CHAPTER 5

BASIC TIMBER DESIGN CONCEPTS FOR BRIDGES

5.1 INTRODUCTION

For thousands of years, timber bridges and other timber structures were built primarily by trial and error and rule of thumb. Designs were based on past experience, and little concern was given to efficient material usage or economy. As the complexity of structures increased, more attention was focused on the importance of accurate engineering methods. Research was undertaken to develop design criteria for wood with the same level of accuracy and reliability available for other engineering materials. As a result, developments in timber design have advanced substantially in this century. Although wood is orthotropic and differs in many respects from other materials, wood structures are designed using many of the same equations of mechanics developed for isotropic materials. Variations in material properties from growth characteristics, manufacturing, and use conditions are compensated for by material grading and stress adjustments applied in the design process. Timber design may seem confusing at first, but with experience it is no more difficult than design with other materials.

This chapter provides an overview of basic design concepts for sawn lumber and glulam used in bridge design. It includes specification requirements and methods for designing beams, tension members, columns, combined axial and bending members, and connections. Applications of these concepts to design situations are given in examples for each member and connection type. More detailed design related to specific bridge types is covered in Chapters 7, 8, and 9.

The discussions and examples in this chapter are based on a number of referenced specifications that were current at the time of publication. The reader is cautioned to verify these requirements against the most recent edition of the specifications before designing a bridge. In no case should the information presented in this chapter be considered a substitute for the most current design specifications.

5.2 DESIGN SPECIFICATIONS AND STANDARDS

The primary specifications for bridge design in the United States are the *Standard Specifications for Highway Bridges,* adopted and published by the American Association of State Highway and Transportation Officials (AASHTO).[1] These specifications are published intermittently and are revised annually through the issuance of interim specifications. They address all areas of bridge design, including geometry, loading, and design requirements for materials. AASHTO specifications are used extensively as the standard for bridge design and are the primary reference for the timber design requirements, procedures, and recommendations addressed in this manual.

The majority of the timber design requirements in AASHTO are based on the *National Design Specification for Wood Construction (NDS).*[26] The NDS is the most widely recognized general specification for timber design and is published periodically by the National Forest Products Association. The specification includes design requirements and tabulated design values for sawn lumber, glulam, and timber piles. Although the NDS does not specifically address detailed bridge design, it does serve as the basis for the timber design concepts and requirements used for bridges. Notation of the NDS as the source of design requirements in this chapter reflects references in AASHTO that specify the NDS as the most current source of timber design information for bridges (AASHTO 13.1.1).

In addition to the NDS, AASHTO periodically references the specifications, standards, and technical publications of the American Institute of Timber Construction (AITC). AITC is the national technical trade association of the glulam industry and is responsible for numerous specifications and technical publications addressing fabrication, design, and construction of glulam. AITC also publishes *AITC 117-Design Standard Specifications for Structural Glued Laminated Timber of Softwood Species (AITC 117-Design),* which is the source of tabulated values for glulam.[4]

Timber design requirements for bridges may differ from those commonly used for buildings and other structures. Although the requirements in AASHTO are based on the NDS and other referenced specifications and standards, modifications have been incorporated in AASHTO to address specific bridge requirements. The designer should become familiar with the content and requirements of current AASHTO, NDS, and AITC specifications. Copies of these specifications and other noted references are available from the parent organizations at the addresses listed in Table 16-10.

5.3 DESIGN METHODS AND VALUES

Timber bridges are designed according to the principles of engineering mechanics and strength of materials, assuming the same basic linear elastic theory applied to other materials. The method used for design is the allowable stress design method, which is similar to service load design for structural steel. In this method, stresses produced by applied loads must be less than or equal to the allowable stresses for the material. A design method called load and resistance factor design (LRFD) is used for timber design in other countries, but not in the United States. Progress is being made toward development of such a method in the United States; however, adoption is several years away.

As discussed in Chapter 3, wood strength and stiffness vary with species, growth characteristics, loading, and conditions of use. As a result, one set of allowable design values for all species and design situations would result in very uneconomical design in most cases. Conversely, tabulated values for all potential conditions would result in so many tables that they would be unusable. Rather than using either of these approaches, timber design is based on published tabulated values that are intended for one set of standard conditions. When these conditions differ from those of the design application, the tabulated values are adjusted by modification factors to arrive at the allowable values used for each design. This approach produces more realistic design values for a specific situation. In general terms, the basic timber design sequence is as follows:

1. Compute load effects and select an initial member size and species.

2. Compute the *applied stress* from applied loads.

3. Obtain the *tabulated stress* published for the specific material.

4. Determine appropriate modification factors and other adjustments required for actual use conditions.

5. Adjust the tabulated stress to arrive at the *allowable stress* used for design.

6. Compare applied stress to allowable stress. The design is satisfactory when applied stress is less than or equal to allowable stress.

SYMBOLS AND ABBREVIATIONS

Timber design uses standard symbols to denote the types of stresses for strength properties. These symbols consist of a stress symbol to designate the type of stress (applied, tabulated, or allowable), followed by a lower case subscript to denote the specific strength property (bending, shear, tension, and so forth). The symbols used for this purpose are shown in Table 5-1. For example, applied, tabulated, and allowable bending stresses are designated f_b, F_b, and F_b', respectively. The same type of designation without the strength property subscript applies to modulus of elasticity, where E denotes the tabulated value and E' denotes the allowable value. For glulam, an additional subscript of x or y may be included to designate

Table 5-1.- Stress symbols for timber components.

Stress symbol	Definition
f	Applied stress from loading
F	Tabulated stress from the applicable design specifications
F'	Allowable stress for design (tabulated stress adjusted by all applicable modification factors)
F^*	Intermediate stress for calculating the tabulated stress for some beams or columns
Property subscript	**Definition**
b	Bending
v	Horizontal shear
t	Tension parallel to grain
c	Compression parallel to grain
$c\perp$	Compression perpendicular to grain
g	End grain in bearing

values about the *x-x* or *y-y* axis of the member (the *x-x* axis for glulam is always parallel to the wide face of the laminations). For example, F_{bx} is the tabulated bending stress about the *x-x* axis. In the absence of such a subscript, it is assumed that stresses act about the *x-x* axis.

TABULATED DESIGN VALUES

Tabulated design values for sawn lumber and glulam are based on testing and grading processes discussed in Chapter 3. These values represent the maximum permissible values for specific conditions of use and normally require adjustments for actual design conditions. In this sense, tabulated values should be viewed only as the basis or starting point for determining the allowable values to be used for design. An abbreviated summary of tabulated values for sawn lumber and glulam is published in AASHTO; however, these values do not include all species and grades and may not be current. For this reason, AASHTO requires that tabulated values comply with those specified in the most current edition of the NDS or AITC specifications (AASHTO 13.1.1 and 13.2.2). The source of tabulated values for sawn lumber is *Design Values for Wood Construction,* which is an integral part of the NDS, but is published as a separate volume. Tabulated values for glulam are given in *AITC 117-Design.* These NDS and AITC specifications represent the most comprehensive and current source of design information and include tabulated values for the following properties:

Bending (F_b)
Horizontal shear (F_v)
Tension parallel to grain (F_t)
Compression parallel to grain (F_c)
Compression perpendicular to grain ($F_{c\perp}$)

End grain in bearing (F_g)
Modulus of elasticity (E)

Tabulated Values for Sawn Lumber

Tabulated values for visually graded and machine stress rated (MSR) sawn lumber are published in the NDS based on the grading rules established by seven grading agencies. Separate tables are included for visually graded sawn lumber, MSR lumber, and end grain in bearing. The values are valid for sawn lumber used in dry applications under normal loading conditions (both of these conditions are discussed later for modification factors). In addition, each table contains an extensive set of footnotes for adjusting values to specific use conditions.

Visually Graded Sawn Lumber

Design values for visually graded sawn lumber are specified in Table 4A of the NDS. A portion of this table is shown in Table 5-2. The table gives tabulated values for F_b, F_t, F_v, F_c, $F_{c\perp}$, and E based on the species, size classification, and commercial grade of the lumber. When using the table, the following considerations will help interpret tabulated values:

1. Wood species may be specified as an individual species or a species combination. When species combinations are used, the individual species of the combination are listed in the Table 4A table of contents.
2. The grading rules agencies for each species are noted in the far right column of the tables. When grading rules for the same species differ among agencies, tabulated values are given separately for each grading agency.
3. Tabulated values for each species are based on the grade and size classification. Although commercial grade designations may be the same, tabulated values can vary among size classifications. For example, the tabulated values for grade No. 1 in the Beams and Stringers (B&S) size classification are not necessarily the same as those for No. 1 in the Posts and Timbers (P&T) size classification.
4. For all dimension lumber that is 2 to 4 inches thick, grading rules and commercial-grade nomenclature are standardized. When sawn lumber is thicker than 4 inches, grades are not standardized, and tabulated values for the same species, size, and grade of member may vary among grading agencies. In situations where conflicting tabulated values are given for different agencies, the designer must either specify the grading rules agency or use the lower tabulated values.
5. The availability of sawn lumber in the species, grade, and size classifications in Table 4A of the NDS may be geographically limited. The designer should verify availability before specifying a particular species, size, or grade.

Table 5-2. —Typical tabulated values for visually graded sawn lumber.

Species and Commercial Grade	Size Classification	Design values in pounds per square inch: Extreme fiber in bending "F_b": Single-member uses	Extreme fiber in bending "F_b": Repetitive-member uses	Tension parallel to grain "F_t"	Horizontal shear "F_v"	Compression perpendicular to grain "$F_{c\perp}$"	Compression parallel to grain "F_c"	Modulus of elasticity "E"	Grading rules agency
COTTONWOOD (Surfaced dry or surfaced green. Used at 19% max. m.c.)									
Stud	2" to 3" thick 2" to 4" wide	525	600	300	65	320	350	1,000,000	NHPMA (See footnotes 1–12)
Construction	2" to 4" thick 4" wide	675	775	400	65	320	650	1,000,000	
Standard		375	425	225	65	320	525	1,000,000	
Utility		175	200	100	65	320	350	1,000,000	
DOUGLAS FIR-LARCH (Surfaced dry or surfaced green. Used at 19% max. m.c.)									
Dense Select Structural	2" to 4" thick 2" to 4" wide	2450	2800	1400	95	730	1850	1,900,000	WCLIB WWPA (See footnotes 1–12 and 20)
Select Structural		2100	2400	1200	95	625	1600	1,800,000	
Dense No. 1		2050	2400	1200	95	730	1450	1,900,000	
No. 1		1750	2050	1050	95	625	1250	1,800,000	
Dense No. 2		1700	1950	1000	95	730	1150	1,700,000	
No. 2		1450	1650	850	95	625	1000	1,700,000	
No. 3		800	925	475	95	625	600	1,500,000	
Appearance		1750	2050	1050	95	625	1500	1,800,000	
Stud		800	925	475	95	625	600	1,500,000	
Construction	2" to 4" thick 4" wide	1050	1200	625	95	625	1150	1,500,000	
Standard		600	675	350	95	625	925	1,500,000	
Utility		275	325	175	95	625	600	1,500,000	
Dense Select Structural	2" to 4" thick 5" and wider	2100	2400	1400	95	730	1650	1,900,000	
Select Structural		1800	2050	1200	95	625	1400	1,800,000	
Dense No. 1		1800	2050	1200 (See Footnote 3)	95	730	1450	1,900,000	
No. 1		1500	1750	1000	95	625	1250	1,800,000	
Dense No. 2		1450	1700	775	95	730	1250	1,700,000	
No. 2		1250	1450	650	95	625	1050	1,700,000	
No. 3		725	850	375	95	625	675	1,500,000	
Appearance		1500	1750	1000	95	625	1500	1,800,000	
Stud		725	850	375	95	625	675	1,500,000	
Dense Select Structural	Beams and Stringers	1900	—	1100	85	730	1300	1,700,000	WCLIB (See footnotes 1–12 and 20)
Select Structural		1600	—	950	85	625	1100	1,600,000	
Dense No. 1		1550	—	775	85	730	1100	1,700,000	
No. 1		1300	—	675	85	625	925	1,600,000	
No. 2		875	—	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	—	1150	85	730	1350	1,700,000	
Select Structural		1500	—	1000	85	625	1150	1,600,000	
Dense No. 1		1400	—	950	85	730	1200	1,700,000	
No. 1		1200	—	825	85	625	1000	1,600,000	
No. 2		750	—	475	85	625	700	1,300,000	
Select Dex	Decking	1750	2000	—	—	625	—	1,800,000	
Commercial Dex		1450	1650	—	—	625	—	1,700,000	
Dense Select Structural	Beams and Stringers	1850	—	1100	85	730	1300	1,700,000	WWPA (See footnotes 1–13 and 20)
Select Structural		1600	—	950	85	625	1100	1,600,000	
Dense No. 1		1550	—	775	85	730	1100	1,700,000	
No. 1		1350	—	675	85	625	925	1,600,000	
Dense No. 2		1000	—	500	85	730	700	1,400,000	
No. 2		875	—	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	—	1150	85	730	1350	1,700,000	
Select Structural		1500	—	1000	85	625	1150	1,600,000	
Dense No. 1		1400	—	950	85	730	1200	1,700,000	
No. 1		1200	—	825	85	625	1000	1,600,000	
Dense No. 2		800	—	550	85	730	550	1,400,000	
No. 2		700	—	475	85	625	475	1,300,000	
Selected Decking	Decking	—	2000	—	—	—	—	1,800,000	
Commercial Decking		—	1650	—	—	—	—	1,700,000	
Selected Decking	Decking	—	2150	(Surfaced at 15% max. m.c. and used at 15% max. m.c.)			—	1,900,000	
Commercial Decking		—	1800				—	1,700,000	

Refer to the latest edition of the NDS for a complete and current listing of tabulated values and footnote explanations. From NDS,[24] © 1986. Used by permission.

MSR Lumber

For MSR lumber, tabulated values are derived by nondestructive stiffness testing of individual pieces that are 2 inches thick or less. Values are specified in Table 4B of the NDS for F_b, F_t, F_c, and E based on the grade designation and size classification of lumber (Table 5-3). Tabulated stresses for F_v and $F_{c\perp}$ are as specified in NDS Table 4A for No. 2 visually graded sawn lumber of the appropriate species.

End Grain in Bearing

The NDS contains a separate table of tabulated stress for end grain in bearing, F_g. These values are specified in Table 2B of the main NDS volume and pertain only to end-grain bearing parallel to grain on a rigid surface. The stresses are given for each species based on member size and use conditions and apply to both visually graded and MSR lumber.

Table 5-3. -Typical tabulated values for MSR sawn lumber.

			Design values in pounds per square inch[10]				
			Extreme fiber in bending "F_b"[8]				
Grade designation[11]	Grading rules agency (see footnotes 1,2,3,4)	Size classification	Single-member uses	Repetitive-member uses	Tension parallel to grain "F_t"	Compression parallel to grain "F_c"	Modulus of elasticity "E"
900f-1.0E	3,4	Machine rated lumber 2" thick or less All Widths	900	1050	350	725	1,000,000
1200f-1.2E	1,2,3,4,		1200	1400	600	950	1,200,000
1350f-1.3E	2,3,4		1350	1550	750	1075.[12]	1,300,000
1450f-1.3E	1,3,4		1450	1650	800	1150	1,300,000
1500f-1.3E	2		1500	1750	900	1200	1,300,000
1500f-1.4E	1,2,3,4		1500	1750	900	1200	1,400,000
1650f-1.4E	2		1650	1900	1020	1320	1,400,000
1650f-1.5E	1,2,3,4		1650	1900	1020	1320	1,500,000
1800f-1.6E	1,2,3,4		1800	2050	1175	1450	1,600,000
1950f-1.5E	2		1950	2250	1375	1550	1,500,000
1950f-1.7E	1,2,4		1950	2250	1375	1550	1,700,000
2100f-1.8E	1,2,3,4		2100	2400	1575	1700	1,800,000
2250f-1.6E	2		2250	2600	1750	1800	1,600,000
2250f-1.9E	1,2,4		2250	2600	1750	1800	1,900,000
2400f-1.7E	2		2400	2750	1925	1925	1,700,000
2400f-2.0E	1,2,3,4		2400	2750	1925	1925	2,000,000
2550f-2.1E	1,2,4		2550	2950	2050	2050	2,100,000
2700f-2.2E	1,2,3,4		2700	3100	2150	2150	2,200,000
2850f-2.3E	2,4		2850	3300	2300	2300	2,300,000
3000f-2.4E	1,2		3000	3450	2400	2400	2,400,000
3150f-2.5E	2		3150	3600	2500	2500	2,500,000
3300f-2.6E	2		3300	3800	2650	2650	2,600,000
900f-1.0E	1,2,3	See footnote 5	900	1050	350	725	1,000,000
900f-1.2E	1,2,3		900	1050	350	725	1,200,000
1200f-1.5E	1,2,3		1200	1400	600	950	1,500,000
1350f-1.8E	1,2		1350	1550	750	1075	1,800,000
1500f-1.8E	3		1500	1750	900	1200	1,800,000
1800f-2.1E	1,2,3		1800	2050	1175	1450	2,100,000

Refer to the latest edition of the NDS for a complete and current listing of tabulated values and footnote explanations. From the NDS;[24] © 1986. Used by permission.

Tabulated Values for Glued-Laminated Timber (Glulam)

Tabulated values for glulam are specified in *AITC 117-Design.* Separate tables are included for bending combinations, axial combinations, and end grain in bearing. Values are given for western species and Southern Pine made with either visually graded or E-rated lumber based on dry-use conditions (moisture content of 16 percent or less) and normal duration of load. Tabulated values for a specific combination symbol of glulam are standardized and are not subject to variations in grading rules or fabrication processes.

Bending Combinations

For bending combinations, tabulated values are given in Table 1 of *AITC 117-Design.* The combination symbols in this table are for members consisting of four or more laminations, stressed primarily in bending with loads applied perpendicular to the wide faces of the laminations (*x-x* axis). The table also includes tabulated values for axial loading and bending with loads applied parallel to the wide faces of the laminations (*y-y* axis); however, the axial combinations are usually better suited for these loading conditions. A limited number of combination symbols, taken from Table 1 from *AITC 117-Design,* are shown in Table 5-4. The first two columns of the table give the combination symbol and species of the member. The remainder of the table is divided into three parts based on the type and direction of applied stress. Columns 3 to 8 contain stresses for members loaded in bending about the *x-x* axis (the most common case). For this condition, stresses for F_b and $F_{c\perp}$ are specified separately for the tension and compression zones of the member. These stresses may be the same for both zones (balanced combination) or may differ significantly. Columns 9 to 13 are for members loaded in bending about the *y-y* axis where stresses in the tension and compression zones are equal. Columns 14 to 16 are for members loaded axially or with a combination of axial and bending loads. The intended use and limitations for groups of combinations are also noted in the table.

Axial Combinations

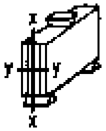
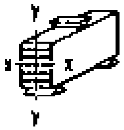
Tabulated values for axial combinations are specified in Table 2 of *AITC 117-Design.* The combinations in this table are intended primarily for members loaded axially or in bending with loads applied parallel to the wide faces of the laminations (*y-y* axis). The table also includes tabulated values for loading perpendicular to the wide faces of the laminations (*x-x* axis), but bending combinations are usually better suited for this condition. A limited number of combination symbols, taken from Table 2 from *AITC 117-Design,* are shown in Table 5-5. The table is organized in three sections based on the type and direction of applied stresses, as in Table 5-4. Tabulated values depend on the number of laminations and are given for members consisting of 2, 3, and 4 or more laminations. For all axial combinations, strength properties are balanced about the neutral axis, and tabulated stresses for F_b and $F_{c\perp}$ are equal in the tension and compression zones.

Table 5-4.- Typical tabulated values for glulam bending combinations.

Combination Symbol [d]	Species Outer Laminations/Core Laminations [e]	Bending About X-X Axis — Loaded Perpendicular to Wide Faces of Laminations — Extreme Fiber in Bending, F_{bx} — Tension Zone Stressed in Tension [f,v] psi	Compression Zone Stressed in Tension [g] psi	Compression Perpendicular to Grain, $F_{c\perp x}$ [u] — Tension Face psi	Compression Face psi	Horizontal Shear, F_{vx} psi	Modulus of Elasticity, E_x $\times 10^6$ psi	Bending About Y-Y Axis — Loaded Parallel to Wide Faces of Laminations — Extreme Fiber in Bending, F_{by} [f,r] psi	Compression Perpendicular to Grain $F_{c\perp y}$ [v] psi	Horizontal Shear, F_{vy} psi	Horizontal Shear, F_{vy} psi (For members with multiple piece laminations which are not edge glued) [q] psi	Modulus of Elasticity, E_y $\times 10^6$ psi	Axially Loaded — Tension Parallel to Grain, F_t psi	Compression Parallel to Grain, F_c psi	Modulus of Elasticity, E $\times 10^6$ psi
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Visually Graded Western Species															
The following four combinations are not balanced and are for either dry or wet use.															
16F-V1	DF/WW			560 [h,l]	560 [h,l]	140 [s,w]	1.3 [x]	950	255	130 [s,w]	65 [s,w]	1.1 [x]	675	975	1.1 [x]
16F-V2	HF/HF			500 [l]	375 [l]	155	1.4	1250	375	135	70	1.3	875	1300	1.3
16F-V3	DF/DF	1600	800	560 [h,l]	560	165	1.5	1450	560	145	75	1.5	950	1550	1.5
16F-V8	DFS/DFS			650	500	165	1.2	1200	500	145	75	1.1	825	1350	1.1
The following two combinations are intended for straight or slightly cambered members for dry use and industrial appearance. [k]															
16F-V4	DF/N3WW			650	560 [h]	90 [l,s,w]	1.5 [x]	900	255	130 [s,w]	65 [s,w]	1.3 [x]	650	600	1.3 [x]
16F-V5	HF/N3DF	1600	800	650	560 [h]	90 [m]	1.6	1000	470	135	70	1.5	750	875	1.5
The following two combinations are balanced and are intended for members continuous or cantilevered over supports and provide equal capacity in both positive and negative bending.															
16F-V4	DF/DF			560 [h,l]	560 [h]	165	1.5	1450	560	145	75	1.4	950	1550	1.5
16F-V5	HF/HF	1600	1600	375 [l]	375 [l]	155	1.4	1200	375	135	70	1.3	850	1350	1.3
The following seven combinations are not balanced and are for either dry or wet use.															
20F-V1	DF/WW			650	560 [h]	140 [s,w]	1.4 [x]	1000	255	130 [s,w]	65 [s,w]	1.2 [x]	750	1000	1.2 [x]
20F-V2	HF/HF			500 [l]	375 [l]	155	1.5	1200	375	135	70	1.4	950	1350	1.4
20F-V3	DF/DF			650	560 [h]	165	1.6	1450	560	145	75	1.5	1000	1550	1.5
20F-V4	DF/DF	2000	1000	590 [h,l]	560 [h]	165	1.6	1450	560	145	75	1.6	1000	1550	1.6
20F-V10	DF/HF			650	560	155	1.5	1300	375	135	70	1.4	950	1500	1.4
20F-V11	DFS/DFS			650	500	165	1.3	1400	500	145	75	1.1	900	1400	1.1
20F-V12	AC/AC			560	560	190	1.5	1200	470	165	80	1.4	900	1500	1.4
The following two combinations are intended for straight or slightly cambered members for dry use and industrial appearance. [k]															
20F-V5	DF/N3WW			650	560 [h]	90 [l,s,w]	1.6 [x]	1000	255	135 [s,w]	70 [s,w]	1.3 [x]	750	725	1.3 [x]
20F-V6	DF/N3DF	2000	1000	650	560 [h]	90 [m]	1.6	1000	470	135	70	1.5	775	900	1.5
The following three combinations are balanced and are intended for members continuous or cantilevered over supports and provide equal capacity in both positive and negative bending															
20F-V7	DF/DF			650	650	165	1.6	1450	560	145	75	1.6	1000	1600	1.6
20F-V8	DF/DF	2000	2000	590 [h,l]	590 [h,l]	165	1.7	1450	560	145	75	1.6	1000	1600	1.6
20F-V9	HF/HF			500 [l]	500 [l]	155	1.5	1400	375	135	70	1.4	975	1400	1.4
The following five combinations are not balanced and are for either dry or wet use.															
22F-V1	DF/WW			650	560 [h]	140 [s,w]	1.6 [x]	1050	255	130 [s,w]	65 [s,w]	1.3 [x]	850	1100	1.3 [x]
22F-V2	HF/HF			500 [l]	500 [l]	155	1.5	1250	375	135	70	1.4	950	1350	1.4
22F-V3	DF/DF	2200	1100	650	560 [l]	165	1.7	1450	560	145	75	1.6	1050	1500	1.6
22F-V4	DF/DF			650	560 [h]	165	1.7	1450	560	145	75	1.6	1050	1550	1.6
22F-V10	DF/DFS			650	560 [h]	165	1.6	1600	500	145	75	1.3	1000	1400	1.3
The following two combinations are intended for straight or slightly cambered members for dry use and industrial appearance. [x]															
22F-V5	DF/N3WW			650	560 [h]	90 [l,s,w]	1.6 [x]	1100	255	135 [s,w]	75 [s]	1.4 [x]	800	725	1.4 [x]
22F-V6	DF/N3DF	2200	1100	650	560 [h]	90 m	1.7	1250	470	135	75	1.6	900	925	1.6
The following three combinations are balanced and are intended for members continuous or cantilevered over supports and provide equal capacity in both positive and negative bending.															
22F-V7	DF/DF			650	650	165	1.8	1450	560	145	75	1.6	1100	1650	1.6
22F-V8	DF/DF	2200	2200	590 [h,l]	590 [h,l]	165	1.7	1450	560	145	75	1.6	1050	1650	1.6
22F-V9	HF/HF			500 [l]	500 [l]	165	1.5	1250	375	135	70	1.4	975	1400	1.4
The following six combinations are not balanced and are for either dry or wet use															
24F-V1	DF/WW			650	650	140 [s,w]	1.7 [x]	1250	255	135 [s,w]	70 [s]	1.4 [x]	950	1300	1.4 [x]
24F-V2	HF/HF			500 [l]	500 [l]	155	1.5	1250	375	135	70	1.4	950	1300	1.4
24F-V3	DF/DF	2400	1200	650	560 [h]	165	1.8	1500	560	145	75	1.6	1100	1600	1.6
24F-V4	DF/DF			650	650	165	1.8	1500	560	145	75	1.6	1100	1650	1.6
24F-V5	DF/HF			650	650	155	1.7	1350	375	140	70	1.5	1100	1450	1.5
24F-V11	DF/DFS			650	560 [h]	165	1.7	1600	500	145	75	1.4	1150	1700	1.4

Refer to the latest edition of *AITC 117—Design* for a complete and current listing of tabulated values and footnote explanations. From *AITC 117—Design*;[4] © 1987. Used by permission.

Table 5-5. Typical tabulated values for glulam axial combinations.

					Axially Loaded			Bending about Y-Y Axis							Bending About X-X Axis		
					Tension Parallel to Grain, F_t	Compression Parallel to Grain, F_c		Loaded Parallel to Wide Faces of Laminations 							Loaded Perpendicular to Wide Faces of Laminations 		
			Modulus of Elasticity, E	Compression Perpendicular to Grain, $F_{c\perp}$ [n]				Extreme Fiber in Bending[f], F_{by}			Horizontal Shear, F_{vy}				Extreme Fiber In Bending[h], F_{bx}		Horizontal Shear,[g] F_{vx}
Combination Symbol	Species[d]	Grade[e]	$\times 10^6$ psi	psi	2 or More Lams. psi	4 or More Lams. psi	2 or 3 Lams. psi	4 or More Lams. psi	3 Lams psi	2 Lams. psi	4 or More Lams. (For members with multiple piece lams)[m] psi	4 or More Lams. psi	3 Lams. psi	2 Lams. psi	2 Lams. to 15 in. Deep[i] psi	4 or More Lams.[i,q] psi	2 or More Lams. psi
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Visually Graded Western Species																	
1	DF	L3	1.5	560[k]	900	1550	1200	1450	1250	1000	75	145	135	125	1250	1500	165
2	DF	L2	1.7	560[k]	1250	1900	1600	1800	1600	1300	75	145	135	125	1700	2000	165
3	DF	L2D	1.8	650	1450	2300	1850	2100	1850	1550	75	145	135	125	2000	2300	165
4	DF	L1CL	1.9	590[k]	1400	2100	1900	2200	2000	1650	75	145	135	125	1900	2200	165
5	DF	L1	2.0	650	1600	2400	2100	2400	2100	1800	75	145	135	125	2200	2400	165
6	DF	N3C	1.4	470	350	875	550	550	550	550	60	120	115	105	450	—	140
7	DF	N3M	1.5	560	900	1550	700	1450	1250	1000	75	145	135	125	1000	—	165
8	DF	N2	1.6	560[k]	1000	1550	1150	1600	1550	1300	75	145	135	125	1350	1600	165
9	DF	N2D	1.8	650	1150	1800	1350	1850	1800	1500	75	145	135	125	1600	1850	165
10	DF	N1	1.8	560[k]	1300	1950	1450	1950	1750	1500	75	145	135	125	1750	2100	165
11	DF	N1D	2.0	650	1500	2300	1700	2300	2100	1750	75	145	135	125	2100	2400	165
12	DF	SS	1.8	560[k]	1400	1950	1650	2100	1950	1650	75	145	135	125	1900	2200	165
13	DF	SSD	2.0	650	1600	2300	1950	2400	2300	1950	75	145	135	125	2200	2400	165
14	HF	L3	1.3	375[k]	800	1100	975	1200	1050	850	70	135	130	115	1100	1300	155
15	HF	L2	1.4	375[k]	1050	1350	1300	1500	1350	1100	70	135	130	115	1450	1700	155
16	HF	L1	1.6	375[k]	1200	1500	1450	1750	1550	1300	70	135	130	115	1600	1900	155
17	HF	L1D	1.7	500	1400	1750	1700	2000	1850	1550	70	135	130	115	1900	2200	155
18	HF	N3	1.3	375	425	900	575	700	700	700	70	135	130	115	575	—	155
19	HF	N2	1.4	375[k]	850	1300	975	1350	1300	1100	70	135	130	115	1150	1350	155
20	HF	N1	1.6	375[k]	975	1450	1250	1550	1500	1250	70	135	130	115	1350	1550	155
21	HF	SS	1.6	375[k]	1100	1450	1350	1750	1650	1400	70	135	130	115	1500	1750	155
22	WW	L3	1.0	255[o]	525	850	675	800	700	550	60[p]	120	115	105	725	850	140
23	WW	N3	1.0	255[o]	275	625	450	450	450	450	60[p]	120	115	105	400	—	140
24	WW	N2	1.1	255[o]	550	900	700	900	875	725	60[c]	120	115	105	775	900	140
25	WW	N1	1.2	255[o]	650	1000	875	1050	1000	850	60[p]	120	115	105	875	1050	140
26	WW	SS	1.2	255[o]	750	1000	1000	1150	1100	925	60[p]	120	115	105	1000	1150	140
59	DFS	L3	1.1	500[k]	800	1400	1050	1200	1050	850	75	145	135	125	1050	1250	165
60	DFS	L2	1.3	500[k]	1050	1750	1400	1750	1550	1150	75	145	135	125	1450	1700	165
61	DFS	L1	1.5	650	1350	2200	1850	2000	1800	1500	75	145	135	125	1850	2200	165
69	AC	L3	1.3	470	700	1150	1150	1000	875	700	80	165	160	140	1000	1150	190
70	AC	L2	1.4	470	1000	1450	1550	1250	1100	925	80	165	160	140	1350	1550	190
71	AC	L1D	1.7	560	1250	1900	2050	1650	1500	1250	80	165	160	140	1700	2000	190
72	AC	L1S	1.7	560	1250	1900	2050	1650	1500	1250	80	165	160	140	1700	2000	190

Refer to the latest edition of ***AITC 117—Design*** for a complete and current listing of tabulated values and footnote explanations. From ***AITC 117—Design;***[4] ©1987. Used by permission

End Grain in Bearing

Tabulated stress for end grain in bearing parallel to grain *(F_g)* is given in Annex A of *AITC 117-Design.* Annex A consists of Tables A-1 and A-2, which specify F_g for bending combinations and axial combinations, respectively. In both tables, F_g is specified by a combination symbol where member bearing is on the full cross section and where bearing is on a partial cross section.

ADJUSTMENTS TO TABULATED DESIGN VALUES

Tabulated values for sawn lumber and for glulam are based on the standard conditions noted in the applicable design tables. When actual use conditions vary from these standard conditions, tabulated values must be adjusted to compensate for (1) differences between the assumptions used to establish tabulated values and actual use conditions, (2) variations in wood behavior related to the type of stress or member orientation, and (3) differences between the physical or mechanical behavior of wood and that of an ideal material assumed in most equations of engineering mechanics.

Requirements for adjusting tabulated values are given in the text of the design specifications (AASHTO, NDS, and *AITC 117-Design)* and as footnotes to tabulated values. The type and magnitude of the adjustments, as well as the manner in which they are applied, vary with the type of material, strength property, and design application. Most adjustments are applied as modification factors that are multiplied by the tabulated values. These modification factors are designated by the letter C, followed by a subscript to denote the type of modification. They include the following:

C_M moisture content factor	C_L lateral stability of beams factor
C_D duration of load factor	C_P lateral stability of columns factor
C_t temperature factor	C_R fire-retardant treatment factor
C_f form factor	C_C curvature factor
C_F size factor	C_I interaction stress factor

Modification factors are applied to tabulated values only, not to applied stresses or loads. In most cases they are cumulative; however, in some cases the more restrictive value of two factors is used. A summary of the applicability of modification factors to various wood properties is given in Table 5-6. The factors C_C and C_I apply to curved and taper-cut glulam beams, respectively, and are not discussed in this chapter. Refer to the AITC *Timber Construction Manual* for additional information on these factors.[6]

Moisture Content Factor C_M)

The strength and stiffness of wood decrease as moisture content increases. To compensate for this effect, tabulated values are adjusted by C_M. This factor, which is also referred to as a wet-use factor or condition-of-use

Table 5-6.- Applicability of modification factors for strength properties and modulus of elasticity.

	Modification factor[1]							
	Duration of load[2]	Moisture content	Temperature	Fire retardant	Size factor	Stability of beams	Form factor	Stability of columns
Design value	C_D	C_M	C_t	C_R	C_F	C_L	C_f	C_P
Bending, F_b	X	X	X	X	X	X	X	—
Tension parallel to grain, F_t	X	X	X	X	—	—	—	—
Compression parallel to grain, F_c	X	X	X	X	—	—	—	X
Compression perpendicular to grain, $F_{c\perp}$	—	X	X	X	—	—	—	—
Horizontal shear, F_v	X	X	X	X	—	—	—	—
End-grain bearing, F_g	X	X	X	X	—	—	—	—
Modulus of elasticity, E	—	X	X	X	—	—	—	—

[1] Factors are not always cumulative.
[2] The duration of load factor for impact does not apply to members pressure-impregnated with preservative salts to the heavy retentions required for marine exposure, or to sawn lumber treated with fire-retardant chemicals.
X = modification factor is applicable.
— = modification factor does not apply.

factor, is applicable to all tabulated values for strength and modulus of elasticity. It adjusts values for changes in strength and stiffness and compensates for variations in cross section caused by shrinkage.

Application of C_M differs for sawn lumber and glulam. For sawn lumber, tabulated values are based on the moisture content specified for each species in the NDS tables. With the exception of Southern Pine and Virginia Pine-Pond Pine, adjustment by C_M is applied when the moisture content of the member in service is expected to exceed 19 percent. For Southern Pine and Virginia Pine-Pond Pine, the C_M adjustment is not required because tabulated values are given in the design tables for three in-service moisture contents. These tabulated values already include the C_M adjustment, and no further adjustment for moisture is required. Values of C_M for all other lumber species are given in the footnotes to the design tables and depend on the member size and specific strength property (Table 5-7).

For glulam, all tabulated values in *AITC 117-Design* are based on a moisture content in service of 16 percent or less. When the moisture content in service is expected to be 16 percent or higher, tabulated values must be multiplied by the wet-use factors given in the design tables. Factor C_M for glulam depends on the strength property only and is independent of species, combination symbol, and member size. Values of C_M for glulam are given in Table 5-7.

In most applications, bridge members are exposed to the weather and should be adjusted by C_M for wet-use conditions. In cases where beams are protected by a waterproof deck, design for dry conditions may be appropriate, as discussed in Chapter 7.

Table 5-7. - Values of the moisture content factor C_M for sawn lumber and glulam.

Property	C_M values						
	F_b	F_t	F_c	$F_{c\perp}$	F_v	F_g	E
Sawn lumber; all species except Southern Pine and Virginia Pine-Pond Pine[a]							
All thicknesses surfaced dry or surfaced green and used at 19% maximum moisture content	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Nominal 4 inches or less in thickness, surfaced green or dry and used at a moisture content greater than 19%	0.86	0.84	0.70	0.67	0.97	—[b]	0.97
Nominal 4 inches or less in thickness, surfaced green or dry and used at a moisture content of 15% or less[c]	1.08	1.08	1.17[d]	1.00	1.05	—[b]	1.05[d]
Nominal 5 inches and thicker used where moisture content exceeds 19%	1.00	1.00	0.91	0.67	1.00	—[b]	1.00
Glulam							
Used at moisture contents of 16% or less (dry conditions of use)	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Used at moisture contents greater than 16% (wet conditions of use)	0.80	0.80	0.73	0.53	0.875	0.57	0.833

[a] Refer to the NDS[24] for adjusted tabulated values for Southern Pine and Virginia Pine-Pond Pine.
[b] Use tabulated values for wet-use conditions given in Table 2 of the NDS.[26]
[c] Refer to the NDS[24] for decking graded to WWPA rules that is surfaced at 15 percent maximum moisture content and used where the moisture content will exceed 15 percent for an extended period of time.
[d] For Redwood, use 1.15 for compression parallel to grain and 1.04 for modulus of elasticity.

Duration of load Factor *(C_D)*

Wood is capable of withstanding much greater loads for short durations than for long periods. This is particularly significant in bridge design where short-term increased loads from vehicle overloads, wind, earthquake, or railing impact must be considered. The tabulated values for sawn lumber and glulam are based on an assumed *normal* duration of load. In this case, a normal duration of load is based on the expectation that members will be stressed to the maximum stress level (either continuously or cumulatively) for a period of approximately 10 years, stressed to 90 percent of the maximum design level continuously for the remainder of the life of the structure, or both. This maximum stress is assumed to occur during the life of the member as a result of either continuous loading or a series of shorter duration loads that total 10 years. When the maximum design loads act for durations that are shorter or longer than these assumed durations, tabulated stresses are adjusted by C_D, (Table 5-8). Factor C_D applies to tabulated strength properties but does not apply to compression perpendicular to grain ($F_{c\perp}$) or modulus of elasticity *(E)*. In most bridge

Table 5-8. Modification factors for duration of load.

Load duration	Duration of load factor C_D
2 months (as for snow and ice)	1.15
7 days (as for snow and ice)	1.25
Wind or earthquake	1.33
5 minutes (rail loads only)	1.65[a]

[a] The duration of load factor for impact does not apply to members pressure-impregnated with preservative salts to the heavy retentions required for marine exposure, or sawn lumber treated with fire-retardant chemicals.

From *AASHTO* Section 13.2.5.1:[1] © 1983. Used by permission.

applications, the permanent load of the structure is small in relation to vehicle loads, and a decrease in tabulated stresses for permanent loading is not necessary

The stresses produced in bridge members are commonly the result of a combination of loads rather than a single load (Chapter 6). For a combination of loads of different durations, C_D for the entire group is the single value associated with the shortest load duration. When applying C_D, the designer must recognize that for a given combination of loads, the most restrictive allowable stress may result from a partial combination involving loads of longer duration. The individual loads in a load combination must be evaluated in various combinations, with the value of C_D depending on the load of shortest duration for that combination. This is accomplished by progressively eliminating the load of shortest duration from the group and applying C_D for the load of next-shortest duration. In other words, the resulting size or capacity of a member required for a load combination must not be less than that required for a partial combination of the longer-duration loads. Application of C_D is discussed in more detail in Appendix B of the NDS and in Chapter 6. Duration of load is generally not applicable in bridge design, except for the design of railing systems.

Temperature Factor *(C_t)*

The strength and stiffness of wood increases as it cools and decreases as it warms. These changes in strength because of temperature occur immediately and depend on the magnitude of the temperature change and the moisture content of the wood. For temperatures up to approximately 150 °F, the immediate effects of strength loss are reversible, and the member will essentially recover its initial strength levels as the temperature is lowered. Prolonged exposure to temperatures higher than 150 °F may cause a permanent and irreversible loss in member strength.

Tabulated design values for sawn lumber and glulam assume that members will be used in normal temperature applications and may occasionally be heated to temperatures up to 150 °F. This applies to most bridge design situations. In cases where a member may be periodically exposed to elevated temperatures, humidity is generally low, and the increase in member strength that results from reduced moisture tends to offset the reduction in strength that results from temporary temperature increases. The design specifications do not require a mandatory adjustment to tabulated values for temperature effects, and as a general rule, none are warranted. In cases where members will be exposed to prolonged temperatures in excess of 150 °F, or will be used at very low temperatures for the entire design life, the modification factor, C_t, given in Table 5-9, may be applied at the discretion of the designer.

Table 5-9. - Temperature factor C_t given as a percentage increase or decrease in design values for each 1 °F decrease or increase in temperature.

Property	Moisture content[1]	Cooling[2] below 68 °F (Min. -100 °F)	Heating above 68 °F (Max. 150 °F)
Modulus of elasticity and tension parallel to grain	0%	+0.09%	–0.11%
	12%	+0.13%	–0.13%
	24%	+0.38%	–0.15%
Other properties and fastenings[2]	0%	+0.14%	–0.19%
	12%	+0.24%	–0.38%
	24%	+0.84%	–0.57%

[1] In-service (equilibrium) moisture content at design temperature.
[2] The effect of low temperatures on the ductility of metal fasteners should be considered.

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Fire-Retardant Treatment Factor *(C_R)*

Fire-retardant treatments are seldom used on bridge members and are unnecessary in most applications. For those situations where fire-retardant chemicals are considered necessary, tabulated values must be adjusted by the fire-retardant treatment factor C_R. The value for this factor depends on specific strength properties and is different for sawn lumber and glulam. C_R is given for sawn lumber in Table 2A of the NDS (Table 5-10). The basis for these values and treatment qualifications are outlined in Appendix Q of the NDS. C_R for glulam depends on the species and treatment combinations involved. The effects on strength properties must be determined for each treatment. However, indications are that 10 to 25 percent reductions in bending strength are applicable.[4,6] The treatment manufacturer should be contacted for more specific C_R values for glulam based on the specific material and design application.

Table 5-10.- Fire-retardant treatment factor for structural lumber.

Property	C_R
Extreme fiber in bending	0.85
Tension parallel to grain	0.80
Horizontal shear	0.90
Compression perpendicular to grain	0.90
Compression parallel to grain	0.90
Modulus of elasticity	0.90
Fastener design loads	0.90

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Size Factor (C_F)

Tabulated bending stresses are based on a square or rectangular member 12 inches deep in the direction of applied loads. For member depths greater than 12 inches, F_b must be adjusted by C_F, as computed by

$$C_F = \left(\frac{12}{d}\right)^{1/9} \tag{5-1}$$

where d is the member depth in inches.

For sawn lumber, C_F does not apply to MSR lumber or to visually graded lumber 2 to 4 inches thick used edgewise. For glulam, the C_F value computed by the above equation is based on a uniformly distributed load on a simply supported beam with a span to depth ratio L/d = 21. In most bridge applications, these assumptions result in reasonable accuracy as variations in loading and L/d result in relatively small deviations in the size factor. In cases where greater accuracy is warranted, C_F may be adjusted for other L/d ratios or loading conditions by the percentages in Table 5-11.

The effect of the size factor for both sawn lumber and glulam is to reduce the tabulated bending stress for members more than 12 inches deep. For members less than 12 inches deep, footnotes to design tables allow an increase in bending stress for sawn lumber members 2 to 4 inches thick used flatwise,[24] and glulam members loaded parallel to the wide faces of the laminations.[4] C_F is generally cumulative with other modification factors, but is normally not cumulative with the lateral stability of beams factor, C_L (see Sections 5.4 and 5.7).

Equation 5-1, used for computing size factor, is being reevaluated for glulam, and alternate forms of the equation are being considered by several industry-related technical committees. Thus, the designer should be aware of the potential for future revisions and refer to the latest editions of the NDS and *AITC 117-Design* for current requirements.

Table 5-11. —Adjustments to C_r for various span-to-depth ratios and loading conditions.

Span-to-depth ratio (L/d)[a]	% change
7	+6.3
14	+2.3
21	0
28	–1.6
35	–2.8
Loading condition for simply supported beams	**% change**
Single concentrated load	+7.8
Uniform load	0
Third point load	–3.2

[a] Use straight line interpolation for other L/d ratios.

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Lateral Stability of Beams Factor (C_L)

The lateral stability of beams factor, C_L, is applied to some bending members where the compressive stress in bending must be limited to prevent lateral buckling. Additional details on the use of C_L are discussed in Section 5.4.

Form Factor (C_f)

Tabulated bending stresses are based on members with a square or rectangular cross section loaded normal to one or more faces. For other member shapes, specifically round or diamond sections, stresses must be modified by the form factor, C_f. C_f does not apply to rectangular or square members and is not commonly used in bridge applications. Refer to the NDS for additional information on the use of C_f.

Lateral Stability of Columns Factor (C_P)

The lateral stability of columns factor, C_P, is applied to some compression members where the compressive stress must be limited to prevent lateral buckling. Additional details on the use of C_P are discussed in Section 5.6.

5.4 BEAM DESIGN

A beam is a structural component with loads applied transversely to the longitudinal axis. In bridge design, beams are the most frequently used structural components. The three most common bridge beams are girders, stringers, and floorbeams. Girders are large beams (normally glulam) that provide primary superstructure support, most often in beam-type superstructures. Stringers are longitudinal beams that support the bridge deck.

They are generally smaller than girders, but there is no clear size definition for either. Floorbeams are transverse beams that directly support the bridge deck or support longitudinal stringers that support the deck. In addition to girders, stringers, and floorbeams, other bridge components are designed as beams, including components of the deck and railing systems.

Beam design involves the analysis of member strength, stability, and stiffness for four basic criteria: (1) bending (including lateral stability), (2) deflection, (3) horizontal shear, and (4) bearing. Of these four criteria, bending, deflection, and shear can directly control member size, while bearing will influence the design of supports. Initial beam design is normally based on bending, then checked for deflection and shear. After an appropriate beam size is determined, bearing stresses are checked at supports to ensure sufficient bearing area.

Beam design requirements discussed in this section are limited to straight or slightly curved (cambered) solid rectangular beams of constant cross-sectional area. Refer to the NDS for design requirements for other beam configurations and shapes and for beams with notches or cutouts. The design of beams loaded in combined bending and axial tension or compression is discussed in Section 5.7.

DESIGN FOR BENDING

Beam design must consider the strength of the material in bending and the potential for lateral buckling from induced compressive stress. For positive and negative bending, compression stress occurs in the top and bottom portions of the beam, respectively. Single, simple spans are subjected to positive bending moments only, while multiple continuous spans and cantilevers will be subjected to both positive and negative moments. This distinction is particularly important for stability considerations, and also when the allowable stresses for positive and negative bending are different, as in some combination symbols of glulam beams.

Initial beam design is somewhat of a trial-and-error process. A beam size is first estimated, and applied stress is computed and checked against the allowable stress in bending. After a suitable beam is determined from strength requirements, it must be verified for lateral stability.

Applied Stress

Applied bending stress in timber beams is determined by the standard formulas of engineering mechanics assuming linear elastic behavior. Stress at extreme fiber in bending, f_b is computed by

$$f_b = \frac{M}{S} \tag{5-2}$$

where M = moment due to applied loads (in-lb), and

S = section modulus of the beam (in^3).

Section modulus values for standard sizes of sawn lumber and glulam are given in Chapter 16.

Lateral Stability and Beam Slenderness

Beams develop compressive stress from induced bending forces. If compression areas are not restrained from lateral movement and rotation, the member may buckle laterally at a bending stress considerably lower than that normally allowed for the material. The potential for lateral buckling depends on the magnitude of applied loads, beam dimensions, and the effectiveness and frequency of lateral restraint. Lateral stability is most critical in long slender beams with a high depth-to-width ratio. It is not critical in beams where the width of the beam exceeds its depth.

One of the primary factors affecting beam lateral stability is the distance between points of lateral support along the beam length. In bridge applications, lateral support is generally provided by cross frames, solid wood diaphragms, or framing connections that prevent beam rotation and lateral displacement (Figure 5-1). The distance between such points of lateral support is termed the unsupported length, or ℓ_u. When the compression edge is continuously supported along its length, ℓ_u is zero. For all other configurations, ℓ_u is simply the distance between cross frames, diaphragms, or bracing that prevent beam rotation and lateral displacement.

The basis for stability design in beams is the beam slenderness factor C_s, given by

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} \leq 50 \tag{5-3}$$

where ℓ_e = effective beam length (in.),

d = beam depth (in.), and

b = beam width (in.).

The effective beam length ℓ_e in Equation 5-3 depends on the beam configuration and loading condition (Figure 5-2). For a single-span beam with a concentrated load at the center, ℓ_u is computed by

$$\ell_e = 1.37\ell_u + 3d \tag{5-4}$$

For a single-span beam with a uniformly distributed load, ℓ_e is computed by

$$\ell_e = 1.63\ell_u + 3d \tag{5-5}$$

For a single-span beam, or cantilever beam, with any load, ℓ_e is computed by

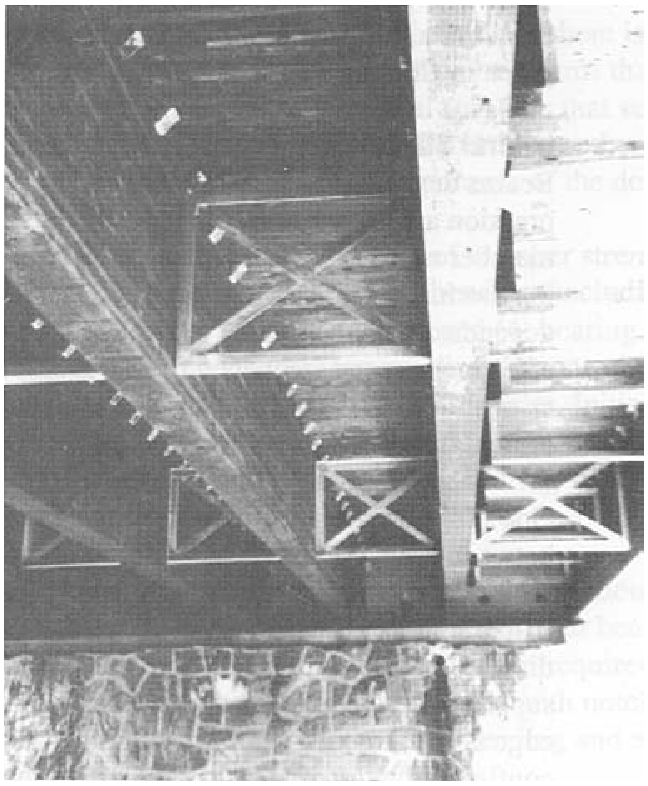


Figure 5-1. - Cross frames fabricated from steel angles are commonly used to provide lateral support for large glulam bridge beams.

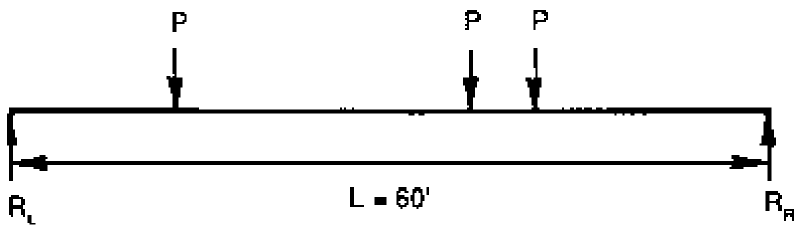
$$\ell_e = 1.84\,\ell_u \qquad \text{when } \ell_u/d \geq 14.3 \tag{5-6}$$

$$\ell_e = 1.63\,\ell_u + 3d \qquad \text{when } \ell_u/d < 14.3 \tag{5-7}$$

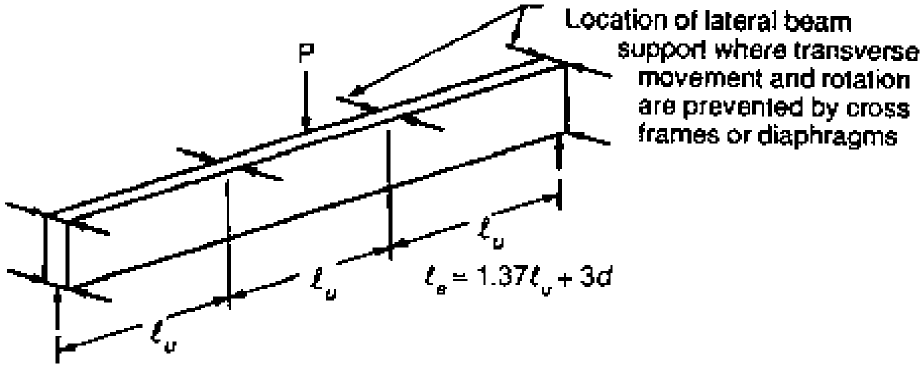
Equations for computing ℓ_e for other beam configurations and loading conditions are given in the NDS. For single-span or cantilever beams, Equations 5-6 and 5-7 give slightly conservative results for any loading condition and are often used in bridge applications where several concentrated loads are positioned on the span.

Example 5-1 - Beam slenderness factor

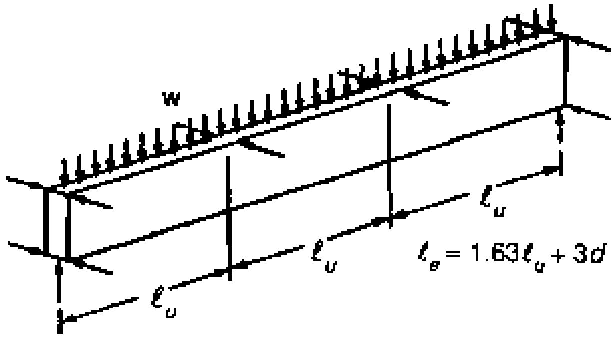
A 10-3/4- by 48-inch glulam beam spans 60 feet and supports the three concentrated loads shown below. Lateral beam support is provided by transverse bracing located at the beam ends and at the third points. Compute the beam slenderness factor, C_s.



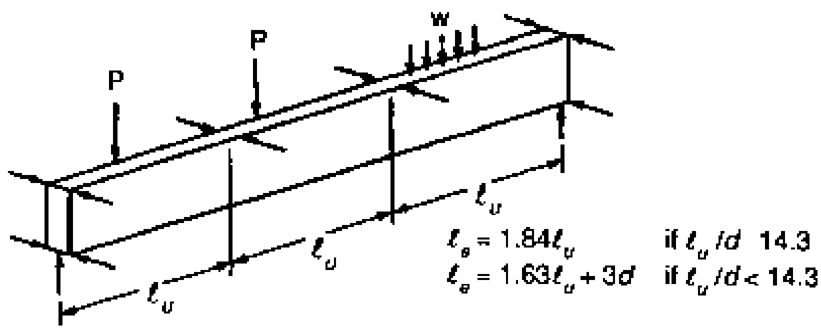
Lateral support at beam ends and at third point



Single-span beam with concentrated load at center



Single-span beam with uniform load



Single-span or cantilever beam with any loading condition

Figure 5-2.—Effective beam length, ℓ_e, for various loading conditions on single-span beams. Refer to the NDS for equations for other beam loads and configurations.

Solution

Lateral support is equally spaced along the beam, giving an unsupported length ℓ_u of 20 feet. Because the beam is loaded with three concentrated loads, the effective beam length ℓ_e will be computed by Equation 5-6 or 5-7, depending on the ratio of the unsupported length to the beam depth:

$$\frac{\ell_u}{d} = \frac{20(12 \text{ in/ft})}{48} = 5.0$$

5.0 < 14.3, so Equation 5-7 applies:

$$\ell_e = 1.63\ell_u + 3d = (1.63)(20)(12 \text{ in/ft}) + (3)(48) = 535.20$$

The slenderness factor is computed by Equation 5-3:

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{535.2(48)}{(10.75)^2}} = 14.9 \leq 50$$

This example illustrates a typical case where transverse bracing is equally spaced and the value of C_s applies to all portions of the beam. In cases where the distance ℓ_u varies substantially along the beam length, C_s should be checked for each unsupported length. With few exceptions, however, C_s for the center portion of the beam, where bending stress is highest, will normally control.

Allowable Stress

The allowable bending stress in beams is controlled either by the size factor C_F, which limits bending stress in tension zone, or by lateral stability, which limits bending stress in the compression zone. *Adjustments for the size factor and lateral stability are not cumulative.* Therefore, the designer must compute allowable bending stress based on both criteria separately, and the lowest value obtained is used for design. In most bridge beams, allowable bending stress is controlled by C_F rather than stability. In addition, beam stability cannot be evaluated until an initial member size is selected. Therefore, it is most convenient and practical to assume that the size factor controls allowable bending stress and to initially design the beam based on the allowable stress given by

$$F_b' = F_b C_D C_M C_R C_t C_F \qquad (5\text{-}8)$$

Values of C_F are normally included in tables of section properties for glulam bending combinations (see Tables 16-3 and 16-4). In addition, most glulam tables include C_F as a noted adjustment to the section modulus. This adjusted value, $S_x C_F$, is included for convenience and facilitates design by adjusting for C_F during initial member selection (see Example 5-3).

After a satisfactory beam size and grade are determined based on the allowable bending stress given by Equation 5-8, the beam must be checked for lateral stability. Criteria for allowable bending stress related to lateral stability are based on beam slenderness for the following three ranges:

$0 < C_s \leq 10$ **Short Beam**

$10 < C_s \leq C_k$ **Intermediate Beam**

$C_k < C_s \leq 50$ **Long Beam**

where C_k is a slenderness factor defined later for intermediate beams.

Short Beams

In short beams with C_s of 10 or less, capacity of the member is controlled by the wood strength in bending rather than by lateral stability. In this case, the size factor is the controlling modification factor, and the allowable bending stress computed by Equation 5-8 is used for design.

Intermediate Beams

Intermediate beams have C_s greater than 10, but less than C_k determined by

$$C_k = 0.811\sqrt{E'/F_b''} \tag{5-9}$$

where C_k = the largest value of C_s at which the intermediate beam equation applies,

$E' = EC_MC_RC_t$ (lb/in²), and

$F_b'' = F_bC_DC_MC_RC_t$ (lb/in²).

In intermediate beams, failure can occur in bending or by torsional buckling from lateral instability. The controlling mode is indicated by the lateral stability of beams factor C_L given by

$$C_L = \left[1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4\right] \tag{5-10}$$

If C_L is less than C_F, bending stress is controlled by stability, and C_L is the controlling modification factor. The allowable bending stress is computed by

$$F_b' = F_bC_DC_MC_RC_tC_L \tag{5-11}$$

If C_L is greater than C_F, bending stress is controlled by strength, and the allowable stress computed by Equation 5-8 is used for design.

Equation 5-9 for lateral stability was developed from theoretical analyses and beam verification tests and is based on the modulus of elasticity of the member. For visually graded sawn lumber, tabulated E values are based on the average modulus of elasticity for the grade and species of material and represent a coefficient of variation of approximately 0.25. For glulam with six or more laminations, the coefficient of variation is 0.10 (less than half that for visually graded sawn lumber). To account for this reduced variability, the NDS allows the designer to use the following modified equation for C_k (Equation 5-12), which more accurately reflects the characteristics of glulam:

$$C_k = 0.956\sqrt{E'/F_b''} \tag{5-12}$$

This equation provides the same factor of safety at the 5-percent exclusion value for glulam that is provided for visually graded sawn lumber with a 0.25 coefficient of variation. Although use of Equation 5-12 is optional, it represents a more realistic approach to glulam beam design and is recommended for bridge applications. For additional information on low-variability equations for glulam beams, refer to Appendix O of the NDS [26] and the *AITC Timber Construction Manual.*[6]

Long Beams

Long beams have a slenderness ratio greater than C_k, but less than or equal to 50. In long beams, bending stress is controlled by lateral stability rather than strength, and the allowable stress is computed using

$$F_b' = \frac{0.438E'}{(C_s)^2} \qquad (5\text{-}13)$$

For glulam beams, the following low-variability equation may be used in lieu of Equation 5-13:

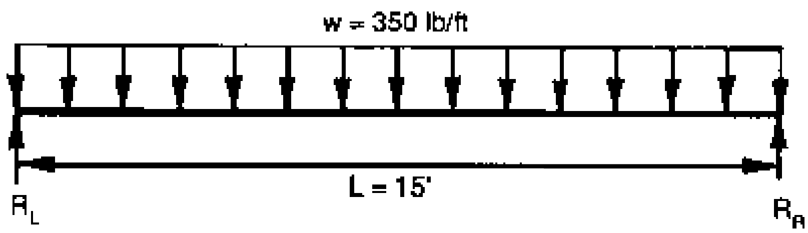
$$F_b' = \frac{0.609E'}{(C_s)^2} \qquad (5\text{-}14)$$

Example 5-2- Beam design based on bending; sawn lumber beam

A sawn lumber beam spans 15 feet center-to-center of bearings and supports a uniform load of 350 lb/ft in addition to its own weight. The beam is laterally supported by blocking placed at the beam ends and at 5-foot intervals along the beam length. Determine the required beam size based on bending, assuming the following:

1. Normal load duration under wet-use conditions (lumber moisture content will exceed 19-percent in service); adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. The beam is surfaced (S4S) Douglas Fir-Larch.



Solution

Beam design is somewhat of a trial-and-error process that starts with either an estimated beam size or a selected lumber species and grade. In this example, Douglas Fir-Larch, visually graded No. 1 in the Joist and Plank size classification is initially selected. The tabulated bending stress and modulus of elasticity for this species and grade are obtained from Table 4A of the NDS:

$$F_b = 1{,}500 \text{ lb/in}^2$$

$$E = 1{,}800{,}000 \text{ lb/in}^2$$

An initial section modulus based on applied moment and tabulated bending stress is computed as follows:

$$M = \frac{wL^2}{8} = \frac{(350)(15)^2}{8} = 9{,}844 \text{ ft-lb}$$

Rearranging Equation 5-2,

$$S = \frac{M}{F_b} = \frac{(9{,}844)(12 \text{ in/ft})}{1{,}500} = 78.8 \text{ in}^3$$

From lumber section properties in Table 16-2, a nominal beam size is selected with a section modulus slightly greater than the required 78.8 in^3. The closest standard nominal size appears to be 4 inches by 14 inches with the following properties:

b = 3.5 in.

d = 13.25 in.

S = 102.41 in^3

Beam weight = 16.1 lb/ft (based on a unit weight for wood of 50 lb/ft^3)

The allowable bending stress is computed using the applicable modification factors given in Equation 5-8. The size factor, C_F is not applicable because it only applies to sawn lumber beams that are more than 4 inches thick. In this case, Equation 5-8 becomes

$$F_b' = F_b C_M$$

From Table 5-7, C_M = 0.86, and

$$F_b' = F_b C_M = 1{,}500(0.86) = 1{,}290 \text{ lb/in}^2$$

Next, the applied bending stress is revised to reflect the beam weight of 16.1 lb/ft:

$$M = \frac{wL^2}{8} = \frac{(350 + 16.1)(15)^2}{8} = 10{,}297 \text{ ft-lb}$$

By Equation 5-2,

$$f_b = \frac{M}{S} = \frac{(10{,}297 \text{ ft-lb})(12 \text{ in/ft})}{102.41 \text{ in}^3} = 1{,}207 \text{ lb/in}^2$$

f_b = 1,207 lb/in^2 < F_b' = 1,290 lb/in^2, so the initial beam is satisfactory in bending. The beam must next be checked for lateral stability.

For lateral support at 5-foot intervals,

$$\ell_u = 5 \text{ ft} = 60 \text{ in.}$$

By Equation 5-5 for a single-span beam with a uniformly distributed load,

$$\ell_e = 1.63\ell_u + 3d = (1.63)(60) + (3)(13.25) = 137.55$$

By Equation 5-3,

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(137.55)\,(13.25)}{(3.5)^2}} = 12.20$$

The value C_s = 12.20 is greater than 10, so further stability calculations are required. From Table 5-7, C_M for modulus of elasticity is 0.97, and

$$E' = EC_M = 1{,}800{,}000(0.97) = 1{,}746{,}000$$

By Equation 5-9,

$$C_k = 0.811\sqrt{\frac{E'}{F_b''}} = 0.811\sqrt{\frac{1{,}746{,}000}{1{,}500\,(0.86)}} = 29.84$$

10 < C_s = 12.20 < C_k = 29.84, so the beam is classified in the intermediate slenderness range. By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{12.20}{29.84}\right)^4 = 0.99$$

The allowable bending stress based on lateral stability is computed by Equation 5-11 using the modification factor C_L:

$$F_b' = F_b C_M C_L = 1{,}500(0.86)(0.99) = 1{,}277 \text{ lb/in}^2$$

f_b = 1,207 lb/in^2 < F_b' = 1,277 lb/in^2, so the beam size, species, and grade are satisfactory in bending.

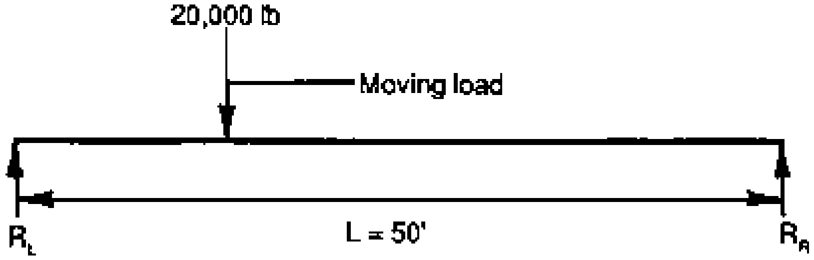
Summary

Based on bending only, the beam will be a nominal 4-inch by 14-inch surfaced Douglas Fir-Larch beam, visually graded No. 1 in the Joists and Planks (J&P) size classification. The applied bending stress, f_b, is 1,207 lb/in^2. The allowable bending stress, F_b', is 1,277 lb/in^2 and is controlled by lateral stability.

Example 5-3 - Beam design based on bending; glulam beam

A glulam beam spans 50 feet center-to-center of bearings and supports a moving concentrated load of 20,000 pounds. Determine the required beam size based on bending for cases where: (A) the beam is laterally supported at the ends and at the third points, and (B) the beam is laterally supported at the ends only. The following assumptions apply:

1. Normal load duration under wet-use conditions (glulam moisture content will exceed 16-percent in service); adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not applicable.
2. The glulam beam is manufactured from visually graded Southern Pine, combination symbol 24F-V2.



Case A: Lateral support is provided at beam ends and at third points
Case B: Lateral support is provided at beam ends only

Solution

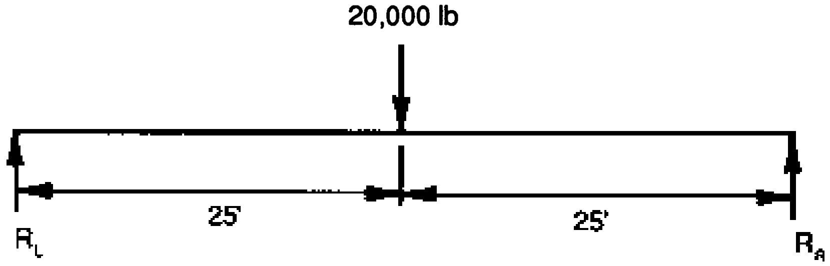
The first step in the design process is to determine the required beam size based on bending stress, adjusted by the size factor, C_F. The suitability of the initial beam size is then checked for each of the two conditions of lateral support.

Tabulated values for bending and modulus of elasticity are obtained from *AITC 117-Design.* Respective values for the moisture content modification factor are obtained from Table 5-7:

F_{bx} = 2,400 lb/in² $\qquad$ C_M = 0.80

E_x = 1,700,000 lb/in² $\qquad$ C_M = 0.833

The maximum applied moment is computed with the moving load positioned at the span centerline:



$$M = \frac{PL}{4} = \frac{(20{,}000)(50)}{4} = 250{,}000 \text{ ft-lb}$$

An initial beam size is determined using procedures similar to those used for sawn lumber beam design. For glulam, however, the size factor, C_F, is included as a noted adjustment to the section modulus (S_xC_F) in Table 16-4. By Equation 5-8,

$$F_b' = F_{bx}C_MC_F$$

Assuming that the applied bending stress equals the allowable bending stress, Equation 5-2 is rearranged to compute the required value of S_xC_F directly:

$$f_b = F_b' = F_{bx}C_MC_F = \frac{M}{S_x} \qquad \text{or} \qquad S_xC_F = \frac{M}{F_{bx}C_M}$$

Based on the moment from the concentrated load only, an initial value of S_xC_F is computed:

$$S_xC_F = \frac{M}{F_{bx}C_M} = \frac{(250{,}000)(12 \text{ in/ft})}{(2{,}400)(0.80)} = 1{,}563 \text{ in}^3$$

From Table 16-4, an initial beam size is selected that provides an S_xC_F value slightly greater than 1,563 in^3. It is usually most convenient to find the closest S_xC_F to that required, then increase the beam depth by one or two laminations to account for the beam dead load. In this case, a 6-3/4-inch by 41-1/4-inch beam is chosen with the following properties:

S_xC_F = 1,668.9 in^3

Beam weight = 96.7 lb/ft (based on a unit weight of 50 lb/ft^3)

Moment from the beam weight is computed and added to that from the concentrated load:

$$\text{Beam } M = \frac{wL^2}{8} = \frac{96.7(50)^2}{8} = 30{,}219 \text{ ft-lb}$$

M = 250,000 + 30,219 = 280,219 ft-lb

The required S_xC_F value is revised:

$$S_x C_F = \frac{(280,219)(12 \text{ in/ft})}{(2,400)(0.80)} = 1,751 \text{ in}^3$$

From Table 16-4, a revised beam size of 6-3/4 inches by 42-5/8 inches is selected with the following properties:

S_x = 2,044 in^3

C_F = 0.87

Beam weight = 99.9 lb/ft (based on a unit weight of 50 lb/ft^3)

Moment from beam weight is revised and the applied bending stress is computed:

$$\text{Beam } M = \frac{wL^2}{8} = \frac{99.9(50)^2}{8} = 31,219 \text{ ft-lb}$$

$$M = 250,000 + 31,219 = 281,219 \text{ ft-lb}$$

$$f_b = \frac{M}{S_x} = \frac{281,219(12 \text{ in/ft})}{2,044} = 1,651 \text{ lb/in}^2$$

Allowable bending stress is computed by Equation 5-8:

$$F_b' = F_{bx} C_M C_F = 2,400(0.80)(0.87) = 1,670 \text{ lb/in}^2$$

f_b = 1,651 lb/in^2 < F_b' = 1,670 lb/in^2, so the beam is satisfactory in bending, assuming that the size factor controls. The beam is next checked for lateral stability.

Case A: Lateral support at beam ends and at third points

For lateral support at the beam ends and at the third points, the unsupported beam length is equal to one-third the span length:

$$\ell_u = \frac{50}{3} = 16.67 \text{ ft} = 200 \text{ in.}$$

Because the maximum moment is produced with the moving load at midspan, the effective beam length is computed using Equation 5-4:

$$\ell_e = 1.37\ell_u + 3d = 1.37(200) + 3(42.63) = 401.89 \text{ in.}$$

By Equation 5-3,

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{401.89\,(42.63)}{(6.75)^2}} = 19.39$$

The value of C_s is greater than 10, so lateral stability must be checked further. By equation 5-12 for low-variability material,

$$E' = E_x C_M = 1{,}700{,}000(0.83) = 1{,}416{,}100 \text{ lb/in}^2$$

$$F_b'' = F_{bx} C_M = 2{,}400(0.80) = 1{,}920 \text{ lb/in}^2$$

$$C_k = 0.956\,\frac{E'}{F_b''} = 0.956\,\frac{1{,}416{,}100}{1{,}920} = 25.96$$

$C_s = 19.39 < C_k = 25.96$, so the beam is in the intermediate beam slenderness range.

By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{19.39}{25.96}\right)^4 = 0.90$$

$C_L = 0.90 > C_F = 0.87$, so the size factor reduction is more severe and controls the allowable bending stress. The selected beam size is therefore satisfactory in bending.

Case B: Lateral support at beam ends only

With lateral support at the beam ends only, the unsupported beam length equals the span length:

$$\ell_u = 50 \text{ ft} = 600 \text{ in.}$$

By Equation 5-4,

$$\ell_e = 1.37\,\ell_u + 3d = 1.37(600) + 3(42.63) = 949.89 \text{ in.}$$

By Equation 5-3,

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{949.89\,(42.63)}{(6.75)^2}} = 29.81$$

The previously computed value C_k = 25.96 is unchanged. In this case, however, $C_k = 25.96 < C_S = 29.81$, so the beam is in the long-beam slenderness range and lateral stability controls design. By low-variability Equation 5-14,

$$F_b' = \frac{0.609E'}{(C_s)^2} = \frac{0.609(1,416,100)}{(29.81)^2} = 970 \text{ lb/in}^2$$

f_b = 1,651 lb/in^2 > F_b' = 970 lb/in^2, so the beam must be redesigned. Using a modified form of Equation 5-2, with the previously computed moment (based on the previous beam size):

$$S_x = \frac{M}{F_b'} = \frac{281,219\,(12 \text{ in/ft})}{970} = 3,479 \text{ in}^3$$

From Table 16-4, a revised beam size of 8-1/2 inches by 50-7/8 inches is selected with the following properties:

S_x = 3,666.7 in^3

Beam weight = 150.2 lb/ft (based on a unit weight of 50 lb/ft^3)

Moment from beam weight is revised and bending stress is computed:

$$\text{Beam } M = \frac{wL^2}{8} = \frac{150.2(50)^2}{8} = 46,938 \text{ ft-lb}$$

$$M = 250,000 + 46,938 = 296,938 \text{ ft-lb}$$

$$f_b = \frac{M}{S_x} = \frac{296,938\,(12 \text{ in/ft})}{3,666.7} = 972 \text{ lb/in}^2$$

F_b' = 970 lb/in^2 < f_b = 972 lb/in^2, but the difference of 2 lb/in^2, or approximately 0.20 percent, is insignificant and the beam size is acceptable.

Summary

Based on bending only, the required size and bending stress for 24F-V2 Southern Pine beams are as follows:

Case A: With lateral support at beam ends and at third points

Beam size = 6-3/4 in. by 42-5/8 in.

f_b = 1,651 lb/in^2

F_b' = 1,670 lb/in^2

Case B: With lateral support at beam ends only

Beam size = 8-1/2 in. by 50-7/8 in.

f_b = 972 lb/in^2

$F_b' = 970 \text{ lb/in}^2$

This example illustrates the effect of lateral support on beam size requirements. When support along the span is eliminated, the required beam size increases substantially. Additional requirements on the placement and design of lateral support for bridge beams are discussed in Chapter 7.

DESIGN FOR DEFLECTION

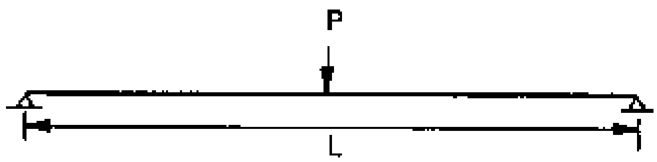
Deflection is the relative deformation that occurs in a beam as it is loaded. Deflection in timber beams results from bending and shear, but shear deformations are small in comparison to bending deformations and are normally not considered. Deflection does not seriously affect the strength of a beam, but it can affect the serviceability and appearance of bridge members and the performance of fasteners.

The length of time a load acts on a member influences its long-term deflection. When loads of relatively short duration are applied, deformation occurs immediately and remains at a relatively constant level for the duration of loading. When the load is removed, the member recovers elastically to the original unloaded position. For permanent loads (dead loads), initial elastic deformation is immediate, but members also develop an additional time-dependent, nonrecoverable deformation. This time-dependent deformation, known as creep, develops at a slow but persistent rate and is more pronounced for members seasoned in place or subject to variations in moisture content and temperature. Creep does not endanger the safety of the beam, but it can influence the performance, serviceability, and appearance of a structure when it is ignored in design. Thus, the two types of deflection considered in timber bridge design are: elastic deflection, and inelastic deflection, or creep.

Deflection Equations

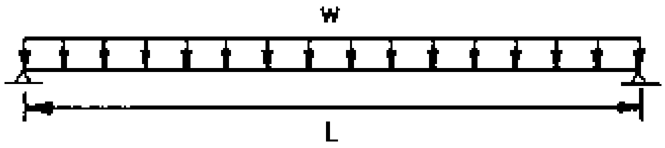
Timber beam deflections are computed by the same engineering methods used for isotropic, elastic materials. Standard equations based on these methods are available in many engineering textbooks and manuals for numerous beam configurations and loading conditions.[6,27] Two of the most commonly used equations for simple beams are given below in Equations 5-15 and 5-16. Additional equations for more specific bridge applications and loads are discussed in Chapters 7, 8, and 9.

For a simply supported beam with one concentrated load at the center of the span:



$$\Delta = \frac{PL^3}{48E'I} \qquad (5\text{-}15)$$

For a simply supported beam with a uniform load:



$$\Delta = \frac{5wL^4}{384E'I} \qquad (5\text{-}16)$$

where P = magnitude of a single concentrated load (lb),

w = magnitude of uniform load (lb/in),

L = beam span (in.),

$E' = EC_MC_tC_R$ (lb/in^2), and

I = moment of inertia about the axis of bending (in^4).

Note that the modification factor for duration of load, C_D, does not apply to E.

Deflection equations such as 5-15 and 5-16 can be used to accurately predict elastic beam deflections. For permanent load deflections, however, it is necessary to increase computed values to compensate for the long-term effects of creep. The magnitude of the increase depends on the type of material and the moisture content of the member at installation. A 50-percent increase in dead load deflection is normally sufficient for glulam and seasoned sawn lumber, while a 100-percent increase is more appropriate for unseasoned lumber (refer to Appendix F of the NDS for additional discussions on dead load deflection increases for creep).

Deflection Criteria

AASHTO specifications do not give deflection criteria for timber bridge members, and selection of an appropriate deflection limit is a matter of designer judgment. The acceptable deflection for a member will depend on specific use requirements and may vary among beam types within the same structure. Deflections in bridge members are important for serviceability, performance, and aesthetics and should not be ignored. From a structural viewpoint, large deflections cause fasteners to loosen and brittle materials, such as asphalt pavement, to crack and break. In addition, members that sag below a level plane present a poor appearance and can give the public a perception of structural inadequacy. Deflections from moving vehicle loads also produce vertical movement and vibrations that annoy motorists and alarm pedestrians.

Bridge deflection is normally expressed as a fraction, the denominator of which is obtained by dividing the beam span in inches by the computed deflection in inches. A deflection of *L/500,* for example, indicates a deflection equal to one five-hundredth of the beam span. The larger the denominator, the smaller the deflection. A brief literature search of bridge-related specifications and publications produced maximum recommended applied-load deflection values ranging from *L/200* to *L/1,200.* For general beam design discussed in this chapter, the recommended maximum deflections for timber beams are as follows:

1. For applied (short-term) loads, the maximum deflection should not exceed *L/360.*

2. For the combination of applied loads and dead load, the maximum deflection should not exceed *L/240,* where the portion of the total deflection from dead load is increased to account for creep.

Additional considerations and recommendations for deflection in timber bridge components are discussed in more detail in Chapters 7, 8, and 9.

Camber

Camber is circular or parabolic upward curvature built into a glulam beam, opposite to the direction of deflection. It is intended to offset dead load deflection and creep and is introduced during the manufacturing process. It is not feasible to camber sawn lumber beams. The amount of camber for bridge beams depends on the length and number of spans. For single spans shorter than approximately 50 feet, camber should be a minimum of 1.5 to 2.0 times the immediate (elastic) dead load deflection, plus one-half the applied load deflection.[6] For single beam spans equal to or longer than 50 feet and multiple-span beams of any span, camber should be a minimum of 1.5 to 2.0 times the immediate dead load deflection (multiple-span bridge beams are normally cambered for dead loads only to obtain acceptable riding qualities for vehicle traffic).

Camber is specified by the designer as a vertical centerline offset to the horizontal line between points of bearing (Figure 5-3). The glulam manufacturer will determine an appropriate radius of curvature based on offset distances and fabrication limitations. On multiple-span continuous beams, camber may vary along the beam and should be specified for each span segment. More specific information on cambering practices and limitations can be obtained from glulam manufacturers and the AITC.

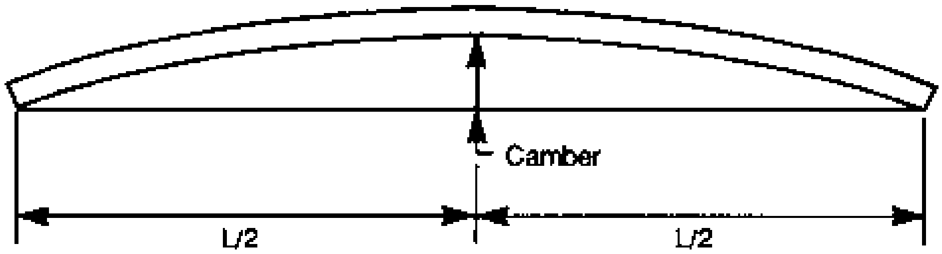
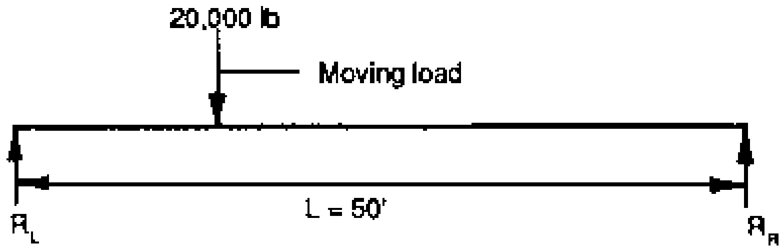


Figure 5-3.- Camber for glulam beams is specified as an upward vertical offset at the span centerline.

Example 5-4- Beam deflection and camber

For the glulam beam of Example 5-3, Case A, determine the deflection from the 20,000-pound moving load and the camber required to offset deflection from the beam weight. The beam spans 50 feet, measures 6-3/4 inches by 42-5/8 inches, and is manufactured from visually graded Southern Pine, combination symbol 24F-V2.



Solution:

The tabulated modulus of elasticity for a 24F-V2 Southern Pine beam is obtained from *AITC 117-Design:*

$$E_x = 1,700,000 \text{ lb/in}^2$$

The allowable modulus of elasticity is computed using the applicable C_M value from Table 5-7:

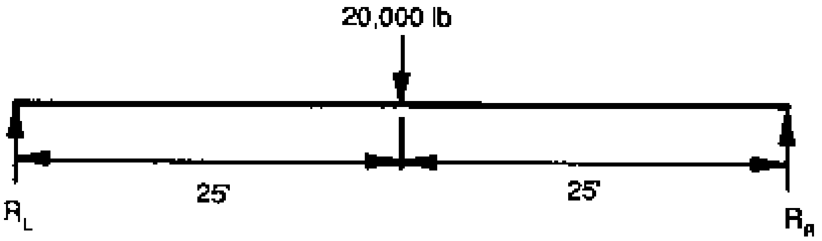
$$E' = E_x C_M = 1,700,000(0.833) = 1,416,100 \text{ lb/in}^2$$

From Table 16-4 for a 6-3/4-inch by 42-5/8-inch Southern Pine beam:

$$I_x = 43,562.8 \text{ in}^4$$

Beam weight = 99.9 lb/ft (based on a beam weight of 50 lb/ft^3)

Deflection for the 20,000-pound moving load is computed with the load at midspan by Equation 5-15:



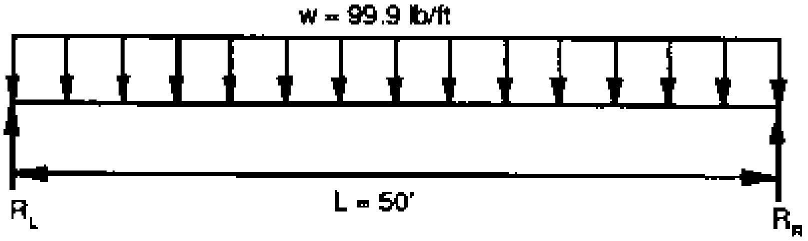
$$\Delta = \frac{PL^3}{48\,E'\,I_x} = \frac{20{,}000\left[50\,(12\text{ in/ft})\right]^3}{48\,(1{,}416{,}100)(43{,}562.8)} = 1.46\text{ in.}$$

Expressing the deflection as a ratio of the bridge span,

$$\Delta = \frac{L}{\left[50\text{ ft}\,(12\text{ in/ft})\right]/1.46\text{ in.}} = \frac{L}{411}$$

$L/411 < L/360$, so deflection is acceptable.

For the beam weight of 99.9 lb/ft, deflection is computed by Equation 5-16:



$$\Delta = \frac{5wL^4}{384\,E'\,I_x} = \frac{5(99.9)\left[50\,(12\text{ in/ft})\right]^4}{384(1{,}416{,}100)(43{,}562.8)(12\text{ in/ft})} = 0.23\text{ in.}$$

Camber of approximately 1/2-inch will be specified at centerline, which is approximately twice the beam dead load deflection.

DESIGN FOR SHEAR

Beams develop internal shear forces that act perpendicular and parallel to the longitudinal beam axis. In timber beams, horizontal shear rather than vertical shear will always control design. As discussed in Chapter 3, horizontal shear forces produce a tendency for the upper portion of the beam to slide in relation to the lower portion of the beam, with shear stresses acting parallel to the grain of the member. The maximum intensity of horizontal shear in rectangular beams occurs at the neutral axis and is proportional to the vertical shear force, *V*. In bridge applications, horizontal shear generally controls beam design only on relatively short, heavily loaded spans.

Shear requirements in AASHTO and the NDS apply at or near the supports for solid beams constructed of such materials as sawn lumber, glulam, or mechanically laminated lumber. Shear design for built-up components containing load-bearing connections at or near supports, such as between a web and chord, must be based on tests or other techniques.

Applied Stress

The applied stress in horizontal shear depends on the magnitude of the vertical shear and the area of the beam. Applied stress in square or rectangular timber beams is computed by Equation 5-17:

$$f_v = \frac{3V}{2bd} = \frac{1.5V}{A} \qquad (5\text{-}17)$$

where f_v = unit stress in horizontal shear (lb/in^2),

V = vertical shear force (lb),

b = beam width at the neutral axis (in.),

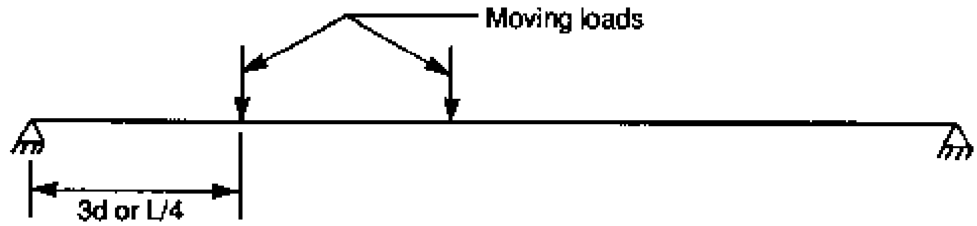
d = beam depth (in.), and

A = beam cross-sectional area (in^2).

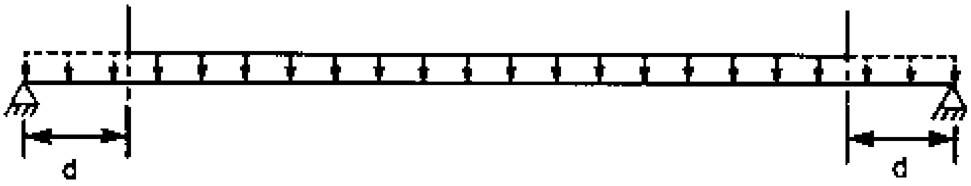
Equation 5-17 does not apply (1) at notches or joints, (2) in regions where the beam is supported by fasteners, or (3) when hanging loads are located at or near the supports. For these conditions, refer to AASHTO and the NDS.

The magnitude off, given by Equation 5-17 is based on the value of the vertical shear force, *V*. Unlike the situation in other construction materials, where the maximum vertical shear is computed at the face of the supports, in timber beams the maximum intensity of horizontal shear is produced by the maximum vertical shear force occurring at some distance from the support. This distance depends on the type of applied loading; different distances are used for moving loads and for stationary loads.

Current AASHTO requirements (AASHTO 13.3.1) specify that horizontal shear in beams from moving (vehicle) loads be computed from the maximum vertical shear *(V)* occurring at a distance from the support equal to three times the beam depth *(3d*, or the span quarter point *(L/4)*, whichever is less (Figure 5-4). The moving loads are positioned on the beam to produce the maximum vertical shear at this location (Chapter 6). For stationary loads (such as dead load), vertical shear is computed at a distance from the support equal to the beam depth, *d*, and all loads occurring within the distance *d* from the supports are neglected. For sawn lumber, shear design requirements given in the NDS vary somewhat based on the beam configuration, loading condition, and wood species. Refer to the latest edition of the NDS for additional shear criteria for sawn lumber.



For moving loads, the loads are positioned to produce the maximum vertical shear at the lesser of 3d or L/4 from the support



For stationary loads, such as dead load, the maximum vertical shear is computed at a distance d from the supports and all loads occurring within a distance d from the supports are neglected

Figure 5-4.- Locations for determining the maximum vertical shear (V) for timber beams.

Although the bases for shear design requirements are widely accepted, specific requirements for computing *V* are somewhat controversial and vary among design specifications. Research is currently under way to develop more accurate design criteria for shear, and the designer should remain familiar with the most current requirements and the potential for future revision.

Allowable Stress

The allowable stress in horizontal shear is computed by

$$F_v' = F_v C_D C_M C_R C_t \tag{5-18}$$

Individual sawn lumber members have a much higher potential for strength-reducing characteristics that reduce the ability of the member to resist horizontal shear. In glulam, most strength-reducing characteristics are excluded at fabrication and any that remain are dispersed throughout the individual laminations in the section. For sawn lumber, strength-reducing characteristics are not dispersed, and members are more susceptible to the development of checks and splits caused by variations in moisture content. As a result, tabulated values of F_v for sawn lumber are considerably lower than those for glulam because they are based on the worst-case assumption that members are split for their entire length. In situations where the length of split, or size of check or shake, can be estimated with reasonable certainty, the tabulated horizontal shear stress can be increased by the shear stress modification factors given in footnotes to the NDS Table 4A (Table 5-12). Application of this factor to specific design situations and materials is left to designer judgment, but the 2.0

increase is commonly used for mechanically laminated lumber and dimension lumber with loads applied perpendicular to the wide face. Additional information on application of the shear stress modification factor is discussed in Chapters 7 and 8.

Table 5-12.- Shear stress modification factor for sawn lumber.

Length of split on wide face of 2" lumber (nominal):	Multiply tabulated "F_v" value by:
No split	2.00
1/2 x wide face	1.67
3/4 x wide face	1.50
1 x wide face	1.33
1-1/2 x wide face or more	1.00
Length of split on wide face of 3" and thicker lumber (nominal):	**Multiply tabulated "F_v" value by:**
No split	2.00
1/2 x narrow laace	1.67
1 x narrow face	1.33
1-1/2 x narrow face or more	1.00
Size of shake[a] in 3" and thicker lumber (nominal):	**Multiply tabulated "F_v" value by:**
No shake	2.00
1/6 x narrow face	1.67
1/3 narrow face	1.33
1/2 x narrow face or more	1.00

[a] Shake is measured at the end between lines enclosing the shake and parallel to the wide face.

Specific horizontal shear values may be established by use of this table when the length of split, or size of check or shake is known and no increase in them is anticipated. For California Redwood, Southern Pine, Virginia Pine-Pond Pine, and Yellow Poplar, refer to the NDS for specific values of F_v for which these adjustments apply.

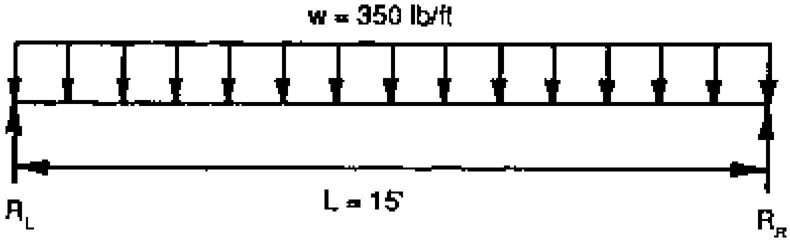
From the NDS;[24] © 1986. Used by permission.

Example 5-5- Horizontal shear in a sawn lumber beam

Determine the adequacy of the beam in Example 5-2 for horizontal shear. The beam measures 4 inches by 14 inches and is surfaced Douglas Fir-Larch, visually graded No. 1 in the J&P size classification. It spans 15 feet and supports a uniform load of 350 lb/ft.

Solution

Tabulated horizontal shear stress for No.1 Douglas Fir-Larch is obtained from Table 4A of the NDS (note that the tabulated shear stress for lumber 2 to 4 inches thick is the same for all grades):



$F_v = 95\ \mathrm{lb/in^2}$

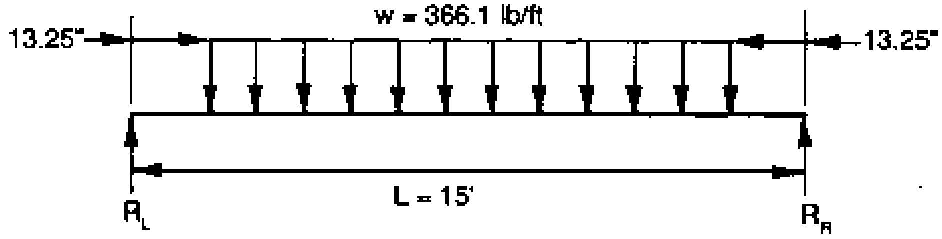
Allowable shear stress is computed by Equation 5-18 using the C_M value obtained from Table 5-7,

$$C_M = 0.97$$

$$F_v' = F_v C_M = 95(0.97) = 92\ \mathrm{lb/in^2}$$

The allowable stress in horizontal shear could be increased by the shear stress modification factor (Table 5-12) if the beam were free of shake, splits or checks, or if the length of such characteristics was known. For lumber bridge beams of this type, it is common for some beam checking to occur, however, its magnitude cannot be accurately predicted. Therefore, no adjustment by the shear stress modification factor will be used.

From Example 5-2, the beam weighs 16.1 lb/ft and has actual dimensions of 3.5 inches by 13.25 inches. The total load acting on the beam is equal to the 350 lb/ft applied load plus the beam weight of 16.1 lb/ft, for a total of 366.1 lb/ft. For a uniformly distributed load, the maximum vertical shear force, *V*, is computed at a distance from the support equal to the beam depth, *d*, and all loads acting within a distance *d* from the supports are neglected:



$$V = R_L = w\left(\frac{L}{2} - d\right) = 366.1\left[\frac{15}{2} - \frac{13.25}{(12\ \mathrm{in/ft})}\right] = 2{,}342\ \mathrm{lb}$$

Horizontal shear stress is computed by Equation 5-17:

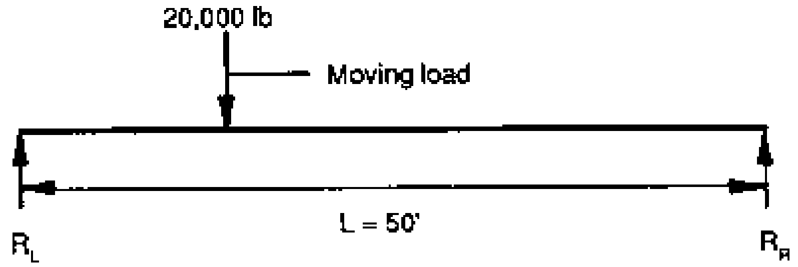
$$A = (3.5 \text{ in.})(13.25 \text{ in.}) = 46.38 \text{ in}^2$$

$$f_v = \frac{1.5V}{A} = \frac{1.5\,(2{,}342)}{46.38} = 76 \text{ lb/in}^2$$

$f_v = 76$ lb/in^2 $< F_v' = 92$ lb/in^2, so horizontal shear is acceptable

Example 5-6- Horizontal shear in a glulam beam.

Check the adequacy of the glulam beam in Example 5-3, Case A, for horizontal shear. The beam measures 6-3/4 inches by 42-5/8 inches and is manufactured from visually graded Southern Pine, combination symbol 24F-V2. It spans 50 feet and supports a moving concentrated load of 20,000 pounds.



The tabulated stress for horizontal shear for a 24F-V2 beam is obtained from *AITC 117--Design,*

$$F_{vx} = 200 \text{ lb/in}^2$$

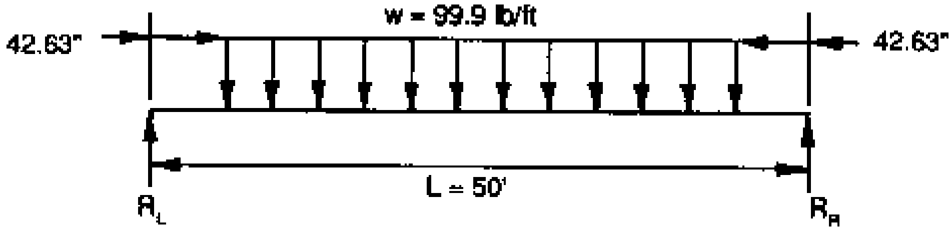
Allowable shear stress is computed by Equation 5-18 using the applicable C_M value obtained from Table 5-7:

$$C_M = 0.875$$

$$F_v' = F_{vx} C_M = 200(0.875) = 175 \text{ lb/in}^2$$

In this case the beam supports two loads; the uniform load from the beam weight and the moving concentrated load. Maximum vertical shear from the uniformly distributed beam weight is computed at a distance from the support equal to the beam depth, *d,* and all loads acting within a distance *d* from the supports are neglected. For the moving concentrated load, maximum vertical shear is computed at a distance from the support equal to three times the beam depth, *3d,* or the span quarter point, *L/4,* whichever is less.

For the uniformly distributed beam weight of 99.9 lb/ft and a beam depth of 42.63 inches,

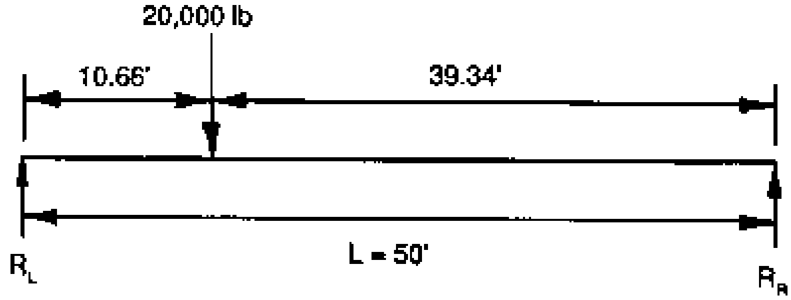


$$V = R_L = w\left(\frac{L}{2} - d\right) = 99.9\left[\frac{50}{2} - \frac{42.63}{(12\ \text{in/ft})}\right] = 2{,}143\ \text{lb}$$

For the moving concentrated load of 20,000 lb,

$$3d = 3\left(\frac{42.63}{12\ \text{in/ft}}\right) = 10.66\ \text{ft} \qquad \frac{L}{4} = \frac{50}{4} = 12.50\ \text{ft}$$

3d < *L/4*, so the maximum vertical shear from the 20,000-pound load is computed at a distance of 10.66 feet from the support:



$$V = R_L = \frac{20{,}000\,(39.34)}{50} = 15{,}736\ \text{lb}$$

From Table 16-4, the cross-sectional area of a 6-3/4-inch by 42-5/8-inch Southern Pine glulam beam is 287.7 in^2. Applied stress is computed by Equation 5-17:

$$f_v = \frac{1.5V}{A} = \frac{1.5\,(2{,}143 + 15{,}736)}{287.7} = 93\ \text{lb/in}^2$$

f_v = 93 lb/in² < F_v' = 175 lb/in², so horizontal shear is acceptable.

DESIGN FOR BEARING

Reactions at beam supports produce bearing stress that acts perpendicular to or at an angle to the grain of the member. Bearing stress causes wood fibers to compress to a degree that depends on the magnitude of load and the area of bearing. The beam bearing area must be large enough to adequately transfer loads without causing the wood to compress or deform excessively.

Applied Stress

Applied bearing stress is computed by

$$f_{c\perp} = \frac{R}{A} \tag{5-19}$$

where $f_{c\perp}$ = unit stress in compression perpendicular to grain (lb/in^2),

R = reaction or bearing force at the support (lb), and

A = net bearing area (in^2).

When computing $f_{c\perp}$ at the end of a beam, no allowance is made for the fact that as the beam bends the pressure on the inner edge of the bearing is greater than that at the end of the beam.

Allowable Stress

The allowable stress for bearing perpendicular to grain is equal to the tabulated stress $F_{c\perp}$ adjusted by all applicable modification factors, except the duration of load factor, C_D, as computed by

$$F_{c\perp}' = F_{c\perp} C_M C_t C_R \tag{5-20}$$

When beam bearing is not perpendicular to grain (Figure 5-5), allowable stress must be computed for compression at an angle to the grain using the Hankinson Formula (Equation 5-21):

$$F_n' = \frac{F_g' F_{c\perp}'}{F_g' \sin^2(\theta) + F_{c\perp}' \cos^2(\theta)} \tag{5-21}$$

where F_n' = allowable stress in compression at an angle to the grain (lb/in^2),

$F_g' = F_g C_M C_D C_t C_R$ (lb/in^2),

$F_{c\perp}' = F_{c\perp} C_M C_t C_R$ (lb/in^2), and

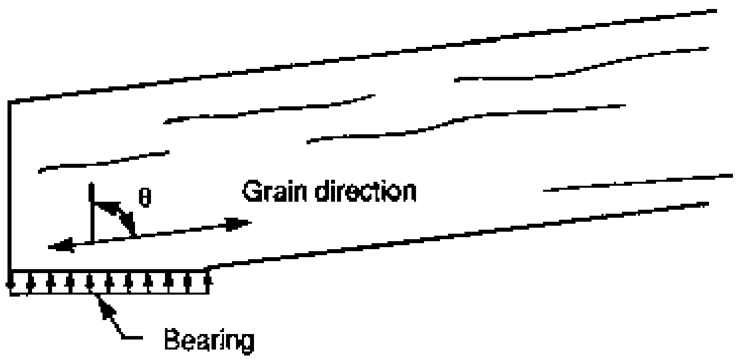


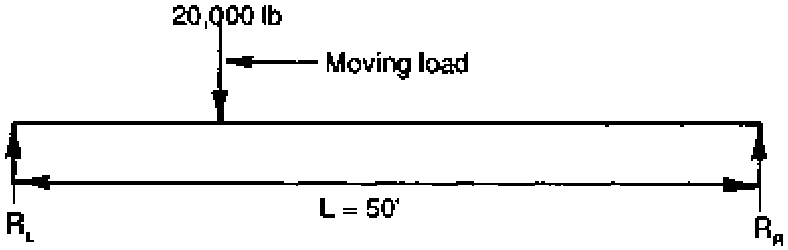
Figure 5-5. -- Beam bearing at an angle to the grain.

θ = angle between the direction of load and the direction of grain (degrees).

Values of $F_{c\perp}$ given in the NDS and *AITC 117-Design* apply to bearings of any length at beam ends and to all bearings 6 inches or more in length at other locations. Refer to the NDS for required adjustments in tabulated stress for bearings less than 6 inches long at locations between beam ends.

Example 5-7 - Beam bearing

For the glulam beam of Example 5-3, Case A, determine the required bearing length and the bearing stress in compression perpendicular to grain. The beam spans 50 feet center-to-center of bearings, is 6-3/4 inches wide and supports a moving concentrated load of 20,000 pounds. It is manufactured from visually graded Southern Pine, combination symbol 24F-V2.



Solution

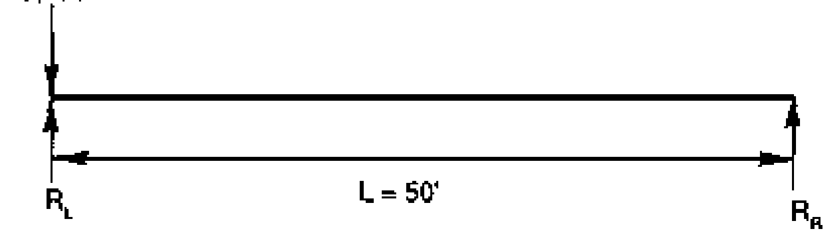
The tabulated stress in compression perpendicular to grain for a 24F-V2 Southern Pine beam is obtained from *AITC 117-Design:*

$$F_{c\perp x} = 650 \text{ lb/in}^2$$

The allowable compression perpendicular to grain is computed using Equation 5-20 and the applicable C_M value from Table 5-7:

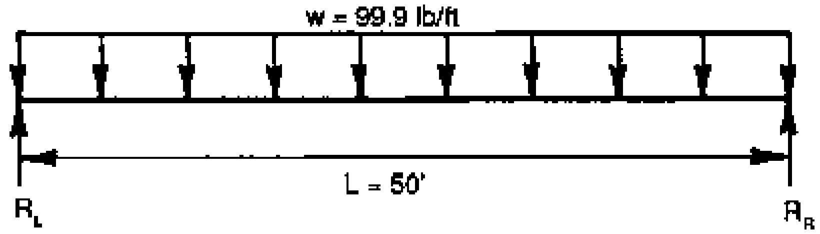
$$F_{c\perp}' = F_{c\perp x} C_M = 650(0.53) = 345 \text{ lb/in}^2$$

The maximum reaction at the beam bearing is equal to the sum of the reactions from the moving concentrated load and the beam weight. The maximum reaction from the moving concentrated load occurs when the load is placed over one support:



R_L = 20,000 lb

The reaction from the beam weight is the same at both supports:



$$R_L = \frac{wL}{2} = \frac{99.9\,(50)}{2} = 2{,}498 \text{ lb}$$

Rearranging Equation 5-19, the minimum required bearing area is computed for the maximum reaction by substituting $F_{c\perp}'$ for $f_{c\perp}$:

$$A = \frac{R}{F_{c\perp}'} = \frac{(20{,}000 + 2{,}498)}{345} = 65.2 \text{ in}^2$$

For a beam width of 6-3/4 inches, the required bearing length is computed by dividing the bearing area by the bearing width:

$$\text{Bearing length} = \frac{A}{b} = \frac{65.30}{6.75} = 9.7 \text{ in.}$$

A bearing length of 10 inches is selected and applied stress is computed by Equation 5-19:

$$f_{c\perp} = \frac{P}{A} = \frac{(20{,}000 + 2{,}498)}{10(6.75)} = 333 \text{ lb/in}^2$$

$f_{c\perp} = 333 \text{ lb/in}^2 < F_{c\perp}' = 345 \text{ lb/in}^2$, so the bearing is satisfactory. For a center-to-center span of 50 feet, a beam length of 50 feet 10 inches will be required.

5.5 DESIGN OF TENSION MEMBERS

A tension member is a structural component loaded primarily in axial tension. In bridge design, tension members are used mostly as truss elements and occasionally as bracing (Figure 5-6). The direction of loading in tension members should always be parallel to the grain of the member. Timber is weak in tension perpendicular to the grain, and loading conditions that produce stress in this direction should be avoided. When loading conditions that induce tension perpendicular to the grain do exist, mechanical reinforcement must be designed to carry the load.

Discussions in this section apply to members loaded in axial tension only. Design criteria for members loaded in combined axial tension and bending are given in Section 5.7.

Figure 5-6.- Tension members in bridge applications are most common in trusses. This timber truss, located at Sioux Narrows, Ontario, Canada, spans 210 feet and is reputed to be the longest clear-span timber bridge in the world.

APPLIED STRESS

Applied stress in tension is computed by Equation 5-22:

$$f_t = \frac{P}{A} \tag{5-22}$$

where P = axial load applied to the member (lb), and

A = net cross-sectional area of the member (in^2).

The net area, A, in Equation 5-22 is the gross area of the member minus the projected area of fastener holes or cuts that reduce the section. Requirements for determining net area for various fasteners are discussed in Section 5.8.

ALLOWABLE STRESS

Allowable stress in tension equals the tabulated stress for tension parallel to grain, F_t, adjusted by all applicable modification factors. This is computed by

$$F_t' = F_t C_D C_M C_F C_t \tag{5-23}$$

For sawn lumber, values of F_t for members 2 to 4 inches thick, and 5 inches and wider, apply to 5- and 6-inch widths only. When wider members are used, a reduction in tabulated stress ranging from 0.9 to 0.6 is required by footnotes to the NDS Table 4A. When glulam is used, the most economical tension members are generally selected from the axial combinations given in *AITC 117-Design.*

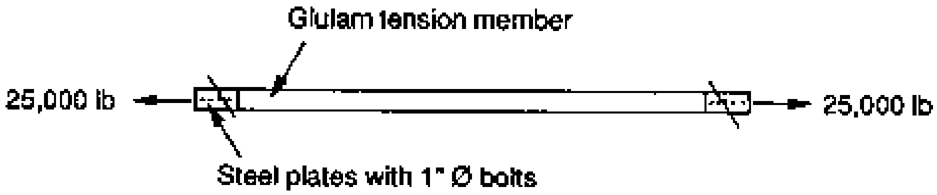
Example 5-8- Glulam tension member

A glulam truss member carries an axial tension load of 25,000 pounds. The ends of the member are attached to steel plates with a single row of 1-inch-diameter bolts aligned in the longitudinal direction. Design this truss member, assuming the following:

1. Normal load duration under wet-use conditions; adjustments for temperature (C_t) and fire-retardant treatment (C_R) are not applicable.

2. Bolt holes at member ends are 1/16 inch larger than the bolt diameter.

3. Glulam is manufactured from visually graded western species.



Solution

The design of a tension member starts with either the selection of a glulam combination symbol or a standard member width. In this example, combination symbol No. 2 is selected and design will involve determining the required member size.

The tabulated stress for tension parallel to grain is obtained for combination symbol No. 2 from *AITC 117-Design:*

$$F_t = 1{,}250 \text{ lb/in}^2$$

The allowable stress for tension parallel to grain is computed by Equation 5-23 using the C_M value obtained from Table 5-7:

$$F_t' = F_t C_M = 1{,}250(0.80) = 1{,}000 \text{ lb/in}^2$$

Next, Equation 5-22 is rearranged to compute an initial member area based on the applied load and the allowable stress in tension parallel to grain:

$$A = \frac{P}{F_t'} = \frac{25{,}000}{1{,}000} = 25 \text{ in}^2$$

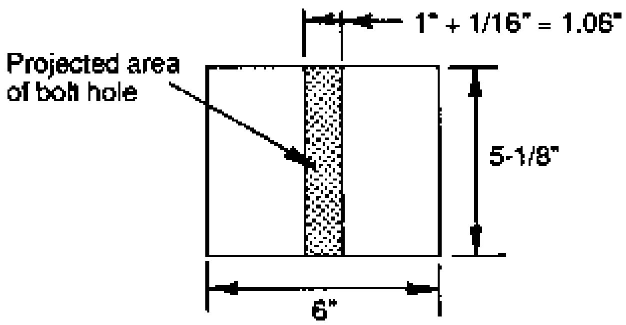
The required member depth is obtained for several standard glulam widths by dividing the required area by the standard width, then rounding the depth up to the next standard depth (based on a 1-1/2-inch lamination thickness for western species). For three standard glulam widths:

Member width	Minimum required depth	Depth rounded up to standard depth	Number of laminations
3-1/8 in.	$\frac{A}{b} = \frac{25}{3.125} = 8.00 \text{ in.}$	9 in.	6
5-1/8 in.	$\frac{A}{b} = \frac{25}{5.125} = 4.88 \text{ in.}$	6 in.	4
6-3/4 in.	$\frac{A}{b} = \frac{25}{6.75} = 3.70 \text{ in.}$	4.5 in.	3

Initial selection of a member width and depth is a matter of designer judgement and depends on size and economic considerations. In this case, the 5-1/8-inch width is selected and the gross member area is computed:

$$A_{GROSS} = b(d) = 5.125(6) = 30.75 \text{ in}^2$$

The net area used for design is equal to the gross area minus the projected area of bolt holes. Assuming that bolts pass through the narrow (5-1/8-inch) dimension,



$$A_{BOLT} = (1.06 \text{ in.})(5.125 \text{ in.}) = 5.43 \text{ in}^2$$

$$A_{NET} = A_{GROSS} - A_{BOLT} = 30.75 - 5.43 = 25.32 \text{ in}^2$$

By Equation 5-22,

$$f_t = \frac{P}{A} = \frac{25{,}000}{25.32} = 987 \text{ in}^2$$

$f_t = 987 \text{ in}^2 < F_t' = 1{,}000 \text{ lb/in}^2$, so a 5-1/8-inch wide by 6-inch deep combination symbol No. 2 member is satisfactory.

A column is a structural component loaded primarily in axial compression parallel to its length. In bridge design, columns are used as supporting components of the substructure, truss elements, and bracing (Figure 5-7). The three general types of columns are simple solid columns, spaced columns, and built-up columns (Figure 5-8). Simple solid columns consist of a piece of sawn lumber or glulam. Spaced columns consist of two or more parallel pieces that are separated and fastened at the ends and at one or more interior points by blocking. Built-up columns consist of a number of solid members joined together with mechanical fasteners. The most common columns for timber bridges are simple solid columns constructed of sawn lumber, glulam (axial combinations), timber piles, or poles. Although spaced and built-up columns may be used for truss elements or other components, they are not common in modem bridge applications.

The column design requirements in this section are limited to simple solid columns of constant cross-sectional area. Loads are applied concentrically, and design is based on the stresses and instability from axial compression and end-grain bearing stress at column ends. Columns loaded in combined compression and bending are discussed in Section 5.7 of this chapter. For additional information on built-up, spaced, and tapered solid columns, refer to the *NDS* and the *AITC Timber Construction Manual.*

Figure 5-7.- Timber columns are common in bridge substructures such as these bents (photo courtesy Wheeler Consolidated, Inc.).

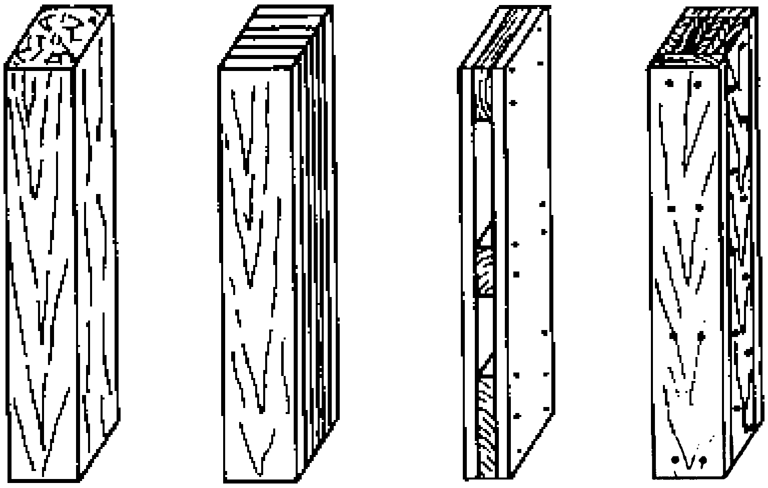


Figure 5-8. - General classes of timber columns.

DESIGN FOR COMPRESSION

Compression in timber columns can induce failure by crushing the wood fibers or by lateral buckling (deformation). The first step in column design is to estimate an initial member size and compute applied stress (several iterations may be required to arrive at a suitable section). After an initial column size is selected, the column slenderness ratio is computed, which serves as the basis for design in compression. From the slenderness ratio, allowable stress is determined from equations given in the NDS and checked against the applied stress.

Applied Stress

Applied column stress in compression parallel to grain, f_c, is computed by

$$f_c = \frac{P}{A} \qquad (5\text{-}24)$$

where P = the total compressive load supported by the column (lb), and

A = the cross-sectional area of the column (in^2).

The value of A used in Equation 5-24 depends on the location of fastener holes that reduce the column section. When the reduced section occurs at points of lateral support, failure occurs by wood crushing, and the gross column area is used without deductions for fastener holes. At locations away from points of lateral support, failure may occur by column buckling, and the net column area (gross column area minus fastener holes) is used. Refer to Section 5.8 for details on computing net area for different fastener types.

Column Slenderness Ratio

The slenderness ratio of a column provides a measure of the tendency of the column to fail by buckling from insufficient stiffness, rather than by crushing from insufficient strength. It is expressed as the ratio of the unsupported column length to its least radius of gyration and is computed for timber in the same manner as for other materials. For convenience in design, however, the slenderness ratio for square or rectangular simple solid columns is given in terms of the column cross-sectional dimension, rather than the radius of gyration, and is computed by

$$\text{Slenderness ratio} = \frac{\ell_e}{d} \tag{5-25}$$

where ℓ_e = effective column length (in.), and

d = cross-sectional dimension corresponding to ℓ_e (in.).

The effective column length in Equation 5-25 is the distance between two points along the column length at which the member is assumed to buckle in the shape of a sine wave. It is computed as the product of the unsupported column length and the effective buckling length factor given by

$$\ell_e = K_e \ell \tag{5-26}$$

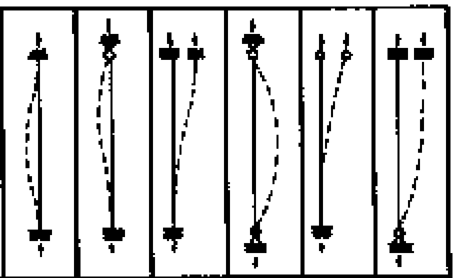
where K_e = effective buckling length factor, and

ℓ = unbraced length between points of lateral support along the column length.

Values of K_e are given in Table 5-13 for various conditions of end fixity and lateral translation at column ends or intermediate points of lateral support. In most applications, timber columns with square-cut ends are fixed against translation but not rotation (approximately pinned connections), and the value of K_e is 1.0. Conditions may be encountered in design where restraint is more or less than this condition, and K_e must be adjusted accordingly based on designer judgment. Additional discussion on effective buckling length factors is given in Appendix N of the NDS.

The slenderness ratio provides an indication of the mode of failure and is the basis for determining the allowable design stress. If a column is loaded to failure by buckling, the buckling will always occur about the axis with the largest slenderness ratio. The task of the designer is to determine the controlling slenderness ratio for a given column configuration. For a rectangular column with the same unbraced length in both directions, the critical slenderness ratio can be determined by inspection (Figure 5-9 A). In this case, the column will obviously buckle about the weaker *(y)* axis, and that is the only slenderness ratio that must be computed (for buckling about the *y* axis the column deflects in the *x* direction). For column configurations where the unbraced length is not the same in both directions,

Table 5-13. - Effective buckling length factor, K_e.

Buckling modes						
Theoretical K_e value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design K_e when ideal conditions approximated	0.65	0.80	1.2	1.0	2.10	2.4
End condition code	Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed,translation free Rotation free, translation free					

From the NDS;[26] © 1986. Used by permission.

the critical slenderness ratio cannot be determined by inspection and the designer must compute slenderness ratios for both directions (Figure 5-9 B). Depending on the spacing of lateral support, conditions may exist where the column design is controlled by buckling about the strong axis.

Allowable Stress

The allowable compressive stress for square or rectangular simple solid columns is computed from equations given in the NDS. These equations are based on the column slenderness for three ranges:

$0 < \ell_e/d$ 11	Short Column
$11 < \ell_e/d$ K	Intermediate Column
$K < \ell_e/d$ 50	Long Column

where K is a slenderness factor defined later in this section for intermediate columns.

The NDS equations have been modified to incorporate the use of the column dimension *(d)* rather than the radius of gyration *(r)*. They may be used for nonrectangular cross sections by substituting *3.46r* for d ($\ell_e/3.46r$ is used in place of ℓ_e/d when determining the column-length class). For the special case of a round column, the NDS states that the load on a round column may be taken as the same as that for a square column of the same cross-sectional area. For round columns, the d used in determing the ℓ_e/d ratio is 0.866 times the diameter of the round column.

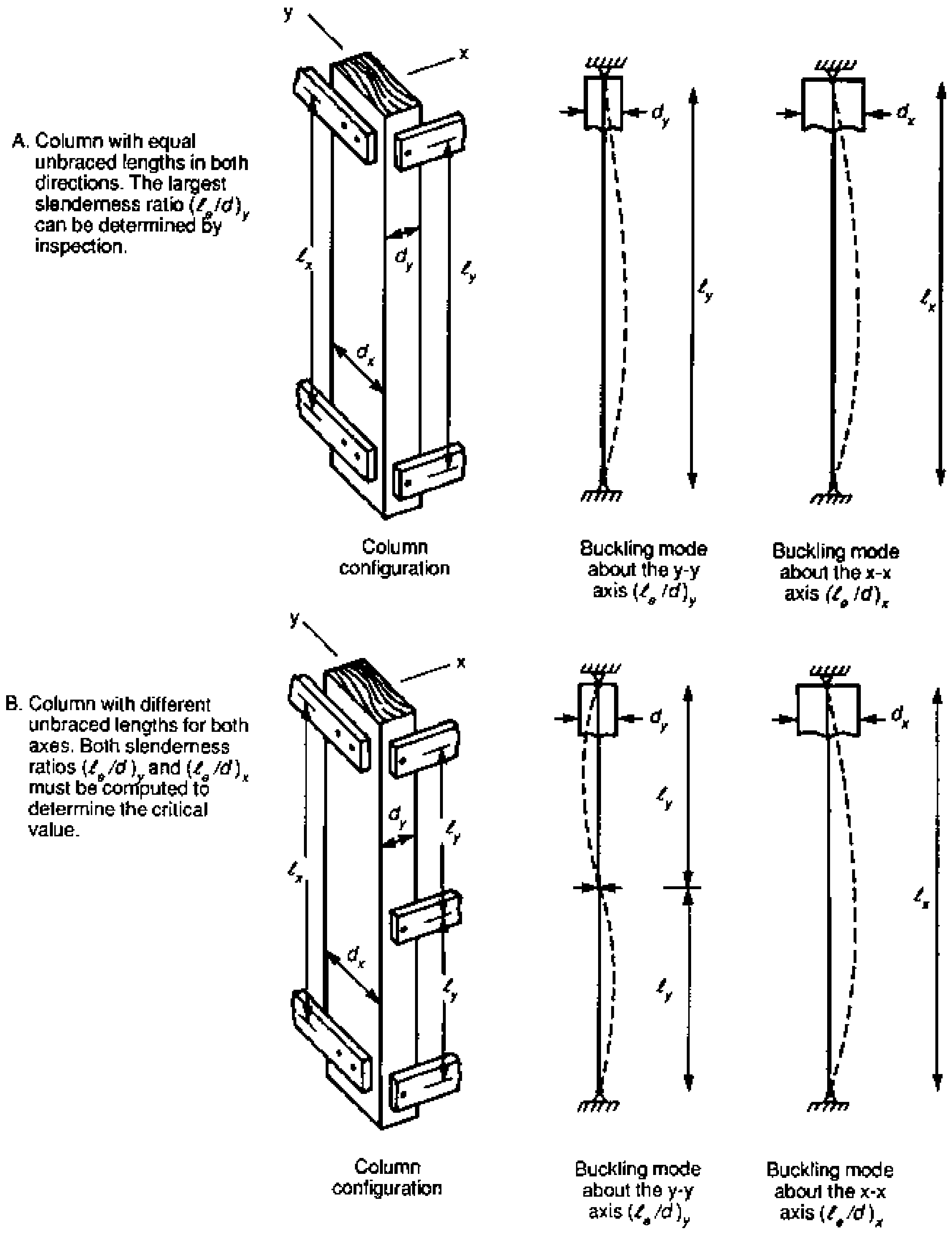


Figure 5-9.- Column slenderness ratios for columns with equal and unequal unbraced lengths.

Short Columns

Short columns are columns with a slenderness ratio of 11 or less. In short columns, the capacity of the member is controlled by the strength in compression parallel to grain, and failure always occurs by crushing of the wood fibers. Allowable stresses for short columns are equal to the tabulated stress in compression parallel to grain adjusted by applicable modification factors, as given by

$$F_c' = F_c C_D C_M C_R C_t \tag{5-27}$$

Intermediate Columns

Intermediate columns have a slenderness ratio greater than 11 but less than K as determined by

$$K = 0.671\sqrt{\frac{E'}{F_c''}} \tag{5-28}$$

where K = minimum value of ℓ_e/d at which the column can be expected to perform as an Euler column,[6]

$E' = EC_M C_R C_t$ (lb/in²), and

$F_c'' = F_c C_D C_M C_R C_t$ (lb/in²).

In intermediate columns, failure can occur by crushing of the wood fibers or by lateral buckling, or both. The allowable stress for intermediate columns is the tabulated stress in compression parallel to grain adjusted by applicable modification factors, including the lateral stability of columns factor, C_P, and is computed by

$$F_c' = F_c C_P C_D C_M C_R C_t \tag{5-29}$$

where

$$C_P = 1 - \frac{1}{3}\left(\frac{\ell_e / d}{K}\right)^4 \tag{5-30}$$

In addition to Equation 5-29, the NDS gives optional column design adjustments for low variability materials (such as glulam) that are similar to those previously discussed for beams. For additional information on these equations, refer to Appendix G of the NDS and the AITC *Timber Construction Manual.*

Long Columns

Long columns are columns with a slenderness ratio greater than K and less than or equal to 50 (the maximum slenderness ratio allowed by the NDS for any column is 50). In long columns, the strength of the member is controlled by stiffness, and failure occurs by lateral buckling. The allowable design stress for long columns is given by

$$F_c' = \frac{0.30E'}{(\ell_e / d)^2} \quad (5\text{-}31)$$

DESIGN FOR BEARING

Column design must also consider bearing on the end grain of the member, given by

$$f_g = \frac{P}{A} \quad (5\text{-}32)$$

where f_g = end-grain bearing stress from applied loads (lb/in^2),

P = total applied load (lb), and

A = net area in bearing (in^2).

The tabulated stress for end grain in bearing is specified in Table 2B of the NDS for sawn lumber and in Tables A-1 and A-2 of *AITC 117-Design* for glulam. The tabulated stress for sawn lumber is given for wet-service and dry-service conditions. For glulam, tabulated stress is for dry-service conditions and must be modified when the moisture content of the member is expected to exceed 16 percent in service (as in most bridge applications). Tabulated end-grain bearing stress is computed for sawn lumber and glulam as follows:

For sawn lumber,

$$F_g' = F_g C_D C_R C_t \quad (5\text{-}33)$$

For glulam,

$$F_g' = F_g C_M C_D C_R C_t \quad (5\text{-}34)$$

where F_g' = allowable stress for end grain in bearing (lb/in^2),

F_g = tabulated stress for end grain in bearing (lb/in^2), and

C_M = moisture modification factor for glulam for end grain in bearing = 0.57.

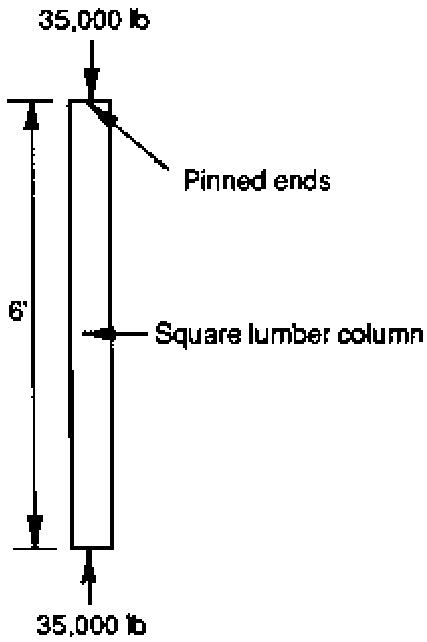
When the bearing stress computed by Equation 5-32 exceeds 75 percent of the allowable stress computed by Equations 5-33 or 5-34, the NDS requires that the bearing be on a metal plate or strap, or on other durable, rigid, homogeneous material of adequate strength.

Example 5-9. - Column design; sawn lumber

A square, sawn lumber column is 6 feet high and supports a concentric load of 35,000 pounds. Lateral support for the column is provided by pinned connections at the column ends only. Design this column, assuming the following:

1. Normal load duration and wet-use conditions; adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. The column is S4S Douglas Fir-Larch, visually graded No. 1 to WCLIB rules in the Posts and Timbers (P&T) size classification.



Solution

The first step in column design is to determine an initial column size. Since column dimensions are initially unknown, it is usually assumed that the column is in the short column slenderness range, and the allowable stress in compression parallel to grain is computed using Equation 5-27:

$$F_c' = F_c C_D C_M$$

From the NDS Table 4A for No. 1 Douglas Fir-Larch in the P&T size classification,

$$F_c = 1{,}000 \text{ lb/in}^2$$

From Table 5-7,

$$C_M = 0.91$$

Substituting values,

$$F_c' = F_c C_D C_M = 1{,}000(1.0)(0.91) = 910 \text{ lb/in}^2$$

An initial column area is obtained by dividing the applied load by F_c':

$$A = \frac{35{,}000 \text{ lb}}{910 \text{ lb/in}^2} = 38.5 \text{ lb/in}^2$$

From Table 16-2, the smallest square lumber size that meets the minimum area requirement is 8 inches by 8 inches, with the following properties:

b = 7.5 in.

d = 7.5 in.

A = 56.25 in^2

The column slenderness ratio must next be computed to determine the actual column slenderness range. The effective column length is computed by Equation 5-26 using an unbraced length of 6 feet and an effective buckling length factor, K_e, of 1.0 for the pinned ends:

$$\ell_e = K_e \ell = 1.0\,(6)(12\text{ in/ft}) = 72\text{ in.}$$

The column slenderness ratio is computed by Equation 5-25:

$$\text{Slenderness ratio} = \frac{\ell_e}{d} = \frac{72}{7.5} = 9.6$$

$\ell_e/d = 9.6 < 11.0$, so the column is in the short column slenderness range as initially assumed. Applied stress is computed by Equation 5-24:

$$f_c = \frac{P}{A} = \frac{35{,}000}{56.25} = 622\text{ lb/in}^2$$

f_c = 622 lb/in^2 < F_c' = 910 lb/in^2, so the column size is satisfactory.

Although normally not a controlling factor in column design, end grain in bearing stress should also be checked. From NDS Table 2B for wet-use Douglas Fir-Larch,

F_g = 1,340 lb/in^2

By Equation 5-33,

$$F_g' = F_g C_D = 1{,}340(1.0) = 1{,}340\text{ lb/in}^2$$

$$0.75F_g' = 0.75\,(1{,}340) = 1{,}005\text{ lb/in}^2$$

Assuming a unit weight for wood of 50 lb/ft^3

$$\text{Column weight} = \frac{56.25\text{ in}^2}{144\text{ in}^2/\text{ft}^2} = (6\text{ ft})\left(50\text{ lb/ft}^2\right) = 117.2\text{ lb}$$

By Equation 5-32,

$$f_g = \frac{P}{A} = \frac{(117.2 + 35{,}000)}{56.25} = 624 \text{ lb/in}^2$$

f_g = 624 lb/in^2 < $0.75F_g'$ = 1,005 lb/in^2, so end-grain bearing is satisfactory, and bearing on a steel plate or other rigid, homogeneous material is not required.

Summary

The column will be nominal 8-inch by 8-inch surfaced Douglas Fir-Larch, visually graded No. 1 in the P&T size classification. The column is classified in the short column slenderness range and f_c = 622 lb/in^2 < F_c' = 910 lb/in^2. End-grain bearing stress is less than 75 percent of the allowable value, so special steel bearing plates are not required.

Example 5-10- Glulam column design

A glulam column is 17 feet long, 8-1/2 inches wide and 12-3/8 inches deep. Determine the column capacity for concentric loading when (A) the column is laterally supported at the ends only, and (B) the column is laterally supported at the ends and at midheight along the 12-3/8-inch dimension. The following assumptions apply:

1. Normal load duration under wet-use conditions; adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not applicable.

2. Glulam is visually graded Southern Pine, combination symbol No. 47.

3. All support connections are pinned.

4. End-grain bearing is on a steel plate.

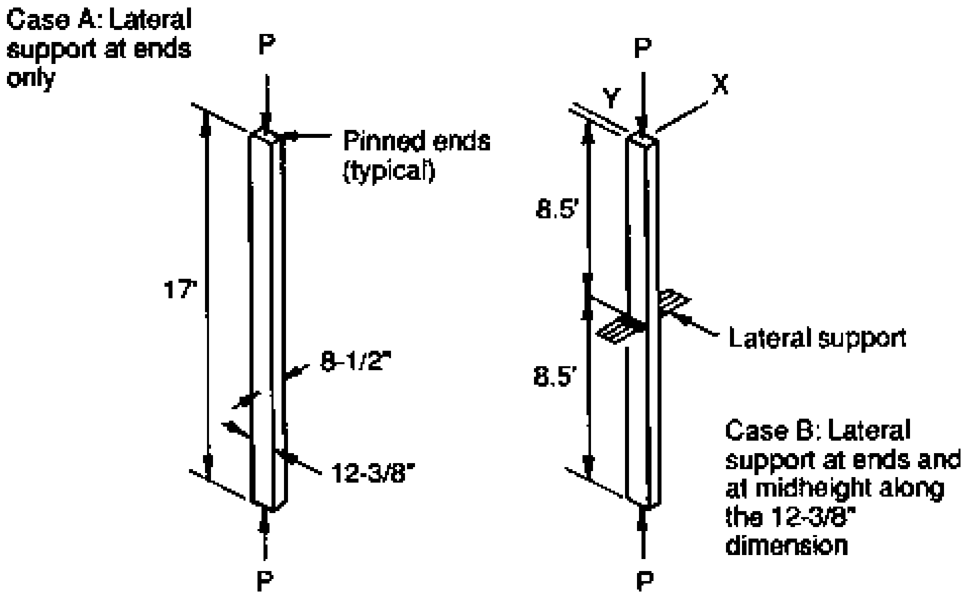
Solution

The procedure for determining the allowable load for each support condition will first involve computing the column slenderness range. From this, the allowable unit stress and load will be determined.

Tabulated values for compression parallel to grain and modulus of elasticity are obtained from *AITC 117--Design.* Respective values for the moisture content modification factor are obtained from Table 5-7:

F_c = 1,900 lb/in^2 $\quad$ C_M = 0.73

E = 1,400,000 lb/in^2 $\quad$ C_M = 0.833



From Table 16-4, the area of an 8-1/2-inch by 12-3/8-inch glulam column is 105.2 in^2.

Case A: Lateral support at column ends only

With lateral support at the column ends only, the effective column length is computed using Equation 5-26. For an unbraced column length of 17 feet and a buckling length factor for pinned ends of 1.0,

$$\ell_e = K_e \ell = 1.0(17)(12\text{ in/ft}) = 204\text{ in.}$$

The column slenderness ratio is computed using Equation 5-25 with the least column dimension, d = 8.5 inches:

$$\text{Slenderness ratio} = \frac{\ell_e}{d} = \frac{204}{8.5} = 24$$

$\ell_e/d = 24 > 11$, so the column is in the intermediate or long slenderness range. The slenderness factor, K, is computed using Equation 5-28:

$$E' = EC_M = 1{,}400{,}000(0.833) = 1{,}166{,}200\text{ lb/in}^2$$

$$F_c'' = F_c C_D C_M = 1{,}900(1.0)(0.73) = 1{,}387\text{ lb/in}^2$$

$$K = 0.671\sqrt{\frac{E'}{F_c''}} = 0.671\sqrt{\frac{1{,}166{,}200}{1{,}387}} = 19.46$$

$\ell_e/d = 24 > K = 19.46$, so the column is in the long slenderness range. Allowable stress in compression parallel to grain is computed by Equation 5-31:

$$F_c' = \frac{0.30E'}{(\ell_e/d)^2} = \frac{0.30\,(1{,}166{,}200)}{(24)^2} = 607 \text{ lb/in}^2$$

The allowable load is the product of the column area and F_c':

$$P = A(F_c') = 105.2(607) = 63{,}856 \text{ lb}$$

Case B: Lateral support at column ends and at midheight along the 12-3/8-inch dimension

With lateral support at the column ends and at midheight along one axis, the slenderness ratio must be checked for both axes. About the *x-x* axis:

$$\ell = (17 \text{ ft})(12 \text{ in/ft}) = 204 \text{ in.}$$

$K_e = 1.0$

$$\ell_e = K_e\,(\ell) = 1.0\,(204) = 204 \text{ in.}$$

$d = 12.38$ in.

$$\frac{\ell_e}{d} = \frac{204}{12.38} = 16.84$$

About the *y-y* axis:

$$\ell = \frac{17 \text{ ft}}{2}(12 \text{ in/ft}) = 102 \text{ in.}$$

$K_e = 1.0$

$$\ell_e = K_e\,(\ell) = 1.0\,(102) = 102 \text{ in.}$$

$d = 8.5$ in.

$$\frac{\ell_e}{d} = \frac{102}{8.5} = 12.00$$

The largest slenderness ratio of 16.48 (about the *x-x* axis) will control design. By previous calculations K = 19.46 > ℓ_e/d = 16.48, so the column is in the intermediate range.

The lateral stability of columns factor, C_p, is computed by Equation 5-30;

$$C_p = 1 - \frac{1}{3}\left(\frac{\ell_e/d}{K}\right)^4 = 1 - \frac{1}{3}\left(\frac{16.48}{19.46}\right)^4 = 0.83$$

Allowable stress in compression parallel to grain is computed by Equation 5-29:

$$F_c' = F_c C_p C_D C_M = 1{,}900(0.83)(1.0)(0.73) = 1{,}151 \text{ lb/in}^2$$

The allowable load is the product of the column area and F_c':

$$P = A(F_c') = 105.2(1{,}151) = 121{,}085 \text{ lb}$$

Check End-Grain Bearing

The tabulated stress for end grain in bearing is obtained from *AITC 117-Design:*

$$F_g = 2{,}300 \text{ lb/in}^2$$

The allowable stress is computed using Equation 5-34:

$$F_g' = F_g C_M C_D = 2{,}300(0.57)(1.0) = 1{,}311 \text{ lb/in}^2$$

$F_g' = 1{,}311$ lb/in^2 is greater than previously computed values of F_c', so bearing stress will not control.

Summary

The allowable compression parallel to grain and maximum load for both column support cases are as follows:

Case A: Column laterally supported at ends only

$F_c' = 607$ lb/in^2

Maximum allowable load = 63,856 lb

Case B: Column laterally supported at ends and at midheight along the 12-3/8-inch dimension

$F_c' = 1{,}151$ lb/in^2

Maximum allowable load = 121,085 lb

This example illustrates the effect that lateral support can have on allowable column loading. When additional support is added at midheight, along the 12-3/8-inch dimension, the allowable load nearly doubles.

One or more loads acting on a column, beam, or other structural member may induce a combination of axial and bending stresses that occur simultaneously. In bridge design, combined loading most commonly occurs as axial compression and bending acting on supporting columns of the substructure (Figure 5-10). Even in columns designed for concentric loads, small eccentricities are created because of construction tolerances, slight member curvature, and material variations. Bending stress also occurs when columns are subjected to transverse loads from wind or earthquakes (see Chapter 6). Other conditions involving combined compression and bending or combined tension and bending are less common in bridge applications, but may occur in truss members or other components.

The design requirements discussed in this section are for combined axial tension or compression acting simultaneously with bending. It is assumed that bending occurs about one axis and that all loads are applied directly to the member. For cases involving axial loads with biaxial bending or loads acting through brackets attached to the member side, refer to references listed at the end of this chapter.[6,7,8,21,26,34]

GENERAL DESIGN CONSIDERATIONS

When members are subjected to simultaneous axial and bending loads, the resulting stress distribution is approximately the sum of the effects of the individual loads. In combined tension and bending, the effect is to reduce the compressive stress on one side of the member and increase the tensile stress on the other side. For combined compression and bending, tensile stress is reduced on one side and compressive stress is increased on the other. The case of combined compression and bending is critical because the higher compression increases the potential for lateral buckling of the member.

Combined stresses are evaluated using an interaction formula. In general terms, the interaction formula contains two expressions, one for the capacity in axial loading and one for the capacity in bending. In its basic form, the interaction formula is expressed by

$$\frac{f_a}{F_a'} + \frac{f_b}{F_b'} \leq 1.0 \qquad (5\text{-}35)$$

where f_a = applied stress in tension f_t or compression f_c (lb/in^2), and

F_a' = allowable stress in tension F_t' or compression F_c' (lb/in^2).

Each of the expressions in Equation 5-35 can be thought of as representing the portion of the total member capacity taken by the respective axial or bending stress. The axial portion of the formula is the ratio of the applied axial stress to the allowable axial stress, assuming the member is loaded

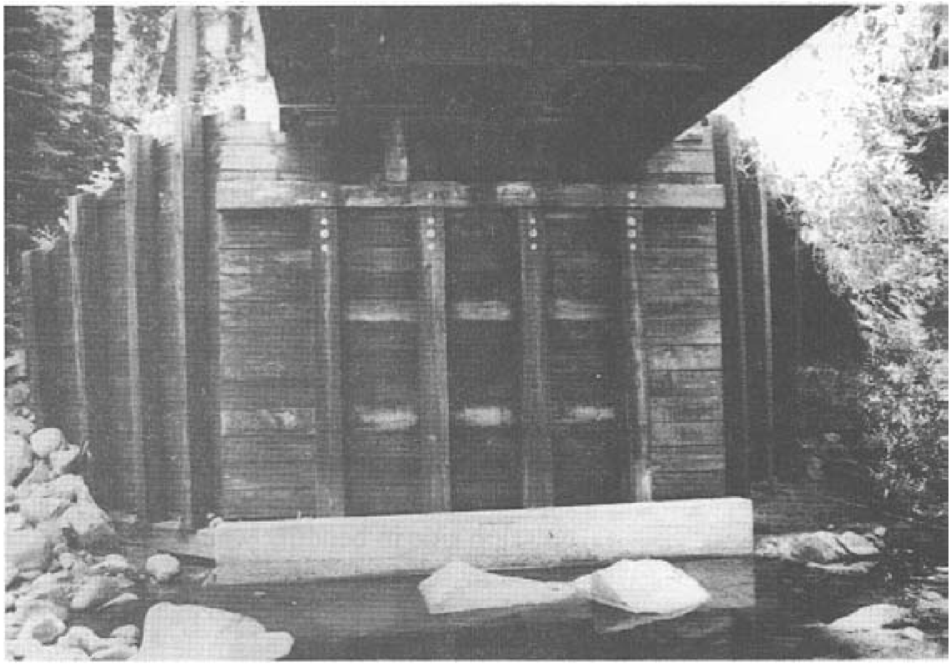


Figure 5-10.- Members subjected to combined axial and bending forces are most common in bridge substructures. The vertical posts of this abutment support compressive loads from the superstructure and lateral loads from the earth pressure on the abutment wall.

with axial forces only. The bending portion is the ratio of the applied bending stress to the allowable bending stress, assuming the member is loaded with bending forces only. The sum of these expressions cannot exceed 1.0, or 100 percent of the member capacity.

When selecting a glulam member for combined axial and bending stresses, the designer should consider the relative magnitude of each type of stress. If tension or compression is the predominant stress, axial combinations are usually most economical. When bending is the predominant stress, bending combinations may be more appropriate.

COMBINED BENDING AND AXIAL TENSION

When members are loaded in combined axial tension and bending, the interaction equations that must be satisfied for design are given by

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b'' C_F} \leq 1.0 \qquad (5\text{-}36)$$

$$\frac{f_b - f_t}{F_b'' C_L} \leq 1.0 \qquad (5\text{-}37)$$

where f_t = applied stress in axial tension computed by Equation 5-22 (lb/in²),

$F_t' = F_t C_D C_M C_R C_t$ from Equation 5-23 (lb/in²),

f_b = applied bending stress computed by Equation 5-2 (lb/in^2), and

$F_b'' = F_b C_D C_M C_R C_t$ from Equation 5-9 (lb/in^2).

In applying the interaction formulas, tension stress is computed for a tension member, as discussed in Section 5.5, and bending stress is computed for a beam, as discussed in Section 5.4. Considerations for tension are relatively straightforward; however, for bending, the member must be checked for strength in the tension zone and stability in the compression zone. In beam design, the size factor, C_F, applies to the tension side of the member where stresses from combined loading are greater than those from bending alone. As a result, C_F is always used as a modification factor in Equation 5-36. The lateral stability of beams factor, C_L, affects the compression side in bending where stresses from combined loading are reduced by the axial tension. When conditions of lateral support are such that the member is classified as an intermediate or long beam, and C_L rather than C_F controls beam design, the member must also meet the stability requirements given in Equation 5-37.

COMBINED BENDING AND AXIAL COMPRESSION

Members subjected to combined axial compression and bending are common in bridge design and are frequently referred to as beam columns. This type of loading is more critical than combined tension and bending because of the potential for lateral buckling and the additional bending stress created by the *P-delta* effect. The *P-delta* effect is produced when bending loads cause the axially loaded member to deflect along its longitudinal axis. When this occurs, an additional moment is generated by the axial load, *P*, acting over a lever arm equal to the deflected distance (Figure 5-11). The potential magnitude of the *P-delta* moment depends on the stiffness of the member and is not computed directly; however, the interaction equations for combined compression and bending include additional terms to compensate for this effect.

The exact analysis of a member with combined axial compression and bending can be a very time-consuming task and is most accurately determined by the secant formula. When timber members are considered, such an exacting analysis is generally not justified because of the material variability in modulus of elasticity and in strength properties and because of the degree of uncertainty in loading conditions. Rather than using a rigorous type of analysis, the NDS gives a simplified interaction formula for combined compression and bending that provides an accuracy well within an acceptable range for bridge applications. These equations are suitable for pin-end members of square or rectangular cross sections and are based on the following assumptions given in the NDS:

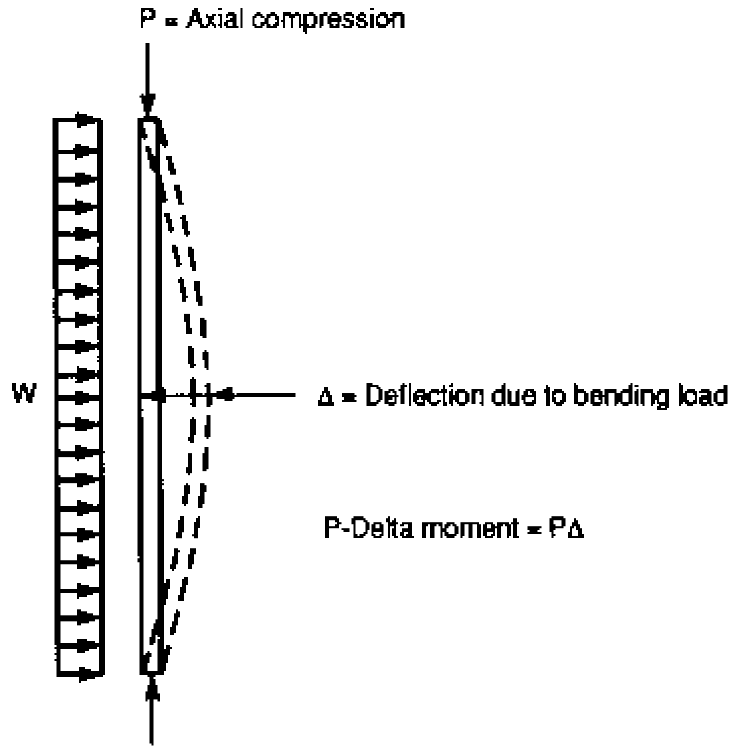


Figure 5-11.- P-delta effect on members loaded in combined axial compression and bending.

1. The stresses that cause a given deflection as a sinusoidal curve are the same as those for a beam with a uniform side load.

2. For a single concentrated side load, the stress under the load can be used, regardless of the position of the load with reference to the length of the column.

3. The stress to use with a system of side loads is the maximum stress from the system (some slight error on the side of overload will occur with large side loads near each end).

4. For columns with a slenderness ratio of 11 or less (short columns), the *P-delta* stress may be neglected.

The NDS interaction formula for combined compression and bending is given below by Equations 5-38 and 5-39. Appendix H of the NDS also gives eight modified forms of this equation for specified loading conditions that may be used at the option of the designer.

$$\frac{f_c}{F_c'}+\frac{f_b+f_c(6+1.5J)(e/d)}{F_b'-J(f_c)}\leq 1.0 \qquad (5\text{-}38)$$

and

$$= \frac{(\ell_e / d) - 11}{K - 11} \qquad 0 \le J \le 1.0 \tag{5-39}$$

where f_c = applied stress in compression parallel to grain computed by Equation 5-24 (lb/in^2),

F_c' = allowable stress in compression parallel to grain by the applicable equations in Section 5.6 for the maximum slenderness ratio (ℓ_e/d), assuming the member is loaded in axial compression only (lb/in^2),

f_b = applied stress in bending from side loads or moments only, by Equation 5-2 (lb/in^2),

F_b' = allowable stress in bending computed by equations in Section 5.4, assuming the member is loaded in bending only (lb/in^2),

e = the eccentricity of an eccentrically applied axial load (in.),

d = the cross-sectional dimension of a rectangular or square column (in.),

J = a unitless convenience factor computed from the ℓ_e/d ratio in the plane of bending and limited to values between zero and 1.0, inclusively,

ℓ_e/d = for computing J, the column slenderness ratio of the member in the plane of bending, and

K = the smallest slenderness ratio ℓ_e/d at which the long column formula applies, from Equation 5-28.

The interaction Equations 5-38 and 5-39 are somewhat confusing at first glance, but become easier to use with experience. When applying the equations, five considerations will provide some clarification for various design applications. First, the compression terms f_c and F_c' are determined by the methods discussed in Section 5.6, in exactly the same manner as if the member was loaded in axial compression only.

Second, the term for bending stress f_b is applicable only when bending is from transverse loads or applied moments. When bending is from eccentric axial loads only, and no side loads or applied moments occur, f_b equals zero and Equation 5-38 becomes

$$\frac{f_c}{F_c'} + \frac{f_c(6 + 1.5J)(e/d)}{F_b' - J(f_c)} \le 1.0 \tag{5-40}$$

Third, the allowable bending stress F_b' is the tabulated bending stress adjusted by all applicable modification factors, assuming the member is loaded in bending only. In most applications, the more restrictive modification factor for size effect, C_F or lateral beam stability, C_L, applies; however, for combined compression and bending, both modification factors are applied cumulatively when the value of C_F is greater than 1.0. This will occur only when axial glulam combinations are less than 12 inches deep and are loaded in bending about the *y-y* axis (see *AITC 117-Design).* In all other cases, only the lowest value computed for C_F or C_L is applied as a modification factor to F_b.

Fourth, in the expression for eccentric loads *e/d, d* is the cross-sectional dimension of the member perpendicular to the axis about which bending is applied. When there are no eccentric axial loads, *e/d* equals zero and Equation 5-38 reduces to

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b' - J(f_c)} \leq 1.0 \qquad \textbf{(5-41)}$$

Fifth, the *J* factor, whose value is limited between zero and 1.0, compensates for the effects of the *P* -delta moment. The column slenderness ratio used to determine *J* is always computed in the plane of bending. For column slenderness ratios of 11 or less (short columns), *P* -delta effects are ignored and the value *J* is zero. For ℓ_e/d values greater than *K* (long columns), the *P* -delta effects are greatest and *J* is at its maximum value of 1.0. When ℓ_e/d is greater than 11 but less than *K, P* -delta effects increase with the slenderness ratio and values of *J* vary linearly from zero to 1.0.

5.8 CONNECTIONS

A connection consists of two or more members joined with one or more mechanical fasteners. Connections are one of the most important considerations in timber bridge design because they provide continuity to the members as well as strength and stability to the system. The connections may consist entirely of wood members but frequently involve the connection of wood to steel or other materials. One advantage of wood as a structural material is the ease with which the members can be joined with a wide variety of fasteners. Progress in the past decade on fastener design and performance has led to reliable design criteria, allowing connections. to be designed with the same accuracy as other components of the structure.

This section discusses connection design for several types of fasteners commonly encountered in bridge construction. The types of connections and fasteners are discussed first, followed by basic design criteria and specific fastener requirements. The scope of coverage is limited to connections with two or three members, where fasteners are loaded perpendicular or parallel to their axis in the side grain of timber members. When fasteners are loaded at an angle to their axis, placed in wood end grain, or used in joints consisting of more than three members, refer to the NDS specifications for design criteria and requirements.

TYPES OF CONNECTIONS AND FASTENERS

There are two basic types of connections in timber bridges: lateral (shear) connections and withdrawal (tension) connections (Figure 5-12). In lateral connections, forces are transmitted by bearing stresses developed between the fastener and the members of the connection. A tight lateral connection also develops some strength by friction between members (at least when initially installed), but this effect is not considered in design. In withdrawal connections, the mechanism of load transfer depends on the type of fastener. For screw-type fasteners, load transfer is by a combination of friction and thread interaction between the fastener and the wood. For driven fasteners, such as nails, load transfer in withdrawal is entirely by friction developed between the fastener and the wood.

Selection of a fastener for a specific design application depends on the type of connection and the required strength capacity. Each connection must be designed to adequately transmit forces and provide good performance for the life of the structure without causing splitting, cracking, or deformation of the wood members. The five fastener types most commonly used for timber bridges are bolts, lag screws, timber connectors, nails or spikes, and drift bolts or pins (Figure 5-13). A brief description of each fastener is given below.

Bolts are the most common timber fastener for lateral connections where moderately high strength is required. They also are used in tension connections where loads are applied parallel to the bolt axis. Bolts used for bridge connections are standard machine bolts and should not be confused with machine screws, which have a much finer thread. Bolts are the only type of fastener that require nuts to maintain tightness of the connection.

Lag screws are pointed threaded fasteners with a square or hexagonal head that are placed in wood members by turning with a wrench. Although they provide a lower lateral strength than a comparable bolted connection, lag screws are advantageous when an excessive bolt length is required or when access to one side of a connection is restricted.

Timber connectors are steel rings or plates placed between members held by a bolt or lag screw. They are used in lateral connections only and provide the highest lateral strength of all fasteners because of the large bearing area provided by the connector.

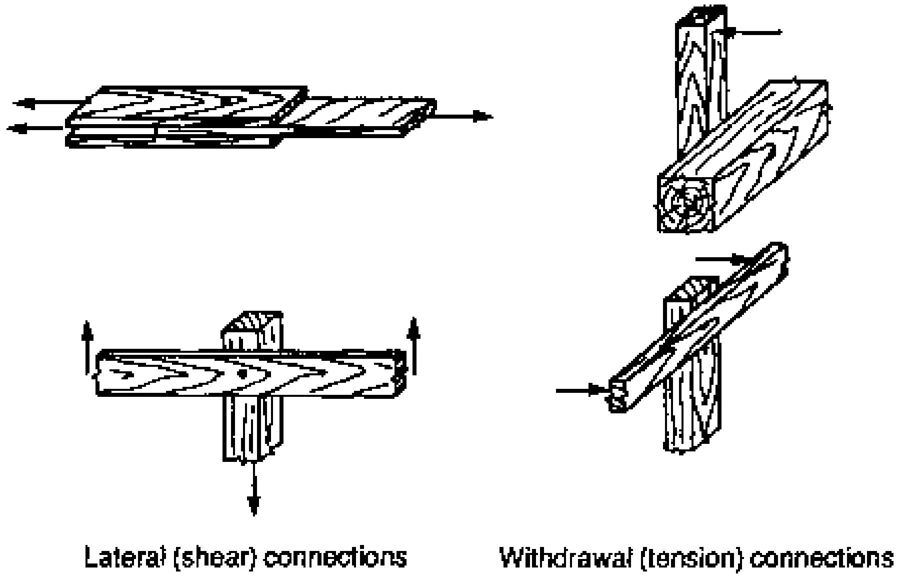


Figure 5-12.- Typical lateral and withdrawal connections for timber members.

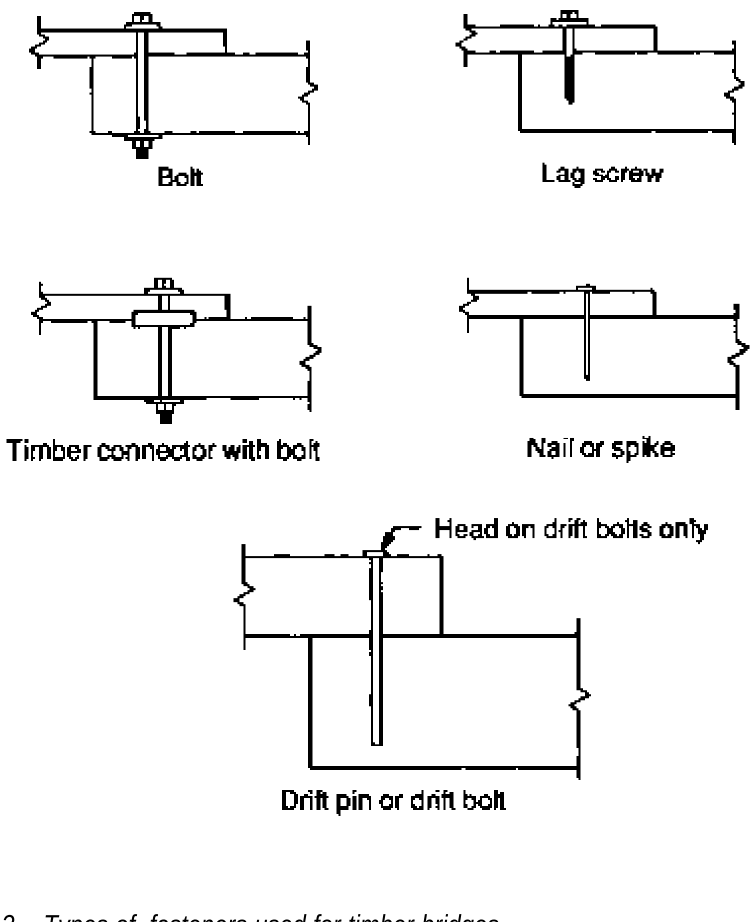


Figure 5-13. - Types of fasteners used for timber bridges.

Nails and spikes are driven fasteners used in bridges primarily for non-structural applications. They are more susceptible than other fasteners to loosening from vibrations and from dimensional changes in the wood caused by moisture content variations.

Drift bolts and drift pins are long unthreaded bolts or steel pins that are driven in prebored holes. Drift bolts have a head on one end, but drift pins have no head. In bridge applications, drift bolts and drift pins are used in lateral connections for large timber members. They are not suitable for withdrawal connections because of their low resistance to withdrawal loads.

When bolts or lag screws are used individually or with timber connectors, they must be provided with washers if the head or nut of the fastener is in wood contact. Washers distribute the load over a larger area to reduce stress and prevent wood crushing under the fastener head when the fastener is tightened. The three primary types of washers are cut washers, plate washers (round or square), and malleable iron washers (Figure 5-14). Cut washers are limited in application because they are thin and may bend from bearing forces. Malleable iron (MI) washers, intended only for timber connections, are most commonly used. Washers are not required when the head or nut of the fastener bears on a steel component; however, when steel components are used, they must be designed for adequate strength in accordance with AASHTO specifications for structural steel (AASHTO Section 10).

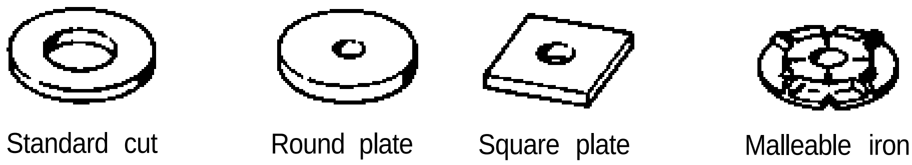


Figure 5-14. - Common washer types for timber connections.

An important factor in connection performance and longevity is protection of the steel fasteners and hardware from corrosion. All steel components should be hot-dip galvanized in accordance with the applicable AASHTO specification M111 or M232. Such finishes as chrome and cadmium plating do not afford suitable protection for the exposure conditions encountered in bridges. When color is an important consideration, components can be painted after galvanizing or be coated with colored epoxy.

BASIC DESIGN CRITERIA

The strength of timber connections is usually controlled by the strength of the wood in bearing or withdrawal rather than by the strength of the fastener. As a result, connection design is affected by many of the same factors that influence the strength properties of wood. In addition to the type, number, and size of fasteners, connection strength depends on such factors as the wood species, direction and duration of load, and conditions of use.

Tabulated design values for different types of fasteners are given in the NDS. These values are based on one fastener, installed and used under specified conditions. Allowable design loads are determined by adjusting tabulated values with modification factors. When more than one fastener is used in a connection, the design value is the sum of the design values for the individual fasteners (for some fastener types, adjustments are required for multiple-fastener connections). It should be noted that the design criteria and tabulated values in the NDS are limited to connections involving the same type of fastener. Methods of analysis and test data for connections made with more than one type of fastener have not been developed.

The basic design procedures for connections are similar to those for structural components. For a given connection and fastener type, the designer must (1) compute fastener load requirements; (2) determine the tabulated value for one fastener based on the species group of the connected members; (3) apply modification factors to the tabulated value to reflect specific conditions; (4) adjust the modified value for lateral loading conditions other than parallel or perpendicular to grain, when applicable; (5) multiply the allowable design value for one fastener by the total number of fasteners in the connection; (6) compute the net section and verify the capacity of the members; and (7) detail the connection to ensure adequate fastener placement and performance.

Species Groups

The strength of timber connections is directly related to the species (density) of wood in which the fastener is installed. For lateral connections, wood species are divided into groups depending on the relative bearing capacity of the species for the specific fastener type. The three species groups consist of Groups 1 to 12 for bolts, Groups A to D for timber connectors, and Groups I to IV for lag screws, nails, spikes, drift bolts, and drift pins. There are no group designations for withdrawal connections, and design values are based on the specific gravity of the member. For both lateral and withdrawal connections, the species groups and specific gravities for sawn lumber (Table 5-14) and axial combinations of glulam (Table 5-15) apply to fasteners in the side grain at any location in the member. For bending combinations of glulam (Table 5-16), the species and grade of laminations vary for different locations in the member, and fastener groups and specific gravities are given separately for the tension face, side face, and compression face.

Modification Factors for Fasteners

Tabulated design values for fasteners are based on the strength of wood components assuming specific conditions of use. To adjust tabulated

Table 5-14.—Sawn lumber species groups for fastener design.

Species	Bolt load group	Timber connector load group[1]	Grouping for lag screws, nails, spikes, drift bolts and drift pins: Load group	Specific gravity[2]
Cedar, Northern White	12	D	IV	0.31
Cedars, Western[3]	9	D	IV	0.35
Coast Species	12	D	IV	0.39
Douglas Fir-Larch[3]	3	B	II	0.51
Douglas Fir-Larch (dense)	1	A	II	0.51[5]
Douglas Fir, South	6	C	III	0.48
Eastern Woods	12	D	IV	0.38
Fir, Balsam	11	D	IV	0.38
Hem-Fir[3]	8	C	III	0.42
Hemlock:				
Eastern-Tamarack[3]	8	C	III	0.45
Mountain	9	C	III	0.47
Western	8	C	III	0.48
Pine:				
Eastern White[3]	11	D	IV	0.38
Idaho White	11	D	IV	0.40
Lodgepole	10	C	III	0.44
Northern	9	C	III	0.46
Ponderosa[4]	11	C	III	0.49
Ponderosa-Sugar	11	C	III	0.42
Red[4]	11	C	III	0.42
Southern	3	B	II	0.55
Southern (dense)	1	A	II	0.55[5]
Western White	11	D	IV	0.40
Spruce:				
Eastern	10	C	III	0.43
Engelmann-Alpine Fir	12	D	IV	0.36
Sitka	10	C	III	0.43
Sitka, Coast	10	D	IV	0.39
Spruce-Pine-Fir	10	C	III	0.42
West Coast Woods (mixed species)	12	D	IV	0.35
White Woods (Western Woods)	12	D	IV	0.35

[1] When stress graded.

[2] Based on weight and volume when ovendry.

[3] Also applies when species name includes the designation "North."

[4] Applies when graded to NLGA rules.

[5] The specific gravity of dense lumber is slightly higher than for medium-grain lumber; however, the design values for this group are based on the average specific gravity of the species.

Load groups and specific gravities apply to all grades of that species unless otherwise noted.

This table contains a limited number of species. Refer to the NDS for a complete species listing.

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Table 5-15. - Glulam axial combination species groups for fastener design.

Combination symbol	Bolt group	Timber connector group	Lag screws, nails, spikes, drift bolts, and drift pins	
			Group	Specific gravity
Visually graded western species:				
1	3	B	II	0.51
2	3	B	II	0.51
3	1	A	II	0.51
4	3	B	II	0.51
5	1	A	II	0.51
Visually graded Southern Pine:				
46	3	B	II	0.55
47	3	B	II	0.55
48	1	A	II	0.55
49	3	B	II	0.55
50	1	A	II	0.55

Applicable to fasteners placed in any face of the member.

This table represents a partial listing of selected combination symbols. Refer to *AITC 117—Design*[4] and the *AITC Timber Construction Manual*[6] for a complete listing of all combination symbols. From *AITC 117—Design*,[4] © 1987. Used by permission.

values for actual design requirements, modification factors are applied to tabulated values in the same manner as those for strength properties. The modification factors for fasteners consist of the following:

- C_M moisture content factor
- C_D duration of load factor
- C_t temperature factor
- C_R fire-retardant treatment factor
- C_e edge-distance factor
- C_n end-distance factor
- C_s spacing factor
- C_g group action factor
- C_{st} steel side-plate factor
- C_{lb} lag-screw factor

A summary of fastener modification factors and their applicability to various fasteners are shown in Table 5-17.

Moisture Content Factor *(C_M)*

The moisture content of timber components affects joint strength in approximately the same manner as it affects other strength properties. For sawn lumber, moisture content must be considered at the time of fabrication (when the fastener is installed) and in service. For glulam, all laminations are dry when fabricated, and moisture effects are considered for in-use conditions only. Tabulated fastener values are based on fasteners that

Table 5-16.—Glulam bending combination species groups for fastener design.

	Tension face				Side face				Compression face			
			Lag screws, nails, spikes, drift bolts, and drift pins				Lag screws, nails, spikes, drift bolts, and drift pins				Lag screws, nails, spikes, drift bolts, and drift pins	
Combination symbol	Bolt group	Timber conn. group	Group	Specific gravity	Bolt group	Timber conn. group	Group	Specific gravity	Bolt group	Timber conn. group	Group	Specific gravity
Visually graded western species												
16F-V3	3	B	II	0.51	3	B	II	0.51	3	B	II	0.51
16F-V6	3	B	II	0.51	3	B	II	0.51	3	B	II	0.51
20F-V3	1	A	II	0.51	3	B	II	0.51	3	B	II	0.51
20F-V7	1	A	II	0.51	3	B	II	0.51	1	A	II	0.51
24F-V4	1	A	II	0.51	3	B	II	0.51	1	A	II	0.51
24F-V8	1	A	II	0.51	3	B	II	0.51	1	A	II	0.51
Visually graded Southern Pine												
16F-V2	3	B	II	0.55	3	B	II	0.55	3	B	II	0.55
16F-V5	3	B	II	0.55	3	B	II	0.55	3	B	II	0.55
20F-V3	3	B	II	0.55	3	B	II	0.55	3	B	II	0.55
20F-V5	1	A	II	0.55	3	B	II	0.55	1	A	II	0.55
24F-V3	1	A	II	0.55	3	B	II	0.55	1	A	II	0.55
24F-V5	1	A	II	0.55	3	B	II	0.55	1	A	II	0.55

This table represents a partial listing of selected combination symbols. Refer to *AITC 117—Design*[4] and the AITC *Timber Construction Manual*[5] for a complete listing of all combination symbols.

Table 5-17.—Applicability of modification factors for fasteners.

	Modification factor									
Fastener type	Duration of load C_D	Moisture content C_M	Temperature C_t	Fire retardant C_R	Edge dist. C_e	End dist. C_n	Spacing C_s	Group action C_g	Steel side plate C_{st}	Lag screw C_d
Timber connectors										
Split rings	X	X	X	X	X	X	X	X	—	X
Shear plates	X	X	X	X	X	X	X	X	X	X
Bolts	X	X	X	X	—	X	X	X	X	—
Lag screws	X	X	X	X	—	X	X	X	—	—
Nails and spikes	X	X	X	X	—	—	—	—	X	—
Drift bolts and drift pins	X	X	X	X	—	X	X	X	X	—

X = modification factor is applicable.

— = modification factor does not apply.

C_e, C_n, and C_s are not cumulative, and the most restrictive of the three values is used for design.

are installed and used in continuously dry conditions that do not exceed 19-percent moisture content for sawn lumber and 16-percent moisture content for glulam. For other conditions, tabulated values must be adjusted by C_M (Table 5-18). Note that C_M values for fasteners may vary from those used for other strength properties.

Duration of Load Factor *(C_D)*

Tabulated fastener values are for conditions where maximum loads are of normal duration. For other loading conditions, values are adjusted by the duration of load factor, C_D, discussed in Section 5.3. The duration of load factor applies to wood members only and is not used for metal components. As a result, load increases due to application of C_D may be limited when the capacity of the connection is controlled by the strength of the steel connector rather than by the strength of the wood. This is discussed further in the following sections on fastener design.

Temperature Factor *(C_t)*

The strength of a wood connection is affected by temperature in the same manner as wood components. In unusual cases where connections will be subjected to prolonged temperatures in excess of 150 $^{\circ}$F, fastener values should be adjusted by the temperature factor, C_t. Values and criteria for this factor are the same as those given in Section 5.3.

Fire-Retardant Treatment Factor *(C_R)*

Fire-retardant treatments are not common in bridge applications. However, when timber components are treated with fire-retardant chemicals, tabulated fastener values must be reduced by the fire-retardant treatment factor, C_R, discussed in Section 5.3.

Table 5-18 Fastener load modification factors for moisture content, C_M.

	Condition of wood[1]				
	Sawn lumber			Glulam	
Type of fastener	At time of fabrication	In service	C_M	In service	C_M
Timber connectors[2]	Dry	Dry	1.0	Dry	1.0
	Partially seasoned[3]	Dry	Note 3	Wet	0.67
	Wet	Dry	0.8		
	Dry or wet	Partially seasoned or wet	0.67		
Bolts or lag screws	Dry	Dry	1.0	Dry	1.0
	Partially seasoned or wet[3]	Dry	See below	Wet	0.67
	Dry or wet	Exposed to weather	0.75		
	Dry or wet	Wet	0.67		
Drift bolts or pins - laterally loaded	Dry or wet	Dry	1.0	Dry	1.0
	Dry or wet	Partially seasoned, wet or subject to wetting and drying	0.70	Wet	0.7
Wire nails and spikes — Withdrawal loads	Dry	Dry	1.0	Dry	1.0
	Partially seasoned or wet	Will remain wet	1.0	Wet	0.25
	Partially seasoned or wet	Dry	0.25		
	Dry	Subject to wetting and drying	0.25		
— Lateral loads	Dry	Dry	1.0	Dry	1.0
	Partially seasoned or wet	Dry or wet	0.75	Wet	10.75
	Dry	Partially seasoned or wet	0.75		
Threaded, hardened steel nails	Dry or wet	Dry or wet	1.0	Dry	1.0
				Wet	1.0

[1] Condition of wood definitions applicable to fasteners are as follows:

"Dry" wood has a moisture content of 19% or less for sawn lumber and 16% or less for glulam.

"Wet" wood has a moisture content at or above the fiber saturation point (approximately 30%) for sawn lumber and above 16% for glulam.

"Partially seasoned" wood has a moisture content greater than 19%, but less than fiber saturation point.

"Exposed to weather" implies that the wood may vary in moisture content from dry to partially seasoned, but is not expected to reach the fiber saturation point at times when the joint is under full design load.

"Subject to wetting and drying" implies that the wood may vary in moisture content from dry to partially seasoned or wet, or vise versa, with consequent effects on the tightness of the joint.

[2] For timber connectors, moisture content limitations apply to a depth of 3/4 inch from the surface of the wood.

[3] When timber connectors, bolts, or laterally loaded lag screws are installed in wood that is partially seasoned at the time of fabrication, but that will be dry before full design load is applied, intermediate values may be used.

From NDS;[26] © 1986. Used by permission.

Moisture modification factors C_M for laterally loaded bolts and lag screws in sawn lumber seasoned in place[1]

Arrangement of bolts or lag screws	Type of splice plate	C_M
One fastener only, or Two or more fasteners placed in a single line parallel to grain, or Fasteners placed in two or more lines parallel to grain with separate splice plates for each line.	Wood or metal	1.0
All other arrangements	Wood or metal	0.4

[1] Factors apply when wood is at or above the fiber saturation point (wet) at time of fabrication but dries to a moisture content of 19% or less (dry) before full design load is applied. For wood partially seasoned when fabricated, adjusted intermediate values may be used.

Edge-Distance Factor *(C_e)*

Edge distance is the distance from the center of a fastener to the edge of the member, measured perpendicular to grain (Figure 5-15). For loads applied perpendicular to the grain, the loaded edge is the edge toward which the load induced by the fastener acts. The unloaded edge is the opposite edge. Tabulated design values for bolts, lag screws, timber connectors, drift bolts, and drift pins are based on the full edge-distance requirements specified for the fastener. For timber connectors, it is permissible to reduce the edge distance provided the tabulated value for the connector is reduced by C_e (design tables for timber connectors include tabulated values reduced by C_e). The edge-distance factor is not cumulative with the end-distance factor *(C_n)* or the spacing factor *(C_s)*. Of the three factors, the most restrictive value is used for design.

End-Distance Factor *(C_n)*

End distance is the distance from the center of a fastener to the end of the member (Figure 5-15). Tabulated values for bolts, lag screws, timber connectors, drift bolts, and drift pins are based on the full end-distance requirements for the fastener. Reduced end distances are permitted if the tabulated fastener value is reduced by C_n. End distance requirements and values of C_n for individual fasteners are discussed later in this section. The end-distance factor is not cumulative with edge-distance factor *(C_e)* or the spacing factor *(C_s)* Of the three factors, the most restrictive value is used for design.

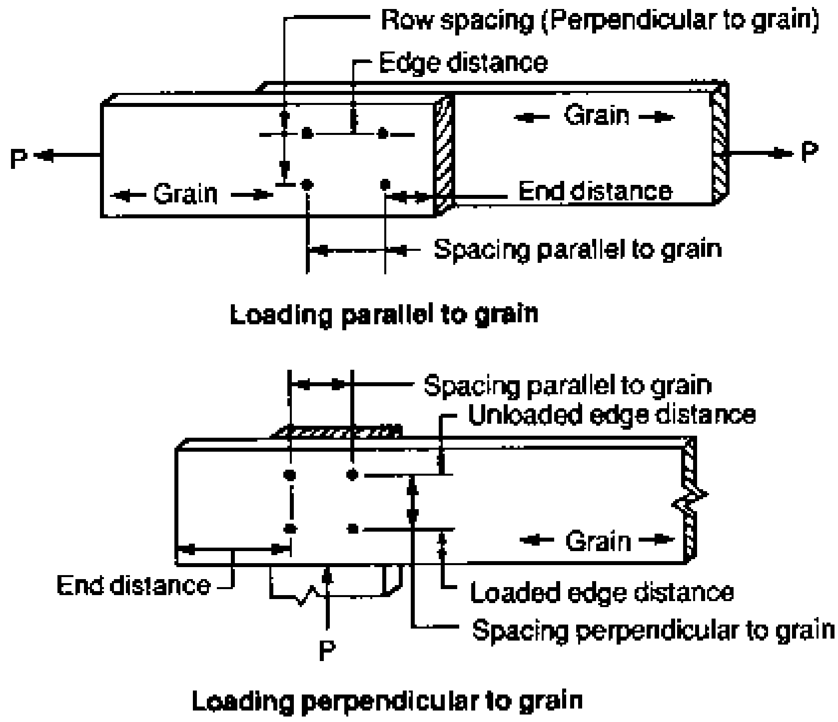


Figure 5-15.-- Edge distance, end distance, and spacing for fasteners.

Spacing Factor (C_s)

Fastener spacing is the center-to-center distance between fasteners, measured parallel or perpendicular to grain (Figure 5-15). Tabulated design values for bolts, lag screws, timber connectors, drift bolts, and drift pins are based on minimum spacing requirements between fasteners. When spacings are less than the minimum, tabulated fastener values must be reduced by the spacing factor, C_s. Spacing requirements and values of C_s depend on the type of fastener and are discussed later in this section. The spacing factor is not cumulative with the edge-distance factor (C_e) or the end-distance factor. Of the three factors, the most restrictive value is used for design.

Group Action Factor (C_g)

A row of fasteners consists of two or more bolts, lag screws, timber connectors, drift bolts, or drift pins aligned in the direction of the applied load. When three or more of these fasteners are used in a row, the capacity of the connection is less than that computed by multiplying the value of an individual fastener by the total number of fasteners. To compensate for this effect, tabulated values for individual fasteners in the row are reduced by the group action factor, C_g. Values of C_g are given in Table 5-19 and are based on the gross areas of the members and the total number of fasteners in the row. It should be noted that the group action factor given in the NDS is applied to the group of fasteners acting as a whole. However, it also may be applied to individual fasteners, as presented here. Applying the factor to individual fasteners is more convenient for design and is more consistent with the application of other modification factors. Procedures for determining C_g are demonstrated in examples later in this section.

Steel Side-Plate Factor (C_{st})

The distribution of stress in a lateral connection depends on the material of the side members. Tabulated fastener values in the NDS are based on the assumption that all side members are wood. When steel side members are used, tabulated values for some fasteners may be increased by C_{st}. The value of C_{st} depends on the type of fastener and direction of loading and is discussed later in this section. For lag screws, a separate table of design values for metal side plates is given in the NDS, and adjustment by C_{st} is not required.

Lag-Screw Factor (C_{lb})

Tabulated values for timber connectors are based on a bolted connection. When lag screws are used instead of bolts, tabulated values must be adjusted by the lag-screw factor C_{lb}.

Loads at an Angle to the Grain

The strength of a laterally loaded wood connection for all fasteners other than nails and spikes depends on the direction of fastener bearing in relation to the grain of the members. Design values in the NDS are

Table 5-19. —Group action modification factor C_g for laterally loaded bolts, lag screws and timber connectors

Connections With Wood Side Plates

A_1/A_2 [2,3]	A_1 (in^2) [4]	Number of fasteners in a row[1]										
		2	3	4	5	6	7	8	9	10	11	12
0.5	<12	1.00	0.92	0.84	0.76	0.68	0.61	0.55	0.49	0.43	0.38	0.34
	12 – <19	1.00	0.95	0.88	0.82	0.75	0.68	0.62	0.57	0.52	0.48	0.43
	19 – <28	1.00	0.97	0.93	0.88	0.82	0.77	0.71	0.67	0.63	0.59	0.55
	28 – <40	1.00	0.98	0.96	0.92	0.87	0.83	0.79	0.75	0.71	0.69	0.66
	40 – <64	1.00	1.00	0.97	0.94	0.90	0.86	0.83	0.79	0.76	0.74	0.72
	>64	1.00	1.00	0.98	0.95	0.91	0.88	0.85	0.82	0.80	0.78	0.76
1.0	<12	1.00	0.97	0.92	0.85	0.78	0.71	0.65	0.59	0.54	0.49	0.44
	12 – <19	1.00	0.98	0.94	0.89	0.84	0.78	0.72	0.66	0.61	0.56	0.51
	19 – <28	1.00	1.00	0.97	0.93	0.89	0.85	0.80	0.76	0.72	0.68	0.64
	28 – <40	1.00	1.00	0.99	0.96	0.92	0.89	0.86	0.83	0.80	0.78	0.75
	40 – <64	1.00	1.00	1.00	0.97	0.94	0.91	0.88	0.85	0.84	0.82	0.80
	>64	1.00	1.00	1.00	0.99	0.96	0.93	0.91	0.88	0.87	0.86	0.85

A_1 = gross cross-sectional area of the main member before boring or grooving.[5]
A_2 = sum of the cross-sectional areas of the side members before boring or grooving.[5]

Connections With Metal Side Plates

A_1/A_2	A_1 (in^2)	Number of fasteners in a row[1]										
		2	3	4	5	6	7	8	9	10	11	12
2–12	5 – <8	1.00	0.78	0.64	0.54	0.46	0.40	0.35	0.30	0.25	0.20	0.15
	8 – <16	1.00	0.85	0.73	0.63	0.54	0.48	0.42	0.38	0.34	0.30	0.26
	16 – <24	1.00	0.91	0.83	0.74	0.66	0.59	0.53	0.48	0.43	0.38	0.33
	24 – <39	1.00	0.94	0.87	0.80	0.73	0.67	0.61	0.56	0.51	0.46	0.42
	39 – <64	1.00	0.96	0.92	0.87	0.81	0.75	0.70	0.66	0.62	0.58	0.55
	64 – <119	1.00	0.98	0.95	0.91	0.87	0.82	0.78	0.75	0.72	0.69	0.66
	119 – <199	1.00	0.99	0.97	0.95	0.92	0.89	0.86	0.84	0.81	0.79	0.78
12–18	17 – <24	1.00	0.94	0.88	0.81	0.74	0.67	0.61	0.55	0.49	0.43	0.37
	24 – <39	1.00	0.96	0.91	0.86	0.80	0.74	0.68	0.62	0.56	0.50	0.44
	39 – <64	1.00	0.98	0.94	0.90	0.85	0.80	0.75	0.70	0.67	0.62	0.58
	64 – <119	1.00	0.99	0.96	0.93	0.90	0.86	0.82	0.79	0.75	0.72	0.69
	119 – <199	1.00	1.00	0.98	0.96	0.94	0.92	0.89	0.86	0.83	0.80	0.78
	>199	1.00	1.00	1.00	0.98	0.97	0.95	0.93	0.91	0.90	0.88	0.87
18–24	40 – <64	1.00	1.00	0.96	0.93	0.89	0.84	0.79	0.74	0.69	0.64	0.59
	64 – <119	1.00	1.00	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.76	0.73
	119 – <199	1.00	1.00	0.99	0.98	0.96	0.94	0.92	0.90	0.88	0.86	0.85
	>199	1.00	1.00	1.00	1.00	0.98	0.96	0.95	0.93	0.92	0.92	0.91
24–30	40 – <64	1.00	0.98	0.94	0.90	0.85	0.80	0.74	0.69	0.65	0.61	0.58
	64 – <119	1.00	0.99	0.97	0.93	0.90	0.86	0.82	0.79	0.76	0.73	0.71
	119 – <199	1.00	1.00	0.98	0.96	0.94	0.92	0.89	0.87	0.85	0.83	0.81
	>199	1.00	1.00	0.99	0.98	0.97	0.95	0.93	0.92	0.90	0.89	0.89
30–35	40 – <64	1.00	0.96	0.92	0.86	0.80	0.74	0.68	0.64	0.60	0.57	0.55
	64 – <119	1.00	0.98	0.95	0.90	0.86	0.81	0.76	0.72	0.68	0.65	0.62
	119 – <199	1.00	0.99	0.97	0.95	0.92	0.88	0.85	0.82	0.80	0.78	0.77
	>199	1.00	1.00	0.98	0.97	0.95	0.93	0.90	0.89	0.87	0.86	0.85
35–42	40 – <64	1.00	0.95	0.89	0.82	0.75	0.69	0.63	0.58	0.53	0.49	0.46
	64 – <119	1.00	0.97	0.93	0.88	0.82	0.77	0.71	0.67	0.63	0.59	0.56
	119 – <199	1.00	0.98	0.96	0.93	0.89	0.85	0.81	0.78	0.76	0.73	0.71
	>199	1.00	0.99	0.98	0.96	0.93	0.90	0.87	0.84	0.82	0.80	0.78

A_1 = gross cross-sectional area of the main member before boring or grooving.[5]
A_2 = sum of the cross-sectional areas of metal side plates before drilling.

1 When fasteners in adjacent rows are staggered, refer to the NDS[26] for requirements for determining the number of fasteners in a row.
2 When $A_1/A_2 > 1.0$, use A_2/A_1.
3 For values of A_1/A_2, between 0 and 1.0, interpolate or extrapolate from tabulated values.
4 When $A_1/A_2 > 1.0$, use A_2 instead of A_1.
5 When a wood member is loaded perpendicular to grain, its equivalent gross cross-sectional area for computing A_1 and A_2 is the product of the thickness of the member and the overall width of the fastener group.

From NDS.[26] © 1986. Used by permission.

tabulated for loads acting parallel to grain *(P)* and perpendicular to grain *(Q)*. When the loads act at some intermediate angle (Figure 5-16), design values are computed using the Hankinson formula given by

$$N' = \frac{P'Q'}{P'\sin^2\theta + Q'\cos^2\theta} \tag{5-42}$$

where N' = allowable design value at an angle to the grain (lb),

P' = allowable value for the fastener parallel to grain (lb),

Q' = allowable value for the fastener perpendicular to grain (lb), and

θ = angle between the direction of load and the direction of grain (degrees).

For bolts, lag screws, drift bolts, and drift pins, the Hankinson formula is applied after tabulated values are adjusted by modification factors, and the value N' is the allowable fastener value used for design. For timber connectors, modification factors for distance and spacing are based on the angle of the load to the grain, and C_e, C_n, and C_s are applied after N is computed by the Hankinson formula.

Member Capacity

The strength of a timber connection depends not only on the strength of the fasteners but also on the structural capacity of the connected members. As a part of the design process, the capacity of all members must be checked to ensure that factors related to the fasteners and connection

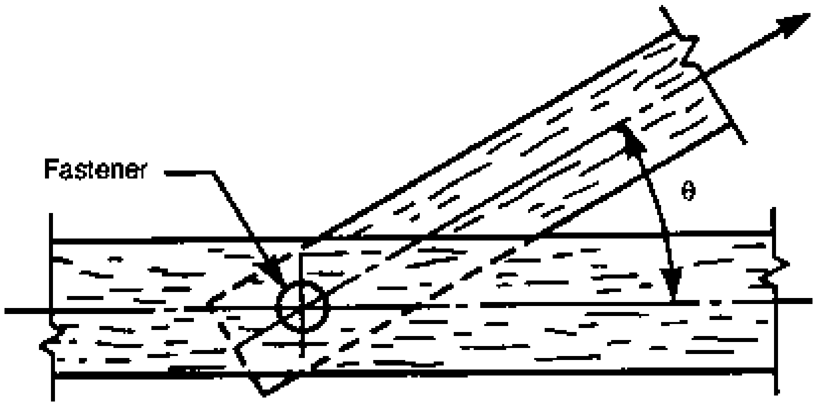


Figure 5-16. - Fastener loading applied at an angle to the grain.

configuration have not reduced the load-carrying capacity of the members. Connection-related factors that may affect member capacity include net area, eccentric loading, and shear capacity.

Net Area

The net area of a member is the cross-sectional area remaining after subtracting the area of material removed for fastener placement. The cross section where the net area is taken is called the critical section. Depending on the type of loading and size and placement of fasteners, the reduction in area for fasteners can significantly reduce member capacity. Requirements for determining net area vary among fasteners and are discussed in more detail later in this section.

Eccentricity

Eccentric loading is produced at connections when the resultant member forces are offset at the connection (Figure 5-17). Eccentricity in connections induces tension perpendicular to grain and can severely reduce the capacity of the members. The strength of eccentric connections is difficult to evaluate, and connections of this type must be avoided unless tests are employed in design to ensure that members can safely carry applied loads.

Shear Capacity

When fastener loads are applied transverse to beams or other components, the capacity of the member in horizontal shear may be reduced. Although not common in most bridge applications, this can occur when beams are supported entirely by fasteners, without bearing on another member, or when fastener loads are applied transverse to the beam (Figure 5-18). When conditions such as these are encountered, refer to the NDS for special provisions on computing horizontal shear in the member.

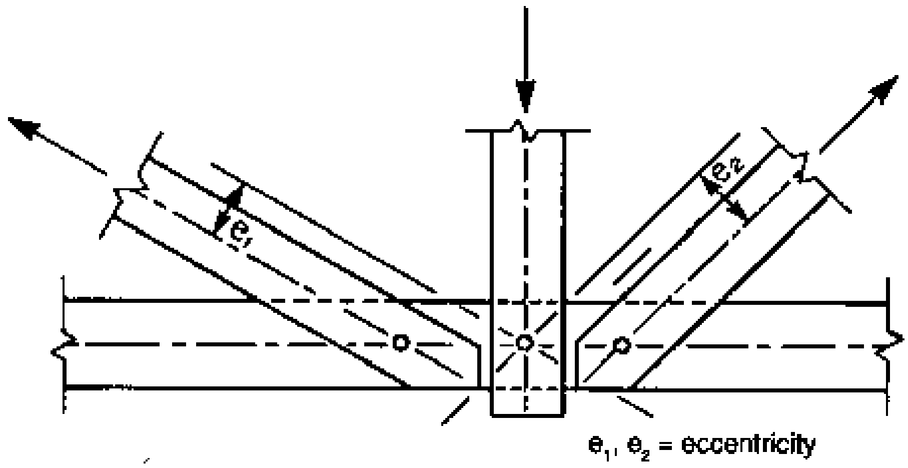


Figure 5-17. - Example of an eccentrically loaded connection. Connections of this type can induce tension perpendicular to the grain and substantially reduce the capacity of connected members.

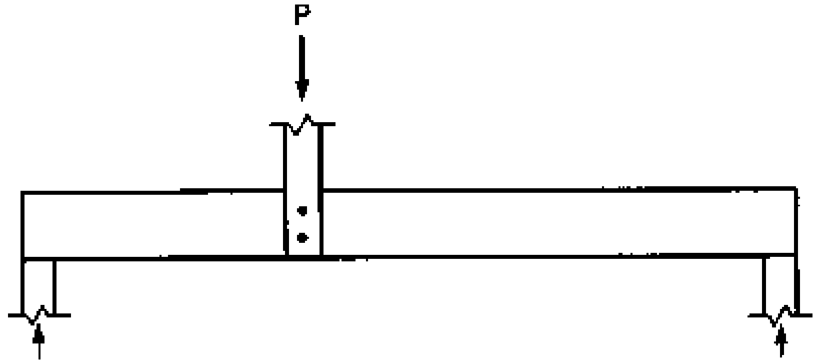


Figure 5-18. - Unsupported fastener loads applied transverse to the beam axis can reduce beam capacity in horizontal shear. Refer to the NDS [26] for design requirements for this loading condition.

BOLTS

Bolts are the most common mechanical fasteners in timber bridge connections. They are used for lateral connections in double shear (three members) or single shear (two members) and in tension connections where the bolt is loaded parallel to its axis. The bolts most commonly used for timber connections conform to *ASTM Standard A307, Low-Carbon Steel Externally and Internally Threaded Standard Fasteners.* The allowable design stresses for these bolts are 20,000 lb/in^2 in tension and 10,000 lb/in^2 in shear. Bolts are generally available in diameters of 1/4 inch to 2 inches and lengths up to 24 inches or more in 1/2-inch increments. However, the designer should verify availability before specifying diameters over 1-1/4 inches or lengths over 16 inches. When long lengths are required, threaded rods conforming to ASTM A307 may be more practical.

Bolts are manufactured in a variety of types based on the configuration of the bolt head. The most common types are the hexagonal head, square head, dome head, and flat head (Figure 5-19). The standard hexagonal or square heads are used when the bolt head is in contact with wood or steel. More specialized bolts, such as the dome head and flat head, provide an

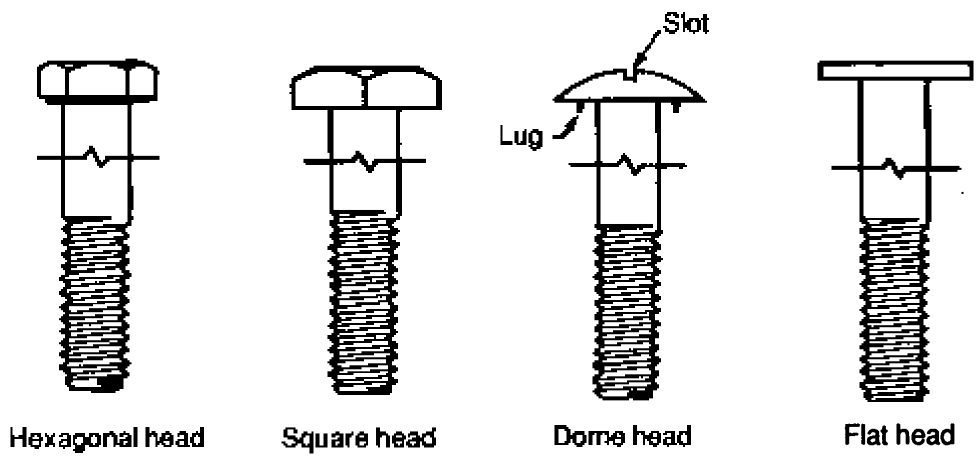


Figure 5-19.- Bolt types used for timber connections.

increased head diameter and are used when the bolt head is in contact with wood. Bolts with dome heads also are referred to as economy bolts or mushroom bolts and may be slotted or provided with lugs to facilitate installation and tightening.

Net Area

The net area at a bolted connection is equal to the gross area of the timber member minus the projected area of the bolt holes at the section (bolt holes are typically 1/32 to 1/16 inch larger than the bolt diameter). For parallel-to-grain loading with staggered bolts, the nearest bolt in the adjacent row is considered to occur at the same critical section unless the parallel-to-grain spacing of bolts in each row is a minimum of eight times the bolt diameter (Figure 5-20). The required net area in tension and compression members is determined by dividing the total load transferred through the critical section by the applicable allowable stress (F_t' or F_c') for the species and grade of material used.

Design of Lateral Connections

The strength of laterally loaded, bolted connections is developed by bearing between the bolt and the wood (Figure 5-21). The capacity of the

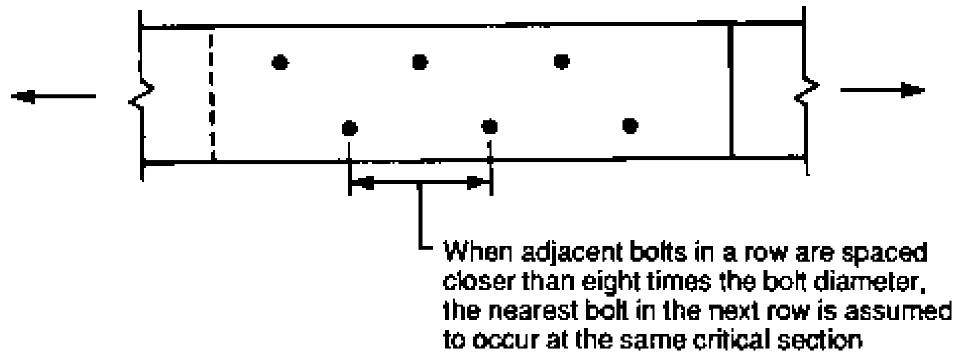


Figure 5-20. - Critical section for determining net area for staggered bolts loaded parallel to grain.

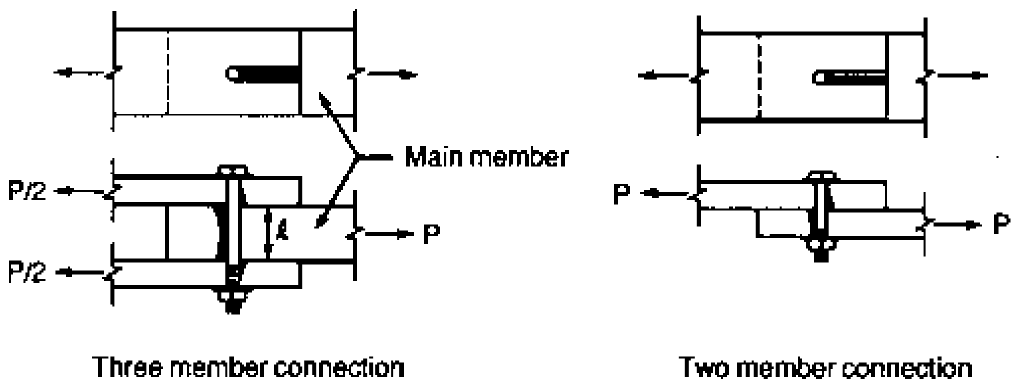


Figure 5-21. - Typical configuration and stress distribution for a laterally loaded bolted connection.

connection depends on the bearing strength of the wood and the slenderness ratio of the bolt. The slenderness ratio is defined as the length of the bolt in the main member (ℓ) divided by the bolt diameter *(D).* For bolted connections with low slenderness ratios, the bolt is relatively stiff, and the full bearing strength of the connection is developed. As the slenderness ratio increases, bolt stiffness is reduced, and bending may occur before full bearing strength is achieved, reducing the capacity of the connection.

The allowable value for one bolt is equal to the tabulated design value adjusted by all applicable modification factors and loading at an angle to the grain, when required. When more than one bolt is used, the allowable connection value is the sum of the design values of the individual bolts, adjusted by the group action factor, C_g. The applicable modification factors for loading parallel to grain *(P)* and perpendicular to grain *(Q)* are given for laterally loaded bolts by

$$P' = PC_D C_M C_t C_R C_n C_s C_g C_{st} \tag{5-43}$$

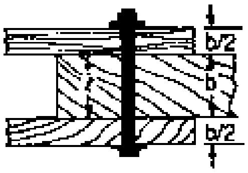
$$Q' = QC_D C_M C_t C_R C_n C_s C_g \tag{5-44}$$

Tabulated Design Values

Tabulated bolt design values are given in the NDS for one A307 bolt in a wood-to-wood, three-member connection where the side members are each a minimum of one-half the thickness of the main member (double shear). A portion of the NDS tables for several species groups is shown in Table 5-20. To determine the tabulated value for one bolt, enter the table with the length of bolt in main member and bolt diameter and read the tabulated values for loading parallel to grain and perpendicular to grain for the applicable species group. When joints have side pieces that are of a species different from that of the main member, the design value is the lesser of that obtained by assuming a comparable joint with all members the same species as the main member, or all members the same species as the side members.

Although tabulated values in the NDS are for a balanced three-member connection, the table also is used for other member thicknesses and two-member connections (Table 5-21). For three-member connections loaded parallel to grain, with side members that are less than one-half the thickness of the main member, the tabulated value is determined by assuming a main member twice the thickness of the thinnest side member. When steel side plates are used, the length of bolt is based on the thickness of the wood member. For a bolted connection consisting of two members of equal thickness loaded parallel to grain (single shear), the tabulated value is one-half that given for a main member the same thickness as the members. When the two members are of unequal thickness, the tabulated value is the lesser of one-half the tabulated value for the thicker member, or one-half the tabulated value for a piece twice the thickness of the thinner member. For a two-member connection consisting of one wood member

Table 5-20.—Tabulated design values for laterally loaded bolts.



				Species Group 1		Species Group 3		Species Group 8		Species Group 9		Species Group 10		Species Group 11	
				DOUGLAS FIR-LARCH (Dense), SOUTHERN PINE (Dense)		CALIFORNIA REDWOOD (Close grain), DOUGLAS FIR-LARCH SOUTHERN PINE, SOUTHERN CYPRESS		EASTERN HEMLOCK TAMARACK, CALIFORNIA REDWOOD (Open grain), HEM-FIR, WESTERN HEMLOCK		MOUNTAIN HEMLOCK, WESTERN CEDARS, NORTHERN PINE		SPRUCE-PINE-FIR, SITKA SPRUCE, YELLOW POPLAR, LODGEPOLE PINE		RED PINE, WESTERN WHITE PINE, PONDEROSA PINE-SUGAR PINE, EAST. WHITE PINE, EAST. SPRUCE, BALSAM FIR, IDAHO WHITE PINE	
Length of bolt in main member	Diameter of bolt		Projected area of bolt	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain
ℓ	D	ℓ/D	$A=\ell \times D$	P	Q	P	Q	P	Q	P	Q	P	Q	P	Q
2-1/2	1/2	5.00	1.250	1480	830	1260	720	1180	460	1100	500	1080	470	1010	340
	5/8	4.00	1.563	2140	950	1820	810	1620	520	1510	560	1410	530	1310	390
	3/4	3.33	1.875	2700	1050	2310	900	1990	580	1860	620	1700	590	1580	430
	7/8	2.86	2.188	3210	1160	2740	990	2330	630	2180	690	1980	650	1840	480
	1	2.50	2.500	3680	1270	3150	1080	2670	690	2490	750	2260	700	2110	520
3	1/2	6.00	1.500	1490	970	1270	860	1210	550	1130	590	1160	560	1080	410
	5/8	4.80	1.875	2290	1130	1960	970	1810	620	1690	670	1640	630	1520	470
	3/4	4.00	2.250	3080	1270	2630	1080	2340	690	2180	750	2030	700	1890	520
	7/8	3.43	2.625	3760	1390	3220	1190	2780	760	2600	820	2380	780	2210	570
	1	3.00	3.000	4390	1520	3750	1300	3200	830	2990	900	2710	850	2530	630
3-1/2	1/2	7.00	1.750	1490	1120	1270	980	1210	640	1130	690	1160	650	1080	480
	5/8	5.60	2.188	2320	1310	1980	1130	1890	720	1760	780	1790	740	1660	550
	3/4	4.67	2.625	3280	1470	2800	1260	2570	810	2400	870	2310	820	2150	610
	7/8	4.00	3.063	4190	1630	3580	1390	3180	890	2970	960	2760	900	2570	670
	1	3.50	3.500	5000	1770	4270	1520	3710	970	3460	1050	3170	990	2950	730
4	1/2	8.00	2.000	1490	1010	1270	1010	1210	700	1130	760	1160	720	1080	550
	5/8	6.40	2.500	2330	1410	1990	1290	1900	830	1770	900	1820	840	1690	620
	3/4	5.33	3.000	3340	1690	2850	1440	2690	920	2510	1000	2520	940	2350	690
	7/8	4.57	3.500	4440	1850	3790	1590	3470	1010	3240	1100	3090	1030	2880	770
	1	4.00	4.000	5470	2030	4670	1730	4150	1110	3880	1200	3600	1130	3360	840
4-1/2	5/8	7.20	2.813	2330	1440	1990	1400	1900	930	1770	1010	1820	950	1690	700
	3/4	6.00	3.375	3350	1830	2860	1620	2730	1040	2550	1120	2610	1060	2440	780
	7/8	5.14	3.938	4540	2110	3880	1790	3630	1140	3390	1240	3360	1160	3130	860
	1	4.50	4.500	5770	2280	4930	1950	4500	1250	4200	1350	3990	1270	3710	940
	1-1/4	3.60	5.625	7970	2670	6810	2280	5930	1460	5540	1580	5080	1490	4740	1100
5-1/2	5/8	8.80	3.438	2330	1390	1990	1410	1900	1010	1770	1090	1820	1030	1690	820
	3/4	7.33	4.125	3350	1930	2860	1880	2730	1260	2550	1360	2620	1280	2440	960
	7/8	6.29	4.813	4570	2400	3900	2180	3720	1400	3470	1510	3560	1420	3320	1050
	1	5.50	5.500	5930	2760	5070	2380	4820	1520	4500	1650	4550	1550	4240	1150
	1-1/4	4.40	6.875	8930	3260	7630	2790	6930	1780	6470	1930	6110	1820	5690	1350
7-1/2	5/8	12.00	4.688	2330	1260	1990	1260	1890	950	1770	1030	1820	960	1690	800
	3/4	10.00	5.625	3350	1820	2860	1820	2730	1320	2550	1430	2620	1340	2440	1110
	7/8	8.57	6.563	4560	2420	3900	2420	3720	1730	3470	1870	3560	1760	3320	1400
	1	7.50	7.500	5950	3090	5080	3030	4850	2060	4520	2230	4650	2100	4330	1570
	1-1/4	6.00	9.375	9310	4290	7950	3800	7580	2430	7070	2630	7260	2480	6770	1840
9-1/2	3/4	12.67	7.125	3350	1640	2860	1640	2730	1250	2550	1350	2620	1270	2440	1060
	7/8	10.86	8.313	4560	2270	3890	2270	3720	1660	3470	1790	3560	1690	3320	1410
	1	9.50	9.500	5950	2960	5080	2960	4850	2130	4530	2300	4650	2170	4330	1790
	1-1/4	7.60	11.875	9310	4510	7950	4450	7580	3030	7070	3280	7270	3090	6770	2330
	1-1/2	6.33	14.250	13420	6070	11470	5520	10930	3540	10200	3830	10470	3610	9760	2680
11-1/2	7/8	13.14	10.062	4560	2060	3900	2060	3700	1590	3460	1730	3570	1630	3330	1370
	1	11.50	11.500	5950	2770	5080	2770	4860	2040	4530	2210	4650	2080	4330	1730
	1-1/4	9.20	14.375	9310	4360	7960	4360	7590	3140	7080	3400	7270	3200	6780	2620
	1-1/2	7.67	17.250	13410	6210	11450	6140	10920	4210	10190	4550	10470	4280	9750	3240
13-1/2	1	13.50	13.500	5960	2530	5280	2530	4850	1970	4540	2140	4670	2020	4350	1680
	1-1/4	10.80	16.875	9300	4160	7950	4160	7590	3030	7080	3280	7260	3080	6770	2570
	1-1/2	9.00	20.250	13400	6040	11450	6040	10930	4340	10200	4700	10460	4420	9750	3600

Tabulated values, in pounds, are for one ASTM A307 bolt loaded in double shear in a three-member wood connection subjected to normal load duration and dry-use conditions. When high strength bolts are used (such as ASTM A325 bolts), values in this table can be used with slightly conservative results.

Use linear interpolation to determine design values for intermediate bolt lengths.

This table is limited to selected species and bolt lengths and is intended for illustrative purposes only. Refer to the current edition of the NDS for a more complete listing of tabulated values.

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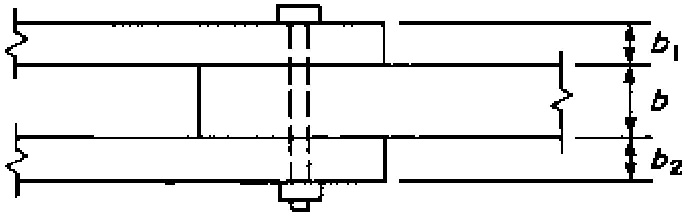
Table 5-21. - Summary of requirements for determining tabulated bolt values for lateral connections.

A. Three-member joints



Wood side members

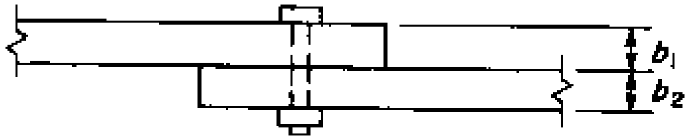
$b_1 = b_2 \geq b/2$ Use the tabulated value for main member thickness b.
$b_1 \leq b_2 < b/2$ Use the tabulated value lor main member twice the thickness ol b_1.
When side members are loaded at a different direction to the grain than the main member, the design value is the lesser of:

a. the tabulated value for the main member, or;
b. the tabulated value for a member twice the thickness of the side members and loaded in the same direction as the side members.

Steel side members

Use the tabulated value for the main (wood) member *b* for the direction of applied loading.

B. Two-member joints



Wood side member

$b_1 = b_2$ Use one-half the tabulated value for a main member of thickness b_1.
$b_1 < b_2$ Use the lesser of one-hall the tabulated value for main member of thickness = b_2 or $2b_1$.
When one member is loaded parallel-to-grain and the other is loaded at an angle to the grain, the design value is the lesser of:

a. one-hall the tabulated value for the thickness ol the parallel to grain loaded member, or;
b. the value obtained from application of the Hankinson formula (Equation 5-42) using one-hall the tabulated parallel-to-grain and perpendicular-to-grain values for a member the thickness of the member loaded at an angle to the grain.

Steel side member

Use one-half the tabulated value for a member the thickness of the wood member for the direction of applied loading.

connected to a steel plate, the design load is one-half of the tabulated value of the thickness of the wood member.

Steel Side Plates

When steel rather than wood side plates are used for lateral connections, the tabulated design values for members loaded parallel to grain only may be increased by the steel side plate factor, C_{st}, given below.

Sawn Lumber		Glulam	
Bolt diameter (in.)	C_{st}	Bolt diameter (in.)	C_{st}
≤1/2	1.75	All	1.25
3/4	1.63		
1	1.50		
1-1/4	1.38		
1-1/2	1.25		

Use linear interpolation to compute C_{st} for intermediate bolt diameters in sawn lumber. It should be noted that the values of C_{st} greater than 1.25 are currently being evaluated for sawn lumber and may be reduced in the future. In addition, AITC recommends that bolts used with steel side plates in glulam not exceed 1 inch in diameter.

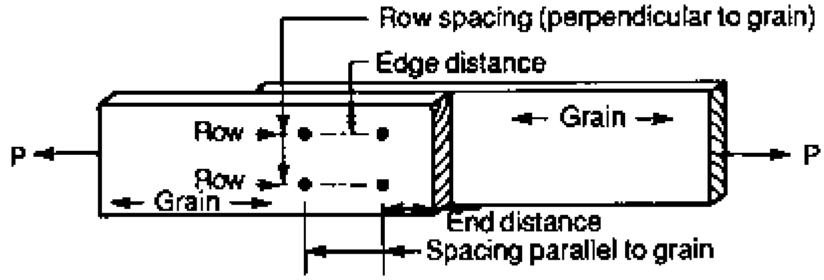
Distance and Spacing Requirements

Tabulated bolt values are based on minimum distance and spacing requirements necessary to develop the full capacity of the connection. These requirements differ for parallel-to-grain loading and perpendicular-to-grain loading and are summarized in Table 5-22. When bolts are placed at the minimum dimension for full tabulated value, no reduction in capacity is required. For end distance and spacing parallel to grain only, the dimensions may be reduced provided the tabulated value is reduced by the modification factors C_n or C_s. For example, when a bolted tension connection is loaded parallel to gram, the minimum end distance to develop the full tabulated value is 7 times the bolt diameter. This distance may be reduced to an absolute minimum of 3.5 times the bolt diameter provided the tabulated value is reduced by 50 percent (C_n = 0.50). When reduced dimensions are used for any bolt in a group, the factors C_n or C_s apply to all bolts in that group. Dimensions less than those given for reduced capacity are not permitted under any circumstances.

Distance and spacing requirements in the NDS are for loading parallel to grain and perpendicular to grain only. When loads act at an angle to the grain, bolt spacing and distance must be based on good engineering judgment. In this case, the gravity axis of the members should pass through the center of resistance of the bolt group to provide uniform stress in the main members and a uniform distribution of load to all bolts.

Table 5-22. - Summary of edge distance, end distance and spacing requirements for bolted connections.

A. Loading parallel to grain



	Minimum dimension for full tabulated value	Minimum dimension for reduced value[1]
Edge distance		
$l/D \leq 6$	$1.5D$	N/A
$l/D > 6$	$1.5D$ or 1/2 row spacing perpendicular to grain, whichever is greater	N/A
End distance		
Tension members	$7D$	$3.5D$ ($C_n = 0.50$)
Compression member	$4D$	$2D$ ($C_n = 0.50$)
Spacing		
Parallel to grain	$4D$	$3D$ ($C_n = 0.75$)
Row spacing perpendicular to grain	$1.5D$	N/A

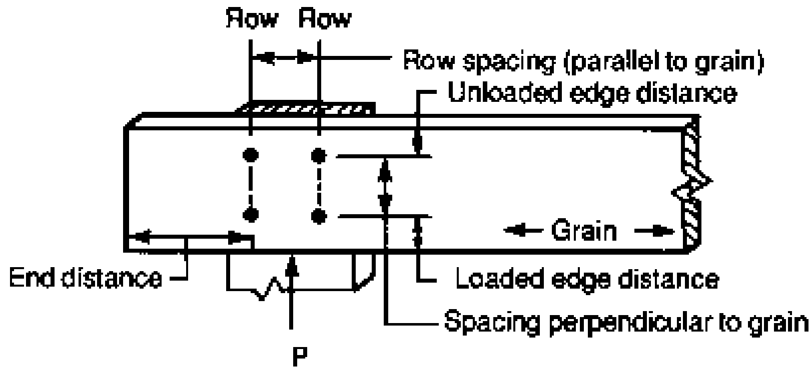
When steel members are used in connections, the spacing and distance requirements are based on the requirements for the timber components, not the steel components. As a practical consideration, the designer should always check to ensure that spacing requirements are sufficient to place washers without overlap.

Design of Tension Connections

In tension connections, the bolt is loaded in axial tension parallel to its axis. This type of connection is common in bridge applications when rail posts are bolted to curbs. The strength of a tension connection depends on the bearing strength of the wood and the tensile strength of the bolt (20,000 lb/in^2 for A307 bolts). The bearing stress under the washer must not exceed the allowable stress for compression perpendicular to grain ($F_{c\perp}'$). To compute bearing stress, the bolt load is divided by the total washer area minus the area of the bolt hole. Distance and spacing requirements for bolts loaded in tension only are not specified in the NDS and should be based on designer judgment.

Table 5-22. - Summary of edge distance, end distance and spacing requirements for bolted connections. (Continued)

B. Loading perpendicular to grain



	Minimum dimension for full tabulated value	Minimum dimension for reduced value[1]
Edge distance		
Loaded edge	$4D$	N/A
Unloaded edge	$1.5D$	N/A
End distance	$4D$	$2D (C_{\Delta} = 0.50)$
Spacing		
Row spacing parallel to grain[2,3]		
$\ell/D = 2$	$2.5D$	N/A
$\ell/D \geq 6$	$5D$	N/A
Perpendicular to grain	See note 4	See note 4

[1] For distances and spacings between the tabulated value and the reduced value use straight line interpolation to compute modification factor value.

[2] The spacing between rows of bolts shall not be more than 5 inches unless seperate splice plates are used for each row of bolts.

[3] For ℓ/D ratios between 2 and 6, spacing requirements are obtained by straight line interpolation.

[4] The spacing of bolts perpendicular to grain is limited by the spacing requirements of the attached member or members (whether of metal or of wood loaded parallel to grain).

All dimensions are measured from the center of the bolt hole.

Bolt Placement

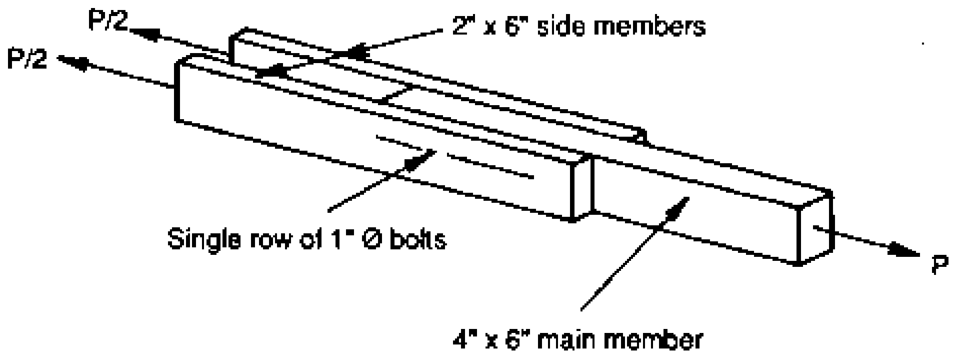
The strength of a laterally loaded, bolted connection can be significantly affected by the diameter of the hole and the manner in which it is bored. When holes are too large, bearing is nonuniform, and the capacity of the connection is reduced. If holes are too small, the bolt cannot be inserted without driving, which may split the wood members. The NDS specifies that bolt holes be a minimum of 1/32 inch to a maximum of 1/16 inch larger than the bolt diameter. In some cases, it may be necessary to slightly enlarge the hole diameter slightly to compensate for galvanized coatings on large fasteners.

When bolts are installed in wood members, washers of the proper size or a steel plate or strap are required under all nuts and under square or hexagonal bolt heads. Nuts must be tightened so that member surfaces are brought into close contact without crushing the wood. Tabulated design values for bolts include an allowance for the loosening of nuts because of member shrinkage. However, when bolts are installed in unseasoned wood it is advisable to retighten connections at least every 6 months until the wood reaches equilibrium moisture content. Self-locking nuts are frequently used for decks and other components that may have a tendency to loosen because of vibrations from moving loads.

Example 5-11 - Lateral bolted connection parallel to grain

A tension splice in a timber truss joins two 2-inch by 6-inch side members to a 4-inch by 6-inch main member. Design the connection to develop the full capacity of the members, assuming the following:

1. Members will be exposed to weathering and carry loads of normal duration; adjustments for temperature (C_t) and fire-retardant treatment (C_R) are not required.
2. The connection is made with a single row of 1-inch-diameter bolts.
3. Lumber is dressed Southern Pine, visually graded No. 1 to SPIB rules.



Solution

This connection involves a three-member configuration loaded in double shear. The design procedure will be to (1) compute the capacity of the lumber members, (2) determine the required number of bolts, and (3) detail the connection for minimum distance and spacing requirements.

Member Capacity

The tabulated stress for No. 1 Southern Pine in tension parallel to grain is obtained from NDS Table 4A. The NDS includes several tables for Southern Pine, and a value F_t = 775 lb/in^2 is selected from the table "surfaced green; used any condition" (footnotes to Table 4A specify use of this table when the moisture content in service is expected to exceed 19 percent). Further adjustment for moisture content is not required.

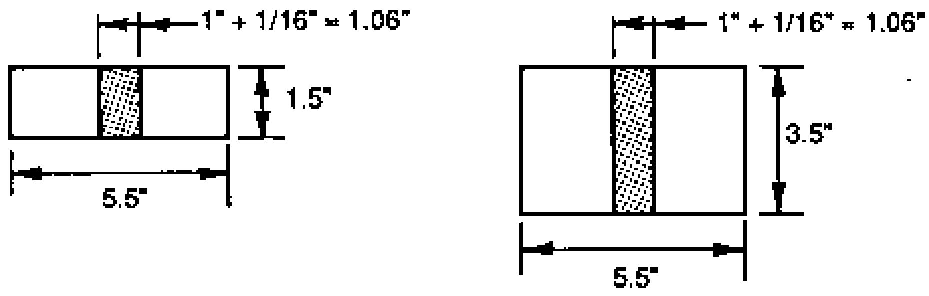
The allowable stress in tension parallel to grain is computed using Equation 5-23:

$$F_t' = F_t C_M = 775(1.0) = 775 \text{ lb/in}^2$$

The capacity of the connection depends on the net area of the lumber members. Gross section properties for nominal 2-inch by 6-inch and 4-inch by 6-inch lumber are obtained from Table 16-2.

For nominal 2-inch by 6-inch lumber	For nominal 4-inch by 6-inch lumber
b = 1.5 in.	b = 3.5 in.
d = 5.5 in.	d = 5.5 in.
A = 8.25 in^2	A = 19.25 in^2

Assuming that bolt holes are 1/16 inch larger than the bolt diameter, the net area of each member is equal to the gross area minus the projected area of the bolt holes:



For two 2-inch by 6-inch members,

$$A_{NET} = 2\ [8.25\ in^2 - (1.06\ in.)(1.5\ in.)] = 13.32\ in^2$$

For a single 4-inch by 6-inch member,

$$A_{NET} = 19.25\ in^2 - [(1.06\ in.)(\ 3.5\ in.)] = 15.54\ in^2$$

Connection capacity will be limited by the smaller area of the two 2-inch by 6-inch members. The maximum connection load in tension, P_T, is equal to the net area times the allowable stress in tension parallel to grain:

$$P_T = A_{NET}(F_t') = 13.32(775) = 10{,}323\ lb$$

Number of Bolts

The next step is to determine the number of 1-inch-diameter bolts that are required to transfer the lateral load of 10,323 pounds. Because this is a three-member connection, tabulated bolt values can be read directly from the NDS bolt design tables (Table 5-20); however, the length of bolt in the main member, ℓ, must first be determined. In this case, the thickness of the side members (1.5 inches) is less than half the thickness of the main member (1.75 inches). From Table 5-21, the main member thickness used to determine the tabulated bolt value is equal to twice the thickness of the thinner side members:

$$\ell = 2(1.5\ in.) = 3\ in.$$

From Table 5-20 for a 1-inch-diameter bolt, Species Group 3, and a bolt length in main member of 3 inches,

$$P = 3{,}750\ lb$$

Assuming that adequate distance and spacing requirements can be met, the allowable load for one bolt loaded parallel to grain is given by Equation 5-43:

$$P' = P C_M C_g$$

From Table 5-18 for a bolted connection that is exposed to weathering:

$$C_M = 0.75$$

This connection will involve more than 2 bolts in a row and adjustment by the group action factor, C_g will be required. To determine C_g from Table 5-19, the number of bolts must be known. At this point, an estimate of the number of bolts is made by assuming adjustment by C_M only:

$$\text{Estimated number of bolts} = \frac{P_T}{P(C_M)} = \frac{10{,}323}{3{,}750\,(0.75)} = 3.7 \text{ bolts}$$

C_g will be determined for a row of 4 bolts. From Table 5-19 for connections with wood side plates:

A_1 = 19.25 in^2

A_2 = (2)(8.25 in^2) = 16.50 in^2

A_1/A_2 = 19.25/16.50 = 1.17 > 1.0, so use A_2/A_1

A_2/A_1 = 16.5/19.25 = 0.86

The value A_2/A_1 is between the values 0.5 and 1.0 given in Table 5-19. Because $A_1/A_2 > 1.0$, A_2 is used instead of A_1. For A_2 = 16.5 and 4 bolts, linear interpolation between C_g = 0.88 for A_2/A_1 = 0.50 and C_g = 0.94 for A_2/A_1 = 1.0 gives a value C_g = 0.92. Using this factor, the allowable bolt load is computed by Equation 5-43:

$$P' = PC_MC_g = 3{,}750(0.75)(0.92) = 2{,}588 \text{ lb/bolt}$$

The required number of bolts is computed by dividing the maximum load by the allowable load per bolt:

$$\text{Required number of bolts} = \frac{P_T}{P'} = \frac{10{,}323}{2{,}588} = 3.99 \text{ or } 4 \text{ bolts}$$

Distance and Spacing Requirements

From Table 5-22, distance and spacing requirements for full connection capacity with loading parallel to grain are as follows:

Edge distance for $\ell/D \le 6 = 1.5D = 1.5$ in.

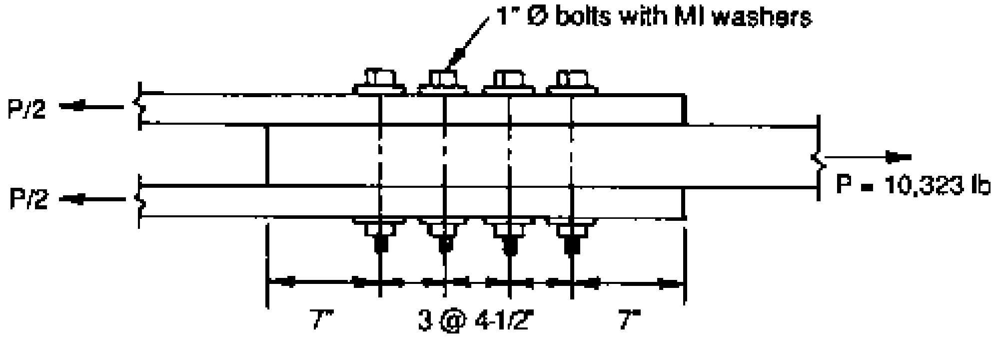
End distance for tension members = $7D$ = 7 in.

Bolt spacing parallel to grain = $4D$ = 4 in.

All distance and spacing requirements for full load can be met; however, washer size should be checked to avoid potential overlapping. In most cases, malleable iron (MI) washers of the sizes given in Table 16-7 are used. For a 1-inch-diameter MI washer the outside washer diameter is 4 inches, which is the same distance required for bolt spacing parallel to grain. Spacing will be increased to 4-1/2 inches to allow for construction tolerances and washer placement.

Summary

The connection will be made with four 1-inch-diameter bolts to develop the member capacity of 10,323 pounds. Detailing is as follows:



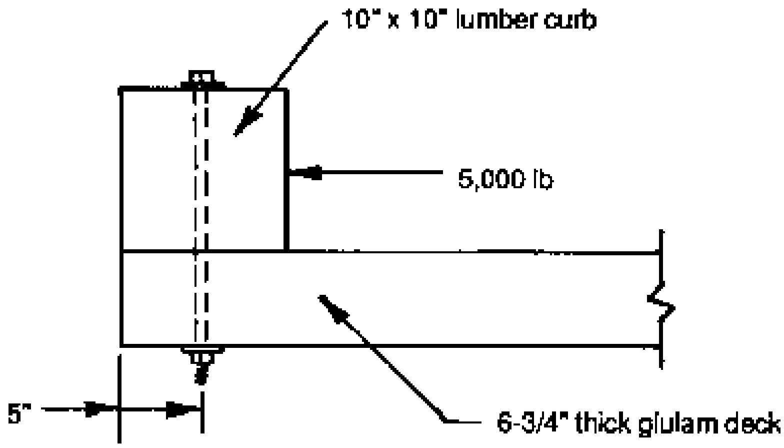
Example 5-12 - Lateral bolted connection perpendicular to grain

A 10-inch by 10-inch lumber curb is bolted along the edges of a 6-3/4-inch-thick transverse glulam deck. A transverse 5,000 pound load with a duration of load of 5 minutes is applied at the curb center. Determine the number of 7/8-inch-diameter bolts that are required to transfer the curb load to the deck, assuming the following:

1. Members will be exposed to weathering (wet-use conditions for glulam); adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. The glulam deck is combination symbol No. 2.

3. The curb is full-sawn Douglas Fir-Larch, visually graded No. 1 to WWPA rules.



Solution

In this connection the curb is loaded perpendicular to grain while the deck is loaded parallel to grain. From Table 5-21, for a two-member connection with members loaded at different angles to the grain, the tabulated design value for one bolt is the lesser of the following:

1. one-half the tabulated parallel-to-grain value, *P*, for the thickness of the member loaded parallel to grain; or

2. one-half the tabulated perpendicular-to-grain value, *Q*, for the thickness of the member loaded perpendicular to grain (application of the Hankinson formula as stated in Table 5-21 is not necessary in this case because loading is perpendicular to grain rather than at some intermediate angle between 0 and 90 degrees).

Tabulated bolt values for loading perpendicular to grain are normally much lower than those for loading parallel to grain. Thus, the design sequence will be to (1) determine the number of bolts required for curb loading perpendicular to grain, (2) check the connection for deck loading parallel to grain, and (3) verify and detail distance and spacing requirements.

Curb Loading Perpendicular to Grain

The allowable design value for one bolt loaded perpendicular to grain is given by Equation 5-44. Assuming minimum distance and spacing requirements can be met, and substituting *Q/2* for *Q* in this single-shear application, Equation 5-44 reduces to

$$Q' = \frac{Q}{2} C_D C_M$$

Using bolt design tables in the NDS (Table 5-20), the tabulated perpendicular to grain value, *Q*, is determined for one 7/8-inch-diameter bolt in Douglas Fir-Larch (Species Group 3), with a length of bolt in main member of 10 inches (ℓ = 10 inches). Table 5-20 does not include ℓ = 10 inches, so interpolation is required.

For ℓ = 9-1/2 in., Q = 2,270 lb

For ℓ = 11-1/2 in., Q = 2,060 lb

By linear interpolation, for ℓ = 10 in., Q = 2,218 lb

The duration of load factor for the 5-minute load duration is obtained from Table 5-8:

$C_D = 1.65$

The moisture-content factor for bolted sawn lumber exposed to weathering is obtained from Table 5-18:

$C_M = 0.75$

Substituting values into Equation 5-44, the allowable perpendicular-to-grain load for one 7/8-inch-diameter bolt is computed:

$$Q' = \frac{Q}{2} C_D C_M = \frac{(2,218)}{2} (1.65)(0.75) = 1,372 \text{ lb}$$

The required number of bolts is computed by dividing the applied load by the allowable load per bolt:

$$\text{Required number of bolts} = \frac{5,000 \text{ lb}}{1,372 \text{ lb/bolt}} = 3.64 = 4 \text{ bolts}$$

Deck Loading Parallel to Grain

The allowable design value for one bolt loaded parallel to grain is given by Equation 5-43. Assuming that minimum distance and spacing requirements can be met, and substituting *P/2* for *P* in this single-shear application,

$$P' = \frac{P}{2} C_D C_M$$

From Table 5-15, glulam combination symbol No. 2 is in Species Group 3 for bolt design. As with curb loading, tabulated values in Table 5-20 do not include a bolt length in main member that matches the required length of ℓ = 6-3/4 inches. However, values for ℓ = 5-1/2 inches and ℓ = 7-1/2 inches are both 3,900 pounds, so the same value also applies to ℓ = 6-3/4 inches:

$P = 3,900$ lb

From Table 5-18 for glulam used under wet-use conditions,

$C_M = 0.67$

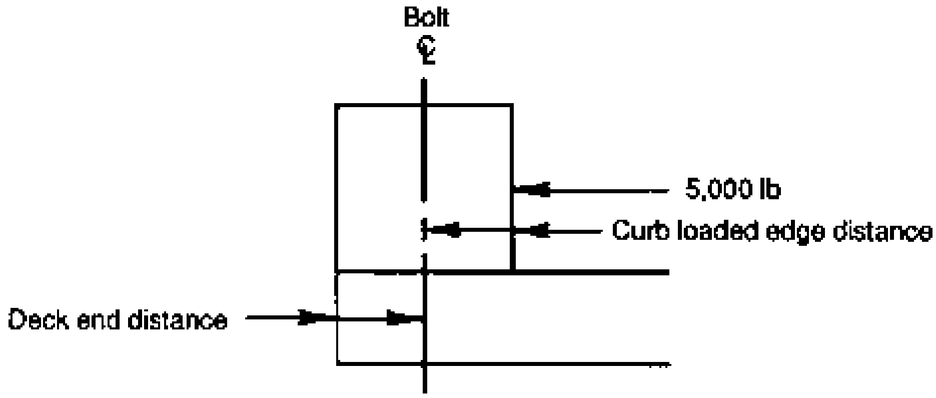
Substituting into Equation 5-43,

$$P' = \frac{P}{2} C_D C_M = \frac{3,900}{2} (1.65)(0.67) = 2,156 \text{ lb}$$

P' = 2,156 lb > Q' = 1,372 lb, so curb loading perpendicular to grain will control design.

Distance and Spacing Requirements

The two most critical distances in this connection are the curb loaded edge distance and the deck end distance:



From Table 5-22, the minimum loaded edge distance for loading perpendicular to grain is four times the bolt diameter,

$$4D = 4(0.875 \text{ in.}) = 3.5 \text{ in.}$$

The actual loaded edge distance of 5 inches exceeds the minimum 3.5 inches, and is sufficient.

For loading parallel to grain, the minimum end distance for full capacity on the glulam deck is seven times the bolt diameter,

$$7D = 7(0.875) = 6.13 \text{ in.}$$

This value is greater than the 5 inches provided. The end distance can be reduced to a minimum value of $3.5D$ = 3.06 inches, provided the allowable load is reduced by 50 percent (C_n = 0.50). By linear interpolation for the actual end distance of 5 inches, C_n = 0.82, and the allowable load is revised as follows:

$$P' = \frac{P}{2} C_p C_M C_n = \frac{3{,}900}{2}(1.65)(0.67)(0.82) = 1{,}768 \text{ lb}$$

The revised value is still greater than Q' = 1,372 pounds, so a reduced end distance of 5 inches will not affect connection capacity.

From Table 5-22, the spacing of bolts parallel to grain on the curb is controlled by $5D$ = 4.38 inches (based on ℓ/d = 10/.875 = 11.4). From Table 16-7, the outside diameter of a 7/8-inch MI washer is 3.5 inches. Bolts will be spaced 4-1/2 inches apart to meet spacing requirements and allow for construction tolerance.

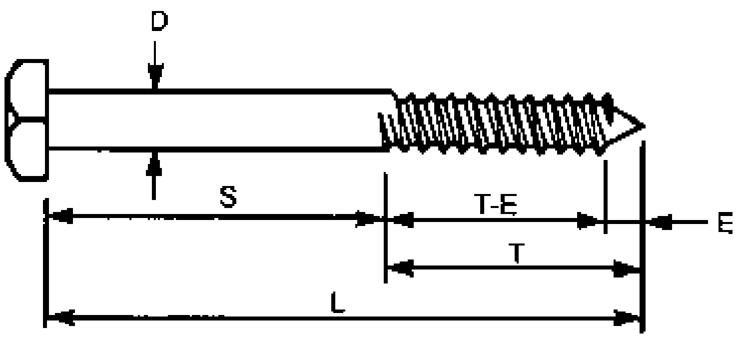
Summary

The connection will be made using four 7/8-inch-diameter bolts for a total capacity of 4(1,372) = 5,488 pounds. The bolts will be spaced 4-1/2 inches on-center and will be provided with malleable iron washers on each end.

LAG SCREWS

Lag screws are used in bridge applications for two-member connections loaded laterally in single shear (two members) or in withdrawal. The strength of a lag screw is less than that of a comparable bolt, but lag screws offer the advantage of being placed from one side of the connection. They are used primarily for convenience or when through bolts are undesirable or impractical. This occurs in connections where access for nut placement is restricted or when an excessively long bolt is required to fully penetrate the connection. Lag screws also may be used instead of spikes in nonstructural applications (such as timber wearing surface attachment) because they are less susceptible to loosening from vibrations and from dimensional changes in the wood.

Lag screws are manufactured of the same material as bolts, conforming to *ASTM Standard A307, Low-Carbon Steel Externally and Internally Threaded Standard Fasteners.* They have a square or hexagonal bolt head and require a washer when the screw head is in wood contact. A diagram of a typical lag screw is shown in Figure 5-22. The specified diameter of the screw corresponds to the diameter of the unthreaded shank portion. Nominal length is the distance from the base of the head to the tip of the threads. Lag screws are commonly available in stock diameters of 3/16 inch to 1-1/4 inch and nominal lengths up to 16 inches, in 1/2-inch increments. The length of the threaded portion varies with the length of the screw. Dimensions of common lag screws are given in Table 16-5.



D = Nominal diameter or shank diameter
L = Nominal length
S = Length of shank
T = Length of thread
E = Length of tapered tip

Figure 5-22. - Lag screw configuration and nomenclature.

Net Area

Net area is computed for lag screws in the same manner as bolts with the same diameter as the shank diameter of the lag screw.

Design of Lateral Connections

The strength of a laterally loaded, lag screw connection is developed by bearing between the screw and the members, and the interaction of the threads in the main member (Figure 5-23). In bridge applications, these connections should be limited to applications where the screw is inserted into the side grain of the member, perpendicular to the wood fiber direction. Refer to the NDS for design criteria when end-grain connections cannot be avoided.

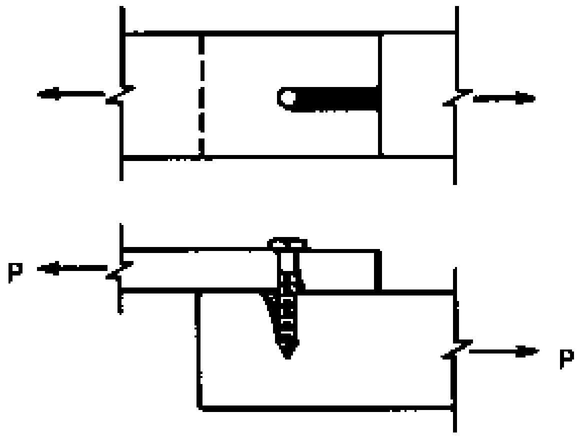


Figure 5-23. - Typical configuration and stress distribution for a laterally loaded lag screw connection.

The allowable value for one laterally loaded lag screw is equal to the tabulated value, adjusted by all applicable modification factors. When more than one lag screw is used, the allowable value for the connection is the sum of the allowable values for the individual fasteners, including adjustment by the group action factor, C_g. Equations 5-45 and 5-46 follow:

$$P' = PC_D C_M C_t C_R C_n C_s C_g \tag{5-45}$$

$$Q' = QC_D C_M C_t C_R C_n C_s C_g \tag{5-46}$$

If loads act at an angle to the grain, the allowable design value is computed using the Hankinson formula.

Tabulated Design Values

Tabulated values are specified in the NDS for one A307 lag screw loaded in single shear in a two-member joint. Unlike other fasteners, separate tables are included for connections with wood side pieces and connections with metal side pieces. Portions of the NDS tables for a limited number of lag screw lengths are given in Tables 5-23 and 5-24. For connections with wood side members, tabulated values are based on the thickness of the side members and the nominal length and diameter of the lag screw (Table 5-23). When side members are 1-1/2 or 2-1/2 inches thick, values are read directly from the table. The NDS does not include tabulated values for other side member thicknesses, but additional tabulated values for other side member thicknesses are given in the AITC *Timber Construction Manual.* [24]

Table 5-23.—Tabulated design values for laterally loaded lag screws with wood side plates.

Thickness of side member (inches)	Length of lag screw (inches)	Diameter of lag screw shank (inches)	Species Group							
			GROUP I		GROUP II		GROUP III		GROUP IV	
			Total lateral load per lag screw in single shear (pounds)		Total lateral load per lag screw in single shear (pounds)		Total lateral load per lag screw in single shear (pounds)		Total lateral load per lag screw in single shear (pounds)	
			Parallel to grain	Perpen-dicular to grain	Parallel to grain	Perpen-dicular to grain	Parallel to grain	Perpen-dicular to grain	Parallel to grain	Perpen-dicular to grain
1-1/2	4	1/4	200	200	170	170	130	130	100	100
		5/16	290	240	220	180	150	130	120	110
		3/8	330	250	250	190	180	140	140	110
		7/16	370	260	280	190	200	140	160	110
		1/2	390	250	290	190	210	140	170	110
		5/8	470	260	360	210	260	160	200	120
	6	1/4	270	260	230	220	210	200	180	180
		5/16	380	320	330	280	290	250	260	220
		3/8	490	370	420	320	370	280	300	230
		7/16	600	420	520	360	410	280	330	230
		1/2	700	460	600	390	430	280	340	220
		5/8	850	510	710	430	510	310	410	250
2-1/2	6	3/8	450	340	380	290	270	210	220	170
		7/16	590	410	440	310	320	220	250	180
		1/2	620	410	470	310	340	220	270	180
		5/8	730	440	550	330	390	240	320	190
		3/4	830	460	630	350	450	250	360	200
		7/8	950	490	720	370	510	270	410	210
		1	1060	530	800	400	570	290	460	230
	8	3/8	560	420	480	370	430	330	380	290
		7/16	730	510	630	440	560	390	450	320
		1/2	890	580	770	500	600	390	480	310
		5/8	1230	740	970	580	700	420	560	340
		3/4	1440	790	1090	600	780	430	630	340
		7/8	1610	840	1220	630	870	450	700	360
		1	1810	910	1370	690	980	490	790	390

Tabulated values are for normal load durations under dry service conditions.

This table contains a limited number of lag screw lengths and is intended for illustrative purposes only. Refer to the current edition of the NDS for a more complete listing of design values.

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Table 5-24. - Tabulated design values for laterally loaded lag screws with metal side plates up to 1/2-inch thick.

Length of lag screw (inches)	Diameter of lag screw shank (inches)	Species Group: GROUP I Total lateral load per lag screw in single shear (pounds)		GROUP II Total lateral load per lag screw in single shear (pounds)		GROUP III Total lateral load per lag screw in single shear (pounds)		GROUP IV Total lateral load per lag screw in single shear (pounds)	
		Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain	Parallel to grain	Perpendicular to grain
4	1/4*	270	210	240	180	210	160	190	150
	5/16	410	280	350	240	290	200	230	160
	3/8	570	350	480	290	340	210	280	170
	7/16	730	410	550	310	390	220	310	180
	1/2	810	420	610	320	440	230	350	180
	5/8	980	470	740	360	530	250	430	200
6	5/16*	450	300	390	260	340	230	300	210
	3/8	630	390	550	330	490	300	430	260
	7/16	850	480	730	410	660	370	540	300
	1/2	1100	570	950	490	760	400	610	320
	5/8	1640	790	1290	620	920	440	740	350
	3/4	1990	870	1500	660	1070	470	860	380
8	7/16*	880	490	760	420	680	380	600	330
	1/2	1140	590	980	510	880	460	780	400
	5/8	1750	840	1510	720	1320	630	1060	510
	3/4	2470	1090	2130	940	1560	690	1250	550
	7/8	3260	1360	2480	1030	1770	470	1420	590
10	5/8*	1790	860	1550	740	1380	660	1220	590
	3/4	2550	1120	2200	970	1970	870	1630	720
	7/8	3430	1420	2960	1230	2340	970	1880	780
	1	4410	1770	3680	1470	2640	1050	2110	850
12	7/8	3490	1450	3020	1260	2700	1120	2320	960
	1	4520	1810	3900	1560	3260	1310	2620	1050
	1-1/8	5670	2270	4890	1960	3630	1450	2910	1170

Tabulated values are for normal load durations under dry service conditions.

The asterisk (*) indicates that greater lengths for the lag screw diameter do not provide higher loads.

This table contains a limited number of lag screw lengths and is intended for illustrative purposes only. Refer to the current edition of the NDS for a more complete listing of design values.

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For lag screw connections with metal side plates, tabulated values in the NDS are based on the nominal length and diameter of the lag screw (Table 5-24). Values for side plates up to 1/2 inch thick are read directly from the table. When side plates are thicker than 1/2 inch, tabulated values must be reduced in proportion to the reduced lag screw penetration, by linear interpolation of table values. Values in Table 5-24 have been adjusted by C_{st}, and further adjustment by this factor is not required or permitted.

Distance and Spacing Requirements

End distance, edge distance, and spacing requirements for lag screws are the same as those for bolts of a diameter equal to the shank diameter of the lag screw (Table 5-22). Bolt modification factors for reduced distance and spacing also apply to lag screws.

Design of Withdrawal Connections

In withdrawal connections, lag screws develop their strength by the interaction of the threads with the wood. The capacity of the connection depends on the specific gravity of the wood and the length of penetration of the lag screw. As shown in Equation 5-47, the allowable value for one lag screw in axial withdrawal is equal to the tabulated value in withdrawal, P_w adjusted by all applicable modification factors:

$$P_w' = P_w C_D C_M C_t C_R \tag{5-47}$$

When more than one lag screw is used, the value for one screw is multiplied by the total number of screws in the connection.

In determining allowable withdrawal values, the washer bearing stress on wood members must be less than the allowable stress in compression perpendicular to grain $F_{c\perp}'$, as discussed for bolts. In addition, the allowable tensile strength of the lag screw at the net (root) section must not be exceeded. The strength of A307 lag screws in axial tension is developed when the penetration depth of the threaded portion is approximately 7 diameters for Group I species, 8 diameters for Group II species, 10 diameters for Group III species, and 11 diameters for Group IV species. When the penetration of the screw exceeds these values, connection strength is generally controlled by the tensile strength of the fastener.

Tabulated Design Values

Tabulated withdrawal values for lag screws are given in the NDS for one A307 lag screw loaded in withdrawal from side grain. A portion of the NDS table for a limited number of specific gravities is shown in Table 5-25. To determine the tabulated value for one lag screw, enter the table with the specific gravity of the member and read the value in pounds per inch of penetration given for the screw diameter. The tabulated value for one screw is computed by multiplying this value times the distance of

Table 5-25. - Tabulated design values for lag screws loaded in withdrawal.

Specific gravity G	Lag Screw Shank Diameter (in.)											
	1/4 0.250	5/16 0.3125	3/8 0.375	7/16 0.4375	1/2 0.500	9/16 0.5625	5/8 0.625	3/4 0.750	7/8 0.875	1 1.000	1-1/8 1.125	1-1/4 1.250
0.55	260	307	352	395	437	477	516	592	664	734	802	868
0.54	253	299	342	384	425	464	502	576	646	714	780	844
0.51	232	274	314	353	390	426	461	528	593	656	716	775
0.49	218	258	296	332	367	401	434	498	559	617	674	730
0.48	212	250	287	322	356	389	421	482	542	599	654	708
0.47	205	242	278	312	345	377	408	467	525	580	634	686
0.46	199	235	269	302	334	365	395	453	508	562	613	664
0.45	192	227	260	292	323	353	382	438	492	543	594	642
0.44	186	220	252	283	312	341	369	423	475	525	574	621
0.43	179	212	243	273	302	330	357	409	459	508	554	600
0.42	173	205	235	264	291	318	344	395	443	490	535	579

Tabulated values are for load in withdrawal in pounds per inch of pentration of the threaded portion of the screw into the side grain of the member holding the point; normal load duration under dry service conditions.

This table is limited to selected values for specific gravity and is intended for illustrative purposes only. Refer to the current edition of the NDS for a more complete listing of design values.

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thread penetration into the member. When determining thread penetration, the screw tip length is not included as a portion of the threads. Refer to Table 16-5 for lag screw thread and tip lengths.

Lag Screw Placement

Lag screws are installed in prebored lead holes of sufficient diameter and length to develop thread strength and prevent the wood from splitting as the screw is installed. This requires that holes be drilled in two diameters, one for the shank and one for the threads (Figure 5-24). The lead hole for the shank is 1/16 inch larger than the shank diameter and is bored to the depth of penetration of the shank. The lead hole diameter for the threaded portion, which is bored at least the length of the threads, is based on the species of the member receiving the point. The NDS requires that for the threaded portion, the lead hole be 65 to 85 percent of the shank diameter in Group I species, 60 to 75 percent in Group II species, and 40 to 70 percent in Group III and IV species (the larger percentile figure in each range applies to screws of greater diameters). Recommended prebore diameters for lag screws are given in Table 5-26. The effect of prebore diameter on the lag screw thread penetration is illustrated in Figure 5-25.

Lag screws must be provided with a washer of the proper size unless the head of the screw bears on steel. When installing lag screws, the threaded portion is inserted in the lead hole by turning with a wrench, not by driv-

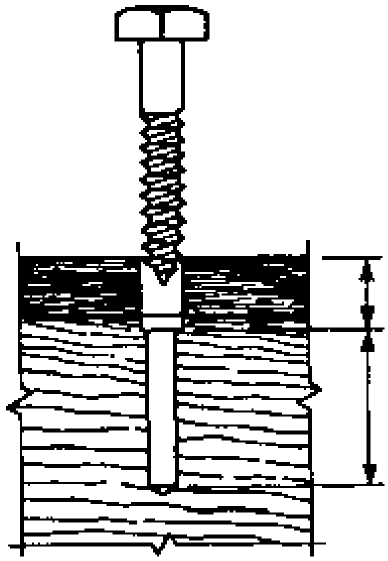

Figure 5-24.- Lead holes for lag screws are prebored in two diameters; one diameter for the shank and a smaller diameter for the threads.

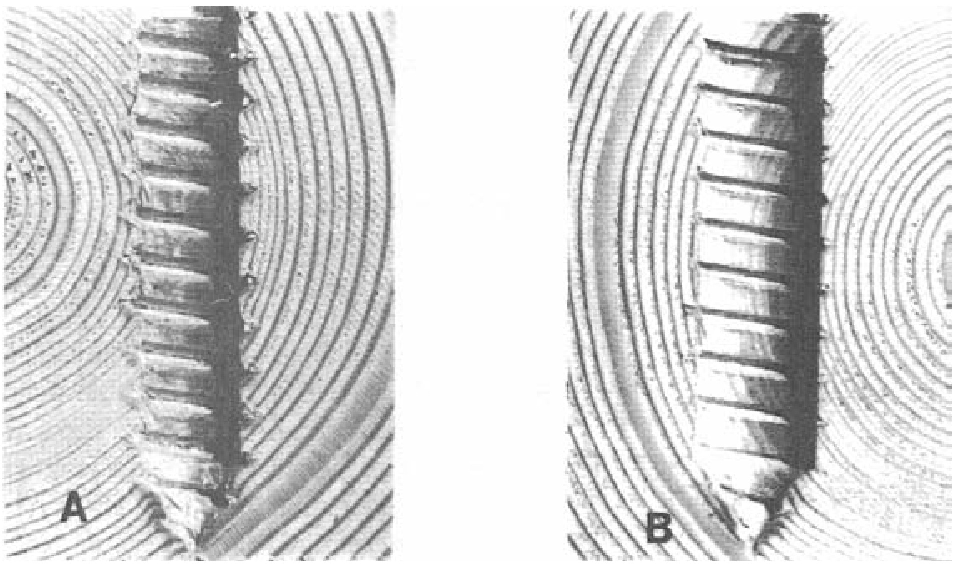

Figure 5-25. - (A) Clean-cut, deep penetration of thread made by a lag screw turned into a lead hole of proper size. (B) Shallow penetration of thread made by a lag screw turned into an oversized lead hole.

ing with a hammer. If screws are difficult to insert, soap or other lubricants can be placed on the screw to facilitate placement. In timber treated with an oil-type preservative, the preservative facilitates placement, and additional lubricants are normally not required.

Table 5-26. - Recommended lead hole diameters for lag screws.

Nominal diameter of lag screw (in.)	Diameter of lead hole (in.)			
	Shank (unthreaded) portion (in.)	Threaded portion		
		Group I species	Group II species	Groups III and IV species[1]
1/4	5/16	3/16	5/32	3/32
5/16	3/8	13/64	3/16	9/64
3/8	7/16	1/4	15/64	11/64
7/16	1/2	19/64	9/32	13/64
1/2	9/16	11/32	5/16	15/64
9/16	5/8	13/32	23/64	9/32
5/8	11/16	29/64	13/32	5/16
3/4	13/16	9/16	1/2	13/32
7/8	15/16	43/64	39/64	33/64
1	1-1/16	51/64	23/32	5/8
1-1/8	1-3/16	59/64	53/64	3/4
1-1/4	1- 5/16	1-1/16	15/16	7/8

[1] When loaded primarily in withdrawal, lag screws of 3/8-inch diameter or less may be inserted into group III and IV species without a lead hole provided that spacings, end distances and edge distances are sufficient to prevent unusual splitting.

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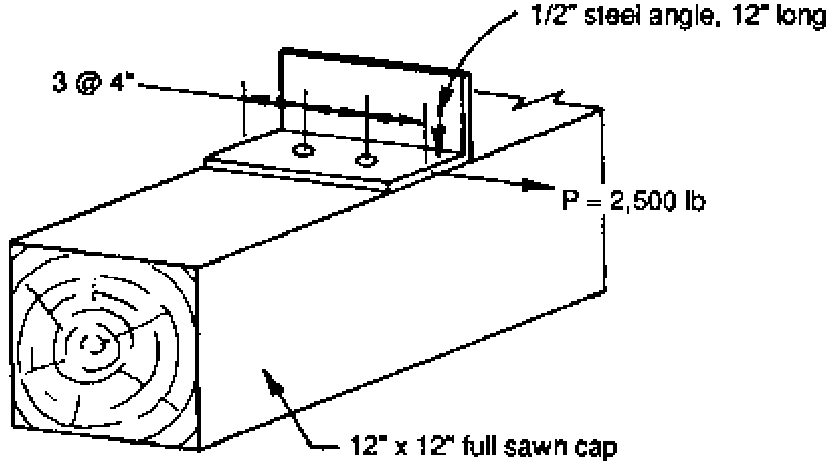
Example 5-13 - Lateral lag screw connection with steel side plates

A beam bearing shoe consists of a pair of 1/2-inch-thick steel angles that are 12 inches long. Each angle is connected to a full-sawn 12-inch by 12-inch pile cap with two lag screws placed at the angle third points. Determine the required diameter and length of lag screws to resist a longitudinal beam load of 2,500 pounds per angle, assuming the following:

1. There is a 2-month duration of load (C_D= 1.15).

2. Members will be exposed to weathering; adjustments for temperature (C_t) and fire-retardant treatment (C_R) are not required.

3. The pile cap is Douglas Fir-Larch, visually graded No. 1 to WWPA rules.



Solution

The allowable load perpendicular to grain for one lag screw, Q', is given by Equation 5-46. Assuming that distance and spacing requirements can be met, Equation 5-46 for this case reduces to

$$Q' = QC_DC_M$$

To facilitate selection of a lag screw from NDS tables, the above equation is rearranged so that the required tabulated lag screw value, Q, is computed directly:

$$Q = \frac{Q'}{C_DC_M}$$

The applied load of 2,500 pounds is resisted by two lag screws. Therefore, the minimum allowable load for one lag screw is one-half the applied load:

$$Q' = \frac{2,500}{2} = 1,250 \text{ lb}$$

From Table 5-18 for lag screws installed in sawn lumber exposed to weathering:

$$C_M = 0.75$$

Substituting values and solving for the required tabulated value perpendicular to grain:

$$Q = \frac{1,250}{1.15\,(0.75)} = 1,449 \text{ lb}$$

Before entering design tables, limitations on lag screw length and diameter must be checked. For the 12-inch pile cap depth, lag screw length will be limited to 12-inches. Limitations on lag screw diameter are checked against distance and spacing requirements given in Table 5-22. For loading perpendicular to grain, the minimum loaded edge distance is four times the lag screw diameter. For the 4-inch distance provided by the angle configuration, requirements for loaded edge distance cannot be met if the lag screw diameter exceeds 1-inch. Thus, design requirements for lag screw selection are as follows:

Tabulated value for loading perpendicular to grain = $Q \geq 1{,}449$ **lb**

Lag screw diameter = $D \leq 1$ **in.**

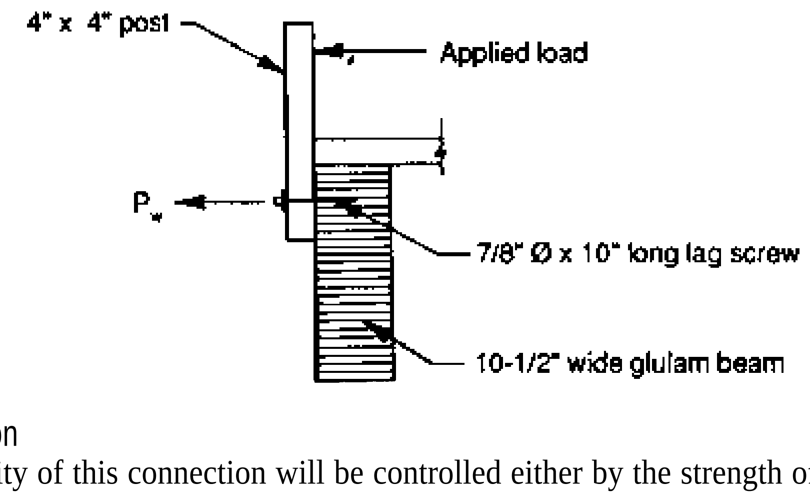
Lag screw length = $L \leq 12$ **in.**

From Table 5-14, No. 1 Douglas Fir-Larch is in Species Group II for lag screw design. Entering Table 5-24 for laterally loaded lag screws with metal side plates, two possible lag screw sizes meet design requirements; a 10-inch-long by 1-inch-diameter lag screw with Q = 1,470 pounds, or a 12-inch-long by 1-inch-diameter lag screw with Q = 1,560 pounds. In this case the 10-inch-long lag screw is selected, but either screw is feasible depending on availability and relative economics.

Example 5-14 - Lag screw loaded in withdrawal

A 4-inch by 4-inch lumber railpost is attached to the side of a 10-1/2-inch-wide glulam beam with a 7/8-inch diameter by 10-inch-long lag screw with malleable iron washer. Determine the capacity of the connection in withdrawal, assuming the following:

1. Members will be exposed to weathering (wet-use conditions) and carry loads of normal duration; adjustments for temperature (C_t) and fire-retardant treatment (C_R) are not required.
2. The railpost is rough-sawn Southern Pine, visually graded No. 1 to SPIB rules.
3. The glulam beam is combination symbol 24F-V5 Southern Pine.



Solution

Capacity of this connection will be controlled either by the strength of the lag screw in withdrawal, or by bearing stress under the washer.

Lag Screw in Withdrawal

Using applicable modification factors from Equation 5-47, the allowable lag screw load in withdrawal is given as follows:

$$P_W' = P_W C_M$$

The strength in withdrawal depends on the length of penetration of the threaded portion minus tip length *(T - E)* into the member receiving the point. From Table 16-5, dimensions for a 7/8-inch-diameter by 10-inch-long lag screw are as follows:

S = Length of shank = 4.75 in.

T = Length of thread = 5.25 in.

T - *E* = Length of thread minus length of tip = 4.75 in.

From Table 5-16, the specific gravity of the side face of a 24F-V5 glulam beam is 0.55. Entering Table 5-25 with a specific gravity of 0.55, and a lag screw shank diameter of 7/8 inch,

P_W per inch of penetration = $P_W/in.$ = 664 lb

P_W is obtained by multiplying $P_W/in.$ by the thread penetration, *T* - *E*,

$$P_W = P_W/\text{in.}\ (T - E) = 664\ (4.75) = 3{,}154\ \text{lb}$$

From Table 5-18, C_M = 0.67 for lag screws in glulam under wet-use conditions, and

$$P'_W = P_W C_M = 3{,}154\ (0.67) = 2{,}113\ \text{lb}$$

Check Washer Bearing Stress

From NDS Table 4A for No. 1 Southern Pine, surfaced green, used any condition:

$$F_{c\perp} = 375\ \text{lb/in}^2$$

Further adjustment for moisture content in excess of 19 percent is not required, and

$$F_{c\perp}' = F_{c\perp} C_M = 375(1.0) = 375\ \text{lb/in}^2$$

From malleable iron washer sizes in Table 16-7, the bearing area of a 7/8-inch-diameter washer is computed by subtracting the hole area from the total washer area:

$$A = A_{TOTAL} - A_{HOLE} = \pi(1.75)^2 - \pi(0.5)^2 = 9.62 - 0.79 = 8.83\ \text{in}^2$$

The allowable bearing load is equal to the bearing area times the allowable stress in compression perpendicular to grain:

$$\text{Bearing capacity} = AF_{c\perp}' = 8.83(375) = 3{,}311 \text{ lb}$$

Summary

The allowable capacity of the connection is limited by the withdrawal strength of the lag screw to 3,113 pounds.

TIMBER CONNECTORS

Timber connectors are round steel rings or plates embedded between members in precut grooves. When used with bolts or lag screws, they develop the highest strength in lateral loading of all fastener types. The two types of timber connectors most common in bridge applications are split rings and shear plates (Figure 5-26). Split rings are round steel rings with slightly tapered edges that wedge the connector in the precut grooves. They are manufactured in diameters of 2-1/2 inches and 4 inches from hot-rolled carbon steel meeting Society of Automotive Engineers Specification SAE-1010. As the name implies, the side of the ring is split to allow the connector to expand as it is placed in the groove. Shear plates are 2-5/8-inch or 4-inch-diameter round steel plates with a flange on one side. The 2-5/8-inch plates are pressed from hot-rolled steel meeting SAE-1010. The 4-inch plates are cast malleable iron manufactured to Grade 32510 of ASTM Standard A47. Typical dimensions for split rings and shear plates are given in Table 16-6.

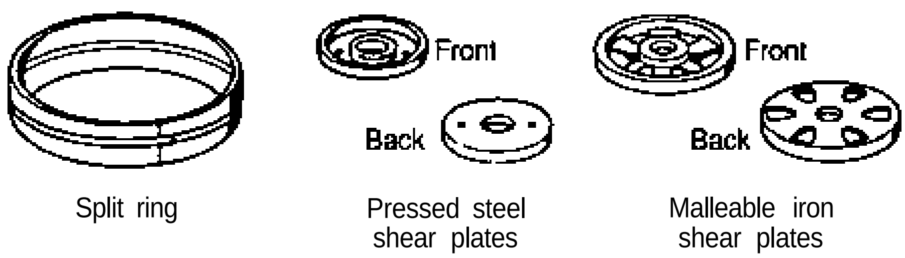


Figure 5-26. - Types of timber connectors.

Timber connectors are used in lateral connections with a bolt or lag screw placed concentrically through the center of the connector (Figure 5-27). Split rings are limited to wood-to-wood connections where one ring is placed at each wood interface. Shear plates are best adapted for wood-to-metal connections but may be used back to back for wood connections; however, one split ring in wood connections is more economical than two shear plates. For both types of connectors, the bolt or lag screw is an integral part of the connector unit and serves to clamp the members together so that the connector functions effectively. For shear plates, the bolt also must transfer the shear across the member interface.

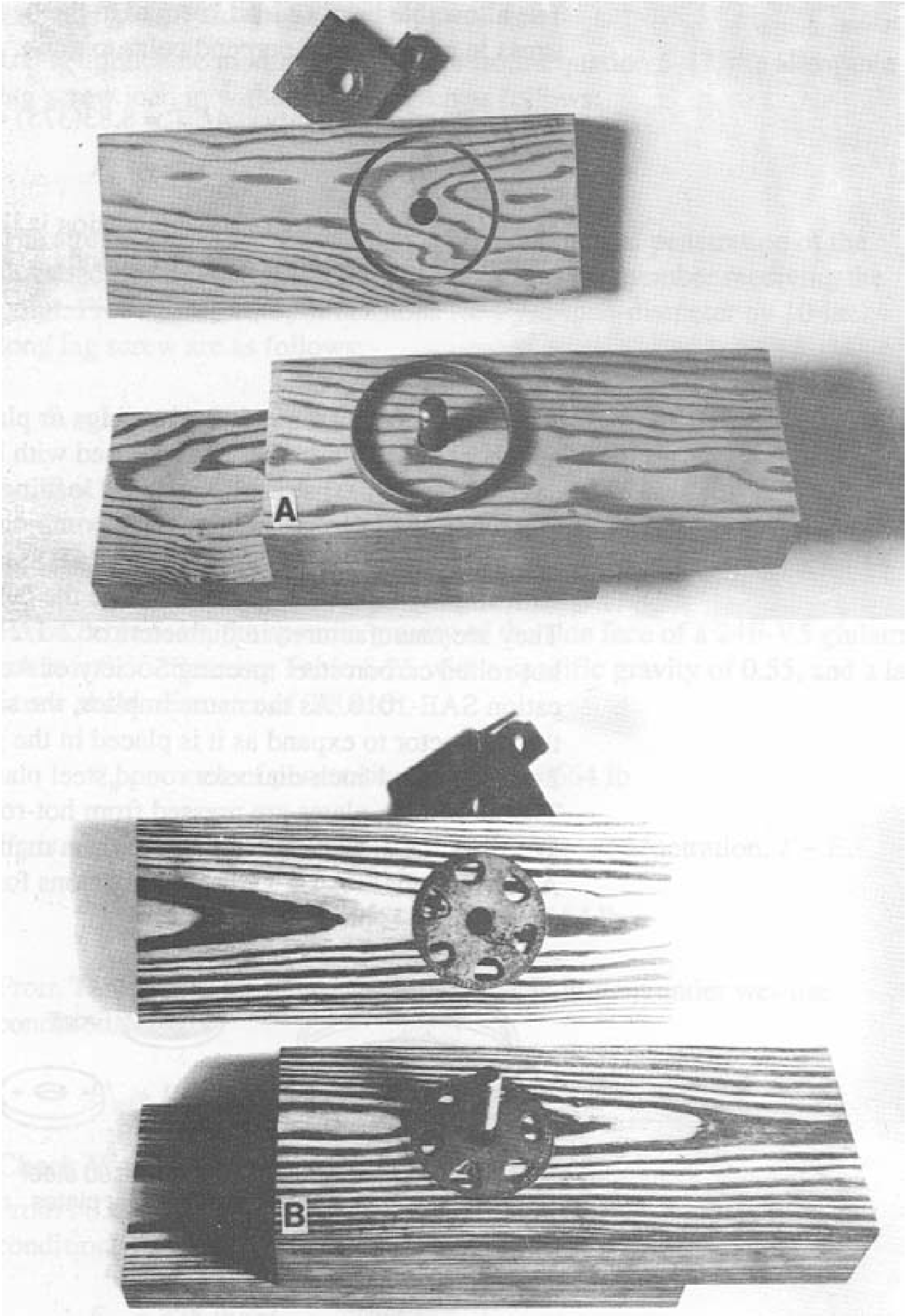


Figure 5-27.- Typical timber connectorjoints: (A) split ring connector between wood members. (B) Shear plates used back-to-back between wood members.

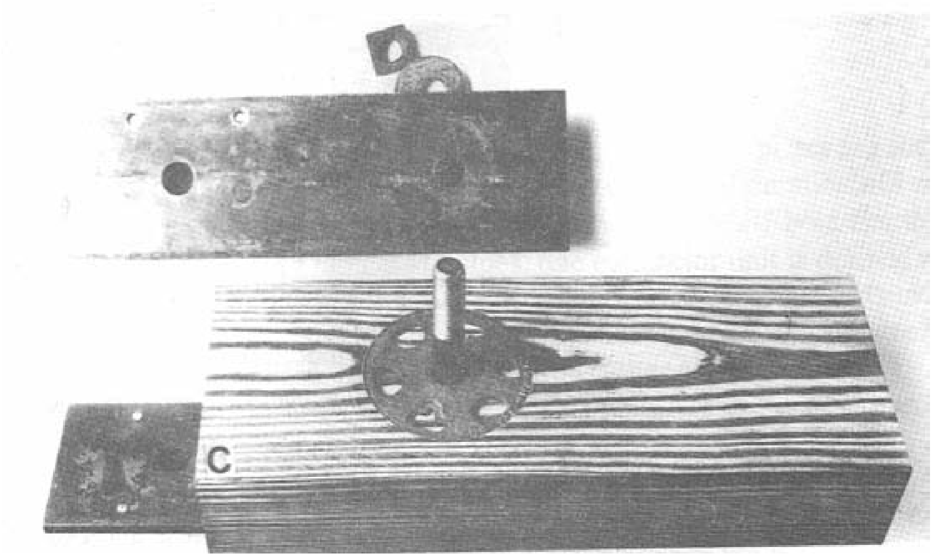


Figure 5-27. - Typical timber connector joints (continued): (C) Shear plate used between wood and steel members.

Net Area

The net area at a timber connection is the gross area of the member minus the projected area of the bolt holes and the projected area of the connector groove within the member (Figure 5-28). When connectors are staggered, adjacent connectors with a parallel to grain spacing equal to or less than one connector diameter are considered to occur at the same critical section. The required net area in tension and compression members is determined by dividing the total load transferred at the connection by the applicable allowable design stress, F_t' or F_c'.

Design of Lateral Connections

As with other types of lateral connections, the strength of timber connectors is developed by bearing between the connector and the wood (Figure 5-29). Design values for connectors are considerably higher than bolts or

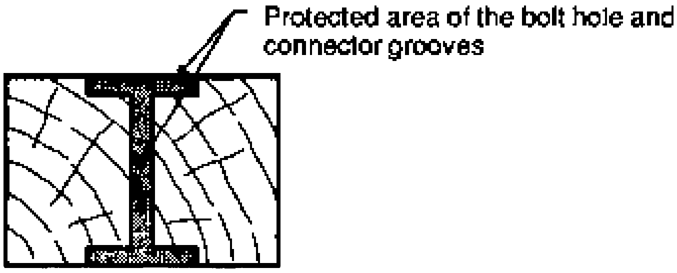
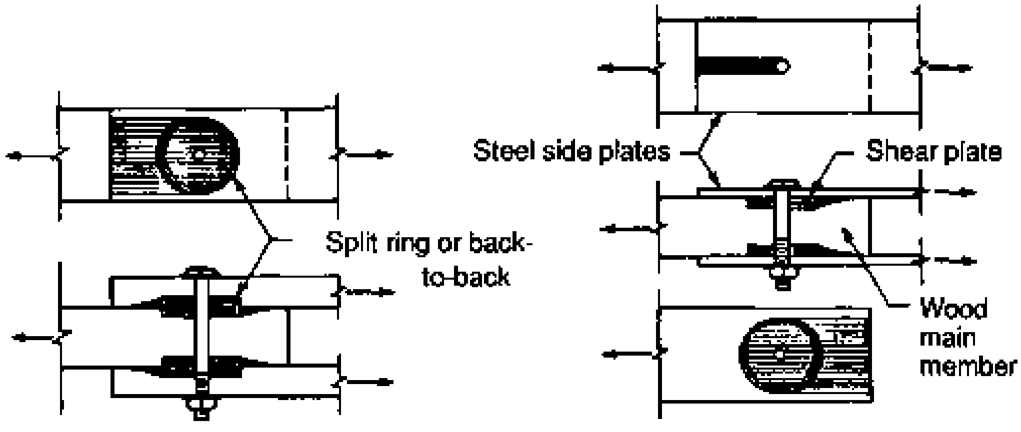


Figure 5-28. - The net area at a timber connector is equal to the gross area of the member minus the projected area of the bolt hole and connector grooves.



Wood-Wood Connection **Wood-Steel Connection**

Figure 5-29. - Typical configuration and stress distribution for laterally loaded timber connectors.

lag screws because they bend less and provide more bearing area. With connectors, the inner surface of the ring bears against the inner core of wood, while the outer surface bears against the outer wall of the groove. Split rings are especially efficient because the tongue and groove split allows expansion, resulting in better load distribution in bearing. In most applications, connector capacity is controlled by the strength of the wood; however, for some shear plates, capacity may be controlled by the strength of the connector. In such cases, maximum design values are limited by the NDS.

The allowable value on one timber connector is equal to the tabulated value adjusted by all applicable modification factors. When several connectors are used, the design value is the sum of the individual connector values adjusted by the group action factor, C_g. Applicable modification factors for timber connectors are given by Equations 5-48 and 5-49 for split rings and by Equations 5-50 and 5-51 for shear plates:

For split rings,

$$P' = PC_D C_M C_t C_R C_e C_n C_s C_g C_{tb} \tag{5-48}$$

$$Q' = QC_D C_M C_t C_R C_e C_n C_s C_g C_{tb} \tag{5-49}$$

For shear plates,

$$P' = PC_D C_M C_t C_R C_e C_n C_s C_g C_{st} C_{tb} \leq P'_{MAX} \tag{5-50}$$

$$Q' = QC_D C_M C_t C_R C_e C_n C_s C_g C_{tb} \leq Q'_{MAX} \tag{5.51}$$

where P'_{MAX} and Q'_{MAX} are the maximum allowable values for shear plates, limited by the strength of the connector.

When timber connectors are loaded at an angle to the grain, modification factors for end distance *(C_n)*, edge distance *(C_e)*, and spacing *(C_s)* are based on the loading angle. As a result, these factors are applied after application of the Hankinson formula (Equation 5-42).

Tabulated Design Values

The tabulated NDS design values for split rings and shear plates are shown in Tables 5-27 and 5-28. The values are based on normal duration of load and dry-use conditions for one connector unit with an A307 bolt. For the purpose of determining tabulated values, one connector unit is defined as (1) one split ring in a wood-wood connection, (2) two shear plates back to back in a wood-wood connection, or (3) one shear plate in a wood-steel connection. In each case, the tabulated value is the load that occurs in single shear at the location of the connector, regardless of the total number of members in the connection.

To determine the tabulated value for either type of connector, enter the appropriate table with the connector diameter and read the tabulated value, by species group, based on the number of faces of the piece with connectors on the same bolt, and the net thickness of the thinnest member in contact with the connector. For loading perpendicular to grain, tabulated values are additionally based on the loaded edge distance of the connector. Values for intermediate member thicknesses and loaded edge distances are determined by linear interpolation.

When determining tabulated connector values, the following considerations apply:

1. Timber connectors cannot be used in members less than the minimum net thickness given in Tables 5-27 and 5-28.

2. The bolt diameter specified for each connector is the minimum diameter A307 bolt required to meet tabulated values. Increasing the bolt diameter is permissible but does not increase the tabulated values.

3. Maximum loads on shear plates (P'_{MAX} and Q'_{MAX}) shall not exceed the following:

   (a) 2,900 pounds for a 2-5/8-inch shear plate,

   (b) 4,400 pounds for a 4-inch shear plate with a 3/4-inch bolt, or

   (c) 6,000 pounds for a 4-inch shear plate with a 7/8-inch bolt.

   When tabulated values exceed P'_{MAX} or Q'_{MAX}, they are marked with an asterisk in Table 5-28.

Table 5-27.—Tabulated split ring design values.

Split-ring diam. (inches)	Bolt diam. (inches)	Number of faces of piece with connectors on same bolt	Net thickness of piece (inches)	Loaded parallel to grain (0°): Minimum edge distance (inches)	Loaded parallel to grain (0°): Design value per connector unit and bolt (pounds): Group A woods	Group B woods	Group C woods	Group D woods	Loaded perpendicular to grain (90°): Edge distance (inches): Unloaded edge, min.	Loaded edge	Loaded perpendicular to grain (90°): Design value per connector unit and bolt (pounds): Group A woods	Group B woods	Group C woods	Group D woods
2-1/2	1/2	1	1 min.	1-3/4	2630	2270	1900	1640	1-3/4	1-3/4 min.	1580	1350	1130	970
										2-3/4 or more	1900	1620	1350	1160
			1-1/2 or more	1-3/4	3160	2730	2290	1960	1-3/4	1-3/4 min.	1900	1620	1350	1160
										2-3/4 or more	2280	1940	1620	1390
		2	1-1/2 min.	1-3/4	2430	2100	1760	1510	1-3/4	1-3/4 min.	1460	1250	1040	890
										2-3/4 or more	1750	1500	1250	1070
			2 or more	1-3/4	3160	2730	2290	1960	1-3/4	1-3/4 min.	1900	1620	1350	1160
										2-3/4 or more	2280	1940	1620	1390
4	3/4	1	1 min.	2-3/4	4090	3510	2920	2520	2-3/4	2-3/4 min.	2370	2030	1700	1470
										3-3/4 or more	2840	2440	2040	1760
			1-1/2	2-3/4	6020	5160	4280	3710	2-3/4	2-3/4 min.	3490	2990	2490	2150
										3-3/4 or more	4180	3590	2990	2580
			1-5/8 or more	2-3/4	6140	5260	4380	3790	2-3/4	2-3/4 min.	3560	3050	2540	2190
										3-3/4 or more	4270	3660	3050	2630
		2	1-1/2 min.	2-3/4	4110	3520	2940	2540	2-3/4	2-3/4 min.	2480	2040	1700	1470
										3-3/4 or more	2980	2450	2040	1760
			2	2-3/4	4950	4250	3540	3050	2-3/4	2-3/4 min.	2870	2470	2050	1770
										3-3/4 or more	3440	2960	2460	2120
			2-1/2	2-3/4	5830	5000	4160	3600	2-3/4	2-3/4 min.	3380	2900	2410	2080
										3-3/4 or more	4050	3480	2890	2500
			3 or more	2-3/4	6140	5260	4380	3790	2-3/4	2-3/4 min.	3560	3050	2540	2190
										3-3/4 or more	4270	3660	3050	2630

Design values in pounds apply to one split ring and bolt in single shear when installed in seasoned wood that will remain dry in service and be subject to normal loading conditions.

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Table 5-28.—Tabulated shear plate design values.

Shear-plate diam. (inches)	Bolt diam. (inches)	Number of faces of piece with connectors on same bolt	Net thickness of piece (inches)	Loaded parallel to grain (0°): Minimum edge distance (inches)	Loaded parallel to grain (0°): Design value per connector unit and bolt (pounds): Group A woods	Group B woods	Group C woods	Group D woods	Loaded perpendicular to grain (90°): Edge distance (inches): Unloaded edge, min.	Loaded edge	Loaded perpendicular to grain (90°): Design value per connector unit and bolt (pounds): Group A woods	Group B woods	Group C woods	Group D woods
2-5/8	3/4	1	1-1/2 min.	1-3/4	3110*	2670	2220	2010	1-3/4	1-3/4 min.	1810	1550	1290	1110
										2-3/4 or more	2170	1860	1550	1330
		2	1-1/2 min.	1-3/4	2420	2080	1730	1500	1-3/4	1-3/4 min.	1410	1210	1010	870
										2-3/4 or more	1690	1450	1210	1040
			2	1-3/4	3190*	2730	2270	1960	1-3/4	1-3/4 min.	1850	1590	1320	1140
										2-3/4 or more	2220	1910	1580	1370
			2-1/2 or more	1-3/4	3330*	2860	2380	2060	1-3/4	1-3/4 min.	1940	1660	1380	1200
										2-3/4 or more	2320	1990	1650	1440
4	3/4 or 7/8	1	1-1/2 min.	2-3/4	4370	3750	3130	2700	2-3/4	2-3/4 min.	2540	2180	1810	1550
										3-3/4 or more	3040	2620	2170	1860
			1-3/4 or more	2-3/4	5090*	4360	3640	3140	2-3/4	2-3/4 min.	2950	2530	2110	1810
										3-3/4 or more	3540	3040	2530	2200
		2	1-3/4 min.	2-3/4	3390	2910	2420	2090	2-3/4	2-3/4 min.	1970	1680	1400	1250
										3-3/4 or more	2360	2020	1680	1410
			2	2-3/4	3790	3240	2700	2330	2-3/4	2-3/4 min.	2200	1880	1570	1360
										3-3/4 or more	2640	2260	1880	1630
			2-1/2	2-3/4	4310	3690	3080	2660	2-3/4	2-3/4 min.	2500	2140	1780	1540
										3-3/4 or more	3000	2550	2140	1850
			3	2-3/4	4830	4140	3450	2980	2-3/4	2-3/4 min.	2800	2400	2000	1720
										3-3/4 or more	3360	2880	2400	2060
			3-1/2 or more	2-3/4	5030*	4320	3600	3110	2-3/4	2-3/4 min.	2920	2500	2090	1800
										3-3/4 or more	3500	3000	2510	2160

Design values in pounds apply to one shear plate unit and bolt in single shear when installed in seasoned wood members that will remain dry in service and be subject to normal loading conditions. Allowable values for shear plates (after adjustment by applicable modification factors) shall not exceed P'_{MAX} or Q'_{MAX} as given below:

a. 2-5/8 inch shear plate .... 2,900 lb.
b. 4-inch shear plate with 3/4 inch bolt .... 4,400 lb.
c. 4-inch shear plate with 7/8 inch bolt .... 6,000 lb.

Loads followed by an asterisk exceed P'_{MAX} or Q'_{MAX} but are necessary for proper determination of values for other angles of load to the grain.

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4. If concentric grooves for two sizes of split rings are cut in the member, rings must be installed in both grooves; however, the tabulated design value is that for the larger ring only.

Lag Screws

Tabulated values for timber connectors are based on a bolted connection. Lag screws may be used, provided the shank diameter of the lag is the same as specified for a bolt and provided the lag screw threads are cut rather than rolled (cut threads hold better). When lag screws are used instead of bolts, tabulated values must be adjusted by the lag screw factor C_{lb} given in Table 5-29.

Steel Side Plates

When steel rather than wood side plates are used, tabulated values may be increased for 4-inch shear plates loaded parallel to grain only (no increase is allowed for 2-5/8-inch shear plates or split rings). Values of C_{st} are given below. However, the adjusted load on any shear plate is limited to the maximum values P'_{MAX} and Q'_{MAX}.

Species Group	**C_{st} for 4-inch shear plates loaded parallel to grain**
A	1.18
B	1.11
C	1.05
D	1.00

Table 5-29. - Lag screw modification factor, C_{lb}

Connector size and type	Side plate	Penetration[1]	Penetration of lag screw into member receiving point (number of shank diameters) Fastener species group: I	II	III	IV	C_{lb}
2-1/2-inch split ring 4-inch split ring 4-inch shear plate	Wood or Metal	Standard	7	8	10	11	1.00
		Minimum	3	3-1/2	4	4-1/2	0.75
2-5/8-inch shear plate	Wood	Standard	4	5	7	8	1.00
		Minimum	3	3-1/2	4	4-1/2	0.75
2-5/8-inch shear plate	Metal	Standard and Minimum	3	3-1/2	4	4-1/2	1.00

[1] Use straight line interpolation for intermediate values.

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Distance and Spacing Requirements

Distance and spacing requirements for timber connectors loaded parallel to grain and perpendicular to grain are summarized in Table 5-30. These requirements are given as the minimum dimension for the full tabulated value and as the minimum dimension for reduced value, as previously discussed for bolts. It is recommended that the minimum dimensions for full tabulated value be used whenever possible. When space is not available, the minimum dimensions for a reduced value may be used provided tabulated values are reduced by the applicable modification factors for edge distance, C_e, end distance, C_n, or spacing, C_s. The edge-distance factor for the loaded edge is already factored into tabulated values, and further application of C_e, is not required when tabulated minimum loaded edge-distance values are used. The modification factors C_e, C_n, and C_s are not cumulative, and the lowest value of the three is used. However, when end distance or spacing is reduced for any connector in a group, the lowest applicable factor applies to all connectors in the group. Modification factor values for intermediate dimensions are determined by straight-line interpolation.

When timber connectors are loaded at an angle to the grain of the member, refer to the NDS and the *AITC Timber Construction Manual* for distance and spacing requirements.

Connector Placement

All holes, grooves, and daps for timber connectors must be precision machined with special cutters for proper connector performance and assembly (Figure 5-30). Fabrication is best suited to a shop environment but can be done in the field when shop fabrication is not possible (Figure 5-31). The holes for bolts and lag screws are prebored in the manner previously discussed for the individual fasteners. Grooves and daps for split rings and shear plates must be appropriate for the type and size of connector. Connectors from different manufacturers may differ slightly in shape or cross section, and cutter heads must be specifically designed to accurately conform to the dimensions and shape of the par-

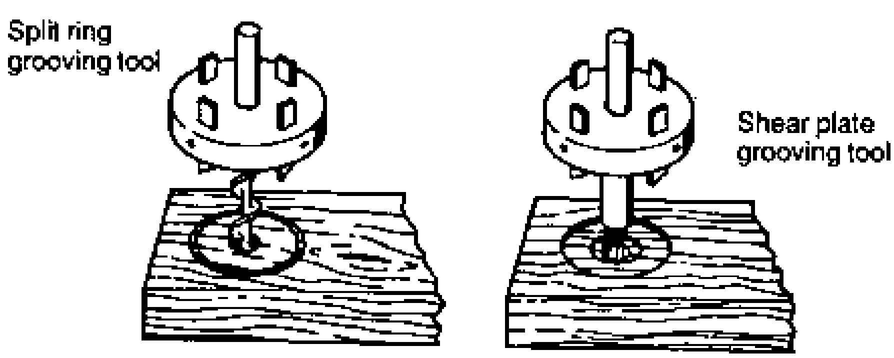
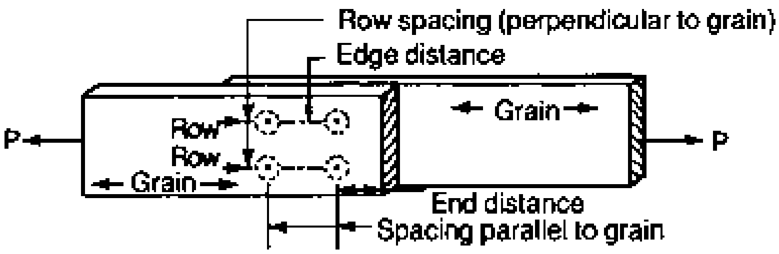


Figure 5-30. - Tools used for grooving wood for timber connectors.

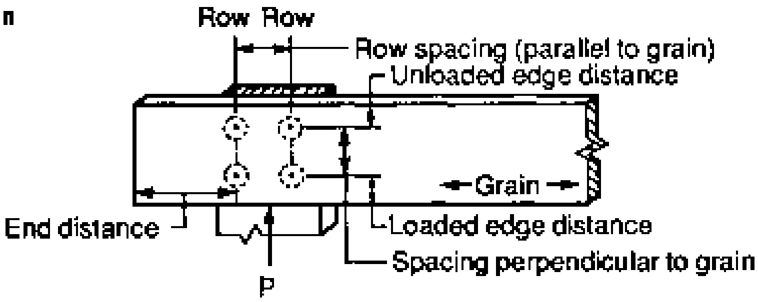
Table 5-30. - Summary of edge distance, end distance, and spacing requirements for timber connectors.

Loading parallel to grain



	2-1/2-inch split rings or 2-5/8-inch shear plates		4-inch split rings or shear plates	
	Minimum dimension for full tabulated value (inches)	Minimum dimension for reduced value[1] (inches)	Minimum dimension for full tabulated value (inches)	Minimum dimension for reduced value[1] (inches)
Edge distance	1-3/4	N/A[2]	2-3/4	N/A
End distance				
Tension members	5-1/2	3/4 (C_n = 0.625)	7	3-1/2 (C_n = 0.625)
Compression members	4	2-1/2 (C_n = 0.625)	5-1/2	3-1/4 (C_n = 0.625)
Spacing				
Parallel to grain	6-3/4	3-1/2 (C_Δ = 0.5)	9	5 (C_Δ = 0.5)
Row spacing perpendicular to grain	3-1/2	N/A	5	N/A

Loading perpendicular to grain



	2-1/2-inch split rings or 2-5/8-inch shear plates		4-inch split rings or shear plates	
	Minimum dimension for full tabulated value (inches)	Minimum dimension for reduced value[1] (inches)	Minimum dimension for full tabulated value (inches)	Minimum dimension for reduced value[1] (inches)
Edge distance				
Unloaded edge	1-3/4	N/A	2-3/4	N/A
Loaded edge[2]	2-3/4	1-3/4	3-3/4	2-3/4
End distance				
Tension members	5-1/2	2-3/4 (C_n = 0.625)	7	3-1/2 (C_n = 0.625)
Compression members	5-1/2	2-3/4 (C_n = 0.625)	7	3-1/2 (C_n = 0.625)
Spacing				
Row spacing parallel to grain	3-1/2	N/A	5	N/A
Perpendicular to grain	4-1/2	3-1/2 (C_Δ = 0.5)	5	5 (C_Δ = 0.5)

[1] For dimensions between the tabulated value and the reduced value use straight line interpolation to compute the modification factor value.
[2] See Table 5-27 and Table 5-28 for reduced design values for minimum loaded edge distance.
All dimensions are measured from the center of the connector.

Figure 5-31. - Field-grooving for a split ring connector for a curb-deck attachment. Field fabrication such as this requires field treating with wood preservative, as discussed in Chapter 12 (photo courtesy of Wheeler Consolidated, Inc.).

ticular connector used. The heavy 4-inch-diameter shear plates may sometimes be cast out-of-round and should be checked for dimensions and roundness before assembly. An out-of-round plate should not be forced into a round groove.

Bolts and lag screws installed with timber connectors must be provided with plate or malleable iron washers between the outside wood member and the head or nut of the fastener. Cut washers are not suitable for use with connectors and are not permitted. The minimum washer size for each type of connector is given in Table 16-6. When an outside member is a steel plate or shape, the washer may be omitted except when desirable to prevent bearing on the fastener threads.

Design values for timber connectors are based on the assumption that the faces of the members will be brought into tight contact when the connectors are installed. When timber connectors are installed in wood with a high moisture content, they should be checked periodically to ensure that shrinkage of the wood has not caused members to separate. It may be necessary to retighten connections as the wood dries.

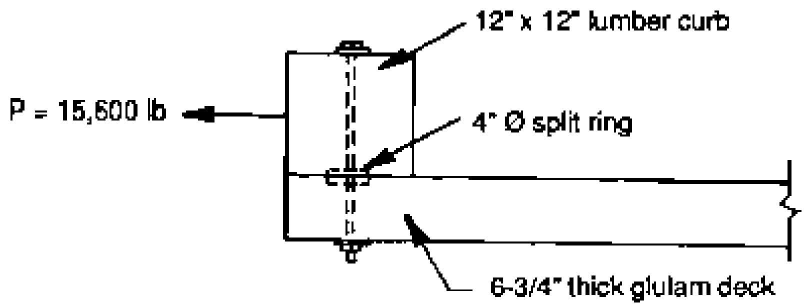
Example 5-15 - Lateral split ring connection

A dressed 12-inch by 12-inch lumber curb is bolted along the edges of a 6-3/4-inch-thick transverse glulam deck. The curb serves at an attachment point for vehicular railing where a transverse reaction of 15,600 pounds is transfered at the center of the curb height. Determine the number of 4-inch-diameter split ring connectors that are required to transfer the curb load to the deck, assuming the following:

1. Members will be exposed to weathering (wet-use conditions) with a duration of load factor, C_D, of 1.65; adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. The glulam deck is combination symbol No. 2.

3. The curb is surfaced Douglas Fir-Larch, graded No. 1 to WWPA rules.



Solution

In this connection, the curb is loaded perpendicular to grain while the deck is loaded parallel to grain. Unlike bolted connections, tabulated values for timber connectors are based on a two-member (single shear) joint and values for loading parallel to grain and perpendicular to grain are read directly from tables in the NDS (Table 5-27). The procedure used here will be to design the connection based on perpendicular-to-grain loading (which normally controls), then check for parallel-to-grain loading.

Curb Loading Perpendicular to Grain

The allowable design value for one split ring loaded perpendicular to grain is given by Equation 5-49. Including possible modification factors for connector distance and spacing, the equation in this case becomes

$$Q' = QC_DC_MC_\Delta C_s$$

From Table 5-14, Douglas Fir-Larch is in Load Group B for timber connector design. The tabulated value for one split ring loaded perpendicular to grain is obtained from Table 5-27. Entering that table for a 4-inch-diameter split ring, 3/4-inch bolt, one member face with a connector on the same bolt, member thickness greater than 1-5/8-inches, and a loaded edge distance greater than 3-3/4-inches:

Q = 3,660 lb

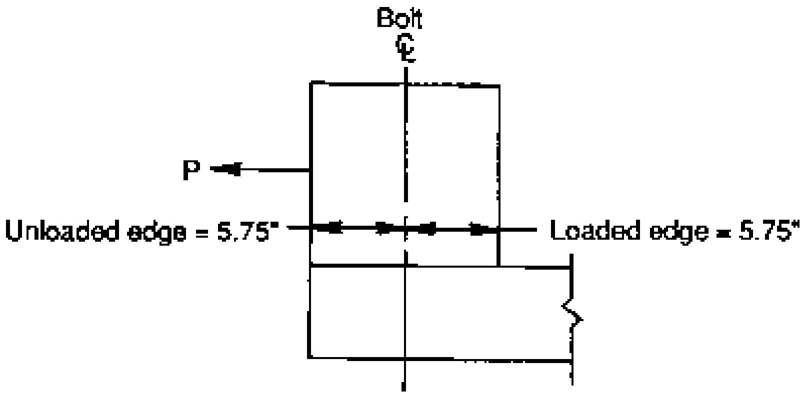
From Table 5-18 for timber connectors used in partially seasoned or wet-condition sawn lumber:

$C_M = 0.67$

Minimum values of connector distance and spacing for full loading are obtained from Table 5-30:

Unloaded edge distance	2.75 in.
Loaded edge distance	3.75 in.
Spacing parallel to grain	5 in.

From Table 16-2, the width of a dressed 12-inch by 12-inch curb is 11.5 inches. Centering the connector on the curb provides a loaded and unloaded edge distance of 5.75 inches:



Using a minimum connector spacing of 5 inches, all distance and spacing requirements for full load are met and values of C_e and C_s each become 1.0.

The allowable load for one split ring is computed by substituting values into the equation for Q':

$$Q' = QC_DC_MC_eC_s = 3{,}660(1.65)(0.67)(1.0)(1.0) = 4{,}046 \text{ lb}$$

The required number of split rings is obtained by dividing the applied load by Q',

$$\text{Number of split rings} = \frac{15{,}600 \text{ lb}}{4{,}046 \text{ lb}} = 3.86 = 4$$

Deck Loading Parallel to Grain

Using the applicable modification factors for this case, the allowable load for one split ring loaded parallel to grain is given by Equation 5-48:

$$P' = PC_D C_M C_t C_n C_s$$

From Table 5-15, glulam combination symbol No. 2 is in Load Group B for timber connector design. The tabulated value for one split ring loaded parallel to grain is obtained from Table 5-27 using the same table values previously used for loading perpendicular to grain:

$$P = 5{,}260 \text{ lb}$$

From Table 5-18 for timber connectors used in glulam under wet-use conditions:

$$C_M = 0.67$$

Minimum values of connector distance and spacing for full loading are obtained from Table 5-30:

Edge distance	2.75 in.
End distance (tension members)	7 in.
Spacing perpendicular to grain	5 in.

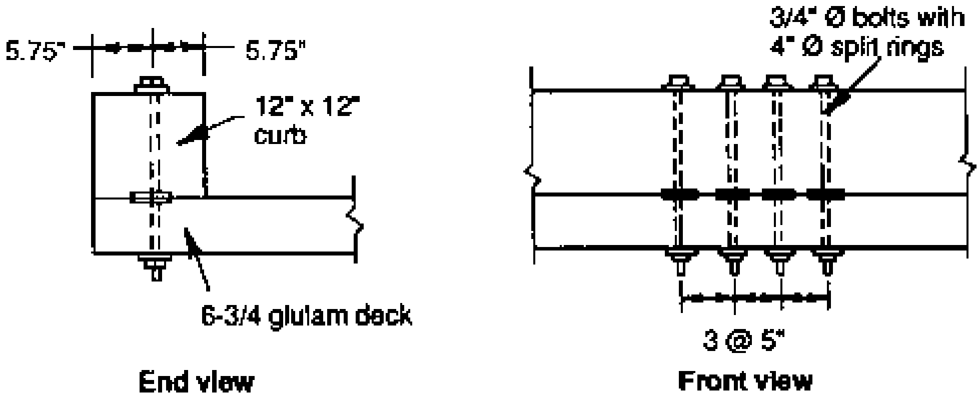
All distance and spacing requirements can be met with the exception of end distance, which is 5.75 inches rather than the 7 inches required for full load (end distance for parallel-to-grain loading is the same as the unloaded edge distance for perpendicular-to-grain loading). From Table 5-30, end distance can be reduced to a minimum of 3.5 inches provided the tabulated load is reduced by C_n = 0.625. Using linear interpolation for the 5.75-inch distance, C_n = 0.87.

Substituting values into the equation for P',

$$P' = PC_D C_M C_t C_n C_s = 5{,}260(1.65)(0.67)(1.0)(0.87)(1.0) = 5{,}059 \text{ lb}$$

P' = 5,059 lb > Q' = 4,046 pounds so connector capacity is controlled by loading perpendicular to grain.

Summary
The connection will be made using four 4-inch-diameter split rings with 3/4-inch-diameter bolts. The capacity of the connection is limited by curb loading perpendicular to grain to 16,184 pounds. The bolts will be spaced 5 inches on-center and will be provided with malleable iron washers at each end:



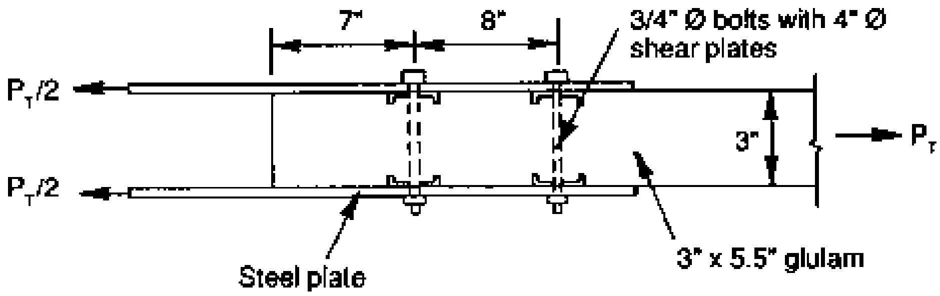
Example 5-16 - Lateral shear-plate connection

A glulam tension member measures 3-inches wide by 5.5 inches deep. The end of the member is held between steel plates by two 3/4-inch-diameter bolts with four 4-inch-diameter shear plates. Determine the capacity of the connection, assuming the following:

1. Members will be exposed to weathering (wet-use conditions) and a normal duration of load; adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. The glulam is combination symbol No. 47.

3. The capacity of the steel plates is satisfactory.



Solution

The capacity of this connection will be controlled either by the strength of the glulam member or the strength of the connectors. Glulam capacity will be computed first, followed by connector capacity.

Capacity of Glulam Member

The capacity of the glulam member in tension is equal to the allowable tensile stress times the net member area. The allowable stress in tension parallel to grain is computed using the applicable modification given by Equation 5-23:

$$F_t' = F_t C_D C_M$$

From *AITC 117-Design* for combination symbol No. 47,

$$F_t = 1{,}200 \text{ lb/in}^2$$

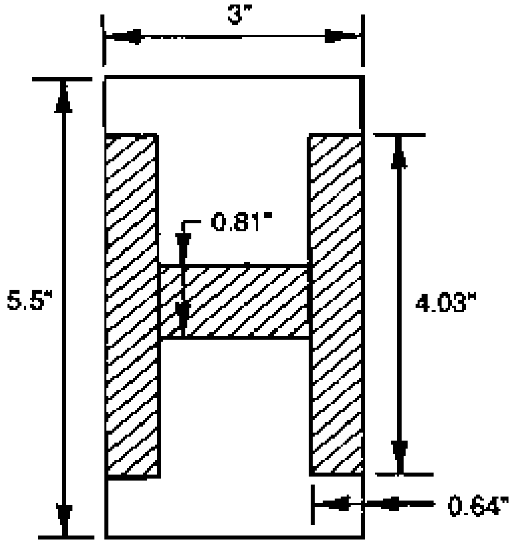
From Table 5-7,

$$C_M = 0.80$$

Substituting,

$$F_t' = F_t C_D C_M = 1{,}200(1.0)(0.80) = 960 \text{ lb/in}^2$$

The net area of the member is equal to the gross area minus the projected area of the shear plates and bolt hole. Dimensions of the shear plates and bolt hole are obtained from timber connector properties given in Table 16-6:



$$\text{Gross area} = (3 \text{ in.})(5.5 \text{ in.}) = 16.5 \text{ in}^2$$

$$\text{Shear plate area} = 2\,[(4.03 \text{ in.})(0.64 \text{ in.})] = 5.16 \text{ in}^2$$

$$\text{Bolt hole area} = [\,(3 \text{ in.}) - 2(0.64 \text{ in.})]\,(0.81 \text{ in}) = 1.39 \text{ in}^2$$

$$A_{NET} = 16.5 \text{ in}^2 - 5.16 \text{ in}^2 - 1.39 \text{ in}^2 = 9.95 \text{ in}^2$$

The member capacity equals the allowable stress times the net area:

$$\text{Member capacity} = F_t'(A_{NET}) = 960\ (9.95) = 9{,}552 \text{ lb}$$

Capacity of Shear Plates

The allowable design value for one shear plate loaded parallel to grain is given by Equation 5-50. Including possible modification factors for connector distance and spacing, the equation in this case becomes

$$P' = PC_DC_MC_tC_nC_sC_gC_{st}$$

From Table 5-15, glulam combination symbol No. 47 is in Load Group B for timber connector design. The tabulated value for one shear plate loaded parallel to grain is obtained from Table 5-28. Entering that table for a 4-inch-diameter shear plate, 3/4-inch-diameter bolt, two member faces with a connector on the same bolt, and a member thickness of 3 inches:

$$P = 4{,}140 \text{ lb}$$

From Table 5-18 for timber connectors used under wet-use conditions in glulam,

$$C_M = 0.67$$

Values of connector distance and spacing for full loading are obtained from Table 5-30:

Edge distance	2.75 in.
End distance (tension members)	7 in.
Spacing parallel to grain	9 in.

All distance and spacing requirements for full load are met except spacing parallel to grain, which is 8 inches instead of 9 inches. Spacing can be reduced to a minimum of 5 inches provided tabulated values are reduced by $C_s = 0.50$. By interpolation for an 8-inch spacing,

$$C_s = 0.88$$

For two connectors in a row, adjustment for group action is not required, and

$C_g = 1.0$

For steel side plates used with Group B species,

$C_{st} = 1.11$

Substituting values into the equation for P',

$$P' = PC_DC_MC_tC_nC_sC_gC_{st}$$

$$= 4{,}140(1.0)(0.67)(1.0)(1.0)(0.88)(1.0)(1.11) = 2{,}709 \text{ lb}$$

For four shear plates,

$4(P') = 4(2709) = 10{,}836$ lb

Summary

The capacity of the connection is 9,552 pounds and is controlled by the capacity of the glulam member in tension parallel to grain.

NAILS AND SPIKES

Nails and spikes are the most common wood fastener for building construction. For bridge applications, however, their use is mostly limited to laminating lumber decks and attaching plank-wearing surfaces. Design is usually based on nailing schedules or specification requirements rather than on structural analysis, but an engineered design may be required in some situations. The primary disadvantage with nails and spikes is their susceptibility to loosening from vibrations or changes in moisture content. Withdrawal connections are not recommended, and discussions in this section are limited to lateral loading conditions only. Refer to the NDS for criteria on withdrawal connections.

Nails and spikes are available in a wide variety of lengths and diameters in four different types: box nails, common wire nails, common wire spikes, and threaded hardened-steel nails and spikes. Size is specified by pennyweight, or by diameter and length for larger spikes (Table 5-31). Spikes are longer and have a larger diameter than nails. Most nails and spikes are manufactured from low- or medium-carbon steel. Threaded hardened-steel nails and spikes are made of high-carbon steel wire that is heat treated and tempered to provide higher strength.

Table 5-31. - Typical sizes of nails and spikes.

Pennyweight	Length (in.)	Wire diameter (in.) Box nails	Common wire nails	Threaded hardened-steel nails	Common wire spikes
6d	2	0.099	0.113	0.120	—
8d	2-1/2	0.113	0.131	0.120	—
10d	3	0.128	0.148	0.135	0.192
12d	3-1/4	0.128	0.148	0.135	0.192
16d	3-1/2	0.135	0.162	0.148	0.207
20d	4	0.148	0.192	0.177	0.225
30d	4-1/2	0.148	0.207	0.177	0.244
40d	5	0.162	0.225	0.177	0.263
50d	5-1/2	—	0.244	0.177	0.283
60d	6	—	0.263	0.177	0.283
70d	7	—	—	0.207	—
80d	8	—	—	0.207	—
90d	9	—	—	0.207	—
5/16	7	—	—	—	0.312
3/8	8-1/2	—	—	—	0.375

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Nail and spike classifications are based on the type of shank, whether smooth or deformed (Figure 5-32). Deformed shanks are generally spiral (helical) or ringed, but patterns may vary. Deformed shanks are used in most bridge applications because they provide greater withdrawal resistance and are less susceptible to loosening from vibrations or changes in wood moisture content.

Net Area

The net area at nailed or spiked connections is normally taken as the gross area of the member. When large-diameter spikes are used, the net area may be computed by subtracting the projected area of the fasteners, based on designer judgment.

Design of Lateral Connections

In laterally loaded nail and spike connections, the capacity of the connection is controlled by deformation (slip) rather than strength. As a result, design values are independent of the direction of loading with respect to the direction of grain. The allowable value for one nail or spike is the tabulated design value from the NDS adjusted by all applicable modification factors, as given by

$$P_N' = P_N C_D C_M C_t C_R C_{st} \qquad (5\text{-}52)$$

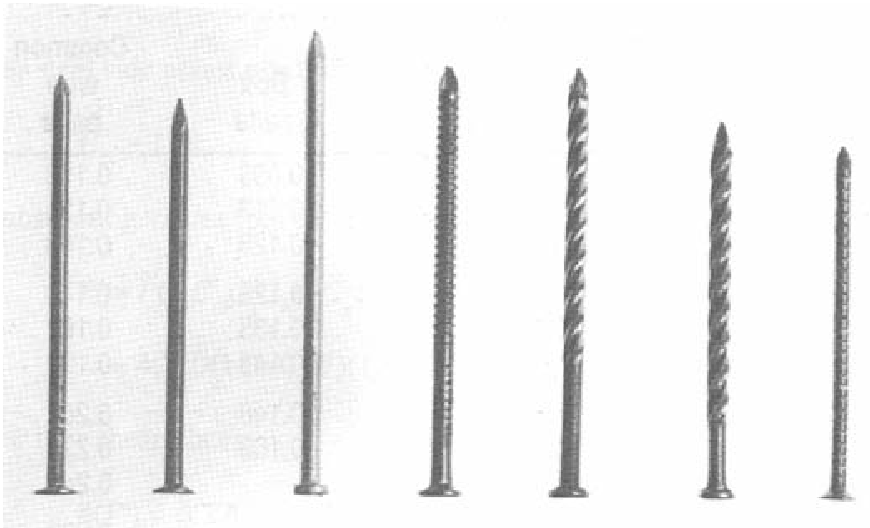


Figure 5-32. - Types of nails: (left to right), bright, smooth wire; cement coated; zinc-coated; annularly threaded; helically threaded; helically threaded and barbed; and barbed.

where P_N' is the allowable lateral load applied at any angle to the grain of the members.

When more than one nail or spike is used, the allowable value for the connection is the sum of the individual design values. Adjustment by the group action factor, C_g, is not required for nails and spikes.

Tabulated Design Loads

Tabulated lateral values for nails and spikes loaded at any angle to grain are given in Table 5-32. The values are for side-grain connections in seasoned wood and are based on the depth of penetration of the nail or spike into the member. In two-member connections, the penetration is measured in the member holding the point. For three-member connections, the penetration is measured in the center member. To determine the tabulated value, enter the table with the type and size of fastener and read horizontally across from the applicable species group (for connections with members of different species, use the higher numbered species group). For full tabulated value, penetration must be a minimum of 10 diameters in Group I species, 11 diameters in Group II species, 13 diameters in Group III species, and 14 diameters in Group IV species. The minimum penetration for any connection cannot be less than one-third of these values. For intermediate penetrations, values are determined by linear interpolation between zero and the tabulated value. However, values cannot be increased for penetrations greater than those required for full tabulated value.

Table 5-32. - Tabulated lateral load design values for nails and spikes.

Common Wire Nails

Pennyweight	6d	8d	10d	12d	16d	20d	30d	40d	50d	60d
Length	2	2-1/2	3	3-1/4	3-1/2	4	4-1/2	5	5-1/2	6
Diameter	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263
10 Diameters	1.13	1.31	1.48	1.48	1.62	1.92	2.07	2.25	2.44	2.63
11 Diameters	1.24	1.44	1.63	1.63	1.78	2.11	2.28	2.48	2.68	2.89
13 Diameters	1.47	1.70	1.92	1.92	2.11	2.50	2.69	2.93	3.17	3.42
14 Diameters	1.58	1.83	2.07	2.07	2.27	2.69	2.90	3.15	3.42	3.68
Species Group I	77	97	115	116	133	172	192	218	246	275
Species Species Group II	63	78	94	94	108	139	155	175	199	223
Species Species Group III	51	64	77	77	88	114	127	144	163	182
Species Species Group IV	41	51	61	61	70	91	102	115	130	146

Threaded Hardened-Steel Nails and Spikes

Pennyweight	6d	8d	10d	12d	16d	20d	30d	40d	50d	60d	70d	80d	90d
Length	2	2-1/2	3	3-1/4	3-1/2	4	4-1/2	5	5-1/2	6	7	8	9
Diameter	0.120	0.120	0.135	0.135	0.148	0.177	0.177	0.177	0.177	0.177	0.207	0.207	0.207
10 Diameters	1.20	1.20	1.35	1.35	1.48	1.77	1.77	1.77	1.77	1.77	2.07	2.07	2.07
11 Diameters	1.32	1.32	1.49	1.49	1.63	1.95	1.95	1.95	1.95	1.95	2.28	2.28	2.28
13 Diameters	1.56	1.56	1.76	1.76	1.92	2.30	2.30	2.30	2.30	2.30	2.69	2.69	2.69
14 Diameters	1.68	1.68	1.89	1.89	2.07	2.48	2.48	2.48	2.48	2.48	2.90	2.90	2.90
Species Group I	77	97	116	116	133	172	172	172	172	172	218	218	218
Species Group II	63	78	94	94	108	139	139	139	139	139	176	176	176
Species Group III	51	64	77	77	88	114	114	114	114	114	144	144	144
Species Group IV	41	51	61	61	70	91	91	91	91	91	115	115	115

Common Wire Spikes

Pennyweight	10d	12d	16d	20d	30d	40d	50d	60d	5/16"	3/8"
Length	3	3-1/4	3-1/2	4	4-1/2	5	5-1/2	6	7	8-1/2
Diameter	0.192	0.192	0.207	0.225	0.244	0.263	0.283	0.283	0.312	0.375
10 Diameters	1.92	1.92	2.07	2.25	2.44	2.63	2.83	2.83	3.12	3.75
11 Diameters	2.11	2.11	2.28	2.48	2.68	2.89	3.11	3.11	3.43	4.13
13 Diameters	2.50	2.50	2.69	2.93	3.17	3.42	3.68	3.68	4.06	4.88
14 Diameters	2.69	2.69	2.90	3.15	3.42	3.68	3.96	3.96	4.37	5.25
Species Group I	172	172	192	218	246	275	307	307	356	468
Species Group II	139	139	155	176	199	223	248	248	288	379
Species Group III	114	114	127	144	163	182	203	203	235	310
Species Group IV	91	91	102	115	130	146	163	163	188	248

Diameters and lengths are in inches; loads are in pounds.
Design values are for lateral loads in single shear (two members) for nails and spikes penetrating not less than 10 diameters in Group I species, 11 diameters in Group II species, 13 diameters in Group III species, and 14 diameters in Group IV species, into the member holding the point. For other diameters and lengths refer to the *Wood Handbook*.[35]
From the NDS;[26] ©1986. Used by permission.

Steel Side Plates

When steel rather than wood side plates are used for lateral connections, the tabulated design values for nails and spikes may be increased by the steel side plate factor *(C_{st} = 1.25)*.

Distance and Spacing Requirements

End distance, edge distance, and spacing of nails and spikes should be sufficient to avoid unusual splitting of the wood. Although no criteria or dimensions are given in AASHTO or the NDS, the following criteria are given in the *Wood Handbook* [35] based on the diameter *d* of the nail or spike:

End distance (tension members)	*15d*
End distance (compression members)	*12d*
Edge distance	*10d*

Nail and Spike Placement

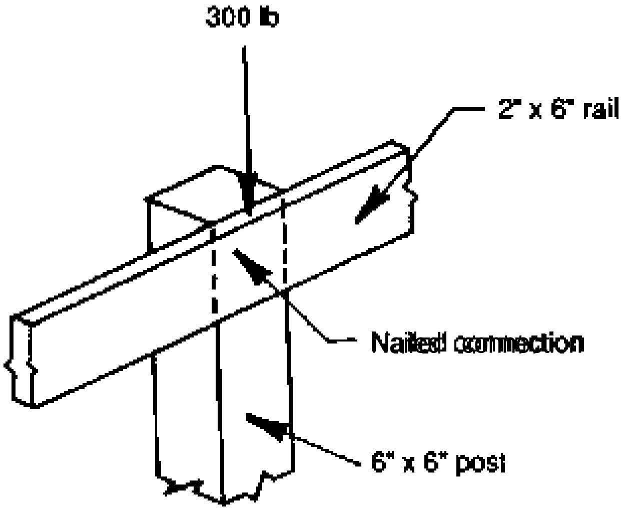
Nails and spikes are generally hand-driven but may be placed with power drivers for smaller diameters and lengths. They should be driven through the thinner member, into a thicker member, and be flush or countersunk to the member surface. Holes for large-diameter fasteners should be prebored to prevent the wood from splitting during placement. In such cases, the diameter of the lead hole must not exceed 0.90 times the fastener diameter for Group I species and 0.75 times the fastener diameter for Group II, III, and IV species. For deformed shanks, the diameter of the nail or spike may vary among types and manufacturers and should be verified before preboring lead holes.

Example 5-17 - Lateral nailed connection

A nominal 2-inch by 6-inch handrail is attached to a 6-inch by 6-inch post with common wire nails. The connection between the rail and post must be capable of resisting a downward force of 300 pounds. Determine the size and number of common wire nails that are required for the connection, assuming the following:

1. Members will be exposed to weathering (wet-use conditions) with a normal duration of load; adjustments for temperature *(C_t)* and fire-retardant treatment *(C_R)* are not required.

2. Lumber is surfaced Southern Pine.



Solution

In this connection, the rail is loaded perpendicular to grain while the post is loaded parallel to grain. For nailed connections, however, allowable loads are independent of load orientation to grain. The allowable load for one nail loaded in either direction is computed using Equation 5-52 with the applicable modification factors:

$$P_N' = P_N C_D C_M$$

The moisture modification factor for nailed connections is obtained from Table 5-18:

$$C_M = 0.75$$

Tabulated values for nails and spikes are given in the NDS (Table 5-32). From Table 5-14, Southern Pine is in Species Group II for nailed and spiked connections. To develop the full tabulated load in this species group, the nail must penetrate a minimum of 11 diameters *(11D)* into the member holding the point (reduced penetration requires reduced load). In this case, the nail length minus *11D* must not be less than the rail thickness of 1-1/2 inches. Using information from Table 5-32, nail sizes are evaluated to determine the minimum nail pennyweight for full penetration:

Pennyweight	Length (in.)	11*D* (in.)	Length – 11*D* (in.)
8d	2.5	1.44	1.06
10d	3	1.63	1.37
12d	3.25	1.63	1.62

A 12d nail is the minimum nail size that provides the required penetration for full load.

Substituting values for the allowable load on one nail,

$$P_N' = P_N C_D C_M = P_N(1.0)(0.75) = P_N(0.75)$$

Using tabulated values from Table 5-32 for nails 12d and larger, a table is compiled of allowable nail loads and the number of nails required:

Pennyweight	P_N (lb)	P_N' (lb)	# nails required
12d	94	70.5	4.3 = 5
16d	108	81.0	3.7 = 4
20d	139	104.3	2.9 = 3
30d	155	116.3	2.6 = 3

In any nailed connection it is desirable to use the minimum diameter and number of nails to minimize the potential for splitting. In this case, the 20d nails will be used because only three nails are required and the increase in diameter from 16d to 20d is small.

Summary
The connection will be made with three 20d nails for a connection capacity of 3(104.3 lb) = 313 lb.

DRIFT BOLTS AND DRIFT PINS

Drift pins and drift bolts are long, unthreaded steel rods that are driven in prebored holes for lateral connections in large timber members. Drift bolts have a head, for use with steel side plates and for convenience in driving, while drift pins have no head (Figure 5-33). In bridge applications, drift bolts and drift pins are used for connecting pile caps to timber piles or posts, or for attaching sawn lumber beams to their supporting cap or sill (Figure 5-34). Manufactured fasteners generally conform to ASTM A307, but pins of concrete reinforcing steel also are used. Because they have poor resistance in withdrawal, drift bolts and drift pins are not recommended for bridge connections subjected to significant withdrawal forces.

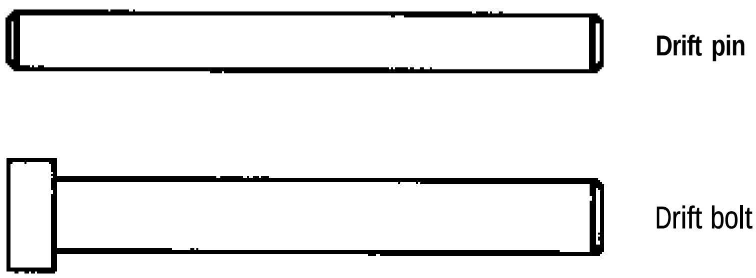


Figure 5-33. - Typical drift pin and drift bolt.

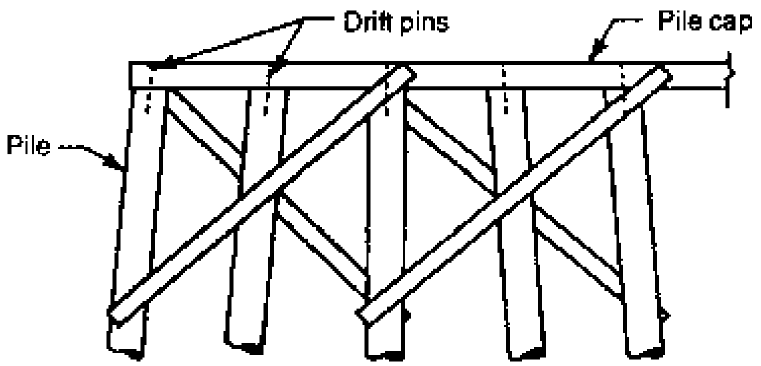


Figure 5-34. - Drift pins or drift bolts are normally used to connect large timber members such as a pile cap to piling.

There is little design information available on drift bolts or drift pins, and requirements for net area, end distance, edge distance, and spacing are taken to be the same as those for a bolt of the same diameter. The NDS specifies that lateral design values in wood side grain not exceed 75 percent of the design value for a comparable bolt of the same diameter and length in the main member. Fastener penetration is left to the judgment of the designer. Drift bolts and drift pins are driven in prebored holes that are 1/8 inch to 1/16 inch smaller in diameter than the fastener.

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