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20

Timber Bridges

Kenneth J. Fridley
Washington State University

20.1 Introduction

Wood is one of the earliest building materials, and as such often its use has been based more on tradition than on principles of engineering. However, the structural use of wood and wood-based materials has increased steadily in recent times, including a renewed interest in the use of timber as a bridge material. Supporting this renewed interest has been an evolution of our understanding of wood as a structural material and our ability to analyze and design safe, durable, and functional timber bridge structures.

An accurate and complete understanding of any material is key to its proper use in structural applications, and structural timber and other wood-based materials are no exception to this requirement. This chapter focuses on introducing the fundamental mechanical and physical properties of wood that govern its structural use in bridges. Following this introduction of basic material properties, a presentation of common timber bridge types will be made, along with a discussion of fundamental considerations for the design of timber bridges.

20.1.1 Timber as a Bridge Material

Wood has been widely used for short- and medium-span bridges. Although wood has the reputation of being a material that provides only limited service life, wood can provide long-standing and serviceable bridge structures when properly protected from moisture. For example, many covered bridges from the early 19th century still exist and are in use. Today, rather than protecting wood

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by a protective shelter as with the covered bridge of yesteryear, wood preservatives which inhibit moisture and biological attack have been used to extend the life of modern timber bridges.

As with any structural material, the use of wood must be based on a balance between its inherent advantages and disadvantages, as well as consideration of the advantages and disadvantages of other construction materials. Some of the advantages of wood as a bridge material include:

- Strength
- Light weight
- Constructibility
- Energy absorption
- Economics
- Durability, and
- Aesthetics

These advantages must be considered against the three primary disadvantages:

- Decay
- Insect attack, and
- Combustibility

Wood can withstand short-duration overloading with little or no residual effects. Wood bridges require no special equipment for construction and can be constructed in virtually any weather conditions without any negative effects. Wood is competitive with other structural materials in terms of both first costs and life-cycle costs. Wood is a naturally durable material resistant to freeze–thaw effects as well as deicing agents. Furthermore, large-size timbers provide good fire resistance as a result of natural charring. However, if inadequately protected against moisture, wood is susceptible to decay and biological attack. With proper detailing and the use of preservative treatments, the threat of decay and insects can be minimized. Finally, in many natural settings, wood bridges offer an aesthetically pleasing and unobtrusive option.

20.1.2 Past, Present, and Future of Timber Bridges

The first bridges built by humans were probably constructed with wood, and the use of wood in bridges continues today. As recently as a century ago, wood was still the dominant material used in bridge construction. Steel became an economical and popular choice for bridges in the early 1900s. Also during the early part of the 20th century, reinforced concrete became the primary bridge deck material and an economical choice for the bridge superstructure. However, important advances were made in wood fastening systems and preservative treatments, which would allow for future developments for timber bridges. Then, in the mid-20th century, glued-laminated timber (or glulams) was introduced as a viable structural material for bridges. The use of glulams grew to become the primary material for timber bridges and has continued to grow in popularity. Today, there is a renewed interest in all types of timber bridges. Approximately 8% (37,000) of the bridges listed in the National Bridge Inventory in the United States having spans greater than 6.10 m are constructed entirely of wood and 11% (51,000) use wood as one of the primary structural materials [9]. The future use of timber as a bridge material will not be restricted just to new construction. Owing to its high strength-to-weight ratio, timber is an ideal material for bridge rehabilitation of existing timber, steel, and concrete bridges.

20.2 Properties of Wood and Wood Products

It is important to understand the basic structure of wood in order to avoid many of the pitfalls relative to the misuse and/or misapplication of the material. Wood is a natural, cellular, anisotropic,

TABLE 20.1 Moisture Content (%) of Wood in Equilibrium with Temperature and Relative Humidity

Temp.	Relative Humidity (%)																			
(°C)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	98
0	1.4	2.6	3.7	4.6	5.5	6.3	7.1	7.9	8.7	9.5	10.4	11.3	12.4	13.5	14.9	16.5	18.5	21.0	24.3	26.9
5	1.4	2.6	3.7	4.6	5.5	6.3	7.1	7.9	8.7	9.5	10.4	11.3	12.3	13.5	14.9	16.5	18.5	21.0	24.3	26.9
10	1.4	2.6	3.6	4.6	5.5	6.3	7.1	7.9	8.7	'9.5	10.3	11.2	12.3	13.4	14.8	16.4	18.4	20.9	24.3	26.9
15	1.3	2.5	3.6	4.6	5.4	6.2	7.0	7.8	8.6	9.4	10.2	11.1	12.1	13.3	14.6	16.2	18.2	20.7	24.1	26.8
20	1.3	2.5	3.5	4.5	5.4	6.2	6.9	7.7	8.5	9.2	10.1	11.0	12.0	13.1	14.4	16.0	17.9	20.5	23.9	26.6
25	1.3	2.4	3.5	4.4	5.3	6.1	6.8	7.6	8.3	9.1	9.9	10.8	11.7	12.9	14.2	15.7	17.7	20.2	23.6	26.3
30	1.2	2.3	3.4	4.3	5.1	5.9	6.7	7.4	8.1	8.9	9.7	10.5	11.5	12.6	13.9	15.4	17.3	20.8	23.3	26.0
35	1.2	2.3	3.3	4.2	5.0	5.8	6.5	7.2	7.9	8.7	9.5	10.3	11.2	12.3	13.6	15.1	17.0	20.5	22.9	25.6
40	1.1	2.2	3.2	4.1	5.0	5.7	6.4	7.1	7.8	8.6	9.3	10.1	11.1	12.2	13.4	14.9	16.8	20.3	22.7	25.4
45	1.1	2.2	3.2	4.0	4.9	5.6	6.3	7.0	7.7	8.4	9.2	10.0	11.0	12.0	13.2	14.7	16.6	20.1	22.4	25.2
50	1.1	2.1	3.0	3.9	4.7	5.4	6.1	6.8	7.5	8.2	8.9	9.7	10.6	11.7	12.9	14.4	16.2	18.6	22.0	24.7
55	1.0	2.0	2.9	3.7	4.5	5.2	5.9	6.6	7.2	7.9	8.7	9.4	10.3	11.3	12.5	14.0	15.8	18.2	21.5	24.2

Adapted from USDA, 1987 [10].

hygrothermal, and viscoelastic material, and by its natural origins contains a multitude of inclusions and other defects.* The reader is referred to basic texts that present a description of the fundamental structure and physical properties of wood as a material [e.g., Refs. 5, 6, 10].

20.2.1 Physical Properties of Wood

One physical aspect of wood that deserves attention here is the effect of moisture on the physical and mechanical properties and performance of wood. Many problems encountered with wood structures, especially bridges, can be traced to moisture. The amount of moisture present in wood is described by the moisture content (MC), which is defined by the weight of the water contained in the wood as a percentage of the weight of the oven-dry wood. As wood is dried, water is first evaporated from the cell cavities, then, as drying continues, water from the cell walls is drawn out. The point at which *free* water in the cell cavities is completely evaporated, but the cell walls are still saturated, is termed the *fiber saturation point* (FSP). The FSP is quite variable among and within species, but is on the order of 24% to 34%. The FSP is an important quantity since most physical and mechanical properties are dependent on changes in MC below the FSP, and the MC of wood in typical structural applications is below the FSP. Finally, wood releases and absorbs moisture to and from the surrounding environment. When the wood equilibrates with the environment and moisture is not transferring to or from the material, the wood is said to have reached its equilibrium moisture content (EMC). Table 20.1 provides the average EMC as a function of dry-bulb temperature and relative humidity. The *Wood Handbook* [10] provides other tables that are specific for given species or species groups and allow designers better estimates of in-service moisture contents that are required for their design calculations.

Wood shrinks and swells as its MC changes below the FSP; above the FSP, shrinkage and swelling can be neglected. Wood machined to a specified size at an MC higher than that expected in service will therefore shrink to a smaller size in use. Conversely, if the wood is machined at an MC lower than that expected in service, it will swell. Either way, shrinkage and swelling due to changes in MC must be taken into account in design. In general, the shrinkage along the grain is significantly less than that across the grain. For example, as a rule of thumb, a 1% dimensional

*The term *defect* may be misleading. Knots, grain characteristics (e.g., slope of grain, spiral grain, etc.), and other naturally occurring irregularities do reduce the effective strength of the member, but are accounted for in the grading process and in the assignment of design values. On the other hand, splits, checks, dimensional warping, etc. are the result of the drying process and, although they are accounted for in the grading process, they may occur after grading and may be more accurately termed *defects*.

change across the grain can be assumed for each 4% change in MC, whereas a 0.02% dimensional change in the longitudinal direction may be assumed for each 4% change in MC. More-accurate estimates of dimensional changes can be made using published values of shrinkage coefficients for various species, [10].

In addition to simple linear dimensional changes in wood, drying of wood can cause warp of various types. Bow (distortion in the weak direction), crook (distortion in the strong direction), twist (rotational distortion), and cup (cross-sectional distortion similar to bow) are common forms of warp and, when excessive, can adversely affect the structural use of the member. Finally, drying stresses (internal stress resulting from differential shrinkage) can be quite significant and can lead to checking (cracks formed along the growth rings) and splitting (cracks formed across the growth rings).

20.2.2 Mechanical Properties of Wood

The mechanical properties of wood also are functions of the MC. Above the FSP, most properties are invariant with changes in MC, but most properties are highly affected by changes in the MC below the FPS. For example, the modulus of rupture of wood increases by nearly 4% for a 1% decrease in moisture content below the FSP. The following equation is a general expression for relating any mechanical property to MC:

$$P_{\mathrm{MC}} = P_{12}\left(\frac{P_{12}}{P_g}\right)^{(12-\mathrm{MC})/(\mathrm{FSP}-\mathrm{MC})} \tag{20.1}$$

where P_{MC} = property of interest at any MC below the FSP, P_{12} = the property at 12% MC, and P_g = property in the green condition (at FSP).

For structural design purposes, using an equation such as (20.1) would be cumbersome. Therefore, design values are typically provided for a specific maximum MC (e.g., 19%) and adjustments are made for "wet use."

Load history can also have a significant effect on the mechanical performance of wood members. The load that causes failure is a function of the rate and duration of the load applied to the member. That is, a member can resist higher magnitude loads for shorter durations or, stated differently, the longer a load is applied, the less able is a wood member to resist that load. This response is termed *load duration effects* in wood design. Figure 20.1 illustrates this effect by plotting the time-to-failure as a function of the applied stress expressed in terms of the short-term (static) ultimate strength. There are many theoretical models proposed to represent this response, but the line shown in Figure 20.1 was developed at the U.S. Forest Products Laboratory in the early 1950s [11] and is the basis for current design "load duration" adjustment factors.

The design factors derived from the relationship illustrated in Figure 20.1 are appropriate only for stresses and not for stiffness or, more precisely, the modulus of elasticity. Related to load duration effects, the deflection of a wood member under sustained load increases over time. This response, termed *creep effects,* must be considered in design when deformation or deflections are critical from either a safety or serviceability standpoint. The main parameters that significantly affect the creep response of wood are stress level, moisture content, and temperature. In broad terms, a 50% increase in deflection after a year or two is expected in most situations, but can easily be upward of 200% given certain conditions [7]. In fact, if a member is subjected to continuous moisture cycling, a 100 to 150% increase in deflection could occur in a matter of a few weeks. Unfortunately, the creep response of wood, especially considering the effects of moisture cycling, is poorly understood and little guidance is available to the designer.

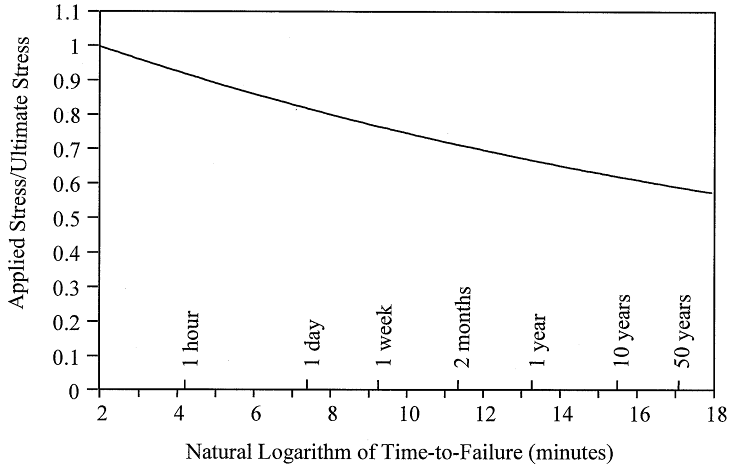


FIGURE 20.1 Load Duration behavior of wood.

Wood, being a fibrous material, is naturally resistant to fatigue effects, particularly when stressed along the grain. However, the fatigue strength of wood is negatively affected by the natural presence of inclusions and other defects. Knots and slope of grain in particular reduce fatigue resistance. Regardless of this, wood performs well in comparison with structural steel and concrete. In fact, the fatigue strength of wood has been shown to be approximately double that of most metals when evaluated at comparable stress levels relative to the ultimate strength of the material [10]. The potential for fatigue-induced failure is considered to be rather low for wood, and thus fatigue is typically not considered in timber bridge design.

20.2.3 Wood and Wood-Based Materials for Bridge Construction

The natural form of timber is the log. In fact, many primitive and "rustic" timber bridges are nothing more than one or more logs tied together. For construction purposes, however, it is simpler to use rectangular elements in bridges and other structures rather than round logs. Solid sawn lumber is cut from logs and was the mainstay of timber bridge construction for years. Solid sawn lumber comes in a variety of sizes including boards (less than 38 mm thick and 38 to 387 mm wide), dimension lumber (38 to 89 mm thick and 38 to 387 mm wide), and timbers (anything greater than 89 by 89 mm). Based on size and species, solid sawn lumber is graded by various means, including visual grading, machine-evaluated lumber (MEL), and machine stress rated (MSR), and engineering design values are assigned.

In the mid-1900s glulam timber began to receive significant use in bridges. Glulams are simply large sections formed by laminating dimension lumber together. Sections as large as 1.5 m deep are feasible with glulams. Today, while solid sawn lumber is still used extensively, the changing resource base and shift to plantation-grown trees has limited the size and quality of the raw material. Therefore, it is becoming increasingly difficult to obtain high-quality, large-dimension timbers for construction. This change in raw material, along with a demand for stronger and more cost-effective material, initiated the development of alternative products that can replace solid lumber such as glulams.

Other engineered products such as wood composite I-joists and structural composite lumber (SCL) also resulted from this evolution. SCL includes such products as laminated veneer lumber (LVL) and parallel strand lumber (PSL). These products have steadily gained popularity and now are receiving widespread use in building construction, and they are beginning to find their way into bridge construction as well. The future may see expanded use of these and other engineered wood composites.

20.2.4 Preservation and Protection

As mentioned previously, one of the major advances in the 20th century allowing for continued and expanded use of timber as a bridge material is pressure treatment. Two basic types of wood preservatives are used: oil-type preservatives and waterborne preservatives. Oil-type preservatives include creosote, pentachlorophenol (or "penta"), and copper naphthenate. Creosote can be considered the first effective wood preservative and has a long history of satisfactory performance. Creosote also offers protection against checking and splitting caused by changes in MC. While creosote is a natural by-product from coal tar, penta is a synthetic pesticide. Penta is an effective preservative treatment; however, it is not effective against marine borers and is not used in marine environments. Penta is a "restricted-use" chemical, but wood treated with penta is not restricted. Copper naphthenate has received recent attention as a preservative treatment, primarily because it is considered an environmentally safe chemical while still giving satisfactory protection against biological attack. Its primary drawback is its high cost relative to other treatments. All these treatments generally leave the surface of the treated member with an oily and unfinishable surface. Furthermore, the member may "bleed" or leach preservative unless appropriate measures are taken.

Most timber bridge applications utilize oil-type preservatives for structural elements such as beams, decks, piles, etc. They offer excellent protection against decay and biological attack, are noncorrosive, and are relatively durable. Oil-type preservatives are not, however, recommended for bridge elements that may have frequent or repeated contact by humans or animals since they can cause skin irritations.

Waterborne preservatives have the advantage of leaving the surface of the treated material clean and, after drying, able to be painted or stained. They also do not cause skin irritations and, therefore, can be used where repeated human and/or animal contact is expected. Waterborne preservatives use formulations of inorganic arsenic compounds in a water solution. They do, however, leave the material with a light green, gray, or brownish color. But again, the surface can be later painted or stained. A wide variety of waterborne preservatives are available, but the most common include chromated copper arsenate (CCA), ammoniacal copper arsenate (ACA), and ammoniacal copper zinc arsenate (ACZA). Leaching of these chemicals is not a problem with these formulations since they each are strongly bound to the wood. CCA is commonly used to treat southern pine, ponderosa pine, and red pine, all of which are relatively accepting of treatment. ACA and ACZA are used with species that are more difficult to treat, such as Douglas fir and larch. One potential drawback to CCA and ACA is a tendency to be corrosive to galvanized hardware. The extent to which this is a problem is a function of the wood species, the specific preservative formulation, and service conditions. However, such corrosion seems not to be an issue for hot-dipped galvanized hardware typical in bridge applications.

Waterborne preservatives are used for timber bridges in applications where repeated or frequent contact with humans or animals is expected. Such examples include handrails and decks for pedestrian bridges. Additionally, waterborne preservatives are often used in marine applications where marine borer hazards are high.

Any time a material is altered due to chemical treatment its microlevel structure may be affected, thus affecting its mechanical properties. Oil-type preservatives do not react with the cellular structure of the wood and, therefore, have little to no effect on the mechanical properties of the material. Waterborne preservatives do react, however, with the cell material, thus they can affect properties. Although this is an area of ongoing research, indications are that the only apparent effect of waterborne preservatives is to increase load duration effects, especially when heavy treatment is used for saltwater applications. Currently, no adjustments are recommended for design values of preservative treated wood vs. untreated materials.

In addition to preservative treatment, fire-retardant chemical treatment is also possible to inhibit combustion of the material. These chemicals react with the cellular structure in wood and can cause significant reductions in the mechanical properties of the material, including strength. Generally, fire retardants are not used in bridge applications. However, if fire-retardant-treated material is used, the designer should consult with the material producer or treater to obtain appropriate design values.

20.3 Types of Timber Bridges

Timber bridges come in a variety of forms, many having evolved from tradition. Most timber bridges designed today, however, are the results of fairly recent developments and advances in the processing and treating of structural wood. The typical timber bridge is a single- or two-span structure. Single-span timber bridges are typically constructed with beams and a transverse deck or a slab-type longitudinal deck. Two-span timber bridges are often beam with transverse decks. These and other common timber bridge types are presented in this section.

20.3.1 Superstructures

As with any bridge, the structural makeup can be divided into three basic components: the superstructure, the deck, and the substructure. Timber bridge superstructures can be further classified into six basic types: beam superstructures, longitudinal deck (or slab) superstructures, trussed superstructures, trestles, suspension bridges, and glulam arches.

Beam Superstructures

The most basic form of a timber beam bridge is a log bridge. It is simply a bridge wherein logs are laid alternately tip-to-butt and bound together. A transverse deck is then laid over the log beams. Obviously, spans of this type of bridge are limited to the size of logs available, but spans of 6 to 18 m are reasonable. The service life of a log bridge is typically 10 to 20 years.

The sawn lumber beam bridge is another simple form. Typically, made of closely spaced 100 to 200-mm-wide by 300 to 450-mm-deep beams, sawn lumber beams are usually used for clear spans up to 9 m. With the appropriate use of preservative treatments, sawn lumber bridges have average service lives of approximately 40 years. A new alternative to sawn lumber is structural composite lumber (SCL) bridges. Primarily, laminated veneer lumber (LVL) has been used in replacement of solid sawn lumber in bridges. LVL can be effectively treated and can offer long service as well.

Glulam timber beam bridges are perhaps the most prevalent forms of timber bridges today. A typical glulam bridge configuration is illustrated in Figure 20.2. This popularity is primarily due to the large variety of member sizes offered by glulams. Commonly used for clear spans ranging from 6 to 24 m, glulam beam bridges have been used for clear spans up to 45 m. Transportation restrictions rather than material limitations limit the length of beams, and, therefore, bridges. Since glulam timber can be satisfactorily treated with preservatives, they offer a durable and long-lasting structural element. When designed such that field cutting, drilling, and boring are avoided, glulam bridges can provide a service life of at least 50 years.

Longitudinal Deck Superstructures

Longitudinal deck (or slab) superstructures are typically either glulam or nail-laminated timber placed longitudinally to span between supports. A relatively new concept in longitudinal deck systems is the stress-laminated timber bridge, which is similar to the previous two forms except that continuity in the system is developed through the use of high-strength steel tension rods. In any case, the wide faces of the laminations are oriented vertically rather than horizontally as in a typical glulam beam. Figure 20.3 illustrates two types of glulam longitudinal decks: noninterconnected and interconnected. Since glulam timbers have depths typically less than the width of a bridge, two or more segments must be used. When continuity is needed, shear dowels must be used to provide interconnection between slabs. When continuity is not required, construction is simplified. Figure 20.4 illustrates a typical stress-laminated section.

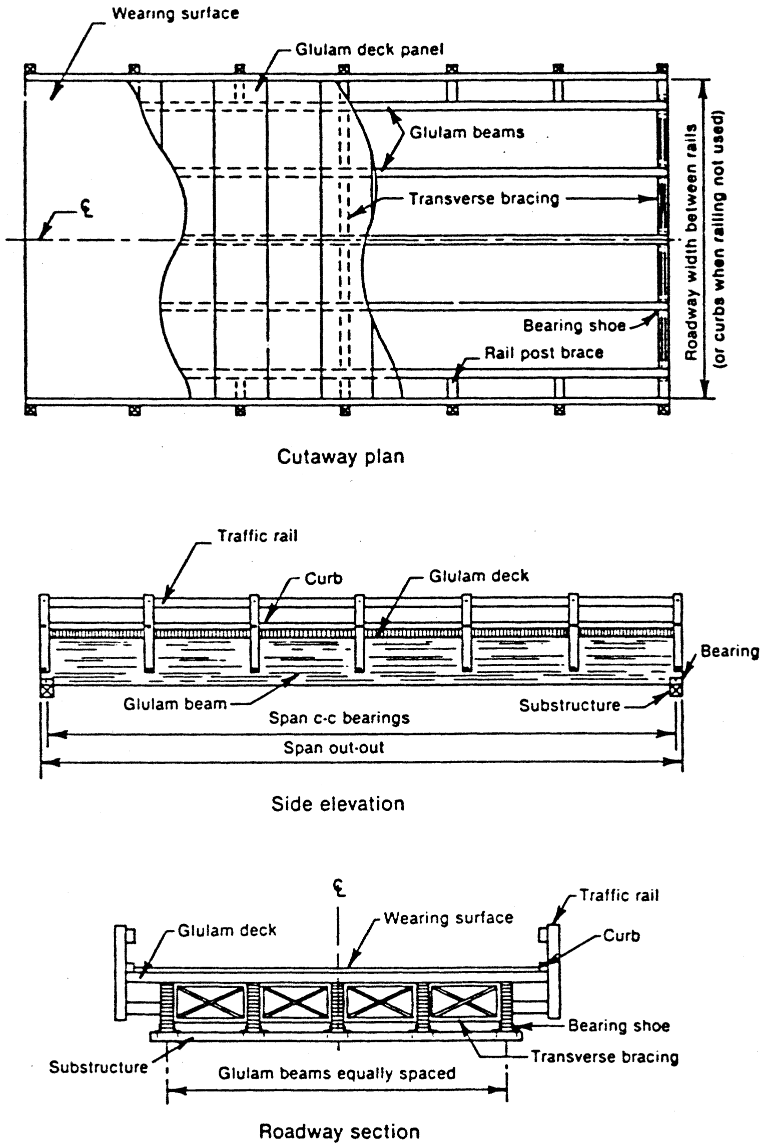


FIGURE 20.2 Glulam beam bridge with transverse deck. (*Source*: Ritter, M.A., EM7700-8, USDA Forest Service, Washington, D.C., 1990.)

Longitudinal deck systems are relatively simple and offer a relatively low profile, making them an excellent choice when vertical clearance is a consideration. Longitudinal decks are economical choices for clear spans up to approximately 10 m. Since the material can be effectively treated, the average service life of a longitudinal timber deck superstructure is at least 50 years. However, proper maintenance is required to assure an adequate level of prestress is maintained in stress-laminated systems.

Trussed Superstructures

Timber trusses were used extensively for bridges in the first half of the 20th century. Many different truss configurations were used including king post, multiple king posts, Pratt, Howe, lattice, long, and bowstring trusses, to name a few. Clear spans of up to 75 m were possible. However, their

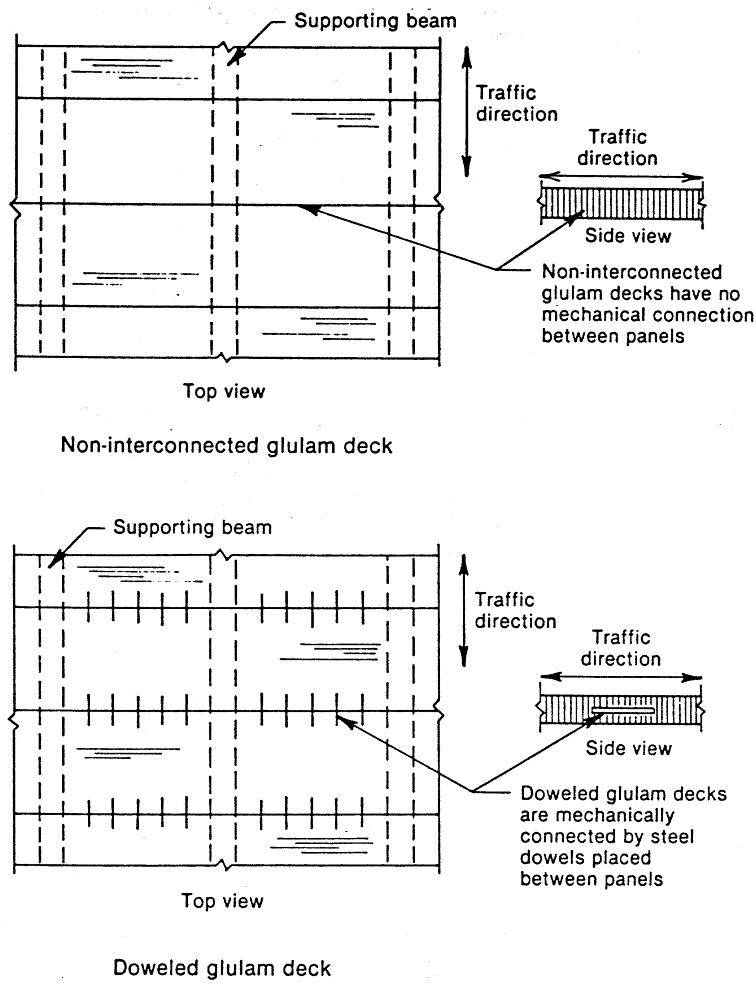


FIGURE 20.3 Glulam longitudinal decks. (*Source*: Ritter, M.A., EM7700-8, USDA Forest Service, Washington, D.C., 1990.)

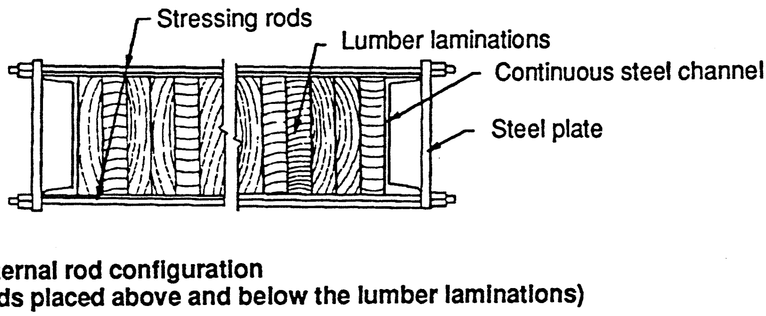
use has declined due primarily to high fabrication, erection, and maintenance costs. When timber trusses are used today, it is typically driven more by aesthetics than by structural performance or economics.

Trestles

Another form of timber bridge which saw its peak usage in the first half of the 20th century was the trestle. A trestle is a series of short-span timber superstructures supported on a series of closely spaced timber bents. During the railroad expansion during the early to mid 1900s, timber trestles were a popular choice. However, their use has all but ceased because of high fabrication, erection, and maintenance costs.

Suspension Bridges

A timber suspension bridge is simply a timber deck structure supported by steel cables. Timber towers, in turn, support the steel suspension cables. Although there are examples of vehicular timber suspension bridges, the more common use of this form of timber bridge is as a pedestrian bridge. They are typically used for relatively long clear spans, upward of 150 m. Since treated wood can be used throughout, 50-year service lives are expected.



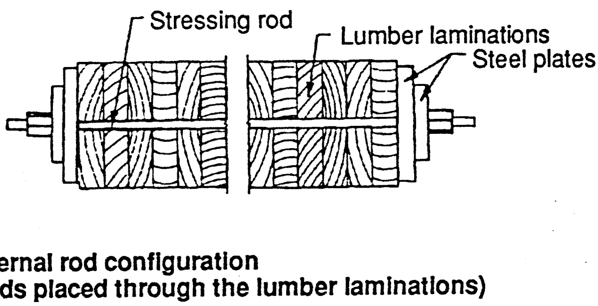


FIGURE 20.4 Stress laminated bridge. (*Source*: Ritter, M.A., EM7700-8, USDA Forest Service, Washington, D.C., 1990.)

Glued Laminated Arches

One of the most picturesque forms of timber bridges is perhaps the glulam arch. Constructed from segmented circular or parabolic glulam arches, either two- or three-hinge arches are used. The glulam arch bridge can have clear spans in excess of 60 m, and since glulam timber can be effectively treated, service lives of at least 50 years are well within reason. Although the relative first and life-cycle costs of arch bridges have become high, they are still a popular choice when aesthetics is an issue.

20.3.2 Timber Decks

The deck serves two primary purposes: (1) it is the part of the bridge structure that forms the roadway, and (2) it distributes the vehicular loads to the supporting elements of the superstructure. Four basic types of timber decks are sawn lumber planks, nailed laminated decks, glulam decks, and composite timber–concrete decks. The selection of a deck type depends mainly on the level of load demand.

Lumber Planks

The lumber plank deck is perhaps the simplest deck type. It is basically sawn lumber, typically 75 to 150 mm thick and 250 to 300 mm wide, placed flatwise and attached to the supporting beams with large spikes. Generally, the planks are laid transverse to the beams and traffic flow, but can be placed longitudinally on cross beams as well. Lumber planks are only used for low-volume bridges. They are also of little use when protection of the supporting members is desired since water freely travels between adjacent planks. Additionally, when a wearing surface such as asphalt is desired, lumber planks are not recommended since deflections between adjacent planks will result in cracking and deterioration of the wearing surface.

Nailed Laminated and Glulam Decks

Nailed laminated and glulam decks are essentially as described previously for longitudinal deck (or slab) superstructures. Nailed laminated systems are typically 38-mm-thick by 89- to 285-mm-deep lumber placed side by side and nailed or spiked together along its length. The entire deck is nailed together to act as a composite section and oriented such that the lumber is laid transverse to the bridge span across the main supporting beams, which are spaced from 0.6 to 1.8 m. Once a quite popular deck system, its use has declined considerably in favor of glulam decks.

A glulam deck is a series of laminated panels, typically 130- to 220-mm thick by 0.9 to 1.5 m wide. The laminations of the glulam panel are oriented with their wide face vertically. Glulam decks can be used with the panels in the transverse or longitudinal direction. They tend to be stronger and stiffer than nailed laminated systems and offer greater protection from moisture to the supporting members. Finally, although doweled glulam panels (see Figure 20.3) cost more to fabricate, they offer the greatest amount of continuity. With this continuity, thinner decks can be used, and improved performance of the wearing surface is achieved due to reduced cracking and deterioration.

Composite Timber–Concrete Decks

The two basic types of composite timber–concrete deck systems are the T-section and the slab (see Figure 20.5). The T-section is simply a timber stem, typically a glulam, with a concrete flange that also serves as the bridge deck. Shear dowels are plates that are driven into the top of the timber stem and develop the needed shear transfer. For a conventional single-span bridge, the concrete is proportioned such that it takes all the compression force while the timber resists the tension. Composite T-sections have seen some use in recent years; however, high fabrication costs have limited their use.

Composite timber–concrete slabs were used considerably during the second quarter of the 20th century, but receive little use today. They are constructed with alternating depths of lumber typically nailed laminated with a concrete slab poured directly on top of the timber slab. With a simple single span, the concrete again carries the compressive flexural stresses while the timber carries the flexural stresses. Shear dowels or plates are driven into the timber slab to provide the required shear transfer between the concrete and the timber.

20.3.3 Substructures

The substructure supports the bridge superstructure. Loads transferred from the superstructures to the substructures are, in turn, transmitted to the supporting soil or rock. Specific types of substructures that can be used are dependent on a number of variables, including bridge loads, soil and site conditions, etc. Although a timber bridge superstructure can be adapted to virtually any type of substructure regardless of material, the following presentation is focused on timber substructures, specifically timber abutments and bents.

Abutments

Abutments serve the dual purpose of supporting the bridge superstructure and the embankment. The simplest form of a timber abutment is a log, sawn lumber, or glulam placed directly on the embankment as a spread footing. However, this form is not satisfactory for any structurally demanding situation. A more common timber abutment is the timber pile abutment. Timber piles are driven to provide the proper level of load-carrying capacity through either end bearing or friction. A backwall and wing walls are commonly added using solid sawn lumber to retain the embankment. A continuous cap beam is connected to the top of the piles on which the bridge superstructure is supported. A timber post abutment can be considered a hybrid between the spread footing and pile abutment. Timber posts are supported by a spread footing, and a backwall and wing walls are added to retain the embankment. Pile abutments are required when soil conditions do not provide adequate support for a spread footing or when uplift is a design concern.

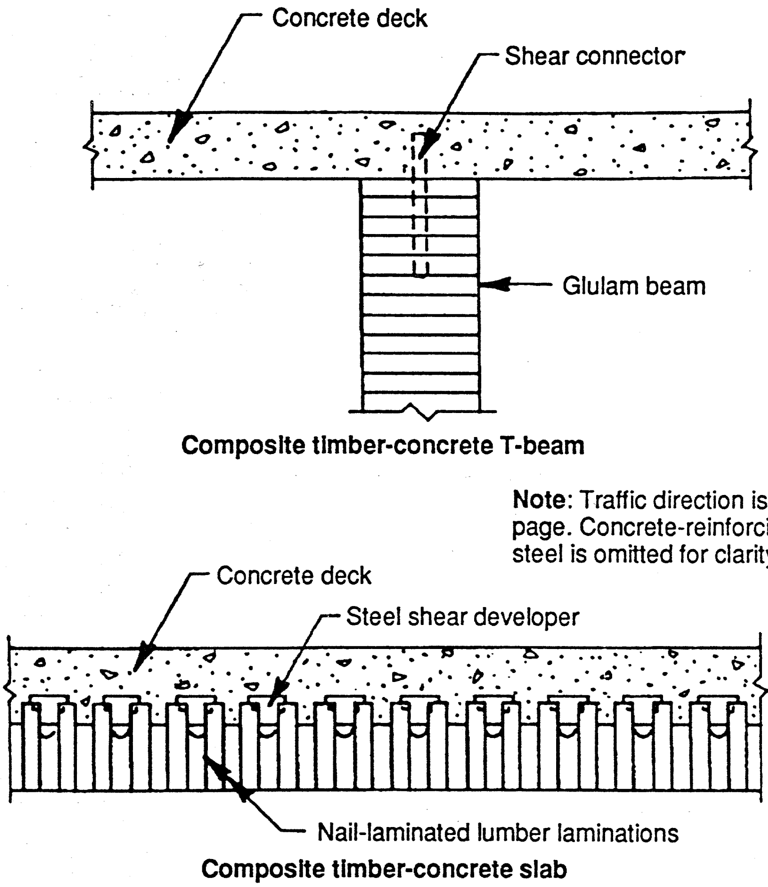


FIGURE 20.5 Composite timber–concrete decks. (*Source*: Ritter, M.A., EM7700-8, USDA Forest Service, Washington, D.C., 1990.)

Bents

Bents are support systems used for multispan bridges between the abutments. Essentially, timber bents are formed from a set of timber piles with lumber cross bracing. However, when the height of the bent exceeds that available for a pile, frame bents are used. Frame bents were quite common in the early days of the railroad, but, due to high cost of fabrication and maintenance, they are not used often for new bridges.

20.4 Basic Design Concepts

In this section, the basic design considerations and concepts for timber bridges are presented. The discussion should be considered an overview of the design process for timber bridges, not a replacement for specifications or standards.

20.4.1 Specifications and Standards

The design of timber bridge systems has evolved over time from what was tradition and essentially a "master-builder" approach. Design manuals and specifications are available for use by engineers involved with or interested in timber bridge design. These include *Timber Bridges: Design, Construction, Inspection, and Maintenance* [8], AASHTO *LRFD Bridge Design Specifications* [1], and AASHTO

Standard Specifications for Highway Bridges [2]. The wood industry, through the American Forest and Paper Association (AF&PA), published design values for solid sawn lumber and glulam timber for both allowable stress design [4] and load and resistance factor design [3] formats. Rather than presenting those aspects of bridge design common to all bridge types, the focus of the following presentation will be on those aspects specific to timber bridge design. Since bridge design is often governed by AASHTO, focus will be on AASHTO specifications. However, AF&PA is the association overseeing the engineering design of wood, much like ACI is for concrete and AISC is for steel, and AF&PA-recommended design procedures will also be presented.

20.4.2 Design Values

Design values for wood are provided in a number of sources, including AF&PA specifications and AASHTO specifications. Although the design values published by these sources are based on the same procedures per ASTM standards, specific values differ due to assumptions made for end-use conditions. The designer must take care to use the appropriate design values with their intended design specification(s). For example, the design should not use AF&PA design values directly in AASHTO design procedures since AF&PA and AASHTO make different end-use assumptions.

AF&PA "Reference" Design Values

The AF&PA *Manual for Engineered Wood Construction*: *Load and Resistance Factor Design* [3] provides nominal design values for visually and mechanically graded lumber, glulam timber, and connections. These values include reference bending strength, F_b; reference tensile strength parallel to the grain, F_t; reference shear strength parallel to the grain, F_v; reference compressive strength parallel and perpendicular to the grain, F_c and $F_{c\perp}$, respectively; reference bearing strength parallel to the grain, F_g; and reference modulus of elasticity, *E*. These are appropriate for use with the LRFD provisions.

Similarly, the Supplement to the NDS® provides tables of design values for visually graded and machine stress rated lumber, and glulam timber for use in allowable stress design (ASD). The basic quantities are the same as with the LRFD, but are in the form of allowable stresses and are appropriate for use with the ASD provisions of the NDS· Additionally, the NDS provides tabulated allowable design values for many types of mechanical connections.

One main difference between the ASD and LRFD design values, other than the ASD prescribing allowable stresses and the LRFD prescribing nominal strengths, is the treatment of duration of load effects. Allowable stresses (except compression perpendicular to the grain) are tabulated in the NDS and elsewhere for an assumed 10-year load duration in recognition of the duration of load effect discussed previously. The allowable compressive stress perpendicular to the grain is not adjusted since a deformation definition of failure is used for this mode rather than fracture as in all other modes; thus the adjustment has been assumed unnecessary. Similarly, the modulus of elasticity is not adjusted to a 10-year duration since the adjustment is defined for strength, not stiffness. For the LRFD, short-term (i.e., 20 min) nominal strengths are tabulated for all strength values. In the LRFD, design strengths are reduced for longer-duration design loads based on the load combination being considered. Conversely, in the NDS, allowable stresses are increased for shorter load durations and decreased only for permanent (i.e., greater than 10 year) loading.

AASHTO-LRFD "Base" Design Values

AASHTO-LRFD publishes its own design values which are different from those of the AF&PA LRFD. AASHTO publishes base bending strength, F_{b0}; base tensile strength parallel to the grain, F_{t0}; base shear strength parallel to the grain, F_{v0}; base compressive strength parallel and perpendicular to the grain, F_{c0} and $F_{c\perp}$, respectively; and base modulus of elasticity, E_0. While the NDS publishes design values based on an assumed 10-year load duration and the AF&PA LRFD assumes a short-term (20-min) load duration, AASHTO publishes design values based on an assumed 2-month duration.

TABLE 20.2 Factors to Convert NDS-ASD Values to AASTHO-LRFD Values

	Property					
Material	F_b	F_v	F_c	$F_{c\perp}$	F_t	E
Dimension lumber	2.35	3.05	1.90	1.75	2.95	0.90
Beams and stringers, posts and timbers	2.80	3.15	2.40	1.75	2.95	1.00
Glulam	2.20	2.75	1.90	1.35	2.35	0.83

Unfortunately, the AASHTO published design values are not as comprehensive (with respect to species, grades, sizes, as well as specific properties) as thsoe of AF&PA. The AASHTO-LRFD does, however, provide for adjustments from AF&PA-published reference design values so they can be used in AASHTO specifications. For design values not provided in the AASHTO-LRFD, conversion factors are provided from NDS allowable stresses to AASHTO-LRFD base strengths. Table 20.2 provides these adjustments for solid sawn and glulam timbers. The designer is cautioned that these conversion factors are from the NDS allowable stresses, *not* the AF&PA-LRFD strength values.

20.4.3 Adjustment of Design Values

In addition to the providing *reference* or *base* design values, the AF&PA-LRFD, the NDS, and the AASHTO-LRFD specifications provide adjustment factors to determine final *adjusted* design values. Factors to be considered include load duration (termed *time effect* in the LRFD), wet service, temperature, stability, size, volume, repetitive use, curvature, orientation (form), and bearing area. Each of these factors will be discussed further; however, it is important to note that not all factors are applicable to all design values, nor are all factors included in all the design specifications. The designer must take care to apply the appropriate factors properly.

AF&PA Adjustment Factors

LRFD reference strengths and ASD allowable stresses are based on the following specified reference conditions: (1) dry use in which the maximum EMC does not exceed 19% for solid wood and 16% for glued wood products; (2) continuous temperatures up to 32°C, occasional temperatures up to 65°C (or briefly exceeding 93°C for structural-use panels); (3) untreated (except for poles and piles); (4) new material, not reused or recycled material; and (5) single members without load sharing or composite action. To adjust the reference design value for other conditions, adjustment factors are provided which are applied to the published reference design value:

$$R' = R \cdot C_1 \cdot C_2 \cdots C_n \tag{20.2}$$

where R' = adjusted design value (resistance), R = reference design value, and C_1, C_2, … C_n = applicable adjustment factors. Adjustment factors, for the most part, are common between LRFD and ASD. Many factors are functions of the type, grade, and/or species of material while other factors are common to all species and grades. For solid sawn lumber, glulam timber, piles, and connections, adjustment factors are provided in the AF&PA LRFD manual and the NDS. For both LRFD and ASD, numerous factors need to be considered, including wet service, temperature, preservative treatment, fire-retardant treatment, composite action, load sharing (repetitive use), size, beam stability, column stability, bearing area, form (i.e., shape), time effect (load duration), etc. Many of these factors will be discussed as they pertain to specific designs; however, some of the factors are unique for specific applications and will not be discussed further. The four factors that are applied to all design properties are the wet service factor, C_M; temperature factor, C_t; preservative treatment factor, C_{pt}; and fire-retardant treatment factor, C_{rt}. Individual treaters provide the two treatment factors, but the wet service and temperature factors are provided in the AF&PA LRFD Manual. For example, when considering the design of solid sawn lumber members, the adjustment

TABLE 20.3 AF&PA LRFD Wet Service Adjustment Factors, C_M

	Size Adjusted[a] F_b			Size Adjusted[a] F_c				
Thickness	≤20 MPa	>20 MPa	F_t	≤12.4 MPa	>12.4 MPa	F_v	$F_{c\perp}$	E, E_{05}
≤90 mm	1.00	0.85	1.00	1.00	0.80	0.97	0.67	0.90
>90 mm	1.00	1.00	1.00	0.91	0.91	1.00	0.67	1.00

[a] Reference value adjusted for size only.

TABLE 20.4 AF&PA-LRFD Temperature Adjustment Factors, C_t

	Dry Use		Wet Use	
Sustained Temperature (°C)	E, E_{05}	All Other Prop.	E, E_{05}	All Other Prop.
32 < T ≤ 48	0.9	0.8	0.9	0.7
48 < T ≤ 65	0.9	0.7	0.9	0.5

values given in Table 20.3 for wet service, which is defined as the maximum EMC exceeding 19%, and Table 20.4 for temperature, which is applicable when continuous temperatures exceed 32°C, are applicable to all design values. Often with bridges, since they are essentially exposed structures, the MC will be expected to exceed 19%. Similarly, temperature may be a concern, but not as commonly as MC.

Since, as discussed, LRFD and ASD handle time (duration of load) effects so differently and since duration of load effects are somewhat unique to wood design, it is appropriate to elaborate on it here. Whether using ASD or LRFD, a wood structure is designed to resist all appropriate load combinations – unfactored combinations for ASD and factored combinations for LRFD. The time effects (LRFD) and load duration (ASD) factors are meant to recognize the fact that the failure of wood is governed by a creep–rupture mechanism; that is, a wood member may fail at a load less than its short-term strength if that load is held for an extended period of time. In the LRFD, the time effect factor, λ, is based on the load combination being considered. In ASD, the load duration factor, C_D, is given in terms of the assumed cumulative duration of the design load.

AASHTO-LRFD Adjustment Factors

AASHTO-LRFD base design values are based on the following specified reference conditions: (1) wet use in which the maximum EMC exceeds 19% for solid wood and 16% for glued wood products (this is opposite from the dry use assumed by AF&PA, since typical bridge use implies wet use); (2) continuous temperatures up to 32°C, occasional temperatures up to 65°C; (3) untreated (except for poles and piles); (4) new material, not reused or recycled material; and (5) single members without load sharing or composite action. AASHTO has fewer adjustments available for the designer to consider, primarily but not entirely due to the specific application. To adjust the base design value for other conditions, AASHTO-LRFD provides the following adjustment equation:

$$F = F_0 \cdot C_F \cdot C_M \cdot C_D \tag{20.3}$$

where F = adjusted design value (resistance), F_0 = base design value, C_F = size adjustment factor, C_M = moisture content adjustment factor, C_D = deck adjustment factor, and C_S = stability adjustment factor.

The size factor is applicable only to bending and is essentially the same as that used by AF&PA for solid sawn lumber and the same as the volume effect factor used by AF&PA for glulam timber. For solid sawn lumber and vertically laminated lumber, the size factor is defined as

$$C_F = \left(\frac{300}{d}\right)^{1/9} \leq 1.0 \tag{20.4}$$

TABLE 20.5 AASHTO-LRFD Moisture Content Adjustment Factors, C_M, for Glulam

Property					
F_b	F_v	F_c	F_{cp}	F_t	E
1.25	1.15	1.35	1.90	1.25	1.20

where d = width (mm). The equation implies if lumber less than or equal to 300 mm in width is used, no adjustment is made. If, however, a width greater than 300 mm is used, a reduction in the published base bending design value is required. For horizontally glulam timber, the "size" factor may more appropriately be termed a *volume* factor (per the AF&PA). The size factor for glulam is given as

$$C_F = \left[\left(\frac{300}{d} \right) \left(\frac{130}{b} \right) \left(\frac{6400}{L} \right) \right]^a \leq 1.0 \tag{20.5}$$

where d = width (mm), b = thickness (mm), L = span (mm), and a = 0.05 for southern pine and 0.10 for all other species glulam. As with the previous size adjustment, if the dimensions of the glulam exceed 130 by 300 by 6400, then a reduction in the bending strength is required.

Unlike the size factor, the moisture factor is applicable to all published design values, not just bending strength. The moisture adjustment factor, C_M, is again similar to that provided by AF&PA; however, it is embedded in the published base design values. Unless otherwise noted, C_M should be assumed as unity. The only exception is when glulams are used and the moisture content is expected to be less than 16%. An increase in the design values is then allowed per Table 20.5. A similar increase is not allowed for lumber used at moisture contents less than 19% per AASHTO. This is a conservative approach in comparison with that of AF&PA.

The deck adjustment factor, C_D, is again specific for the bending resistance, F_b, of 50- to 100-mm-wide lumber used in stress-laminated and mechanically (nail or spike) laminated deck systems. For stress-laminated decks, the bending strength can be increased by a factor of C_D = 1.30 for select structural grade lumber, and C_D = 1.5 for No. 1 and No. 2 grade. For mechanically laminated decks, the bending strength of all grades can be increased by a factor of C_D = 1.15.

Since, as discussed, AF&PA LRFD and ASD handle time (duration of load) effects so differently and since duration of load effects are somewhat unique to wood design, it is appropriate to elaborate on it here and understand how time effects are accounted for by AASHTO-LRFD. Implicit in the AASHTO-LRFD Specification, λ = 0.8 is assumed for vehicle live loads. The published base design values are reduced by a factor of 0.80 to account for time effects. For strength load combination IV, however, a reduction of 75% is required. This load combination is for dead load only. The rationale behind this reduction is found in the AF&PA-LRFD time effects factors. For live-load-governed load combinations, AF&PA requires λ = 0.8; and for dead load only, λ = 0.6 is used. The ratio of the dead-load time effect factor to the live-load time effect factor is 0.6/0.8 = 0.75.

20.4.4 Beam Design

The focus of the remaining discussion will be on the design provisions specified in the AASHTO-LRFD for wood members. The design of wood beams follows traditional beam theory. The flexural strength of a beam is generally the primary concern in a beam design, but consideration of other factors such as horizontal shear, bearing, and deflection are also crucial for a successful design.

Moment Capacity

In terms of moment, the AASHTO-LRFD design factored resistance, M_r, is given by

$$M_r = \phi_b M_n = \phi_b F_b S C_s \tag{20.6}$$

where ϕ_b = resistance factor for bending = 0.85, M_n = nominal adjusted moment resistance, F_b = adjusted bending strength, S = section modulus, and C_s = beam stability factor.

The beam stability factor, C_s, is only used when considering strong axis bending since a beam oriented about its weak axis is not susceptible to lateral instability. Additionally, the beam stability factor need not exceed the value of the size effects factor. The beam stability factor is taken as 1.0 for members with continuous lateral bracing; otherwise C_s is calculated from

$$C_s = \frac{1+A}{1.9} - \sqrt{\frac{(1+A)^2}{3.61} - \frac{A}{0.95}} \le C_F \tag{20.7}$$

where

$$A = \frac{0.438\,EB^2}{L_e d F_b} \quad \text{for visually graded solid sawn lumber} \tag{20.8}$$

and

$$A = \frac{0.609\,Eb^2}{L_e d F_b} \quad \text{for mechanically graded lumber and glulams} \tag{20.9}$$

where E = modulus of elasticity, b = net thickness, d = net width, L_e = effective length, and F_b = adjusted bending strength. The effective length, L_e, accounts for both the lateral motion and torsional phenomena and is given in the AASHTO-LRFD specification for specific unbraced lengths, L_u, defined as the distance between points of lateral and rotations support. For $L_u/d < 7$, the effective unbraced length, $L_e = 2.06L_u$; for $7 \le L_u/d \le 14.3$, $L_e = 1.63L_u + 3d$; and for $l_u/d > 14.3$, $L_e = 1.84L_u$.

While the basic adjustment factor for beam stability is quite similar between AASHTO and AF&PA, the consideration of beam stability and size effects combined differs significantly from the approach used by AF&PA. For solid sawn lumber, AF&PA requires both the size factor and the beam stability factor apply. For glulams, AF&PA prescribes the lesser of the volume factor or the stability factor be used. AASHTO compared with AF&PA is potentially nonconservative with respect to lumber elements and conservative with respect to glulam elements.

Shear Capacity

Similar to bending, the basic design equation for the factored shear resistance, V_r, is given by

$$V_r = \phi_v V_n = \phi_v \frac{F_v bd}{1.5} \tag{20.10}$$

where ϕ_v = resistance factor for shear = 0.75, V_n = nominal adjusted shear resistance, F_v = adjusted shear strength, and b and d = thickness and width, respectively. Obviously, the last expression in Eq. (20.10) assumes a rectagular section, the nominal shear resistance could be determined from the relationship

$$V_n = \frac{F_v Ib}{Q} \tag{20.11}$$

where I = moment of inertia and Q = statical moment of an area about the neutral axis.

In timber bridges, notches are often made at the support to allow for vertical clearances and tolerances as illustrated in Figure 20.6; however, stress concentrations resulting from these notches

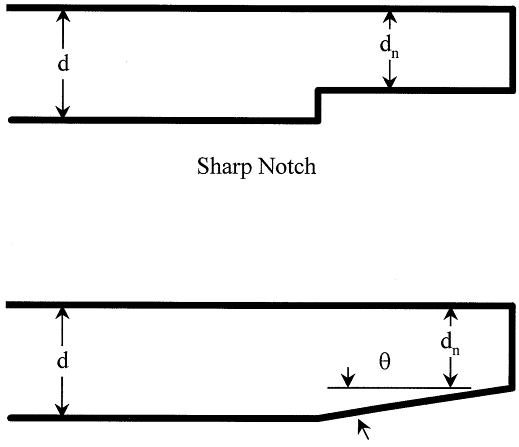


FIGURE 20.6 Notched beam: (a) sharp notch; (b) tapered notch.

significantly affect the shear resistance of the section. AASHTO-LRFD does not address this condition, but AF&PA does provide the designer with some guidance. At sections where the depth is reduced due to the presence of a notch, the shear resistance of the notched section is determined from

$$V' = \left(\frac{2}{3} F_v' b d_n\right)\left(\frac{d_n}{d}\right) \tag{20.12}$$

where d = depth of the unnotched section and d_n = depth of the member after the notch. When the notch is made such that it is actually a gradual tapered cut at an angle θ from the longitudinal axis of the beam, the stress concentrations resulting from the notch are reduced and the above equation becomes

$$V' = \left(\frac{2}{3} F_v' b d_n\right)\left(1 - \frac{(d - d_n)\sin\theta}{d}\right) \tag{20.13}$$

Similar to notches, connections too can produce significant stress concentrations resulting in reduced shear capacity. Where a connection produces at least one half the member shear force on either side of the connection, the shear resistance is determined by

$$V' = \left(\frac{2}{3} F_v' b d_e\right)\left(\frac{d_e}{d}\right) \tag{20.14}$$

where d_e = effective depth of the section at the connection which is defined as the depth of the member less the distance from the unloaded edge (or nearest unloaded edge if both edges are unloaded) to the center of the nearest fastener for dowel-type fasteners (e.g., bolts).

Bearing Capacity

The last aspect of beam design to be covered in this section is bearing at the supports. The governing design equation for factored bearing capacity perpendicular to the grain, $P_{r\perp}$, is

$$P_{r\perp} = \phi_c P_{n\perp} = \phi_c F_{c\perp} A_b C_b \tag{20.15}$$

where ϕ_c = resistance factor for compression = 0.90, P_{np} = nominal adjusted compression resistance perpendicular to the grain, F_{cp} = adjusted compression strength perpendicular to the grain, A_b = bearing area, and C_b = bearing factor.

The bearing area factor, C_b, allows an increase in the compression strength when the bearing length along the grain, l_b, is no more than 150 mm along the length of the member, is at least 75 mm from the end of the member, and is not in a region of high flexural stress. The bearing factor C_b is given by AF&PA as

$$C_b = (l_b + 9.5)/l_b \tag{20.16}$$

where l_b is in mm. This equation is the basis for the adjustment factors presented in the AASHTO-LRFD. For example, if a bearing length of 50 mm is used, the bearing strength can be increased by a factor of (50 + 9.5)/50 = 1.19.

20.4.6 Axially Loaded Members

The design of axially loaded members is quite similar to that of beams. Tension, compression, and combined axial and bending are addressed in AASHTO-LRFD.

Tension Capacity

The governing design equation for factored tension capacity parallel to the grain, P_{rt}, is

$$P_{rt} = \phi_t P_{nt} = \phi_t F_t A_n \tag{20.17}$$

where ϕ_t = resistance factor for tension = 0.80, P_{nt} = nominal adjusted tension resistance parallel to the grain, F_t = adjusted tension strength, and A_n = smallest net area of the component.

Compression Capacity

In terms of compression parallel to the grain, the AASHTO-LRFD design factored resistance, P_{rc} is given by

$$P_{rc} = \phi_c P_{nc} = \phi_c F_c A C_p \tag{20.18}$$

where ϕ_c = resistance factor for bending = 0.85, and P_{nc} = nominal adjusted compression resistance, F_c = adjusted compression strength, A = cross-sectional area, and C_p = column stability factor.

The column stability factor, C_p, accounts for the tendency of a column to buckle. The factor is taken as 1.0 for members with continuous lateral bracing; otherwise, C_p is calculated from one of the following expressions, depending on the material:

For sawn lumber:

$$C_p = \frac{1+B}{1.6} - \sqrt{\frac{(1+B)^2}{2.56} - \frac{B}{0.80}} \tag{20.19}$$

For round timber piles:

$$C_p = \frac{1+B}{1.7} - \sqrt{\frac{(1+B)^2}{2.89} - \frac{B}{0.85}} \tag{20.20}$$

For mechanically graded lumber and glued laminated timber:

$$C_p = \frac{1+B}{1.8} - \sqrt{\frac{(1+B)^2}{3.24} - \frac{B}{0.9}} \quad (20.21)$$

where

$$B = \frac{4.32Ed^2}{L_e{}^2 F_b} \quad \text{for visually graded solid sawn lumber} \quad (20.22)$$

and

$$B = \frac{60.2Ed^2}{L_e{}^2 F_b} \quad \text{for mechanically graded lumber and glulams} \quad (20.23)$$

where E = modulus of elasticity, d = net width (about which buckling may occur), L_e = effective length = effective length factor times the unsupported length = KL_u, and F_b = adjusted bending strength.

Combined Tension and Bending

AASHTO uses a linear interaction for tension and bending:

$$\frac{P_u}{P_{rt}} + \frac{M_u}{M_r} \leq 1.0 \quad (20.24)$$

where P_u and M_u = factored tension and moment loads on the member, respectively, and P_{rt} and M_r are the factored resistances as defined previously.

Combined Compression and Bending

AASHTO uses a slightly different interaction for compression and bending than tension and bending:

$$\left(\frac{P_u}{P_{rc}}\right)^2 + \frac{M_u}{M_r} \leq 1.0 \quad (20.25)$$

where P_u and M_u = factored compression and moment loads on the member, respectively, and P_{rc} and M_r are the factored resistances as defined previously. The squared term on the compression term was developed from experimental observations and is also used in the AF&PA LRFD. However, AF&PA includes secondary moments in the determination of M_u, which AASHTO neglects. AF&PA also includes biaxial bending in its interaction equations.

20.4.7 Connections

The final design consideration to be discussed in this section is that of connections. AASHTO-LRFD does not specifically address connections, so the designer is referred to the AF&PA LRFD. Decks must be attached to the supporting beams and beams to abutments such that vertical, longitudinal, and transverse loads are resisted. Additionally, the connections must be easily installed in the field. The typical timber bridge connection is a dowel-type connection directly between two wood components, or with a steel bracket.

The design of fasteners and connections for wood has undergone significant changes in recent years. Typical fastener and connection details for wood include nails, staples, screws, lag screws, dowels, and bolts. Additionally, split rings, shear plates, truss plate connectors, joist hangers, and many other types of connectors are available to the designer. The general LRFD design checking equation for connections is given as follows:

$$Z_u \le \lambda \phi_z Z' \tag{20.26}$$

where Z_u = connection force due to factored loads, λ = applicable time effect factor, ϕ_z = resistance factor for connections = 0.65, and Z' = connection resistance adjusted by the appropriate adjustment factors.

It should be noted that, for connections, the moisture adjustment is based on both in-service condition and on conditions at the time of fabrication; that is, if a connection is fabricated in the wet condition but is to be used in service under a dry condition, the wet condition should be used for design purposes due to potential drying stresses which may occur. It should be noted that C_M does not account for corrosion of metal components in a connection. Other adjustments specific to connection type (e.g., end grain factor, C_{eg}; group action factor, C_g; geometry factor, C_Δ; penetration depth factor, C_d; toe-nail factor, C_{tn}; etc.) will be discussed with their specific use. It should also be noted that when failure of a connection is controlled by a nonwood element (e.g., fracture of a bolt), then the time-effects factor is taken as unity since time effects are specific to wood and not applicable to nonwood components.

In both LRFD and ASD, tables of reference resistances (LRFD) and allowable loads (ASD) are available which significantly reduce the tedious calculations required for a simple connection design. In this section, the basic design equations and calculation procedures are presented, but design tables are not provided herein.

The design of general dowel-type connections (i.e., nails, spikes, screws, bolts, etc.) for lateral loading are currently based on possible yield modes. Based on these possible yield modes, lateral resistances are determined for the various dowel-type connections. Specific equations are presented in the following sections for nails and spikes, screws, bolts, and lag screws. In general, however, the dowel bearing strength, F_e, is required to determine the lateral resistance of a dowel-type connection. Obviously, this property is a function of the orientation of the applied load to the grain, and values of F_e are available for parallel to the grain, $F_{e\|}$, and perpendicular to the grain, $F_{e\perp}$. The dowel bearing strength or other angles to the grain, $F_{e\theta}$, is determined by

$$F_{e\theta} = \frac{F_{e\|} F_{e\perp}}{F_{e\|} \sin^2 \theta + F_{e\perp} \cos^2 \theta} \tag{20.27}$$

where θ = angle of load with respect to a direction parallel to the grain.

Nails, Spikes, and Screws

Nails, spikes, and screws are perhaps the most commonly used fastener in wood construction. Nails are generally used when loads are light such as in the construction of diaphragms and shear walls; however, they are susceptible to working loose under vibration or withdrawal loads. Common wire nails and spikes are quite similar, except that spikes have larger diameters than nails. Both a 12d (i.e., 12-penny) nail and spike are 88.9 mm in length; however, a 12d nail has a diameter of 3.76 mm while a spike has a diameter of 4.88 mm. Many types of nails have been developed to provide better withdrawal resistance, such as deformed shank and coated nails. Nonetheless, nails and spikes should be designed to carry laterally applied load and not withdrawal. Screws behave in a similar manner to nails and spikes, but also provide some withdrawal resistance.

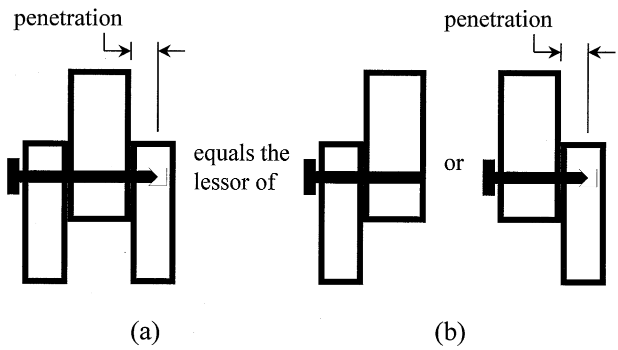


FIGURE 20.7 Double-shear connection: (a) complete connection; (b) left and right shear planes.

Lateral Resistance

The reference lateral resistance of a single nail or spike in single shear is taken as the least value determined by the four governing modes:

$$\mathbf{I_s}:\quad Z = \frac{3.3\,Dt_sF_{es}}{K_D} \tag{20.28}$$

$$\mathbf{III_m}:\ Z = \frac{3.3\,k_1DpF_{em}}{K_D\left(1+2\,R_e\right)} \tag{20.29}$$

$$\mathbf{III_s}:\ Z = \frac{3.3\,k_2Dt_sF_{em}}{K_D\left(2+R_e\right)} \tag{20.30}$$

$$\mathbf{IV}:\quad Z = \frac{3.3D^2}{K_D}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}} \tag{20.31}$$

where D = shank diameter; t_s = thickness of the side member; F_{es} = dowel bearing strength of the side member; p = shank penetration into member (see Figure 20.7); R_e = ratio of dowel bearing strength of the main member to that of the side member = F_{em}/F_{es}; F_{yb} = bending yield strength of the dowel fastener (i.e., nail or spike in this case); K_D = factor related to the shank diameter as follows: $K_D = 2.2$ for $D \le 4.3$ mm, $K_D = 0.38D + 0.56$ for 4.3 mm $< D \le 6.4$ mm, and $K_D = 3.0$ for $D > 6.4$ mm; and k_1 and k_2 = factors related to material properties and connection geometry as follows:

$$k_1 = -1 + \sqrt{2\left(1+R_e\right) + \frac{2F_{yb}\left(1=2R_e\right)D^2}{3F_{em}p^2}} \tag{20.32}$$

$$k_2 = -1 + \sqrt{\frac{2\left(1+R_e\right)}{R_e} + \frac{2F_{yb}\left(1+2R_e\right)D^2}{3F_{em}t_s^2}} \tag{20.33}$$

Similarly, the reference lateral resistance of a single wood screw in single shear is taken as the least value determined by the three governing modes:

$$\mathbf{I}_s\text{:}\quad Z = \frac{3.3\,Dt_s F_{es}}{K_D} \tag{20.34}$$

$$\mathbf{III}_s\text{:}\quad Z = \frac{3.3\,k_3 D t_s F_{em}}{K_D\left(2+R_e\right)} \tag{20.35}$$

$$\mathbf{IV}\text{:}\quad Z = \frac{3.3\,D^2}{K_D}\sqrt{\frac{1.75\,F_{em}F_{yb}}{3\left(1+R_e\right)}} \tag{20.36}$$

where K_D is defined for wood screws as it was for nails and spikes, and k_3 = a factor related to material properties and connection geometry as follows:

$$k_3 = -1 + \sqrt{\frac{2\left(1+R_e\right)}{R_e} + \frac{F_{yb}\left(2+R_e\right)D^2}{2F_{em}t_s^2}} \tag{20.37}$$

For nail, spike, or wood screw connections with steel side plates, the above equations for yield mode $\mathbf{I}_s$ is not appropriate. Rather, the resistance for that mode should be computed as the bearing resistance of the fastener on the steel side plate. When double shear connections are designed (Figure 20.7a), the reference lateral resistance is taken as twice the resistance of the weaker single shear representation of the left and right shear planes (Figure 20.7b).

For multiple nail, spike, or wood screw connections, the least resistance, as determined from Eqs. (20.28) through (20.31) for nails and spikes or Eqs. (20.34) through (20.36) for wood screws, is simply multiplied by the number of fasteners, n_f, in the connection detail. When multiple fasteners are used, the minimum spacing between fasteners in a row is 10D for wood side plates and 7D for steel side plates, and the minimum spacing between rows of fasteners is 5D. Whether a single or a multiple nail, spike, or wood screw connection is used, the minimum distance from the end of a member to the nearest fastener is 15D with wood side plates and 10D with steel side plates for tension members, and 10D with wood side plates and 5D with steel side plates for compression members. Additionally, the minimum distance from the edge of a member to the nearest fastener is 5D for an unloaded edge and 10D for a loaded edge.

The reference lateral resistance must be multiplied by all the appropriate adjustment factors. It is necessary to consider penetration depth, C_d, and end grain, C_{eg}, for nails, spikes, and wood screws. For nails and spikes, the minimum penetration allowed is 6D, while for wood screws the minimum is 4D. The penetration depth factor, $C_d = p/12D$, is applied to nails and spikes when the penetration depth is greater than the minimum, but less than 12D. Nails and spikes with a penetration depth greater than 12D assume a $C_d = 1.0$. The penetration depth factor, $C_d = p/7D$, is applied to wood screws when the penetration depth is greater than the minimum, but less than 7D. Wood screws with a penetration depth greater than 7D assume a $C_d = 1.0$. Whenever a nail, spike, or wood screw is driven into the end grain of a member, the end grain factor, $C_{eg} = 0.68$, is applied to the reference resistance. Finally, in addition to C_d and C_{eg}, a toe-nail factor, $C_{tn} = 0.83$, is applied to nails and spikes for "toe-nail" connections. A toe-nail is typically driven at an angle of approximately 30° to the member.

Axial Resistance

For connections loaded axially, tension is of primary concern and is governed by either fastener capacity (e.g., yielding of the nail) or fastener withdrawal. The tensile resistance of the fastener (i.e., nail, spike, or screw) is determined using accepted metal design procedure. The reference withdrawal resistance for nails and spikes with undeformed shanks in the side grain of the member is given by

$$Z_w = 31.6\,DG^{2.5}pn_f \tag{20.38}$$

where Z_w = reference withdrawal resistance in newtons and G = specific gravity of the wood. For nails and spikes with deformed shanks, design values are determined from tests and supplied by fastener manufactures, or Eq. (20.38) can be used conservatively with D = least shank diameter. For wood screws in the side grain,

$$Z_w = 65.3\, DG^2\; pn_f \tag{20.39}$$

A minimum wood screw depth of penetration of at least 25 mm or one half the nominal length of the screw is required for Eq. (20.39) to be applicable. No withdrawal resistance is assumed for nails, spikes, or wood screws used in end grain applications.

The end grain adjustment factor, C_{eg}, and the toe-nail adjustment factor, C_{tn}, as defined for lateral resistance, are applicable to the withdrawal resistances. The penetration factor is not applicable, however, to withdrawal resistances.

Combined Load Resistance

The adequacy of nail, spike, and wood screw connections under combined axial tension and lateral loading is checked using the following interaction equation:

$$\frac{Z_u \cos\alpha}{\lambda\phi_z Z'} + \frac{Z_u \sin\alpha}{\lambda\phi_z Z'_w} \le 1.0 \tag{20.40}$$

where α = angle between the applied load and the wood surface (i.e., 0° = lateral load and 90° = withdrawal/tension).

Bolts, Lag Screws, and Dowels

Bolts, lag screws, and dowels are commonly used to connect larger-dimension members where larger connection capacities are required. The provisions presented here are valid for bolts, lag screws, and dowels with diameters in the range of 6.3 mm ≤ D ≤ 25.4 mm.

Lateral Resistance

The reference lateral resistance of a bolt or dowel in single shear is taken as the least value determined by the six governing modes:

$$\mathbf{I_m}: \quad Z = \frac{0.83\, Dt_m F_{em}}{K_\theta} \tag{20.41}$$

$$\mathbf{I_s}: \quad Z = \frac{0.83\, Dt_s F_{es}}{K_\theta} \tag{20.42}$$

$$\mathbf{II}: \quad Z = \frac{0.93 k_1 D F_{es}}{K_\theta} \tag{20.43}$$

$$\mathbf{III_m}: \quad Z = \frac{1.04\, k_2 D t_m F_{em}}{K_\theta\left(1 + 2R_e\right)} \tag{20.44}$$

$$\mathbf{III_s}: \quad Z = \frac{1.04\, k_3 D t_s F_{em}}{K_\theta\left(2 + R_e\right)} \tag{20.45}$$

$$\mathbf{IV}: \quad Z = \frac{1.04\, D^2}{K_\theta} \sqrt{\frac{2F_{em}F_{yb}}{3\left(1 + R_e\right)}} \tag{20.46}$$

where D = shank diameter; t_m and t_s = thickness of the main and side member, respectively; F_{em} = F_{es} = dowel bearing strength of the main and side member, respectively; R_e = ratio of dowel bearing strength of the main member to that of the side member = F_{em}/F_{es}; F_{yb} = bending yield strength of the dowel fastener (i.e., nail or spike in this case); K_θ = factor related to the angle between the load and the main axis (parallel to the grain) of the member = $1 + 0.25(\theta/90)$; and k_1, k_2, and k_3 = factors related to material properties and connection geometry as follows:

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1+R_t+R_t^2\right)+R_t^2R_e^3} - R_e\left(1+R_t\right)}{1+R_e} \tag{20.47}$$

$$k_2 = -1 + \sqrt{2\left(1+R_e\right) + \frac{2F_{yb}\left(1+2R_e\right)D^2}{3F_{em}t_m^2}} \tag{20.48}$$

$$k_3 = -1 + \sqrt{\frac{2\left(1+R_e\right)}{R_e} + \frac{2F_{yb}\left(1+2R_e\right)D^2}{3F_{em}t_s^2}} \tag{20.49}$$

where R_t = ratio of the thickness of the main member to that of the side member = t_m/t_s.

The reference lateral resistance of a bolt or dowel in double shear is taken as the least value determined by the four governing modes:

$$\mathbf{I_m}: \quad Z = \frac{0.83\, Dt_mF_{em}}{K_\theta} \tag{20.50}$$

$$\mathbf{I_s}: \quad Z = \frac{1.66\, Dt_sF_{es}}{K_\theta} \tag{20.51}$$

$$\mathbf{III_s}: \quad Z = \frac{2.08\, k_3Dt_sF_{em}}{K_\theta\left(2+R_e\right)} \tag{20.52}$$

$$\mathbf{IV}: \quad Z = \frac{2.08\, D^2}{K_\theta}\sqrt{\frac{2\, F_{em}F_{yb}}{3\left(1+R_e\right)}} \tag{20.53}$$

where k_3 is defined by Eq. (20.49)

Similarly, the reference lateral resistance of a single lag screw in single shear is taken as the least value determined by the three governing modes:

$$\mathbf{I_s}: \quad Z = \frac{0.83\, Dt_sF_{es}}{K_\theta} \tag{20.54}$$

$$\mathbf{III_s}: \quad Z = \frac{1.19\, k_rDt_sF_{em}}{K_\theta\left(1+R_e\right)} \tag{20.55}$$

$$\mathbf{IV}: \quad Z = \frac{1.11\, D^2}{K_\theta}\sqrt{\frac{1.75\, F_{em}F_{yb}}{3\left(1+R_e\right)}} \tag{20.56}$$

where k_4 = a factor related to material properties and connection geometry as follows:

$$k_4 = -1 + \sqrt{\frac{2\left(1+R_e\right)}{R_e} + \frac{F_{yb}\left(2+R_e\right)D^2}{2\,F_{em}t_s^2}} \tag{20.57}$$

When double shear lag screw connections are designed, the reference lateral resistance is taken as twice the resistance of the weaker single shear representation of the left and right shear planes as was described for nail and wood screw connections.

Wood members are often connected to nonwood members with bolt and lag screw connections (e.g., wood to concrete, masonry, or steel). For connections with concrete or masonry main members, the dowel bear strength, F_{em}, for the concrete or masonry can be assumed the same as the wood side members with an effective thickness of twice the thickness of the wood side member. For connections with steel side plates, the equations for yield modes $\mathbf{I_s}$ and $\mathbf{I_m}$ are not appropriate. Rather, the resistance for that mode should be computed as the bearing resistance of the fastener on the steel side plate.

For multiple bolt, lag screw, and dowel connections, the least resistance is simply multiplied by the number of fasteners, n_f, in the connection detail. When multiple fasteners are used, the minimum spacings, edge distances, and end distances are dependent on the direction of loading. When loading is primarily parallel to the grain, the minimum spacing between fasteners in a row (parallel to the grain) is $4D$, and the minimum spacing between rows (perpendicular to the grain) of fasteners is $1.5D$ but not greater than 127 mm.* The minimum edge distance is dependent on l_m = length of the fastener in the main member for spacing in the main member or total fastener length in the side members for side member spacing relative to the diameter of the fastener. For shorter fasteners ($l_m/D \leq 6$), the minimum edge distance is $1.5D$, while for longer fasteners ($l_m/D > 6$), the minimum edge distance is the greater of $5D$ or one half the spacing between rows (perpendicular to the grain). The minimum end distance is $7D$ for tension members and $4D$ for compression members. When loading is primarily perpendicular to the grain, the minimum spacing within a row (perpendicular to the grain) is typically limited by the attached member but not to exceed 127 mm,* and the minimum spacing between rows (parallel to the grain) is dependent on l_m. For shorter fastener lengths ($l_m/D \leq 2$), the spacing between rows is limited to $2D$; for medium fastener lengths ($2 < l_m/D < 6$), the spacing between rows is limited to $(5l_m + 10D)/8$; and for longer fastener lengths ($l_m/D \geq 6$), the spacing is limited to $5D$; but never should the spacing exceed than 127 mm.* The minimum edge distance is $4D$ for loaded edges and $1.5D$ for unloaded edges. Finally, the minimum end distance for members loaded primarily perpendicular to the grain is $4D$.

The reference lateral resistance must be multiplied by all appropriate adjustment factors. It is necessary to consider group action, C_g, and geometry, C_Δ for bolts, lag screws, and dowels. In addition, penetration depth, C_d, and end grain, C_{eg}, need to be considered for lag screws. The group action factor accounts for load distribution between bolts, lag screw, or dowels when one or more rows of fasteners are used and is defined by

$$C_g = \frac{1}{n_f}\sum_{i=1}^{n_r} a_i \tag{20.58}$$

where n_f = number of fasteners in the connection, n_r = number of rows in the connection, and a_i = effective number of fasteners in row i due to load distribution in a row and is defined by

$$a_i = \left(\frac{1+R_{EA}}{1-m}\right)\left[\frac{m\left(1-m^{2n_i}\right)}{\left(1+R_{EA}m^{n_i}\right)\left(1+m\right)-1+m^{2n_i}}\right] \tag{20.59}$$

*The limit of 127 mm can be violated if allowances are made for dimensional changes of the wood.

where

$$m = u - \sqrt{u^2 - 1} \tag{20.60a}$$

$$u = 1 + \gamma \frac{s}{2}\left(\frac{1}{(EA)_m} + \frac{1}{(EA)_s}\right) \tag{20.60b}$$

and where γ = load/slip modulus for a single fastener; s = spacing of fasteners within a row; $(EA)_m$ and $(EA)_s$ = axial stiffness of the main and side member, respectively; R_{EA} = ratio of the smaller of $(EA)_m$ and $(EA)_s$ to the larger of $(EA)_m$ and $(EA)_s$. The load/slip modulus, γ, is either determined from testing or assumed as $\gamma = 0.246D^{1.5}$ kN/mm for bolts, lag screws, or dowels in wood-to-wood connections or $\gamma = 0.369D^{1.5}$ kN/mm for bolts, lag screws, or dowels in wood-to-steel connections.

The geometry factor, C_Δ, is used to adjust for connections in which either end distances and/or spacing within a row does not meet the limitations outlined previously. Defining a = actual minimum end distance, $a_{\min}$ = minimum end distance as specified previously, s = actual spacing of fasteners within a row, and $s_{\min}$ = minimum spacing as specified previously, the lesser of the following geometry factors are used to reduce the adjusted resistance of the connection:

1. End distance:	for, $a \geq a_{\min}$,	$C_\Delta = 1.0$
	for $a_{\min}/2 \leq a < a_{\min}$,	$C_\Delta = a/a_{\min}$
2. Spacing:	for, $s \geq s_{\min}$,	$C_\Delta = 1.0$
	for $3D \leq s < s_{\min}$,	$C_\Delta = s/s_{\min}$

In addition to group action and geometry, the penetration depth factor, C_d, and end grain factor, C_{eg}, are applicable to lag screws (not bolts and dowels). The penetration of a lag screw, including the shank and thread less the threaded tip, is required to be at least $4D$. For penetrations of at least $4D$ but not more than $8D$, the connection resistance is multiplied by $C_d = p/8D$, where p = depth of penetration. For penetrations of at least $8D$, $C_d = 1.0$. The end grain factor, C_{eg}, is applied when a lag screw is driven in the end grain of a member and is given as $C_{eg} = 0.67$.

Axial Resistance

Again, the tensile resistance of the fastener (i.e., bolt, lag screw, or dowel) is determined using accepted metal design procedure. Withdrawal resistance is only appropriate for lag screws since bolts and dowels are "through-member" fasteners. For the purposes of lag screw withdrawal, the penetration depth, p, is assumed as the threaded length of the screw less the tip length, and the minimum penetration depth for withdrawal is the lesser of 25 mm or one half the threaded length. The reference withdrawal resistance of a lag screw connection is then given by

$$Z_w = 92.6\, D^{0.75} G^{1.5} p n_f \tag{20.61}$$

where Z_w = reference withdrawal resistance in newtons and G = specific gravity of the wood.

The end grain adjustment factor, C_{eg}, is applicable to the withdrawal resistance of lag screws and is defined as $C_{eg} = 0.75$.

Combined Load Resistance

The resistance of a bolt, dowel, or lag screw connection to combined axial and lateral load is given by

$$Z'_\alpha = \frac{Z' Z'_w}{Z' \sin^2\alpha + Z'_w \cos^2\alpha} \tag{20.62}$$

where Z'_α = adjusted resistance at an angle and α = angle between the applied load and the wood surface (i.e., 0° = lateral load and 90° = withdrawal/tension).

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Further Reading

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