Defining Appropriate Limit States for Design of Timber Connections in Australia and New Zealand

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ABSTRACT: Both Australia and New Zealand design standards for timber structures are in limit states format, but these are first generation soft conversions of previous working stress design standards. It is anticipated that development of a new combined Australian and New Zealand standard for design of timber structures will commence shortly. There is a shared view amongst some researchers that the current approach is limited, since it does not distinguish between serviceability and "ultimate" strength events and is not particularly relevant for connections in high performance timber structures. This paper discusses these matters and presents an overview of relevant literature and research work that has been undertaken to date, with recommendations for future development.

1 INTRODUCTION

1.1 *Background*

The 1997 edition of the Australian Timber Structures Code AS 1720.1 (SAA 1997) was the first Australian code to provide a limit states design (LSD) procedure for timber construction. However, whilst some revisions were included to reflect new data and information resulting from research and development that had occurred since the previous edition, the LSD edition was essentially a soft conversion of the previous working stress design (WSD) edition (SAA 1988). It is important to note that the code provisions for design of connections were entirely "soft-converted" to the LSD format.

1.2 *Current Limit States Format of AS1720.1*

A number of researchers believe that the current approach is limited, since it does not distinguish between serviceability and "ultimate" strength events and is not particularly relevant for connections in large or highly loaded timber structures, such as LVL portal frames where the material is being used in a strength critical application

Most connections in timber structures are designed for the strength limit state and whilst Appendix C3 of AS1720.1 contains some approximate methods for estimating the initial stiffness and deformation of nailed and bolted joints, the joint deformation models are quite simplistic and are based on limited empirical data. As such, they have limited value and as a result serviceability limit states of connections are seldom checked.

The inherent ductility of the metal fasteners means that the failure of connections can often be ductile, provided the timber members being "joined" together have adequate thickness. However, the large range of connection types and the fact that a single connection between two members may involve many connectors (fasteners) each of which may carry residual stresses and non uniform load distribution, means that connection behaviour is often quite difficult to accurately model. As such, AS1720.1 in its current form presents some rather simplified models for predicting the capacity of timber connections, based upon the performance an individual fastener loaded in single shear.

Section 4 of AS1720.1 contains clauses for design of a limited range of connection types (noted below) and specifies the relevant design equations, capacities and modification factors to use. Design provisions are included for type 1 (lateral loading) and type 2 (axial loading) joints using the following fasteners: nails, screws, bolts and coach screws. Design provisions for less commonly used shear-plate fastener connections are also included. There are currently no design provisions for dowel (snug fit) or epoxy injected connectors.

The LSD edition of AS1720.1 has the following generic format for design of connections:

$$(\phi N_j) \geq N^*$$

where:

$$(\phi N_j) = \phi\, k_1 \ldots k_j\, n\, Q_k$$

and
N^* = factored design action effect
ϕ = capacity factor
k_i = modification factors
n = number of fasteners
Q_k = characteristic capacity of a fastener

The characteristic capacity for any connector is dependent upon the fastener diameter, the joint group (classified on the basis of timber density groups and whether the timber is seasoned or unseasoned), the effective thickness of the timber and the number of shear planes for the fastener. As in most other codes throughout the world, a duration of load factor is included, based upon the 'Madison curve' and other factors are included to reflect the influence of head rotation and non linear / non uniform load distribution in connections consisting of multiple rows of fasteners.

For the purposes of the remainder of this paper, the scope of discussion will be essentially limited to performance and design of bolted connections.

2 HISTORICAL DEVELOPMENTS:

Australian joint design procedures are essentially based on an empirical fit of test data, which to a large degree, was derived from early North American studies, particularly those undertaken by Trayer (1932). Additional research was subsequently undertaken to supplement Trayer's work for application to Australian timbers - particularly hardwoods (Langlands and Thomas 1939). Allowable bolt loads for different joint groups were derived on the basis of establishing a permissible average stress under a bolt as a function of "t / D", where "t" is the minimum thickness of a timber member in the connection and "D" is the diameter of the bolt.

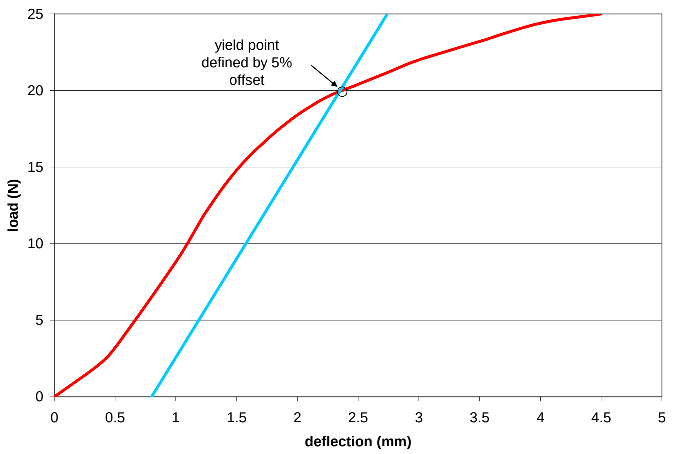

Figure 1 – example of yield point defined by 5% proof off-set

Stress ratios proposed by Langlands accord directly with Trayer's original plots, for a particular joint group, and these were subsequently re-plotted as "load" vs. "t/D" graphs (Pearson et al. 1958). For "t/D" ratios of 4 or greater, the permissible capacity is essentially defined by the yield point of the fastener, usually defined by a proof offset of the load vs. deflection curve of 5% of the faster diameter, as noted in Figure 1, for an M16 bolt.

A comparison of current design capacities with those derived from this early testing (which was primarily undertaken by CSIRO), confirms that there have been no essential changes from the original "allowable strength" values as published in the "Timber Engineering Design Handbook" (Pearson et al. 1958), other than inclusion of capacities for kiln dried material (25% increase permitted), inclusions of joint groups J5 / JD5 and J6 / JD6, metrication and soft conversion to the limit states format (Lhuede 1988). Leicester (1993) has also provided a brief but valuable overview of the development of the design procedures for connections contained in AS1720.1.

3 CURRENT SITUATION:

From the previous section it is noted that current provisions for connection design are based on empirical tests, and were (particularly for bolts) developed from background studies undertaken in the United States in the first half of the 20th century, which were adapted for use with unseasoned native hardwood timbers. This has raised a number of issues over the past 15 or so years, particularly as new materials and engineered wood products (EWP's) such as LVL have emerged and with them, more widespread use of such products in non residential applications.

Whilst the 1997 edition of AS1720.1 attempted to address reliability issues associated with non residential applications for design of timber members, such as defining capacity reduction factors that reflect the type of member, the consequence of failure and the inherent material variability and quality control in its production; the soft conversion of the provisions for connection design means that consistent reliability provisions for joining the structural members together, have yet to be adequately defined.

Not only have new timber products emerged, but also new (at least for Australia) types of connections for which there are no existing code provisions have also become available and are being used in practice. Examples of these include the use of steel fin plates with snug fitting dowels, type 17 "tek" screws (commonly used as type 2 fasteners for securing steel roofing) used as type 1 - lateral loaded connections, epoxy injected steel bars and nail plate reinforced bolted connections.

3.1 *Important Developments*

At the same time two other important developments have shaped current thinking about future requirements of connection design. The first is the development of a joint Australian – New Zealand standard (actually a series of standards known as AS/NZS BBBB), which are intended for evaluation of complete joint systems that specify actual in-service configurations and loads. Unlike the current standard AS 1649 for determining characteristic strengths of mechanical fasteners (SAA – 2001), AS/NZS BBBB contains rational and consistent methods for determining the characteristic strength and applying relevant load factors to obtain joint properties.

The second development has been interest in overseas research on the use of analytical behaviour models of connectors, which now form the basis for design codes in Europe and North America (Soltis and Wilkinson – 1987). The principles underlying timber connection modelling have been essentially based upon a theory first developed by Johansen in 1949, commonly referred to as "European Yield Theory" (EYT).

Significant work has already been undertaken in Australia to validate Johansen's equations for simple connections using Australian timbers.

Both theoretical and experimental investigations into the potential of EYT for design of bolted connections have also been undertaken at the University of Technology, Sydney (UTS) since 1990, the results indicating that good agreement has been found between experimental results and yield theory for a number of commercially available timber species, using single fasteners in three member joints. Stringer (1993) examined the applicability of European Yield Theory to nailed connections using pine, LVL and hardwood species and an extensive research program has been recently completed to develop yield model based design procedures for both nailed and bolted pine connections (Foliente et al – 2001).

4 DEVELOPMENT OF YIELD THEORY MODELS:

Johansen's equations predict the ultimate strength of a laterally loaded "dowel-type" connection due to either a failure of the joint members (ie. in the timber), or the development of localised crushing in the joint members while the fasteners show plastic behaviour similar to that illustrated in Figure 2.

Using these models, the mode of failure is determined by the joint geometry and the material properties - the fastener yield moment and the embedment strengths of the timber or wood based materials, as noted in Figure 3.

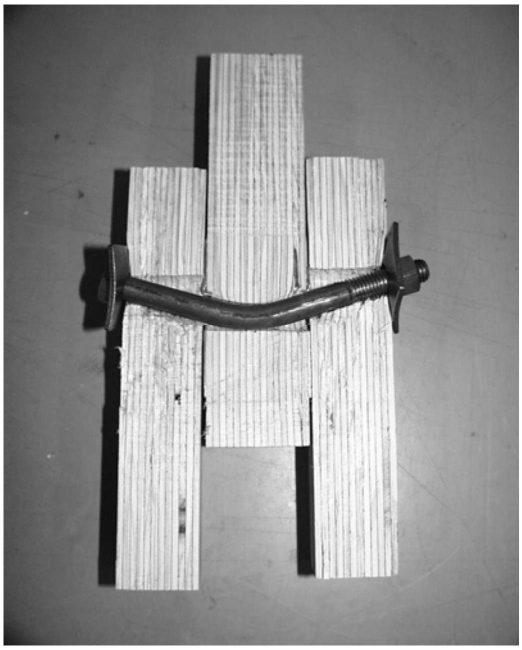

Figure 2 – section through bolted connection indicating plastic behaviour

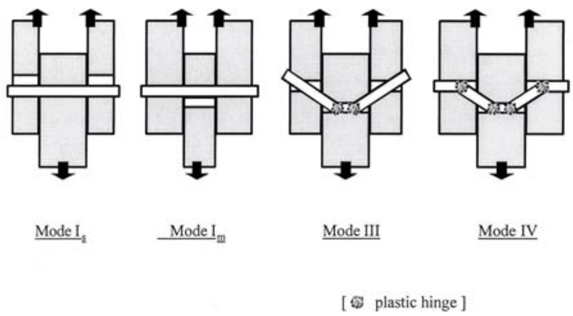

Figure 3 – EYT failure modes for 3 member joints (Foliente and Smith – 2000)

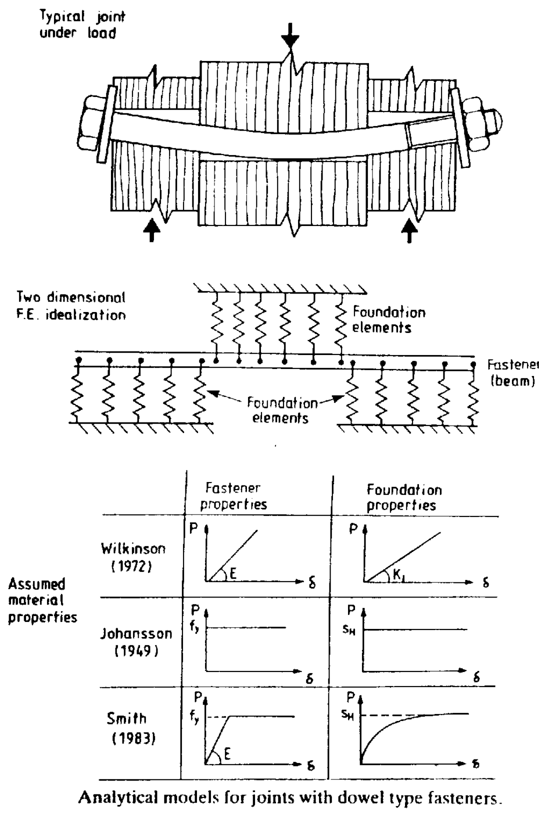

Figure 4 - analytical techniques for representing ductile behaviour of steel dowels and / or bolts (from Smith et al, 1987)

Johansen's equations have been further developed to more accurately model interaction of the timber and fasteners under laterally loading (Hilson et al, 1990), as noted in Figure 4. These modified equations have now been incorporated in the Eurocode 5 design procedures for connections.

5 CHALLENGES FOR THE FUTURE:

At the present time in Australia, large spanning timber structures which utilise high capacity, sophisticated connection systems are generally undertaken by specialist timber engineering consultants. This is because these types of connection systems are beyond the present scope of AS1720.1. In a number of cases the large connections used in such structures have been developed from advanced analytical models of connection behaviour and confirmed with full scale prototype testing of the connection, using AS/NZS BBBB.

Therefore, both the development of a testing standard such as AS/NZS BBBB, and the application of yield theory models are essential for developing an appropriate reliability based methodology for defining limit states design procedures for connections. It is the author's view that mechanical based formulas allow greater flexibility for designers than the use of traditional empirical based design equations. There also appears to be international acceptance of yield theory models as being an appropriate way of estimating the capacity of joints with single "type 1" laterally loaded fasteners, be they nails, bolts or dowels. However, there are still significant challenges to be addressed.

5.1 *Lack of International Consensus*

The importance of test standards such as AS/NZS BBBB must not be understated, as a potential means for determining "universally accepted" characteristic values that are necessary for any harmonization of design codes. With the exception of nailed connections, where there is a general acceptance of "a sensibly linear relationship between ultimate capacity and the number of nails" (Smith and Foliente – 2002), there is little international consensus on methods for determination of joint capacity where there are multiple connectors.

In most non residential timber structures the magnitude of loads is relatively high and few if any connections will be constructed using a single fastener. This can lead to a significant problem in that the "system" behaviour of a connection or joint can be brittle, even though the behaviour of the individual fasteners is quite ductile. Thus it is important to develop mechanically based models that have been verified by full scale testing, to ensure adequate strength whilst maintaining appropriate "ductile" failure modes that ensure partial, rather than catastrophic collapse under extreme loading events.

It is also important to recognise that in most "real" structures utilising engineered wood products the behaviour of connection systems is generally non-linear due to various effects including:

- time dependent deformation under long duration loads (creep)
- short duration deformations are often non-linear with respect to load. Wood tends to respond in a non-linear fashion under the high bearing stresses generated against each connector in Type 1 connections.
- joint deformations vary under successive load applications. As the wood fibre crushes against the connectors, load is redistributed (both between the fasteners in the connection, and even along the shank of a single fastener). The response characteristics of the connection change each time load is applied.

These effects can occur in the connection system, even though the behaviour of the fasteners themselves remain linear elastic. Other effects that influence deformation under serviceability loads (particularly for bolted connections) are:

- clearance between fasteners and holes
- tolerances in hole position and alignment
- residual or secondary load effects in the structure
- lack of fit of elements
- load transfer in the structure may not be as assumed or determined by the analysis
- eccentricities in the connection

6 FUTURE NEEDS:

Noting the challenges in the previous section, it is important that future research seeks to develop models that quantify the extent of non linearity and permanent deformations that can be safely sustained in connection systems, particularly at ultimate load limit states. These characteristics of connections were recognised by the authors who developed the AS/NZS BBBB procedures.

An example of recent research and development at UTS, of high capacity connections using LVL illustrates this point. The characteristic capacity of an M16 bolt in 45mm thick LVL (Joint group JD4) loaded parallel to grain in single shear, indicates a characteristic capacity of 12800 N. The only departure from code requirements was that the bolts were in snug fitting holes. Testing of 10 specimens using AS/NZS BBBB resulted in a normalised (taking into account the AS1720 phi factor of 0.7) characteristic capacity of 12750 N – so there was no significant difference in the predicted capacity, even though the test method allowed for a limited amount of deformation at the maximum load. This is because, the characteristic capacity is based upon a 5th percentile value (of maximum loads), which is not all that different a value from that obtained using a proof offset.

However, if the same connection is modified by reinforcing it with nail plates (Figure 6), the performance changes quite dramatically and the load deflection curve exhibits a significant amount of ductility with increasing load – well beyond the "proportional limit", as seen in Figure 5 (noting that specimens are 3 member joints with 2 shear planes).

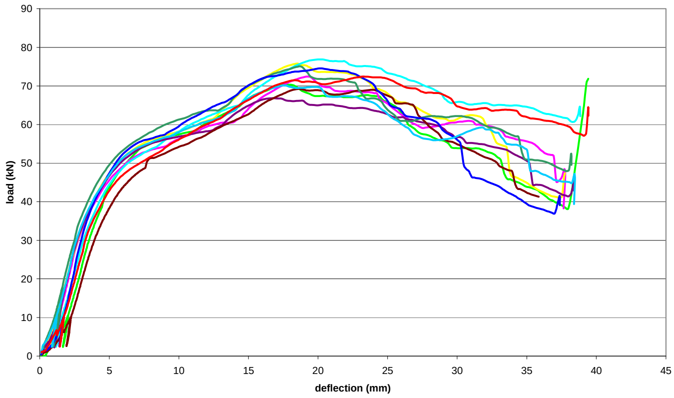

Figure 5 – testing of M16 bolts in reinforced LVL


Figure 6 – reinforced LVL connection

In the case of the reinforced connection, the basic characteristic capacity is 32000 N whilst the normalised characteristic capacity increases to 36900 N, which is close to the average and is nearly three times that specified in AS1720.1. At these loads, deformations in the joint are of the order of 15mm, as opposed to 2 to 2.5mm at the proportional limit. Failure in the wood occurs only after the bolt has torn through the nail plate, at deformations exceeding 25mm.

7 CONCLUSIONS:

For this type of "high performance" connection the benefits of high strength can only be realised if permanent deformations are permitted in joints at the strength limit state. Under normal serviceability loads, the behaviour of a single fastener is effectively linear elastic. As loads increase, some permanent deformation results; although it is interesting to note that significant recovery occurs when the load is released prior to reaching about 30000 N.

Whilst yield theory models are appropriate for predicting the behaviour of "normal" bolted or dowel connections made from single fasteners, new models are needed to quantify system behaviour and ensure ductile performance at the strength limit state. New models will also need to be developed for design of high capacity connections, which could be described as having bi-linear behaviour and demonstrate considerable load capacity beyond the point where initial yielding of the fastener occurs. The effect of such joint deformations on the structure as a whole needs to be understood, but it is likely this can be accurately modelled using structural analysis software if the stiffness / ductility of the connection itself is quantified.

There is a compelling argument to separate characteristic capacities for timber connections for serviceability and ultimate limit states - provided the behaviour of a connection is effectively linear elastic under serviceability loads and ductile up to the ultimate capacity. The potential benefits are significant and given the relative cost of connections as a proportion of the total cost of a large building, use of high capacity connections that exhibit both adequate stiffness and ductility could lead to increased cost competitiveness for large timber structures in non residential markets.

Clearly considerable further research is required to understand and quantify the extent of "damage" or permanent deformations that can be safely sustained at the strength limit state and the level of ductility that is appropriate for given load events. This will need to be done not only for single fasteners, but also systems of connections and appropriate models will need to be developed to predict the behaviour of both individual fasteners and connection systems. It is the author's view that such work is essential for developing a reliability basis for connection design in future codes.

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