Development of Limit States Design Method for Joints with Dowel Type Fasteners

Part 1: Literature Review
Publication: Development of Limit States Design Method for Timber Joints with Dowel Type Fasteners Part 1 Literature Review

The Forest and Wood Products Research and Development Corporation (“FWPRDC”) makes no warranties or assurances with respect to this publication including merchantability, fitness for purpose or otherwise. FWPRDC and all persons associated with it exclude all liability (including liability for negligence) in relation to any opinion, advice or information contained in this publication or for any consequences arising from the use of such opinion, advice or information.

This work is copyright and protected under the Copyright Act 1968 (Cth). All material except the FWPRDC logo may be reproduced in whole or in part, provided that it is not sold or used for commercial benefit and its source (Forest and Wood Products Research and Development Corporation) is acknowledged. Reproduction or copying for other purposes, which is strictly reserved only for the owner or licensee of copyright under the Copyright Act, is prohibited without the prior written consent of the Forest and Wood Products Research and Development Corporation.

Project no: PN02.1908 (Part 1)

Researchers:

I. Smith, G.C. Foliente
CSIRO Building, Construction and Engineering
PO Box 56, Highett, Victoria 3190
Development of Limit States Design Method for Joints with Dowel Type Fasteners

Literature Review

Prepared for the
Forest & Wood Products Research & Development Corporation

by

G. Foliente
I. Smith
## Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXECUTIVE SUMMARY</td>
<td>4</td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>6</td>
</tr>
<tr>
<td>Background</td>
<td>6</td>
</tr>
<tr>
<td>Elements of a Joint Design Method</td>
<td>7</td>
</tr>
<tr>
<td>REVIEW OF JOINT DESIGN STANDARDS</td>
<td>9</td>
</tr>
<tr>
<td>Introduction</td>
<td>9</td>
</tr>
<tr>
<td>Australian Code</td>
<td>11</td>
</tr>
<tr>
<td>US Code</td>
<td>15</td>
</tr>
<tr>
<td>Canadian Code</td>
<td>23</td>
</tr>
<tr>
<td>European Code</td>
<td>28</td>
</tr>
<tr>
<td>Summary of Issues</td>
<td>33</td>
</tr>
<tr>
<td>RECOMMENDATIONS</td>
<td>35</td>
</tr>
<tr>
<td>General</td>
<td>35</td>
</tr>
<tr>
<td>Factors and Parameters to Consider</td>
<td>36</td>
</tr>
<tr>
<td>REFERENCES</td>
<td>38</td>
</tr>
<tr>
<td>APPENDIX A. The European Yield Model</td>
<td>44</td>
</tr>
<tr>
<td>APPENDIX B. Analysis of Stringer's (1991) Results</td>
<td>47</td>
</tr>
<tr>
<td>APPENDIX C. Notations</td>
<td>50</td>
</tr>
</tbody>
</table>
Executive Summary

In this project, we aim to develop a limit states design procedure for timber joints. Towards this goal, we reviewed the current Australian practice, and the design codes in the USA, Canada and Europe, and identified the issues and parameters that need to be considered in an experimental program aimed at evaluating the use of the European Yield Model (EYM) for joints with dowel-type fasteners. We also reviewed the previous research by Stringer (1991) investigating the use of EYM for nailed joints with Australian timbers; Appendix B provides a commentary and re-analysis of his data.

In Phase 2, we will conduct a focused experimental program, after consultation with PRI. With promising results, we will then prepare a draft design procedure in Phase 3 and submit a set of recommendations on required data and activities for full implementation. Before presenting the recommended design procedure, the draft procedure will be trialed in the field by selected practicing engineers. Their comments and recommendations will be taken into account in revising or refining the design procedure.

Based on the review of literature and design standards here and overseas, our primary recommendation is to fully develop the use of EYM for the design of dowel-type fasteners in Australia. As a first step, we should consider nails, and, possibly, slender bolts. This process will involve developing:

- an experimental program to establish properties and apply the EYM concept to a range of variables common in Australian condition; and
- the infrastructure required to support the use of this design procedure.

The latter will include development of guidelines and/or standards to determine the input properties needed in EYM-based design. Despite the amount of development and data available from other countries, a fair amount of work still needs to be done to evaluate its applicability to a wide range of Australian joints with dowel-type fasteners, and to introduce simplifications to make the proposed design procedures easier to use.

In developing LSD procedures for timber joints, we need to determine the appropriate resistance factors, develop a method to estimate joint variability based only on data from fastener and timber components of the joint, and conduct reliability analysis to calibrate the method to accepted levels of performance.
We also recommend further development of the standard for testing whole joint systems under realistic loading conditions; this is currently in draft form as AS/NZS BBBB. The concept is state-of-the-art, and many aspects of this suite of standards provide typically missing elements (evident in the review of overseas codes) that are needed to support design procedures for timber joints.
Introduction

Background

The new Timber Structures Code AS 1720.1 (SAA 1997) provides a limit states design (LSD) procedure for timber construction. Although the original intent was to do only a ‘soft conversion’ from the previous version (SAA 1988), some revisions were nevertheless included to reflect new data and information that have been generated or obtained from timber testing and research since 1988. The code provisions for connections, however, have only been soft-converted to LSD format.

The National Timber Framing Code AS 1684 (SAA 1999), which has been providing ‘deemed to comply’ provisions for timber construction since 1975, has also been revised to reflect new timber engineering technology, following the new LSD code. The new suite of framing standards (now in four volumes) caters for a widening variety of wood-based materials and evolving design conditions. The framing standards incorporated results of an experimental program conducted at CSIRO (Foliente et al. 1996) to address the lack of knowledge associated with joints that are widely used throughout the domestic construction industry but are currently not adequately covered by AS 1720.1, and for which conventional code design procedures cannot be applied.

Since the new AS 1684 framing tables were generated following the provisions of AS 1720.1, a timber building constructed following the new AS 1684 framing tables can be said to be ‘engineered’. For timber members, this is true; but for the joints and connections in that building, this is not the case. All the engineering design spent on getting the right member sizes and spans may be for naught because if the building is subjected to extreme loads, the joints could fail and the structure could be severely damaged.

Joint systems are typically one of the most critical components of any structure. They can govern the overall strength and serviceability of structural systems, and should be given primary considerations in designing or detailing for 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). Because of the importance of joints in the overall performance of buildings, it is often said that:

‘A structure is a constructed assembly of joints separated by members.’
In this project, we aim to develop a limit states design procedure for timber joints. Towards this goal, we planned to revisit the current Australian practice, review the design practice in North America and Europe, and plan an experimental program to evaluate the use of the European Yield Model (EYM) for joints with dowel-type fasteners. With promising results, we will then prepare a draft design procedure and submit a set of recommendations on required data and activities for full implementation. Before presenting the recommended design procedure, the draft procedure will be trialed in the field by selected practicing engineers. Their comments and recommendations will be taken into account in revising or refining the design procedure.

This report covers the review of Australian and overseas practice, and includes general recommendations on the scope of the experimental program proposed for the next phase of the project.

**Elements of A Joint Design Method**

Any design method requires a number of supporting guidelines/documents and standards. Figure 1 shows some of the key elements needed to develop or support a design procedure for timber joints. First, a set of guidelines or standards that specify the type, nature and extent/scope of testing should be in place (blocks A1 and A2 in Fig. 1). This is typically specified for specific classes of fasteners (block A1). Alternatively, the test procedure can be for the whole joint system (block A2). Then, we need a method to obtain nominal design values from test data (block B). Ideally, this method should provide a consistent way of assigning design values at a specified level of confidence based on number of specimens and variability in test data.

![Figure 1. Overview of key elements needed for developing a joint design procedure](image-url)
The two testing procedures in blocks A1 and A2 (Fig. 1) are complementary. Test procedures for whole joint systems can be used to establish the design properties and/or evaluate the performance of a specific joint system under single or combined loads. These can also be used for individual fastener types (e.g. nails, bolts, screws, etc.) but they are most useful for jointing systems (i.e. complete joint system of similar or mixed materials) to:

- assess the performance and allocate design properties of a joint system under a specific loading configuration
- assess design theories for joint systems (e.g., block C)

In the following section, the review of different LSD design procedures for timber joints in selected countries/regions of the world will cover relevant information related to each block in Fig. 1. Primary supporting documents and standards are cited.
Review of Joint Design Standards

Introduction

Overview
The purpose of this section is to review the existing joint design standards in selected countries/regions of the world where timber engineering technology is well developed and where engineering practice is well recognised. Only a brief overview of the Australian code provisions is given, but extensive comments are made. The review of overseas codes is more detailed but focused heavily on dowel-type fasteners because these are the most common in timber construction. The presentation covers critical elements of the design method development process (Fig. 1), and includes specific comments that may be relevant to this project. Primary supporting documents and standards are included. The most important issues that need to be considered in the development of a new LSD method for timber joints are summarised.

Types of Joints
Joints will be categorised as those with (Madsen 1998):

- dowel type fasteners, and
- surface connections.

Dowel type fasteners include drift pins, bolts, coach screws (called lag screws in North America), wood screws, nails, spikes and timber rivets (also known as glulam rivets). Coach screws are a relatively large and special type of wood screw, and normally used in steel-to-wood connections. Timber rivets are a special type of nail for making single shear steel-to-wood splice joints. With the exception of the timber rivets, dowel type fasteners almost always have a nominally circular cross-section. Nails, spikes and timber rivets are typically installed without a lead-hole by hammering or using an air gun. Screws and coach screws are installed in lead holes. Drift pins (plain dowels) are installed by hammer into holes of the same nominal diameter. Bolts are installed in oversize holes. Typically the hole is made with a drill 1 to 2 mm greater than the bolt shank diameter. Bolts always have a head and nut, and washers beneath either the head or nut if they bear onto the face of a timber member. Literature is occasionally confusing because some authors from mainland Europe use the term bolt when they mean a plain dowel. All types of dowel fastener are permitted in applications where they resist loads by bending action (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 utilised 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’ and ‘split-ring’ connectors, which were developed in the USA in the mid-1930s. Shear-plates are used in timber-to-timber or steel-to-timber joints, while split-ring connectors are used in timber-to-timber joints. Both types of connector are used in conjunction with a bolt (or coach screw) which passes through the centre. For split-rings the function of the bolt is to prevent separation of the members so they do not ‘work themselves’ off the rings. A split-ring is embedded in both members and bridges the joint plane. It transfers all force across the joint. Shear-plates locally increase the fastener diameter, and therefore the capacity. One is placed either side of a joint plane in a wood-to-wood joint, but they do not bridge the joint plane. They transfer the force to the bolt passing through them, and it must be capable of resisting the full joint load in shear. Manufacture of joints with either type of connector is labour intensive. Both types are only suitable for use with large dimension timbers. In parts of Europe ‘toothed plate’ connectors are used. This is for all intents a low capacity type of shear-plate connector, but is much less labour intensive to install and can be used with small dimension timber (lumber). Punched metal plate connectors and various folded plate connectors manufactured from light gauge steel plate are used to make lumber-to-lumber joints. Punched metal plate connectors can have integral teeth or are fastened to the lumber with nails. Folded plate connectors are fastened to the lumber with nails. Punched metal and folded plate connectors are typically proprietary products, i.e., they are not manufactured to a standardised form.

Not all types of fasteners and connectors mentioned are covered by the structural design standards of various countries. Design capacities of proprietary products usually fall outside the scope of written codes. This does not mean that they cannot be used for building purposes. All developed countries have some sort of deemed to comply specifications for small to medium size timber buildings, and a process of certification of products. Certification often includes recommended design capacities that engineers can use.

Site-made glued joints are usually excluded from engineering design codes, because of concern about the ability of ‘workers’ to apply proper 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 written product standards or product certification, and not design codes.

**Australian Code**

*Overview of Design Method*

To design timber joints, an engineer typically needs only the Timber Structures Code AS 1720.1 (SAA 1997), which gives him/her the design equations, capacities and modification factors to use. Design provisions include the following:

- Nailed joints,
- Screwed joints,
- Bolted joints,
- Coach screwed joints, and
- Shear-plate fastener joints.

Applications of these provisions to design situations are demonstrated in the Timber Design Handbook (Boughton and Crews 1998).

The new LSD code for joint design has the following generic format:

\[(\bar{N}_j) \geq N^*\]

where

\[(\bar{N}_j) = \bar{N}_1 \ldots \bar{N}_j n Q_k\]

and

- \(N^*\) = factored design action effect
- \(\bar{N}\) = capacity factor
- \(k_i\) = modification factor
- \(n\) = number of fasteners
- \(Q_k\) = characteristic capacity of a fastener

Timber density groups classify joint capacities. As in most other codes elsewhere, the duration of load factor follows the ‘Madison curve’ (Wood 1951).

*Technical Basis*

Australian joint design procedures are, to a large degree, derived from early North American studies (e.g., Trayer 1932). This was supplemented by studies on Australian timbers,
particularly hardwoods, and local practice and conditions. Key contributors to this development include Langlands and Thomas (1939), the Department of Works and Housing (1946), Pearson (1965, Pearson et al. 1958), Mack (1966, 1978, 1979) and Lhuede (1986, 1988). The current design provisions are primarily due to the work of former CSIRO researchers, J.J. Mack and E.P. Lhuede. Leicester (1993) has provided a brief overview of timber joint code development in Australia.

The joint design provisions of the code are based solely on an empirical fit of test data. Since these provisions in the current LSD AS 1720.1 are essentially the same as in the previous version of the code, the commentary by Lhuede (1988) on the latter is still very relevant.

There has been some interest in Australia in the use of the European Yield Model (EYM) for dowel-type fasteners for some time. (This model is discussed in later sections of the report; Appendix A has a set of equations that describe the model). Mack (1960, 1962) investigated this first, followed by Lhuede (1988, 1990) and then by Stringer (1991, 1993). Stringer’s work and results are discussed under Comments and in Appendix B.

Supporting Standards
The following standards provide support and information needed to use the design procedures. In Fig. 2, they are placed in the context of the diagram given in Fig. 1.

Comments

Because current design procedures are based on empirical rules, their applicability to new products/materials in the marketplace may be questionable. For example in the last several decades, we have seen:

- a shift in the house framing market from the use of unseasoned hardwoods to seasoned softwoods,
- the introduction of different variety of fasteners, and
- the introduction of new sheathing panels and other composite timber products.

For these reasons, Stringer (1991) examined the applicability of the mechanics-based EYM to nailed timber joints using four Australian timber species, plywood from slash pine veneers and Radiata pine laminated veneer lumber (LVL). He concluded that EYM has the potential to be used in Australia, and he presented a draft design method for nailed joints based on EYM. He also listed a number of recommended research work in the future. In Appendix B, we have provided some comments regarding his work and a re-analysis of some of his data using the ‘simplified EYM’ presented by Blass et al. (1999), which gives a better prediction than the original EYM that he used.

We should note that seemingly small procedural differences in test methods and data interpretation techniques are important in evaluating the applicability of EYM. Furthermore, tests on single nail joints are not very realistic in performance and with respect to variability in joint strength. Thus, while we intend to use all of Stringer’s data and results as much as possible, (as we have done in Appendix B), in planning our experimental program, we may...
also re-work some of Stringer’s tests, as we consider other factors that may affect the use of EYM in Australia.

Based on Fig. 2, it would seem that current support for the design of joints in AS 1720.1 through existing and draft standards is adequate. But before AS/NZS BBBB was drafted, the need for better and more realistic joint testing and evaluation procedures was highlighted (Leicester 1993). There were three primary problems with the previous operational system (Foliente and Leicester 1996):

- First, the AS 1649 test provides data on only one type of failure mode whereas at least ten types of failure modes have been observed in laboratory tests. Some examples include:
  - failure of metal (tension, bending, shear, buckling),
  - failure of wood parallel to the grain (tension, compression, shear), and
  - failure of wood perpendicular to the grain (tension, compression).
  An example of major concern is the failure of bolted joints due to splitting; AS 1649 does not address the problem of splitting.
- The second flaw is that the load factors currently used in AS 1649 are questionable. They were derived on the basis of fitting existing practice. It is not known whether the controlling criteria, then in use, were related to strength or serviceability (deformation); also the load factors vary considerably from one connector type to another for no obvious reasons.
- The third flaw in the current procedure arises from the fact that there are very few theories to predict the strength of joint systems on the basis of the characterisation of single connectors. Thus, there is obviously a need for a standard not only to describe the testing of new fasteners, but also for whole joint systems.

The draft standard AS/NZS BBBB is 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 standard is analogous to AS 4063 Timber—Stress-graded—In-grade Strength and Stiffness Evaluation (SAA 1992), i.e., 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. Additional information about this set of standards can be found in Foliente and Leicester (1996), and Foliente et al. (1998).

With regards to the method of establishing characteristic values and nominal design values (block B in Fig. 2), AS BBBB uses the proposal by Leicester (1986). Bryant and Hunt (1999) conducted recently a numerical study using Monte Carlo simulation to evaluate different methods of computing characteristic values of joints. Since the ‘Leicester method’ provided good predictions for reasonable sample sizes, they recommended its use, especially where the coefficient of variation is calculated from the lower tail of experimental results. It should be noted that the Leicester method was derived with the latter in mind; it is conservative when the coefficient of variation is calculated from all the test data.

US Code

Overview of Design Methods
A vast majority of American engineers design timber structures based on the Working Stress Design (WSD) approach. The factor of safety is 1.3 for members loaded in bending, tension and compression (Madsen 1992). For timber joints the factor of safety is inconsistent across various types of joints, and cannot actually be defined. The approach for splice joints with dowel type fasteners is to take the allowable load as a fraction of the load at the ‘proportional limit’ load or an approximation to it (USDA 1987). For joints made with split-ring or shear-plate connectors the allowable load is based on consideration of both the peak load and proportional limit load. The proportional limit load, as observed in a short-term monotonic loading test, has traditionally been believed to be the point at which irreversible damage processes begin. If observations of joint slip (relative displacement of adjacent members at the joint location) are made using modern high resolution Linear Variable Displacement Transformers (LVDT’s), and these are placed in such a way that all member deformation is excluded, load versus slip relationships are much more curvaceous than was thought. Existence of a proportional limit is largely an illusion and reflects use of low precision methods such as manually read dial gauges (Smith 1998). Withdrawal capacities of nails, wood screws and coach screws are based on peak load. Peak loads are multiplied by fractions to account for safety, variability and moisture effects (USDA 1987). The fraction of peak load used varies between types of fasteners.
Unadjusted allowable loads apply to a single fastener/connector subjected to normal duration (10 year) loading, and a dry service condition. Deviations from reference conditions are accounted for by a sequence of modification factors. Concepts and much of the data that underpin working stress design of joints are the result of research at the US Forest Products Laboratory in the 1930s and 1940s (e.g., Trayer 1932). It is generally accepted that the working stress method embodies inconsistencies in capacities for different types of fasteners (McLain 1993). For most types of mechanical joint the ultimate (peak) load lies well above the estimated proportional limit load. The ratio of ultimate to proportional limit load can approach 3.5 for nailed softwood joints and 7.0 for hardwood joints (USDA 1987). Thus, the allowable load for a joint is often only a small fraction of its ultimate strength. Joints tend to be strong elements within structural systems. Overloading will therefore tend to cause failures in members rather than joints.

There have been some relatively recent changes in the basis of US design provisions for joints, especially with regard to dowel type fasteners (AFPA 1991). One important change is that the lateral load capacities of dowel type fastener joints are now based on Johansen’s yield model (Johansen 1949, Larsen 1979). The model presumes that both the fastener and the wood foundation upon which it bears behave as ideal rigid plastic materials, and therefore that the yield and ultimate capacities coincide. The basis of the theory is exactly the same as for the well-known limit analysis (plastic design) theory applied to steel frames. The Johansen theory is often referred to as the European Yield Model (EYM) in the USA. The most recent working stress design code National Design Specification for Wood Construction was published in 1997 (AFPA 1997). Specific provisions and their basis are discussed in the next section of this report.

Another important change is that, for load combinations including short-duration loads such as wind or earthquake, NDS now prescribes a load duration factor of 1.6 for all connections. However, the Uniform Building Code (UBC) (ICBO 1997) prescribes different load duration factors for joints with dowel-type fasteners under these loading conditions. The UBC permits a load duration factor of 1.6 for joints exhibiting failure modes III or IV, but a load duration factor of 1.33 must be used for joints that exhibit yield modes I or II. The reason for this requirement is that there are available data that demonstrate that modes III or IV joints have sufficient ductility under cyclic loading (Dolan et al. 1995, 1996) but there is a lack of data for modes I or II joints. The NDS provides the designer with multiple tables containing capacities for common connections but yield modes are not identified. Thus, the designer must carry out potentially cumbersome calculations first simply to determine the governing yield mode.
before he/she can apply the appropriate UBC load duration factor. To address this issue, Tucker et al. (2000) have produced tables that extend NDS tables to contain connection capacities and yield modes to accelerate the design process.

The American Society of Civil Engineers (ASCE) recently published a code for limit states design of timber construction (ASCE 1996a). This is based on a partial-coefficient approach that in the USA is known as Load and Resistance Factor Design (LRFD). Technically this is equivalent to many other, and somewhat older, limit states design partial-coefficient design codes. As in other countries, partial factors are assigned based on committee judgement aided by structural reliability analysis. It is not clear to what extent the ASCE code is actually used by design engineers. 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 WSD and LRFD methods (or just WSD 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 in California and some other states. Eventually the WSD should cease to be referenced and therefore disappear from use, but informal comments from colleagues in the USA imply that at present very limited use is made of LRFD.

Common connections are those made using nails, bolts, screws, punched metal plate connectors, folded plate connectors, and hangers. Of these types, only capacities of those made with generic fasteners (nails, bolts and screws) are dealt with by design codes. Thus, only use of generic dowel type fasteners is discussed further.

**Technical Basis**

The research background and rationale underpinning much of the US code provisions have been discussed by McLain (1984) and in a recent ASCE publication entitled *Mechanical Connections in Wood Structures* (ASCE Manuals and Reports on Engineering Practice No. 84) (ASCE 1996b). The latter covers the following:

- Nails, spikes and staples,
- Lag (or coach) screws and wood screws,
- Bolts, drift bolts and pins,
- Metal connector plates, and
- Timber connectors (e.g., shear-plate and split ring connectors).
For each fastener type, the ASCE manual also contains sections on details, fabrication and installation practices, and a brief review of ‘foreign’ design practices or specifications. We recommend those interested in background technical details to consult the ASCE manual. In the rest of this section, we focus the discussion on laterally loaded fasteners.

Yield limit load (or EYM) equations for bolt, coach screw, wood screw, nail, spike and drift-pin joints are given in the *National Design Specification for Wood Construction* (AFPA 1997). The approach was first adopted in the 1991 edition of that Specification (AFPA 1991), at which time it had been decided that mechanics-based formulae are the most rational means of allowing designers to account for the many arrangement specific parameters that affect the capacity of a joint (McLain 1993).

The yield limit load equations are a generalised form of the original EYM by Johansen (1949). Figure 3 shows the failure modes that are possible for a single fastener joint loaded in single shear (two-member arrangement). For the purposes of this report, the left member is the head-side member and the right member the point-side member for joints with nails, spikes, wood screws or coach screws. Comparable modes are applicable to multiple shear arrangements. Figure 4 shows the failure modes that are possible for a symmetric double shear (three-member) arrangement. Geometric variables are fastener diameter, $d$, and length of penetration in each member ($l$, $\alpha l$) [where $l$ is thickness of side/head-side member in a single-shear joint, and $\alpha$ is ratio of the thickness of main member to thickness of side member(s)]. Joint capacity increases as thickness of the members is increased up to the point where a mode IV failure occurs. It is assumed that:

- the members are rectangular in cross-section,
- the fastener is inserted normal to the faces of the members, and
- members are loaded concentrically.

Neglecting force other than that normal to the axis of the nail, the yield load (per shear plane) for a joint, $P_Y$, is 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 made of rigid-plastic material, it can only translate and/or rotate as a rigid body (or segments of it behave as rigid bodies for modes where one or more hinges form in the fastener). Failure can involve bearing failure in just one member or formation of a plastic hinge in just one member. Consequently, there are a total of six potential failure equations to consider for two-member joints, and four equations in the case
of three-member joints. The NDS equations (AFPA 1997) are based on the 'Original EYM' (Appendix A), although the notation differs from that used here.

Figure 3. Yield modes for two-member joints

Figure 4. Yield modes for three-member joints

Although the NDS equations follow the Original EYM, there is a very specific meaning attached to the yield limit load and the material properties that enter the equations for the various failure modes. The NDS yield limit load relates to the 5% offset load (Fig. 5). This is a value intermediate between the proportional limit and ultimate loads. In an analogous
manner, the bearing embedment (bearing) strength of the wood beneath a fastener and the yield moment for the fastener foundation are 5% offset values. The product of the calculations yields the nominal capacity for a joint with one fastener, i.e., they are values that have not been modified for design purposes.

![Figure 5. Five percent offset method for defining joint yield load (based on McLain 1993)](image)

The nominal design value for a given combination of fastener and member parameters is divided by applicable reduction terms. Reduction terms differ depending upon the type of fastener. For bolts, drift pins and coach screws they are a function of the EYM mode that governs (which reflects the slenderness of the fastener), and the maximum angle to the grain at which the fastener loads any wooden member. For nails, spikes and wood screws the reduction factor accounts only for the duration of the loading and depends upon the fastener shank diameter. In the NDS, the reduction terms are incorporated directly into the denominator of the ‘yield limit’ equations. The purpose of these adjustments is to give capacities that are representative of nominal proportional limit based design capacities in earlier editions of the specification (soft calibration).

Material properties which are input to the yield limit equations reflect the type of fastener, its size, the materials from which members are made and the direction to the grain at which wood members are loaded. Embedment strength was determined according to the procedures that have since been formalised in ASTM D 5764 (ASTM 1997a). Embedment strength is taken as a function of fastener type, wood density, and direction of loading relative to the grain in the case of larger diameter fasteners. Embedment properties are available for steel and concrete as well as wood. Fastener bending strengths were estimated based on ASTM F 1575 (ASTM 1995b), the tensile strength according to ASTM
F606 (ASTM 1995a) or from tabulated tensile values. Methods of conversion have been established for calculating the 5% offset bending strength from more readily available tensile strength information (AFPA 1999).

In deriving NDS values no allowance was made for the possibility that presence of, for example, a nail head or head of a bolt and the washer underneath it that might restrain the end of and a fastener. Any restraint to the end(s) of a fastener will increase its capacity unless members are thick enough to develop a mode IV failure. Similarly, no allowance was made for friction between members which also enhances the capacity of a joint. Influences such as those mentioned are ignored because they can not be reliably obtained in practice.

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. This is generally regarded as a reasonable assumption at loads approaching the ultimate capacity (Smith 1998). However for joints having a number of bolts, drift pins or coach 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 a simplified analogue where an assemblage of linear elastic springs represents the fasteners and member segments between them (Lantos 1967, Cramer 1968). For design purposes the number of fasteners adjustment factor depends upon the type of fastener, their spacing and cross-section sizes of the joined members. The approach can be manipulated to yield an explicit prediction of the effective loss in capacity, per fastener, as a function of the number that are placed in a row (Zahn 1991).

The NDS requires that certain minimum requirements 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 wood members. Adjustments must be made to ‘basic’ NDS capacities accounting for the effects of non-reference moisture and duration of loading conditions.

Supporting Standards
The following standards provide support and information needed to use the design procedures. In Fig. 6, they are placed in the context of the diagram given in Fig. 1.

- ASTM F606-95b ‘Determining the mechanical properties of externally threaded fasteners, washers, and rivets’, Volume 15.08 Fasteners, ASTM, West Conshohocken, PA.

![Diagram](image)

**Figure 6.** Key elements of the US joint design practice

**Comments**

As far as we know, there is no explicit method in the US of evaluating characteristic values and nominal design values from experiments of timber joints (block B in Fig. 6). In principle, the method used in evaluating the characteristic values of lumber properties can be used for joints. Since there are typically much less samples in joint tests than in lumber tests, the applicability of the non-parametric method of evaluating characteristic values of lumber in ASTM D1990-91 (ASTM 1996) for use in connections needs to be confirmed.

Although the USA has followed other countries in adopting the EYM as an approach for calculating single fastener (reference) joint capacities, the NDS approach is unique in respect to: (1) the use of 5% offset yield values for joint load, (2) the properties entering the ‘model’, and (3) the use of different load duration factors for different failure modes and for load combinations that include wind or earthquake loads. Also it is unique in that it adopts
fastener slenderness and duration of loading related adjustments to nominal proportional limit values to arrive at reference strengths in the NDS. Compared to EYM design capacities employed elsewhere, the NDS values tend to be low. There is fairly extensive recent research evidence that the NDS method for accounting for the effect of the number of fasteners is non-conservative, certainly in the case of bolted joints (Mohammad et al. 1997). It should be remembered however that the USA practice is actually calibrated to give, in a global sense, traditionally accepted solutions. It can be argued, therefore, that it is inappropriate to dissect the calculation method and then consider its parts in isolation.

Although the specifics of the LRFD and WSD methods differ, the technical basis of the two codes is rather similar with respect to joints. Thus, the specifics of the LRFD code will not be discussed here. The same should not be inferred for member design.

**Canadian Code**

*Overview of Design Methods*

Like the USA, Canada permits engineers to design to either WSD provisions (CSA 1984) or partial-coefficients limit states provisions (LSD/LRFD) (CSA 1994). It has, however, been accepted for some time that national, provincial and municipal building codes will cease to reference the working stress version of the timber design code. This is expected to become established fact within the next three to five years. The WSD provisions were very similar to those used in the USA (pre-1991). Because WSD is essentially obsolete in Canada, it is not discussed further in this document.

Canada introduced its first LSD code for timber structures in 1984 (CSA 1984). For timber members this was a soft conversion from the working 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 re-analysed so that joint capacities would reflect the ultimate (strength) limit state and serviceability limit state considerations. Strength and serviceability limits were separately identified for joints made with truss plates, nails and coach screws. Only the ultimate limit state was considered in other cases, e.g., laterally loaded bolts, nails loaded in withdrawal. Ultimate capacities of laterally loaded bolts and nails were calculated based on the EYM (Johansen 1949, Larsen 1979). This use was not transparent in 1984, as capacities were given in the traditional tabular form for different combinations of fastener diameter and commercial species group, plus member thickness and direction of loading in the case of bolts. Use of a Johansen type
approach became transparent for bolted joints in the 1989 edition of the Canadian timber code. Resistance factors, $\phi$ values, were assigned based on engineering judgement. Conceptually an ultimate limit state design equation takes the form: $\phi R \geq \sum \alpha F$ where $\alpha$ values are the load factors and $F$ values the non-factored load effects. Despite selective refinements, code provisions for joints remain essentially unaltered since 1984 (Lepper and Smith 1995, CSA 1994).

Common connections are those made using nails, bolts, drift pins, punched metal plate connectors, folded plate connectors, and hangers. Of these types, only capacities of those made with generic fasteners (nails, bolts and drift pins) are dealt with by design codes. Thus, only use of generic dowel type fasteners is discussed further. Note that no capacities are currently given for wood screws in the Canadian code. This omission is expected to be rectified in the next edition.

Technical Basis

(Note: Since many of the technical details parallel those discussed in connection with the US Code, emphasis is placed here on aspects specific to Canadian practice.)

Yield load (EYM) equations for bolt, drift pin and coach screw joints are given in the CSA Standard 086.1 Engineering design in wood (limit states design) (CSA 1994). The designer does not use EYM equations directly in the case of joints made with nails or spikes. For bolt, coach screw and drift pin joints the failure modes considered are those shown in Figures 3 and 4. The equations used are an empirical approximation for some modes, and not the original EYM equations. The approximations used are due to Whale et al. (1987) and given in Appendix A. As for the Original EYM, geometric variables are fastener diameter, $d$, and length of penetration in each member ($l, \alpha l$). 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 1989). The values represent the ultimate (peak) bearing capacity (Fig. 7). Thus, the embedment values adopted in Canada do not have the same basis as those in the USA. Embedment properties for wood members are related to the short-term ultimate strength of the material. Empirical rules apply the yield load equations to timber-to-concrete connections. 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. It is assumed $M_y = \text{yield stress} \times \text{plastic section modulus} \left(\frac{d^3}{6}\right)$. The output from the EYM
calculation is an estimated 5\textsuperscript{th} percentile short-term ultimate strength. A factor is embedded within the CSA equations. This converts from short-term test to Standard Term loading, and has the value of 0.8 irrespective of which EYM mode governs. Standard Term does not have an associated duration of the loading. The 0.8 value was derived from structural reliability analysis on lumber members, this being the only product for which there was a creep-rupture model available (Foschi et al. 1989). It was a committee decision to apply the same factor to all products and connections, in the absence of other information. Standard term loading corresponds to the snow plus dead load combination for roofs and the live plus dead load combination for floors. The way the 0.8 factor is applied is hard to justify for joints where failure involves failure in both the fastener and the wood, e.g., slender bolts.

![Diagram](image)

**Figure 7.** Canadian method for defining embedment strength

Tabulated specified strengths for nail and spike joints are based on the Original EYM (Appendix A), 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). The embedment strength is taken to be 1.1 x ‘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 and inter-member friction (Keenan et al. 1982). Allowance for these effects inflates the specified strength values by up to about 30 percent. Recent, as yet unpublished, work at the University of New Brunswick, Canada indicates that neither the string effect or inter-member friction should be employed. Capacities of nail and spike joints are independent of the angle to the grain at which the fastener(s) loads the timber member(s). Various multiplicative modification factors are applied to account for variations in the arrangement from the reference geometry. These
account for factors such as the number of joint planes, 'substandard' member thickness, nail 'clinching' and toe nailing. Without exception, these modification factors are based on empirical evidence.

CSA capacities for dowel type fasteners make no allowance for the possibility that restraint at the head of a nail, spike or bolt will enhance the capacity of a joint. 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 coach screws capacities are adjusted to account for uneven force distribution. Prior to the 1989 edition of CSA Standard 086.1 (CSA 1989), the approach for accounting for this was the same as in the NDS (AFPA 1997). Since then, 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 CSA 'group effect' adjustments are rather constricting and preclude use of connections with a large number of bolts (very little increase in capacity is available for more than about 20 bolts).

CSA Standard 086.1 requires that certain minimum requirements be met with regard to the arrangement of fasteners in a joint, so as to reduce the possibility of premature brittle failure. Adjustments must be made to specified strengths to account for the effects of non-reference moisture and duration of loading conditions.

Supporting Standards and Documents
The following standards provide support and information needed to use the design procedures. In Fig. 8, they are placed in the context of the diagram given in Fig. 1.

- CSA Standard 086.1-94. *Engineering design in wood (limit states design)*. Canadian Standards Association, Toronto, ON.
Comments
As seen in Fig. 8, there are currently no standard test method for whole joint systems (block A2) and no agreed method to evaluate characteristic values and nominal values for design of joints (block B). The latter is typically done on an ad-hoc basis by either individual researchers or the standards committee. Thus, the method of establishing design values is neither consistent nor transparent.

The CSA 086 Technical Committee that is responsible for the Canadian timber design code is in the process of a major overhaul of provisions pertaining to joints. Their objectives include:

- the 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 formula based methods whenever feasible.

By the time of the next edition, there should be almost totally new provisions in place for nail, spike, bolt, drift pin and coach screw joints. Wood screw joints should be introduced for the first time. The most likely publication date is 2003.

**Figure 8.** Key elements of the Canadian joint design practice
European Code

Overview of Design Methods
The situation in Europe is somewhat complicated owing to the many national and multinational technical/code writing bodies that are involved. The European Committee for Standardisation (CEN) has published a model design code *Structural Timber Design Code* with the designation Eurocode 5 (CEN 1995). This is a partial coefficient based LSD format. There is a raft of supporting European test standards detailing how joints and joint components are to be tested. Many of them are being ‘fast-tracked’ to become ISO standards. Various countries within Europe have developed/are developing National Application Documents that meld Eurocode 5 (EC5) with local needs, tradition and design practices. Although the documents are available it does not mean that they have yet superseded prior codes. As discussed in respect of the US code, what is permitted within political jurisdictions is controlled from within. It is understood that both WSD and LSD methods are used over wide areas of Europe. Scandinavia was the first region to fully embrace LSD. Because of the difficulty in saying what exactly is the situation, this document takes a liberal approach in describing the state of the ‘design art’.

Technical Basis
The EC5 is written to a partial coefficient LSD format, and is intended to be consistent with partial coefficient loading codes. Conceptually the form of the design equation for a member or joint is:

\[ R_d \geq \sum \alpha_i F_i \]

where

\[ R_d = R_k k_{mod} / \alpha_m \]

and

- \( R_d \) = design resistance
- \( R_k \) = characteristic resistance
- \( k_{mod} \) = composite modification factor
- \( \alpha_m \) = material resistance partial coefficient = \( \phi^{-1} \)
  (inverse of the resistance factor \( \phi \) that is used in non-European countries)
- \( \sum \alpha_i F_i \) = summation of factored load effects, *schematic*
  (\( \alpha_i \) = partial load coefficient, \( F_i \) = load effect from wind, snow, etc)
The characteristic resistances \( R_k \) is a 5th percentile value determined from laboratory tests of about 5 minute duration to failure, using wood materials conditioned at 20°C and 65% relative humidity (about 12% moisture content for solid wood). Partial coefficients are assigned values by the appropriate body within any country via the National Application Document. Values are suggested in EC5 as a guide to national bodies. The composite modification factor adjusts from reference to design conditions, e.g., to account for the service environment and duration of the loading.

Rules are given covering the design of connections made with dowel type fasteners (nails, bolts, screws, etc), and surface connections made with shear-plates, split-rings and toothed plate connectors. As in other sections of this review, discussion is limited to design of dowel type fastener joints.

Characteristic resistances for dowel type fastener joints are derived using the Original EYM equations (Johansen 1949), Appendix A. The effects of tension in the dowel, inter-member friction, and restraint at the ends (e.g., due to nail or bolt heads) are ignored. In these respects, the approach is equivalent to assumptions accepted in the USA (AFPA 1997). The embedment strengths of members, \( s_H \) values, are based on the maximum bearing resistance (Fig. 7). Embedment properties are a function of fastener diameter, wood density and angle of loading relative to the grain (except for nails). Fastener yield moment is determined by testing, or by calculation assuming \( M_Y = (\sigma_U + \sigma_Y)d^3/12 \) where \( \sigma_U \) and \( \sigma_Y \) are the ultimate and yield stresses in tension. (Note: the notation has been changed from the original so as to be consistent with the rest of this document.) Values of \( s_H \) and \( M_Y \) entering the EYM equations are first adjusted to account for changes in properties from reference to design conditions, and to include partial safety factors. Assuming use of timber for all members and a steel dowel fastener, the approach is as follows:

\[
\begin{align*}
    s_{H,d} & = k_{mod} s_{H,k} / \alpha_{m,wood} \\
    M_Y & = M_{Y,k} / \alpha_{m,steel}
\end{align*}
\]

where

- \( s_{H,d} \) = design embedment strength
- \( s_{H,k} \) = characteristic embedment strength
- \( \alpha_{m,wood} \) = material partial coefficient for wood (> 1.0, typical value 1.3)
- \( M_Y \) = design moment resistance
- \( M_{Y,k} \) = characteristic moment resistance
- \( \alpha_{m,steel} \) = material partial coefficient for steel (> 1.0, typical value 1.1)
Thus, for mode I and II failures $R_d / R_k = k_{mod} / \alpha_{m,\text{wood}}$, while for mode IV failures $R_d / R_k = [k_{mod} / (\alpha_{m,\text{wood}} \alpha_{m,\text{steel}})]^{1/2}$. The ratio $R_d / R_k$ is intermediate for mode III failures. The details of the approach are not the same as in non-European countries. However, there are similarities to the approach taken in the USA, in that adjustments for non-reference conditions depend upon the slenderness 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:

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

where $n$ is the number of fasteners aligned in a row. This is a very liberal reduction rule compared with those used elsewhere (USA, Canada). It is understood that the EC5 rule is based on judgement rather than data. More sophisticated approaches to the group of fasteners effect on the strength of a connection are permitted, provided that they are at least as stringent. For nailed joints, $n_{\text{ef}} = n$.

In the case of joints with steel plate members (not thin sheet steel) the EYM equations are simplified by setting the embedment strength for such a member(s) equal to infinity. This results in mode I embedment failure in the wood member(s), or a mode III or IV failure (with a hinge located on the joint plane(s)) governing. This is a non-conservative approach.

Composite modification factor $k_{mod}$ is arrived at in a manner unique to EC5. The factor accounts simultaneously for duration of the loading and the moisture conditions in service. A curtailed set of $k_{mod}$ values is shown in Table 1. It is understood that the values are based on ad-hoc test data, experience and committee judgement.
Table 1. Values of composite modification factor $k_{\text{mod}}$ for solid and glued-laminated timber and plywood in Eurocode 5

<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 unlimited)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent (&gt;10 yrs.)</td>
<td>0.60</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>Long-term (6 mo. to 10 yrs.)</td>
<td>0.70</td>
<td>0.70</td>
<td>0.55</td>
</tr>
<tr>
<td>Medium-term (1 wk. to 6 mo.)</td>
<td>0.80</td>
<td>0.80</td>
<td>0.65</td>
</tr>
<tr>
<td>Short-term (&lt;1 wk.)</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 values refer to those moisture contents typical of solid wood in the service class.

Fairly extensive guidance is given on calculation of slip in joints, and as for strength, is a function of both the service class and the duration of the loading. Rules are given regarding aspects of connection design such as placement of fasteners to avoid premature brittle failure. Nail spacing provisions are a function of the wood density, and whether or not there is a predrilled lead hole.


Supporting Standards

The following standards provide support and information needed to use the design procedures. In Fig. 9, they are placed in the context of the diagram given in Fig. 1.

• EN 26891. ‘Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics’, European Committee for Standardisation (CEN), Brussels, Belgium.

![Diagram](image.png)

**Figure 9.** Key elements of the Eurocode 5 joint design provisions

**Comments**

In many respects, the EC5 approach to the design of dowel type fastener joints has much to commend it relative to other codes (e.g., NDS, CSA Standard 086.1). It is certainly logical to incorporate modification for the effects of moisture content and duration of loading within the EYM calculations. It is not easy however to argue the merits of incorporating partial safety coefficients within the EYM calculations, without specific evidence that variability in resistances for joints is proportional to the relationships between joint yield load, \( P_Y \), and component strength properties (\( s_H \) and \( M_Y \)) implied by the EYM. This could be verified based on test data in the literature. The coupling between the effects of service class (moisture content) and the duration of loading class is philosophically correct, and is perhaps the most innovative aspect of the EC5 method. The use of five duration of loading classes upholds long-term practices, but seems in conflict with results from application of creep-rupture models within structural reliability calculations (Foschi et al. 1989). This, and the rather weak treatment of how having a group of fasteners affects the strength of a connection are the major weaknesses of the EC5 provisions.
Summary of Issues

Based on the review of design practices and standards in Australia, USA, Canada and Europe (based on Eurocode 5), we offer the following observations:

- Mechanics-based formulae allow designers to account for the many arrangement specific parameters that affect the capacity and failure mode of a joint. They are preferred over design equations that are based on empirical relationships alone because of the continuing introduction and/or evolution of products, and the changes that occur in the timber construction market.

- Methods for testing of whole joint systems that specify actual in-service joint configuration and realistic in-service loads can be used to establish joint design properties and/or evaluate the performance of a specific joint system. They are also useful in evaluating design theories for joint systems (e.g., assessing EYM). The current set of draft standards AS BBBB in Australia and New Zealand is one such method.

- There is a need for a rational and consistent method to establish characteristic values and nominal values for design of joints. This is typically done, in most places, in a way that is neither consistent nor transparent. Applicability of methods used in establishing design values for timber/lumber should be investigated. The current Australian approach, called the ‘Leicester method’, provides a simple way that seems to work well.

The European Yield Model (or its adaptations) provides a set of mechanics-based equations for the design of joints with dowel-type fasteners that allows consideration of failure modes in design and the flexibility of being applicable for joints with non-traditional members (e.g., new species or new composite). Having said this, we note that the use of EYM in design codes and standards is not universally harmonious as generally believed. Many issues still need careful consideration. These include the following:

- EYM equations still need empirical adjustments and simplifications that best suit local conditions and/or practice;

- EYM predictions depend on the type of test used in deriving input properties; seemingly small procedural differences in test methods and data interpretation techniques are important;

- Tests on single nail joints are not very realistic in performance and with respect to variability in joint strength;

- Applicability to large diameter bolts and other types of dowel-type fasteners (other than nails and slender bolts) require further empirical adjustments;
• Effects of modification factors (e.g., moisture, duration of load, etc.) are not well established; traditional modification factors specified in existing codes are used by default;
• A species (or possibly density) related rules for minimum member thickness to avoid premature splitting failures is necessary;
• A method to calculate variance in joint strength from variance in nail and wood properties is needed; and
• Reliability-based calibration of resistance factors is needed.
Recommendations

General

Our primary recommendation is to fully develop the use of the European Yield Model (EYM) for the design of dowel-type fasteners in Australia. As a first step, we should consider nails, and, possibly, slender bolts. This process will involve developing:

1. an experimental program to establish properties and try the EYM concept to a range of variables common in Australian condition; and
2. the infrastructure required to support the use of this design procedure.

The latter includes guidelines and/or standards to determine the input properties needed in EYM-based design. Despite the amount of development and data available from other countries, a fair amount of work still needs to be done to evaluate its applicability to a wide range of Australian joints with dowel-type fasteners, and to introduce simplifications to make the proposed design procedures easier to use.

Figure 10 shows the overall logic of joint design development. The experimental program involves evaluation of material properties to determine input values for the EYM equations and of complete joint systems to validate the use of EYM theory and to provide data for simplification of the equations. With acceptable methods of deriving characteristic values, then new code rules can be developed and put into appropriate language.

In developing LSD procedures for timber joints, it is ideal to determine the appropriate resistance factors, develop a method to estimate joint variability based only on data from fastener and timber components of the joint, and conduct reliability analysis to calibrate the method to accepted levels of performance.
Finally, we also recommend further development of the standard for testing whole joint systems under realistic loading conditions; this is currently in draft form as AS/NZS BBBB. The concept is state-of-the-art, and many aspects of this suite of standards provide typically missing elements (evident in the review of overseas codes) that are needed to support design procedures for timber joints.

Factors and Parameters to Consider

To determine the practical scope of the experimental program in Phase 2 of this project, we need to consider first the parameters and factors that affect the development of EYM-based design equations. Suggested priority rating is given as follows:

- ♠ low priority
- ♠♠ medium priority
- ♠♠♠ high priority
- ♠ fair amount of overseas data/information that can be used

The final rating will be determined after consultation with representatives of the timber industry and timber engineering design profession.
• **Characteristic embedment values.** Variables:
  - Type of fastener (nail, bolt, screw) ♠♠♠♠♠
  - Size of fastener ♠♠♠
  - Surface: (plain, profiled, coated) ♠♠
  - Type of member
    - Lumber ♠♠♠♠♠
    - LVL ♠♠
    - Plywood (thickness should only be a weak influence, but needs to be checked) ♠♠♠♠♠
  - Influence of moisture
    - Conditioning (delay between fabrication and testing) ♠♠♠
    - Cycling ♠♠♠
    - Minimum member thickness to avoid premature splitting failures ♠♠♠♠♠
    - Influence of load cycling, duration of load effects ♠♠(nails), ♠♠♠(bolts)

• **Characteristic yield stress/moment.** Variables:
  - Type of fastener (nail, bolt, screw) ♠♠♠♠♠
  - Type of metal ♠♠♠
  - Size of fastener ♠♠♠

• **Joint validation tests** (to check whether EYM works)
  - Number of fasteners ♠♠♠♠
  - Effects of fastener spacing, end and edge distances ♠♠♠♠
  - Comparison with findings from overseas studies

• **Reliability studies**
  - Characterisation of variability for various types of connection ♠♠♠♠
  - Calibration of resistance factors ♠♠♠
  - Calibration of modification factors ♠♠♠

• **Putting rules into code language** ♠

Short-term embedment strength and nail (not bolt or screw) yield tests are quick and fairly cheap to do, so there is no real point in economising on them. Other tests need to be thought about rather more, as the cost implications can be significant.
References


5. ASCE 1996b, Mechanical Connections in Wood Structures, ASCE Manuals and Reports on Engineering Practice No. 84, American Society of Civil Engineers, New York, NY.


18. CEN 1995d, EN 26891. ‘Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics’, European Committee for Standardisation (CEN), Brussels, Belgium.


24. Department of Works and Housing 1946, “A report on the structural soundness of unseasoned timbers used in structures erected for war purposes”, Department of Works and Housing, Melbourne, Australia.


41. Leicester, R.H. 1986, “Confidence in estimates of characteristic values”, Proceedings of 19th Meeting of CIB W18, Firenze, Italy.


Appendix A

The European Yield Model

Original Model

The following equations describe the original model proposed by Johansen (1949) and Larsen (1979). Notations are defined in Appendix C.

- 2-Member joints

\[
P_y = s_H(dl)\begin{cases}
1.0 + \frac{\alpha}{\alpha_1} & I_s \\
\sqrt{\beta + 2\beta^2(1 + \alpha + \alpha^2) + \beta^2 + \beta^2 - \beta(1 + \alpha)} & II_s \\
\frac{2\beta(1 + \beta) + 4\beta(2 + \beta)M_y}{s_H dl^2} - \beta & III_s \\
\alpha \left( \sqrt{2\beta^2(1 + \beta) + 4\beta(2 + \beta)M_y} - \frac{M_y}{s_H d\alpha^2 l^2} \right) & III_m \\
\min \left\{ \sqrt{4\beta M_y} \right\} & IV
\end{cases}
\]
• 3-Member joints

\[
P_y = s_H dl \begin{align*}
\left( \begin{array}{c}
1.0 \\
\frac{\alpha \beta}{2} \\
(1 + \alpha \beta) / 5 \\
\end{array} \right)
\end{align*}
\begin{align*}
I_s & \text{II}_m \\
I_m & \text{II} \\
& \text{III}_s \\
& \text{III}_m \\
& \text{IV} \\
\end{align*}

\[
\sqrt{2\beta(1 + \beta) + 4\beta(2 + \beta)} \frac{M_y}{s_H dl^2} - \beta \\
\frac{\beta M_y}{(1 + \beta)s_H dl^2 + \frac{1}{5}} \\
\frac{\beta M_y}{(1 + \beta)s_H dl^2 + \frac{\alpha}{5}} \\
\min \left\{ \sqrt{s_H dl^2 (1 + \beta)} \right\}
\]

Empirical Approximation

The following equations describe the simplification proposed by Whale et al. (1987).

• 2-Member joints

\[
P_y = s_H dl \begin{align*}
\left( \begin{array}{c}
1.0 \\
\alpha \beta \\
(1 + \alpha \beta) / 5 \\
\end{array} \right)
\end{align*}
\begin{align*}
I_s & \text{II}_m \\
& \text{II} \\
& \text{III}_s \\
& \text{III}_m \\
& \text{IV} \\
\end{align*}

\[
\sqrt{\beta M_y}{(1 + \beta)s_H dl^2 + \frac{1}{5}} \\
\frac{\beta M_y}{(1 + \beta)s_H dl^2 + \frac{\alpha}{5}} \\
\min \left\{ \sqrt{s_H dl^2 (1 + \beta)} \right\}
\]
• 3-Member joints

\[ P_y = s_H dl \]

\[
\begin{align*}
&= \begin{cases} 
1.0 & \text{Is} \\
\frac{\alpha \beta}{2} & \text{Im} \\
\sqrt{\frac{\beta M_y}{(1 + \beta)s_H dl^2}} + \frac{1}{5} & \text{III}_{\beta} \\
\sqrt{\frac{4\beta M_y}{s_H dl^2 (1 + \beta)}} & \text{IV}
\end{cases}
\end{align*}
\]
Appendix B

Analysis of Stringer’s (1991) Results

Comments

- Stringer (1991) determined embedment properties using a modified Nordtest (1981) method. Owing to inevitable misalignment in the specimen, a two-nail test cannot avoid creation of moment that the nails must resist. This will promote premature failure due to splitting. Embedment strength was calculated based on maximum attained load.

- He compared nail yield stresses obtained from tension and 3-point bending tests on nails, and tensile quality control tests on wire. Used three nail diameters (2.87, 3.06 and 3.33mm). Obtained similar values for the two tensile tests, but the 3-point bending values were about 5 percent higher. Is not clear whether quoted tension values are actually yield or maximum stresses. If we had to guess we would say they are maximum values. Otherwise they are too close to the bending values. Stringer used the tension values for nails in EYM calculations. Values are in the ‘normal’ range (about 800 MPa). It may be relevant to point out here that Ehlbeck et al. (1990) carried out comparative tests of a wide variety of nails and nail wires in Europe to determine the yield moment of nails. They observed that the yield moment in nails cannot be derived accurately from yield stress obtained from regular nail wire tension tests, and recommended that the yield moment be obtained from a direct nail bending test using the apparatus described in Ehlbeck and Larsen (1993), and which is now in the European Standard EN 409.

- Joint tests were in accordance with AS 1649. Used a two-nail specimen. Left a 0.8mm gap between joint members. Although intended for machine driving, nails were driven by hand using a hammer. There were no pre-drilled lead holes. Tests were stopped following attainment of a peak load, or at 7.5mm deformation. Stringer recommended the use of four or more nails in testing of joints with thin plywood members to get a more stable joint configuration.

- There seems to be an inconsistency in the way 5th percentile characteristic values are calculated from embedment (best-fit value) and joint (tolerance-interval value based on Leicester method) data.

- Results imply that for mode III, the EYM over-estimates the capacity if the head-side member is thin (e.g., tests with a 7mm plywood member). The implication is that the ‘simplified EYM’ proposed by Blass et al. (1999) will give a better prediction.
Re-Analysis of Data

The proposal by Blass et al. is given below:

\[ P_Y = s_H d l \begin{cases} 1 / [1 + ((1 + \beta)/\beta)^{1/2}] & \text{interpolation} \\ \alpha \beta /[1 + (1 + \beta)^{1/2}] & \text{interpolation} \\ \min \left\{ [(4 \beta M_Y)/(s_H d l^2 (1 + \beta))]^{1/2} \right\} & \text{IV} \end{cases} \]

*NOTE*: interpolation equations apply when members are too thin to generate a mode IV failure.

Accepting the material properties given by Stringer, the model by Blass et al. predicts the mean ultimate capacity \( (N) \) in the following table:

<table>
<thead>
<tr>
<th>Side member</th>
<th>( l - \ell_p : \alpha ) (mm)</th>
<th>( s_H ) (MPa)</th>
<th>Main member</th>
<th>( \beta )</th>
<th>( P_Y – \text{Test} ) (N)</th>
<th>( P_Y – \text{Blass} ) (N)</th>
<th>( P_Y – \text{Test} ÷ P_Y – \text{Blass} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radiata</td>
<td>35: 1.00</td>
<td>40.09</td>
<td>Radiata</td>
<td>1.0</td>
<td>1224 (III)</td>
<td>900 (III)</td>
<td>1.24 (1.24)</td>
</tr>
<tr>
<td>Slash ply</td>
<td>24: 1.54</td>
<td>66.11</td>
<td>Rad. LVL</td>
<td>0.771</td>
<td>1376 (III)</td>
<td>1186 (III)</td>
<td>1.16 (1.16)</td>
</tr>
<tr>
<td>Slash ply</td>
<td>7: 5.00</td>
<td>65.40</td>
<td>Radiata</td>
<td>0.589</td>
<td>732 (II)</td>
<td>530 (int. I)</td>
<td>1.38 (0.95)</td>
</tr>
<tr>
<td>Slash ply</td>
<td>7: 5.00</td>
<td>64.87</td>
<td>Slash</td>
<td>0.863</td>
<td>716 (II)</td>
<td>563 (int. I)</td>
<td>1.27 (0.82)</td>
</tr>
<tr>
<td>Radiata</td>
<td>35: 0.80</td>
<td>39.76</td>
<td>Messmate</td>
<td>1.552</td>
<td>1432 (III)</td>
<td>1087 (III)</td>
<td>1.32 (1.32)</td>
</tr>
</tbody>
</table>

*Notes*: 1- Nail diameter 3.06mm in all cases.  
2- \( M_Y \) taken as 3991Nmm  
3/ \( \alpha \) is ratio \( l_p / (l - \ell_p) \), where: \( l = 75 \text{mm and } \ell_p \text{ is penetration into point-side member.} \)  
4/ In columns denoted \( P_Y – \text{Test} \) or \( P_Y – \text{Blass} \) values in parentheses indicate the failure mode according to the ‘original EYM’ designation.  
5/ In column denoted \( P_Y – \text{Test} ÷ P_Y – \text{Blass} \) values in parentheses are the ratio calculated by Stringer who used the full (original) set of EYM equations.

Additional Observations/Conclusions

- The simplified EYM due to Blass et al. is always (could say excessively) conservative. Conversely the original EYM is too liberal in cases where failure involves formation of
only one plastic hinge, which for the joint arrangements studied was located in the point-side member.

- It is controversial/debatable how to estimate strength of a joint from a test record exhibiting a very ductile response, which is usually the case for joints with laterally loaded nails. Bearing this in mind, ratios of test to predicted strength quoted by any author are subjective.
Appendix C

Notations

The following notations are used in this report:

- \( \alpha \) = ratio of thickness of main/centre member to thickness of side member
- \( \alpha_m \) = material resistance partial coefficient = \( \phi^{-1} \)
- \( \alpha_{Fi} \) = partial load coefficient
- \( \sum \alpha_{Fi} F_i \) = factored design action effect
- \( \beta \) = ratio of embedment strength of main member to embedment strength of side member
- \( \Box \) = capacity factor (known as a resistance factor in North America)
- \( (\Box N) \) = \( \Box_k \ldots k_j n Q_k \) = factored resistance
- \( d \) = diameter of fastener
- \( EYM \) = European Yield Model
- \( F_i \) = load effect from wind, snow, etc
- \( k_i \) = modification factor accounting for departures from reference conditions for \( Q_k \)
- \( k_{mod} \) = composite modification factor
- \( l \) = thickness of side member(s)
- \( l_p \) = thickness of head-side member in a single-shear nailed joint
- \( l_p \) = penetration into point-side member
- \( LRFD \) = Load and Resistance Factor Design
- \( LSD \) = Limit States Design
- \( M_Y \) = yield moment of fastener
- \( n \) = number of fasteners
- \( n_{ef} \) = effective number of fasteners
- \( N^* \) = factored design action effect
- \( P_Y \) = yield load per fastener per joint plane
- \( Q_k \) = characteristic capacity of a fastener
- \( R_d \) = \( R_k k_{mod} / \alpha_m \) = factored resistance
- \( R_k \) = characteristic resistance
- \( s_H \) = embedment strength of side member(s)
- \( WSD \) = Working Stress Design