.

The tooth holding stress due to moment, V_M , is determined in Section 8.7.2 using a plastic theory method, which has been shown by testing to be more accurate than elastic assumptions.²⁵ Because this stress (V_M) acts in all directions, it must be considered as acting in combination with the resultant vector of the other tooth holding stresses (i.e., from axial and shear forces), and the sum of these stresses must be limited to the allowable tooth stress at the angle of the resultant vector. This check is addressed by Equation E8.7-3. A second check is also required to ensure that the moment stress does not exceed the allowable tooth holding stresses at any orientation, which is checked by Equation E8.7-2. These two checks can be summarized as follows:

[Moment Stress + Shear/Axial Stress]	\leq Allowable stress at angle of Shear/Axial Stress
Moment Stress	\leq Allowable stress at any angle for the plate/wood

Figure C8.7-2 illustrates the plastic method to determine tooth holding stress due to moment, calculated as $4M_A / (A_{ef}D)$, where D is the diagonal of a rectangle equivalent to the effective plate area, A_{ef} .

§8.8 NET SECTION LUMBER CHECK (h')

Another critical factor in the design of Metal Connector Plates, aside from the plate tooth holding, shear, and tensile strength considerations, is the wood strength of the reduced net section where a Metal Connector Plate is attached, especially for tension members.

§8.8.1 Reduced Net Section Checks.

The requirement that the allowable axial stress (F_c or F_t) of the Wood Member is not exceeded on the reduced net section has been specified since the 1985 edition. Figure 8.8-1 illustrates the reduced net lumber section on three typical Truss member joints. The dimension designated h' on the tension members indicates the effective width of the wood tension member which is stressed at the actual location of the joint. The dimension ' d_2 ' represents the thickness of the member (usually 1½ in.). Therefore, the effective cross sectional area at the joint resisting the tension force equals $h' \times d_2$. When the axial force in the tension member divided by the effective cross sectional area exceeds the allowable tension stress, F_t , for the wood

²⁵ A. Kevanmaki, *Moment capacity and stiffness of punched metal plate fastener joints*, Proceedings of the 1996 International Wood Engineering Conference, New Orleans, LA, Vol. 1, 385-392, 1996.

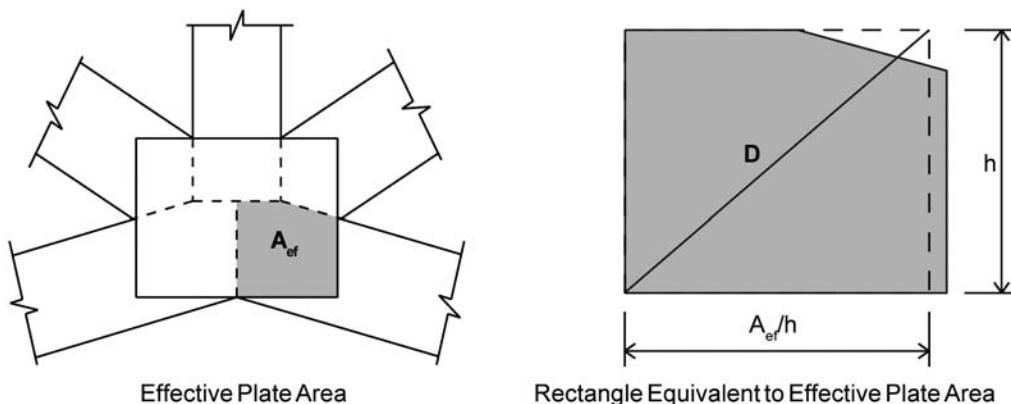


Figure C8.7-2. Tooth Holding Stress Due to Moment, $V_M = 4M_A/A_{ef}D$.

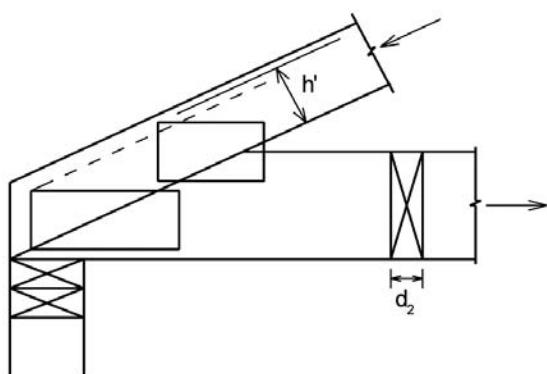


Figure C8.8-1. For the Top Chord in this Detail, h' is Defined By the Metal Connector Plate that Covers the Largest Depth of the Chord Member.

tension member, the potential of wood tension failure at the reduced net cross section exists. This mode of failure can be consistently reproduced in load test studies.

Note that for a heel joint, the h' dimension on the Bottom Chord extends from the top of the plate on the Bottom Chord to the lower edge of the Bottom Chord (see Figure 8.8-1). If the effective width of the Bottom Chord at the heel joint were considered only to extend to the bottom of the plate, then the shear capacity of the wood along the bottom of the plate would be added to the total capacity. However, a shear failure at this location (the shear line being the bottom edge of the plate) is not consistent with the failures observed at heel joints, which are always vertical fractures. Thus, assuming relatively small Edge Distances from the bottom of the Metal Connector Plate to the bottom of the Chord, the effective width of the Bottom Chord at a heel joint can be considered as extending to the bottom of the Chord.

The dimension designated h' on the compression members indicates the effective width of the wood compression member which is stressed at the actual location of the joint. The effective cross section at the joint resisting the compression force also equals $h' \times d_2$. When the axial force in the compression member divided by the effective cross sectional area exceeds the full allowable compression stress, F_c , for the wood compression member, the potential for a wood compression or shear failure exists. The dimension, h' , for the Top Chord of a heel joint with overlapping double Metal Connector Plates, as shown in Figure C8.8-1, is defined by the Metal Connector Plate that covers the largest depth of the Top Chord. The dimension, h' , for the Bottom Chord of this same joint is defined by adding the effective width of each plate. The distance between the plates in this joint should be evaluated based on the allowable horizontal shear and tension values of the Bottom Chord Wood Member.

§8.8.2 Limit on Tension Introduced into a Wood Member.

Test data from applications with plates on the Narrow Faces of 2x4 lumber has shown that the relatively high stress concentrations in the outermost portion of the narrow lumber faces where the Teeth penetrate can result in premature wood fracture, especially when a knot or other grain deviation occurs near the plate.

To check against a wood “chunk-out” failure at tension joints for wood thickness greater than 2 in. (38 mm) nominal, particularly with narrow-face plating, the tension introduced in the Wood Member should not exceed that specified by the Truss Design Engineer.

The provision in Section 8.8.2 is intended to address a design consideration for a possible mode of failure where high tension forces create a shear failure parallel to the

grain of the wood directly beneath the connector plate Teeth. The appropriate technical term for this type of failure mode may be shear-out, block shear or shear-plug failure, but it is actually a failure due to shear fracture as well as tension fracture. This type of failure can potentially occur when a high tensile force must be transferred from a Metal Connector Plate to a Wood Member and the plate is fastened to the Narrow Face at the end of a member. It is this transfer of load and the resulting non-uniform stress distribution that can prompt the wood to shear underneath the plate, along with lumber tensile fracture at or near the end of the plate, leaving a significant portion of wood embedded in the plate as the joint pulls apart. Figure C8.8-2 illustrates this failure mode.

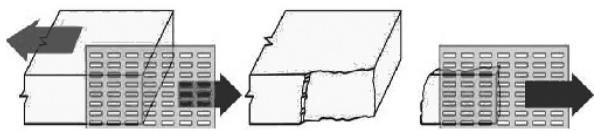


Figure C8.8.2. Illustration of Wood “Chunk-Out” Failure Mode.

This is a difficult condition to address by making use of standardized wood design values, as the small areas of lumber subject to these stresses are subject to larger variations in strength than are addressed by standard wood grading procedures. The TPI 1 Project Committee (PC) and TPI Technical Advisory Committee (TAC) concluded that there are circumstances where this failure mode can occur at a load below the tensile strength of the connecting Wood Member, the plate tooth withdrawal and associated plate values and should therefore be considered as a separate design criterion. In 2006, TPI TAC reviewed the available data and recommended the design value of 2300 lbs. tension per inch of wood depth. A prior limit of 1600 lbs. tension per in. of wood depth (2400 lbs. for a 4x2) in the 2002 edition of the Standard (mandatory language) was replaced by the non-mandatory note in the current Standard recommending a limit of 2300 lbs. per in. (3450 lbs. for a 4x2). The prior limit of 1600 lbs./in. was not based on significantly lower data values than the 2300 lbs. per in. limit currently recommended, and was lower primarily due to the limited number of data points. Additional testing done by members of TPI TAC to address this limited sampling along with reports of 4x2 Truss tests from a WTCA member resulted in the evaluation by the TPI TAC that the 2300 lbs. value was a reasonable recommendation. This limit was also modified to not apply at locations other than where the Wood Member ends are under or within 1 ft. of the edge of the Truss plate. The mandatory wording in this section speci-

fies that the limit to the value of tensile force in a given circumstance shall be defined by the Truss Design Engineer as they are best qualified to account for any effects unique to their design situation. The value of 2300 lbs. per in. in the non-mandatory note as being recommended by the TPI TAC is intended to provide guidance to Truss Design Engineers who do not have access to further data.

The typical Truss design conditions where the limits would pertain include:

- Short span, Top Chord bearing floor Trusses with heavy loads.
- Floor Trusses with tension Webs at very shallow angles.
- Heavily loaded commercial floor Trusses.
- 4x2 purlin Trusses in panelized roof systems.

Designing to avoid this failure mode may consist of increasing the thickness of the wood member or modifying the Truss configuration to minimize the tension force in the member, such as flipping the direction of Webs within a panel or increasing the pitch of the tension web. The intent of the industry is to continue to gather data, when possible, and conduct further research on this issue to more thoroughly understand the nuances of this occurrence. It has been theorized that the likelihood of this failure mode may be able to be minimized by avoiding any grain deviations (knots or slope of grain) near the plate, so that may be another technique that may be considered by Truss Design Engineers and Truss Manufacturers.

§8.9.1 Lumber Dowel Bearing Strength Increase.

The provisions for increasing dowel bearing strength of lumber with the use of Metal Connector Plates are based on testing of over 200 joints. Bolt capacities have been found to have two to three times standard design values when nail plates were used to reinforce the wood.²⁶

§8.9.2 Bolts and other Fasteners.

The 1995 and earlier editions did not address the situation where fasteners, such as hanger nails or bolts that were not included in the Truss design, are driven through a Metal Connector Plate for the purposes of installation. Thus, questions regarding the acceptance of any extraneous fasteners were necessarily directed back to the Truss Designer.

²⁶ A. Kevarinmaki and J. Kangas, *Nail plate reinforced bolt joints of Kerto-LVL structures*, Proceedings of the 1996 International Wood Engineering Conference, New Orleans, LA, Vol.3, 271-278, 1996.

Because the addition of other fasteners at a Truss joint is not uncommon, the 2007 edition of the Standard recognizes this situation as being acceptable as long as the fastener hole is away from the critical steel shear or tension plane in the joint. The 1-in. limit that defines the critical location relative to the joint line is based on engineering judgment having achieved general consensus.

§8.10 TENSION PERPENDICULAR TO GRAIN

The bite of a Metal Connector Plate onto a Chord member that is subject to perpendicular to grain forces is critical in relation to the amount of axial force that can be transferred through the joint. This is due to the low resistance of lumber to horizontal shear and tension perpendicular to grain stresses.²⁷ Inasmuch as this becomes a member design issue, Section 8.10 refers to Section 7.5.3.3 to determine the amount by which to extend the Metal Connector Plate past the member centerline. The modifications to the tension perpendicular to grain limits mentioned in Chapter 7 for girder Truss plated connections also apply to all Trusses (see Sections §7.5.3.2 and §7.5.3.2.1 for the basis of the 800-lb. limit and additional discussion).

§8.11 PLATE POSITIONING TOLERANCE

Section 8.11 relates design to manufacturing tolerances by establishing design-based quality criteria to be used in the manufacture of metal-plate-connected wood Trusses.

§8.11.2 Joint QC Detail.

Section 8.11.2 specifies the minimum amount of information that must be depicted on the Joint QC Detail, a full-scale graphical detail of a Truss joint that shows positioning tolerances calculated by the Truss Designer for any particular joint of a Truss. This information, at a minimum, ensures that the Joint QC Detail can be effectively aligned on the actual Truss joint and accurately display the region on the Truss within which the center of the actual plate must be located to be in conformance with the Truss design. The region of allowable plate position may in some cases be a single point, it may also be a polygon defined by a minimum of four points, or it may be an irregular shape that is defined by many more than four points.

In order for the Joint QC Detail to be an effective tool for use by the Truss Manufacturer in an In-Plant Quality Assurance Program, the Joint QC Detail must be able to be printed on an 8- $\frac{1}{2}$ in. x 11 in. template, such as a sheet

of velum. The Joint QC Detail may not always show all member or plate edges at the joint. It is only necessary that a sufficient number of reference lines depicting the edges of the Wood Members and the center point of the plate at its design position are shown in order to align the detail on the joint. Additional items that may be shown on the Joint QC Detail to assist in the Truss Manufacturer's inspection of the joint include angled reference lines defining the maximum allowable plate rotation (positive or negative) from the primary axis of the plate, and shapes that represent, to scale, the tolerance percentage of the plate contact area on each member at the joint. This tolerance percentage of the plate contact area at each member is permitted to contain any combination of lumber characteristics (i.e., a loose knot, wane, etc.) and flattened Teeth, and the Truss Manufacturer will conduct a visual inspection that this amount is not exceeded. Figure 3.7-1 shows an example of a Joint QC Detail, which depicts these additional items.

New to the 2007 edition of the Standard, the tolerances for the plate positioning shall be shown on the Joint QC Detail for the selected fabrication tolerance, as well as for a 0 percent fabrication tolerance ($C_q = 100$).

§8.11.3 Joint Stress Index.

The Joint Stress Index has been defined in the 2007 edition to effectively quantify the stress level of a plate contact area, given its location (as it relates to the number of Teeth in the plate contact area, net section steel, and net section lumber) and all applicable design forces acting at the joint. Because it is computed by taking the ratio of applied force to allowable design force, the Joint Stress Index should never exceed 1.0, similar in concept to a combined stress index (CSI) for Wood Members, except the JSI is computed for each joint design force separately, with the maximum JSI controlling.

Defining a Joint Stress Index provides two useful purposes in the context of the associated quality criteria for the manufactured Truss (see Chapter 3). First, it provides a means to characterize the amount a plate can be translated from its design position in any given direction before it becomes critical (i.e., JSI = 1.0). Thus, the JSI is used to determine a plate positioning tolerance and create a Joint QC Detail that is customized to any joint.

Second, the JSI can be used as an indicator of those joints in a Truss that are critical to the structural performance of the finished product, and thus are more critical in terms of proper plate placement during the manufacturing process. A joint that is fully, or near fully, stressed to its allowable limit will be critical such that the performance of the

²⁷ W. R. McAlpine and O. A. Grossthanner, *Proposed design methods for three typical truss details*, Proceedings of the 1979 Metal Plate Wood Truss Conference, P-79-28, Forest Products Research Society, Madison, WI, 227-231, 1979.

Truss will depend on its proper placement. In contrast, a joint where the connector plate is stressed to only half its allowable capacity will unlikely influence Truss performance, even if potentially located “out of tolerance” by an amount that is not apparent without inspection. Thus, the JSI of the joint is also used to determine which joints of a Truss must be inspected, at a minimum, as part of a Truss Manufacturer’s In-Plant Quality Assurance Program (see Section 3.7.1).

The wording in Section 8.11.3 to use “the selected C_q value” permits the use of either the C_q value corresponding to the fabrication tolerance selected by the fabricator or the maximum C_q value of 1.0. The practice per the *TPI 1-2002* standard was to use only the C_q value corresponding to the fabrication tolerance selected by the fabricator, however it was noted that in many instances the only purpose of the selection of a lower C_q value was in order to reduce the JSI in order to minimize the number of critical joints, which were defined as joints that must be inspected on any Trusses selected for inspection per Section 3.2.2, rather than to accommodate greater joint fabrication tolerances as was its intent. As presented in Section §3.7.1, the *TPI 1-2007* edition introduced a number of changes that permit the Truss fabricator to better control the number and type of joints that must be inspected. The use of a C_q value of 1.0 for purposes of determining JSI, regardless of the selected fabrication tolerance, is one such change.

§8.11.4 Minimum Plate Positioning Tolerances.

The maximum positioning tolerance in any direction shall be limited by the JSI, which is defined for each joint as the largest ratio of applied force to allowable design force determined. The 1995 edition contained a plate location tolerance that permitted plate translation of up to $\frac{1}{2}$ in. for all plates (and $\frac{3}{4}$ in. for plates over a given length and width), provided the effective tooth requirements were met. Thus, it is reasonable to allow a plate, whose translation in any given direction is not controlled by tooth grip, to have a minimum $\frac{1}{2}$ in. tolerance for translation in that direction. For the rare case where a steel stress check would otherwise indicate a maximum allowable translation of less than $\frac{1}{2}$ in. based on steel JSI, it can be assumed that the steel safety factor adequately accounts for this. New to the 2007 edition of the Standard, the plate placement limit need be no less than $\frac{1}{8}$ in. for lateral resistance limits. Also, the plate placement limit is now to be determined with consideration of the rotation limits per Section 8.11.5.

§8.11.5 Plate Rotation.

This topic was previously contained in Section 8.2 of *TPI 1-2002*. Earlier editions of the Standard did not include a plate rotation tolerance as an explicit design check within the joint design procedures. Prior to the 2002 edition, plate rotation was addressed solely by a maximum allowable plate rotation provision contained within the quality criteria for the manufactured metal-plate-connected wood Truss (see Figure C8.11-1). The maximum amount of allowable plate rotation set forth by this provision was considered to be acceptable without the need for explicit design consideration or reanalysis.

The 2007 edition includes provisions for customized positioning tolerances, which are specific to each metal connected joint as calculated by the Truss Manufacturer. Since the new calculated positioning tolerance accounts for plate translation in any direction only, and thus gives the Truss Manufacturer only the maximum allowable plate translation at any joint, a separate explicit design check is required for plate rotation.

A rotation tolerance of ten degrees from the plate design position was determined to be a reasonable limit to set forth in the quality criteria in Chapter 3 (see Section 3.7.3). A plate rotation of ten degrees is within the range of plate rotation permitted by the standardized approach in earlier editions (per Figure C8.11-1), and it is reasonable to expect that this plate rotation limit can be met in the manufacturing process.

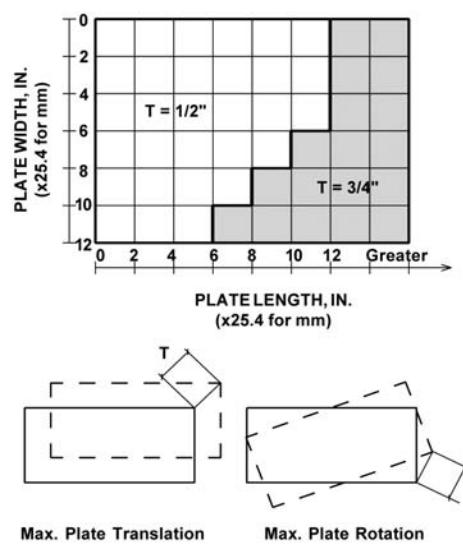


Figure C8.11-1. Earlier Editions Used this Standardized Metal Connector Plate Location Tolerance (T) for All Joints.

The exception if the geometry of the plate at its design position at the joint is such that a ten-degree or less rotation of the plate would result in a portion of the plate extending beyond the outside perimeter of the Truss, was removed in the 2007 edition of the Standard since that requirement would not impact the structural integrity.

**APPENDIX A
(NON-MANDATORY)
COMMENTARY ON PERMANENT TRUSS BRACING**

A.1 GENERAL

TPI 1-2007 provides for the design of individual metal-plate-connected wood Trusses as structural components only. The need for, and location of, Lateral Restraints to reduce the buckling length of individual Truss members is determined as part of the wood Truss design and is the only permanent restraint/bracing requirement that will be specified by the Truss Design Engineer or Truss Designer [see *TPI 1* Sections 2.3.5.5(o) and 2.4.5.4(o)]. The permanent bracing specified on the Truss Design Drawing does not indicate the restraint/bracing requirements for an entire roof or floor system, nor for the structural safety or overall stability of the Building.

As the Building Designer is responsible for incorporating the wood Truss component into the building structure, the Building Designer must determine the Lateral Restraining/bracing details, including method of connection and transfer of member buckling forces to the structure. This commentary provides guidance to the Building Designer regarding permanent Truss restraint/bracing requirements, including Top Chord bracing, Bottom Chord bracing, and Web Member bracing requirements. The size, connection, and anchorage of Permanent Individual Truss Member Restraints may be determined using the approach given in this Appendix for computing brace forces (see Section A.2). Other acceptable methods for bracing wood Trusses may be determined by the Building Designer, and additional sources of information on this topic are provided at the end of this Appendix including the *BCSI* booklet published by WTCA and TPI.¹

Other special building design requirements to achieve total structural integrity, such as wind bracing, portal bracing, seismic bracing, diaphragms, shear walls, or other load transfer elements and their connections to the wood Trusses, must be considered separately by the Building Designer (see *TPI 1* Sections 2.3.3 and 2.4.3). For a discussion on permanent restraint/bracing to resist lateral forces imposed on the completed building by wind or seismic forces, the reader is referred

to *Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses*).²

A.2 TRUSS BRACING FORCES

Permanent Truss restraint/bracing must provide support at right angles to the plane of the Truss to hold every Truss member in the position assumed for it in the design and to reduce buckling lengths.

The brace force (BF) is the force applied at right angles to the Truss member, which will hold the member in a stable lateral alignment. The BF acting on each Lateral Restraining may be assumed to be two percent of the maximum calculated axial compression force in the Truss member. The BF should be considered to act on the Lateral Restraining in both tension and compression.

To be effective, Lateral Restraints must be used in conjunction with diagonal bracing or some equivalent means of bracing. The cumulative lateral restraint force (P) that must be braced is equal to the product of the BF times the number of Trusses supported (NT). When diagonal bracing is used for bracing, the number of Trusses supported, NT, is the number of Trusses between sets of diagonal braces. P must not exceed the capacity of the bracing members, bracing nails, or any other connection.

A.3 TOP CHORD PLANE BRACING

Top Chord plane permanent bracing must be designed to resist lateral movement of the Top Chord. Rated sheathing or metal roofing and other approved materials may be used to act as the permanent Lateral Restraining if designed as a diaphragm. Where metal roofing materials are used to act as a diaphragm, the Building Designer should specify how the metal roofing is to be properly lapped and fastened to transfer the BF.

Purlin spacing must not exceed the design buckling length of the Top Chord as specified on the Truss Design Drawing, and must be adequately attached to the Top Chord. In the absence of an adequate diaphragm to prevent lateral shifting of the purlins, the Building Designer can specify permanent diagonal bracing to be attached to the underside of the Top Chord. Figure A1

¹ *Building Component Safety Information (BCSI): Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses*, Published by the Truss Plate Institute, Inc. and WTCA – Representing the Structural Building Components Industry. www.sbcindustry.com/bcsi.php

² J. Meeks, *Commentary for permanent bracing of metal plate connected wood trusses*, WTCA – Representing the Structural Building Components Industry, Madison, WI, 1999. www.sbcindustry.com

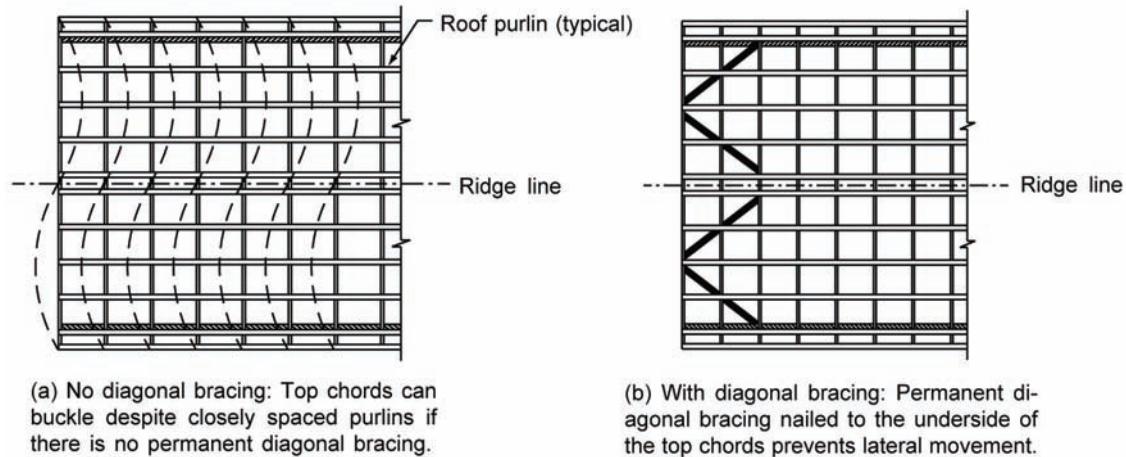


Figure A1. Use of Roof Purlins for Top Chord Permanent Bracing.

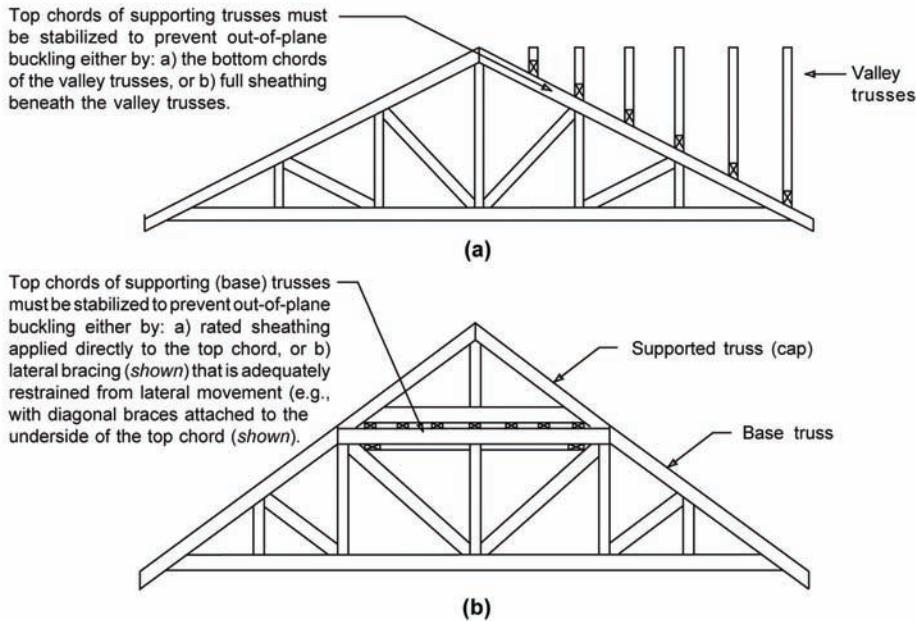


Figure A2. Special Top Chord Bracing Conditions for: (a) Valley Truss Framing and (b) Piggyback Trusses.

illustrates the necessity for applying permanent diagonal bracing in the plane of the Top Chord despite the use of closely spaced purlins.

A.4 SPECIAL TOP CHORD BRACING CONDITIONS

Special permanent bracing requirements exist when one Truss is placed on top of another, such as in the case of a valley set or piggyback Truss. The Top Chord of the lower supporting Truss, subjected to axial compression, must be stabilized to prevent out-of-plane movement (see Figure A2).

When the lower Chord of a valley set is used to stabilize the compression Chord of the supporting Truss below, the greater value of l_e/d , using panel lengths for in-plane l_e/d or using the valley set spacing adjusted for pitch (distance between points of lateral support) for out-of-plane l_e/d , will be used for the design of the affected Truss panels of the supporting Truss. Alternately, full sheathing under the valley set may be specified on the Truss Design Drawings of the supporting Trusses.

A.5 BOTTOM CHORD BRACING

Bottom Chord plane permanent bracing is required to maintain the Truss design spacing and to provide lateral

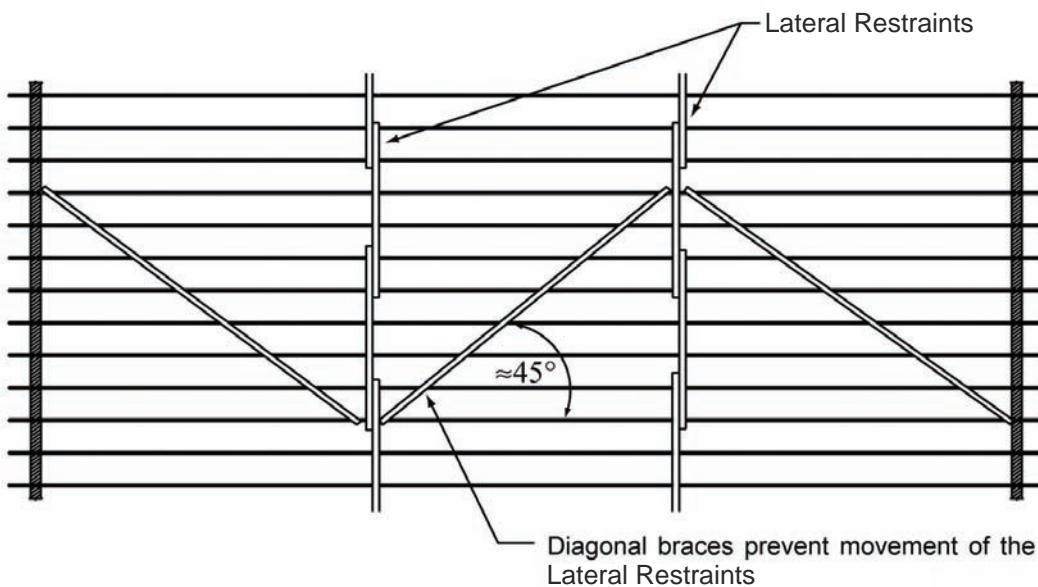


Figure A3. Permanent Bracing in the Plane of the Bottom Chord.

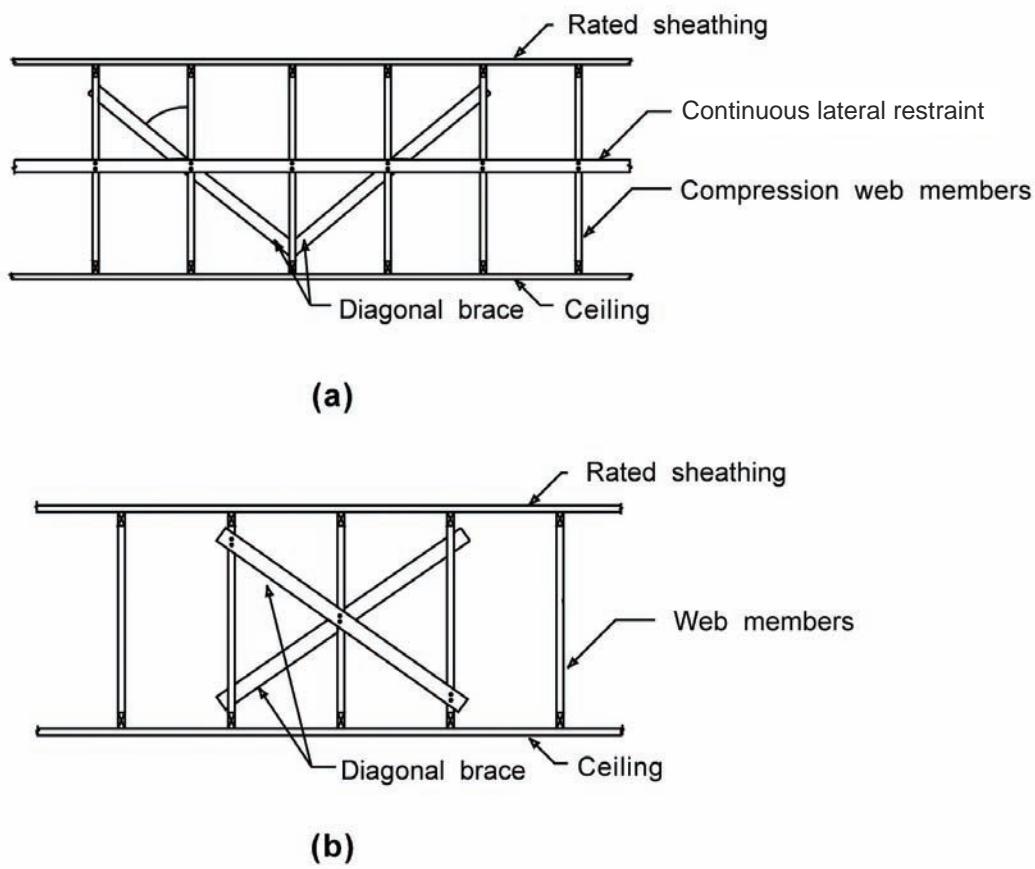


Figure A4. Permanent Diagonal Bracing in the Plane of the Web Members Serves to: (a) **Restrain the Continuous Lateral Restraint, or** (b) **Provide Additional Lateral Rigidity to the Entire Roof System When Continuous Lateral Restraint is Not Required for Any Web Members.**

support to the Bottom Chord to resist buckling forces in the event of stress reversal due to wind uplift or loading conditions that produce compression forces.

In multiple bearing Trusses or Cantilever conditions, portions of the Bottom Chord become compression members and must be restrained/braced laterally to resist buckling in the same manner as the Top Chord of simple span Trusses. Figure A3 illustrates the use of permanent diagonal bracing in combination with Lateral Restraints in the plane of the Bottom Chord.

Lateral Restraints and associated diagonal bracing may not be required if the Bottom Chords of Trusses are braced by an engineered horizontal diaphragm or gypsum board sheathing designed and attached in accordance with the requirements of *ASTM C840* (see *TPI 1* Section 1.4).

A.6 WEB MEMBER BRACING

Web member plane bracing holds the Trusses in a vertical position and maintains the design spacing. In addition, when permanent Lateral Restraints are required to shorten the buckling length of a Web member, the required location of the permanent Lateral Restraining will be specified on the Truss Design Drawing. The Lateral Restraining must be braced by permanent diagonal bracing in the plane of the Web members, or by some other equivalent means, as determined by the Building Designer [see *TPI 1* Sections 2.3.3 and 2.4.3 and Figure A4(a)]. An alternative to the Web reinforcement/bracing approach depicted in Figure A4(a) is the use of T-reinforcement formed by adding a member to the narrow edge of the Truss Web. The T-reinforcement method is useful in cases where the Webs may not line up such as in the hip ends of a hip roof or where Trusses are widely spaced making the installation of lateral and diagonal braces more difficult to install. For the case of widely spaced Trusses, T-reinforcement can be installed on the ground prior to the Truss being lifted onto the roof. An engineering design procedure for T-reinforcement based on the published allowable properties of the Truss Web and connected brace is available in the literature.³

When Permanent Individual Truss Member Restraints are not required for any of the Web members per the Truss design, permanent diagonal bracing, at intervals or continuous as specified by the Building Designer, may be required to provide additional lateral stability to the roof system [see Figure A4(b)]. Permanent diagonal bracing in the Web member plane may also serve as a means to

control deflections and/or vibration, per the discretion of the Building Designer. Permanent bracing intended to serve this purpose should comply with the provisions for strongbacking (see *TPI 1* Section 7.6.2.4).

A.7 ADDITIONAL LITERATURE

1. Hoyle, R. J., Jr., and H. C. Sorenson. Stability and bracing of parallel-chord wood trusses. *Forest Products Journal*: 36(2):74-80; 1986. www.forestprod.org
2. Truss Plate Institute. Recommended Design Specification for Temporary Bracing of Metal Plate Connected Wood Trusses. *DSB-89*. Alexandria, VA; 1989. www.tpininst.org
3. Underwood, C.R., Woeste, F. E., Dolan, J.D., Holzer, S. M. Permanent bracing design for MPC wood roof truss webs and chords. *Forest Products Journal*. Vol. 51, No. 7/8: 73-81; 2001. www.forestprod.org/fpjover.html
4. Waltz Jr., M.C., McLain, T.E., Miller, T.H., and Leichti, R.J. Discrete bracing analysis for light-frame wood-truss compression webs. *Journal of Structural Engineering*. Vol. 126, No. 9: 1086-1093; 2000. <http://scitation.aip.org/sto>
5. Zuk, W. Lateral bracing forces on beams and columns. *Journal of the Engineering Mechanics Division*, Vol. 82, No. EM 3, July; 1956.

³ D. Shrestha, D. A. Bender and F. E. Woeste, 2003, *T-Brace design for MPC wood truss webs*, *Forest Products Journal*, Vol. 53, No.1, 61-66, 2001.

**APPENDIX B
(NON-MANDATORY)
PROOF LOAD TEST FOR SITE-SELECTED TRUSSES**

INTRODUCTION

The methods and criteria for proof testing of site selected Trusses described herein are based upon the collective experience of leading personnel in the Metal Connector Plate industry, Truss Manufacturers, researchers, architects and engineers in general practice. The information presented should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability and applicability by a licensed professional engineer or architect. The publication of the material contained herein is not intended as a representation or warranty on the part of the Truss Plate Institute or any other organization named herein, that this information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use.

B.1 SAFETY CONSIDERATIONS

B.1.1 Trusses that have been subjected to proof loading shall not be used in the structure.

B.1.2 This guideline may involve hazardous materials, operations, and equipment. This guideline does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this Standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

B.2 SCOPE

This guideline is intended for those who may require information in testing the structural integrity of a Truss or Trusses that have been fabricated and selected from the site to which they have been delivered. These guidelines are not intended as material quality control guidelines, in-plant fabrication quality assurance or quality control guidelines which are provided in *ANSI/TPI 1-2007 "National Design Standard for Metal Plate Connected Wood Truss Construction"* (*TPI 1*), published by the Truss Plate Institute.

B.3 DEFINITIONS

B.3.1 Proof Tests - A proof test is made to provide assurance that the Truss designed in accordance with *TPI 1* "National Design Standard for Metal Plate Connected Wood Truss Construction" will support a stated load or to

determine the performance and structural response under a specified loading.

B.3.2 Truss Design Drawings - Engineering drawings as discussed in this Standard are defined as roof or floor Truss Design Drawings (see *TPI 1* Section 2.2) containing the engineering data outlined in *TPI 1* Sections 2.3.5.5 and 2.4.5.4.

B.3.3 Construction Documents - Construction documents are the current building design drawings as prepared by the Building Designer as defined in *TPI 1* Section 2.2.

B.3.4 Truss Placement Diagrams - Per *TPI 1*, Truss Placement Diagrams are an illustration that identifies the assumed location of each Truss. Truss Placement Diagrams are not engineering drawings prepared by the Building Designer and are not part of the Construction Documents for the project.

B.3.5 Site Selected Trusses - Site selected Trusses are defined as Trusses which have been delivered to the site and which have not yet been installed in the roof or floor location for which they were originally designed, and were selected in accordance with the requirements of Section B.5.

B.4 APPLICABLE DOCUMENTS

The following documents are recommended to be made available in order that an accurate determination can be made of the loading conditions the Trusses are intended to carry:

- (a) Truss Design Drawings**
- (b) Truss Placement Diagrams**
- (c) Construction Documents (Including Building Designer's system bracing information)**

B.5 SITE SELECTED TRUSSES FOR TESTING

B.5.1 GENERAL

Trusses shall be selected from the site and shall not have been installed.

B.5.2 FIELD BREAKAGE PRIOR TO TESTING

Trusses which have been broken and repaired shall have repair Truss Design Drawings available to verify the details of repair prior to load testing.

B.5.3 TRUSS QUALITY

Records are to be made as to the condition and quality of Trusses found prior to selection for testing.

B.6 TEST PROCEDURE FOR SITE SELECTED TRUSSES**B.6.1 TESTING SITE SELECTED TRUSSES IN PAIRS**

B.6.1.1 The test units shall be supported and attached to firm stable supports. Use of strapping or metal framing connectors is suggested to resist tipping of the units during the load application. The Top Chords shall be sheathed with 4 to 6 ft. lengths of any of the generally accepted sheathing materials (e.g., board, plywood, reconstituted panel products, etc.). In any case, each piece shall be separated at least $\frac{1}{8}$ in. A recommended practice is to use cross-bracing between the Trusses to stabilize the setup; however, the Trusses shall not be braced by tying the setup to adjacent permanent supports or other supports which may affect the free deflection of the Trusses.

B.6.1.2 Deflection readings shall be taken and recorded for each Truss as required for the purpose of the tests, and shall include vertical movement at midspan of the Truss. Deflection readings shall be permitted to be averaged between each pair of Trusses at corresponding locations.

B.6.1.3 Deflection measurements can be taken by any one of several methods. Support settlement, including wood crushing and movement at bearings shall be removed from deflection measurements.

This is best accomplished by line, scale and mirror arrangement. The line is supported on free-rolling bearings and kept under tension by means of weights. Deflection readings are taken by reading on the scale, which is attached to the Truss, adjacent to the line, adjusting the eye position so that the line covers its image on the mirror and removes parallax from the scale reading. Scale readings shall be referenced to either the top or bottom edge of the line. Dial gages shall be permitted to be used to measure deflection; however, a precise measurement must be taken of settlement at the supports. The other deflection readings are then subject to correction for settlement.

B.6.2 TESTING SITE SELECTED TRUSSES SINGLY

B.6.2.1 The site selected Trusses shall be permitted to be tested as single units in either a vertical or horizontal position in a properly designed test facility. The load shall be applied uniformly or at concentrated points along the Chords. Wood strips shall be attached along the Top Chords or frictionless brackets or attachments shall be permitted to be used to prevent the load brackets from slipping down the Top Chords on steep-sloped Trusses.

B.6.2.2 Deflection measurements shall be identical to the procedures described in Sections B.6.1.2 and B.6.1.3.

B.6.3 PROOF LOADING**B.6.3.1 TOTAL DESIGN LOAD**

Site selected Trusses used for testing shall be proof loaded to total design load conditions following this procedure.

B.6.3.2 LOADING PROCEDURE

B.6.3.2.1 Prior to conducting the load test, a preliminary load equal to the design dead load shall be applied and held for five minutes. This preliminary load shall then be removed.

B.6.3.2.2 Apply the design dead load and hold for no less than five minutes. Read and record deflection measurements at the end of this period. These deflection measurements are known as the basic dead load (BDL) position.

B.6.3.2.3 With the dead load applied, increase the load to design total load at a continuous rate not to exceed 0.1 units of design total load per minute or by step-increments of 25 percent, or less, of total design load. For continuous loading systems, read and record deflection measurements at least twice during this load period. For step-loading systems, after each incremental load is applied, hold the new load level for a period of no less than five minutes, recording deflection readings at the end of each hold period.

B.6.3.2.4 Hold the load at design total load for no less than 30 minutes. Read and record the deflections at the beginning and end of this 30-minute period. During this hold period, examine the Wood Members and metal plate connectors for signs of distress and record any such signs.

B.6.3.2.5 Remove the design live load portion of the load. For loading systems in which accurate load distribution upon unloading cannot be achieved, it shall be permitted that the design dead load be removed and immediately reimposed in order to achieve the correct design dead load distribution.

B.6.3.2.6 Hold at design dead load for up to 15 minutes. Read and record the minimum values of each deflection measurement during this period.

B.7 EVALUATION CRITERIA

B.7.1 There shall be no visible signs of distress in the Wood Members and metal plate connectors during the loading procedure.

B.7.2 The average maximum deflection at design load relative to the BDL position, for all tests of identical Trusses, shall not exceed the limiting deflection specified by *TPI 1* based on overall span, or other limiting deflection, as specified by the applicable Building Code or Building Designer, by more than 25 percent or $\frac{1}{8}$ in., whichever is greater.

B.7.3 The site selected Truss shall pass if the deflection recovery, during the loading steps in Section B.6.3.2.5 and Section B.6.3.2.6, equals or exceeds 85 percent of the maximum total deflection, relative to BDL deflection measured at the end of the load step given in Section B.6.3.2.4.

B.8 REPORT

The load testing report shall follow the format and contain the information set forth in *ASTM E 575*.

B.9 ADDITIONAL DOCUMENTS

Additional reference documents are shown below for additional information and/or for clarification on specific references.

1. *ANSI/TPI 1-2007*, National Design Standard for Metal Plate Connected Wood Truss Construction. Published by the Truss Plate Institute, Inc. www.tpinst.org

2. *ASTM E73-83(2007)*, Standard Practice for Static Load Testing of Truss Assemblies. www.astm.org

3. *ASTM E575-05*, Standard Practice for Reporting Data from Structural Tests of Building Constructions, Elements, Connections, and Assemblies.

4. *ASTM E196-06*, Standard Practice for Gravity Load Testing of Floors and Low Slope Roofs.

5. National Design Specification® for Wood Construction, *ANSI/AF&PA NDS-2005*, published by the American Forest & Paper Association. www.awc.org

6. *Building Component Safety Information (BCSI)*: Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses. Pub-

lished by the Truss Plate Institute, Inc. and WTCA – Representing the Structural Building Components Industry. www.sbcindustry.com/bcsi.php

7. Minimum Design Loads for Buildings and Other Structures, *ASCE/SEI 7-05*, published by American Society of Civil Engineers. www.asce.org

**APPENDIX C
(NON-MANDATORY)
GUIDE SPECIFICATIONS**

**SECTION 06 17 53
SHOP FABRICATED WOOD TRUSSES**

PART 1 - GENERAL

1.01 SECTION INCLUDES

- A. Design, manufacture, and supply wood Trusses as shown on the Construction Documents and as specified.

1.02 REFERENCES AND DEFINITIONS

- A. *BCSI*: Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses jointly produced by WTCA – Representing the Structural Building Components Industry and the Truss Plate Institute.
- B. *Building Designer*: Owner of the Building or the person that contracts with the Owner for the design of the Framing Structural System and/or who is responsible for the preparation of the Construction Documents. When mandated by the Legal Requirements, the Building Designer shall be a Registered Design Professional.
- C. *Building*: Structure used or intended for supporting or sheltering any use or occupancy.
- D. *Building Code*: As it applies to a Building, any set of standards set forth and enforced by a Jurisdiction for the protection of public safety.
- E. *Building Official*: Officer or other designated authority charged with the administration and enforcement of the Building Code, or a duly authorized representative.
- F. *Construction Documents*: Written, graphic and pictorial documents prepared or assembled for describing the design (including the Framing Structural System), location and physical characteristics of the elements of a Building necessary to obtain a Building Permit and construct a Building.
- G. *Contractor*: Owner of a Building, or the person who contracts with the Owner, who constructs the Building in accordance with the Construction Documents and the Truss Submittal Package. The term “Contractor” shall include those subcontractors who have a direct contract with the Contractor to construct all or a portion of the construction.
- H. *Cover/Truss Index Sheet*: Sheet that is signed and sealed, where required by the Legal Requirements, by the Truss Design Engineer, and depending on the Legal Requirements shall be permitted to contain the following information: (1) identification of the Building, including Building name and address, lot, block, subdivision, and city or county; (2) identification of Construction Documents by drawing number(s) with revision date; (3) specified Building Code; (4) computer program used; (5) roof dead and live loads; (6) floor dead and live loads; (7) wind load criteria from a specifically defined code (e.g., *ASCE 7*) and any other design loads (such as ponding, mechanical loads, etc.); (8) name, address and license number of Registered Design Professional for the Building, if known; (9) a listing of the individual identification numbers and dates of each Truss Design Drawing referenced by the Cover/Truss Index Sheet; and (10) name, address, date of drawing and license number of Truss Design Engineer.
- I. *Framing Structural System*: Completed combination of Structural Elements, Trusses, connections and other systems, which serve to support the Building's self-weight and the specified loads.
- J. *Jurisdiction*: Governmental unit that is responsible for adopting and enforcing the Building Code.
- K. *Legal Requirements*: Any applicable provisions of all statutes, laws, rules, regulations, ordinances, codes, or orders of the governing Jurisdiction.
- L. *Owner*: Person having a legal or equitable interest in the property upon which a Building is to be constructed, and: (1) either prepares, or retains the Building Designer or Registered Design Professional to prepare the Construction Documents; and (2) either constructs, or retains the Contractor to construct the Building.
- M. *Registered Design Professional*: Architect or engineer, who is licensed to practice their respective design profession as defined by the Legal Requirements of the Jurisdiction in which the Building is to be constructed.
- N. *ANSI/TPI 1-2007*: National Design Standard for Metal Plate Connected Wood Truss Construction (*TPI 1*).
- O. *Structural Element*: Single structural member (other than a Truss) that is specified in the Construction Documents.

- P.** *Truss:* Individual metal-plate-connected wood component manufactured for the construction of a Building.
- Q.** *Truss Design Drawing:* Written, graphic and pictorial depiction of an individual Truss that includes the information required in *TPI 1* Sections 2.3.5.5 and 2.4.5.4.
- R.** *Truss Design Engineer:* Person who is licensed to practice engineering as defined by the Legal Requirements of the Jurisdiction in which the Building is to be constructed and who supervises the preparation of the Truss Design Drawings.
- S.** *Truss Designer:* Person responsible for the preparation of the Truss Design Drawings.
- T.** *Truss Manufacturer:* Person engaged in the fabrication of Trusses.
- U.** *Truss Placement Diagram:* Illustration identifying the assumed location of each Truss.
- V.** *Truss Submittal Package:* Package consisting of each individual Truss Design Drawing, and, as applicable, the Truss Placement Diagram, the Cover/Truss Index Sheet, Lateral Restraint and Diagonal Bracing details designed in accordance with generally accepted engineering practice, applicable *BCSI* defined Lateral Restraint and Diagonal Bracing details, and any other structural details germane to the Trusses.

1.03 DESIGN REQUIREMENTS

Trusses shall be designed in accordance with this Specification and where any applicable design feature is not specifically covered herein, design shall be in accordance with the applicable provisions of the latest edition of *TPI 1* and the American Forest & Paper Association's (AF&PA) *National Design Specification® (NDS®) for Wood Construction* and all applicable legal requirements.

1.04 SUBMITTALS

- A.** Truss Manufacturer shall furnish Truss Design Drawings prepared in accordance with all applicable legal requirements.
- B.** If required by the Construction Documents and the Truss Manufacturer's contract, the Truss Manufacturer shall furnish a Truss Placement Diagram which shall provide at a minimum the location assumed for each Truss based on the Truss Manufacturer's interpretation of the Construction Documents.
- C.** Where required by the Truss Manufacturer's contract, any local building official or applicable legal requirements, the Truss Manufacturer shall

submit the Truss Submittal Package to the Building Designer and/or the local Building Official for review and approval prior to the manufacturing of the Trusses.

- D.** The Truss Design Drawings shall include, at a minimum, the information specified below (per *TPI 1* Sections 2.3.5.5 and 2.4.5.4):
 - 1.** Building Code used for Design, unless specified on Cover/Truss Index Sheet.
 - 2.** Slope or depth, span and spacing.
 - 3.** Location of all joints and support locations.
 - 4.** Number of plies if greater than one.
 - 5.** Required bearing widths.
 - 6.** Design loads as applicable, including:
 - Top Chord live load (for roof Trusses, this shall be the controlling case of live load or snow load);
 - Top Chord dead load;
 - Bottom Chord live load;
 - Bottom Chord dead load;
 - Additional loads and locations;
 - Environmental Load Design Criteria (wind speed, snow, seismic, and all applicable factors as required to calculate the Truss loads); and
 - Other lateral loads, including drag strut loads.
- 7.** Adjustments to Wood Member and Metal Connector Plate design values for conditions of use.
- 8.** Maximum reaction force and direction, including maximum uplift reaction forces where applicable.
- 9.** Metal Connector Plate type, manufacturer, size, and thickness or gauge, and the dimensioned location of each Metal Connector Plate except where symmetrically located relative to the joint interface.
- 10.** Size, species and grade for each Wood Member.
- 11.** Truss-to-Truss connection and Truss field assembly requirements.
- 12.** Calculated span to deflection ratio and/or maximum vertical and horizontal deflection for live and total load and K_{CR} (creep factor) as applicable.

13. Maximum axial tension and compression forces in the Truss members.
 14. Fabrication tolerance per *TPI 1* Section 6.4.10.
 15. Required Permanent Individual Truss Member Restraint location and the method of Restraint/Bracing to be used per *TPI 1* Sections 2.3.3 and 2.4.3.
- E.** Truss Submittals and any supplementary information provided by the Truss Manufacturer shall be provided by the Contractor to the individual or organization responsible for the installation of the Trusses.

PART 2 - PRODUCTS

2.01 MATERIALS

A. LUMBER:

1. Lumber used shall be identified by grade mark of a lumber inspection bureau or agency approved by the American Lumber Standards Committee, and shall be the size, species, and grade as shown on the Truss Design Drawings, or equivalent as approved by the Truss Design Engineer/ Truss Designer.
2. Adjustment of value for duration of load or conditions of use shall be in accordance with *NDS*.
3. Fire retardant treated lumber, if applicable, shall meet the specifications of the fire retardant chemical manufacturer, *TPI 1*, and the Truss design. The fire retardant treated lumber shall be re-dried after treatment in accordance with the American Wood-Preservers' Association (AWPA) Standard *C20 Structural Lumber – Fire Retardant Treatment by Pressure Processes*. Allowable values must be adjusted in accordance with *NDS*. Lumber treator shall supply certificate of compliance.

B. METAL CONNECTOR PLATES:

1. Metal Connector Plates shall be manufactured by a Truss Plate Institute (TPI) member plate manufacturer and shall comply with *TPI 1*. Working stresses in steel are to be applied to Effectiveness Ratios for plates in accordance with the Standard.
2. [This section not applicable unless the blank space has been completed with information regarding the corrosive environment.] Due to exposure to a highly corrosive environment consisting of _____, special applied coatings

per *TPI 1* Chapter 6 or stainless steel shall be required.

3. At the request of the Building Designer, the plate manufacturer shall furnish a certified record that materials comply with steel specifications.

2.02 FABRICATION

- A.** Trusses shall be manufactured to meet the quality requirements outlined in *TPI 1* Chapter 3 and in accordance with the information provided in the final approved Truss Design Drawings.

PART 3 - EXECUTION

3.01 HANDLING, INSTALLING, RESTRAINING AND BRACING

- A.** Trusses shall be handled during manufacturing, delivery and by the Contractor at the job site so as not to be subjected to excessive bending.
- B.** Trusses shall be unloaded in a manner so as to minimize lateral strain. Trusses shall be protected from damage that might result from on-site activities and environmental conditions. Trusses shall be handled in such a way so as to prevent toppling when banding is removed.
- C.** Contractor shall be responsible for the handling, installation, and temporary restraint/ bracing of the Trusses in a good workmanlike manner and in accordance with the recommendations set forth in the latest edition of *BCSI*.
- D.** Apparent damage to Trusses, if any, shall be reported to Truss Manufacturer prior to erection.
- E.** Trusses shall be set and secured level and plumb, and in correct location. Each Truss shall be held in correct alignment until specified permanent restraint and bracing is installed.
- F.** Cutting and altering of Trusses is not permitted. If any Truss should become broken, damaged, or altered, written concurrence and approval by the Truss Manufacturer and Building Designer is required.
- G.** Concentrated loads shall not be placed on top of Trusses until all specified restraint and bracing has been installed and decking is permanently nailed in place. Specifically avoid stacking full bundles of plywood or other concentrated loads on top of Trusses.
- H.** Trusses shall be permanently restrained and braced in a manner consistent with good building practices as outlined in *BCSI* and in accordance with the requirements of the Construction Documents.

Trusses shall furthermore be anchored or restrained to prevent out-of-plane movement so as to keep all Truss members from simultaneously buckling together in the same direction. Such permanent Lateral Restraint shall be accomplished by including permanent diagonal bracing in the plane of the Lateral Restraints or other suitable means.

- I.** Materials used in temporary and permanent restraint and bracing shall be furnished by Contractor.

**APPENDIX D
(NON-MANDATORY)**

ABBREVIATIONS, TERMINOLOGY, DEFINITIONS & NOMENCLATURE

D.1 ABBREVIATIONS

AISI	American Iron and Steel Institute 1140 Connecticut Ave., NW Suite 705 Washington, D.C. 20036 www.steel.org
AITC	American Institute of Timber Construction 7012 S. Revere Pkwy Suite 140 Centennial, CO 80112 www.aitc-glulam.org
ALSC	American Lumber Standards Committee P.O. Box 210 Germantown, MD 20874 www.alsc.org
ANSI	American National Standards Institute 1819 L Street, NW 6 th Floor Washington, DC 20036 www.ansi.org
ASTM	ASTM International 100 Barr Harbor Drive PO Box C700 West Conshohocken, PA 19428 www.astm.org
BCSI	Building Components Safety Information: <i>Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses</i> jointly produced by WTCA – Representing the Structural Building Components Industry and the Truss Plate Institute.
NDS[®]	National Design Specification for Wood Construction
AF&PA	American Forest & Paper Association 1111 19th St. NW Suite 800 Washington, D.C. 20036 www.afandpa.org

D.2 DEFINITIONS (SEE ALSO TPI 1 CHAPTERS 1 AND 2)

D.2.1 TRUSS TERMS

Anchorage - Connection between the roof or floor framing members (e.g., Trusses, bracing, etc.) and the building structure, which is required to transfer the forces from these members into the building.

Beam Pocket - Void or cut-out built into Truss to allow beam support.

Bearing - Structural support, usually a beam or wall that is designed by the Building Designer to carry the Truss reaction loads to the foundation.

Built-Up Beam - Single component composed of two or more Wood Members fastened together to serve as a bending member with greater load carrying capability as well as lower deflection than by the sum of the capacities of the individual Wood Members (e.g., garage door, stairwell and fireplace headers).

Butt Cut - Vertical cut at outside edge of a Truss bottom, typically $\frac{1}{4}$ in.

Camber - Upward curvature built into a Truss Bottom Chord to compensate for deflection due to loading conditions.

Center Bearing Truss - Truss with structural support at center of Truss span as well as at heel points.

Chase Opening - Rectangular opening in a floor or sloping flat Truss for the purpose of running utilities through it, such as heating and air conditioning ducts.

Clear Span (Clear Opening) - Indicates the inside or interior frame-to-frame dimensions. Not to be confused with Span (Nominal Span) as defined below.

Cutting Sheets (Cut Lists) - Diagram of lumber lengths and angles of cut for Truss Web members and Chords.

Dimension Lumber - Type of lumber from nominal 2 in. through 4 in. thick and 2 in. or wider.

Dual Pitch Truss - Truss that changes pitch at the Top Chord peak joint.

Fascia - Trim board applied to ends of Overhang.

Gable End Frame - A component manufactured to complete the end wall of a building. The Bottom Chord of the gable end frame is continuously supported by the end bearing wall. Verticals between the Top and Bottom Chords are typically spaced at 24 in. on center. The verticals function as load carrying members and as attachment members for sheathing or other end wall coverings. The gable end frame must be incorporated into the end shear wall by the Building Designer.

Girder Truss - Truss designed to carry heavy loads from other structural members framing into it. Usually a multiple-ply Truss.

Grade - Designation of the quality of a manufactured piece of lumber with respect to its load-carrying ability.

Header - Structural member located above wall openings or at end(s) of openings wider than joist, rafter, or Truss spacing and serving to transfer load to studs, joists, rafters, or Trusses adjacent to the openings.

Heel - Point on the Truss where the Top and Bottom Chords intersect.

Heel Cut - See Butt Cut.

Hip Roof - Roof system in which the slope of the roof at the end walls of the building is perpendicular to the slope of the roof along the sides of the building.

Hip Set - Series of step down Trusses of the same span and Overhang, that decrease in height to form the end slope of a hip roof system.

Let Tails Run - When a piece of lumber making up the Top Chord is not cut off to a specified Overhang length, but retains the length of the lumber used for the purpose of meeting specific Overhang requirements in the field.

Level Return - Lumber filler placed horizontally from the end of an Overhang to the outside wall to form a soft-fit.

Monopitch Truss - Truss that has a single Top Chord, pitch (slope) greater than 1.5/12.

Multi-Ply Truss - A Truss designed to be installed as an assembly of two or more individual Trusses fastened together to act as one. Ply-to-ply connections of multi-ply Trusses are specified on the Truss Design Drawing.

Nail-On Plate - Metal Connector Plates without integral Teeth but with pre-punched holes or identifying marks through which nails are driven by hand or pneumatic means into the lumber. Their typical use is in repairs.

Nominal Size - As applied to products such as lumber, traditionally the approximate rough-sawn commercial size by which it is known and sold in the market. Actual rough-sawn sizes may vary from the nominal. Reference to standards or grade rules is required to determine nominal/actual finished size relationships, but dry dimension lumber usually is intended to be a half inch smaller than nominal dimensions up to 6 in. and $\frac{3}{4}$ in. smaller than nominal dimensions beyond 6 in.

Overall Truss Depth - Vertical distance between bearing and the uppermost point of the peak.

Panel - Chord segment defined by two adjacent panel points.

Panel Length - Distance between the centerlines of two consecutive panel points along the Top or Bottom Chord.

Panel Point - Location on a Truss where the Web members and Top or Bottom Chords intersect and are connected by Metal Connector Plates.

Parallel Chord Truss (PCT) - Truss with matching Top and Bottom Chord slopes and Trusses with flat (zero pitch) Top and Bottom Chord slopes less than 1.5/12.

Peak - Point on the Truss where the sloped Chords meet.

Penny - Designation for common nail length, abbreviated by "d"; for example 10d indicates a 10 penny nail length. Typical designations and corresponding lengths are 8d (2.5 in.), 10d (3 in.), and 16d (3.5 in.).

Piggyback Truss - Truss made in two pieces usually consisting of a hip type Truss with a triangular cap fastened to it. Designed when shipping or manufacturing constraints limit overall Truss height.

Pitch - Incline of the roof described as inches of rise or vertical change over inches of run or horizontal dimension (e.g., 5/12 is 5 in. of rise over 12 in. of run).

Plumb Cut - Top Chord end cut to provide for vertical (plumb) installation of fascia.

Purlin - Horizontal member attached perpendicular to the Truss Chord that serves to laterally restrain the Chord and/or support other materials (e.g., corrugated roofing or plywood and shingles).

Scissors Truss - Dual pitch, triangular Truss with dual pitched Bottom Chords.

Setback - Distance from the outside edge of the wall exclusive of veneer to the face of a girder Truss.

Set-up - Manufacturing term for the equipment configuration used to make Trusses of a specific design.

Slope - See Pitch.

Soffit - Level return or underside of an Overhang or Truss Cantilever end.

Span (Nominal Span) - Horizontal distance between outside edges of exterior bearings.

Splice - Location at which two Chord members are joined together end-to-end to form a single member. It may occur at a panel point or between panel points.

Split Truss - Truss used where an opening for a fireplace or other purpose intersects the Truss span in a run of otherwise similar Trusses. A split Truss can be defined also as a stub Truss or as a monopitch Truss.

Square Cut - End of Top Chord perpendicular to the slope of the member. Cut made at 90° to the length of the member.

Structural Building Components - Specialized structural building products designed, engineered and manufactured under controlled conditions for a specific application. Examples are wood or steel roof Trusses, floor Trusses, floor panels, wall panels, I-joists, or engineered beams and headers.

Stub Truss - Truss that is shortened in length but maintains the original profile.

Symmetrical Truss - Truss with the same configuration of members, occurring on each side of Truss centerline.

Truss Heel Height - The vertical depth of the Truss at the outside face of bearing.

Truss Spacing - On-center distance between Trusses.

Valley Set - Set of triangular components used to frame the shape of dormers and to complete the roof framing where Trusses intersect at perpendicular corners.

D.2.2 DESIGN TERMS

Axial Force - Push (compression) or pull (tension) acting along the length of a member. Usually measured in pounds, kips (1,000 lbs.), tons (2,000 lbs.) or the metric equivalents.

Axial Stress - Axial force acting at a point along the

length of a member, divided by the cross sectional area of the member (usually measured in lbs. per square in.).

Bending Moment - Measure of the bending force.

Bending Stress - Force per square inch of area acting at a point along the length of a member resulting from the bending moment applied at that point. Usually measured in pounds per square inch (psi) or metric equivalent.

Combined Stress - Combination of axial and bending stresses acting on a member simultaneously, such as occurs in the Top Chord (compression + bending) or Bottom Chord (tension + bending) of a Truss.

Concentrated Load - Loading applied at a specific point, such as a load-bearing wall running perpendicular to a Truss, or a roof-mounted A/C unit supported by a Truss.

Dead Load - Any permanent load such as the weight of the Truss itself, purlins, sheathing, roofing and ceiling.

Deflection - Amount a member sags or displaces under the influence of forces.

Design Loads - Dead and live loads for which a Truss is designed to support.

Duration of Load - Duration of stress or the time during which a load acts on a member. In wood, a design consideration for modifying allowable stresses, based on the accumulated loadings anticipated in the life of a structure.

Live Load (roof or floor) - Loads produced by the occupancy of the building or structure, not including dead loads or other types of live loads including construction loads or environmental loads such as wind loads, snow loads, rain loads, earthquake loads, or flood loads. A distinction is made between dead load, roof or floor live loads, and environmental live loads due to how they are considered by engineering design standards. Live loads, other than environmental, on a roof are those produced during maintenance by workers, equipment and materials, and during the life of a structure by people and moveable objects.

Reaction - Total load transmitted by a framing member to its bearing.

Short Term Increase - Increase allowed for design loads of short duration.

Stress - Force per unit of area.

Uniform Load - Load that is equally distributed over a given length, usually expressed in pounds per lineal foot

(plf), or a given area, usually expressed in pounds per square foot (psf).

D.2.3 QUALITY TERMS

S-Dry Lumber - Lumber which has been seasoned or dried to a moisture content of 19 percent or less at time of surfacing. Dry lumber is identified on the approved grade stamp with one of the following designations: KD-15, KD-19, S-Dry, MC-15.

S-Green Lumber - Lumber having a moisture content in excess of 19 percent at time of surfacing. Green lumber is identified on the approved grade stamp with the designation S-Grn.

Wet Lumber - Lumber at time of plate embedment that exhibits moisture contents in excess of 19 percent. Metal plate connector properties in lateral resistance shall be reduced in accordance with *TPI 1*, Section 6.4.6 to account for expected lumber shrinkage.

D.3 TERMINOLOGY

D.3.1 4x2 TRUSS

4x2 Trusses are usually used in floor Truss design, and sometimes in roof Truss design. These Trusses usually have parallel Chords and Webs of 2x4 (1.5-in. by 3.5-in.) lumber oriented so the major cross-sectional axis of the lumber is horizontal so the Trusses are 3.5 in. thick. This type of Truss is often manufactured with duct chase openings so wiring, utilities, piping, air conditioning and/or heating ducts can be run between the Chords of the Truss.

D.3.2 2x_n TRUSS

Generally used in roof Truss design. These Trusses have the minor cross-sectional axis of the lumber members oriented horizontally so the Trusses are 1.5 in. thick.

Parallel Chords

The slope can be built into the Truss or drainage can be provided by using different opposite side wall heights to slope the Truss. Roof slopes should be at least $\frac{1}{4}$ in. per foot of span. Common web configurations include Pratt, Howe and Warren.

Other Chord Geometry

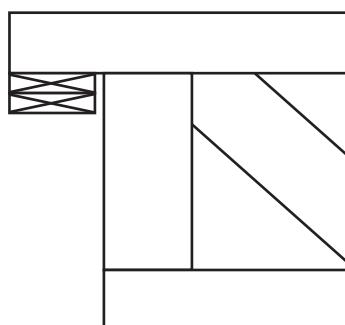
Configurations will vary depending on the loading conditions and roof or ceiling slopes required. Common configurations are shown in the following pages. Note that Trusses with pitched Bottom Chords may show relatively large horizontal movement occurring at the bearing points under load. The Building Designer must consider this movement and design the supporting structure and

Truss to wall connection accordingly.

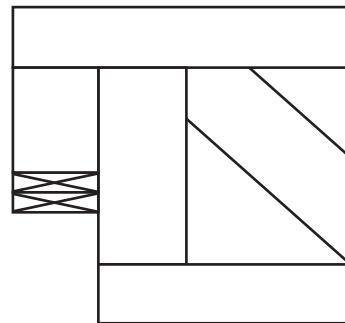
Common configurations of the above Trusses are shown in the following pages.

D.3.3 BEARING CONDITION

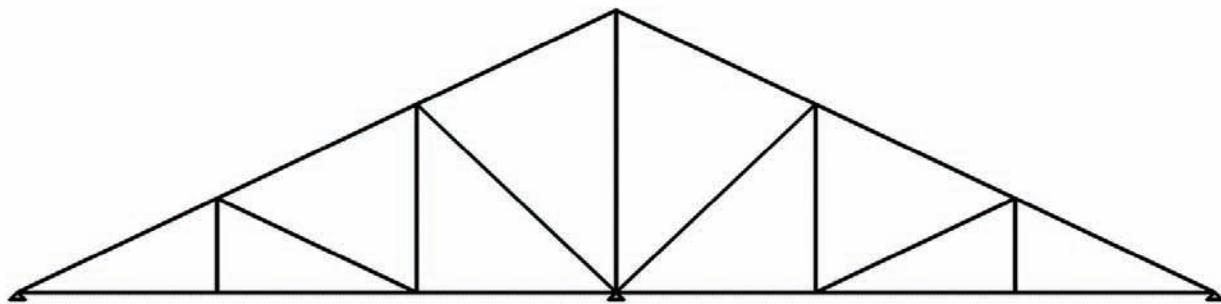
The location on the Truss where the bearing contacts the Truss can vary and may be the Bottom Chords, intermediate height (member other than Top or Bottom Chord), or Top Chord locations. The location and number of bearing points influences the configuration of Trusses. Several examples are shown below.



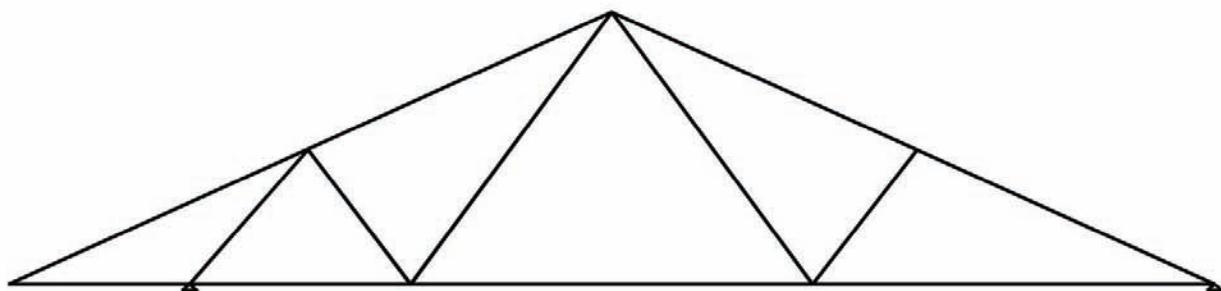
Top Chord Bearing



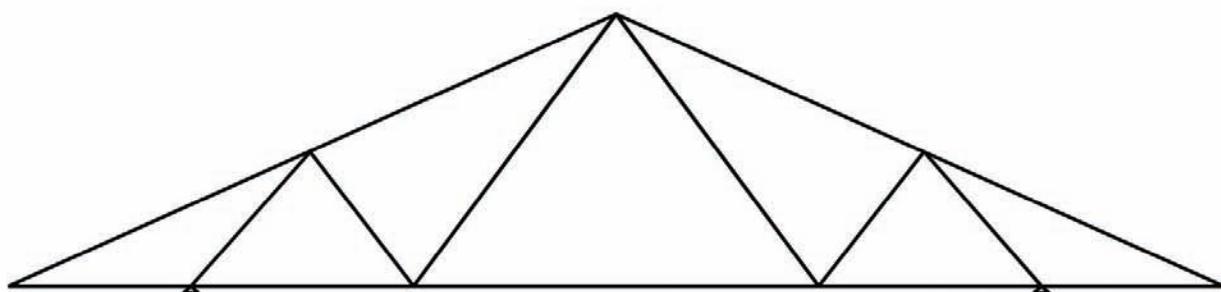
Intermediate Height Bearing



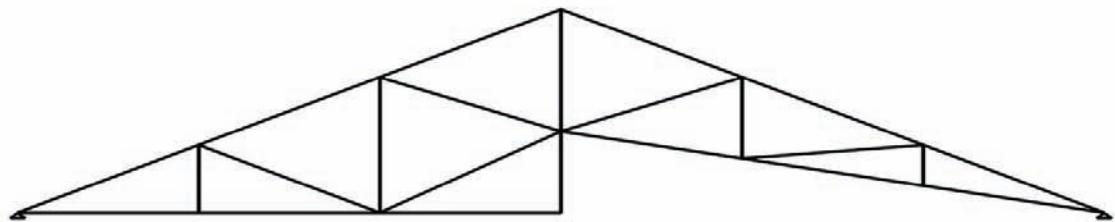
Tri-Bearing



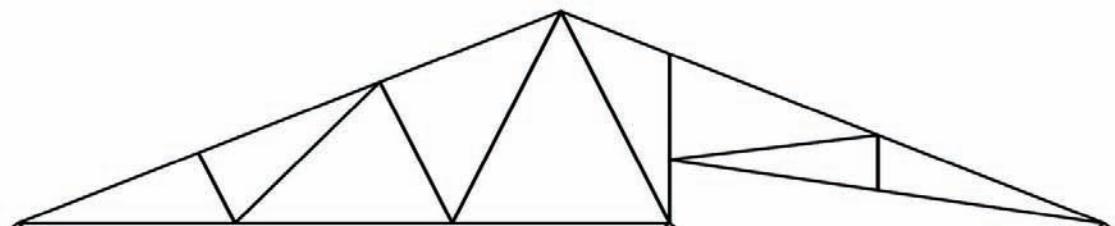
Cantilever



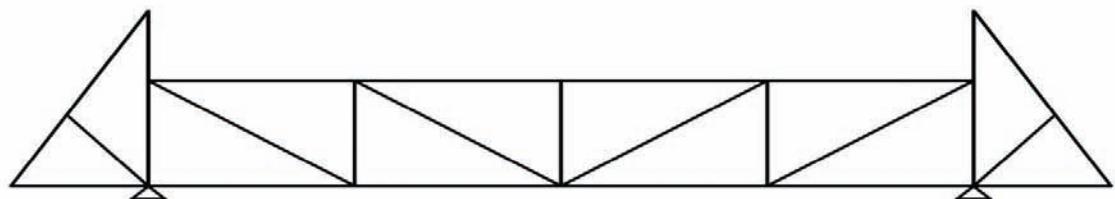
Double Cantilever



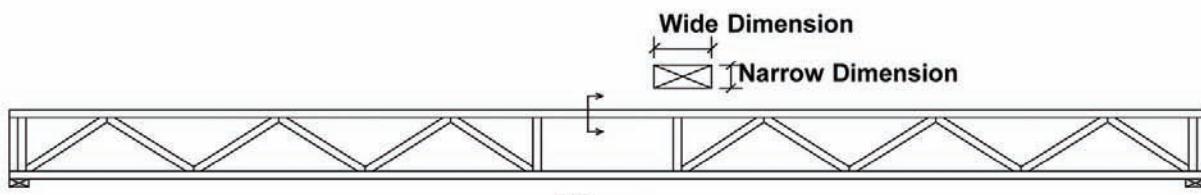
Vaulted Ceiling (Studio) With Two Bearing Points



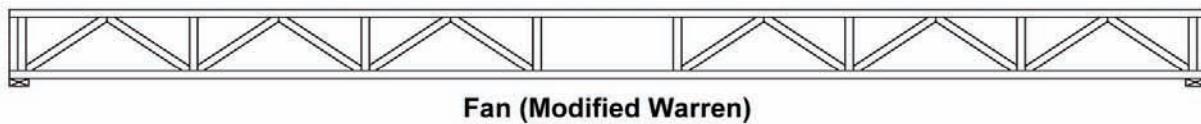
Vaulted Ceiling (Studio) With Three Bearing Points



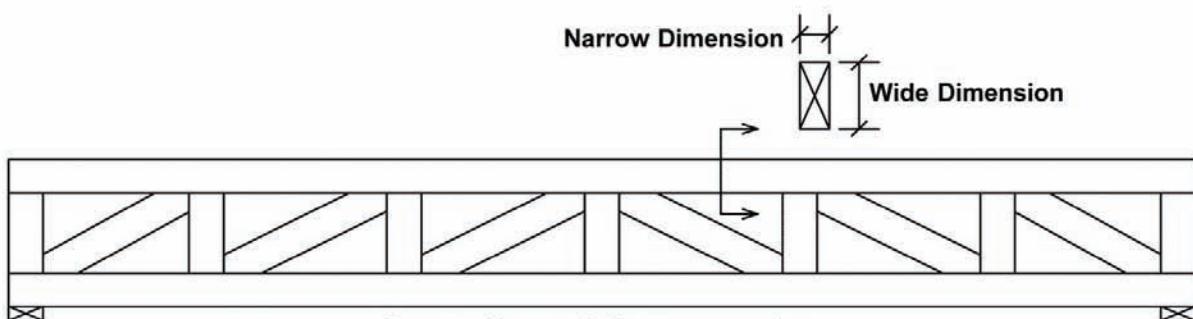
Double Cantilever With Parapets



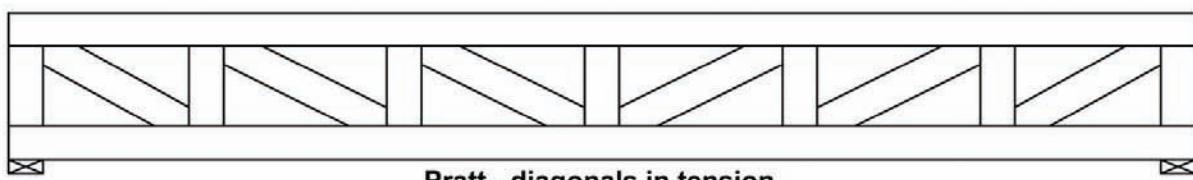
Warren



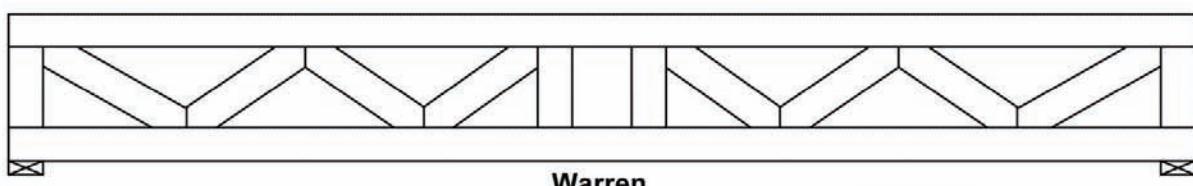
Fan (Modified Warren)



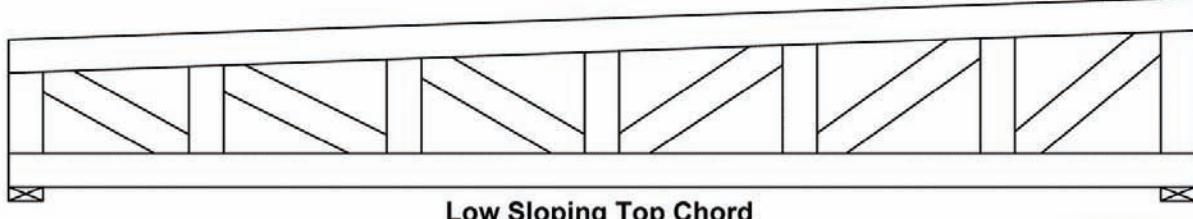
Howe - diagonals in compression



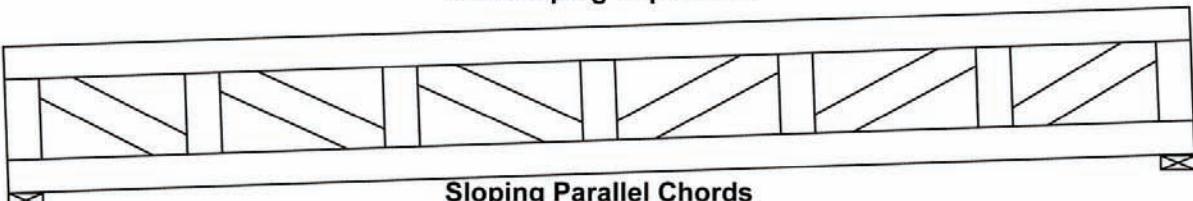
Pratt - diagonals in tension



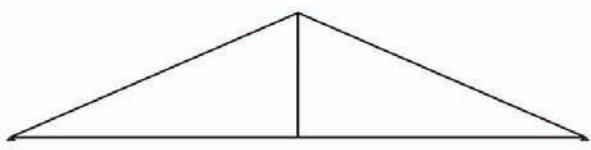
Warren



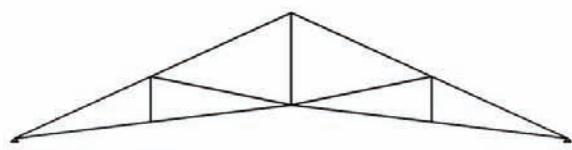
Low Sloping Top Chord



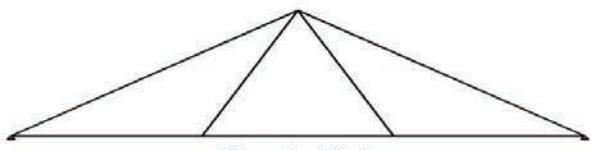
Sloping Parallel Chords



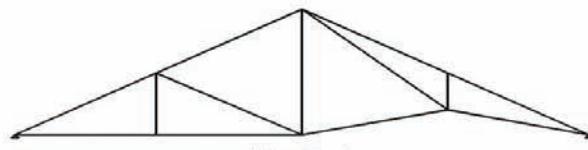
King Post



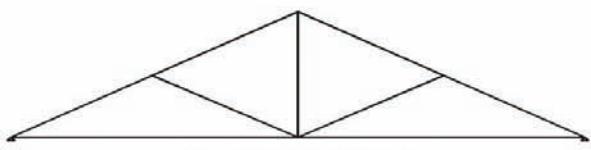
Howe Scissors



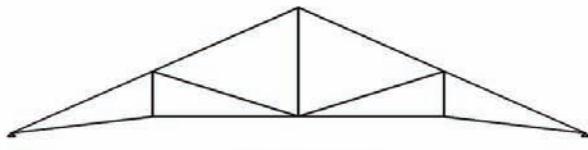
Simple Fink



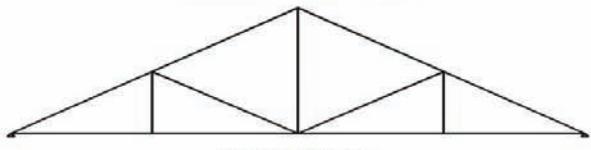
Vaulted



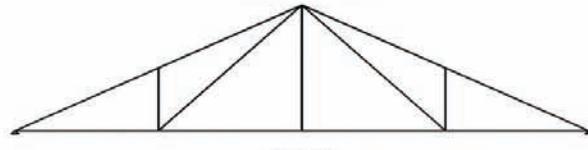
Queen Post (Fan)



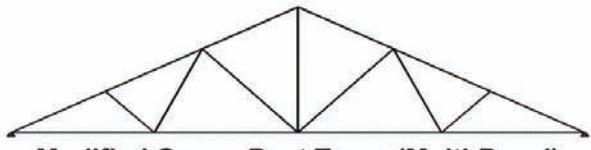
Cambered



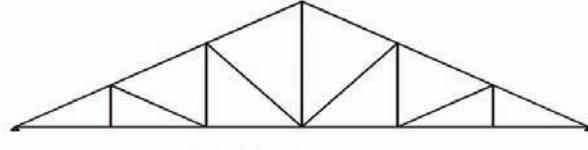
Howe Truss



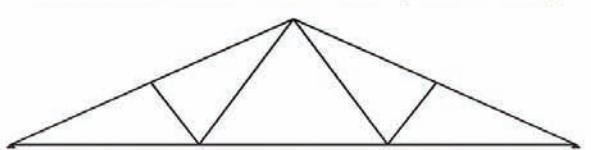
Pratt



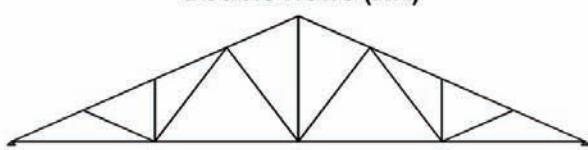
Modified Queen Post Truss (Multi-Panel)



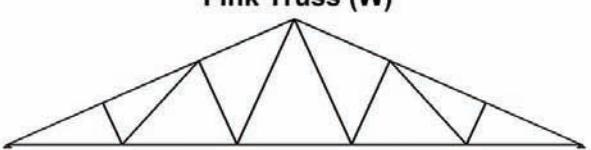
Double Howe (KK)



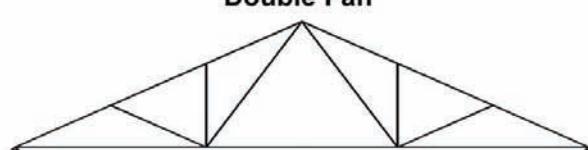
Fink Truss (W)



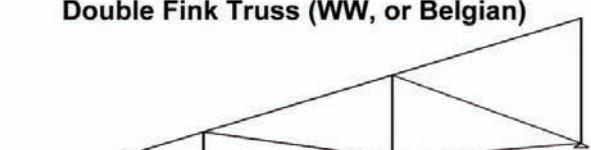
Double Fan



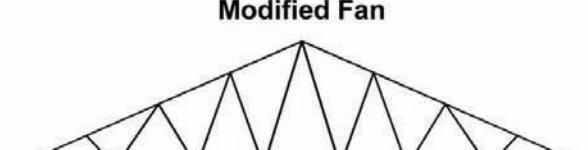
Double Fink Truss (WW, or Belgian)



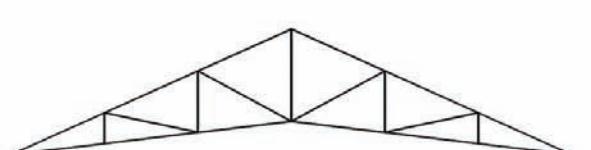
Modified Fan



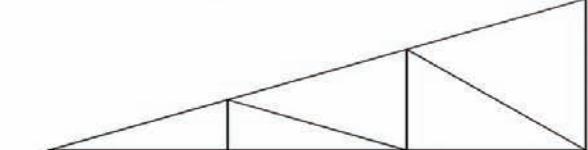
Scissors Mono



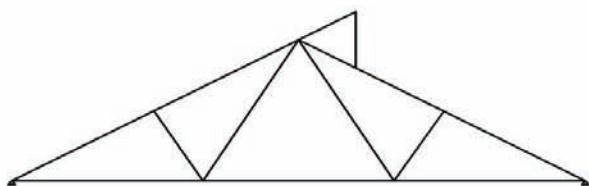
Triple Fink (WWW)



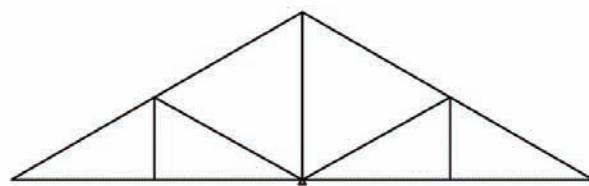
Double Howe Scissors



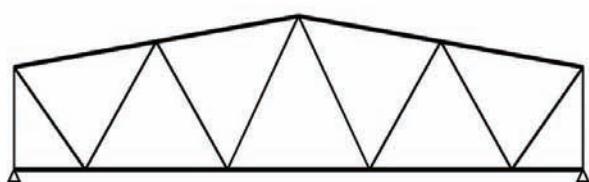
Mono



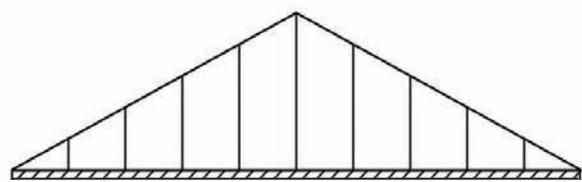
Clear Story (Clerestory)



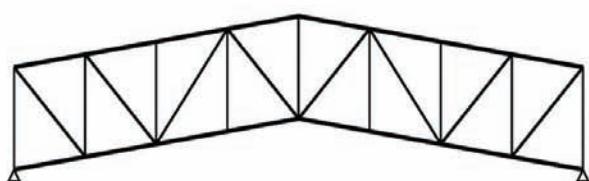
Umbrella Truss



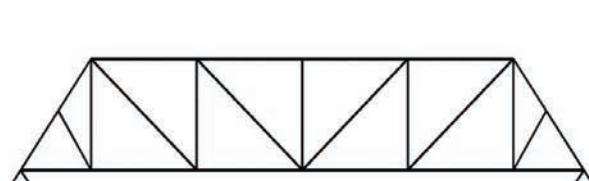
Pitched Warren Truss



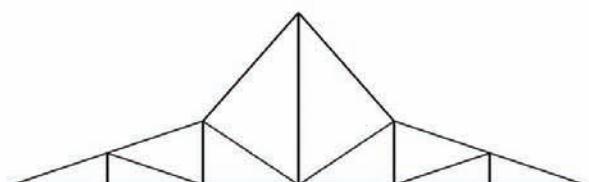
Gable End



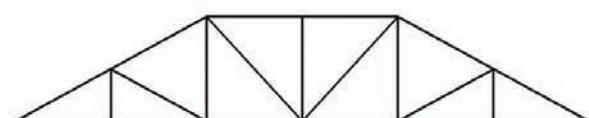
Scissored Warren Truss



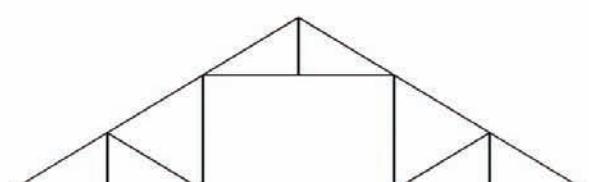
Mansard



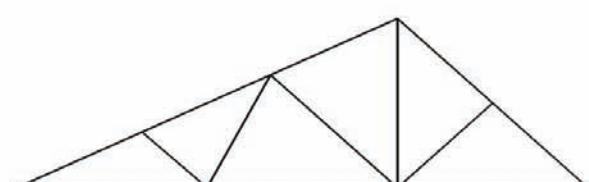
Polynesian (Duo Pitch)



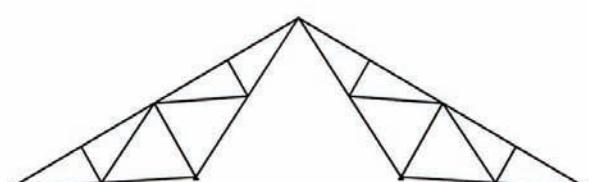
Hip



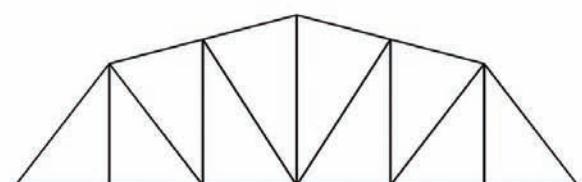
Room-in-Attic



Dual Slope (Double Pitch)

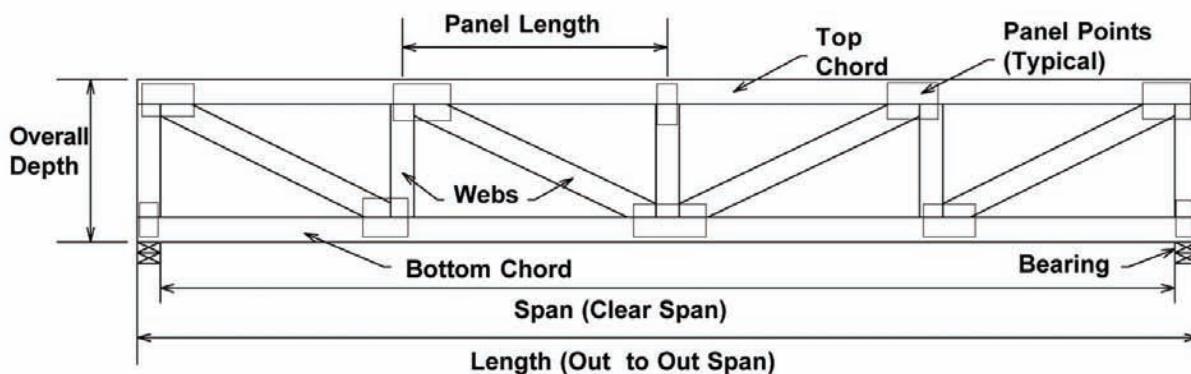


Double Inverted

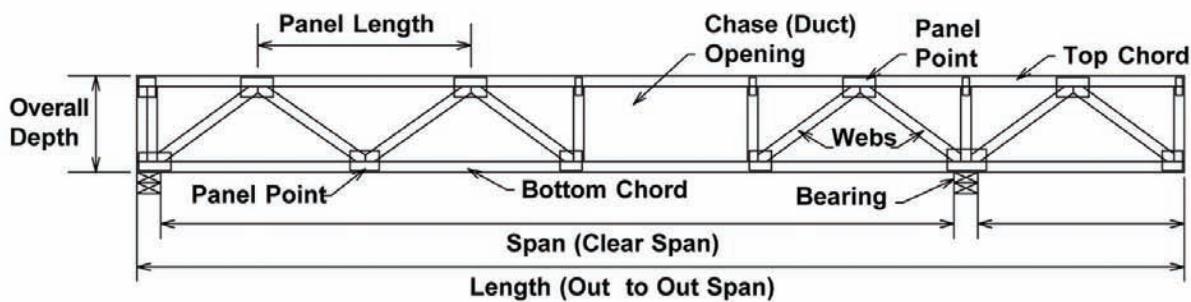


Gambrel

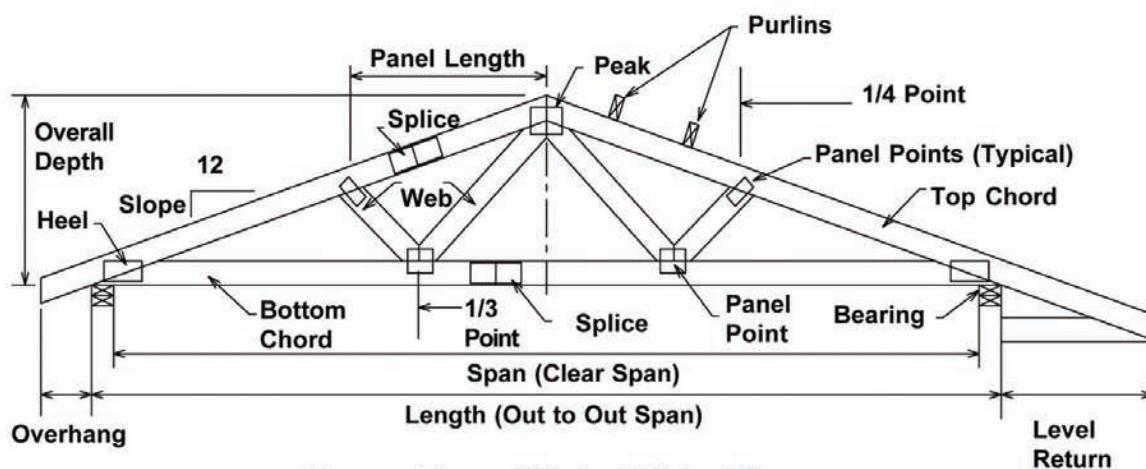
D.3.4 TRUSS NOMENCLATURE



Nomenclature of Typical Parallel Chord Truss With Lumber Oriented Vertically.



Nomenclature of Typical Parallel Chord Truss With Lumber Oriented Horizontally.



Nomenclature of Typical Pitched Truss

**APPENDIX E
(NON-MANDATORY)**

**TPI 2-2002: STANDARD FOR TESTING
METAL PLATE CONNECTED WOOD TRUSSES**

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INTRODUCTION

Testing procedures in this document are intended solely to provide information to evaluate the load carrying capacity of a specific Truss design for Trusses manufactured in conformance to *ANSI/TPI 1, National Design Standard for Metal Plate Connected Wood Truss Construction*, and should not be construed as to be a compulsory part of *TPI 1*. Testing procedures contained in this document are not intended to establish design values. This document is not intended to verify a particular design methodology, nor to endorse similar designs by extrapolating test results from one particular combination of lumber, plates, and geometry.

It is the sole responsibility of the user to apply the criteria in this document. The Truss Plate Institute and the metal-plate-connected wood Truss industry at large expressly disclaim any liability arising from the use, application or reference to the present document.

ACKNOWLEDGEMENT

TPI 2-2002: Standard for Testing Metal Plate Connected Wood Trusses was developed through a consensus process following TPI's Project Committee Method to Achieve Consensus (PCMAC). The publication has been approved by the Truss Plate Institute's Project Committee (PC). *TPI 2-2002* is a revision of the earlier *ANSI/TPI 2-1995* edition that was made with regard to the 2002 edition of the *TPI 1* standard. It should be pointed out,

however, that final submission of this edition to ANSI was not pursued. TPI would like to acknowledge the efforts of its Technical Advisory Committee and its Project Committee for their many hours of work in developing this document. Thanks also go out to those participating in the call for public comment. TPI's PC approval of this document does not necessarily imply that all PC members voted for its approval. At the time the PC approved *TPI 2-2002*, the PC had the following membership.

Stu Lewis	Alpine Engineering
Bob Shupe	Tee-Lok
Brad Cameron	Keymark
David Gromala	Weyerhaeuser
Frank Woeste	Virginia Tech
Gary Sweatt	Sweatt Engineering
Hamid Rashti	Schuck Component Systems, Inc.
James Kainz	Applied Research Associates, Inc.
Kirk Grundahl	Qualtim, Inc.
Marvin Strzyzewski	Tee-Lok
Phil O'Regan	Robbins Engineering
Rachel Smith	WTCA
Rakesh Gupta	Oregon State University
Robert Emerson	Oklahoma State University
Scott Lockyear	AF&PA
Steve Cabler	MiTek
Steven Cramer	University of Wisconsin-Madison
Ted Kolanko	Cherokee Metal

E.1 GENERAL

E.1.1 This Standard provides procedures for testing and evaluating wood Trusses designed and manufactured with Metal Connector Plates in accordance with *TPI 1*. This Standard is intended to define the minimum limits for performance expected from such Trusses. Satisfactory compliance with this Standard is not, by itself, sufficient to imply adequacy of any Truss or Truss design. This Standard is not intended to be used in lieu of *TPI 1* or other applicable Truss design criteria. Destructive load tests of full scale Trusses in accordance with these procedures provide a means of demonstrating that minimum adequate performance is obtainable from specific Metal Connector Plates, various lumber types and grades, a particular Truss design, or particular manufacturing procedures.

E.1.2 These are general procedures and their provisions are not appropriate for all Trusses or testing objectives.

In such cases, alternate provisions shall be specified. The information presented shall not be used or relied upon for a specific application without competent professional review by a licensed professional engineer who has had experience with design standards for metal-plate-connected wood Trusses.

E.2 SAFETY

E.2.1 This Standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this Standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

E.2.1.1 Testing Trusses is hazardous. Personnel shall be kept out of contact from and out from underneath Trusses under load. Safety procedures for such tests shall address the adequacy of the Truss supports, connections of the Truss to its supports, lateral bracing of the Truss and its members, protection of personnel and equipment, and prevention of catastrophic Truss failure.

E.2.1.2 Testing Trusses under gravity load is extremely dangerous. Great care shall be exercised to laterally stabilize the system and to protect personnel loading the system from the movement of the units of mass from vertically or horizontally acting failures.

E.3 SIGNIFICANCE AND USE

E.3.1 Conformance to procedures presented in this Standard provides a common basis for testing and evaluation of test results. This Standard is intended to provide assurance that Truss strength and stiffness performance are adequate to meet load requirements for their specific application. These procedures are intended to evaluate Truss strength and stiffness when subjected to a static load test.

E.3.2 It is the responsibility of the user of this Standard to verify that the testing and evaluation procedures provided herein are compatible with the user's test objectives. Section E.11 to this Standard provides information on the compatibility of specific test objectives with this Standard.

E.3.3 The evaluation criteria provided in this procedure are intended to be applied only to the design of the Truss and the specific Truss being tested. Extrapolation or other applications of test results obtained under this Standard shall not be used to evaluate Trusses or Truss designs that differ from the tested Truss.

E.3.4 When evaluation of deflection recovery is specified, loading procedures shall be as prescribed in Section E.12 of this Standard.

E.4 TERMINOLOGY

E.4.1 Design Drawings - Design drawings refer to drawings that specify the quality and quantity of materials, the manufacturing methods, and the use of the Truss, including information on the design loads acting on the Truss and the methods used to design the Truss members and connections. This includes engineering drawings, Truss Placement Drawings, architectural working drawings (including the Building Designer's bracing information), shop drawings, computer design output and/or design calculations.

E.4.2 Ultimate Load - the maximum load that the Truss is capable of carrying. Ultimate load is determined by loading the Truss until the Truss fails and will no longer carry the load that caused the failure. This failure is known as ultimate failure.

E.4.3 Normal Duration of Load - refers to the expected cumulative duration of the design load of 10 years. The load duration factor, C_D , for this case is 1.0.

E.4.4 Original Design Load - the design total unfactored load that the Truss is intended to carry as indicated in the original Truss Design Drawing or appropriate documents. This load is based on unfactored loads. The original design load includes load reductions for multiple loads.

E.4.5 Adjusted Design Load - the design total load obtained per Sections E.5.2 and E.6.3.1, where the load is calibrated to normal duration of load and test conditions. This load is based on unfactored loads.

E.4.6 Unfactored Loads - nominal loads prior to multiplication by any load factors as required in Load and Resistance Factor Design (LRFD).

E.5 MANUFACTURE AND SELECTION OF TRUSSES FOR TESTING

E.5.1 Information on specifying the sampling of Trusses is provided in Section E.11. Unless otherwise specified, no less than three Trusses shall be tested.

E.5.2 Trusses designed with a load duration factor different from 1.00 require determination of an adjusted design load. Design drawings or documents showing the equivalent information for the Trusses shall be available.

E.5.3 Trusses shall be manufactured at least one week (seven days) prior to testing.

E.5.4 Inspection of the Trusses shall be made for deviations from the design drawings, manufacturing inaccuracies, or damage to the Trusses selected for testing. Inspection shall be performed with detailed measurements recorded as to lumber breaks or splits; knots or wane under connector plates; flattened (angled) connector plate Teeth; misplaced, partially embedded, damaged or missing Metal Connector Plates; and any special reinforcement that is not indicated on the Truss Design Drawings but which is required by the building design or used in the Truss test. Deficiencies in manufacturing relative to *TPI 1*, the Truss Design Drawing, and applicable quality standards are to be noted and any necessary corrections are to be completed prior to testing of Trusses. Any required repair drawings and reinforcement of damaged Trusses shall be completed prior to testing, provided the repairs do not change the design assumptions of the Truss. If the Trusses cannot be repaired correctly, they shall not be tested.

E.5.5 The moisture content of each Wood Member within the Trusses shall be measured with a moisture meter in accordance with *ASTM D 4444* at time of Truss selection or manufacturing. When moisture content variation is to be limited, Truss storage and load testing shall be under controlled environmental conditions.

E.6 EQUIPMENT

E.6.1 Instrumentation used to measure loads shall be calibrated following the procedures of *ASTM E4*, and shall be found to have an accuracy within two percent, in the loading range of interest.

E.6.2 Deflection Measurement Instrumentation

E.6.2.1 Deflection measurements shall be taken with an accuracy of $\frac{1}{32}$ in. Measurement methods that have been found suitable for use are discussed in the commentary.

E.6.2.2 Deflection measurements, recording the movement of a specified point in a specified direction in the plane of the Truss, shall be taken at no less than the following locations. Vertical deflection readings shall be taken at midspan of the Bottom Chord and at ends of Cantilevers and shall be adjusted for support settlement. Horizontal deflections shall be taken at the ends of Trusses in which neither Top nor Bottom Chords are perpendicular to the direction of load application. If Trusses are tested in pairs, deflection readings shall be permitted to be averaged at corresponding locations.

E.6.3 Loading System

E.6.3.1 The loading system shall apply loads to the Truss in a manner consistent with that assumed for the design of the Truss, including provisions for bracing, sheathing and repetitive members. When design conditions cannot be met using the test equipment, then the design conditions shall be altered to match the test conditions and the original design load reduced as needed to satisfy *TPI 1*, resulting in an adjusted design load.

E.6.3.2 The loading system shall not affect the strength of the Truss through unintended restriction of lateral movement. When required to prevent the loading apparatus from slipping down the Top Chords of pitched Trusses or generating unintended horizontal thrust, short wood blocks or other non-structural attachments shall be attached along the Top Chords. No such attachments shall be permitted that would stiffen the Truss.

E.6.3.3 For testing Trusses designed for a uniformly applied load, it shall be permitted that test loads be imposed as equal concentrated loads at points spaced along the Chord no more than the greater of 12 in. or $\frac{1}{3}$ the panel length, where the panel length is the inside distance measured along the Chord between Web-to-Chord joints.

E.6.3.4 When testing Trusses designed primarily for a Top Chord loading, the Bottom Chord is not prohibited from being loaded with a load that is equal to the design Bottom Chord load and that remains constant throughout the test, i.e., the Bottom Chord load need not be increased proportionally with the Top Chord load during the test.

E.6.3.5 Loading units shall be separated to prevent the possibility of arch action or bridging and shall be no wider than 12 in. measured parallel to the Truss span.

E.7 TEST PROCEDURE

E.7.1 Truss Positioning/Setup

E.7.1.1 The Trusses shall be supported on firm, stable reactions. Support at reactions shall provide freedom to deflect in a manner similar to that assumed in design and provided by the intended use.

E.7.1.2 Truss members shall be laterally braced consistent with the permanent bracing defined by the Truss Design Drawing. There shall be no restriction of vertical or horizontal movement in the plane of the Truss by the lateral bracing system, unless the Truss is specifically designed to permit such contribution of braces to Truss load capacity.

E.7.1.2.1 If Trusses are tested in pairs under gravity load, they shall be spaced at a distance sufficient to permit

adequate lateral bracing between the Trusses. For tests designed to simulate Truss performance in a sheathed assembly, sheathing panels shall not be used as cross bracing between Top Chords unless adjacent pieces are no more than 4 ft. long and are separated by a $\frac{1}{8}$ -in. gap. For Trusses to be tested without the influence of a sheathing diaphragm, equal length battens oriented perpendicular to the length of the Trusses shall be permitted to be single nailed to each Top Chord, separated by no less than $\frac{1}{8}$ -in. In either case, the sheathing or battens shall be of sufficient depth to carry the loading units. Application of loading units shall be based on the Truss spacing for which the Trusses were designed, not the actual Truss spacing used in the test.

E.7.1.2.2 For tests of single Trusses designed to simulate Truss performance in a sheathed assembly, sheathing panels no more than 4 ft. long and separated from adjacent panels by a $\frac{1}{8}$ -in. gap shall be nailed to the Top Chord using nail sizes and spacing consistent with the intended application.

E.7.2 Loading Procedure

E.7.2.1 All references to loads in Section E.7.2 are with respect to the adjusted design loads.

E.7.2.2 Prior to conducting the load test, a preliminary load equal to the design dead load shall be applied and held for five minutes. This preliminary load shall then be removed.

E.7.2.3 Apply the design dead load and hold for no less than five minutes. Record deflection measurements. These deflection measurements are known as the basic dead load (BDL) position.

E.7.2.4 With the dead load applied, increase the load to design total load at a rate not to exceed 0.1 units of design total load per minute. Record deflection measurements at least twice during this loading period.

E.7.2.5 Hold the load at design total load for no less than five minutes. Record the deflections at the beginning and end of this five-minute period. During this hold period, examine the Wood Members and metal plate connectors for signs of distress and record any such signs.

E.7.2.6 After the five-minute hold period at design total load, continue to load the Truss at a rate not to exceed 0.1 units of design total load per minute until ultimate failure of the Truss. The loading rate and loading increments shall be limited to that at which ultimate load can be measured with sufficient accuracy.

E.7.3 Test Failures

E.7.3.1 After failure, record the failure load, failure location and the type of failure.

E.7.3.2 The moisture content of each Wood Member at failure locations in the Truss shall be measured at the time of Truss testing with a moisture meter in accordance with *ASTM D4444* or immediately after the Truss test from samples taken from the Truss in accordance with *ASTM D4442*.

E.8 EVALUATION CRITERIA

E.8.1 The maximum Truss deflection adjusted to remove the effects of support settlement shall be obtained for each Truss tested at a load level equal to the original design load. The determination of the deflection at the original design load by linear interpolation or extrapolation using the two nearest data points is not prohibited. The maximum deflection at design load, relative to the BDL position, averaged from all tests of identical Trusses shall not exceed the deflection predicted in design by more than 25 percent, or $\frac{1}{8}$ in., whichever is larger, nor shall it exceed the limiting criteria specified by *TPI 1*, or other deflection limitations specified by the Building Designer and used in the design of the Trusses.

E.8.2 The Truss shall have shown no signs of distress during the hold period at the adjusted design load. Examples of Truss distress include, but are not limited to, plate buckling, out-of-plane buckling of Wood Members, wood crushing, and/or partial plate tooth withdrawal.

E.8.3 Each Truss shall have carried the load implied by the design methodology for the appropriate mode of failure. Required loads for wood and steel failure modes shall be as given in Sections E.8.3.1 and E.8.3.2.

E.8.3.1 If failure occurred due to fracture of the steel in a connector plate, the Truss shall have carried no less than a load as defined in Equation E8.3.1.

$$\text{UL}_{\text{required}} = 2.0/\text{LF} \times \text{ODL} \quad (\text{E8.3.1})$$

where:

$\text{UL}_{\text{required}}$ = minimum required ultimate load based on a plate steel-fracture failure mode

LF = 1.33 for Truss designs governed by wind or seismic loads using allowable stress design if a 1.33 increase on allowable steel shear and tension values was used in the original design; otherwise, LF equals 1.00

ODL = original design load

E.8.3.2 If failure occurred in the wood or due to separation of a connector plate from the wood in which it is embedded, the Truss shall have carried no less than a load as defined in Equation E8.3.2.

$$\text{UL}_{\text{required}} = C \times \text{ADL} \quad (\text{E8.3.2})$$

where:

$\text{UL}_{\text{required}}$ = minimum required ultimate load based on a wood or plate tooth-holding failure mode

ADL = adjusted design load

C = 2.1 if the failure occurred due solely to wood tensile fracture, wood bending fracture, or separation of a connector plate from the wood in which it is embedded; or

C = 1.9 if the failure occurred due to wood compression parallel to grain crushing, regardless of whether it occurred in a member subject to axial stress alone or in a member subject to combined axial and bending stresses or if the failure occurred due to excessive compression perpendicular-to-grain crushing and resulting effects on joint or member strengths (such as out-of-plane instability or tooth withdrawal due to c-perp crushing); or

C = 1.67 if the failure occurred due to lateral buckling of a compression member or a bending member.

If failure occurs due to a combination of the failure modes listed above, the value of C shall be taken as the lowest for any of the failure modes which occurred, unless it can be shown that one failure mode was the initial failure and other failure modes occurred as a result of that initial failure mode.

E.8.4 In the event that all Trusses tested do not meet the requirements of Sections E.8.1, E.8.2, and E.8.3, acceptance or rejection of Trusses as a result of tests conducted following this Standard shall be the responsibility of the licensed professional engineer reviewing the test results. The commentary provides information when only one Truss fails and its failure does not provide conclusive evidence of inadequacy in design or manufacturing.

E.9 TEST REPORT

E.9.1 The written report shall include: method and location of testing, including any variation from the pro-

cedures given in this Standard; objective of the testing; lumber grades; lumber moisture content at time of Truss selection or manufacturing and at time of Truss test; plate type and size at each joint; midspan deflection at the original design load adjusted for support settlement; ultimate load and the methods used to measure load; personnel conducting and witnessing tests; a detailed drawing of each Truss and the findings of the Truss inspection, including any variations from the design drawing, *TPI 1* or other applicable standards. Photographs and/or sketches of the test setup and failure location shall be included in the test report.

E.10 REFERENCED STANDARDS

1. *ANSI/TPI 1-2007*, National Design Standard for Metal Plate Connected Wood Truss Construction. Published by the Truss Plate Institute, Inc. www.tpinst.org
2. *ASTM D4442-92* (1997), Standard Test Methods for Direct Moisture Content Measurement of Wood and Wood-Based Materials. www.astm.org
3. *ASTM D4444-92* (1998), Standard Test Methods for Use and Calibration of Hand-Held Moisture Meters.
4. *ASTME4-96*, Standard Methods of Load Verification of Testing Machines.

TPI 2 APPENDICES (NON-MANDATORY)

E.11 TPI 2 APPENDIX 1 (NON-MANDATORY): OBJECTIVE FOR TRUSS TESTING

E.11.1 This section provides guidance regarding the compatibility of specific test objectives with the given procedure for load testing of Trusses.

E.11.2 Testing is useful for verifying performance of Trusses for a variety of reasons. The number of Trusses selected for testing and the manner in which they are selected vary depending upon the test objective and the evaluation criteria. The evaluation criteria provided in the standard is suitable only for establishing a minimum level of acceptance for the specific Truss and Truss design being tested.

E.11.3 Sections E.11.3.1 through E.11.3.3 describe test objectives that may be compatible with this Standard.

E.11.3.1 Testing in accordance with this procedure may be useful in situations in which there is good cause in the judgment of a recognized authority to question the structural integrity of the Trusses. This section does not imply that satisfaction of this Standard is sufficient to replace Truss design or quality standards.

E.11.3.2 Testing in accordance with this procedure may be useful to demonstrate that adequate performance may be obtained from a Truss designed in accordance with the *ANSI/TPI 1 National Design Standard for Metal Plate Connected Wood Truss Construction* with a specific Metal Connector Plate configuration, type of lumber, type of Truss fabrication equipment or value of a design parameter.

E.11.3.3 Testing in accordance with this procedure may be useful as part of a testing program conducted to assure that wood Truss performance is compatible with that implied by the design specifications.

E.11.4 Sampling requirements vary depending upon the test objective. Sections E.11.4.1 through E.11.4.3 provide guidance for the test objectives described in Section E.11.3.

E.11.4.1 For tests to examine Trusses whose structural integrity has been questioned as described in Section E.11.3.1, a variable number of Trusses may be required depending upon the situation. In some cases, it may be suitable to select that Truss, or the two or three Trusses, with the worst combination of characteristics that caused the structural integrity to be questioned.

E.11.4.2 For demonstration tests such as described in Section E.11.3.2, as few as one to three tests may be required. Depending upon the test objective, special care may be required during the design and fabrication of Trusses for demonstration tests to assure a meaningful study of selected items of interest. The Truss fabrication and design may be modified so as to isolate a particular item of interest.

E.11.4.3 For tests conducted as part of a quality assurance program, as mentioned in Section E.11.3.3, a relatively large number of tests may be necessary and the testing may need to be conducted on a periodic basis. For such situations, the quality assurance program should specify appropriate Truss test evaluation criteria, which may differ from that provided in this Standard. It may be desirable to specify that test Trusses be selected from the Truss production plant in a random manner.

E.11.5 For some uses of this test Standard, it may be necessary for testing to be verified by an approved agent. Such verification of test results and definition of an approved agent shall be specified when required.

E.11.6 This Truss testing procedure is not intended to be used to provide for acceptance of Trusses when structural calculations have not been provided to support the design

of the Trusses. This Truss testing procedure is not to be used as an alternate to a specified design procedure.

E.12 TPI 2 APPENDIX 2 (NON-MANDATORY): DEFLECTION RECOVERY

E.12.1 This section provides guidance regarding a loading procedure that provides for a measurement of deflection recovery and a criterion for evaluation of the deflection recovery.

E.12.2 For Truss tests in which load and deflection recovery data are to be acquired, the following loading procedure is suggested unless an alternate procedure is specified.

E.12.2.1 Following the loading procedure given in Sections E.7.3.1 through E.7.3.5.

E.12.2.2 Continue to hold the Truss at design total load for an additional 25 minutes. Read and record deflection measurements at the beginning and end of this period.

E.12.2.3 Remove the design live load portion of the load. For loading systems in which accurate load distribution upon unloading cannot be achieved, it shall be permitted that the design dead load be removed and immediately reimposed in order to achieve the correct design dead load distribution.

E.12.2.4 Hold the load at design dead load for up to 15 minutes. Read and record the minimum values of each deflection measurement during this time period.

E.12.2.5 Reload the Truss to design live load and continue with the loading procedure given in section E.7.3.6.

E.12.3 Evaluation of the deflection recovery of the Truss shall be made using the following criterion. The Truss shall pass if the deflection recovered during the loading steps given in Sections E.12.2.3 and E.12.2.4 equals or exceeds 85 percent of the maximum total deflection beyond basic dead load measured during the loading step given in Section E.12.2.2.

COMMENTARY TO TPI 2-2002 - STANDARD FOR TESTING METAL PLATE CONNECTED WOOD TRUSSES

The Commentary portion of this document is intended to provide background and supplementary information to the provisions of *TPI 2-2002*. Paragraph numbers in the Commentary marked with a “§” symbol refer to the corresponding section number in *TPI 2-2002*.

§E.1.1 *TPI 2-2002* assumes sample confirmation tests of a limited number of Trusses demonstrate a minimum level of performance. Trusses designed per *ANSI/TPI 1: National Design Standard for Metal Plate Connected Wood Truss Construction* and built per the Quality Criteria for Metal Plate Connected Wood Trusses (*ANSI/TPI 1*, Section 3) should meet this Standard. Prior to 1995, TPI design specifications gave similar objectives for full-scale Truss testing, i.e., “*verify the deflection characteristics at critical spans,*” “*substantiation of new types of connection systems by tests of at least three substantially different designs to establish their performance*” and “*...tests... are generally required only to verify the performance characteristics of wood Trusses at critical spans.*” One prior issue of the TPI specification stated as follows:

“To qualify all spans and slopes of Trusses to be designed in accordance with these specifications and to enable the code agency to judge the efficiency of the Metal Connector Plate system, the initial acceptance of such systems may require full scale Truss tests to be performed. The object of the load tests is to obviate the need at later times for the performance testing of each and every variation on a given form of construction and with these results to design structural performance at the design stage without the need for continued testing.”

This issue of the TPI specification went on to specify test Trusses for accepting a Truss plate system to be the longest span permitted for a Fink or Howe Truss with only 2x4 members and a 2.5/12 and a 4/12 pitch.

These previous test objectives are broad and have been considered as requiring “confirmation” tests, meaning a small sample of full-scale Truss tests that, although not statistically adequate to demonstrate compliance with design specifications, are adequate to 1) show that large non-conservative deviations from the design procedure were not taken and, 2) demonstrate that the Truss component allowable stresses and the Truss analysis and design method can result in a Truss that does not show unacceptable performance. *TPI 2 -2002* continues to address such confirmation tests. This document differs from prior versions of Truss testing standards in that it more closely matches the minimum criteria upon which *TPI 1* Truss design methodology is based.

This document does not address proof-load tests, i.e., non-destructive tests, as there was concern that users would not be able to discern damage in a proof-loaded Truss, if it did occur, and the damaged proof-loaded Truss may subsequently be used in a structure. It was the intent of the committee to not have this Standard permit Trusses

that have been tested to later be used in a structure, nor to test Trusses already installed in a structure.

§E.2 Among safety issues related to Truss testing is prevention of catastrophic failure. Catastrophic failure refers to failure of many components of the Truss following failure of an initial component and is likely to involve splintering of wood, throwing of wood and steel material outward, or other sudden motion of the Trusses. Catastrophic failure can often be prevented by either removing the load immediately after the initial failure and/or by maintaining support just below the Truss to catch it at failure, i.e., providing additional supports several inches below the Truss at midspan and the quarter-points. The height of additional supports should be low enough to not interfere with the Truss test but close enough to prevent catastrophic failure.

§E.3.2 Testing and evaluation in accordance with this Standard is performed for a variety of reasons by the user, manufacturer or designer of wood Trusses. Guidance regarding objectives was placed in Section E.11, which is intended to be non-mandatory and general, since objectives for the testing addressed in this Standard are often general in nature and without an anticipated need. Provisions in *TPI 2-2002* do not apply for evaluating performance of metal-plate-connected Trusses under cyclic loading, such as that from earthquakes or bridge loads. Rather, it is adequate for assessing capacity under code-specified static equivalents of earthquake loads.

§E.3.3 *TPI 2-2002* was written from the viewpoint that the test objective was to verify that a Truss designed in accordance with *TPI 1* design specifications was not unusually below specified load capacity. This is not the same as if it were intended to verify that a Truss has sufficient strength, or that the design methodology used for the Truss is adequate. The assumption is made that the *TPI 1* design provisions are adequate and, therefore, the great majority of Trusses designed in accordance with the *TPI* design provisions should pass the criteria given in the testing procedure. The testing procedure is not intended to be used to establish design values. From this basis, it is appropriate to assume that a Truss selected for testing has wood at the lower 5 percent exclusion limit and steel at the minimum specified strength. Since the basis for this test procedure is to show that the Truss is not unusually weak, the procedure is not appropriate for showing that the Truss is sufficiently strong, which would require more rigorous sampling and testing. For this reason, results should not be applied to anything other than the Truss being tested.

The clause “unless otherwise specified” at the end of Section 3.3 was included because past uses of earlier TPI Truss testing Standard (i.e., *TPI-85 Appendix D* [retired] type tests) expressly permitted extrapolation of test results to other Trusses, (i.e., test one design of a Truss “series” to qualify the whole series).

§E.3.4 Unless specified by the user of *TPI 2-2002*, provisions in Section E.12 remain unnecessary. In the event that deflection recovery is of interest to the party performing the testing, the use of test evaluation procedures in Section E.12 should be prescribed. This language is intended to serve as the only acceptable procedure to assess deflection recovery in metal-plate-connected wood Trusses.

§E.4.2 In some cases, failure of a component of a Truss during testing may cause increased deflection and a decrease in load in hydraulic and pneumatic loading systems, prior to the ultimate failure of the Truss.

§E.4.6 The evaluation criterion in this Standard was developed from allowable stress design methodology. Section E.4.6 states that design loads are based on unfactored loads for clarity in the event that a Truss designed for LRFD is desired to be tested. However, it must be recognized that Trusses designed in accordance with LRFD methodology may require somewhat different evaluation criteria from those presented in this Standard.

§E.5.1 While this is a small sample size, it is felt that the cost of testing and many of the objectives of testing do not warrant a larger sample size. In many cases, a sample size of three will determine if there are large deviations from design assumptions.

§E.5.2 Test Trusses are required to have a test load (the “adjusted design load”) based on a load duration factor (C_D) of 1.00 because the performance criteria specified in Section E.8 are based on normal duration loads and cannot be easily adjusted to other load durations. The reason for this is that the allowable column and beam stresses for Wood Members are not always linearly adjusted with C_D . If all of the allowable stresses for a Truss were linearly adjusted by C_D , then the 2.1 factor in Section E.8.3.2 could be simply divided by C_D to determine the required strength. However, slender wood beams and columns are practically unaffected by C_D , and dividing 2.1 by C_D would not be conservative if such members governed a design. The most accurate way to adjust the performance criteria to other load durations is to adjust the Truss design load by redesigning the Truss for normal duration design loads.

The adjusted design load is the maximum design load permitted using the normal duration of load factor ($C_D = 1.00$) and a Truss configuration (lumber grades and sections, plating and Truss geometry) identical to the original design drawings. No adjustment is permitted in the strength of the steel due to duration of load. The loading pattern and other design variables specified in Section E.6.4.1 used in determining the adjusted design load should match the application of these variables in the Truss test. For Trusses designed for shorter than normal durations of load ($C_D > 1.00$), the adjusted design load will be less than or equal to the original design load. For Trusses designed for normal durations of load ($C_D = 1.00$) and tested in a manner consistent with that assumed in the original design, with regard to the variables referred to in Section E.6.4.1, the adjusted design load will be equal to the original design load.

The adjusted design load does not have to exceed the original design load since a Truss that is “overbuilt” for its intended purpose should not be required to have a higher test strength than a Truss that is designed and built efficiently. This exception was permitted because many Truss tests may be done with a specific end use in mind, where the load required to be carried by the Truss will be less than the maximum allowed by analysis. For such a case, the Truss may be judged acceptable based only upon the intended end use, rather than all possible loads up to and including the maximum allowable design load for the Truss. This exception does not exempt the Truss design from complying with standard design specifications, as stated in Section E.3.1.

The following examples are presented to clarify the determination of the adjusted design load.

§E.5.3 The one-week age requirement is based on the relaxation of wood fibers that occurs around the plate Teeth, resulting in higher tooth holding strengths for fresh joints compared to aged joints that are otherwise identical. This provision came from series of tests (Wilson, 1978 and Arbek, 1979) that showed this reduction to be on the order of 15-20 percent and that most of the reduction in strength due to this fiber relaxation occurs in the first 7 days. In instances where it is necessary to test Trusses less than seven days after manufacturing, it is appropriate to increase the required load level for tooth holding failures by as much as 15 percent. The evaluation criteria given in Section E.8.3.2 is a single criteria that applies to wood fracture as well as tooth holding failure (withdrawal of plate Teeth from wood), however this adjustment based on the age of the Truss should only be applied to tooth holding failures.

§E.5.4 Design deviations and manufacturing quality should be noted as these factors may affect the Truss performance.

§E.5.5 The only wood property specified for measurement was moisture content since it can be done easily on a manufactured Truss. Specific gravity, MOE, etc., were not specified as they are difficult to measure without damaging the Truss and may be of little use in many tests. In some cases, it may be desirable to measure specific gravity in accordance with *ASTM D 2395* from Sample Blocks cut from the Truss members after the test. Moisture content of the wood should be measured at the time of Truss manufacturing, if possible, using a nondestructive method such as an electric moisture meter in accordance with *ASTM D 4444*. Moisture content of samples trimmed from the ends of members may also be used to determine moisture content in accordance with *ASTM D 4442*, but this is usually more difficult than using an electric meter and often the trim from the various members is not available or cannot be identified. Where moisture content of the wood at the time of manufacturing is not available, it should be noted that moisture content history, where changes in moisture content have occurred, may affect Truss strength.

Adequate weather protection should be furnished to prevent adverse effects of moisture on the Trusses and to prevent variation of the load caused by the absorption of water by the loading units. Environmental control is especially important for basic research testing. To minimize the number of variables which may affect Truss performance, the Trusses should be manufactured from wood at an equilibrium moisture content and then stored and tested at conditions that maintain that equilibrium moisture content. If variation of moisture content is a factor being studied, or if environmental control is not possible, environmental conditions and moisture content should be periodically monitored. When environmental conditions cannot be controlled, moisture content of the Truss(es) should periodically be measured with an electric moisture meter in accordance with *ASTM D 4444* to monitor variation in moisture content.

§E.6.2.1 Calibration of the deflection measuring equipment is not specified, but it is mandatory that the operator of the equipment verify that deflection measurement equipment has the required accuracy. Deflection measurement methods that have been found suitable are described below. Caution is advised in use of deflection measurement methods that require personnel to be near the Trusses while under load as sudden failure of Trusses poses significant danger.

A) Line, scale & mirror method. A line is supported on free-rolling bearings and kept under tension by means of weights hanging on the ends of the line. The free-rolling bearings should be located directly above the Truss bearings and attached to the Truss. The line should pass parallel to and in front of the Truss. Behind the line, a linear scale with markings at no less than every $\frac{1}{32}$ -in. and mirror should be attached to the Truss at points where deflection is to be measured. The scale should be oriented to measure distance in the direction in which deflection is to be measured. Deflection readings are taken by reading the location of the line on the scale, with the individual's eye positioned so that the line covers its image in the mirror. Scale readings should be taken with consistent reference to either the top or bottom edge of the line. There are several drawbacks to this method in some tests, namely that the pulley attachment may interfere with a connector plate or the support, the weights required to obtain a taut line may be relatively large, and deflection measurements at ends of Cantilevers cannot be obtained. These problems can be remedied by tying the line to nails over each reaction, in which case it is necessary to assure that line tension remains constant throughout the test, and/or attaching the line to fixed locations other than the Truss and placing scales on the Truss over the reactions to explicitly measure support settlement.

B) Mechanical Dial Gauge Method. Dial gauges should be positioned at points of interest and oriented in the direction of interest. Dial gauges should be positioned in contact with the Truss over the reactions to measure deflection of the Truss at the reaction (support settlement). In most cases, dial gauges cannot be positioned directly under the Truss at the bearing location due to interference with the Truss support; however, gauges may be placed immediately adjacent to each side of the bearing location and the readings averaged, or the dial gauge at the bearing can be located above the Truss, when appropriate. Measurements should be adjusted to remove that portion of the deflection due to support settlement, as measured by the gauges over the reactions.

C) Electronic Instrument Method. This method is identical to method (B) except that electronic instruments are used instead of mechanical dial gauges.

§E.6.2.2 Directions are specified with reference to the intended orientation of the Truss, (i.e., the directions given are for a Truss intended to be installed, and tested, in a vertical plane). Deflections at supports must be used for

reference of measurement of deflection at other locations in order to account for support settlement. Adjustment for support settlement at a particular point should be found by linearly interpolating between measured support settlements based on the distance from the particular point to each support. Note that deflection due to compression perpendicular to grain crushing at the Truss bearings is normally considered support settlement and subtracted from Truss deflection.

Horizontal deflections should be measured for Trusses that are expected to exhibit significant horizontal deflection, such as scissors Trusses or other similar Trusses in which both Top and Bottom Chords are not perpendicular to the direction of load application.

§E.6.3.1 Most loading systems for Truss tests have limitations in the types of loads, bracing, sheathing and repetitive members that can be applied. Certain load distributions, such as wind pressure distributions, girder load applications and loading of repetitive member systems, are difficult to apply with most loading systems. As different load conditions from those used in the design of the Truss change the distribution of stresses within the Truss, it is essential that the load distribution applied in a test match, within reasonable tolerances, the load distribution applied in design. In addition to the actual location, direction and magnitude of loads, other design conditions that should be matched in the Truss test are the level and location of lateral bracing, support conditions and other deflection restraints, and presence of supplementary structural members such as plywood sheathing or adjacent members which may share load. For example, if sheathing is not applied to the Truss Chords and a repetitive system of Trusses are not tested, the buckling stiffness (C_T) and repetitive bending (C_r) factors should not be used in the design of the Trusses.

The design of some Trusses, such as those with more than two bearings or Cantilevered over a bearing, may be governed by a loading condition when only a portion of the Truss is fully loaded. In such cases, design of some Truss members may be governed by one loading condition while design of other Truss members may be governed by another loading condition. In such cases, Trusses should show adequacy for both types of loading conditions. However, due to one loading condition often controlling most members, or due to limited funds for testing, it is often desirable to specify only one loading condition for Truss testing. It is the responsibility of the user to determine which loading conditions should be used to test the Trusses.

§E.6.3.2 The intent of Section E.6.4.2 is to prevent use of

a loading system that applies significant horizontal loads to a pitched Truss that was designed only with vertical loads, thus restraining lateral movement of the Truss, i.e., by using a pneumatic hose or balloon loading system on the Top Chord of a pitched Truss that was designed only for vertical loads. Such a loading can inhibit vertical Truss deflection and affect failure loads. This problem is especially apparent when testing Trusses that exhibit significant horizontal deflection when loaded solely by vertical loads, such as scissors Trusses or other similar Trusses in which both Top and Bottom Chords are not perpendicular to the direction of load application.

§E.6.3.3 This loading exception permits equal concentrated loads at a spacing up to the greater of either 12 in. or $\frac{1}{3}$ the panel length to be considered uniform load. The $\frac{1}{3}$ panel length specification is to ensure that the bending moment in the Chord is approximately the same as that obtained from uniform loading. The 12-in. criteria prevents unnecessarily short spacing between loading points than would otherwise be required by odd short panels in a Truss (at or near an open chase, near a bearing, etc.) if only the $\frac{1}{3}$ panel length criteria were used, but which are not really necessary as the Chord stresses in such short panels are not as greatly affected by panel bending loads due to their short panel length

§E.6.3.4 Due to the difficulty in applying both Top and Bottom Chord loads for most tests, the Bottom Chord load need not be increased proportionately with the Top Chord load when this does not significantly affect the performance of the Truss. This condition applies to Trusses designed primarily for a Top Chord loading which includes those Trusses without concentrated loads whose Bottom Chord uniform load is no greater than 25 percent of the total uniform load (i.e., 33 percent of the Top Chord uniform load). This permits a single application of uniform load equivalent to Bottom Chord design dead load at the beginning of the test, which need not be increased during the test. If design live load is imposed on the Bottom Chord or if the Truss carries a large portion of its design load as Bottom Chord dead load, the Bottom Chord uniform test load must be increased in the same proportion as the Top Chord uniform load throughout the test.

§E.6.3.5 The intent of Section E.6.4.5 is to: 1) prevent spacing of loading units so close together that, as the Truss deflects, the loading units contact each other, possibly transferring load to the reactions through "arch action," and 2) prevent using loading units that are stiffer than the Truss or Top Chord and long enough that they act as a stiff beam once the Truss deflects and only distributing load to the Top Chord at locations that result in moments other than those that would result from a true

uniform load, i.e., loaded pallets that only transfer load to the Truss at panel points.

§E.7 Additional considerations regarding test procedure are provided in *ASTM E 73, E 196, E 529 and E 1080* and it may be useful to review one or more of these standards.

§E.7.1.1 Reaction supports and their restraint on the Truss should match those used for Truss design. If Lateral Restraint is imposed at both reactions, two-way load cells should be used to measure the horizontal force being resisted by the reaction, as well as the vertical force resisted by the reaction. Likewise, any reaction moment restraint should be noted and magnitudes measured.

§E.7.1.2.1 One of the intents of Section E.7.2.2.1 is to specify, when appropriate, a load distributing element for Truss pairs tested under gravity loads that avoids composite action with the Trusses. By using spaced battens attached with a single nail to each Truss, the Truss Top Chords are laterally fastened and vertical loads are distributed but composite action between the load distributing element (battens) and the Truss Chord is prevented.

When testing in pairs, the Trusses need not be spaced at the spacing for which they were designed. However, the loads applied to the Truss must correspond to the Truss spacing at which they were designed. For example, if Trusses are designed for a design load of 50 psf (2 feet on center, sufficient load must be added over a pair of Trusses so that a total load of 200 plf (100 plf/Truss) is applied at design total load, regardless of the spacing between Trusses during the test.

§E.7.2 The loading procedure is intended to check those requirements imposed by the design procedure, namely that the live load deflection and the failure load are no less than the minimum implied by the design methodology. No hold periods longer than 5 minutes are specified. The only hold period required is that time necessary to gather deflections. Longer hold periods are not included as they are not used or implied by the procedures used to establish Truss, member and joint design loads. Section E.12 provides an optional procedure for measuring creep and deflection recovery involving longer hold periods.

§E.7.2.2 This preliminary load application allows the Truss to settle into position and provides an opportunity for checking alignment, equipment positioning and overall setup.

§E.7.2.6 The given test procedure will permit loading to design total load in 20 minutes followed by a five-minute hold, and loading to two times design total load in an-

other 10 minutes, if the loading and measurement system is suitable for this rate of loading. This exceeds the usual five- to ten-minute duration of ramp loading tests used for establishing strength of individual wood and steel components, but permits the gathering of deflection data up to design load. The permitted increase in the rate of loading beyond design load is provided since no deflection measurements are necessary beyond design load and this rate more closely matches the duration of those tests used to establish wood and steel strengths. Long-duration ramp loadings or hold periods at high load levels may cause wood-based failure to occur at lower levels than would otherwise result. If loading is accomplished by addition of gravity weight, the increment in weights should be kept small enough so that the ultimate failure load can be determined with sufficient accuracy. If failure occurs during the addition of a gravity load increment, the failure load may be anywhere from just above the previous load increment to the current load increment, or possibly higher if the current load increment was not slowly applied to the Truss.

§E.7.3.2 *ASTM D 4444* pertains to electric moisture meters. Moisture content may also be measured after the test in a destructive manner by cutting blocks from the Truss members and determining moisture content in accordance with *ASTM D 4442*. If this is done, the blocks should be cut and weighed immediately after the Truss test.

§E.8 The evaluation criteria are those implied by the design procedure for weak member strength. This is a simplistic extrapolation from design to implied failure loads and is based on tests of single member components.

§E.8.1 *TPI 2-2002* requires the average deflection at design load from all tests to not exceed those limits specified by *TPI 1* design specification or alternate limits set by the Truss Designer, if more restrictive. The application of this criteria to the average live load deflection from all tests, rather than to the individual live load deflection from each test, is due to the value of modulus of elasticity (MOE) used in design of Wood Members being an average value. Since it is common for any particular piece of wood to be more or less flexible than the average grade MOE, it would be expected that individual Trusses may fail this criteria. However, on average, they should meet this deflection criteria. This Standard may be somewhat restrictive since the average deflection of any three Trusses will vary above and below the average of the population. When few Trusses are tested, evaluation of deflection may be adjusted if the Trusses were manufactured from wood with MOE less than the average grade MOE. In such cases, tests of the Truss members may be desirable after the Truss test to determine MOE,

or specific gravity if reliable MOE measurements cannot be obtained. These measurement can aid in determining if excessive Truss deflection, relative to the given deflection limitations, is due to lumber characteristics. The live load deflection limitations specified in the National Design Standard for Metal Plate Connected Wood Truss Construction are L/180 for roof Trusses without ceilings, L/240 for roof Trusses supporting flexible ceilings, and L/360 for roof Trusses supporting plaster ceilings or floor Trusses.

It is also recommended that measured Truss deflection be compared against theoretical deflections calculated by the Truss Designer. While deflections of individual Trusses may vary widely due to variation in lumber MOE, the result of a large number of Trusses built from randomly selected lumber should match the theoretical deflection.

Other than measuring deflection at the beginning and end of the 5 minute hold period at design total load, no provision is made in this Standard for measuring creep or deflection recovery and no suitable criteria are provided for determining acceptable levels of creep or deflection recovery. Past creep performance of metal-plate-connected wood Trusses has been acceptable. Creep rates at design load are expected to be equal to or slightly higher than that of solid-sawn lumber due to the presence of connections. An alternative loading procedure that includes provisions for measuring creep, along with a proposed evaluation criteria, is provided in Section E.12.

§E.8.2 The Truss should show no signs of distress at design load. Signs of distress include visible yielding, buckling or other deformation of the material within a steel plate or Wood Member; withdrawal of a plate from wood; formation or extension of cracks, splits or fractures in steel or wood; and excessive lateral deflection of a member. Audible noise from the Truss is not by itself an indication of distress.

§E.8.3 As this Standard is for confirmation Truss tests, the required failure loads are based on the loads expected to fail the “weakest members” that may typically occur in a Truss, where “weakest members” are defined for wood as members with strength at the lower fifth percentile of the strength distribution, and for steel as members with the minimum specified yield and ultimate strengths. This is appropriate given that only a small number of replicates are usually tested and tests are for confirmation only, i.e., results are not being used to calculate or assign design values. The given minimum failure loads are those implied by the design methodology for single component testing. By specifying loads based on these “weak member” strengths, the variability of the material used

to manufacture the Truss is taken into account. Variability inherent in the structural analysis (i.e., in the conversion of applied loads on the Truss to individual member stresses) and in the design (i.e., in the sizing of members with given strengths for the stresses resulting from the structural analysis) are not taken into account.

§E.8.3.1 Failure loads for steel fracture are based on the lowest safety factor implied by *TPI 1* design specifications, typically fracture of a plate loaded in shear. The safety factor implied by *TPI 1* design specifications for shear is 1.44 on yield stress, which is obtained from dividing the factor on tensile yield stress used to establish allowable shear stresses (0.4) into the theoretical ratio of shear strength to tensile strength (0.577) and no less than 2.0 on ultimate stress. The safety factor on yielding of 1.44 results in a variable safety factor on ultimate strength, which can be found by multiplying 1.44 by the ratio of the minimum steel ultimate stress (F_u) to the steel design yield stress (F_y). For currently produced plates, ratios of F_u/F_y vary from 1.17 to 1.41, which would result in a range of safety factors for ultimate strength in shear between 1.68 and 2.03. For plates made from steel with F_u/F_y ratios less than 1.39, the alternate safety factor of 2.0 governs. This results in a minimum basic steel safety factor for shear of 2.0 for all Truss plates, with some plates possibly having a higher safety factor on ultimate shear strength due to the requirement of a safety factor of 1.44 on yield shear strength. Similar limitations, including a minimum safety factor of 2.0 on strength, exist for tensile stresses on Truss plates, except that the specified safety factor on yielding is 1.67.

While the 2.0 specified failure load level may be expected, it should be noted that a five percent deviation in this failure load (meaning the 2.0 would drop to 1.9) may occur due to a five percent deviation permitted in steel design from actual steel thickness.

When the one third stress increase for steel design of Truss plates is used per *TPI 1*, the LF parameter shall be set to 1.33 and used as a divisor on the original design load when establishing the required ultimate load level. This stress increase has only been used for wind and seismic load cases, so it should not apply to original design loads based on other types of live load. This stress increase has been permitted for use by the Truss Designer with the 1997 *Uniform Building Code* and other Building Codes issued prior to 2000, provided that the 0.75 factor permitted to reduce multiple live loads imposed simultaneously is not also used. This is applied as a stress increase for steel design in the *TPI 1* Standard as this has traditionally been typical practice for steel design but not for wood design. Other steel standards, such as the AISI

Specification for the Design of Cold-Formed Steel Structural Members, have expressed this as a 0.75 load factor, rather than a 1.33 factor on steel strength, but the same was not done in the *TPI 1* Standard because this 0.75 load factor would then apply to all materials and it is not applicable to wood.

§E.8.3.2 Failure loads for wood and tooth holding fracture are based on the divisors applied to the 5th percentile strength level for the different wood failure modes to establish design values. The factor of 2.1 is given in Table 9 of the *ASTM D245* for tensile and bending stresses of softwood lumber and is also used by *TPI 1* in determination of the adjustment factors used in determining allowable tooth holding values. The adjustment factors used for compression parallel-to-grain and lateral buckling of wood are 1.9 and 1.67, respectively.

Compression perpendicular to grain is assigned a C factor of 1.9, indicating that failure loads for this mode of failure are expected to be at least this multiple of design load. However, there have been informal reports that compression perpendicular to grain failure modes may occur at multiples as low as 1.3 times design load for design loads determined in accordance with standard compression perpendicular to grain design values. *TPI 2-2002* uses the 1.9 value based on reports to TPI and elsewhere (one such report is: Testing and analysis of parallel chord trusses, by C. Lum and E. Varoglu, in Proceedings of the 1988 International Conference on Timber Engineering, Seattle, Vol. 1, pp. 460-466), and the lack of adequately documented test data illustrating lower multiples than 1.9. Users of this Standard may want to investigate this issue further if failures in compression perpendicular to grain occur at values lower than 1.9 times the design load, as there is no prescribed value for ultimate loads for this failure mode in the standards used to establish allowable design values for compression perpendicular to grain. Further, it may be appropriate to use a 1.3 multiple since the well-accepted multiple of 2.1 for wood failure in bending or tension may be divided by 1.6 when there are no duration of load effects as is true for compression perpendicular to grain.

The above safety factors and adjustment factors are those applied to the allowable design stress, which is based on the “weak” member strength, i.e., the lower 5th percentile of the strength distribution for wood. On average, it is expected that higher loads would be attained. It is also expected that about 5 percent of the time failure will occur at lower loads.

The C-factors specified in Section E.8.3.2 do not include any adjustment for the variability of the wood strength,

whereas allowable stress design values normally include an adjustment such that, in addition to the factor of safety, 95% of the members will have a strength greater than or equal to the design value. For this reason, Section 1.1 states that “...satisfactory compliance with this Standard is not, by itself, sufficient to imply adequacy of any Truss design...”. Section E.3.1 goes on to state: “...this Standard is intended to provide assurance that Truss strength and stiffness performance are adequate to meet load requirements for their specific application...”. The minimum sample size of three Trusses per Section E.5.1 or one to three Trusses per Section E.11.4.1, is not adequate to establish a lower 5th percentile determined with 75 percent confidence, which is typically required when initially establishing wood strength design levels. A randomly selected sample of three Trusses which perform adequately will serve to demonstrate that it is possible to obtain adequate performance from the Truss type. While these additional tests may provide increased level of assurance that the Truss performs adequately, this additional level of assurance has not been quantified and cannot be relied upon to justify a design capacity without supporting structural calculations. If Trusses used for testing are selected through careful examination of the Truss group to identify the worst case combinations of lowest wood grade, on-grade wood characteristics such as knots, and manufacturing characteristics such as mislocated plates and bent Teeth, as discussed in Section E.11.4.1, it may be possible to determine adequacy of all members in the group of Trusses being questioned. The only other possible method of determining 95 percent exclusion by test would be to test enough Trusses to establish statistical distributions.

The design of Wood Members and wood fasteners is dependent upon the length of time that the design load is imposed upon the wood. This is accounted for by the use of a duration of load factor (DOL). Although adjustment of Truss performance criteria for duration of load has often been done by linear adjustment, i.e., division of the required load factor by the load duration factor used in the design, this adjustment is only accurate when the design strength is linearly related to load duration factor, i.e., for plate tooth holding, members in tension, or fully braced members in bending or compression. Since design loads governed by buckling are not adjusted, or only partially adjusted, for duration of load, and there are no duration of load effects for compression perpendicular to grain, division of the required load level by the design duration of load factor may be liberal. For this reason, it is required in Section E.5.2 that a duration of load factor of 1.0 be used to design Trusses that will be tested.

§E.11.4.2 As shown in Section E.11.4.2, various aspects of a Truss design may be isolated by modifying the design and manufacture of the test Trusses. For example, Trusses on a smaller scale or with a shorter span than full scale Trusses are often tested with variations on a Top Chord bearing detail in order to maximize the number of tests that can be done with limited resources.

§E.12 The loading procedures and evaluation criteria given for creep/deflection recovery in Section E.12 are provided simply for guidance. Although not quantified in the methods used to establish material design values, it is commonly assumed that use of typically established material design values result in acceptable levels of creep. Although acceptable levels of creep or deflection recovery in short-term tests may not be related to long-term performance, the loading procedure and evaluation criteria given in Section E.12 are suggested for use.



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