3.1 Strength limit state

The strength limit state models the failure of structural elements. Timber in tension generally has quite brittle behaviour. Because of the level of safety contained in the load factors and the capacity factor, the probability of exceeding the strength limit state (and causing failure) is very low, as discussed in Section 2.3.3. The strength limit state design inequality for tension members is given as *equation 3.1*.

$N_{d,t} \geq N_t^*$	*equation 3.1* <equation 3.4(1)>

$N_{d,t}$	=	Design tensile capacity parallel to grain	<3.4.1>
N_t^*	=	Design axial load for strength limit state	<AS/NZS 1170.0 4.2.2>

The design tensile capacity $N_{d,t}$ of a member represents the tensile failure load of the weakest pieces of timber in the population of all timber in that grade. At tensile failure, the member will completely separate into at least two parts that compromises any load path through the tension member. The capacity factor ensures that the design tensile capacity $N_{d,t}$ gives the appropriate level of reliability to the structure. The design tensile capacity is compared directly with the strength limit state tensile forces from the design loads.

The design tensile capacity of timber is influenced by a large number of characteristics of the timber and the ambient conditions that the timber is under at the time of the strength limit state loading. These are accounted for in the use of a number of modification factors, the k factors. Even the nature of the strength limit state loading itself can influence the failure load of the timber.

The design tensile capacity of the member $N_{d,t}$ is given by:

$N_{d,t} = \phi \, k_1 \, k_4 \, k_6 \, f'_t \, A_t$	*equation 3.2* <equation 3.4(2)>

$N_{d,t}$	=	Design tensile capacity parallel to grain	<3.4.1>
ϕ	=	Capacity factor	<Table 2.1> (Section 2.3.4)
k_1	=	Duration of load factor allowing for the duration of strength limit state load	<2.4.1.1> (Section 3.1.2)
k_4	=	Partial seasoning factor for correcting the strength for the moisture content in the timber at the strength limit state load	<2.4.2> (Section 3.1.3)
k_6	=	Temperature factor that reduces the strength of timber used in high temperature conditions	<2.4.3> (Section 3.1.4)
f'_t	=	Design characteristic tensile strength parallel to grain for the grade and size of the timber selected	<Table H2.1> and others (Section 3.1.1)
A_t	=	Minimum net cross-sectional area of the member	<3.4.1>

The cross-section used in the calculation of the tensile capacity of a member is the minimum cross-sectional area using design dimensions: • For a member where holes have been drilled, this may be the net area of section after deduction of the cross-section for the holes (*equation 3.3*). • Nails driven directly into the timber do not reduce the tensile area of the cross-section. Where the nails are driven into pre-drilled holes, the tensile area will be less than the gross cross-sectional area. • For a tapered tension member, this may be the cross-sectional area at the small end of the taper.

Design dimensions are the smallest dimensions allowed within the tolerance of the product.

- for seasoned timber, the design dimensions are the same as the nominal dimensions;
- for unseasoned timber, the design dimensions are 3 mm less than the nominal dimensions;
- for glulam, the difference between the design dimensions and the nominal dimensions varies with size and is presented in Appendix A; and
- for LVL, the design dimensions are the same as the nominal dimensions.

The design dimensions of common sizes of sawn timber, glulam, and LVL are given in Appendix A.

$A_t = A - n_r D b$	(for holes diameter D, through narrow dimension)	*equation 3.3*

A_t	=	Net cross-sectional area of tension member	<3.4.1>
A	=	Gross cross-sectional area of the section ($b \times d$)	
n_r	=	Number of holes across the cross-section (holes per row)	
D	=	Hole diameter	
b	=	Narrow cross-sectional dimension	

Members with notches incorporate stress concentrations and an eccentricity that induces a bending moment into the tension member. They must be designed according to the provisions of <E11>.

The capacity factor ϕ can be found in <Table 2.1>, and has been outlined in Section 2.3.4. For timber, the ϕ factor is a function of the following:

- the type of structural element (primary or secondary);
- the role of the structure (housing, other or post-disaster function); and
- the structural timber material (stress grade).

Note:

Where a designer is in doubt about the structural timber material or the role of the structural element, a conservative approach should be adopted. A conservative approach will lead to the selection of a lower ϕ value.

3.1.1 Characteristic tensile strength (f'_t)

Characteristic tensile strength is an estimate of the 5th percentile strength of the entire population of timber members of a grade for each size. It can be found in the following places depending on the grade allocated to the selected product:

- For most products, tension strength is size specific.
- Where sawn timber has been awarded an F-grade, the characteristic tensile strength can be found in <Table H2.1>. Tension strength of larger sizes is reduced by *equation 3.4*. Where the manufacturer supports the F-grade product with design characteristic values based on in-grade tension strength data, the manufacturer's design characteristic value may be used instead of the characteristic strength from <Table H2.1>.
- For MGP grades and A17 stress grades design characteristic strength values can be found in <Table H3.1>. The strength of each depth is listed separately.
- For plywood, the characteristic tensile strength for F-grades is given in <Table 5.1>. This strength is independent of size. (AS 1720.1 can only be used with plywood that complies with AS/NZS 2269.)
- For glulam products, the characteristic tensile strength for GL grades is given in <Table 7.1>. Tension strength of larger sizes is reduced by *equation 3.4*. These properties only apply to products that comply in every way with AS/NZS 1328.
- For LVL members, the tensile strength must be taken from manufacturer's published literature. Tension strength of larger sizes is reduced by *equation 3.4*. (AS 1720.1 can only be used with LVL that complies with AS/NZS 4357.)

$$f'_t = \left(\frac{150}{d}\right)^{0.167} \times \text{nominal } f'_t \qquad \textit{equation 3.4}$$

The grading process involves estimation of the strength of the entire population of timber not individual pieces. There is a finite (but very small) probability that any one piece of timber will not achieve the characteristic stress documented. The ϕ factor and the load factors ensure that in spite of this uncertainty, the overall level of safety in the complete structure is not compromised by pieces which have a strength less than that of the 5th percentile.

All timber products except plywood have lower tensile strength for larger specimens in order to model size effects shown by tension test data. Brittle fracture mechanics has led to mathematical models for finding the probability of brittle failures in materials where flaws are randomly distributed through the member. Timber in tension shows brittle behaviour, and has naturally occurring characteristics that reduce its strength. These include knots, gum veins, shakes and slope of grain. Brittle fracture models point to a higher probability of finding a flaw in a critical location with larger pieces. The larger the volume of material, then the larger is the chance of having a flaw in a critical location.

Size effects for MGP and A17 grades have been determined by product specific test programs. So for those products tension strength is presented for each size. However, for other products, *equation 3.4*, which was originally derived from US tension data but checked on Australian products, can be applied. In this equation, the size is represented by *d*, the larger cross sectional dimension. Longer members tend to be deeper, so members with larger *d* have a larger volume of wood in tension.

✍ **Example 3.1 Design characteristic tensile strength f'_t**

Determine the design characteristic tensile strength for the following members:

(a) 240 × 45 mm MGP12

(b) 250 × 50 mm unseasoned F14 hardwood

(c) 330 × 65 mm GL12

(d) 90 × 35 mm MGP10

✍ **Solution**

(a) f'_t of 240 × 45 mm MGP12 member = 11 MPa <Table H3.1>

(a small reduction from the 12 MPa for the smaller section sizes)

(b) f'_t of 250 × 50 mm unseasoned F14 hardwood member

$$= 22 \times \left(\frac{150}{250}\right)^{0.167} = 20.2 \quad \text{MPa}$$ <Table H2.1>

(c) f'_t of 330 × 65 mm GL12 member

$$= 11 \times \left(\frac{150}{330}\right)^{0.167} = 9.6 \text{ MPa}$$ <Table 7.1>

(d) f'_t of 90 × 35 mm MGP10 member = 7.7 MPa <Table H3.1>

(no reduction as it is one of the smaller section sizes)

3.1.2 Duration of load and k_1 factor

Timber has time-dependent properties. Under long-term stresses, the fibres in the wood stretch and move relative to one another. These movements are in addition to the short-term elastic response to load, and they result in creep under long-term loads. The creep effect increases the long-term deflections and is represented by the j_3 factor which for tension members is discussed in Section 3.2.1.

The movement of the fibres can cause damage on a microscopic scale. Over a very long period of loading, this can lead to a loss of strength. Standard testing of timber typically takes less than five minutes. This loading period is taken as the base strength in the limit states version of AS 1720.1. Longer periods of sustained load will lead to lower strengths.

All timber design Standards include the duration of load effect and many of them are based on the Madison Curve (Wood, 1951) or slight modifications of it. In effect, the designer is allowing for the accumulation of microscopic damage caused by peak load events. Minor load events will not cause enough stress to lead to loss of capacity.

The duration of load effect (DoL) on strength for all timber elements is represented by the k_1 factor which is given in <Table 2.3>. Values for members are plotted in Figure 3.1. This duration of load model differs a little from the Madison curve, and is based on some early tests on small clear specimens of Australian species. It may be conservative. Other models for duration of load effect on strength have been given in the list of references at the back of this chapter. (Madsen 1992) (Pearson 1972) (Foschi, Folz, Yao 1989) (Neilsen 1992).

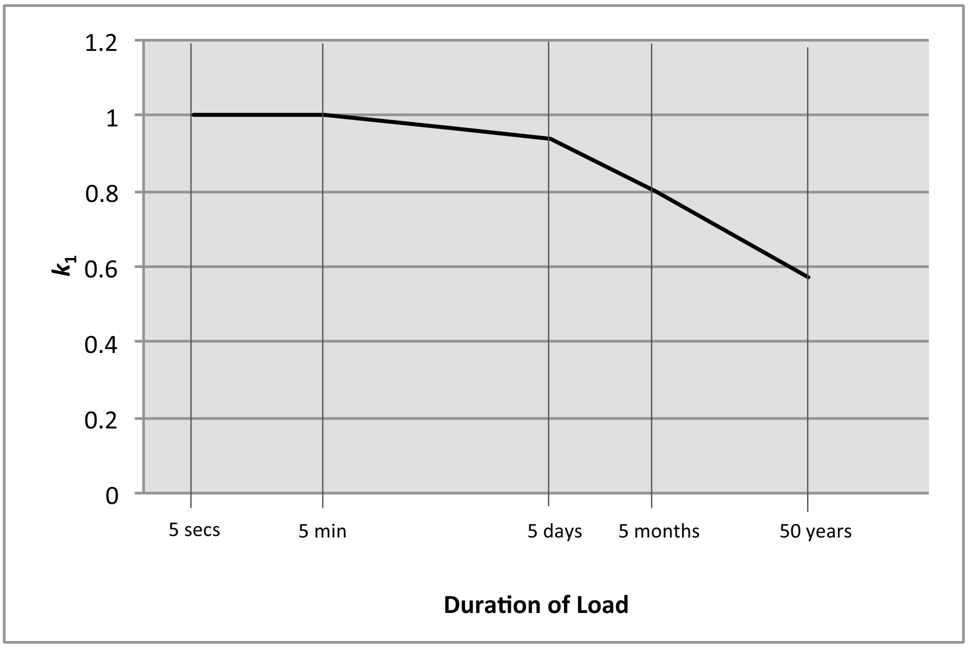


Figure 3.1 Duration of load effect for members

LVL, glulam, plywood, sawn and round timber all use the same duration of load factors. However, connections use different duration of load factors. See Section 7.2.3

Duration of load (strength limit state)

There are different definitions for duration of load for the strength limit state and the serviceability limit states.

Duration of load for the strength limit state is the total time over the life of the structure for which the element will be loaded at or above the designated load level.

For strength, it is the accumulated duration of the specified loading over the life of the structure that is estimated.

AS 1720.1 uses a set of times that are used as the effective duration of peak action i.e. 50 or more years, 5 months, 5 days, 5 hours, 5 minutes or 5 seconds. The classifications all indicate an order of magnitude rather than a precise measure of time.

Figure 3.2 depicts the load history of a structural element typical of permanent and imposed actions. It shows that within the life of the structural element there are a number of significant load events. A designer has to anticipate these (and unless an experienced clairvoyant, the designer will have trouble). The Standard <Appendix G> provides some help for the anticipation of the duration of load for some common load cases. Designers must estimate the duration of the load and then use Figure 3.1 or <Table 2.3> to obtain the appropriate k_1 factor.

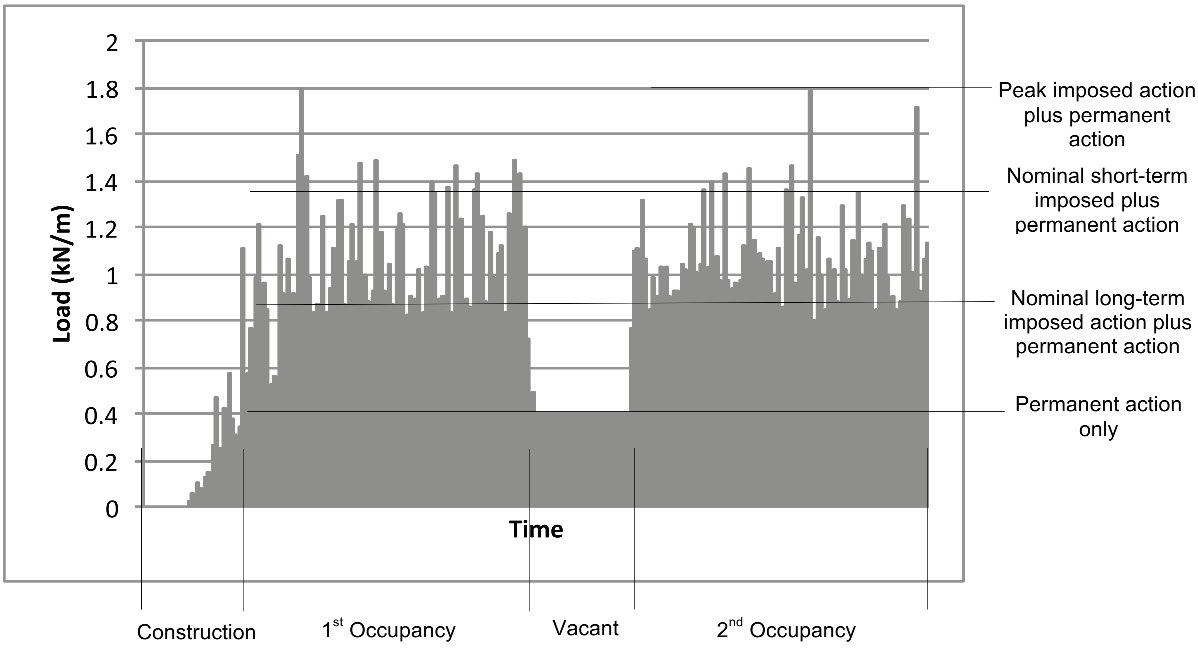


Figure 3.2 Anticipated history of loading on a structural element

In Figure 3.2, the peak load only occurred once in the life of the structure, and there were few load events that were close to it. Figure 3.2 shows a combination of permanent and imposed actions. During construction, the permanent actions increase steadily as the construction proceeds, with occasional peaks due to the construction imposed actions. Once construction is complete, the permanent action is constant for the remainder of the building life, but the imposed actions vary with the use of the building. The nominal short-term infrequent imposed action occurred a few times in the life of the structure and consists of rare imposed loading events superposed on the longer duration imposed action. The aggregate effect of a load at or above the nominal short-term load level in the life of the structure can be regarded as a load of a few days duration. Imposed actions such as furniture, partitions, and storage, which will remain in place for long periods of time, are considered as the long-term imposed loading and these loads also contribute to the short-term imposed loading. Guidance on duration of loading in <2.4.1.1> is summarised in Table 3.1.

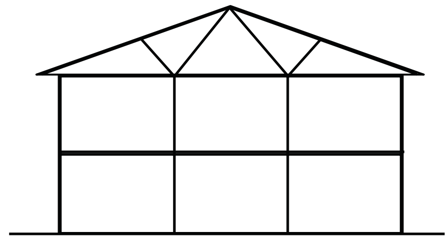
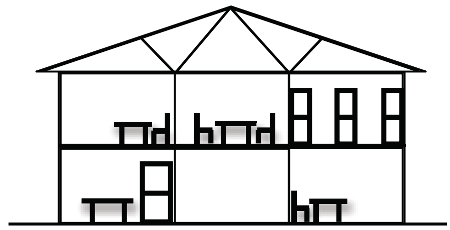
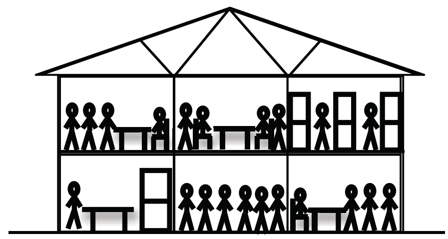
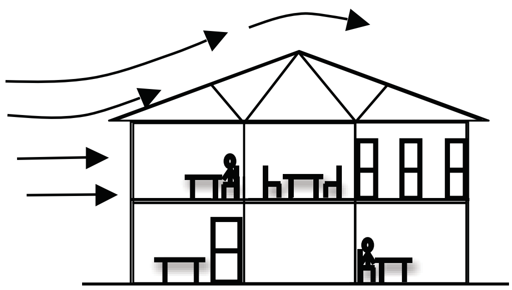
Table 3.1 illustrates different combinations of loads that must be considered as part of the ultimate limit state design. Each of them relates to different configurations of imposed actions on the same building.

Imposed actions in combinations

AS/NZS 1170.1 gives imposed actions for floors and roofs separately.

- Nominal floor imposed actions (Q) represent the total of furniture, fittings, and people with a return period of around 20 years. The loading due to furniture and fittings alone can be taken as $\psi_l\, Q$.
- Floor imposed actions in some industrial or commercial buildings may need to take into account the weight of installed machinery in addition to the nominal floor imposed actions.
- Nominal roof imposed actions (Q) normally represent only construction activities. Where known long-duration imposed actions like heavy light fittings, hanging screens, or air-conditioners have to be considered, these loads must be added in separately. (In some cases, roofs also have floor functions, e.g. as a roof-top car park, and must be designed for floor loadings as well.)

Table 3.1 Character of loading in strength limit states combinations

Load case	**Permanent only**	
Strength limit state magnitude	$1.35G$	
Strength limit state duration	50+ years	
Imposed action	Q not included – only structure self weight	
Load case	**Permanent + long-term imposed**	
Strength limit state magnitude	$1.2G + 1.5\psi_l Q$	
Strength limit state duration	50+ years	
Imposed action	$\psi_l Q$ is the weight of furniture and fittings that remain for long periods	
Load case	**Permanent + short-term imposed**	
Strength limit state magnitude	$1.2G + 1.5Q$	
Strength limit state duration	5 days	
Imposed action	Q is the nominal imposed load, which includes furniture, fittings, people	
Load case	**Permanent + imposed + wind**	
Strength limit state magnitude	$1.2G + \psi_c Q + w_u$	
Strength limit state duration	5 seconds	
Imposed action	$\psi_c Q$ is the nominal point in time imposed action and includes all furniture, fittings, and maybe a few people	

In any given load combination, it is the sum of the durations of the shortest component that gives the duration of load for the whole combination. This can be seen for the nominal short-term load in Figure 3.2 where the duration of load is the sum of the durations of the total loads that extend above that line. Table 3.2 presents appropriate cumulative durations for common types of load.

The strength limit state duration of load factor k_1 can be found as follows for each load combination:

* For each load in the combination, the duration of load can be estimated, or the load can be classified as 50+ years, 5 months, 5 days, 5 hours, 5 minutes, or 5 seconds.
* Only the shortest of all of the loads in the combination is considered to find k_1.
* The k_1 factor is applied to the whole combination.

Table 3.2 Estimated duration of load for different types of loading

Load type		**Assumed duration**	k_1
Permanent actions	(structure loads)	50+ years	0.57
Long duration imposed actions	(furniture and partitions)	50+ years	0.57
Frequent imposed actions	(vehicles or people)	5 months	0.80
Regular snow loads	(alpine regions)	5 months	0.80
Infrequent imposed actions	(crowds, construction)	5 days	0.94
Rare snow loads	(sub-alpine regions)	5 days	0.94
Ultimate wind gust loads	(from V_u in AS/NZS 1170.2)	5 seconds	1.00
Earthquake loads	(from AS 1170.4)	5 seconds	1.00

Example 3.2 Duration of load factor k_1

(a) Determine the strength duration of load factor (k_1) for a roof member with a load combination of $1.2\,G + 1.5\,Q$ (Q is a construction load)

(b) Determine the strength duration of load factors (k_1) for an office floor member with load combinations:

$1.2G + 1.5Q$ and $1.2G + 1.5\,\psi_l\,Q$

Solution

(a) The permanent action G has a duration of 50+ years.

The construction imposed action can be considered as load with cumulative duration 5 days (Table 3.1)

Of the two loads in the combination, the construction load is the shorter (5 days), and k_1 for 5 day loads is 0.94

$k_1 = 0.94$ will be used for the whole combination <Table 2.3>

(b) The permanent action G has a duration of 50+ years.

$1.2G + 1.5Q$

The peak floor imposed action in offices is normally due to crowds, and can be considered as load with cumulative duration 5 days (Table 3.2)

Of the two loads in the combination, the imposed load is the shorter (5 days), and k_1 for 5 day loads is 0.94

$k_1 = 0.94$ will be used for the whole combination $1.2G + 1.5Q$ <Table 2.3>

$1.2G + 1.5\,\psi_l\,Q$

The longer term floor imposed action in offices is only due to furniture, and can be considered as a load with cumulative duration 50+ years (Table 3.2).

The two loads in the combination have the same duration (50+ years) and k_1 for 50+ year loads is 0.57

k_1 = 0.57 will be used for the whole combination $1.2G + 1.5\ \psi_l\ Q$ <Table 2.3>

Normally the selection of the strength limit state duration of load factor can be performed by inspection. The above example could be shown in calculations as:

(a) $1.2\ G + 1.5\ Q$ (construction) – 5 days ⇒ $k_1 = 0.94$ <Table 2.3>

(b) $1.2\ G + 1.5\ Q$ (crowds) – 5 days ⇒ $k_1 = 0.94$ <Table 2.3>

$1.2G + 1.5\ \psi_l\ Q$ – 50+ years ⇒ $k_1 = 0.57$ <Table 2.3>

Identification of the critical load combination

The calculation of the strength limit state capacity is a function of the duration of load used in the strength limit state loads. In *equation 3.1* the particular load combination used to find N^* will have a duration of load associated with it, and this will dictate the use of k_1 in the $N_{d,t}$ term on the left side of the same equation. The link between the left and right sides of the equation means that this equation must be evaluated for each and every load case. (Unlike other materials, the largest load is not necessarily the critical load case because for timber the duration of load must be considered.) A method of calculation is provided in Section 3.3.1 to enable a designer to select the critical load case without having to design for each load combination in its entirety.

3.1.3 Partial seasoning and k_4 factor

Timber responds to atmospheric conditions by taking in moisture if the air is damp or by drying out if the air is dry. In the production of timber, this is used to season the timber (dry it to a moisture content of less than 15%). Refer to Section 1.3.4.

Timber is normally specified, sold and supplied as either seasoned or unseasoned. Refer to Section 1.3.3.

- Seasoned timber has a moisture content less than or equal to 15%.
- Unseasoned timer has a moisture content greater than or equal to 25%.
- Between these two categories is a third; partially seasoned, timber. Timber with a moisture content between 15% and 25% is also sold and classified at sale as unseasoned, although it is, strictly speaking, partially seasoned. This is conservative. Partial seasoning of timber may also occur in service.

Partial seasoning in service

In service, a piece of timber will respond to changes in air moisture:

- If the timber is seasoned prior to installation, but is used in an environment in which the relative humidity of the air is very high for long periods, it will take up moisture to become partially seasoned.
- Timber sold as unseasoned timber (with initial moisture content above 15%, but mostly above 25%), placed in a low humidity environment will lose moisture and hence experience partial seasoning.

Equilibrium moisture content

Equilibrium moisture content (*EMC*) is the moisture content of the timber when there is no moisture movement to or from the air. The moisture in the timber is at equilibrium with the moisture in the air. It is a function of the temperature and relative humidity of the air that surrounds the timber. The relationship between the relative humidity and temperature of the air and equilibrium moisture content of the timber is shown in Figure 3.3.

Moisture tends to move between timber and the air that surrounds it until the moisture in the timber is in equilibrium with the moisture in the air (the timber is at *EMC*). The movement of moisture within wood is relatively slow, even if the wood is unpainted, but it can be very slow where the timber surface is coated. It is, therefore, the 12-month average conditions that are important in the determination of the extent of partial seasoning that will occur in the timber.

Where seasoned timber is used in a protected environment, the timber will remain seasoned. Where unseasoned timber is used indoors, it may become partially seasoned over a period of time (roughly one year for every 50 mm of thickness when exposed to good air movement, but much slower if the air movement is restricted).

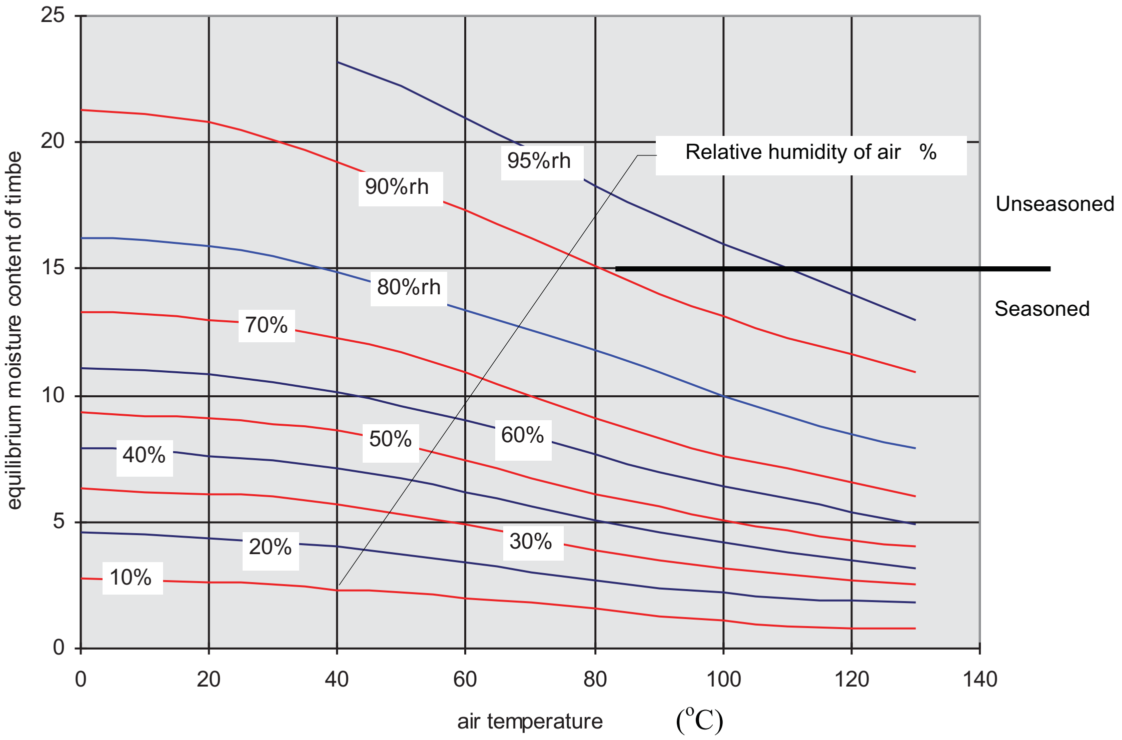


Figure 3.3 Equilibrium moisture content of timber in service (CSIRO 1983)

Work on clear wood early in the 20th century (Wilson, 1932) showed a relationship between moisture content and strength and stiffness. It showed that there was a general decrease in most of the major structural properties with an increase in moisture content. Moisture within the cell walls lubricates the fibre to fibre interface and allows more slippage. This decreases the strength of the wood fibre. Because original work on the effect of moisture content on wood strength was performed on small clear specimens, it may have limited relevance to commercial sized timber. Nonetheless, it forms the basis of most of the structural timber standards world-wide. The relationships between strength and moisture content need to be updated for limit states characteristic strengths based on the lower 5th percentile of in-grade data (Boughton & DeLeo, 1997). Results of some earlier work show that the