Load and Resistance Factor Design of Timber Joints:
International Practice and Future Direction

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Abstract: International practice for load and resistance factor design (LRFD) of mechanical structural timber joints is reviewed. Attention is on design provisions in the U.S., Canada, Europe, and Australia, those being locations where LRFD codes for wood construction exist. There are broad similarities between various codes, in that all countries adopt an element-based approach to design. Most codes base capacity design checks for joints with dowel-type fasteners on the so-called European yield model. There are a number of systematic differences in detailed implementation of LRFD concepts between countries. No country has yet used structural reliability concepts in derivation and/or calibration of design equations for joints as the level of safety cannot be formally assessed except for relatively simple problems. This contrasts with the situation for members, as several countries have already implemented reliability concepts in design of wood members. Thus, there is imbalance in the principles of design for members and joint in timber systems. Suggestions are made regarding actions necessary to place member and joint design on an equal footing.


CE Database keywords: Load and resistance design; Joints; Wood.

Introduction

It is often said that: “A structure is a constructed assembly of joints separated by members” (after McLain 1998). Joints are often the most critical components of any engineered structure and can govern the overall strength, serviceability, durability, and fire resistance. Assessments of timber buildings damaged after extreme wind and earthquake events often point to inadequate connections as the primary cause of damage (Foliente 1998).

Most parts of the world have available timber design codes predicated on the limit states design (LSD) approach and implemented via a partial factors format. The implementation in the United States is termed the load and resistance factor design (LRFD). Originally, LRFD methods were implemented as “soft conversions” of allowable stress design (ASD) methods, but attention is shifting to structural reliability (probability-based) calibration techniques. All contemporary LRFD codes for timber are written assuming an element by element design process for “sizing” components, with forces calculated assuming linear elastic system behavior. Essential requirements are that the entire structural system is statically stable, individual elements meet strength and stiffness requirements, and global deflections do not exceed appropriate limits. Currently, LRFD codes in the U.S., Canada, and Australia employ structural reliability concepts as the basis of LRFD design of timber members (ASCE 1996; CSA 1994; Standards Australia 1997). This advance was possible following collection of extensive data to characterize statistical variation in mechanical properties of components of lumber, glued-laminated timber, or wood-based panels. Data were combined with statistical knowledge of load effects to generate the resistance factors for members (Foschi et al. 1989; Foschi 2000). Timber codes have been revised in recent years to reflect improved data and information on joints, but no code in the world has implemented reliability-based design of joints. LRFD of joints is universally soft-converted from the ASD format.

Reasons for the imbalance in progress of design provisions for members and joints reflects the inherent relative simplicity of characterizing mechanical behavior of members, as compared to joints. Joints have infinite variety in arrangement, which usually precludes the option of testing large numbers of replicates for reliable statistical representation of strength or stiffness characteristics. The same is not true for members which have been characterized through representative sampling and testing in full size (Madsen 1992). Redressing the imbalance between member and joint design methods is important. All the engineering design spent on getting the right member sizes and spans may be for naught, because if buildings are subjected to extreme loads the joints could fail and the building be severely damaged. Conversely, it is possible that joints are both too stiff and too strong relative to the members under present design provisions. If so, the tendency will be for members to fail, which is undesirable because, unlike steel, failure is inherently brittle and can lead to catastrophic system failure. Although not always done, it is quite possible to design ductile connections (Madsen 1998; Rodd 1998).

Fig. 1 shows key elements needed to develop or support a design procedure for timber joints. First requirement is a set of guidelines or standards that specify the type, nature, and scope of testing (blocks A1 and A2, Fig. 1). This is typically specified for certain classes of fasteners (block A1). Alternatively, the test pro-

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procedure can be for the whole joint system (block A2). The second requirement is a method to obtain nominal design values from test data (block B). Ideally, this provides a consistent way of assigning design values at a desired level of confidence based on the number of specimens and variability in test data. Procedures in blocks A1 and A2 are complementary. Test procedures for whole joint systems can be used to establish design properties and/or evaluate the performance of specific joint systems under single or combined loads. They can also be used to test or experimentally validate theories or models of joint performance.

Timber joints can be categorized as those made with dowel-type fasteners, and surface connections (Madsen 1998). Dowel-type fasteners include drift pins, bolts, lag screws, wood screws, nails, spikes, and timber rivets. All types of dowel fasteners are permitted in applications where they resist loads by bending action. This is referred to as lateral loading because force from the members is applied perpendicular to the axis of the fastener. All types other than drift pins are permitted in applications where they resist forces along the axis, referred to as axial loading or withdrawal loading in the case of nails and screws. Axial force resistance capabilities of nails, wood screws, and lag screws are relatively unreliable, and so withdrawal resistance can only be utilized for load combinations of “lesser” cumulative duration (e.g., wind or earthquake loads in Canada). Surface, or skin, connections include those made using “shear-plate,” “split-ring,” or “toothed-plate” connectors. Manufacture of timber connector joints is labor intensive and their popularity is diminishing. Large capacity connections formerly made with connectors are now often made with timber rivets instead (Madsen 1998). Punched-metal-plate connectors and various folded-plate connectors and hangers manufactured from light gauge sheet steel are used to make low-capacity lumber-to-lumber joints.

Not all types of fasteners and connectors are covered by the structural design standards of various countries. Design capacities of proprietary products usually fall outside the scope of written codes. Their use is regulated through “deemed to comply” specifications for small to medium size timber buildings and a process for certification of such products. Site-made glued joints are usually excluded from engineered structures because of concerns about quality control on the “job site.” There are exceptions to this, e.g., in Germany on-site manufacture of large finger joints in glued-laminated timbers is permitted. Glued joints are used extensively in the manufacture of engineered wood components, but this is dealt with via manufacturing standards and product certification.

This paper reviews international practice for design of mechanical timber joints, and discusses the scope of work needed to elevate LRFD of joints so it is on a comparable footing to LRFD of timber members. Of the common types of joint, only capacities of those made with generic dowel-type fasteners (nails/spikes, screws, drift pins, and bolts) are dealt with by design codes in all countries. Thus, only use of generic dowel-type fasteners is discussed further in this paper. The presentation covers all elements of the design method development process as defined in Fig. 1.

**European Yield Model for Dowel-Fastener Joints**

Currently, design codes use the behavior of a joint with one fastener as the reference condition from which to estimate behavior of structural connections with several or many fasteners. Predicting the behavior of joints with one dowel-type fastener is, therefore, very important. One-fastener joints usually exhibit ductile behavior involving bearing failure of wood beneath the fastener. If the fastener is slender there is also bending failure in the fastener, Fig. 2. It can be seen from Fig. 2 that inelastic deformation governs the ultimate load behavior. This provides a basis for relatively simple mechanics-based prediction of joint capacities.

Codes in the U.S., Canada, and Europe all base the design of joints with dowel-type fasteners on Johansen’s yield model (Johansen 1949; Larsen 1979). This model presumes both the fastener and the wood foundation upon which it bears behave as ideal rigid plastic materials. Model assumptions are the same as for the well-known limit analysis (plastic design) theory applied to steel frames. Johansen’s theory is referred to as the European yield model (EYM) in North America. Figs. 3 and 4 show possible modes of failure for a single fastener joint loaded in single shear and double shear, respectively. For the purposes of this paper, the left member is regarded as the head-side member and the right as the point-side member for single-shear joints with nails, spikes, or screws. Geometric variables are fastener diameter \(d\) and length of penetration in each member \((l, a_i)\). Joint capacity increases as thickness of the members is increased up to the point where a mode IV failure occurs. Neglecting force other than that
normal to the axis of the fastener, the yield load (per joint plane) for a joint, \( P_Y \), can be determined for any mode from static equilibrium, or from the principle of virtual work. The governing mode is that giving the lowest estimate of \( P_Y \). Because the fastener is presumed to be rigid plastic, it can only translate and/or rotate as a rigid body (or segments of it can behave as rigid bodies for modes where one or more plastic hinges form in the fastener). Failure can involve bearing failure in just one member or formation of a plastic hinge in just one member. There are six potential failure equations to consider for two-member joints, and four equations in the case of three-member joints. The Appendix gives the equations for the EYM as developed by Johansen (1949).

Currently, design codes in various countries apply one or more versions of the EYM. It is presumed that capacities of joints are not sensitive to possible restraint at the “head end” of nails, spikes, screws, and bolts, or at the “nut end” of bolts. This potentially leads to quite conservative predictions in the circumstance where mode II (two-member joints only) or III\(_5\) governs.

### Overview of LRFD Methods for Timber Joints

All written LRFD timber design codes specify that joints be designed according to the generic capacity design equation:

\[
(\phi R_j) \geq N^* 
\]

where

\[
(\phi R_j) = \phi k_1 \cdot k_2 \cdot k_3 R_k 
\]

and \( R_j \) = resistance adjusted for all design specific considerations; \( \phi \) = resistance factor; \( k_1 \) = modification factor; \( n_f \) = number of fasteners; \( R_k \) = characteristic resistance per fastener; and \( N^* \) = factored design effect. \( R_k \) values are near minimum, usually the fifth percentile, or mean values, determined at reference conditions. \( \phi \) values are assigned based on past experience to achieve “traditionally acceptable” solutions in terms of the number of fasteners for various end-use applications (DeGrace 1986; McLain 1984). Modification factors account for any deviations between design conditions and reference conditions for \( R_k \), e.g., duration of loading, climatic conditions. Without exception, modification factors are based on empirical evidence. In the past, most codes based duration of loading modification factors on the “Madison curve” developed from a study on small clear Douglas fir bending specimens (Wood 1951). Currently, the Canadian and U.S. codes apply duration of loading adjustments that were derived from in-grade test data for softwood lumber to everything, including joints (Foschi et al. 1989). The model pan-European code (Eurocode 5, CEN 1995) uses an experience-based composite modification factor that accounts for the interactive effects between duration of load and moisture conditions during service. The Australian code is still based on the Madison curve.

Codes require that certain minimum specifications be met with regard to the arrangement of fasteners in a joint, so as to reduce the possibility of premature brittle failure due to splitting of members. The tacit presumption is that if minimum spacing, end distance, edge distance, and member thickness requirements are all respected, loss of strength will not occur prematurely whether failures are brittle or ductile. There is no explicit guidance in any code about whether brittle failure is likely for a particular joint design. LRFD of joints, unlike LRFD of members, does not usually explicitly address the question of deformation (slip). Focus is on the above-mentioned strength limit state calculations. The logic underpinning this is that the deformations of most concern are due to take up of “initial slack,” e.g., take up of tolerance in bolt holes. Such movements are often eliminated during construction as self-weight of the structural system increases. For structural arrangements with the possibility of load reversals, good practice is to not use types of joints that produce initial slack. Irrespective of the argument just advanced, design codes provide information on the calculation of slip in joints. For example, the Canadian CSA Standard 086.1 (CSA 1994) and Eurocode 5 (CEN 1995) provide guidance on calculation of slip in joints as a function of the service environment and the duration of loading. Slip information enables designers to, for example, estimate localized relative movement between timber components that might cause cracking of nonstructural overlays, or estimation of deflections in mechanically laminated components.

Mechanical moment connections are notoriously difficult to make in timber and it can be argued that their use invalidates assumptions underpinning codes premised on the linear elastic structural response of systems (Larsen 1998). Currently, most codes avoid the subject of mechanical moment connections. This presumes timber frameworks will be braced by triangulation or using in-fill panels made of wood-based or other materials. However, there has been much attention in Europe, Australia, and New Zealand to mechanical moment connections over the last decade with potential systems having been proven at full scale in the laboratory and applied in prototype structures (Rodd 1998).

The discussion in the remainder of this section focuses on strength design provisions in the U.S., Canada, Europe, and Australia.

### U.S. Code

The vast majority of engineers design timber structures based on the ASD approach. For splice joints with dowel-type fasteners the allowable load is a fraction of the “proportional limit” load or an approximation to it (USDA 1987). Proportional limit load, as observed in a short-term monotonic load test, is believed to be the point at which irreversible damage processes begin. Unadjusted allowable loads apply to a single fastener/connector subjected to normal duration (10 year) loading, and a dry service condition. Concepts and much of the data that underpin working stress de-
Design of joints are the result of research at the U.S. Forest Products Laboratory in the 1930s and 1940s (USDA 1987). It is generally accepted that ASD embodies inconsistencies in capacities for different types of fasteners (McLain 1993). For most types of mechanical joints the ultimate (peak) load lies well above the estimated proportional limit load. As an example, the ratio of ultimate to proportional limit load can approach 3.5 for nailed softwood joints and 7.0 for nailed hardwood joints (USDA 1987). Joints are often strong and stiff elements within structural systems and overloading will tend to cause failures in members (Smith 1998). Factors of safety are inconsistent across various types of joints and cannot actually be defined, as there is no means of separating global adjustments from test to design capacities into their constituent parts.

Major changes have been made to design provisions for joints with dowel-type fasteners (AFPA 1999). Capacities are now based on the EYM. The ASD code “National Design Specifications for Wood Construction” (NDS) was last published in 1997 (AFPA 1997). Alternatively, the American Society of Civil Engineers has published a “Standard for Load and Resistance Factor Design for Engineered Wood Construction” that is technically equivalent to international LRFD codes (ASCE 1996). As in other countries, “partial factors” on the resistance side of the design equation are assigned based on committee judgement. Partial factors on the load side of the design equation are based on reliability analysis. These are the same for all structural materials (Elingwood et al. 1980). Although the specifies of the LRFD and ASD methods differ slightly (Pellicane 2000), the technical basis of the two codes is similar with respect to joints. Member design by contrast differs significantly between the two documents. Research background and rationale underpinning much of the U.S. code provisions for timber joints have been discussed elsewhere (McLain 1984; Task Committee... 1996). Use of any design code becomes mandatory once it is referenced in the building code for a political jurisdiction. Most jurisdictions are regional and adopt one of several “model” building codes, e.g., Uniform Building Code. Some large cities have their own building codes. Building codes reference both ASD and LRFD methods (or just ASD if they have not recently been updated). Normally, the two methods are not mixed in design of any one structure, but the practice is permitted (Pellicane 2000). Eventually, the ASD should cease to be referenced and, therefore, disappear from use. Informal information implies that at present very limited use is made of LRFD in the U.S.

U.S. yield load equations (AFPA 1997) allow for gaps between members and variation in the fastener yield moment between members. Neglecting differences in notations, they are otherwise the same as equations of the “original EYM” (Appendix). As defined in the U.S., the yield limit load and material properties that enter the equations have a very specific and unique meaning. Fig. 5. Embedment strength of wood beneath a fastener and the yield moment for a fastener are 5% offset values. Values are intermediate between proportional limit and ultimate loads. The product of EYM calculations is the nominal capacity for a joint with one fastener. In the NDS, nominal design capacity is divided by a factor greater than 1. The magnitude of the factor differs depending upon the type of fastener. For bolts, drift pins, and lag screws, adjustments from nominal capacities are a function of the EYM mode that governs, and the maximum angle to the grain at which the fastener loads any timber member. For nails, spikes, and wood screws, adjustments depend only upon the fastener diameter. The purpose of these adjustments is to give capacities representative of nominal proportional limit based design capacities in earlier editions of the specification. For slender and relatively flexible fasteners such as nails or screws, it is presumed that the capacity of a joint is linearly proportional to the number of nails. For joints having a number of bolts, drift pins, or lag screws arranged in a row parallel to the direction of the load, there is an uneven distribution of force between the fasteners. This promotes premature failure because, when a first fastener reaches its capacity (local failure) the other fasteners usually cannot accommodate the redistributed force, triggering an unstable unzipping in the row (Tan and Smith 1999). Adjustment factors allowing for this are based on simplified analogues where an assembly of linear elastic springs represent fasteners and member segments between them (Cramer 1968; Lantos 1967; Zahn 1991). Recent research evidence indicates the NDS method for accounting for the effect of the number of fasteners is nonconservative, certainly in the case of bolted joints (Mohammad et al. 1997). It should be remembered, however, that the U.S. practice is actually calibrated to give, in a global sense, traditionally accepted solutions.

In the U.S., American Society for Testing and Materials (ASTM) standards for testing complete joints and their components support the NDS and ASCE design codes. There is a lack of guidance for testing whole joint systems under a variety of loads that might be expected in service (McLain 1998). Also, there is no explicit method for evaluating characteristic values, $R_k$, and nominal design values from experiments on timber joints (block B, Fig. 1). Since there are typically not more than ten replicates in joint tests, nonparametric methods such as those used for evaluating characteristic strength properties of lumber (ASTM 1996) may not be appropriate. A potentially suitable method is that developed by Leicester (1986) in Australia (discussed later).

**Canadian Code**

Like the U.S., Canada permits engineers to design to either ASD (CSA 1984a) or LRFD (CSA 1994). However, it has been accepted for some time that national, provincial, and municipal building codes will cease to reference ASD. This is expected to become established fact by 2003. The ASD provisions are very similar to pre-1991 provisions in the U.S.

The first LRFD code for timber structures was introduced in 1984 (CSA 1984b). For timber members this was a soft conversion from the allowable stress method, although account was taken of “in-grade” test data for small dimension lumber. There was a fundamental shift with regard to joint design (DeGrace 1986). Data from various sources were reanalyzed so that joint capacities would reflect the ultimate (strength) and serviceability limit state considerations rather than being related to the propor-
The limit state design equation for joints takes the form:

\[ \phi R_j = \sum \alpha_{F_i} F_i \]  

(2)

where \( R_j \) = resistance; \( \alpha_{F_i} \) = load factor; and \( F_i \) = nonfactored load effects. Despite selective refinements, code provisions for joints remain essentially unaltered since 1984 (Lepper and Smith 1995).

EYM yield load equations for bolt, drift pin, and lag screw joints are given in the CSA Standard 086.1 “Engineering Design in Wood (Limit States Design)” (CSA 1994). Equations used are an empirical approximation for some modes, rather than original EYM equations (Whale et al. 1987). Embedment properties for timber members are calculated as a function of fastener diameter, wood density, and angle of loading relative to the grain, based on tests on bolts (Smith et al. 1988; Whale and Smith 1986; 1987; 1989). The values represent the ultimate (peak) bearing capacity, Fig. 6. Thus, the embedment values adopted in Canada do not have the same basis as those in the U.S. Embedment properties for wood members are related to the short-term ultimate strength of the material. Fastener yield moments are estimated from data in the literature (McLain and Thangjitham 1983), and minimum permitted yield strength for the grade of bolt used. The plastic moment capacity of a fastener with circular cross section is taken to be

\[ M_y = \sigma_y d^3/6 \]  

(3)

where \( \sigma_y \) = yield stress and \( d^3/6 \) = plastic section modulus. The output from the EYM calculation is an estimated fifth percentile short-term ultimate strength. An adjustment factor of 0.8 is embedded within the CSA equations to convert from short-term test to “standard term” loading. Its value is the same irrespective of which EYM mode governs. There is no specific load duration associated with standard term loading. The factor 0.8 is derived from reliability studies on lumber components and extrapolated to joints. Its purpose is to scale the resistance factor when account is taken of duration of load effect, under dead plus live snow or dead plus occupancy loads, versus when it is neglected (Foschi et al. 1989). The value 0.8 is hard to justify for joints where failure involves both the fastener and the wood, e.g., slender bolts.

Tabulated specified resistances for nail and spike joints are based on the original EYM, assuming a single shear arrangement and that the nail penetrates two thirds of its length into the point-side member (Keenan et al. 1982). This arbitrary calibration arrangement resulted in either mode III or IV governing, depending upon the nail size and the timber species. Embedment strength and the nail yield moment adopted were based on those suggested by Johansen (1949), with embedment strength taken as 1.1 \( \times \) “near-minimum” short-term compressive strength parallel to the grain. An allowance is made for the so-called string resistance that can develop in nails after large displacement and intermember friction (Keenan et al. 1982). These effects inflate specified resistances by up to about 30%. Recent work indicates that neither the string effect or intermember friction should be employed (Smith et al. 2001).

For nails and spikes it is presumed that the capacity of a joint is linearly proportional to the number of nails. However, for joints having a number of bolts, drift pins, or lag screws, capacities are adjusted to account for uneven force distribution. Prior to the 1989 edition of CSA Standard 086.1, the approach for accounting for this was the same as in the NDS (AFPA 1997). Since then, for axially loaded members empirical adjustments have been established to account for the number of bolts in a row, the loaded end distance (distance between the last fastener and the end of the member), and the number of rows of fasteners. Only the “number of rows” adjustment is applied when bolts load the timber member(s) perpendicular to the grain. Essentially, these were an emergency measure, in response to data from two ad hoc series of tensile tests on steel—timber—steel bolted joints having one or more rows of bolts aligned parallel to the axis of a glued-laminated-timber member (Masse et al. 1989; Yasumura et al. 1987). The “group effect” adjustments are rather constraining and preclude use of connections with a large number of bolts (Smith 1994).

Unlike other countries, Canada does not produce test standards in support of its timber design codes. Reliance is placed on the code committee’s judgment to select appropriate background information and data. There is no standard method to evaluate characteristic values and nominal values for design of joints (block B, Fig. 1). The CSA 086 Technical Committee is in the process of a major overhaul of provisions pertaining to joints. Their objectives include elimination of inconsistencies in the approaches used to assign capacities to different types of fasteners, making the nature of the predicted failure mode transparent, and adoption of mechanics-based methods whenever feasible.

**Eurocode 5**

In Europe, the situation is somewhat complicated as many national and multinational code writing bodies are involved. The European Committee for Standardisation (CEN) based on Belgium has published a model design code “Structural Timber Design Code” with the designation Eurocode 5 (CEN 1995), which is in LRFD format. Eurocodes are model codes developed by representatives from member states of the European Economic Community and some neighboring countries. Various countries within Europe have developed/are developing National Application Documents that meld Eurocode 5 with local needs, tradition, and design practices. Although such documents are available, it does not mean that they have yet superseded prior codes within...
all political jurisdictions. It is understood that both ASD and LSD methods are used in Europe. Scandinavia was the first region to embrace LRFD. This paper takes a somewhat liberal approach in describing the state of the “design art” because of the difficulty in saying what exactly is the situation. Design equation constants such as partial coefficients are assigned values by the appropriate body within any country via the National Application Document. Values suggested in Eurocode 5 are purely guides to code writing bodies. Conceptually, the form of the design equation for a member or joint is

\[ R_d = \sum \gamma F_i / F_i \]  

where \( R_d \) = design resistance = \( R_k \cdot k_{mod} / \gamma_m \); \( R_k \) = characteristic resistance; \( k_{mod} \) = composite modification factor; \( \gamma_m \) = material resistance partial coefficient; and \( \sum \gamma F_i / F_i \) = summation of factored load effects.

Characteristic resistances for dowel-type fastener joints are derived using the original EYM equations (Johansen 1949). As in Canada, embedment strengths of members are based on the maximum bearing resistance, Fig. 6. Embedment properties are a function of fastener diameter, wood density, and angle of loading relative to the grain (except for nails). Fastener characteristic yield moment is determined by testing, or by calculation, assuming

\[ M_{Y,k} = (\sigma_{U,k} + \sigma_{Y,k})d^3/12 \]  

where \( \sigma_{U,k} \) = characteristic ultimate strength in tension and \( \sigma_{Y,k} \) = characteristic yield strength in tension. Values of embedment strength, \( s_h \), entering the EYM equations are first adjusted to account for changes in properties from reference to design conditions. Both \( s_{H,d} \) and \( M_Y \) include a partial safety factor. Assuming use of timber for all members and a steel dowel fastener, the approach is as follows:

\[ s_{H,d} = k_{mod} s_h / \gamma_m \]  

\[ M_Y = M_{Y,k} / \gamma_m \]  

where \( s_{H,d} \) = design embedment strength for a fastener of diameter \( d \); \( s_{H,k} \) = characteristic embedment strength; \( \gamma_m \) = material partial coefficient for wood (1.0, typical value 1.3); \( M_Y \) = design moment resistance; \( M_{Y,k} \) = characteristic moment resistance; and \( \gamma_m \) = material partial coefficient for steel (1.0, typical value 1.1). Thus, for mode I and II failures, Eqs. 3 and 4, \( R_d / R_k = k_{mod} / \gamma_m \), while for mode IV failures \( R_d / R_k = (k_{mod} / \gamma_m) \) \( 1/2 \). The ratio \( R_d / R_k \) is intermediate for mode III failures. Although details of the EYM approach are not the same as in non-European countries, there are similarities to the approach taken in the U.S. In both Europe and the U.S., adjustments for nonreference conditions depend upon the slender-ness of the fastener. The product of the calculations is the design resistance of a joint with one fastener. For bolt and dowel (drift pin) joints with fasteners arranged in a row parallel to the force in a member, the capacity is discounted in proportion to the number of fasteners if there are more than six:

Effective number of fasteners = \( n_{ef} = 6 + 2(n - 6)/3 \)  

where \( n \) = number of fasteners aligned in a row. This is a nonconservative reduction rule compared with those used elsewhere. The Eurocode 5 rule is based on judgment rather than data. More sophisticated approaches are permitted, provided that they are at least as stringent. For nailed joints, \( n_{ef} \) it taken to equal \( n \).

The composite modification factor \( k_{mod} \) is arrived at in a manner unique to Eurocode 5 as the factor accounts for interaction between the load and moisture conditions in service. Table 1 shows a curtailed set of \( k_{mod} \) values. Entries in Table 1 are based on ad hoc test data and experience.

In Europe, a raft of test standards published by CEN provides support to Eurocode 5 and derived national design codes. Like in the U.S., there is no standard for testing whole joint systems under a variety of loads that might be expected in service. There is no standard method to evaluate characteristic values and nominal values for design of joints (block B, Fig. 1), but this is not surprising bearing in mind that it is local prerogative within each country.

Eurocode 5 has much to commend its approach to the design of joints. It is logical to incorporate modification factors for moisture condition and duration of load within the embedment strength that enters EYM calculations [Eq. (6)]. However, it is not easy to argue the merits of incorporating partial safety coefficients in calculation of material properties [Eqs. (6) and (7)]. This is only justified if variability in resistances for joints is proportional to the relationships between joint yield load \( P_y \) and \( s_y \) and \( M_Y \) implied by the EYM equations. The coupling between the effects of service class (moisture conditions) and the duration of loading class is philosophically correct, and perhaps the most innovative aspect of Eurocode 5. The use of five duration of loading classes upholds long-term practices and contrasts with Canadian practice where only three loading classes were found necessary based on reliability analysis by Foschi et al. (1989).

Fuller details of joint provisions in Eurocode 5 are given elsewhere (Blass et al. 1995; Whale et al. 1987).

### Australian Code

In Australia, design in timber is in accordance with the standard AS1720.1 “Timber Structures, Part 1: Design Methods” (Standards Australia 1997). Joint provisions of that document are based solely on an empirical fit of test data. As far as joints are concerned, the code is essentially the same as in the previous ASD version (Lhuede 1988). Background studies are largely foreign work in the first half of the twentieth century, supplemented by studies on Australian timbers, particularly hardwoods. The current design rules are primarily due to the work of Mack (1978; 1979) and Lhuede (1988; 1990).

### Table 1. Values of Composite Modification Factor \( k_{mod} \) for Solid and Glued-Laminated Timber and Plywood [Based on Eurocode 5 (CEN 1995)]

<table>
<thead>
<tr>
<th>Load duration class</th>
<th>Services class 1 (MC&lt;12%)</th>
<th>Services class 2 (12%&lt;MC&lt;20%)</th>
<th>Services class 3 (MC&gt;20%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent (&gt;10 years)</td>
<td>0.60</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>Long term (6 months to 10 years)</td>
<td>0.70</td>
<td>0.70</td>
<td>0.55</td>
</tr>
<tr>
<td>Medium term (1 week to 6 months)</td>
<td>0.80</td>
<td>0.80</td>
<td>0.65</td>
</tr>
<tr>
<td>Short term (&lt;1 week)</td>
<td>0.90</td>
<td>0.90</td>
<td>0.70</td>
</tr>
<tr>
<td>Instantaneous</td>
<td>1.10</td>
<td>1.10</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Note: MC = moisture content typical of solid timber in the service class.
Because design procedures are based on empirical rules, their applicability to new products/materials may be questionable. In the last several decades there has been a shift in the house framing market from use of unseasoned hardwoods to seasoned softwoods. There has also been introduction of a wider variety of fasteners, new sheathing panels, and composite timber products that is not reflected by the code. This sparked interest in the EYM for dowel-type fasteners, but as yet, it is not adopted. Stringer (1993) examined the applicability of the EYM to nailed timber joints using four Australian timber species, plywood from slash pine veneers, and Radiata pine LVL. Recently, an extensive experimental program has been conducted to develop new joint design procedures for Australian pine based on EYM equations (Foliente et al. 2001).

Standards Australia publishes test standards in support of the design code (AS1720.1). Supporting standards fulfill the same function as ASTM standards in the U.S. The joint Australian–New Zealand draft standard AS/NZS BBBB (Standards Australia 1998) is actually a set of standards for evaluating complete joint systems that specifies actual in-service joint configuration, specifies realistic in-service loads, has a consistent method of obtaining characteristic strength, and has a consistent method of applying load factors to obtain joint design properties. The spirit of this set of joint systems standards is that it may be used to test any joint under any loading configuration but it is not necessary to test every joint under every loading configuration (Foliente and Leicester 1996; Foliente et al. 1998).

With regards to the method of establishing characteristic values and nominal design values (block B, Fig. 1), when sample size $N$ is equal to or greater than 10, AS BBBB uses a method due to Leicester (1986):

$$R_{k,\text{est}} = R_{0.05\text{data}} \left[ 1 - 2.7 (V/N^{1/2}) \right]$$

(9)

where $R_{k,\text{est}}$ = characteristic value estimated with 75% confidence level, $R_{0.05\text{data}}$ = fifth percentile estimated from the data, and $V$ = coefficient of variation estimated from the data. For $N < 10$, $R_{k,\text{est}}$ is computed as: $R_{k,\text{est}} = R_{\min}(N/27)^{1/2}$, where $R_{\min}$ is the minimum value in the sample. Then, a load factor, which is also a function of $V$, is multiplied to $R_{k,\text{est}}$ to obtain the nominated load capacity of the joint.

In evaluating methods to establish uplift design values for metal connectors, Rosowsky et al. (1998) noted the need to explicitly consider variability of the test data and the assumed probability distribution. Bryant and Hunt (1999) conducted Monte Carlo simulation to evaluate different methods of computing characteristic values for joints. They found that the “Leicester method” provides good predictions even for reasonably small sample sizes. The intent of the method is that the coefficient of variation be determined from the lower tail of test distribution. $R_{k,\text{est}}$ tends to be conservative if the coefficient of variation is calculated from all the test data. Simplicity and reasonable accuracy of Eq. (9) argue in favor of its adoption elsewhere in the world.

Summary of Issues

Mechanics-based formulas allow designers to account for the many arrangement-specific parameters that affect the strength and failure mode of a joint. They are preferable to traditional empirically based design equations and facilitate introduction and/or evolution of products. There is international acceptance of this premise and consensus that EYM-type calculations are an appropriate means of estimating capacities of joints with one dowel-type fastener. There is, however, little uniformity regarding specifics of implementing EYM equations in design codes or test methods for estimating input properties. Divergence in practices between countries or regions of the world seems unnecessary. European test standards have or are being “fast tracked” to become ISO standards and many countries are now beginning to actively participate in drafting ISO timber standards. Application of the EYM may not be completely harmonized in the near future but there are promising moves in that direction.

There is a need for a rational and consistent method of establishing characteristic values and nominal values for design of joints. Direct adoption of material sampling and statistical analysis methods applicable to timber/lumber may not be appropriate. Because a product such as lumber is a commodity, it is feasible to sample the population in a representative manner with typically hundreds of replicates per “test cell.” Even for joints made with one type of fastener there is need to characterize the strength of many possible arrangements under a range of loading scenarios. Statistical distributions that best represent joint strength are not always the same as those used to represent distributions of member strength (Smith 1982). A desirable approach for joints is to combine statistical characterization of component properties (e.g., fastener yield moment and member embedment strength) with methods for predicting variability in joint capacities from information about properties of components.

Few structural connections are made with one dowel-type fastener. Except for nailed joints for which it is generally accepted that there is a sensibly linear relationship between ultimate capacity and the number of nails, calculation of capacities of joints with multiple fasteners is an issue upon which there is almost no international consensus. There can be transition from ductile to brittle failure when the number of fasteners is increased. Furthermore, variability in strength is a function of the number of fasteners. It is important in design against extreme load events such as hurricanes and earthquakes to be able to predict if under a particular design solution, components, including the joints, are likely to exhibit a brittle or ductile failure. Transparency of failure modes is another compelling argument in favor of the shift to mechanics based models.

Modification factors used to adjust from reference to service conditions are not well established. Adjustments associated with duration of loading and moisture content are of primary importance (Leicester and Lhue 1992; McLain 1998). Ad hoc test evidence indicates that there are substantial interactions between moisture conditioning between fabrication and application of load, moisture fluctuations during loading, and duration of the loading (Mohammad and Smith 1994; 1996). Currently, only Eurocode 5 links moisture class and duration of loading. It is a practice that should be followed elsewhere.

Because not all joint systems are amenable to mechanics-based modeling, there is need for “blackbox” methods of assessing and assigning design properties to whole joint systems. Limited blackbox methods are available in the U.S. and Europe. In Australia and New Zealand, draft standard AS BBBB can be used to evaluate specific fasteners or whole joint systems that are required to transmit a range of load types (e.g., moment, shear) acting alone or in combination at a range of intensity.

Overarching all of the above is the need to incorporate probabilistic concepts in calibrating the parameters in LRFD of joints. It is necessary as part of this to decide whether joints are required to be weak or strong links in timber systems, and whether design is to be member or system based. Once this is done, it should be...
relatively straightforward in principle to calibrate parameters because structural reliability methods are well established for these purposes (Melchers 1999; Foschi 2000). There have already been preliminary studies based on first- and second-order reliability methods to characterize reliability of simple wood joints (e.g., Smith 1985, Zahn 1992). However, general treatment of the topic requires account of issues such as aging/damage accumulation, relaxation and force redistribution, and transitions in failure modes with time under load. Further research is required in these areas.

Tables 2 and 3 summarize specific items that need to be considered in order to raise the technical basis of LRFD methods for wood members. Tables 2 and 3 indicate the authors’ assessment of the status of knowledge and priority they attach to remedying specific deficiencies. Priorities also reflect the perceived impact on design practice and discussion within code committees in Australia and Canada.

### Concluding Comments

Structural reliability analysis has almost reached the stage where it can be applied on a turnkey basis, but its meaningful applica-

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**Table 2.** Items to Be Considered for Probability Based Load and Resistance Factor Design of Joints: Application of European Yield Model (EYM)

<table>
<thead>
<tr>
<th>Items</th>
<th>Status</th>
<th>Priority</th>
<th>Focus in new work</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characterize embedment</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Nails</td>
<td>Significant data for short-term loading.</td>
<td>medium</td>
<td>Mechanosorpitive(^a) effects; longer-term loading; improved nails,(^b) wood composites</td>
</tr>
<tr>
<td>• Wood screws</td>
<td>Some data for short-term loading.</td>
<td>medium</td>
<td>Longer-term loading; wood composites</td>
</tr>
<tr>
<td>• Lag screws</td>
<td>Some data for short-term loading.</td>
<td>medium</td>
<td>Mechanosorpitive(^a) effects; longer-term loading</td>
</tr>
<tr>
<td>• Bolts</td>
<td>Significant data for short-term loading.</td>
<td>medium</td>
<td>Mechanosorpitive(^a) effects; longer-term loading</td>
</tr>
<tr>
<td>• Drift pins</td>
<td>Significant data for short-term loading, especially for European softwood species.</td>
<td>medium</td>
<td>Mechanosorpitive(^a) effects; longer-term loading</td>
</tr>
<tr>
<td>• Test method/ interpretation of data</td>
<td>Standardized methods exist in U.S., Europe, ISO.</td>
<td>high</td>
<td>International harmonization</td>
</tr>
</tbody>
</table>

| Fastener moment capacity | | | |
| • Nails | Significant data. | low | Improved and nonsteel nails |
| • Wood screws | Small data. | medium | All types |
| • Lag screws | Small data. | medium | All types |
| • Bolts | Significant data. | low | Nonsteel bolts (e.g., plastic) |
| • Drift pins | Significant data. | low | Nonsteel bolts (e.g., plastic) |
| • Test method/ interpretation of data | Standardized methods exist in U.S., Europe, ISO. | high | International harmonization |

| Models | | | |
| • Development | Several versions exist. | medium | Simplification, if justified by verification studies |
| • Verification | Extensive verification for joints with drift pins (plain steel dowels) and nails. | high | Assessment of whether all modes are possible for joints with wood screws, lag screws, or bolts |
| • Harmonization | Alternative form of EYM used in various codes. | high | International harmonization |

\(^a\)Mechanosorpitive effect=combined effects of moisture and load histories.
\(^b\)Improved nails=other than plain shank steel wire nails (e.g., twisted, ring-shank, and coated nails).
tion depends upon quality of the input for load effects and material resistance. Presently, knowledge of strength of wood joints is incomplete. Until the most important gaps in knowledge are narrowed, it is doubtful that reliability concepts can improve upon "engineering judgment" of committees responsible for LRFD in timber within various countries. There is need for new work on wood joints and on developing key standards, as has been suggested in this paper. The task is feasible within the next several years, particularly if effort is shared internationally. Hopefully, this will be the case.

Appendix. European Yield Model

The following equations describe the model as originally proposed by Johansen (1949) and popularized by Larsen (1979). Corresponding failure modes are illustrated in Figs. 3 and 4. Subscripts $s$ and $m$ applied to mode numbers indicate in the case of mode I failures whether it is the side or main member, respectively, that fails in bearing. The designation $III_m$ indicates there is only bearing failure in the main member, while there is a plastic hinge in the side member combined with bearing failure of the

<table>
<thead>
<tr>
<th>Item</th>
<th>Status</th>
<th>Priority</th>
<th>Focus in new work</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple-fastener connections</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Nails</td>
<td>Some data.</td>
<td>medium</td>
<td>Improved and nonsteel nails; fatigue and long-term loading.</td>
</tr>
<tr>
<td>• Wood screws</td>
<td>Very limited experimental studies.</td>
<td>medium</td>
<td>All types.</td>
</tr>
<tr>
<td>• Lag screws</td>
<td>Very limited experimental studies.</td>
<td>low</td>
<td>Steel-to-timber connections.</td>
</tr>
<tr>
<td>• Bolts</td>
<td>Significant data for short-term load parallel to grain.</td>
<td>high</td>
<td>Load perpendicular to grain; nonsteel bolts (e.g., plastic); fatigue and long-term loading.</td>
</tr>
<tr>
<td>• Drift pins</td>
<td>Significant data for short-term load parallel to grain, some data for long-term loading.</td>
<td>medium</td>
<td>Load perpendicular to grain; nonsteel bolts (e.g., plastic); fatigue and long-term loading.</td>
</tr>
<tr>
<td>• Test method/interpretation of data</td>
<td>Appropriate standardized methods exist in some countries U.S., Europe, Australia, ISO.</td>
<td>high</td>
<td>Evaluation of whole joint systems; international harmonization.</td>
</tr>
<tr>
<td>Mechanics-based models</td>
<td>Some exist for specific applications.</td>
<td>high</td>
<td>Development of generalized approaches; experimental verification; closed-form methods.</td>
</tr>
<tr>
<td>Reliability studies</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Theory</td>
<td>Generalized theory exists and can be used. Problem is lack of input data for &quot;resistance.&quot;</td>
<td>low</td>
<td>Specific tools for application of generalized theory.</td>
</tr>
<tr>
<td>• Variability in capacities of joints.</td>
<td>Ad hoc data, especially for joints with one fastener.</td>
<td>high</td>
<td>Variability in ultimate capacities of multiple fastener connections; methods for predicting from variability in timber and fastener properties.</td>
</tr>
<tr>
<td>• Calibration of LRFD equations</td>
<td>Soft conversion from ASD. Solutions judged adequate based on experience.</td>
<td>high</td>
<td>Hard conversion to probabilistic basis; reconciliation of discrepancies versus existing design solutions; focus on system rather than component reliability.</td>
</tr>
</tbody>
</table>
The following symbols are used in this paper:

\[ P_y = s \mu dl \]

Two-member joints

\[
\begin{align*}
P_y &= s \mu dl \\
&= \frac{1.0}{\alpha \beta} \sqrt{\beta + 2\beta^2(1 + \alpha + \alpha^2) + \beta^2\alpha^2 - \beta(1 + \alpha)} \\
&\quad \frac{\sqrt{2\beta(1 + \beta) + 4\beta(2 + \beta) M_y}}{s \mu dl^2 - \beta} \\
&\quad \frac{\alpha (\sqrt{2\beta^2(1 + \beta) + 4\beta(1 + 2\beta) M_y}}{s \mu dl^2 - \beta} \\
&\quad \frac{\sqrt{4\beta M_y}}{s \mu dl^2(1 + \beta)}
\end{align*}
\]

Three-member joints

\[
\begin{align*}
P_y &= s \mu dl \\
&= \frac{1.0}{\alpha \beta/2} \sqrt{2\beta(1 + \beta) + 4\beta(2 + \beta) M_y} \\
&\quad \frac{\sqrt{2\beta(1 + \beta) + 4\beta(2 + \beta) M_y}}{s \mu dl^2 - \beta} \\
&\quad \frac{\sqrt{4\beta M_y}}{s \mu dl^2(1 + \beta)}
\end{align*}
\]

Notation

The following symbols are used in this paper:

- \( d \) = fastener diameter;
- \( F_i \) = load effect from wind, snow, etc.;
- \( k_i \) = modification factor accounting for departures from reference conditions for \( R_k \);,
- \( k_{mod} \) = composite modification factor;
- \( l \) = thickness of side member(s);
- \( M_y \) = yield moment of fastener;
- \( N \) = number of samples;
- \( N^* \) = factored design action effect;
- \( n \) = number of fasteners;
- \( n_{ef} \) = effective number of fasteners;
- \( R_d \) = design resistance;
- \( R_k \) = characteristic resistance per fastener;
- \( R_{k,\text{est}} \) = characteristic value estimated with 75% confidence level;
- \( R_j \) = \( \phi k_i \cdot k_i \cdot n_i R_k \) = resistance adjusted to account for all design specific considerations;
- \( R_{\text{min}} \) = minimum strength value in the sample;
- \( R_{0.05\text{data}} \) = fifth percentile strength estimated from the data;
- \( V \) = coefficient of variation estimated from test data;
- \( \alpha \) = ratio of thickness of main/center member to thickness of side member;
- \( \beta \) = ratio of embedment strength of main member to embedment strength of side member;
- \( \gamma_m \) = material resistance partial coefficient = \( \phi^{-1} \);
- \( \gamma_{F_i} \) = partial load coefficient;
- \( \Sigma \gamma_{F_i} F_i \) = factored design action effect; and
- \( \phi \) = resistance factor.

References


Task Committee on Fasteners of Committee on Wood, American Society of Civil Engineers. (1996). Mechanical Connections in Wood Structures, ASCE Manuals and Reports on Engineering Practice No. 84, ASCE, New York.


