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Timber Structures

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9.1 Introduction

Wood is one of the earliest building materials, and as such its use often has been based more on tradition than principles of engineering. However, the structural use of wood and wood-based materials has increased steadily in recent times. The driving force behind this increase in use is the ever-increasing need to provide economical housing for the world's population. Supporting this need, though, has been an evolution of our understanding of wood as a structural material and ability to analyze and design safe and functional timber structures. This evolution is evidenced by the recent industry-sponsored development of the *Load and Resistance Factor Design* (LRFD) *Standard for Engineered Wood Construction* [1, 5].

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 section introduces the fundamental mechanical and physical properties of wood that govern its structural use, then presents fundamental considerations for the design of timber structures. The basics of beam, column, connection, and structural panel design are presented. Then, issues related to shear wall and diaphragm, truss, and arch design are presented. The section concludes with a discussion of current serviceability design code provisions and other serviceability considerations relevant to the design of timber structures. The use of the new LRFD provisions for timber structures [1, 5] is emphasized in this section; however, reference is also made to existing allowable stress provisions [2] due to their current popular use.

9.1.1 Types of Wood Products

There are a wide variety of wood and wood-based structural building products available for use in most types of structures. The most common products include solid lumber, glued laminated timber, plywood, and orientated strand board (OSB). Solid sawn lumber was the mainstay of timber construction and is still used extensively; however, 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. Engineered products such as wood composite I-joists and structural composite lumber (SCL) were the result of this evolution. These products have steadily gained popularity and now are receiving wide-spread use in construction.

9.1.2 Types of Structures

By far, the dominate types of structures utilizing wood and wood-based materials are residential and light commercial buildings. There are, however, numerous examples available of larger wood structures, such as gymnasiums, domes, and multistory office buildings. Light-frame construction is the most common type used for residential structures. Light-frame consists of nominal "2-by" lumber such as 2×4s (38 mm $\times$ 89 mm) up to 2×12s (38 mm $\times$ 286 mm) as the primary framing elements. Post-and-beam (or timber-frame) construction is perhaps the oldest type of timber structure, and has received renewed attention in specialty markets in recent years. Prefabricated panelized construction has also gained popularity in recent times. Reduced cost and shorter construction time have been the primary reasons for the interest in panelized construction. Both framed (similar to light-frame construction) and insulated (where the core is filled with a rigid insulating foam) panels are used. Other types of construction include glued-laminated construction (typically for longer spans), pole buildings (typical in so-called "agricultural" buildings, but making entry into commercial applications as well), and shell and folded plate systems (common for gymnasiums and other larger enclosed areas). The use of wood and wood-based products as only a part of a complete structural system is also quite common. For example, wood roof systems supported by masonry walls or wood floor systems supported by steel frames are common in larger projects.

Wood and wood-based products are not limited to building structures, but are also used in transportation structures as well. Timber bridges are not new, as evidenced by the number of covered bridges throughout the U.S. Recently, however, modern timber bridges have received renewed attention, especially for short-span, low-volume crossings.

9.1.3 Design Specifications and Industry Resources

The *National Design Specification for Wood Construction,* or NDS® [2], is currently the primary design specification for engineered wood construction. The NDS® is an allowable stress design (ASD) specification. As with the other major design specifications in the U.S., a *Load and Resistance Factor Design* (LRFD) *Standard for Engineered Wood Construction* [1, 5] has been developed and is recognized by all model building codes as an alternate to the NDS®. In this section, the LRFD approach to timber design will be emphasized; however, ASD requirements as provided by the NDS®, as well as other wood design specifications, also will be presented due to its current popularity and acceptance. Additionally, most provisions in the NDS® are quite similar to those in the LRFD except that the NDS® casts design requirements in terms of allowable stresses and loads and the LRFD utilizes nominal strength values and factored load combinations.

In addition to the NDS® and LRFD Standard, other design manuals, guidelines, and specifications are available. For example, the *Timber Construction Manual* [3] provides information related to engineered wood construction in general and glued laminated timber in more detail, and the *Plywood Design Specification* (PDS®)[6] and its supplements present information related to plywood properties and design of various panel-based structural systems. Additionally, various industry associations such as the APA–The Engineered Wood Association, American Institute of Timber Construction (AITC), American Forest & Paper Association–American Wood Council (AF&PA – AWC), Canadian Wood Council (CWC), Southern Forest Products Association (SFPA), Western Wood Products Association (WWPA), and Wood Truss Council of America (WTCA), to name but a few, provide extensive technical information.

One strength of the LRFD Specification is its comprehensive coverage of engineered wood construction. While the NDS® governs the design of solid-sawn members and connections, the *Timber Construction Manual* primarily provides procedures for the design of glued-laminated members and connections, and the PDS® addresses the design of plywood and other panel-based systems, the LRFD is complete in that it combines information from these and other sources to provide the engineer a comprehensive design specification, including design procedures for lumber, connections, I-joists, metal plate connected trusses, glued laminated timber, SCL, wood-base panels, timber poles and piles, etc. To be even more complete, the AF&PA has developed the *Manual of Wood Construction: Load & Resistance Factor Design* [1]. The Manual includes design value supplements, guidelines to design, and the formal LRFD Specification [5].

9.2 Properties of Wood

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, hyrgothermal, and viscoelastic material, and by its natural origins contains a multitude of inclusions and other defects.[1] The reader is referred to any number of basic texts that present a description of

[1]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, may occur after grading and may be more accurately termed "defects".

the fundamental structure and physical properties of wood as a material (e.g., [8, 11, 20]).

One aspect of wood that deserves attention here, however, is the affect of moisture on the physical and mechanical properties and performance of wood. Many problems encountered with wood structures 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 moisture content 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). Tables are available (see [20]) that provide the EMC for most species as a function of dry-bulb temperature and relative humidity. These tables allow designers to estimate in-service moisture contents that are required for their design calculations.

In structural applications, wood is typically dried to a MC near that expected in service prior to dimensioning and use. A major reason for this is that wood shrinks as its MC drops below the FSP. Wood machined to a specified size at a MC higher than that expected in service will therefore shrink to a smaller size in use. Since the amount any particular piece of wood will shrink is difficult to predict, it would be very difficult to control dimensions of wood if it was not machined after it was dried. Estimates of dimensional changes can be made with the use of published values of shrinkage coefficients for various species (see [20]).

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 lead to checking (cracks formed along the growth rings) and splitting (cracks formed across the growth rings).

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. For structural design purposes, design values are typically provided for a specific maximum MC (e.g., 19%).

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 duration and/or rate the load is 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 a wood member is to support that load. This response is termed "load duration" effects in wood design. Figure 9.1 illustrates this effect by plotting the time-to-failure as a function of the applied stress expressed in terms of the short term (static) strength. There are many theoretical models proposed to represent this response, but the line shown in Figure 9.1 was developed at the U.S. Forest Products Laboratory in the early 1950s [20] and is the basis for design provisions (i.e., design adjustment factors) in both the LRFD and NDS®.

The design factors derived from the relationship illustrated in Figure 9.1 are appropriate only for stresses and not for stiffness or, more precisely, the modulus of elasticity. Very much related to load duration effects, the deflection of a wood member under sustained load increases over time. This response, termed creep effect, must be considered in design when deflections are critical from either a safety or serviceability standpoint. The main parameters that significantly affect the creep response

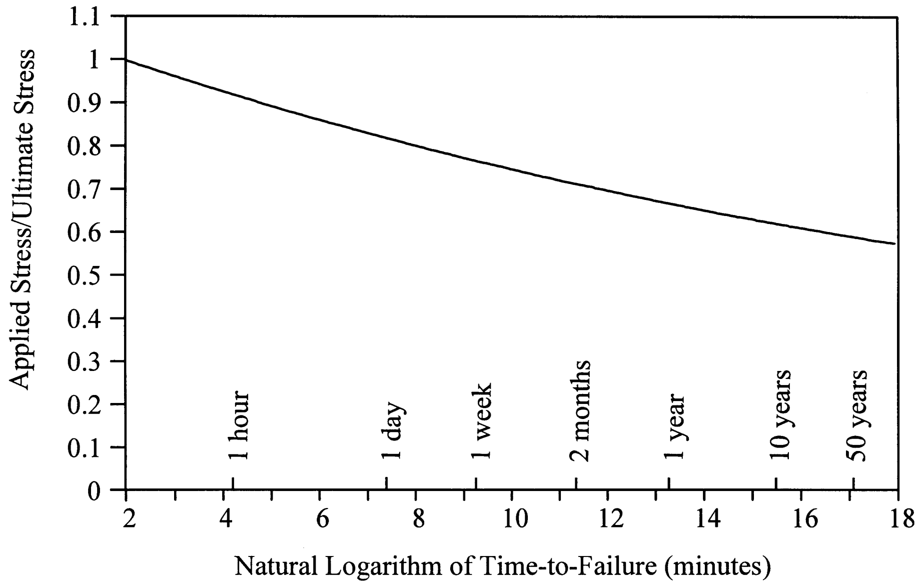

FIGURE 9.1: Load duration behavior of wood.

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 upwards of 100% given the right conditions. 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.

9.3 Preliminary Design Considerations

One of the first issues a designer must consider is determining the types of wood materials and/or wood products that are available for use. For smaller projects, it is better to select materials readily available in the region; for larger projects, a wider selection of materials may be possible since shipping costs may be offset by the volume of material required. One of the strengths of wood construction is its economics; however, the proper choice of materials is key to an efficient and economical wood structure. In this section, preliminary design considerations are discussed including loads and load combinations, design values and adjustments to the design values for in-use conditions.

9.3.1 Loads and Load Combinations

As with all structures designed in the U.S., nominal loads and load combinations for the design of wood structures are prescribed in the ASCE load standard [4]. The following basic factored load combinations must be considered in the design of wood structures when using the LRFD specification:

$$1.4D \tag{9.1}$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \tag{9.2}$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \tag{9.3}$$

$$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \tag{9.4}$$

$$1.2D + 1.0E + 0.5L + 0.2S \tag{9.5}$$

$$0.9D - (1.3W \text{ or } 1.0E) \tag{9.6}$$

where

D = dead load
L = live load excluding environmental loads such as snow and wind
L_r = roof live load during maintenance
S = snow load
R = rain or ice load excluding ponding
W = wind load
E = earthquake load (determined in accordance in with [4])

For ASD, the ASCE load standard provides four load combinations that must be considered: D, $D + L + (L_r$ or S or $R)$, $D + (W$ or $E)$, and $D + L + (L_r$ or S or $R) + (W$ or $E)$.

9.3.2 Design Values

The AF&PA [1] *Manual of Wood Construction: Load and Resistance Factor Design* provides nominal design values for visually and mechanically graded lumber, glued laminated 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; and are appropriate for use with the LRFD provisions. In addition, the *Manual* provides design values for metal plate connections and trusses, structural composite lumber, structural panels, and other pre-engineered structural wood products. (It should be noted that the LRFD Specification [5] provides only the design provisions, and design values for use with the LRFD Specification are provided in the AF&PA *Manual*.)

Similarly, the Supplement to the NDS® [2] provides tables of design values for visually graded and machine stress rated lumber and glued laminated timber. 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. Allowable design values for many proprietary products (e.g., SCL, I-joist, etc.) are provided by producers in accordance with established standards. For structural panels, design values are provided in the PDS® [6] and by individual product producers.

One main difference between the NDS® and LRFD design values, other than the NDS® 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 years) loading.

9.3.3 Adjustment of Design Values

In addition to providing *reference* design values, both the LRFD and the NDS® 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 not all factors are applicable to all design values, and the designer must take care to properly apply the appropriate factors.

LRFD reference strengths and NDS® 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:

$$R' = R \cdot C_1 \cdot C_2 \cdots C_n \tag{9.7}$$

where R' = adjusted design value (resistance), R = reference design value, and $C_1, C_2, \ldots C_n$ = applicable adjustment factors. Adjustment factors, for the most part, are common between the LRFD and the NDS®. Many factors are functions of the type, grade, and/or species of material while other factors are common across the broad spectrum of materials. For solid sawn lumber, glued laminated timber, piles, and connections, adjustment factors are provided in the NDS® and the LRFD *Manual*. For other products, especially proprietary products, the adjustment factors are provided by the product producers. The LRFD and NDS® list numerous factors 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 across the board 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}. The two treatment factors are provided by the individual treaters, but the wet service and temperature factors are provided in the LRFD *Manual*. For example, when considering the design of solid sawn lumber members, the adjustment values given in Table 9.1 for wet service, which is defined as the maximum EMC exceeding 19%, and Table 9.2 for temperature, which is applicable when continuous temperatures exceed 32°C, are applicable to all design values.

TABLE 9.1 Wet Service Adjustment Factors, C_M

	Size adjusted[a] F_b			Size adjusted[a] F_c				
Thickness	$\leq$ 20 MPa	$>$ 20 MPa	F_t	$\leq$ 12.4 MPa	$>$12.4 MPa	F_v	$F_{c\perp}$	E, E_{05}
$\leq$ 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.

Since, as discussed, the LRFD and the NDS® 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 the NDS® or LRFD, a wood structure is designed to resist all appropriate load combinations — unfactored combinations for the NDS® and factored combinations for the LRFD. The time effects (LRFD) and load duration (NDS®) 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,

TABLE 9.2 Temperature Adjustment Factors, C_t

	Dry use		Wet use	
Sustained temperature (°C)	E, E_{05}	All other prop.	E, E_{05}	All other prop.
$32 < T \leq 48$	0.9	0.8	0.9	0.7
$48 < T \leq 65$	0.9	0.7	0.9	0.5

the time effect factor, λ, is based on the load combination being considered as given in Table 9.3. In the NDS®, the load duration factor, C_D, is given in terms of the assumed cumulative duration of the design load. Table 9.4 provides commonly used load duration factors with the associated load combination.

TABLE 9.3 Time Effects Factors for Use in LRFD

Load combination	Time effect factor, λ
$1.4D$	0.6
$1.2D + 1.6L + 0.5(L_r$ or S or $R)$	0.7 when L from storage 0.8 when L from occupancy 1.25 when L from impact[a]
$1.2D + 1.6(L_r$ or S or $R) + (0.5L$ or $0.8W)$	0.8
$1.2D + 1.3W + 0.5L + 0.5(L_r$ or S or $R)$	1.0
$1.2D + 1.0E + 0.5L + 0.2S$	1.0
$0.9D - (1.3W$ or $1.0E)$	1.0

[a] For impact loading on connections, $\lambda = 1.0$ rather than 1.25.
From *Load and Resistance Factor Design (LRFD) for Engineered Wood Construction*, American Society of Civil Engineers (ASCE), AF&PA/ASCE 16-95. ASCE, New York, 1996. With permission.

TABLE 9.4 Load Duration Factors for Use in NDS®

Load duration	Load type	Load combination	Load duration factor, C_D
Permanent	Dead	D	0.9
Ten years	Occupancy live	$D + L$	1.0
Two months	Snow load	$D + L + S$	1.15
Seven days	Construction live	$D + L + L_r$	1.25
Ten minutes	Wind and earthquake	$D + (W$ or $E)$ and $D + L + (L_r$ or S or $R) + (W$ or $E)$	1.6
Impact	Impact loads	$D + L$ (L from impact)	2.0[a]

[a] For impact loading on connections, $\lambda = 1.6$ rather than 2.0.
From *National Design Specification for Wood Construction and Supplement*, American Forest and Paper Association (AF&PA), Washington, D.C., 1991. With permission.

Adjusted design values, whether they are allowable stresses or nominal strengths, are established in the same basic manner: the reference value is taken from an appropriate source (e.g., the LRFD *Manual* [1] or manufacture product literature) and is adjusted for various end-use conditions (e.g., wet use, load sharing, etc.). Additionally, depending on the design load combination being considered, a time effect factor (LRFD) or a load duration factor (NDS®) is applied to the adjusted resistance. Obviously, this rather involved procedure is critical, and somewhat unique, to wood design.

9.4 Beam Design

Bending members are perhaps the most common structural element. The design of wood beams follows traditional beam theory but, as mentioned previously, allowances must be made for the conditions and duration of loads expected for the structure. Additionally, many times bending members are not used as single elements, but rather as part of integrated systems such as a floor or roof system. As such, there exists a degree of member interaction (i.e., load sharing) which can be accounted for in the design. Wood bending members include sawn lumber, timber, glued laminated timber, SCL, and I-joists.

9.4.1 Moment Capacity

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. Strength considerations will be addressed here while serviceability design (i.e., deflection, etc.) will be presented in Section 9.13. In terms of moment, the LRFD [5] design equation is

$$M_u \leq \lambda \phi_b M' \tag{9.8}$$

where

M_u = moment caused by factored loads
λ = time effect factor applicable for the load combination under consideration
ϕ_b = resistance factor for bending = 0.85
M' = adjusted moment resistance

The moment caused by the factored load combination, M_u, is determined through typical methods of structural analysis. The assumption of linear elastic behavior is acceptable, but a nonlinear analysis is acceptable if supporting data exists for such an analysis. The resistance values, however, involve consideration of factors such as lateral support conditions and whether the member is part of a larger assembly.

Published design values for bending are given for use in the LRFD by AF&PA [1] in the form of a reference bending strength (stress), F_b. This value assumes strong axis orientation; an adjustment factor for flat-use, C_{fu}, can be used if the member will be used about the weak axis. Therefore, for strong $(x - x)$ axis bending, the moment resistance is

$$M' = M'_x = S_x \cdot F'_b \tag{9.9}$$

and for weak $(y - y)$ axis bending

$$M' = M'_y = S_y \cdot C_{fu} \cdot F'_b \tag{9.10}$$

where

M' = M'_x = adjusted strong axis moment resistance
M' = M'_y = adjusted weak axis moment resistance
S_x = section modulus for strong axis bending
S_y = section modulus for weak axis bending
F'_b = adjusted bending strength

For bending, typical adjustment factors to be considered include wet service, C_M; temperature, C_t; beam stability, C_L; size, C_F; volume (for glued laminated timber only), C_V; load sharing, C_r; form (for non-rectangular sections), C_f; and curvature (for glued laminated timber), C_c; and, of course, flat-use, C_{fu}. Many of these factors, including the flat-use factor, are functions of specific product types and species of materials, and therefore are provided with the reference design values. The two factors worth discussion here are the beam stability factor, which accounts for possible lateral-torsional buckling of a beam, and the load sharing factor, which accounts for system effects in repetitive assemblies.

The beam stability factor, C_L, 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 and the volume effects factor for glued laminated timber are not used simultaneously. Therefore, when designing an unbraced, glued laminated beam, the lessor of C_L and C_V is used to determine the adjusted bending strength. The beam stability factor is taken as 1.0 for members with continuous lateral bracing or meeting limitations set forth in Table 9.5.

TABLE 9.5 Conditions Defining Full Lateral Bracing

Depth to width (d/b)	Support conditions
≤ 2	No lateral support required.
> 2 and < 5	Ends supported against rotation.
≥ 5 and < 6	Compression edge continuously supported.
≥ 6 and < 7	Bridging, blocking, or X-bracing spaced no more than 2.4 m, or compression edge supported throughout its length and ends supported against rotation (typical in a floor system).
≥ 7	Both edges held in line throughout entire length.

When the limitations in Table 9.5 are not met, C_L is calculated from

$$C_L = \frac{1+\alpha_b}{2c_b} - \sqrt{\left(\frac{1+\alpha_b}{2c_b}\right)^2 - \frac{\alpha_b}{c_b}} \tag{9.11}$$

where

$$\alpha_b = \frac{\phi_s M_e}{\lambda \phi_b M_x^*} \tag{9.12}$$

and

c_b = beam stability coefficient = 0.95
ϕ_s = resistance factor for stability = 0.85
M_e = elastic buckling moment
M_x^* = moment resistance for strong axis bending including all adjustment factors except C_{fu}, C_V, and C_L.

The elastic buckling moment can be determined for most rectangular timber beams through a simplified method where

$$M_e = 2.40 E'_{05} \frac{I_y}{l_e} \tag{9.13}$$

where

E'_{05} = adjusted fifth percentile modulus of elasticity
I_y = moment of inertia about the weak axis
l_e = effective length between bracing points of the compression side of the beam

The adjusted fifth percentile modulus of elasticity is determined from the published reference modulus of elasticity, which is a mean value meant for use in deflection serviceability calculations, by

$$E'_{05} = 1.03 E'(1 - 1.645 \cdot COV_E) \tag{9.14}$$

where E' = adjusted modulus of elasticity and COV_E = coefficient of variation of E. The factor 1.03 recognizes that E is published to include a 3% shear component. For glued laminated timber, values of E include a 5% shear component, so it is acceptable to replace the 1.03 factor by 1.05 for the design of glued laminated timber beams. The COV of E can be assumed as 0.25 for visually graded lumber, 0.11 for machine stress rated (MSR) lumber, and 0.10 for glued laminated timber [2]. For other products, COVs or values of E'_{05} can be obtained from the producer. Also, the only adjustments needed to be considered for E are the wet service, temperature, and any preservative/fire-retardant treatment factors. The effective length, l_e, accounts for both the lateral motion and torsional phenomena and is given in the LRFD specification [1, 5] for numerous combinations of span types, end conditions, loading, bracing conditions, and actual unsupported span to depth ratios (l_u/d). Generally, for $l_u/d < 7$, the effective unbraced length, l_e, ranges from $1.33l_u$ to $2.06l_u$; for $7 \leq l_u/d \leq 14.3$, l_e ranges from $1.11l_u$ to $1.84l_u$; and for $l_u/d > 14.3$, l_e ranges from $0.9l_u + 3d$ to $1.63l_u + 3d$ where $d =$ depth of the beam.

The load sharing factor, C_r, is a multiplier that can be used when a bending member is part of an assembly, such as the floor system illustrated in Figure 9.2, consisting of three or more members

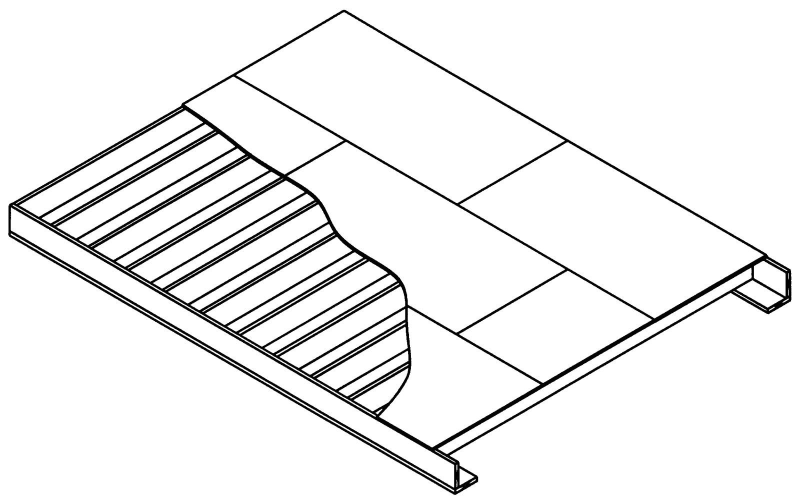

FIGURE 9.2: Typical wood floor assembly.

spaced no more than 610 mm on center and connected together by a load-distributing element, such as typical floor and roof sheathing. The factors recognize the beneficial effects of the sheathing in distributing loads away from less stiff members and are only applicable when considering uniformly applied loads. Assuming a strong correlation between strength and stiffness, this implies the load is distributed away from the weaker members as well, and that the value of C_r is dependent of the inherent variability of the system members. Table 9.6 provides values of C_r for various common framing materials.

TABLE 9.6 Load Sharing Factor, C_r

Assembly type	C_r
Solid sawn lumber framing members	1.15
I-joists with visually graded lumber flanges	1.15
I-joists with MSR lumber flanges	1.07
Glued laminated timber and SCL framing members	1.05
I-joists with SCL flanges	1.04

9.4.2 Shear Capacity

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

$$V_u \leq \lambda \phi_v V' \tag{9.15}$$

where

V_u = shear caused by factored loads

λ = time effect factor applicable for the load combination under consideration

ϕ_v = resistance factor for shear = 0.75

V' = adjusted shear resistance

Except in the design of I-joists, V_u is determined at a distance d (depth of the member) away from the face of the support if the loads acting on the member are applied to the face opposite the bearing area of the support. For other loading conditions and for I-joists, V_u is determined at the face of the support.

The adjusted shear resistance is computed from

$$V' = \frac{F'_v I b}{Q} \tag{9.16}$$

where

F'_v = adjusted shear strength parallel to the grain

I = moment of inertia

b = member width

Q = statical moment of an area about the neutral axis

For rectangular sections, this equation simplifies to

$$V' = \frac{2}{3} F'_v b d \tag{9.17}$$

where d = depth of the rectangular section.

The adjusted shear strength, F'_v, is determined by multiplying the published reference shear strength, F_v, by all appropriate adjustment factors. For shear, typical adjustment factors to be considered include wet service, C_M; temperature, C_t; size, C_F; and shear stress, C_H. The shear stress factor allows for increased shear strength in members with limited splits, checks, and shakes and ranges from $C_H = 1.0$ implying the presence of splits, checks, and shakes to $C_H = 2.0$ implying no splits, checks, or shakes.

In wood construction, notches are often made at the support to allow for vertical clearances and tolerances as illustrated in Figure 9.3; however, stress concentrations resulting from these notches significantly affect the shear resistance of the section. At sections where the depth is reduced due to the presence of a notch, the shear resistance of the notched section is determined from

$$V' = \left(\frac{2}{3} F'_v b d_n\right)\left(\frac{d_n}{d}\right) \tag{9.18}$$

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

$$V' = \left(\frac{2}{3} F'_v b d_n\right)\left(1 - \frac{(d - d_n)\sin\theta}{d}\right) \tag{9.19}$$

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

$$V' = \left(\frac{2}{3} F'_v b d_e\right)\left(\frac{d_e}{d}\right) \tag{9.20}$$

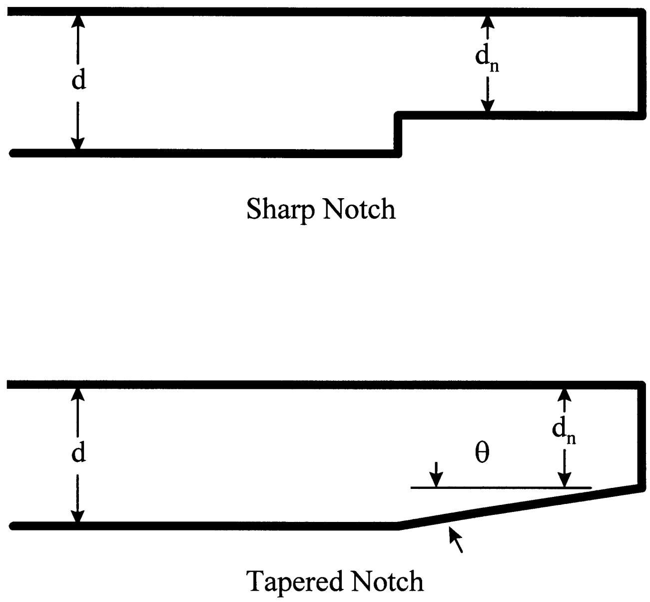

FIGURE 9.3: Notched beam: (a) sharp notch and (b) angled notch.

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). For additional information regarding connector design, see Section 9.8.

9.4.3 Bearing Capacity

The last aspect of beam design to be covered in this section is bearing at the supports. The governing design equation for bearing is

$$P_u \leq \lambda \phi_c P'_{\perp} \tag{9.21}$$

where

P_u = the compression force due to factored loads
λ = time effects factor corresponding to the load combination under consideration
ϕ_c = resistance factor for compression = 0.90
$P'_{\perp}$ = adjusted compression resistance perpendicular to the grain

The adjusted compression resistance, $P'_{\perp}$, is determined by

$$P'_{\perp} = A_n F'_{c\perp} \tag{9.22}$$

where

A_n = net bearing area
$F'_{c\perp}$ = adjusted compression strength perpendicular to the grain

The adjusted compression strength, $F'_{c\perp}$, is determined by multiplying the reference compression strength perpendicular to the grain, $F_{c\perp}$, by all applicable adjustment factors, including wet service, C_M; temperature, C_t; and bearing area, C_b. The bearing area factor, C_b, allows an increase in the compression strength when the bearing length, l_b, is no more than 150 mm along the length of the member and is at least 75 mm from the end of the member, and is given by

$$C_b = (l_b + 9.5)/l_b \tag{9.23}$$

where l_b is in mm.

9.4.4 NDS® Provisions

In the ASD format provided by the NDS®, the design checks are in terms of allowable stresses and unfactored loads. The determined bending, shear, and bearing stresses in a member due to unfactored loads are required to be less than the adjusted allowable bending, shear, and bearing stresses, respectively, including load duration effects. The basic approach to the design of a beam element, however, is quite similar between the LRFD and NDS® and is based on the same principles of mechanics. One major difference between the two specifications, though, is the treatment of load duration effects with respect to bearing. In the LRFD, the design equation for bearing (Equation 9.21) includes the time effect factor, λ; however, the NDS® does not require any adjustment for load duration for bearing. The allowable compressive stress perpendicular to the grain as presented in the NDS® is not adjusted because the compressive stress perpendicular to the grain follows a deformation definition of failure rather than fracture as in all other modes; thus, the adjustment is considered unnecessary. Conversely, the LRFD specification assumes time effects to occur in all modes, whether it is strength- (fracture) based or deformation-based.

9.5 Tension Member Design

The design of tension members, either by LRFD or NDS®, is relatively straightforward. The basic design checking equation for a tension member as given by the LRFD Specification [5] is

$$T_u \le \lambda \phi_t T' \tag{9.24}$$

where

T_u = the tension force due to factored loads
λ = time effects factor corresponding to the load combination under consideration
ϕ_t = resistance factor for tension = 0.80
T' = adjusted tension resistance parallel to the grain

The adjusted compression resistance, T', is determined by

$$T' = A_n F_t' \tag{9.25}$$

where A_n = net cross-sectional area and F_t' = adjusted tension strength parallel to the grain. The adjusted compression strength, F_t', is determined by multiplying the reference tension strength parallel to the grain, F_t, by all applicable adjustment factors, including wet service, C_M; temperature, C_t; and size, C_F.

It should be noted that tension forces are typically transferred to a member through some type of mechanical connection. When, for example as illustrated in Figure 9.4, the centroid of an unsymmetric net section of a group of three or more connectors differs by 5% or more from the centroid of the gross section, then the tension member must be designed as a combined tension and bending member (see Section 9.7).

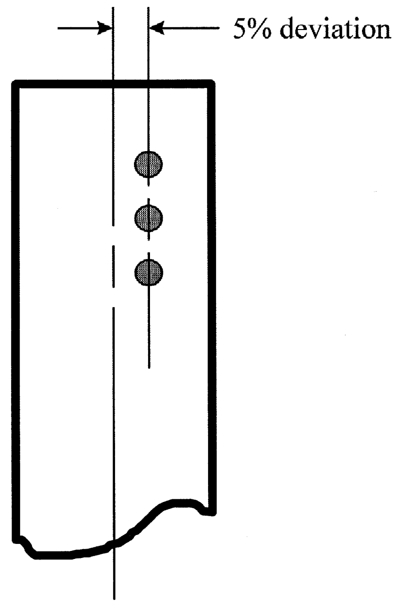

FIGURE 9.4: Eccentric bolted connection.

9.6 Column Design

The term *column* is typically considered to mean any compression member, including compressive members in trusses and posts as well as traditional columns. Three basic types of wood columns as illustrated in Figure 9.5 are (1) simple solid or traditional columns, which are single members such as sawn lumber, posts, timbers, poles, glued laminated timber, etc.; (2) spaced columns, which are two or more parallel single members separated at specific locations along their length by blocking and rigidly tied together at their ends; and (3) built-up columns, which consist of two or more members joined together by mechanical fasteners such that the assembly acts as a single unit.

Depending on the relative dimensions of the column as defined by the *slenderness ratio,* the design of wood columns is limited by the material's stiffness and strength parallel to the grain. The slenderness ratio is defined as the ratio of the effective length of the column, l_e, to the least radius of gyration, $r = \sqrt{I/A}$, where I = moment of inertia of the cross-section about the weak axis and A = cross-sectional area. The effective length is defined by $l_e = K_e l$, where K_e = effective length factor or buckling length coefficient and l = unbraced length of the column. The unbraced length, l, is measured as center to center distance between lateral supports. K_e is dependent on the column end support conditions and on whether sidesway is allowed or restrained. Table 9.7 provides values of K_e for various typical column configurations. Regardless of the column type of end conditions, the slenderness ratio, $K_e l/r$, is not permitted to exceed 175.

9.6.1 Solid Columns

The basic design equation for an axially loaded member as given by the LRFD Specification [5] is given as

$$P_u \leq \lambda \phi_c P_c' \tag{9.26}$$

where

P_u = the compression force due to factored loads

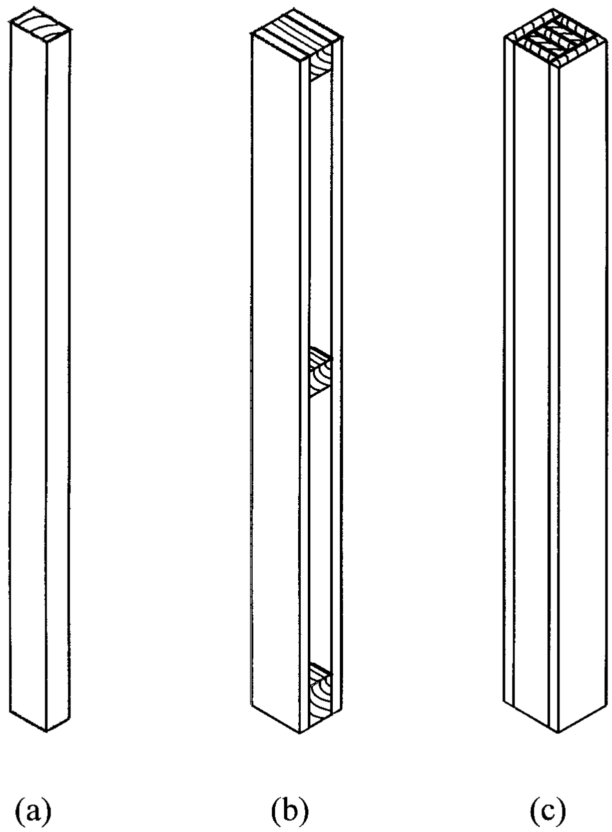

FIGURE 9.5: Typical wood columns: (a) simple wood column, (b) spaced column, and (c) built-up column.

TABLE 9.7 Effect Length Factors for Wood Columns

Support conditions	Sidesway restrained	Theoretical K_e	Recommended K_e^a
Fixed–fixed	Yes	0.50	0.65
Fixed–pinned	Yes	0.70	0.80
Fixed–fixed	No	1.00	1.20
Pinned–pinned	Yes	1.00	1.00
Fixed–free	No	2.00	2.10
Fixed–pinned	No	2.00	2.40

[a] Values recommended by [5].

λ = time effects factor corresponding to the load combination under consideration
ϕ_c = resistance factor for compression = 0.90
P_c' = adjusted compression resistance parallel to the grain.

The adjusted compression resistance, P_c', is determined by

$$P_c' = AF_c' \tag{9.27}$$

where A = gross area and F_c' = adjusted compression strength parallel to the grain. The adjusted compression strength, F_c', is determined by multiplying the reference compression strength parallel to the grain, F_c, by all applicable adjustment factors, including wet service, C_M; temperature, C_t; size, C_F; and column stability, C_P.

The column stability factor, C_P, accounts for partial lateral support for a column and is given by

$$C_p = \frac{1+\alpha_c}{2c} - \sqrt{\left(\frac{1+\alpha_c}{2c}\right)^2 - \frac{\alpha_c}{c}} \tag{9.28}$$

where

$$\alpha_c = \frac{\phi_s P_e}{\lambda \phi_c P_0'} \tag{9.29}$$

$$P_e = \frac{\pi^2 E_{05}' A}{\left(\frac{K_e l}{r}\right)^2} \tag{9.30}$$

and $c =$ coefficient based on member type, $\phi_s =$ resistance factor for stability $= 0.85$, $\phi_b =$ resistance factor for compression $= 0.90$, $\lambda =$ time effect factor for load combination under consideration, $P_e =$ Euler buckling resistance, $P_0' =$ adjusted resistance of a fully braced (or so-called "zero-length") column, $E_{05}' =$ adjusted fifth percentile modulus of elasticity, and $A =$ gross cross-sectional area. The coefficient $c = 0.80$ for solid sawn members, 0.85 for round poles and piles, and 0.90 for glued laminated members and SCL. E_{05}' is determined as presented for beam stability using Equation 9.14, and P_0' is determined using Equation 9.27, except that the reference compression strength, F_c, is *not* adjusted for stability (i.e., assume $C_P = 1.0$).

Two common conditions occurring in solid columns are notches and tapers. When notches or holes are present in the middle half of the effective length (between inflection points), and the net moment of inertia at the notch or hole is less than 80% of the gross moment of inertia, or the length of the notch or hole is greater than the largest cross-sectional dimension of the column, then P_c' (Equation 9.27) and C_P (Equation 9.28) are computed using the net area, A_n, rather than gross area, A. When notches or holes are present outside this region, the column resistance is taken as the lesser of that determined without considering the notch or hole (i.e., using gross area) and

$$P_c' = A_n F_c^* \tag{9.31}$$

where $F_c^* =$ the compression strength adjusted by all applicable factors *except* for stability (i.e., assume $C_P = 1.0$).

Two basic types of uniformly tapered solid columns exist: circular and rectangular. For circular tapered columns, the design diameter is taken as either (1) the diameter of the small end or (2) when the diameter of the small end, D_1, is at least one-third of the large end diameter, D_2,

$$D = D_1 + X(D_2 - D_1) \tag{9.32}$$

where $D =$ design diameter and $X =$ a factor dependent on support conditions as follows:

1. Cantilevered, large end fixed: $X = 0.52 + 0.18(D_1/D_2)$ (9.33a)
2. Cantilevered, small end fixed: $X = 0.12 + 0.18(D_1/D_2)$ (9.33b)
3. Singly tapered, simple supports: $X = 0.32 + 0.18(D_1/D_2)$ (9.33c)
4. Doubly tapered, simple supports: $X = 0.52 + 0.18(D_1/D_2)$ (9.33d)
5. All other support conditions: $X = 0.33$ (9.33e)

For uniformly tapered rectangular columns with constant width, the design depth of the member is handled in a manner similar to circular tapered columns, except that buckling in two directions must be considered. The design depth is taken as either (1) the depth of the small end or (2) when the depth of the small end, d_1, is at least one-third of the large end depth, d_2,

$$d = d_1 + X(d_2 - d_1) \tag{9.34}$$

where d = design depth and X = a factor dependent on support conditions as follows:

For buckling in the tapered direction:

1. Cantilevered, large end fixed: $X = 0.55 + 0.15(d_1/d_2)$ (9.35a)
2. Cantilevered, small end fixed: $X = 0.15 + 0.15(d_1/d_2)$ (9.35b)
3. Singly tapered, simple supports: $X = 0.35 + 0.15(d_1/d_2)$ (9.35c)
4. Doubly tapered, simple supports: $X = 0.55 + 0.15(d_1/d_2)$ (9.35d)
5. All other support conditions: $X = 0.33$ (9.35e)

For buckling in the non-tapered direction:

1. Cantilevered, large end fixed: $X = 0.63 + 0.07(d_1/d_2)$ (9.35f)
2. Cantilevered, small end fixed: $X = 0.23 + 0.07(d_1/d_2)$ (9.35g)
3. Singly tapered, simple supports: $X = 0.43 + 0.07(d_1/d_2)$ (9.35h)
4. Doubly tapered, simple supports: $X = 0.63 + 0.07(d_1/d_2)$ (9.35i)
5. All other support conditions: $X = 0.33$ (9.35j)

In addition to these provisions, the design resistance of a tapered circular or rectangular column cannot exceed

$$P_c' = A_n F_c^* \tag{9.36}$$

where A_n = net area of the column at any cross-section and F_c^* = the compression strength adjusted by all applicable factors *except* for stability (i.e., assume $C_P = 1.0$).

9.6.2 Spaced Columns

Spaced columns consist of two or more parallel single members separated at specific locations along their length by blocking and rigidly tied together at their ends. As defined in Figure 9.5b, L_1 = overall length in the spaced column direction, L_2 = overall length in the solid column direction, L_3 = largest distance from the centroid of an end block to the center of the mid-length spacer, L_{ce} = distance from the centroid of end block connectors to the nearer column end, d_1 = width of individual components in the spaced column direction, and d_2 = width of individual components in the solid column direction. Typically, the individual components of a spaced column are considered to act individually in the direction of the wide face of the members. The blocking, however, effectively reduces the unbraced length in the weak direction. Therefore, the following L/d ratios are imposed on spaced columns:

1. In the spaced column direction: $L_1/d_1 \le 80$ (9.37a)
   $L_3/d_1 \le 40$ (9.37b)
2. In the solid column direction:[2] $L_2/d_2 \le 50$ (9.37c)

Depending on the length L_{ce} relative to L_1, one of two effective length factors can be assumed for design in the spaced column direction. If sidesway is not allowed and $L_{ce} \leq 0.05L_1$, then the effective length factor is assumed as $K_e = 0.63$; or if there is no sidesway and $0.05L_1 < L_{ce} \leq 0.10L_1$, then assume $K_e = 0.53$. For columns with sidesway in the spaced column direction, an effective length factor greater than unity is determined as given in Table 9.7.

9.6.3 Built-Up Columns

Built-up columns consist of two or more members joined together by mechanical fasteners such that the assembly acts as a single unit. Conservatively, the capacity of a built-up member can be taken as the sum of resistances of the individual components. Conversely, if information regarding the rigidity and overall effectiveness of the fasteners is available, the designer can incorporate such information into the analysis and take advantage of the composite action provided by the fasteners; however, no codified procedures are available for the design of built-up columns. In either case, the fasteners must be designed appropriately to resist the imposed shear and tension forces (see Section 9.8 for fastener design).

9.6.4 NDS® Provisions

For rectangular columns, which are common in wood construction, the slenderness ratio can be expressed as the ratio of the unbraced length to the least cross-sectional dimension of the column, or L/d where d is the least cross-sectional dimension. This is the approach offered by the NDS® [2] which differs from the more general approach of the LRFD [5] and is identical to that used in the LRFD for spaced columns. Often, the unbraced length of a column is not the same about both the strong and weak axes and the slenderness ratios in both directions should be considered (e.g., $r_1 = L_1/d_1$ in the strong direction and $r_2 = L_2/d_2$ in the weak direction). One common example of such a case is wood studs in a load bearing wall where, if adequately fastened, the sheathing provides continuous lateral support in the weak direction and only the slenderness ratio about the strong axis needs to be determined. The slenderness ratio is not permitted to exceed 50[2] for single solid columns or built-up columns, and is not permitted to exceed 80 for individual members of spaced columns; however, when used for temporary construction bracing, the allowable slenderness ratio is increased from 50 to 75 for single or built-up columns. All other provisions related to column design are equivalent between the NDS® and LRFD.

9.7 Combined Load Design

Often, structural wood members are subjected to bending about both principal axes and/or bending combined with axial loads. The bending can come from eccentric axial loads and/or laterally applied loads. The adjusted member resistances for moment, M', tension, T', and compression, P'_c, defined in Sections 9.4, 9.5, and 9.6 are used for combined load design in conjunction with an appropriate interaction equation. All other factors (e.g., the resistance factors ϕ_b, ϕ_t, and ϕ_c, and the time effect factor, λ) are also the same in combined load design as defined previously.

[2]For rectangular columns, the provision $L/d \leq 50$ is equivalent to the provision $KL/r \leq 175$.

9.7.1 Combined Bending and Axial Tension

When a tension load acts simultaneously with bending about one or both principal axes, the following interaction equations must be satisfied:

$$\text{1. Tension face:} \quad \frac{T_u}{\lambda\phi_t T'} + \frac{M_{ux}}{\lambda\phi_b M'_s} + \frac{M_{uy}}{\lambda\phi_b M'_y} \leq 1.0 \tag{9.38}$$

$$\text{2. Compression face:} \quad \frac{\left(M_{ux} - \frac{d}{6}T_u\right)}{\lambda\phi_b M'_x} + \frac{M_{uy}}{\lambda\phi_b\left(1 - \frac{M_{ux}}{\phi_b M_e}\right)^2} \leq 1.0 \tag{9.39}$$

where

T_u	=	tension force due to factored loads
M_{ux} and M_{uy}	=	moment due to factored loads about the strong and weak axes, respectively
M'_x and M'_y	=	adjusted moment resistance about the strong and weak axes, respectively
M_e	=	elastic lateral buckling moment (Equation 9.13)
M'_s	=	M'_x computed assuming the beam stability factor $C_L = 1.0$ but including all other appropriate adjustment factors, including the volume factor C_V
d	=	depth of the member

Equations 9.38 and 9.39 assume rectangular sections. If a non-rectangular section is being designed, the quantity $d/6$ appearing in Equation 9.38 should be replaced by S_x/A where S_x = the section modulus about the strong axis and A = gross area of the section.

9.7.2 Biaxial Bending or Combined Bending and Axial Compression

When a member is being designed for either biaxial bending or for combined axial compression and bending about one or both principal axes, the following interaction equation must be satisfied:

$$\left(\frac{P_u}{\lambda\phi_c P'_c}\right)^2 + \frac{M_{mx}}{\lambda\phi_b M'_x} + \frac{M_{my}}{\lambda\phi_b M'_y} \leq 1.0 \tag{9.40}$$

where

P_u	=	axial load due to factored loads
P'_c	=	adjusted compression resistance assuming the compression acts alone (i.e., no moments) for the axis of buckling providing the lower resistance value
M_{mx} and M_{my}	=	moments due to factored loads, including any magnification resulting from second-order moments, about the strong and weak axes, respectively
M'_x and M'_y	=	adjusted strong and weak axes moment resistances, respectively, assuming the beam stability factor $C_L = 1.0$

The moments due to factored loads, M_{mx} and M_{my}, can be determined either of two ways: (1) using an appropriate second-order analysis procedure or (2) using a simplified magnification method. The moment magnification method recommended in the LRFD is given as follows:

$$M_{mx} = B_{bx}M_{bx} + B_{sx}M_{sx} \tag{9.41}$$

$$M_{my} = B_{by}M_{by} + B_{sy}M_{sy} \tag{9.42}$$

where M_{bx} and M_{by} = factored strong and weak axis moments, respectively, from loads producing no lateral translation or sidesway determined using an appropriate first-order analysis; M_{sx} and M_{sy} = factored strong and weak axis moments, respectively, from loads producing lateral translation or sidesway determined using an appropriate first-order analysis; and B_{bx}, B_{sx}, B_{by}, and B_{sy} = moment magnification factors to account for second-order effects and associated with M_{bx}, M_{sx}, M_{by}, and M_{sy}, respectively, and are determined as follows:

$$B_{bx} = \frac{C_{mx}}{\left(1 - \frac{P_u}{\phi_c P_{ex}}\right)} \geq 1.0 \tag{9.43}$$

$$B_{by} = \frac{C_{mx}}{\left[1 - \frac{P_u}{\phi_c P_{ex}} - \left(\frac{M_{ux}}{\phi_b M_e}\right)^2\right]} \geq 1.0 \tag{9.44}$$

$$B_{sx} = \frac{1}{\left(1 - \frac{\Sigma P_u}{\phi_c \Sigma P_{ex}}\right)} \geq 1.0 \tag{9.45}$$

$$B_{sy} = \frac{1}{\left(1 - \frac{\Sigma P_u}{\phi_c \Sigma P_{ey}}\right)} \geq 1.0 \tag{9.46}$$

where

P_{ex} and P_{ey}	=	Euler buckling resistance about the strong and weak axes, respectively, as determined by Equation 9.30
$\sum P_u$	=	sum of all compression forces due to factored loads for all columns in the sidesway mode under consideration
$\sum P_{ex}$ and $\sum P_{ey}$	=	sum of all Euler buckling resistances for columns in the sidesway mode under consideration about its strong and weak axes, respectively
C_{mx} and C_{my}	=	factor relating the actual moment diagram shape to an equivalent uniform moment diagram for moment applied about the strong and weak axes, respectively

All other terms are as defined previously. The factors C_{mx} and C_{my} are determined for one of two conditions:

1. For members braced against lateral joint translation with only end moments applied:

$$C_{mx} \text{ or } C_{my} = 0.60 - 0.40\frac{M_1}{M_2} \tag{9.47}$$

   where M_1/M_2 = ratio of the smaller magnitude end moment to the larger end moment in the plane of bending ($x - x$ or $y - y$) under consideration, with the ratio defined as being negative for single curvature and positive for double curvature.

2. For members braced against joint translation in the plane of bending under consideration with lateral loads applied between the joints:

   a) C_{mx} or $C_{my} = 0.85$ for members with ends restrained against rotation, or

   b) C_{mx} or $C_{my} = 1.00$ for members with ends unrestrained against rotation.

For members not braced against sidesway, all four moment magnification factors need to be determined; however, for members braced against sidesway, only B_{bx} and B_{sx} need to be determined.

9.7.3 NDS® Provisions

The primary difference between the LRFD approach for members subjected to combined load and that of the NDS® is in the design of members for combined axial and bending loads. The LRFD combines moments from all sources, including moments from eccentrically applied axial loads and moments from transversely applied loads. The NDS® provides the following general interaction equation:

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{bx} + f_c\left(\frac{6e_y}{d_y}\right)\left(1 + 0.234\frac{f_c}{F_{cEx}}\right)}{F'_{bx}\left(1 - \frac{f_c}{F_{cEx}}\right)} + \frac{f_{by} + f_c\left(\frac{6e_x}{d_x}\right)\left(1 + 0.234\frac{f_c}{F_{cEy}}\right)}{F'_{by}\left[1 - \frac{f_c}{F_{cEy}} - \left(\frac{f_{bx}}{F_{bE}}\right)^2\right]} \leq 1.0 \tag{9.48}$$

where

f_c	=	compression stress due to unfactored loads
f_{bx} and f_{by}	=	bending stress about the strong and weak axes, respectively, due to unfactored loads
F'_c	=	adjusted allowable compression stress
F'_{bx} and F_{by}	=	adjusted allowable bending stress about the strong and weak axes, respectively
F_{cEx} and F_{cEy}	=	allowable Euler buckling stress about the strong and weak axes, respectively
F_{bE}	=	allowable buckling stress for bending
e_x and e_y	=	eccentricity in the x and y directions, respectively
d_x and d_y	=	cross-sectional dimension in the x (narrow dimension) and y (wide dimension) directions, respectively

The allowable buckling values are determined from

$$F_{cEx} = \frac{K_{cE}E'}{(l_{ex}/d_x)^2} \tag{9.49}$$

$$F_{cEy} = \frac{K_{cE}E'}{\left(l_{ey}/d_y\right)^2} \tag{9.50}$$

$$F_{bE} = \frac{K_{bE}E'}{(R_b)^2} \tag{9.51}$$

where

K_{ce}	=	0.3 for visually graded and machine evaluated lumber, or 0.418 for products with a coefficient of variation on E less than or equal to 11% (e.g., MSR lumber and glued laminated timber)
K_{bE}	=	0.438 for visually graded and machine evaluated lumber, or 0.609 for products with a coefficient of variation on E less than or equal to 11%
E'	=	adjusted modulus of elasticity
R_b	=	slenderness ratio for bending as given by

$$R_b = \sqrt{\frac{l_{ey}d_y}{d_x^2}} \tag{9.52}$$

Note that l_{ey} is used in Equation 9.52 since lateral buckling in beams is only possible about the weak axis.

9.8 Fastener and Connection Design

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:

$$Z_u \leq \lambda \phi_z Z' \tag{9.53}$$

where

Z_u = connection force due to factored loads
λ = applicable time effect factor
ϕ_z = resistance factor for connections = 0.65
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. Also, C_M does not account for corrosion of metal components in a connection. Other adjustments specific to connection type (e.g., diaphragm factor, C_{di}; 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 the time effects factor, λ, is not allowed to exceed unity for connections as noted in Table 9.3. Additionally, when failure of a connection is controlled by a non-wood 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 non-wood components.

In both the LRFD Manual [1] and the NDS® [2], tables of reference resistances (LRFD) and allowable loads (NDS®) 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 such as those given in the LRFD Manual and the NDS® are not provided here.

The design of general dowel-type connections (i.e., nails, spikes, screws, bolts, etc.) for lateral loading are currently based on possible yield modes. Formerly (i.e., all previous editions of the NDS®), empirical behavior equations were the basis for the design provisions. Figure 9.6 illustrates the various yield modes that must be considered for single and double shear connections. 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, though, 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, $F_{e\parallel}$, and perpendicular to the grain, $F_{e\perp}$. The dowel bearing strength or other angles to the grain, $F_{e\theta}$, is determined by

$$F_{e\theta} = \frac{F_{e\parallel} F_{e\perp}}{F_{e\parallel} \sin^2\theta + F_{e\perp} \cos^2\theta} \tag{9.54}$$

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

9.8.1 Nails, Spikes, and Screws

Nails and spikes are perhaps the most commonly used fasteners 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

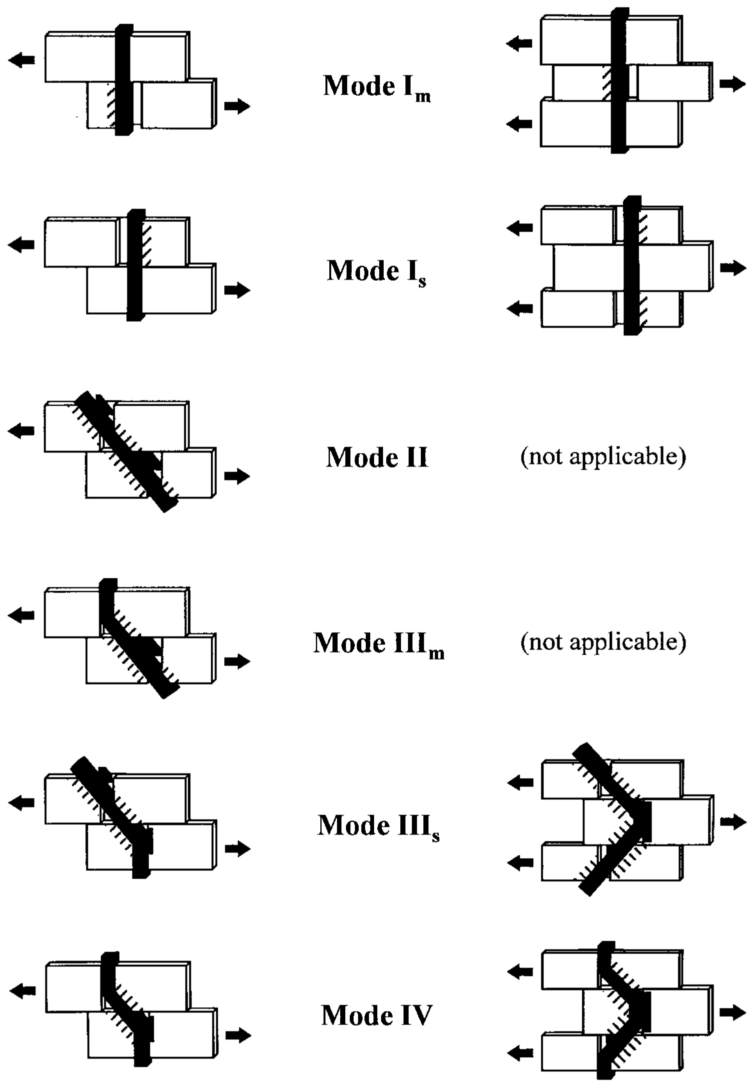

FIGURE 9.6: Yield modes for dowel-type connections. (Courtesy of American Forest & Paper Association, Washington, D.C.)

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.

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:

$$I_s: \quad Z = \frac{3.3 D t_s F_{es}}{K_D} \tag{9.55}$$

$$III_m: \quad Z = \frac{3.3 k_1 D p F_{em}}{K_D (1 + 2R_e)} \tag{9.56}$$

$$III_s: \quad Z = \frac{3.3k_2 D t_s F_{em}}{K_D(2+R_e)} \tag{9.57}$$

$$IV: \quad Z = \frac{3.3D^2}{K_D}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}} \tag{9.58}$$

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 9.7)
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
k_1 and k_2	=	factors related to material properties and connection geometry as follows:

$$K_1 = -1 + \sqrt{2(1+R_e) + \frac{2F_{yb}(1+2R_e)D^2}{3F_{em}p^2}} \tag{9.59}$$

$$K_2 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_{yb}(1+2R_e)D^2}{3F_{em}t_s^2}} \tag{9.60}$$

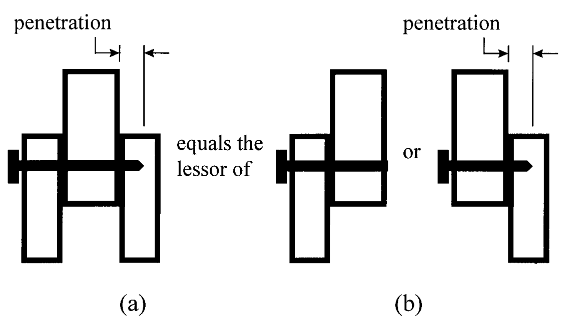

FIGURE 9.7: Double shear connection: (a) complete connection and (b) left and right shear planes.

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:

$$I_s: \quad Z = \frac{3.3Dt_sF_{es}}{K_D} \tag{9.61}$$

$$III_s: \quad Z = \frac{3.3K_3Dt_sF_{em}}{K_D(2+R_e)} \tag{9.62}$$

$$IV: \quad Z = \frac{3.3D^2}{K_D}\sqrt{\frac{1.75F_{em}F_{yb}}{3(1+R_e)}} \tag{9.63}$$

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:

$$K_3 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{F_{yb}(2+R_e)D^2}{2F_{em}t_s^2}} \tag{9.64}$$

For nail, spike, or wood screw connections with steel side plates, the above equations for yield mode I_s are not appropriate. Rather, the resistance for that mode should be computed as the bearing resistance of the fastener on the steel side plate. Also, when double shear connections are designed (Figure 9.7a), the reference lateral resistance is taken as twice the resistance of the weaker single shear representation of the left and right shear planes (Figure 9.7b).

For multi-nail, spike, or wood screw connections, the least resistance, as determined from Equations 9.55 through 9.58 for nails and spikes or Equations 9.61 through 9.63 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 this 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 $C_d = 1.0$. 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 $C_d = 1.0$. 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

$$Z_w = 31.6DG^{2.5}pn_f \tag{9.65}$$

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 Equation 9.65 can be used conservatively with D = least shank diameter. For wood screws in the side grain,

$$Z_w = 65.3DG^2pn_f \tag{9.66}$$

A minimum wood screw depth of penetration of at least 25 mm or one-half the nominal length of the screw is required for Equation 9.66 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:

$$\frac{Z_u \cos\alpha}{\lambda\phi_z Z'} + \frac{Z_u \sin\alpha}{\lambda\phi_z Z'_w} \leq 1.0 \tag{9.67}$$

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

9.8.2 Bolts, Lag Screws, and Dowels

Bolts, lag screws, and dowels are commonly used to connect larger dimension members and when 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 $\leq D \leq$ 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:

$$I_m: \quad Z = \frac{0.83 D t_m F_{em}}{K_\theta} \tag{9.68}$$

$$I_s: \quad Z = \frac{0.83 D t_s F_{es}}{K_\theta} \tag{9.69}$$

$$II: \quad Z = \frac{0.93 K_1 D F_{es}}{K_\theta} \tag{9.70}$$

$$III_m: \quad Z = \frac{1.04 K_2 D t_m F_{em}}{K_\theta (1 + 2R_e)} \tag{9.71}$$

$$III_s: \quad Z = \frac{1.04 K_3 D t_s F_{em}}{K_\theta (2 + R_e)} \tag{9.72}$$

$$IV: \quad Z = \frac{1.04 D^2}{K_\theta}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} \tag{9.73}$$

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 $= 1 + 0.25(\theta/90)$

K_1, K_2, and K_3 = factors related to material properties and connection geometry as follows:

$$K_1 = \frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{1 + R_e} \tag{9.74}$$

$$K_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}t_m^2}} \tag{9.75}$$

$$K_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}t_s^2}} \tag{9.76}$$

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:

$$I_m: \quad Z = \frac{0.83 D t_m F_{em}}{K_\theta} \tag{9.77}$$

$$I_s: \quad Z = \frac{1.66 D t_s F_{es}}{K_\theta} \tag{9.78}$$

$$III_s: \quad Z = \frac{2.08 K_3 D t_s F_{em}}{K_\theta(2 + R_e)} \tag{9.79}$$

$$IV: \quad Z = \frac{2.08 D^2}{K_\theta}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} \tag{9.80}$$

where K_3 is defined by Equation 9.76.

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:

$$I_s: \quad Z = \frac{0.83 D t_s F_{es}}{K_\theta} \tag{9.81}$$

$$III_s: \quad Z = \frac{1.19 K_4 D t_s F_{em}}{K_\theta(2 + R_e)} \tag{9.82}$$

$$IV: \quad Z = \frac{1.11 D^2}{K_\theta}\sqrt{\frac{1.75 F_{em}F_{yb}}{3(1 + R_e)}} \tag{9.83}$$

where K_4 = a factor related to material properties and connection geometry as follows:

$$K_4 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{F_{yb}(2 + R_e)D^2}{2F_{em}t_s^2}} \tag{9.84}$$

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 non-wood 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 I_s and 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 multi-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 dominantly 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.[3] 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 dominantly 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,[3] 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.[3] The minimum edge distance is $4D$ for loaded edges and $1.5D$ for unloaded edges. Finally, the minimum end distance for a member loaded dominantly perpendicular to the grain is $4D$.

The reference lateral resistance must be multiplied by all the 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

$$C_g = \frac{1}{n_f} \sum_{i=1}^{n_r} a_i \tag{9.85}$$

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

$$a_i = \left(\frac{1 + R_{EA}}{1 - m}\right) \left[\frac{m(1 - m^{2n_i})}{(1 + R_{EA} m^{n_i})(1 + m) - 1 + m^{2n_i}}\right] \tag{9.86}$$

where

$$m = u - \sqrt{u^2 - 1} \tag{9.87a}$$

$$u = 1 + \gamma \frac{s}{2} \left(\frac{1}{(EA)_m} + \frac{1}{(EA)_s}\right) \tag{9.87b}$$

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.

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

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 lessor of the following geometry factors are used to reduce the connection's adjusted resistance:

1. End distance:

$$\begin{array}{lll} \text{for,} & a \geq a_{\min}, & C_\Delta = 1.0 \\ \text{for,} & a_{\min}/2 \leq a < a_{\min}, & C_\Delta = a/a_{\min} \end{array}$$

2. Spacing:

$$\begin{array}{lll} \text{for,} & s \geq s_{\min}, & C_\Delta = 1.0 \\ \text{for,} & 3D \leq s < s_{\min}, & C_\Delta = s/s_{\min} \end{array}$$

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 lessor of 25 mm or one-half the threaded length. The reference withdrawal resistance of a lag screw connection is then given by

$$Z_w = 92.6 D^{0.75} G^{1.5} p n_f \tag{9.88}$$

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:

$$Z'_\alpha = \frac{Z' Z'_w}{Z' \sin^2 \alpha + Z'_w \cos^2 \alpha} \tag{9.89}$$

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

9.8.3 Other Types of Connections

A multitude of other connection types are available for design, including split rings, shear plates, truss plate connectors, joist hangers, and many other types of connectors. Many of the connection types are proprietary (e.g., truss plates, joist hangers, etc.), and as such their design resistances are provided by the fastener manufacture/producer.

9.8.4 NDS® Provisions

The basic approach used for the design of dowel type connections is identical between that presented here based on the LRFD Specification and that of the NDS®. The NDS® is less restrictive and, perhaps, helpful with respect to minimum edge distances, end distances, and spacings. For nails, spikes, and wood screws, the NDS® stipulates that the minimum edge distances, end distances, and spacings must be such that splitting of the wood is avoided. The other notable difference between the LRFD procedure and that of the NDS® is the group action factor for bolts, lag screws, and dowels. The group action factor, C_g, prescribed in the NDS® is given by

$$C_g = \left(\frac{1 + R_{EA}}{1 - m}\right)\left[\frac{m(1 - m^{2n})}{n[(1 + R_{EA}m^n)(1 + m) - 1 + m^{2n}]}\right] \tag{9.90}$$

where n = number of fasteners in the row, and all other factors are as defined previously. This equation is essentially equivalent to the factor a_i used to calculate the LRFD group action factor. The difference between this group action factor and that presented for the LRFD is that the LRFD accounts for load sharing in all rows, while the NDS® bases the adjustment of load sharing within one row of fasteners.

9.9 Structural Panels

Structural-use panels are wood-based panel products bonded with waterproof adhesives. Currently, structural-use panels include plywood, oriented strand board (OSB), and composite panels (reconstituted wood-based material with wood veneer faces). The intended use for structural-use panels is primarily for floor, roof, and wall sheathing in residential, commercial, and industrial applications; therefore, these products typically must resist bending and shear stress in the panel without excessive deformation, and resist racking shear (i.e., diaphragm behavior). Due to the numerous types and formulations of structural-use panels, a performance-based system was developed [21]. Structural-use panels are qualified based on performance specifications for specified span ratings. As such, panels of various thickness and composition may be qualified for the same span rating; therefore, the designer is required to specify both a panel thickness and span rating. A span rating is a set of two numbers separated by a slash (e.g., 24/0, 32/16, 48/24, etc.) or a single number (e.g., 16 o.c., 24 o.c., etc.). When two numbers are provided, the first number indicates the allowable span (in inches) if the panel is used as a roof sheathing with the primary axis perpendicular to the rafters, and the second number indicates the allowable span (in inches) if the panel is used as a floor sheathing with the primary axis perpendicular to the joists. When only one number is provided, it is the maximum allowed spacing of the supporting members (in inches), and use is typically specific to either floor or wall applications.

The APA qualifies structural-use, performance rated panels into four types: Rated Sheathing, Structural I Rated Sheathing, Rated Sturd-I-Floor, and Rated Siding. Rated Sheathing is intended for use as subflooring and wall and roof sheathing, and will carry a span rating such as 24/16, 40/20, etc. Structural I Rated Sheathing is intended for use where shear and cross-panel properties are of importance, such as in diaphragms and shearwalls. A very common span rating for Structural I sheathing is 32/16. Common thicknesses for both Rated and Structural I Rated Sheathing range from nominally 8 mm to 19 mm. Sturd-I-Floor is specifically designed as a floor sheathing that can be used as both subflooring and underlayment for carpeted floors. Since the intended use is for floor applications only, one span number is provided in the rating (e.g., 16 o.c., 20 o.c., etc.). Nominal thicknesses range from 15 mm to 29 mm. Rated Siding is produced for exterior siding and can be manufactured to included textured surfaces for visual appearance. Since the intended use is for exterior siding applications only, one span number is provided in the rating (e.g., 16 o.c., 24 o.c., etc.) which indicates the maximum stud spacing allowed for use in an APA Sturd-I-Wall application. Nominal thicknesses range from 9 mm to 16 mm. It should be noted that the span ratings can be modified (i.e., decreased or increased) by local codes.

Although the performance-based product standards provide a simplified method for selecting an appropriate sheathing material, in many instances the designer is required to calculate resistances directly and/or the specific design is not well-suited or covered by the limited performance-based applications. In these cases, the LRFD Manual [1] provides reference properties for various structural-use panels, and the APA [6] provides similar properties for allowable stress design. Additionally, due to the composite nature of structural-use panels, design section properties are provided by the LRFD Manual and APA.

9.9.1 Panel Section Properties

Structural-use panels are composite, non-homogeneous, anisotropic panels, and as such would be difficult to analyze for routine application. To simplify the design process, effective section properties are provided for the primary and secondary directions on a per-unit-width basis. When the normal stress is parallel to the primary axis, the section properties for that direction should be used. Such would be the case when a structural-use panel is used to sheath a floor or roof assembly by laying the primary axis perpendicular to the joists or rafters (see Figure 9.2). As an example, the effective moment of inertia for plywood is calculated using transformed sections assuming a ratio of 35:1 for the modulus of elasticity parallel vs. perpendicular to the grain.

9.9.2 Panel Design Values

Design stresses (nominal in the case of LRFD and allowable for ASD), are based on two basic factors: the Grade Stress Level and Species Group. The Grade Stress Level is the grade of the panel and is designated by S-1 (highest grade), S-2, and S-3. The Species Group is used to classify the panel into one of four groups, designated by 1, 2, 3, or 4. Often, panels are made from more than one species, thus the necessity of creating Species Groups. If span-rated panels are used, then a relationship between the performance type (e.g., Structural I), panel thickness, span-rating, and the appropriate Grade Stress Level and Species Group is provided to determine applicable design stresses.

To further simplify the derivation of design stresses, tables of nominal resistances (i.e., M', V', T', $E'I$, etc.) are provided for the basic panel types based on span rating and thickness. This alleviates the need to determine both effective section properties and design stresses, both of which are complicated by (1) the composite, anisotropic nature of the products; (2) the diversity of structural-use panel products; as well as (3) the proprietary nature of the panel industry.

Design values and nominal resistances for structural-use panels, as with all structural wood products, are provided at specified reference conditions. In addition to the standard adjustments for wet service, C_M, and temperature, C_t, adjustments for width effects, C_w, and grade/construction, C_G, must be considered. The width factor, C_w, accounts for increased panel resistance for narrow sections. The published values are for 610 mm or wider. The grade/construction factor, C_G, is used when the properties of a particular panel differ from that of the reference grade or when panel materials have layups different than that for which the reference values are published. In both cases, the factors are specified by the product producer.

9.9.3 Design Resources

The NDS® does not cover the design of structural panels. The APA and Structural Board Association (SBA) are the primary groups responsible for providing design information to the designer for structural-use panels. The LRFD *Manual* [1] has included significant information relevant to load and resistance factor design of panels, but traditional ASD procedures are maintained by APA and SBA (e.g., see [6, 7, 17]). Additionally, a complete presentation of the allowable stress design of panels is given by McLain [15].

9.10 Shear Walls and Diaphragms

The primary lateral force-resisting systems used in timber structures are shear walls and diaphragms. Shear walls are structural wall assemblies designed to resist lateral forces applied in the plane of the wall and transmit those forces down to the base of the wall. A typical wood stud shear wall is illustrated in Figure 9.8. Diaphragms are horizontal (or nearly horizontal) systems which transmit lateral forces to the shear walls or other lateral force-resisting systems. A floor (see Figure 9.2) or roof system is often designed as a diaphragm as well as a gravity-load resisting system.

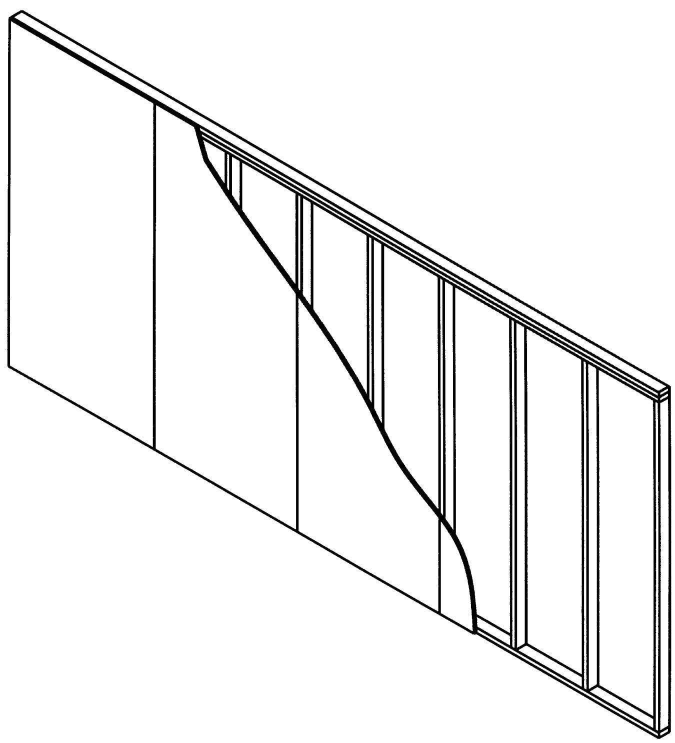

FIGURE 9.8: Typical shear wall assembly.

Shear walls and diaphragms are designed such that

$$D_u \leq \lambda \phi_z D' \tag{9.91}$$

where

D_u = shear wall or diaphragm force due to factored loads

D' = adjusted shear wall or diaphragm resistance

λ = time effect factor

ϕ_z = resistance factor for shear walls and diaphragms = 0.65

Typically, D_u and D' are taken as the force applied and resisted, respectively, per unit length of wall or diaphragm. The LRFD Specification allows a simplified design procedure based on a beam analogy; however, more refined and accurate analysis and design procedures are allowed if the designer wishes to use one (e.g., finite element analysis). In the beam analog, the shear wall and/or diaphragm and its individual components are considered as thin, deep beams with the sheathing resisting the in-plane shear and the boundary members resisting the axial forces. This equates the sheathing to the web of an I-beam section and the boundary elements to the flanges of the I-beam. Boundary elements (e.g., studs, sills, rim-joists, etc.) must be included at all shear wall and diaphragm perimeters and around interior openings or other discontinuities.

9.10.1 Required Resistance

The required resistance of a shear wall or diaphragm, D_u, comes directly from the governing factored lateral load combination considering wind and/or seismic loads. Consideration must be made of loads acting both along each of the structure's principal axes and orthogonal effects. For more on the governing load combinations and their applications, refer to the ASCE Load Standard [4].

9.10.2 Shear Wall and Diaphragm Resistance

The reference resistance of a shear wall and/or diaphragm is generated one of several ways: (1) through experimental tests for specific assemblies, (2) using the beam analogy, or (3) through the use of more rigorous analysis procedures (e.g., finite element analysis). To determine the resistance of a shear wall or diaphragm, consideration should be given to sheathing, framing connection resistance and spacing, sheathing capacity and configuration, blocking, and framing capacity and spacing. For many typical assemblies (e.g., specific type and layout of structural sheathing, nail and nail spacing, and size, species, and spacing framing members), tables are available which provide a reference resistance per unit length (e.g., APA has numerous technical and design publications related to diaphragms and shear walls). Finally, a complete and adequate load transfer path from the shear wall or diaphragm to the supporting system must be designed.

The adjusted resistance of a shear wall and/or diaphragm, D', is determined by adjusting the reference resistance by all the adjustment factors previously discussed in this section. In addition, a diaphragm factor, $C_{di} = 1.10$, is used to account for the increase in resistance of nails used in diaphragms over that specified for single nail connections, recalling that no group action factor was available for nail connections.

9.10.3 Design Resources

Guidelines for shear wall and diaphragm design is provided in a number of publications by APA and is actually prescribed in many model building codes, but is not covered in the NDS®. The LRFD brings information from APA and other sources into a single document and covers shear wall and diaphragm design; but, additional information, such as that provided by the APA, may be helpful. Additionally, a good discussion of diaphragm and shear wall design is presented by Diekmann [9].

9.11 Trusses

One of the most popular structural uses of structural lumber is in wood trusses. This includes both trusses designed for field construction with bolted or nailed gusset plates, as well as the pre-engineered truss. Individual members of both truss types are sized following the procedures outlined previously in this section for bending, tension, and compression members. The connections for pre-engineered trusses, however, typically involve proprietary metal plate connectors (MPC). MPCs are light gauge galvanized steel and come in a variety of sizes, shapes, and configurations, depending on the plate producer. The sheet stock is punched to produce projections (teeth) that are pressed into the truss members to form virtually any planer joint configuration required for a truss. Since MPCs are proprietary and not interchangeable, metal plate connected trusses, including the connections, are designed by the truss supplier per design data provided by the building designer. The responsibilities of each party are outlined by the WTCA [22]. The building designer is required to design the structure suitable for supporting the truss and specifications for all truss, including orientations, spans, profiles, and minimum design loads and deflection performance. The building designer is also required to insure adequate permanent bracing and connection of the truss(es) to the structure. The truss supplier is required to design the truss per accepted standards (e.g., [5, 19]) to meet the requirements set forth by building designer, temporary and permanent bracing requirements, permitted loads, and calculated deflections. A good overview of the truss industry and the design and construction of wood truss systems is provided by Trus Joist MacMillan [18].

9.12 Curved Beams and Arches

The provisions discussed in this section are applicable to the design of glued laminated timber (as well as solid sawn lumber and SCL); however, the advantage of the laminating process, beyond providing large dimensioned members, is the ability to fabricate curved and arched sections. Such members are quite popular for medium to large structures requiring large open spans, such as churches, school gymnasiums, etc. Curved beams and arches of constant cross-section are discussed here; however, design provisions and procedures are available in the LRFD Specification and Manual [1, 5] for pitched and tapered curved beams and arches.

9.12.1 Curved Beams

The design of curved beams is identical to that of straight members, except that a curvature adjustment factor, C_c, is used to adjust the reference moment resistance and radial stresses need to be checked. A curved beam is defined as a member with a radius of curvature of the inside face of a lamination, R_f, defined such that $R_f \geq 100t$ for hardwoods and southern pine, and $R_f \geq 125t$ for other softwoods.

Moment Resistance

The adjusted moment resistance of a straight section was given by Equation 9.9. This basic equation is still valid for curved beams and arches, and the curvature factor, C_c, is used to modify the published resistance of a straight, glued, laminated section of constant cross-section to account for curvature on the bending capacity. The curvature factor is defined by

$$C_c = 1 - 2000 \left(\frac{t}{R_f} \right)^2 \tag{9.92}$$

where t = thickness of individual laminations. This factor is applied to the published reference bending strength or reference moment resistance just as any other adjustment factor. The remainder of the flexural design is then identical to that of straight beams.

Radial Stresses

Radial stresses occur in curved members in the radial direction (or transverse to the bending stresses). Tension occurs when the bending moments act to flatten the beam (or increase the radius of curvature), and compression occurs when moments tend to increase curvature (or decrease the radius of curvature). The result is commonly a tension stress perpendicular to the grain.

To account for radial stresses induced by bending moments in curved beams, the bending resistance limited by radial stress can be calculated using:

$$M' = \frac{2}{3} R_m b d F_r' \tag{9.93}$$

where

M' = adjusted moment resistance of the curved beam limited by radial stress
R_m = radius of curvature at the mid-depth of the cross-section
b = width of the section
d = depth of the section
F_r' = adjusted strength in the radial direction

The value of F_r' depends on the type of material, whether the stress is tension or compression, and whether radial reinforcement is provided. For example, $F_r' =$ adjusted radial tension (perpendicular to the grain) strength when the radial stress is tension and no reinforcement is provided; $F_r' = F_v'/3$ when the radial stress is tension, Douglas-fir-Larch, Douglas-fir-South, Hem-Fir, Western Woods, or Canadian softwood species are used, and the design load is either wind or earthquake, or if reinforcement is provided to carry the radial force; or $F_r' =$ adjusted radial compression (perpendicular to the grain) strength when the radial stress is compression. The radial strengths should be adjusted for other factors, such as temperature and moisture content.

9.12.2 Arches

The primary difference between curved beams and arches is the degree of arching; that is, when the radius of curvature is small enough, arch action is induced and axial forces become an integral part of the structural resistance. Arches can be assumed when $R_f < 100t$ for hardwoods and southern pine, and $R_f < 125t$ for other softwoods. The two basic types of glued laminated arches are three-hinged and two-hinged arches (See Figure 9.9), and include a variety of styles such as tudor, gothic, radial, parabolic, etc. The three-hinged arch is statically determinate owing to the moment release at the peak. The design of three-hinged arches are, therefore, relatively straightforward and should consider the combined bending moment and compression parallel to the grain along the length of the arch, and shear near the member ends. Two-hinged arches are statically indeterminate, and an appropriate method of analysis must be used to determine design moments, shears, and axial loads along the length of the arch. Once these forces and moments are determined, the design is again straightforward and should consider the combined bending moment and compression parallel to the grain along the length of the arch, and shear near the member ends. Two-hinged arches are slightly more efficient than three-hinged arches; however, constructability and transportation issues often dictate the use of three-hinged arches over two-hinged arches.

Moment Resistance

Again, the adjusted moment resistance of a straight section was given by Equation 9.9. This basic equation is still valid for arches, except now, in addition to the curvature factor defined for curved beams, the volume effect factor, C_V, is for arches as follows:

1. For $F_b'(1 - C_V) \le f_c$: $\quad C_V' = 1.0 \quad (9.94)$
2. For $F_b'(1 - C_V) > f_c$: $\quad C_V' = C_V + f_c/F_b' \quad (9.95)$

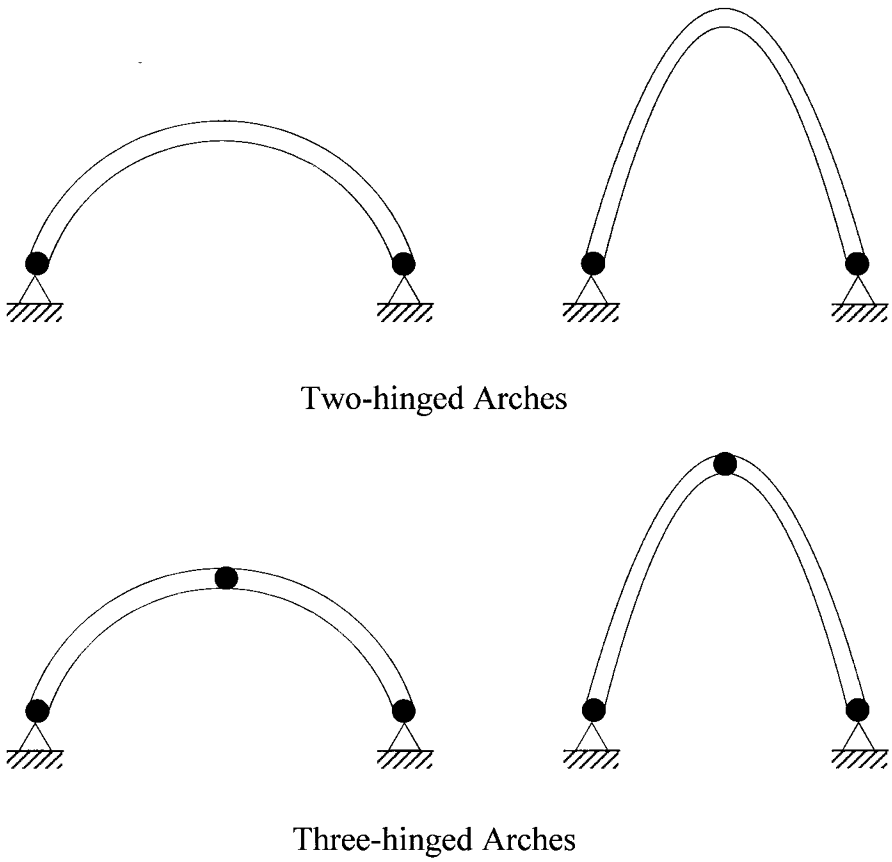

FIGURE 9.9: Two- and three-hinge arches.

where

F_b' = adjusted bending strength of the arch, including all applicable adjustment factors except C_V

C_V' = modified or corrected volume effect factor

f_c = compressive stress in the arch considering factored loads

The remainder of the flexural design is then identical to that of straight and curved beams.

Axial Compression Resistance

Again, the main difference between a curved beam and an arch is that the curvature has been increased to a point where axial forces become significant in arches. The procedures outlined previously for axial resistance are considered valid for arches. Typically, arches are considered to be fully braced against buckling about the weak axis and unbraced about the strong axis; however, design for buckling about the strong axis is typically not required due to arch action. Regardless, the bracing requirements presented for compression and lateral torsional buckling are considered applicable to arches.

Interaction Equation for Arches

The interaction of bending moments and axial forces is accounted for in the same manner as presented for straight columns. However, since arches are often fully braced against buckling in the weak direction, the factored moment need not be magnified. Therefore, the interaction equation given by Equation 9.40 reduces to

$$\left(\frac{P_u}{\lambda \phi_c P_c'}\right)^2 + \frac{M_{bx}}{\lambda \phi_b M_x'} \leq 1.0 \tag{9.96}$$

for arches where P_u = factored axial load; P_c' = adjusted axial compression resistance; M_{bx} = factored moment about the strong axis; M_x' = adjusted moment capacity about the strong axis; λ = time effect factor; and ϕ_c and ϕ_b = resistance factors for axial compression and bending, respectively.

Radial Stresses

Due to the arch action and geometry, radial stresses are induced in arches and must be checked. The procedure to determine a moment resistance is limited by radial stress and is identical to that presented for curved beams.

9.12.3 Design Resources

The NDS® [2] provides design procedures essentially identical to that used in the LRFD Specification [5]. A good overview of glued laminated curved beam and arch deign is provided by Kasaguma [14]. Additional and more detailed design information and guidelines for curved, pitched, and tapered beams and arches are provide by the American Institute of Timber Construction (AITC) in numerous design aids as well as in the *Timber Construction Manual* [3].

9.13 Serviceability Considerations

Previous discussion in this section has focused entirely on strength design (i.e., strength resistance), which is intended to insure adequate safeguard against structural collapse. As with all types of structure, the serviceability performance of a timber structure is critical, not from a safety viewpoint, but rather for occupant satisfaction. Structural serviceability considerations (e.g., deflections, vibrations, etc.) often govern the design of a timber structure. Additionally, the lack of designer attention to detailing, especially detailing which protects wood from moisture, ultraviolet, and/or insect attack, often leads to performance and even structural problems.

9.13.1 Deflections

The most common serviceability design check is to compare the static (short-term) deflection resulting from service loads to some limiting quantity. Historically, this limit has been *span*/360 for floors subjected to a full uniform floor live load and *span*/240 for roof members subjected to roof live load. Model and local building codes may delineate more specific limits for static deflections.

To calculate the deflection used to compare to the limiting value, the appropriate design load, span, and material properties must be used. The following unfactored load combinations are used to determine the deflection:

$$D + L \tag{9.97}$$

$$D + 0.5S \tag{9.98}$$

where D, L, and S are the nominal design dead, live, and snow loads. The design span is taken as the clear span plus one-half the required bearing length at each support, and the adjusted modulus of elasticity, E', is used for serviceability calculations. The reference modulus of elasticity, E, for lumber is a mean value and includes a 3% reduction to account for shear effects; therefore, if shear deflections are calculated separately, it is appropriate to increase the reference value by 3%. For glued laminated timber, E includes a 5% decrease for shear. In some critical applications, it may be appropriate to use a fifth-percentile value of E rather than a mean value. In this case, E_{05} can be determined as presented in Equation 9.14.

In addition to short-term deflections, the effects of creep must be considered in the design of timber structures. The LRFD suggests using the following load combination when considering creep:

$$D + 0.5L \tag{9.99}$$

where it is assumed approximately half the live load has a sustained component which may attribute to creep. In addition, the LRFD suggests increasing the calculated deflection (using the above load combination) by 50 to 100%. A creep multiplier of 1.5 is suggested for glued laminated timber and seasoned sawn lumber, and a factor of 2.0 is used for unseasoned lumber. Recent research by Fridley [12] and Philpot et al. [16] provide a more complete treatment of creep for design, including various creep factors dependent on the load combination and reference period (time) under consideration. Regardless, the multiplier is used to magnify the calculated elastic deflection to account for creep effects.

9.13.2 Vibrations

Current serviceability design of wood joist floor systems is based on static deflection checks as discussed previously. In the U.S., there are no set guidelines to account for floor vibrations. This often leads to objectionable vibrations due to occupant loading. With the use of I-joists and other new, lightweight products, increasing spans can be achieved along with a decrease in the weight of the floor. These aspects tend to cause an increase in the number of complaints about annoying vibrations. Manufactures of proprietary products (e.g., wood I-joists) have moved toward providing span tables for serviceability which address vibrations. Often, this is as simple as limiting deflection to *span*/480 rather than *span*/360. A number of more rigorous methods for vibration design have been presented in the literature (e.g., see Kalkert et al. [13]). The LRFD, in its commentary, reviews several of these approaches, but makes no formal recommendation as to a preferred method of analysis or design.

9.13.3 NDS® Provisions

The LRFD adopted the same serviceability deflection design methodology as that provided by NDS® with respect to static (or short-term) and creep (or long-term) deflection. Being an ASD specification, the NDS® does not, however, provide separate load combinations for serviceability design. Rather, the same working/service loads are used for strength considerations. The NDS® offers no recommendations for vibrations either.

9.13.4 Non-Structural Performance

Often out of the control of the structural designer, the misconstruction and mis-installation of wood, wood materials, and building envelops can effectively shorten the life of a timber structure. From moisture intrusion to insect attack and decay, wood must be protected, or at least guarded from detrimental environments and conditions. A complete presentation of detailing to avoid problems is beyond the scope of this section, but Dost and Botsai [10] provide a comprehensive guide to detailing timber structures for performance with specific attention to protecting the structure and wood-based materials from deterioration. With proper detailing, and attention to details, a timber structure can enjoy a long and useful life, even in exposed environments.

9.14 Defining Terms

Dimensioned lumber: Solid members from 38 to 89 mm in thickness and 33 mm or greater in width (depth).

Dowel-type fasteners: Fasteners such as bolts, lag screws, wood screws, nails, spikes, etc.

Dry service: Structures wherein the maximum equilibrium moisture content of the wood does not exceed 19%.

Equilibrium moisture content: The moisture content at which wood neither gains moisture from nor loses moisture to the surrounding air.

Fiber saturation point: The point at which free water in the cell cavities is completely evaporated, but the cell walls are still saturated.

Glued laminated timber (Glulam): A composite wood member with wood laminations bonded together with adhesive and with the grain of all laminations oriented parallel longitudinally.

Grade: Classification of structural wood products with reference to strengh and use.

Green lumber: Lumber that has a moisture content exceeding 19%.

I-joists: Prefabricated manufactured members using solid or composite lumber flanges and structural panel webs.

Laminated veneer lumber (LVL): Composite lumber products comprised of wood veneer sheets oriented with the grain parallel to the lenght of the member.

Load-duration (time-effect): The strenght of wood is a function of the duration and/or rate the load is 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 a wood member is to support that load.

Machine-evaluated lumber (MEL): Lumber non-destructively evaluated and classified into strength classifications.

Machine stress-rated (MSR) lumber: Lumber non-destructively evaluated and assigned bending strength and modulus of elasticity design values.

Moisture content: The amount of moisture present in wood defined by the weight of water contained in the wood as a percentage of the weight of the oven-dry wood.

Oriented strandboard (OSB): Structural panel comprised of thin flakes oriented in cross-aligned layers.

Parallel strand lumber (PSL): Composite lumber products comprised of wood strands oriented with the grain parallel to the lenght of the member.

Plywood: Structural panel comprised of thin veneers oriented in cross-aligned layers.

Pressure-treated wood: Products treated to make them more resistant to biological attack or environmental conditions.

Repetitive member systems: A system such as a floor, roof, or wall assembly which consists of closely spaced parallel members exhibiting load sharing among the members.

Seasoned lumber: Lumber that has been dried to a maximum of 19% moisture content.

Shear plate: Circular plate embedded between two wood members or a wood member and a steel side plate with a single bolt to transfer shear from the wood to the bolt.

Sheathing: Lumber or structural panel attached to framing members, typically forming a wall, floor, or roof.

Span rating: Index used for structural panels identifying the maximum spacing of supporting framing members for roof, floor, and/or wall applications.

Split ring: Metal ring embedded into adjacent wood members to transmit shear force between them.

Stressed skin panel: Type of construction wherein the framing members and sheathing act as a composite system to resist load.

Structural composite lumber (SCL): Composite lumber products typically comprised of wood strands or veneers oriented with the grain parallel to the length of the member.

Stud: Vertical framing member used in walls, typically 2×4 and 2×6.

Timbers: Lumber of greater than 114 mm in its least dimension.

Truss-plate connectors: Light gage steel plates with punched teeth typically used in prefabricated wood truss systems.

Visually graded lumber: Lumber graded visually and assigned engineering design values.

Wet service: Structures wherein in the maximum equilibrium moisture content of the wood exceeds 19%.

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