1. INTRODUCTION

Design Capacity Tables for Timber are a design aid for structural engineers. They provide assistance in the limit states modelling of timber behaviour and can be used for:

- design - the selection of members to carry given design loads
- capacity determination - performance prediction of known members

The use of the tables follows sensible design methods, and allows a designer to feel as though they are still in control of the design process. A designer is still required to choose appropriate design parameters such as capacity factors and some of the modification factors.

In design, the use of the tables effectively eliminates time consuming iterations associated with normal "guess and check" design methods. The tables use limit states loads and comply with the requirements of the limit states version of AS1720.1:1997 and amendments.

The Tables are mounted on the web as pdf files and can be downloaded one stress grade at a time. The tables can then be printed out for hardcopy use.

There are two pdf files that give an introduction to the DCTT and examples on the use of the DCTT. This is the first of those documents and covers:

- An overview of the DCTT including its objective
- An introduction to the processes of limit states design
- Background information on the development of the DCTT

The file “Guidance.pdf” gives:

- Guidance and examples on the use of the DCTT for performance (capacity) evaluation
- Guidance and examples on the use of the DCTT for design calculations.

In this document, the conventions are as follows:

- [Table F5s.bend.1] refers to Table F5s.bend.1 of the tables.
- [Table *.bend.1] refers to the first bending table for any grade of timber. Eg it includes [Table F5s.bend.1], [Table MGP10.bend.1],... [Table A17s.bend.1],...
- <Table 2.5> refers to Table 2.5 in AS1720.1 (1997) limit states version of the Timber Design Code
- Other codes are referenced in the book, but have the code within the brackets. Eg <AS/NZS1170.1:2002 2.3> refers to clause 2.3 in AS/NZS1170.1 – Structural Design Actions.
- References to sections in the DCTT have been shown as follows: §1.2.3.
- Tables of design data for timber sections are presented in the DCTT as separate pdf files for each grade.
- Appendix A has general tables of design information (mainly drawn from AS1720.1:1997)
1.1 Brief Overview

The full Design Capacity Tables for Timber (DCTT) are published by Forest and Wood Products Research and Development Corporation. Their format is similar to that of the Design Capacity Tables for Structural Steel (AISC, 1994). This will make it easier for practicing structural engineers to navigate.

However, the differences in behaviour between steel and timber, and the differences in code models for the two materials will mean that there are some differences between ways in which the steel and timber capacity tables can be used. The differences are primarily due to the use of modification factors ($k_1$ to $k_9$) in the Limit States Timber Design Code.

The DCTT provides information for:

- F grades in seasoned and unseasoned timber
- MGP grades (Australian pine)
- A17 grade (seasoned Australian ash-type hardwoods)

Bending
Compression
Tension

State and national timber advisory services and some national suppliers were contacted about most common sizes and grades for inclusion in the Tables. There are plans for extending the Tables to cover other generic grades as nationally available material in uniform sizes becomes available.

1.1.1 Objectives of DCTT

The principle objective of the DCTT is to provide a design tool that simplifies the calculations associated with limit states code models of timber behaviour under load. It does not address domestic framing members (though it could be used to design framing). Domestic framing is most efficiently accomplished using appropriate timber framing tables in AS1684. The DCTT is a more general timber design aid, which will assist structural engineers in timber design of other structures.

In terms of the structural design processes associated with ensuring appropriate timber members are used in structures, a number of steps are required. The DCTT addresses only the step associated with structural performance modelling (shown bold below):

- Selection of structural geometry - spans and spacings.
- Determination of design loads (AS/NZS1170).
- Structural analysis to determine member actions.
- Selection of timber species and grading based on cost, availability and durability performance modelling.
- Detailing and drawing of members and structure.
- Construction.

The concept of a design tool can be subdivided into a number of more specific requirements. The data should:

- reflect commonly available grades and cross sections of timber members.
- give information on serviceability performance over a range of structural actions.
- give information on strength performance over a range of structural actions.
- be available for key actions and for different restraint configurations.

1.1.2 Target Users

The DCTT has been developed for use by structural engineers. Structural engineering skills are required to perform all of the steps in the structural design of members as listed in section §1.1.1 above. The use of the
DCTT will not release an engineer from their responsibilities in selecting loads and the detailing of the structure.

Structural engineers generally perform two main tasks for which the capacity of members is required:
- Design - in which the loads and geometry are known and a member must be selected so that it has sufficient capacity to withstand the loads. This is the process of sizing members for structures.
- Performance modelling - in which the member and geometry are known and the designer must find the capacity of the member in that context so that the capacity can be compared with some applied loadings. Examples of performance modelling include the checking of designs, and the evaluation of the performance of existing structures.

Extra background detail and explanation has been included in section §1 to enable the DCTT to be used by students and by professionals who are not as familiar with timber design.
1.2 Limit States Models of Timber Behaviour

This section contains a very brief description of the design of structural timber members using the limit states code AS1720.1 (1997). It only presents enough information to enable the use of the DCTT. It cannot be used in lieu of AS1720.1. Readers are referred to the full text of AS1720.1, and to text materials for more complete explanations. The following text has been written with an Australian perspective.


(Other international limit states timber design books are available):

1.2.1 Limit States Design

All structural elements experience a huge range of action events over their lifetime. The resulting loads vary in magnitude and direction from day-to-day in most structures, and for bridge elements, from second to second. The limit states philosophy is that the structural elements must all continue to deliver appropriate performance over their lifetime. An appropriate performance is rather hard to define. It is probably best thought of as an absence of unsatisfactory performance, because unsatisfactory performance is much easier to define:

- **Unsatisfactory strength**: Where a structural element fails (breaks or buckles or becomes detached so that it is no longer able to sustain its structural function), that element has displayed unsatisfactory strength performance. In most cases, unsatisfactory strength performance poses a risk to life, so society would regard it as an event that should have a very low probability of occurrence. This is modelled in structural codes using an equation of the form of equation (1) where the loads used are factored loads with a very low probability of exceedence. The design capacity expression ($\phi R$) is the appropriate structural action at which failure could be expected.

- **Unsatisfactory serviceability**: Where a structural element deforms so much that it creates problems for the occupants of the building, then it is not performing satisfactorily. Examples of this include; sagging that is unsightly, deflection that causes water to pond or interferes with drainage paths, deformation that causes cracking, deflection that means structural members rest on non-structural elements such as glass partitions, deformation that interferes with the normal opening and closing operations of doors and windows. These events cause annoyance and inconvenience, but do not pose a threat to life. Most occupants would tolerate any of these events providing they are very rare. Regular occurrences would certainly cause a problem. These events are modelled using loads that occur at about the same frequency as regular maintenance - once in five to twenty years. At these loads, the structural elements can be modelled elastically. *Equation (2) in §1.2.2.4 gives an example.*

Structural design processes should involve evaluation of performance at each of these limit states.

\[
\phi R \geq R^* \tag{1}
\]

\[
(\phi R) = \text{Design capacity (strength limit state)} \quad <\text{AS1720.1}>
\]

\[
R^* = \text{Design load combination for strength limit state} \quad <\text{AS1170.1}>
\]
1.2.2 Serviceability Limit State

The serviceability limit state is one that is concerned with the satisfactory function of the building. Violations of the serviceability limit state can be caused by:

- unacceptable appearance which reduces the impact or feeling of safety in a structure (e.g., sagging beams)
- malfunction of building equipment or services (e.g., jamming windows or doors)
- load transmission through unwanted paths that may damage non-structural items (e.g., doors and windows)

1.2.2.1 Serviceability actions

Serviceability limit state combinations of actions have a probability of exceedence of around 5% per year. This gives an acceptable risk of non-performance. The serviceability combinations of actions can be found by using <AS/NZS1170.0 4.3>. Any deformation limit may only apply to a combination of a few of the actions listed in the standard. Some examples are:

- \( G + \psi_{l} Q \) may be used where the total deformation under long-term actions is required for clearance under a beam.
- \( G + \psi_{s} Q \) may also be used where the total deformation is required for clearance under a beam. (In this case, the short-term imposed action (live load) will be larger than the long term imposed action, but \( j_{2} \) for the short-term imposed action will be less due to its smaller duration.)
- \( \psi_{s} Q \) may be used where the change in deformation under imposed actions (live loads) can cause movement that will be detected by the human eye, or cause discomfort. Here, as the permanent action (dead load) is constant, it is not a problem as it will not cause any movement. It is only those loads that vary quite rapidly with time that will be the problem loads.

Serviceability actions are in general close to the nominal loads on the element. The permanent action (dead load) used in serviceability combinations is that which is estimated by a designer. The imposed action (live load) is close to and less than the nominal imposed action level given in the standard. Typically long-term serviceability imposed actions (live loads) are around 40% of the nominal imposed action, and short-term serviceability imposed actions are around 70% of the imposed action specified in <AS/NZS1170.1 3>.

1.2.2.2 Deformation (deflection) limits

<AS1720.1 Appendix B> gives some comment on the deflection limits for beams. Simply stated, deflection limits must be agreed between designer and client or client's representative (architect). Each deflection limit represents a particular type of non-performance of the serviceability limit state that the designer is trying to anticipate. Each deformation limit should have:

- limit either absolute (e.g., 20 mm) or relative (e.g., span/250)
- load case to which the limit applies (e.g., short-term imposed actions alone)
- consequence of exceedence of the limit (e.g., unwanted load path through glass window)

1.2.2.3 Design Modulus of Elasticity

While exceeding the serviceability limits does not usually compromise safety, some serviceability scenarios are more serious than others. Designers must evaluate the consequence of exceedence carefully to enable them to select an appropriate \( E \) value. As timber properties vary between different lengths of timber, there is always a finite chance that the \( E \) value of any particular length will be less than the value assumed in design. Lower assumed \( E \) values have a lower probability of exceedence. Designers frequently make use of two \( E \) values:

- \( E_{k} \) characteristic Modulus of Elasticity (close to the average value and the main value used \( E_{av} \))
- \( E_{5\%} \) an estimate of the 5\th percentile of \( E \) (used where extra conservatism is needed).
### 1.2.2.4 Serviceability model

The serviceability limit state is modelled using elastic deflection formulae, but a correction factor is applied to increase the calculated deflections to allow for the effects of creep under longer-term loads. The timber design code allows the use of an equation such as *equation (2)* - which is appropriate for prediction of deformation of a simply supported beam under uniformly distributed load.

\[
\delta_{\text{total}} = \frac{5}{384} \sum (w_i j_2) L^4 \quad \text{for bending (udl over a SS span)} \quad (2)
\]

- \(\delta_{\text{total}}\) = Total long-term deflection of beam under serviceability loads \(\Sigma w_i\)
- \(L\) = Design span
- \(E\) = Design Modulus of Elasticity
- \(I\) = Design second moment of area of cross section about bending axis
- \(j_2\) = Duration of load factor for serviceability (compression and bending)
- \(w_i\) = a uniformly distributed loading in a serviceability combination of actions

#### 1.2.2.5 Duration of load - serviceability \(j_2\)

The correction factor for creep effects is \(j_2\) and is applied to each component of the load combination. It is a function of the duration of each component of the load, and is also affected by the initial moisture content of the timber. The actual value of the \(j_2\) factor is independent of the size of the timber member used.

Beams are in some cases limited by their serviceability performance. For long and medium span beams, designers will often select a beam on the basis of its serviceability and then check that its strength is adequate. The deflection of a beam under bending can be modelled using *equation (3)*. Note that *equation (2)* is a special case of this more general formula for the case of a simply supported span with a uniformly distributed loading.

\[
\delta_{\text{total}} = \sum \left( g_a j_2 (w_i P_i) \right) \quad \text{for general bending members} \quad (3)
\]

- \(g_a\) = Geometry factor for type of loading and support over the design span \(L\)
- \(j_2\) = Duration of load factor for serviceability applies to each contributing load in the combination
- \(w_i\) = a uniformly distributed loading in a serviceability load combination
- \(P_i\) = a concentrated load in a serviceability load combination

The part of *equation (3)* in \([\ ]\) is the data that is tabulated in relevant tables in the DCTT.
• $g_a$ is a function of the loading and support configuration and for some simple cases can be found in Table A1. $g_a$ is independent of the size of the cross sectional dimensions chosen for the member, but it incorporates an expression for the design span $L$.

• $j_2$ is found in <2.4.1.2>. The relationships for $j_2$ are reproduced here as Table A2 and Figure A1. $j_2$ values are independent of cross section.

1.2.3 Strength Limit State

The strength limit state is one that is concerned with the satisfactory safety of the structure. Violations of the strength limit state can be caused by:

• fracture of the material of the structural member
• buckling of the structural member
• fracture of a significant part of the structural member (eg the glue line or finger joint in a glulam beam)
• fracture of a connection or of the member at a connection

Any of these problems can cause the member to lose its capacity to perform its designated structural function. (It will no longer be capable of resisting its design loads.)

1.2.3.1 Strength limit state loads

Strength limit state load combinations have a probability of exceedence of roughly 5% over the lifetime of the structure (a return period of roughly once in a thousand years). This gives an acceptable risk of non-performance. The strength load combinations can be found in <AS/NZS1170.0 4.2>. Because any violation of the strength limit state in most structures, poses a threat to human life, all possible strength load combinations must be checked by a designer. This is in contrast with the serviceability load combinations in which only some may be deemed to apply. All loads that apply during a given loading scenario must be included in the combination. (For example, permanent actions (dead loads) apply to all combinations because they are always there.) Some examples of strength limit state load combinations are:

• $1.2G + 1.5Q$ is a valid combination for most building elements. Here the imposed action (live load) is a peak live load and the loading is necessarily a short-term action.

• $1.2G + 1.5 (\psi_c Q)$ is also valid for most building elements. Here $(\psi_c Q)$ is a long-term imposed action (live load).

• $1.35G$ is a valid load combination and is of permanent duration. (In many cases the previous combination gives higher loads.)

• $1.2G + \psi_c Q + W_u$ is a load combination that is appropriate where wind actions cause loads in the same sense (direction) as the gravity loads caused by permanent and imposed actions (dead and live loads).

• $0.9G + W_u$ is a load combination involving wind actions in which the wind action is in the opposite sense (direction) to the permanent actions (and imposed actions.) The addition is a vectorial addition so in this case, involves subtracting two numbers.

Strength limit state loads are in general much larger than the nominal loads on the element. The permanent action (dead load) used in strength limit state load combinations involving only gravity loads is 1.2 times that which is estimated by a designer. This is a conservative estimate of the permanent actions over the lifetime of the structure and allows for variation in density of materials. The short-term strength limit state imposed action (live load) is much larger than the nominal imposed action given in the structural design actions standard. This allows for the wide range in loadings associated with varying use of a structure. Typically long-term strength imposed actions $(\psi_c Q)$ are around 40% of the nominal imposed action, and short-term serviceability imposed actions are around 70% of the <AS/NZS1170.1> imposed action.

All combinations of actions that may possibly act on the element over the life of the structure must be evaluated and checked.

• For all structural materials, the direction of load can be critical for structural response. Restraint of bending members has different effects on the strength of the element depending on whether it is on the tension or compression edge. A lower net upward force may cause
more problems for the element than a higher gravity load where the only lateral restraint is to the top edge.

- For timber and other materials where the strength is a function of the duration of the load, combinations involving long-term imposed actions (live loads) must be evaluated as well as those based on the short-term (peak) imposed actions.

### 1.2.3.2 Design Strength Properties

In manual design calculations, the design strength is derived from a characteristic strength times a geometric property. The selection of the appropriate characteristic strength is a function of the material and its grading or production. This has been incorporated into the DCTT so that a designer need not understand the derivation of the property to find its design capacity.

However the design capacity is also a function of the capacity factor $\phi$ and the grading method needs to be known to find $\phi$. The grading is generally included in the specification of the structural element. Further detail is given later in this section.

### 1.2.3.3 Strength modelling

The strength limit state is checked by ensuring that the design capacity of the member is greater than the load effect (factored for the strength limit state). The general case is illustrated in equation (1). The combined effects of the load factors and the capacity factor $\phi$ give a level of safety that is uniform across all of the major building materials.

The strength model for timber will be illustrated in this section using the design capacity of bending members. The form of the equation for bending members at the strength limit state is given by equation (4). The design capacity must be greater than the strength limit state bending effect on the member. The design bending strength is given in equation (5). It shows that the behaviour model is based on the elastic modulus of the cross section $Z$ and the ultimate bending strength $f'_b$. The basic bending strength $\phi f'_b Z$ is modified for the design setting by use of modification $k$ factors.

$$
(\phi M)_{M^*} \geq M^* 
$$

(for bending strength)  

$$
(\phi M) = \phi k_1 k_4 k_6 k_0 [k_{11} k_{12} f'_b Z] 
$$

(for bending strength)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M^*$</td>
<td>Design bending moment (factored for strength limit state)</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Capacity factor</td>
</tr>
<tr>
<td>$k_1$</td>
<td>Duration of load factor for strength</td>
</tr>
<tr>
<td>$k_4$</td>
<td>Partial seasoning factor</td>
</tr>
<tr>
<td>$k_6$</td>
<td>Temperature factor</td>
</tr>
<tr>
<td>$k_0$</td>
<td>Strength sharing factor</td>
</tr>
<tr>
<td>$k_{11}$</td>
<td>Size factor</td>
</tr>
<tr>
<td>$k_{12}$</td>
<td>Stability factor for beams</td>
</tr>
<tr>
<td>$f'_b$</td>
<td>Characteristic bending strength</td>
</tr>
<tr>
<td>$Z_x$</td>
<td>Section modulus about the xx (major axis)</td>
</tr>
<tr>
<td>$Z_y$</td>
<td>Section modulus about the yy (minor axis)</td>
</tr>
</tbody>
</table>

The part of equation (5) in [ ] is the data that is tabulated in relevant tables in the DCTT.
1.2.3.4 Capacity factor ($\phi$)

The capacity factor for structural timber differs a little in its form from the capacity factor for other structural materials. It is a function of the method in which the structural properties were derived in the first place, and the role of the element in the structure. <Table 2.5> shows the capacity factor $\phi$ for members. Different values are used for connections. (See <Table 2.6>.)

The capacity factor $\phi$ can be found using the following steps:

**Step 1** The type of material is selected. This is a decision that has to be made early in the design process. It is quite possible to perform designs for a number of different materials, but each will have their own capacity factor and will use different tables in the DCTT. Examples may include A17, MGP material or F-graded material. ESL, Glulam, LVL may also be chosen, though the tables do not cover those materials at this stage.

**Step 2** For some materials, the type of grading used to produce and stamp the material must also be selected. (This will need to be included in the specification for the purchase of the material, so finding it is not wasted effort.) MGP, A17, LVL, and Glulam are all products for which the production method is specified and no categorisation of the grading is necessary. F-graded material has different $\phi$ factors for the different grading methods. The grading method is shown on the grade stamp on the material, and stockists will be able to give advice on the grading standard used for specific products they carry.

**Step 3** The type of action is identified. Bending, shear, bearing, compression or tension are selected.

**Step 4** The role of the element in the structure must be selected. There are four choices:
- Any elements in housing. In domestic housing, the timber members tend to be used in conjunction with many other similar members. This gives the structure an inherent resilience as the members can share load with each other. A premature failure of one member will invariably shed load to other members, and while the structure may deform, a significant structural failure is not likely. Even failure of principal structural elements such as roof beams which do not have redundancy, may cause loss of only a small part of the structure and may not present a serious risk to life. These elements can be found in the leftmost column of $\phi$ factors in <Table 2.5> of AS1720.1:1997.
- Secondary elements in other types of structures. These elements do not have a significant structural function. Their failure would not induce a significant structural failure. Examples include: cladding support members, partition framing, wall framing in cases where it is not part of the principle structural support system. Because of the low risk to life presented by failure of any of these elements, they are treated the same as housing elements and their $\phi$ factor can be found in the leftmost column of $\phi$ factors in <Table 2.5>.
- Primary elements in normal structures. These elements have a significant structural function, so their failure would generally cause a significant collapse and present risk to life. The higher degree of reliability required for these elements means that they have lower $\phi$ factors than the secondary elements. Their $\phi$ factors are found in the central column of $\phi$ factors in <Table 2.5>.
- Primary elements in structures with a specific post-disaster function. These elements require an even greater degree of reliability, so must have even lower $\phi$ factors. (This is in addition to the special importance classes for wind and earthquake actions which must also be applied.) Their $\phi$ factors are found in the rightmost column of $\phi$ factors in <Table 2.5>.

**Step 5** The capacity factor $\phi$ is found by using <Table 2.5> in AS1720.1:1997 or [Table A4] in the DCTT, finding the appropriate row of the table for the material and grading selected at Step 1 and Step 2 and for the action found in Step 3. The appropriate column for the structural role is found in Step 4.
1.2.3.5 Modification factors for timber

Bending, tension and compression members all use different combinations of the same $k$ factors, with bending members using the largest number of them. They can be grouped as follows:

**Modification factors that are not functions of the size of the member**

- $k_1$: duration of load factor (strength) function of load characteristics
- $k_4$: partial seasoning factor function of environment
- $k_6$: temperature factor function of environment
- $k_9$: strength sharing factor function of geometry of structural system

**Modification factors that are functions of the size of the member**

- $k_{11}$: size factor function of cross section dimensions
- $k_{12}$: stability factor function of dimensions and restraint

Timber’s behaviour is a function of its remarkable microstructure, and each of the above effects represents a complex relationship between strength and the parameter indicated. The behaviour model presented in AS1720.1 (1997) is quite simplified. The modifications are assumed to be independent, and this is largely the case.

In terms of the design process, $k_4$, $k_6$, and $k_9$ can be found immediately after the structural scheme has been selected and $k_1$ after the design loads calculated. Their values will also be common to many members in the entire structure so values previously calculated for other members may be used. Designers will need to evaluate these factors for all timber members, whether they use the DCTT or not. None are difficult to find.

$k_{11}$ and $k_{12}$ are a function of the member cross section and so need to be evaluated each time that a member is sized. (They also happen to be the most difficult to calculate.)

- Under manual design processes, the size factor $k_{11}$ is tabulated in the code or manufacturer’s information for some timber materials and must be evaluated from a formula for others. In common with steel member design, the stability factor $k_{12}$ requires careful evaluation of the lateral restraint system, determination of a slenderness parameter and then calculation of a stability factor $-\frac{\alpha_s}{\alpha_c}$ or $\alpha_c$ and for timber $- k_{12}$.
- When using the DCTT, these two factors are incorporated in the capacities given in the table and do not have to be evaluated explicitly. (They are inside [ ] in equation (5).)

1.2.3.6 Duration of load factor - strength ($k_1$)

Duration of load for the strength limit state is the total time over the life of the structure for which the element is expected to be loaded at or above the designated load level. It is the accumulated duration of the specified loading that is important. At the time of design, this will not be known but can be estimated from the character of the loading on the structure.

In any given load combination, it is the shortest-term loading that will determine the duration of load factor that must be used for that load combination. AS1720.1 uses a set of nomenclature for different classes of load duration. Each of these classes of load duration can be associated with a period of loading:

- permanent,
- long-term,
- medium-term,
- standard test,
- instantaneous or wind gust.

[Table A5] gives some recommended durations of load and appropriate values of $k_1$ for various types of imposed action (live load).
As the character of the loading affects the value of $k_1$ for each load combination and this in turn affects the final design strength of the member, all possible load combinations must be considered before the critical design case can be identified.

### 1.2.3.7 Critical load case for strength limit state

The capacity of a member is a function of the duration of loading for the load combination under consideration. Each strength limit state load combination can have a magnitude of load and a duration of load attached to it. The duration of load assigned will influence the design capacity. Long duration loads result in lower strengths. Thus the critical load for the strength limit state need not be the largest load, a lesser load of longer duration may prove more onerous for design. The critical load combination can be found by evaluating a duration of load parameter $D_L$ for each of the strength limit state load combinations. $D_L$ is given by equation (6).

$$D_L = \frac{w^*}{k_1}$$

The critical load case for the strength limit state can be found by following these steps:

**Step 1** Find all possible load combinations for the strength limit state. <AS/NZS1170.0 4.2>

**Step 2** Evaluate the appropriate design action for the member. eg for a bending member find the bending moment, for a compression member, the axial compression, or for a tension member, the axial tension force.

**Step 3** For each load combination, find the duration of load and appropriate $k_1$ factor using <Table 2.7> in AS1720.1 or [Table A5] in the DCTT.

**Step 4** For each load combination, find the duration of load parameter $D_L$ using equation (6).

**Step 5** The load combination with the highest value of $D_L$ is the critical load combination for the strength limit state. If the strength limit state inequality is satisfied for the critical load combination, then all other load combinations will also satisfy the inequality for that design action.

### 1.2.3.8 Partial seasoning factor ($k_4$)

The partial seasoning factor allows for the fact that the properties of timber can change as moisture moves into or out of the timber.

AS1720.1 gives some models for the partial seasoning effect in timber. These were derived using data collected from tests on small clear specimens of wood. The code models the two scenarios in which timber may become partially seasoned:

- Timber which is initially seasoned may become partially seasoned when it picks up moisture from the air in extremely damp environments. (Seasoned timber has an initial moisture content of less than 15%, and becomes partially seasoned when its moisture content rises above 15%.)
- Timber which is initially unseasoned may become partially seasoned when it loses moisture in service. This generally happens in most indoor environments, and many external environments as well. (Unseasoned timber has an initial moisture content in excess of 25% and becomes partially seasoned when it has fallen to less than 25%). Some unseasoned timber which has been held in a yard for some time may even be delivered to site in a partially seasoned condition.

The code models for partial seasoning <2.4.2> have been presented in section §A6 of this document and is detailed in [Table A6].
In most circumstances, \( k_4 \) is 1.0.

- It is conservative to assume that \( k_4 = 1.0 \) for all unseasoned timber.
- It is only in very wet environments, such as cooling towers, that the \( k_4 \) given by AS1720.1 for seasoned timber is less than 1.0.

1.2.3.9 Temperature factor \((k_6)\)

The temperature factor \( k_6 \) allows for the fact that early work on the strength of small clear wood specimens showed that the strength of wood decreased as temperature increased. The timber design code uses geographical regions to divide Australia into two regions - normal temperatures and hot temperatures. This is presented in section §A7 of this document and detailed in [Table A7]

1.2.3.10 Strength sharing factor \((k_9)\)

This factor accounts for the fact that where a member is used in a system in which a number of closely coupled similar elements are combined, then the strength of the system is closer to the average strength of the elements than the strength of the weakest one. [Table A8] shows the members for which \( k_9 \) applies and the appropriate numbers to include in the code formula for calculating the value of \( k_9 \) Where unsure of the \( k_9 \) factor, it is conservative to use \( k_9 = 1.0 \)

All glulam and LVL members have \( k_9 = 1.0 \).

1.2.4 Bending Members

In some cases, the design of bending members is limited by the strength limit state. Where tight deflection limits are applied, the serviceability limit state may govern the design of the member.

1.2.4.1 Deflection of bending members

The serviceability of bending members has been detailed already in section §1.2.2.4. Deflection of a bending member is modelled using equation (2).

1.2.4.2 Bending capacity \((\phi M)\)

The flexural capacity of a member has been presented as equation (5). However the performance under ultimate limit states may also be limited by shear or by bearing capacity. The terms in brackets in equation (5) represent the data that is tabulated as the bending capacity in the DCTT. Bending capacity is limited by either material failure or for more slender members loaded in bending about their major axis only, by a buckling failure. Both material failure and buckling modes are included in the data presented in the DCTT.

Buckling of bending members is a lateral torsional buckling failure in which the member simultaneously twists about its longitudinal axis and moves laterally. Lateral torsional buckling is only possible for members that are bending about their major axis. Restraint against this type of failure is quite complex.

- The most effective restraint against lateral torsional buckling is restraint on the compressive edge of the member.
- Limited restraint against lateral torsional buckling can also be achieved by lateral restraint to the tensile edge of the bending member.
- Restraint against lateral torsional buckling can also be achieved by giving both lateral restraint to the tensile edge of the member and restraint against twisting. Twisting restraint is most frequently achieved in practice by “blocking” parallel members against each other, or by the provision of fly bracing.

_in order to determine the flexural capacity of a bending member, the restraint against buckling must be correctly evaluated._

AS1720.1 gives models for the following types of restraint of bending members:
• lateral restraint of compression edge at $L_{ay}$ (effectively, both lateral and torsional restraint at $L_{ay}$)
• lateral restraint of tension edge at $L_{ay}$ (effectively only lateral restraint at $L_{ay}$)
• Full Lateral Restraint (FLR) of the tension edge (either continuous restraint or restraint at spacings less than $L_{ay}$) with separate torsional restraint at $L_{ay}$

The maximum restraint spacing to give Full Lateral Restraint, $L_{ay}$ (FLR), can be found using equation (7).

$$L_{ay} (FLR) = 64 d \left( \frac{b}{\rho_b d} \right)^2$$  \hspace{1cm} (7)

$L_{ay} (FLR) =$ Distance between lateral restraints for Full Lateral Restraint
$d =$ Depth of member (largest cross sectional dimension)
$b =$ Width of member (smallest cross sectional dimension)
$\rho_b =$ Material factor for bending elements

1.2.4.3 Shear capacity ($\phi V$)

Shear is not a common failure mode in timber beams, but none the less, short bending members can have performance that is limited by the member shear capacity, particularly if they are short span beams. The shear force design inequality is shown as equation (8). This reflects the same limit states design philosophy espoused for bending members, but with shear forces rather than bending moments as the design action. Shear capacity of a member with a rectangular cross section is given by equation (9). The terms in brackets represent the data that is tabulated as the shear capacity in the DCTT.

$$\left( \phi V \right) \geq V^*$$ \hspace{1cm} (8)

$$\left( \phi V \right) = \phi k_1 k_4 k_6 k_{11} f'_s (0.667 A)$$ \hspace{0.5cm} (rectangular cross section only) \hspace{1cm} (9)

$(\phi V) =$ Design shear capacity of a timber beam
$V^* =$ Design shear force (factored for strength limit state)
$\phi =$ Capacity factor
$k_1 =$ Duration of load factor for strength
$k_4 =$ Partial seasoning factor
$k_6 =$ Temperature factor
$k_{11} =$ Size factor
$f'_s =$ Characteristic shear strength
$A =$ Design cross sectional area of member

As the design shear strength $f'_s$ cannot be found by testing with the same level of reliability as the design flexural strength $f'_b$, the capacity factor $\phi$ is frequently lower for shear than for bending actions. This can be seen in [Table A4].

The area used in equation (9) is the shear area for rectangular cross sections - 0.667 $A$. For other cross sectional shapes, a different shear area must be used. The DCTT contains only rectangular sections, so this is the area used in the tables.
1.2.5 Compression Members

For nearly all compression members, the strength limit state governs the performance of a given cross section under load. It is rare that the serviceability limit state is checked.

1.2.5.1 Deformation of compression members

Elastic deformation of axially loaded timber compression members is given by equation (10). This shows the classic elastic compression model modified by the serviceability duration of load factor $j_2$.

$$\delta_{\text{total}} = \sum \left( j_2 \frac{N_i}{A E} \right) L \quad (10)$$

- $\delta_{\text{total}}$ = Total long-term deformation of axially loaded members under serviceability loads $\Sigma N_i$
- $L$ = Design length of member
- $E$ = Design Modulus of Elasticity
- $A$ = Design cross sectional area of member
- $j_2$ = Duration of load factor for serviceability (compression and bending)
- $N_i$ = an axial loading in a serviceability load combination

The part of equation (10) in [ ] is the data that is tabulated in relevant tables in the DCTT.

The duration of load factor used for the serviceability of compression members is identical to the one used for the serviceability of bending members and is presented in [Table A2].

1.2.5.2 Compression capacity ($\phi N_c$)

The design inequality for compression is presented as equation (11), and the compression capacity of an axially loaded member can be found in equation (12).

$$\left( \phi N_c \right) \geq N^* \quad (11)$$

$$\left( \phi N_c \right) = \phi k_1 k_4 k_6 k_9 \left[ k_{11} k_{12} f'_{c} \right] A \quad (12)$$

- $(\phi N_c)$ = Design compressive capacity of a timber member
- $N^*$ = Design axial force (factored for strength limit state)
- $\phi$ = Capacity factor
- $k_1$ = Duration of load factor for strength
- $k_4$ = Partial seasoning factor
- $k_6$ = Temperature factor
- $k_{11}$ = Size factor
- $k_{12}$ = Stability factor
- $f'_{c}$ = Characteristic compression strength
- $A$ = Design cross sectional area of member

The part of equation (12) in [ ] is the data that is tabulated in relevant tables in the DCTT.

The compression capacity of a member is often limited by the potential for buckling failure. There are two potential buckling axes to consider - the major axis of the member and the minor axis of the member. In calculation of the compression capacity of a member, a designer must carefully examine the restraint provided by other elements in the structure to give the spacing (measured parallel to the length of the compression member) between restraining elements that prevent lateral movement in the minor axis direction ($L_{ax}$) and those that prevent lateral movement in the major axis direction ($L_{ay}$). Figure 1.2 shows the restraint against lateral buckling in the two principle directions, and the definition of spacing of restraining elements.
The compression capacity of the member will be the lower value of the compression capacity limited by major axis buckling ($\phi N_{cx}$ a function of $L_{ax}$) and the compression capacity limited by minor axis buckling ($\phi N_{cy}$ a function of $L_{ay}$). The DCTT tabulates these two capacities separately. Figure 1.3 presents a flow chart for the determination of compression capacity by examining the restraint against lateral buckling under axial load.

Note that the compression capacity expression previously contained the k9 factor to be used with members built up by nail lamination, but this factor was removed from the compression capacity in an amendment.

1.2.6 Tension Members

For nearly all tension members, the strength limit state governs the performance of a given cross section under load. It is rare that the serviceability limit state is checked.

1.2.6.1 Deformation of tension members

Elastic deformation of axially loaded timber tension members is given by equation (13). This shows the classic elastic extension model modified by the serviceability duration of load factor $j_3$. $j_3$ is the duration of load factor for tension members. (Note that it is a different factor to the one used for bending and compression members.)

$$\delta_{total} = \sum \left( j_3 \cdot N_i \right) \frac{L}{A \cdot E}$$

(13)

The part of equation (13) in [ ] is the data that is tabulated in relevant tables in the DCTT.

The duration of load factor used for the serviceability of tension members is less than the one used for the serviceability of bending and compression members and is presented in [Table A3]. Observation of creep in timber show that it is less pronounced for timber in tension than for timber in compression.

1.2.6.2 Tensile capacity ($\phi N_t$)

The design inequality for tension is presented as equation (14), and the tensile capacity of an axially loaded member can be found in equation (15).

The tensile capacity is proportional to the tensile area $A_t$. $A_t$ is the minimum net area of section, and this is frequently a function of the method of connection of the member. The tensile capacity of a member is a function of the number and size of holes used in the connections along the length of the member. $A_t$ is found as the gross cross sectional area minus the area of the cross section removed by any holes.

Nailed connections do not reduce the timber cross section, so are counted as “no bolt” connections.
\[(\phi N_t) \geq N^* \]

\[(\phi N_t) = \phi k_1 k_4 k_6 [k_{11} f', A_t] \]

\(N^*\) = Design axial force (factored for strength limit state) \(<\text{AS1170.1}>\)

\(\phi\) = Capacity factor \(<\text{Table 2.5}>\)

\(k_1\) = Duration of load factor for strength \(<\text{Table 2.7}>\)

\(k_4\) = Partial seasoning factor \(<\text{2.4.2}>\)

\(k_6\) = Temperature factor \(<\text{2.4.3}>\)

\(k_{11}\) = Size factor \(<\text{2.4.6}>\)

\(f'\) = Characteristic tensile strength \(<\text{Table 2.4 and others}>\)

\(A_t\) = Design tensile cross sectional area of member \(<\text{3.4.1}>\)

The part of equation (15) in [] is the data that is tabulated in relevant tables in the DCTT.
1.3 Detail of DCTT

The DCTT presents data on the performance of timber members in such a way that it is of use to structural designers. In general, it only presents the data that is a function of the size and shape of the cross section.

For the strength limit states, all of the factors that are independent of cross sectional dimensions - \( \phi, k_1, k_4, k_6, k_9 \) - can be selected by a designer once a material has been selected for use on the job, near the beginning of the design process. Subsequent selection of a cross section will not affect these factors, so they are not included in the capacities tabulated in the DCTT. This gives a designer maximum flexibility in the design of any member, as the design parameters that reflect the conditions of use can be carefully chosen for each individual application.

However, the parts of the design strength expressions that are enclosed within the brackets in equation (5) will be affected by the cross sectional dimensions of the member selected. This is the value tabulated in the DCTT. Any of the design capacity expressions – for example equation (5) – can be rewritten in the form presented as equation (16). This shows the way in which the bending data given in the tables can be related to the design bending strength. This leads to a design process in which the equation (17) can be used directly to select an appropriate member from the tables.

\[
(\phi M) = \phi k_1 k_4 k_6 [k_{11}, k_{12}, f', b, Z] \\
\text{for bending strength (5)}
\]

\[
(\phi M) = \phi k_1 k_4 k_6 [M_T] \\
\text{with } [k_{11}, k_{12}, f', b, Z] = [M_T] \\
\text{for bending data (16)}
\]

\[
(\phi M) \geq M^* \\
\text{for bending strength (4)}
\]

gives

\[
[\phi k_1 k_4 k_6 [M_T] \geq M^* \\
\text{design expression for bending members (17)}
\]

The left hand side of equation (17) can be found directly from the DCTT and the right hand side is given by the strength limit state design action divided by some design factors that are independent of member size.

For the serviceability limit state, the deformation expression can be used to select members that have sufficient section property. For serviceability, \( EI \) or \( EA \) values can be used to select members.

For example equation (2) gave the deformation under serviceability loading. For a given design scenario, the deformation must be less than the deformation limit.

\[
\delta_{\text{total}} \leq \delta_{\text{lim}} \\
\delta_{\text{total}} = \sum (g_a j_2 (w_i, P_i)) / [E I] \\
\text{for general bending members (3)}
\]

hence

\[
[E I] \geq \sum (g_a j_2 (w_i, P_i)) / \delta_{\text{lim}} \\
\text{for serviceability design of bending members}
\]

Two different limits are used:

- \([EI_{\text{avg}}]\) or \([EA_{\text{avg}}]\) for limits appropriate to appearance or comfort
- \([EI_{5\%}]\) or \([EA_{5\%}]\) for limits appropriate to structural load paths or damage
1.3.1 Information in Tables
Data has been presented in the DCTT for a number of selected grades of timber. The grades have been selected as those that are available in most Australian major centres, and are produced in uniform sizes around the nation. It does not imply:

- that these are the only grades available at any one locality, or
- that all of the grades tabulated in the DCTT will be available in every Australian centre.

Other grades will be added to the DCTT when their availability can be demonstrated.

1.3.1.1 Common information to all tables
All tables have the following information:

Size
This is the nominal size by which the section is generally known. This column gives the dimensions that would be used on the drawings, specifications, and bill of quantities for the work.

Area
This is the design cross sectional area of the section, and it is provided in the second column of every table to give a basis for comparison between the sections that may be appropriate for a given application. (In most cases, the cost of a member is well correlated with its cross sectional area.) In the case of seasoned timber, the nominal cross sectional dimensions are used to calculate the design cross section. For unseasoned timber, 3 mm is subtracted from the nominal dimensions to allow for tolerances. The reduced dimensions are the design dimensions and the cross sectional area is calculated on them.

Serviceability data
This information is given for each structural action (bending, tension, or compression) and is presented as $[EI]$ or $[EA]$ as appropriate.

Strength data
The capacity data presented in the DCTT includes some (but not all) of the $k$ factors. An example is $[k_{11}, k_{12}, f_{b}Z]$ for bending data. For different loading configurations, the slenderness of beams will change, and so $k_{12}$ will also change. The bending data is presented in separate tables for different support and restraint conditions. Other actions tabulate combinations of size factors, stability factors strengths and geometric section properties.

1.3.2 Bending Data
Data on the bending performance of beams of each grade has been presented in four tables. These four tables model different restraint conditions for beams in service.

1.3.2.1 Restraint against lateral torsional buckling
It was observed in section §1.2.4 that the bending capacity of a member was a function of the restraint against lateral torsional buckling. [Figure 1.1] presents a flow chart that enables a designer to select an appropriate model and hence an appropriate table for the restraint conditions.

To determine the correct table to use for the bending capacity, a series of questions must be answered.

- **Bending moment axis** - lateral torsional buckling is only a potential problem for major axis bending. Minor axis bending does not produce any tendency to lateral torsional buckling, so the capacity is not a function of the restraint of the member. It is simply a function of the cross sectional properties and is given in the first of the bending tables for each material [Table *.bend.1]. For major axis bending moments, the capacity is a function of the restraint of the member, and further questions are required.
• **Restraint to Compression/Tension edge** - restraint of the compression edge is more effective than restraint to the tension edge, as it gives both lateral and torsional restraint. Different models are used for restraint of tension and compression edges, so different tables are used. Capacity of beams with compression edge restraint is given in [Table *.bend.2]. For tension edge restraint, further information is required to determine which table is appropriate.

• **Torsional restraint** - tension edge restraint is less effective than compression edge restraint, but can be enhanced in some circumstances by also providing torsional (twist) restraint. This only really improves the buckling response where the lateral restraint is close enough for the beam to be regarded as Fully Laterally Restrained (FLR). Where the spacing of the restraint of the tension edge is at greater centres than that for FLR, the capacity is given by [Table *.bend.3]. Where the restraint of the tension edge is at closer centres than the distance for FLR and there is torsional restraint, capacity is given by [Table *.bend.4].

• **Spacing of restraints** - for all of [Table *.bend.2], [Table *.bend.3] and [Table *.bend.4], the capacity of the member is a function of the spacing of the restraints. The required spacing to give Full Lateral Restraint is given in [Table *.bend.1].

---

**Figure 1.1 - Flow chart for assessment of restraint of beams in DCTT**

Bending Moment Axis?

- Major axis - (potential buckling)

- Minor axis - (no buckling)

Lateral Restraint of comp or tens edge?

- Compression edge restrained (@ $L_{ay}$)

- Tension edge restrained (@ $L_{ay}$)

$L_{ay} < L_{ay}$ (FLR)?

- Yes, $L_{ay} < L_{ay}$ (FLR)

- No, $L_{ay} > L_{ay}$ (FLR)

$L_{ay}$ as distance between lateral restraints.

$L_{ay}$ (FLR) from [Table 2.*.1] or [Table 2.*.4]

$L_{ay} = \text{dist between torsional restraints}$ (use "none" if no torsional restraints)

- [Table *.bend.2]

- [Table *.bend.4]

- [Table *.bend.3]

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1.3.2.2 General information on bending [Table *.bend.1]

This table contains basic information needed for the design of bending members. It is used for the following tasks:

- **Serviceability modelling of beams.** This is the only place in which serviceability data for beams can be found.
- **Minor axis strength modelling.** This is the only table in which minor axis bending data is presented.
- **Major axis strength of fully restrained beams.** Where spacing of restraints to lateral movement and / or twisting are less than the distances presented in this table, the major axis strength can be found from the values tabulated in this table. Where the restraint against lateral torsional buckling is at greater distances than this, the other bending tables should be used.

[Table *.bend.1] contains the following information:

**Size and Area** - see section §1.3.1.1

Flexural stiffness - EI

In finding the serviceability performance of a beam, the properties associated with the flexural stiffness of the member are incorporated in $EI$. The general equation for deflection of beams, *equation (3)*, has this property in the denominator. The DCTT presents a range of different flexural stiffness values for each cross section. They are all given as $10^9$ N mm^2 as follows:

- **Major axis [$EI_{avg}$]**
  - This property is used for members that are in major axis bending and for which the structure has some ability to share loads among parallel members. It can also be used where violation of the deflection limit causes visual problems only.

- **Major axis [$EI_{5%}$]**
  - This property is used for members that are in major axis bending and for which the structure does not have ability to share loads among parallel members. It is used where violation of the deflection limit may cause some damage or may affect drainage paths.

- **Minor axis [$EI_{avg}$]**
  - This property is used for members that are in minor axis bending and for which the structure has some ability to share loads among parallel members. It can also be used where violation of the deflection limit causes visual problems only.

- **Minor axis [$EI_{5%}$]**
  - This property is used for members that are in minor axis bending and for which the structure does not have ability to share loads among parallel members. It is used where violation of the deflection limit may cause some damage or may affect drainage paths.

Major axis bending properties

Beams are most efficiently used in major axis bending. In this configuration, any cross sectional area has the maximum value of $Z$. Hence, the beam has the potential to return the highest design moment capacity which is proportional to $f_b Z$. However, for beams bending about their major axis, the design capacity may be influenced by potential for lateral torsional buckling.

- **Bending capacity - major axis (FLR)** $\left[ M_{Tx} \right] \left( k_{12} = 1 \right)$
  - This is the term $[k_1, k_{12} f_b Z]$ when $k_{12} = 1.0$. It is the maximum bending moment that the beam can carry, and only really applies where the beam can experience no lateral torsional buckling (ie it has Full Lateral Restraint). Where the beam does not have full lateral restraint, its capacity must be found by reference to [Tables *.bend.2] - [Tables *.bend.4], depending on the type and frequency of spacing of restraint. In the DCTT, it has moment units - (kN m).
  - To find the design moment capacity ($\phi M_{Tx}$), $[M_{Tx}]$ is multiplied by $\phi k_1 k_4 k_6 k_9$

- **$L_{avg}$ for compression edge restraint that gives Full Lateral Restraint** $\left[ L_{avg}(FLR) \right]$
  - This is the maximum spacing of lateral restraints on the compression edge that will give Full Lateral Restraint. Where the lateral restraints are on the compression edge, and the spacing
of the lateral restraints ($L_{ay}$) is less than $L_{ay}$ (FLR), the moment capacity of the member is the one tabulated as $[M_{Ty}]$ ($k_{12} = 1$) – in the column immediately to the left of this figure. Units for this spacing are metres.

- $L_{a\phi}$ for spacing of torsional restraint to give Full Torsional Restraint ($L_{a\phi}$ (FTR)) This is the maximum spacing of torsional restraints in addition to lateral restraints at spacing less than $L_{ay}$ (FLR) on the tension edge to give full restraint so that $k_{12} = 1.0$. Where the spacing of the torsional restraints is less than $L_{a\phi}$ (FTR) and the spacing of tension edge lateral restraints is less than $L_{ay}$ (FLR), the moment capacity of the member is the one tabulated as $[M_{Ty}]$ ($k_{12} = 1$) – in the column to the left of this figure. Units for this spacing are metres. The key point for the use of this figure is that there must be Full Lateral Restraint by providing resistance to out-of-plane movement of the tension edge, AND additional torsional restraints at spacing $L_{a\phi}$.

**Bending capacity - minor axis (FLR)**

This is the term $[k_{11} k_{12} f'_b Z_y]$ for minor axis bending. With minor axis bending $k_{12}$ always equals 1.0. The minor axis bending capacity is independent of any restraint as lateral torsional buckling is not a possibility with minor axis bending. No further tabulation of minor axis capacity is necessary. It is only a function of the cross section, not the span or restraint. In the DCTT, it has moment units - (kN m).

To find the design moment capacity ($\phi M_y$), $[M_{Ty}]$ is multiplied by $\phi k_1 k_4 k_6 k_9$

**Shear capacity**

This is the term $[k_{11} f'_s A_s]$. It is the shear capacity of the cross section divided by those design parameters that are not dependent on the actual cross section chosen. This capacity is independent of any restraint on the member or the length of span. It is not tabulated in any other part of the DCTT. The units are kN.

To find the design moment capacity ($\phi V$), $[V_T]$ is multiplied by $\phi k_1 k_4 k_6$

### 1.3.2.3 Bending - lateral restraint of compression edge [Table *.bend.2]

This table contains information on major axis bending capacity of beams where there is lateral restraint of the *compression* edge of the beam. It is used for the following tasks:

- **Major axis strength of beams with lateral restraint of the compression edge** Where there are some other elements in the structure that prevent lateral movement of the compression edge of the beam, the major axis strength can be found from the values tabulated in this table. Other tables should be used where the restraint is not connected to the compression edge of the beam.

[Table *.bend.2] contains the following information:

**Size and Area** - see section §1.3.1.1

$L_{ay}$ for compression edge restraint to give Full Lateral Restraint ($L_{ay}$ (FLR)) This is the maximum spacing of lateral restraints on the compression edge to give Full Lateral Restraint of the beam. Where the spacing of the compression edge restraints is less than $L_{ay}$ (FLR), the moment capacity of the member can be read from [Table *.bend.1], or the column for $L_{ay} = 0$ in [Table *.bend.2]. If the spacing of lateral restraint of the compression edge are at closer centres than this figure, then decreasing the spacing of the restraint will not increase the bending capacity of the beam.
Bending capacity - major axis \[ M_{TS} \]

This is the term \([k_{11} k_{12} f'_{b} Z_{x}]\). In this table the major axis bending capacity has been evaluated and presented for a number of different spacings of lateral restraint. The spacing of lateral restraint is shown as \(L_{ay}\) and is given in metres across the top of this table. Linear interpolation of \(L_{ay}\) is permitted. The bending capacity has moment units - (kN m).

To find the design moment capacity \((\phi M_{x})\), \([M_{TS}]\) is multiplied by \(\phi k_{1} k_{4} k_{6} k_{9}\)

1.3.2.4 Bending - lateral restraint of tension edge [Table 2.*.3]

This table contains information on major axis bending capacity of beams where there is lateral restraint of the tension edge of the beam. It is used for the following tasks:

- Major axis strength of beams with lateral restraint of the tension edge. Where there are some other elements in the structure that prevent lateral movement of the tension edge of the beam, the major axis strength can be found from the values tabulated in this table. [Table *.bend.4] may be used where the lateral restraint is at closer centres than \(L_{ay}\) (FLR), and there is torsional restraint as well.

[Table *.bend.3] contains the following information:

Size and Area - see section §1.3.1.1

Bending capacity - major axis \[ M_{TS} \]

This is the term \([k_{11} k_{12} f'_{b} Z_{x}]\). In this table, the major axis bending capacity has been presented for a number of different spacings of lateral restraint. The spacing of lateral restraint is shown as \(L_{ay}\) and is given in metres across the top of this table. Linear interpolation of \(L_{ay}\) is permitted. The bending capacity has moment units - (kN m).

To find the design moment capacity \((\phi M_{x})\), \([M_{TS}]\) is multiplied by \(\phi k_{1} k_{4} k_{6} k_{9}\)

1.3.2.5 Bending - FLR of tension edge and torsional restraint [Table 2.*.4]

This table contains information on major axis bending capacity of beams where there is lateral restraint of the tension edge of the beam at spacings less than \(L_{ay}\) (FLR), together with torsional restraint. There are two critical spacings to be checked here:

- \(L_{ay}\) is the spacing of the lateral restraint on the tension edge. (For rafters in roofs under uplift, this would be the batten or purlin spacing.)
- \(L_{a\phi}\) is the spacing of the torsional restraint on the beam. (For many applications, this will be the spacing between the fly braces or blocking pieces.)

In order for this table to apply, there are two characteristics of the restraint system that must be identified.:

- FLR of the tension edge. Some elements in the structure must prevent lateral movement of the tension edge of the beam. Where these are discrete elements, they must be spaced at less than the distance required for Full Lateral Restraint \([L_{ay}\) (FLR)]]. (This is presented in both this table and in [Table *.bend.1].)
- Torsional restraint. There must be some intermediate torsional restraint of the beam. This can be provided by fly braces as shown in the illustration, or by blocking pieces between adjacent parallel beams.
Table *.bend.4 contains the following information:

Size and Area - see section §1.3.1.1

Restraint data

Data on lateral restraint for this configuration is quite complicated. Lateral restraint of the tension edge must be very effective. A maximum spacing is given.

- \( L_{ay} \) for lateral restraint to give Full Lateral Restraint \([L_{ay} (FLR)]\) This is the maximum spacing of lateral restraints to give Full Lateral Restraint. The spacing of the lateral restraints (which for this table are on the tension edge) must be less than \( L_{ay} \) (FLR). The data for \( L_{ay} \) (FLR) is also given in [Table*.bend.1], but it is reproduced in this Table for ease of checking in the use of this table.

- \( L_{a\phi} \) for lateral restraint to give Full Torsional Restraint \([L_{a\phi} (FTR)]\) Given that there is the required lateral restraint as given above, then \( L_{a\phi} \) (FTR) is the maximum spacing of torsional restraints to give Full Torsional Restraint. (This means that \( k_{12} = 1.0 \) and the capacity will be given by the major axis capacity \([M_{TX}\]) \([k_{12}=1]\) in [Table *.bend.1]. This information is not required to make use of this table. However, if the spacing of torsional restraint of the beam is at closer centres than this figure, then decreasing the spacing of the torsional restraint will not increase the bending capacity of the beam.

Bending capacity - major axis \([M_{TX}]\)

This is the term \([k_{11}, k_{12}, \phi, Z_{a}]\). In this table, the major axis bending capacity has been presented for a number of different spacings of torsional restraint. The spacing of torsional restraint is shown as \( L_{a\phi} \) and is given in metres across the top of this table. Lateral restraint of the tension edge must also be provided. The lateral restraint must be closer than the spacing required for this to be regarded as Full Lateral Restraint - \( L_{ay} \) (FLR). Linear interpolation of \( L_{a\phi} \) is permitted. The bending capacity has moment units - (kN m)

To find the design moment capacity \( (\phi M_{a}) \), \([M_{TX}]\) is multiplied by \( \phi, k_1, k_4, k_6, k_9 \)

1.3.3 Compression Data

Data on the compression performance of timber members of each grade has been presented in two tables. These two tables model buckling performance about the two principle buckling axes.

1.3.3.1 Restraint against lateral movement under axial buckling

The capacity of compression members is often limited by buckling under axial load. The axial load capacity is the lower of its major axis capacity and its minor axis capacity. The capacity in either direction is affected by the restraint offered to the member by other elements in the structure as shown in Figure 1.2.

Figure 1.2 illustrates the nomenclature for restraint against axial buckling, and Figure 1.3 presents a flow chart that enables a designer to select an appropriate model and hence an appropriate table for the restraint conditions.

To determine the compressive capacity of an axially loaded member, both of the compression member tables must be used.

- Compression with major axis buckling - buckling that gives bending about the major axis is major axis buckling. It can be affected by other elements of the structure that prevent movement of the member which gives rise to major axis bending. These restraints are shown in Figure 1.2 and their spacing is \( L_{ax} \).

- Compression with minor axis buckling - buckling that gives bending about the minor axis is minor axis buckling. It can be affected by other elements of the structure which also prevent movement of the member which gives rise to minor axis bending. These restraints are also shown in Figure 1.2 and their spacing is \( L_{ay} \).
Lateral restraints against buckling under axial loads are most effective if they are positioned at the centroid of the section. The restraints against minor axis buckling in Figure 1.2 offer restraint to the full cross section. Where restraints for minor axis buckling are fixed to only one edge, there is a possibility that some torsional buckling will occur within the length of the member. Torsional buckling under axial load is quite difficult to model. For most compression members it does not reduce the strength of the member below that modelled by <AS1720.1> and presented in the DCTT.

### 1.3.3.2 Compression - major axis buckling [Table *.comp.1]

This table contains data needed to find the capacity of compression members where the strength is limited by buckling about the major axis. It needs to be used in conjunction with [Table *.comp.2] to find the limiting compressive capacity of the member. The effective length for major axis buckling $L_{ax}$ can be found using the following steps:

- Look for restraining elements that prevent movement of the member parallel to the $d$ (or larger cross sectional dimension). $L_{ax}$ is the largest spacing between these restraints. (There is one intermediate restraint (within the length of the member) in this direction in Figure 1.2.)
- Where there are no intermediate restraints in this direction, then $L_{ax}$ is found from equation (18).
- The value of $L_{ax}$ used for design is the appropriate value found in the previous steps.

$$L_{ax} = g_{13} L \quad \text{(no intermediate major axis restraints)} \quad (18)$$

- $L_{ax}$ = Effective length of a column for major axis buckling
- $g_{13}$ = Effective length factor
- $L$ = Design length of member

<Table A9>
Both major axis and minor axis capacities must be found

Potential for Major axis buckling

Identify elements that prevent lateral movement parallel to the $d$ dimension. These limit major axis buckling and are major axis restraints.

Restraint within the length of member?

No - Only Major axis buckling restraint @ end

$L_{ax} = g_{13} L$
($g_{13}$ from Table A9)

$\phi N_{cx}$ from [Table *.comp.1]

Yes - Major axis buckling restraints @ $L_{ax}$

Potential for Minor axis buckling

Identify elements that prevent lateral movement parallel to the $b$ dimension. These limit minor axis buckling and are minor axis restraints.

Restraint within the length of member?

Yes - Minor axis buckling restraints @ $L_{ay}$

No - Only Minor axis buckling restraint @ end

$L_{ay} = g_{13} L$
($g_{13}$ from Table A9)

$\phi N_{cy}$ from [Table *.comp.2]

$\phi N_{cy} < \phi N_{cx}$?

Yes

$\phi N_{c} = \phi N_{cy}$

No

$\phi N_{cy} < \phi N_{cx}$?

Yes

$\phi N_{c} = \phi N_{cx}$

No

$\phi N_{cx} < \phi N_{cy}$

Yes

$\phi N_{c} = \phi N_{cy}$

Figure 1.3 - Flow chart for design capacity of compression members in DCTT

[Table *.comp.1] contains the following information:

Size and Area - see section §1.3.1.1

Axial stiffness - $EA$

In finding the serviceability performance of a compression member, the properties associated with the axial stiffness of the member are incorporated in $EA$. The general equation for deformation of compression members, equation (10), has this property in the denominator. The DCTT presents a range of different axial stiffness values for each cross section. They are all given as $10^6$ N as follows:

- $[EA_{avg}]$ This property is used for members for which the structure has some ability to share loads among parallel members. It can also be used where violation of the deflection limit causes visual problems only.

- $[EA_{5%}]$ This property is used for members for which the structure does not have ability to share loads among parallel members. It is used where violation of the deflection limit may cause some damage or may affect drainage paths.
Compression capacity - Major axis buckling \[N_{Tcx}\]

This can be used to find the axial load capacity of a compression member - considering buckling about the major axis only. It is a function of the restraint spacing for movement in the major axis direction. For each cross section, ten different restraint spacings are given. These correspond to different effective lengths for major axis buckling. The effective length is given as \(L_{ax}\), and can be found by following the steps outlined above. The design capacity can be found from the value at the intersection of the section size (row) and \(L_{ax}\) (column). Linear interpolation can be used if required for values of \(L_{ax}\) not given in the tables.

\[N_{Tcx}\] represents the term \([k_{11} k_{12} f'c A_c]\) with \(k_{12}\) evaluated using the slenderness about the major axis. The cross sectional area taken is the gross cross sectional area. It is appropriate where there are no unfilled holes in the cross section. Bolt holes with bolts in them are quite OK. If there are unfilled holes in the cross section, then designers must evaluate the appropriate compression area \((A_c)\) removing the cross sectional area of the unfilled holes and then correct the capacity by multiplying \([N_{Tcx}]\) by \(A_c/A\).

To find the axial capacity of the section limited by buckling about the major axis (\(\phi N_{cx}\)), \([N_{Tcx}\]) is multiplied by \(\phi k_1 k_4 k_6\).

1.3.3.3 Compression - minor axis buckling [Table *.comp.2]

This table contains data needed to find the capacity of compression members where the strength is limited by buckling about the minor axis. It needs to be used in conjunction with [Table *.comp.1] to find the limiting compressive capacity of the member. The effective length for major axis buckling \(L_{ay}\) can be found using the following steps:

- Look for restraining elements that prevent movement of the member parallel to the \(b\) (or smaller cross sectional dimension). \(L_{ay}\) is the largest spacing between these restraints. (There are two intermediate restraints (within the length of the member) in this direction in Figure 1.2.)
- Where there are no intermediate restraints in this direction, then \(L_{ay}\) is found from equation (19).
- The value of \(L_{ay}\) used for design is the appropriate value of the two values found in the previous steps.

\[
L_{ay} = g_{13} L
\]  \hspace{1cm} (19)

\[
L_{ay} = \text{Effective length of a column for minor axis buckling}
\]

\[
g_{13} = \text{Effective length factor} \hspace{1cm} <\text{Table 3.2}>
\]

\[
L = \text{Design length of member} \hspace{1cm} [\text{Table A9}]
\]

[Table *.comp.2] contains the following information:

Size and Area - see section §1.3.1.1

Axial stiffness - \(EA\)

In finding the serviceability performance of a compression member, the properties associated with the axial stiffness of the member are incorporated in \(EA\). The general equation for deformation of compression members, equation (10), has this property in the denominator. The DCTT presents a range of different axial stiffness values for each cross section. They are all given as \(10^6\) N as follows:

- \([EA_{avg}]\) This property is used for members for which the structure has some ability to share loads among parallel members. It can also be used where violation of the deflection limit causes visual problems only.
• \([EA_{5\%}]\) This property is used for members for which the structure does not have ability to share loads among parallel members. It is used where violation of the deflection limit may cause some damage or may affect drainage paths.

**Compression capacity - Minor axis buckling \([N_{Tcy}]\)**

This can be used to find the axial load capacity of a compression member - considering buckling about the minor axis only. It is a function of the restraint spacing for movement in the minor axis direction. For each cross section, ten different restraint spacings are given. These correspond to different effective lengths for minor axis buckling. The effective length is given as \(L_{ay}\), and can be found by following the steps outlined above. The design capacity can be found from the value at the intersection of the section size (row) and \(L_{ay}\) (column). Linear interpolation can be used if required for values of \(L_{ay}\) not given in the tables.

\([N_{Tcy}]\) represents the term \([k_{11} k_{12} f'c A_c]\) with \(k_{12}\) evaluated using the slenderness about the minor axis. The cross sectional area taken is the gross cross sectional area. It is appropriate where there are no unfilled holes in the cross section. Bolt holes with bolts in them are quite OK. If there are unfilled holes in the cross section, then designers must evaluate the appropriate compression area \((A_c)\) removing the cross sectional area of the unfilled holes and then correct the capacity by multiplying \([N_{Tcy}]\) by \(A_c/A\).

To find the axial capacity of the section limited by buckling about the major axis \((\phi N_{cy})\), \([N_{Tcy}]\) is multiplied by \(\phi k_1 k_4 k_6\).

### 1.3.4 Tensile data

Data on the performance of tension members is presented in one table for each grade. This is perhaps the simplest data to model in limit states.

#### 1.3.4.1 Tensile capacity [Table *.tens]

These tables contain data needed to find the capacity of tension members. For F-graded timber, the tensile strength differs for softwood and hardwood species. However, each table indicates whether it applies for softwoods or for hardwoods.

Tensile strength is limited by material fracture and will occur at the position along the length where the cross sectional area is smallest. Any holes in the member will reduce its tensile capacity. Tensile capacity is a function of the number and size of the holes in the member.

A row of connectors is defined as a group of connectors along a line that is perpendicular to the line of action of the force. The line of action of force in a tension member is parallel to the grain. Thus a single row of fasteners is a group that is perpendicular to the grain. The number of bolt holes in a member for the calculation of tensile capacity is the number in a single row. Holes can be allowed for using the following steps:

- Examine the connections along the length of the tension member. Count the number of drilled holes **across the cross section** to find the number of holes in a single row.
- Find the diameter of the holes. For bolted connections, the hole diameter will be given by the size of the bolt. For unseasoned timber, the additional clearance for the hole has been allowed. Where screws or coach screws are used, the hole will be smaller than the shank diameter of the screw. (In these cases, the nearest bolt diameter to the diameter of the hole should be used.)

**Note:** Nailed connections that do not have pre-drilled holes have no material removed by the installation of the connectors. The capacity for members with these connections can be taken from the "No bolts" column of data.
[Table * .tens] contains the following data:

**Size and Area** - see section §1.3.1.1

### Axial stiffness - $EA$
In finding the serviceability performance of a tension member, the properties associated with the axial stiffness of the member are incorporated in $EA$. The general equation for deformation of tension members, equation (13), has this property in the denominator. The DCTT presents a range of different axial stiffness values for each cross section. They are all given as $10^6$ N as follows:

- **$[EA_{avg}]$** This property is used for members for which the structure has some ability to share loads among parallel members. It can also be used where violation of the deflection limit causes visual problems only.
- **$[EA_{5\%}]$** This property is used for members for which the structure does not have ability to share loads among parallel members. It is used where violation of the deflection limit may cause some damage or may affect drainage paths.

### Tensile capacity $[N_{Tt}]$
For each cross section, ten different values are given. These correspond to different combinations of bolt holes across the section. Interpolation of this table is not appropriate as it is not feasible to have non-integral numbers of holes in a single row.

In some cases, no figure is given in a particular column. This is because it is not possible to fit the number of bolts of that size across the section and comply with bolt spacings given in AS1720.1.

- **No bolts** - This column gives the tensile capacity of the member using the full cross sectional area. It is appropriate for members with nailed connections (installed without pre-drilling), or for parts of members in which there are no holes.
- **Bolt sizes** - Bolt diameters are given for standard metric bolts. For sawn timber members, the bolt sizes used are M10, M12 and M16 being the most commonly used sizes of bolts in smaller sized members. For the larger section sizes, larger bolt sizes have been used recognising that the larger members are capable of carrying much larger forces and will therefore need to have larger bolts - M12, M16 and M20 bolts are given in the tables.
- **Number of bolts** - this is the number of bolts in a single row of connectors (taken across the grain of the tension member.
- **Tensile capacity $[N_{Tt}]$**

This term represents $[k_1 f', A_t]$ and it can be used to find the axial load capacity of a tension member. In the table, if $d$ is too small to allow the use of the number of bolts per row with the required edge distances and spacings from <4.4.4.2>, then the position in the table is left blank. This data is given in kN.

To find the tensile capacity ($\phi N_t$), $[N_{Tt}]$ is multiplied by $\phi k_1 k_4 k_6$. 