UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION

Vienna International Centre, P.O. Box 300, 1400 Vienna, Austria

Tel: (+43-1) 26026-0 · www.unido.org · unido@unido.org

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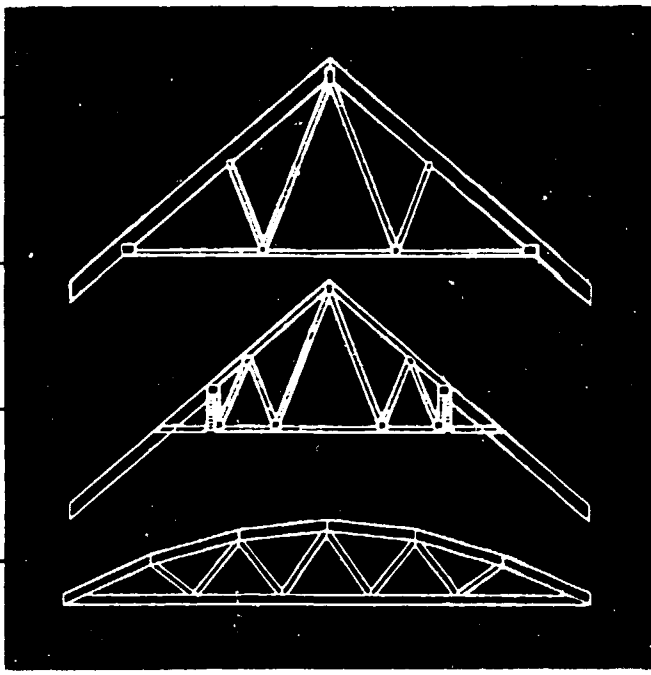
Timber Construction for Developing Countries



UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION

General Studies Series

TIMBER CONSTRUCTION FOR DEVELOPING COUNTRIES

Structural Timber and Related Products

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
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Explanatory notes

The following technical abbreviations are used:

FF	form factor
GR	grade ratio
MDF	medium-density fibreboard
MDR	modulus of rupture
MSR	machine stress-rated (lumber)
PRF	phenol resorcinol formaldehyde adhesive
OSB	oriented strandboard
VPS	vacuum-pressure soak (treatment)

Abbreviations of organizations:

ASTM	American Society for Testing Materials
CSIRO	Commonwealth Scientific and Industrial Research Organization
FAO	Food and Agriculture Organization of the United Nations
IUFRO	International Union of Forestry Research Organizations
PAA	Plywood Association of Australia
SAA	Standards Association of Australia

PREFACE

Whether grown in a particular country or not, wood is a virtually universal material that is familiar to people all over the world. It is used for many purposes but principally for construction, furniture, packaging and other specialized uses such as transmission poles, railway ties, matches and household articles. The United Nations Industrial Development Organization (UNIDO), which was established in 1967 to assist developing countries in their efforts to industrialize, has the responsibility within the United Nations system for assisting in the development of secondary woodworking industries and has carried out this responsibility since its inception at the national, regional and interregional levels by means of projects both large and small. UNIDO also assists by preparing manuals on topics that are common to the woodworking sectors of most countries.*

The lectures presented at the Timber Engineering Workshop (TEW), held from 2 to 20 May 1983 at Melbourne, Australia, are part of the continuing efforts of UNIDO to help engineers and specifiers appreciate the role that wood can play as a structural material. Collected in the form of 38 chapters, these lectures have been entitled Timber Construction for Developing Countries, which forms part of the General Studies Series. Ten of the chapters make up the second volume of the collection, Structural Timber and Products. The TEW was organized by UNIDO with the cooperation of the Commonwealth Scientific and Industrial Research Organization (CSIRO) and was funded by a contribution made under the Australian Government's vote of aid to the United Nations Industrial Development Fund. Administrative support was provided by the Department of Industry and Commerce of the Australian Government. The remaining lectures (chapters), which cover a wide range of subjects, including case studies, are contained in four additional volumes, as shown in the table of contents.

Following the pattern established for other specialized technical training courses in this sector, notably the course on furniture and joinery and that on criteria for the selection of woodworking machinery,** the lectures were complemented by visits to sites and factories, discussion sessions and work assignments carried out by small groups of participants.

It is hoped that the publication of these lectures will lead to the greater use of timber as a structural material to help satisfy the tremendous need in the developing countries for domestic, agricultural, industrial and commercial buildings and for structures such as bridges. It is also hoped that the lectures will be of use to teachers in training institutes as well as to engineers and architects in public and private practice.

Readers should note that the examples cited often reflect Australian conditions and thus may not be wholly applicable to developing countries,

*These activities are described more fully in the booklet UNIDO for Industrialization: Wood Processing and Wood Products (PI/78).

**The lectures for these two courses were collected and published as Furniture and Joinery Industries for Developing Countries (United Nations publication, Sales No. E.88.III.E.7) and Technical Criteria for the Selection of Woodworking Machines (UNIDO publication, Sales No. 92.1.E).

despite the widespread use of the Australian timber stress grading and strength grouping systems and despite the wide range of conditions encountered on the Australian subcontinent. Moreover, it must be remembered that some of the technology that is mentioned as having been new at the time of the Workshop (1983) may since then have been further developed. Similarly, standards and grading systems that were just being developed or introduced at that time have now become accepted. Readers should also note that the lectures were usually complemented by slides and other visual aids and by informal comments by the lecturer, which gave added depth of coverage.

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*For the reader's convenience the contents of the four complementary volumes are also given here.

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INTRODUCTION

Many developing countries are fortunate in having good resources of timber, but virtually all countries make considerable use of wood and wood products, whether home-grown or imported, for housing and other buildings, in both structural and non-structural applications, as well as for furniture and cabinet work and specialized uses. Although wood is a familiar material, it is all too often misunderstood or not fully appreciated since it exists in a great variety of types and qualities.

Some species, such as teak, oak and pine, are well known almost everywhere while others, such as beech, eucalyptus, acacia, mahogany and rosewood, are known primarily in particular regions. Still others, notably the merantis, lauans and keruing, which come from South-East Asia, have only recently been introduced to widespread use. Very many more species exist and are known locally and usually used to good purpose by those in the business. Also, plantations are now providing an increasing volume of wood.

The use of timber for construction is not new and, in fact, has a very long tradition. In many countries this tradition has unfortunately given way to the use of other materials - notably, concrete, steel and brick - whose large industries have successfully supported the development of design information and the teaching of methods for engineering them. This has not been so much the case for timber, despite considerable efforts by some research and development institutions in countries where timber and timber-framed construction have maintained a strong position. Usually the building methods are based on only a few well-known coniferous (softwood) species and a limited number of standard sizes and grades. For these, ample design aids exist, and relatively few problems are encountered by the very many builders involved.

Recent developments in computer-aided design and in factory-made components and fully prefabricated houses have led to better quality control and a decreased risk of site problems. Other modern timber engineering developments have enabled timber to be used with increasing confidence for an ever wider range of structures. This has been especially so in North America, Western Europe, Australia and New Zealand.

UNIDO feels that an important means of transferring this technology is the organization of specialized training courses that introduce engineers, architects and specifiers to the subject and draw their attention to the advantages of wood, as well as its disadvantages and potential problem areas, and also to reference sources. In this way, for particular projects or structures, wood will be fairly considered in competition with other materials and used when appropriate. Comparative costs, aesthetic considerations and tradition must naturally be taken into account in the context of each country and project, but it is hoped that the publication of these lectures will lead those involved to a rational approach to the use of wood in construction and remove some of the misunderstandings and misapprehensions all too often associated with this ancient yet modern material.

I. CHARACTERISTICS OF STRUCTURAL TIMBER

Robert H. Leicester*

Introduction

In design procedures, timber is treated in a manner similar to steel. However, there are at least three basic ways in which timber differs from steel with respect to its physical properties:

(a) The clear wood is essentially orthotropic;

(b) Structural timber contains natural defects that have very complex structural properties;

(c) The properties of timber vary in a random manner from stick to stick and from one location to another within a stick.

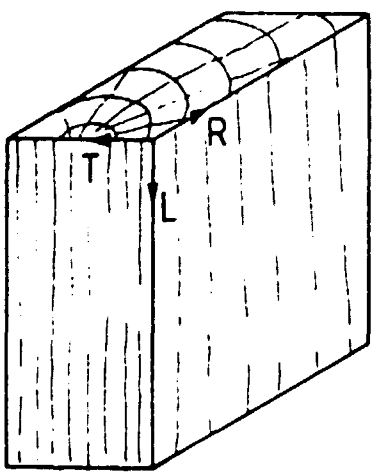
Other differences, such as the creep characteristics of timber, are discussed in other sections.

Because of the above, there are significant differences between the structural characteristics of small, clear pieces of wood; clear, structural-size timber; structural-size timber containing natural defects; and pole timbers. The differences between these forms of timber will be emphasized in the following discussion. Unless otherwise stated, all timber refers to sawn sticks of structural size.

A. Orthotropic elasticity

In both small and large sizes of clear wood, the principal axes lie along the longitudinal, radial and tangential directions, as shown in figure 1, where these directions are denoted by L, R and T. Timber is considerably stiffer, and stronger, in the longitudinal direction than along the other two principal axes.

Figure 1. Principal axes in timber



*An officer of CSIRO, Division of Building Research, Melbourne.

Typical values of the elastic moduli along the principal axes may be stated in terms of E_L, the stiffness along the longitudinal direction:

$$E_R = 0.10\ E_L$$

$$E_T = 0.05\ E_L$$

Similarly, typical shear moduli, G, may be stated as follows:

$$G_{LT} = 0.060\ E_L$$

$$G_{LR} = 0.075\ E_L$$

$$G_{RT} = 0.018\ E_L$$

and typical Poisson's ratios are as follows:

$$\mu_{LR} = 0.40$$

$$\mu_{RL} = 0.04$$

$$\mu_{LT} = 0.40$$

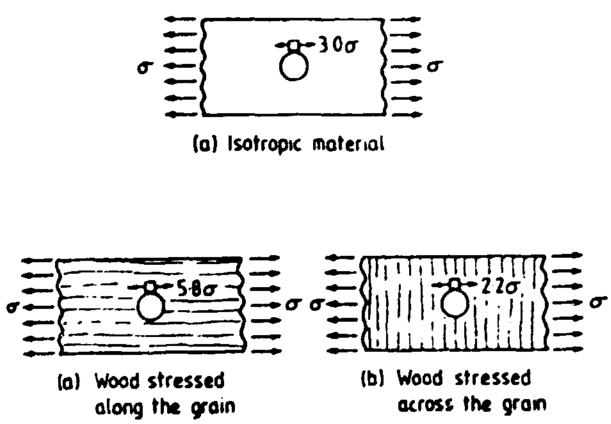
$$\mu_{TL} = 0.10$$

$$\mu_{RT} = 0.50$$

$$\mu_{TR} = 0.25$$

It is outside the scope of this paper to analyse the various effects of orthotropicity. As an example, however, figure 2 shows the effects of orthotropicity on stress concentrations at the edge of a circular hole.

Figure 2. Effect of orthotropicity on stress concentrations



B. Shear deformations

In both small and structural-size timber, the low shear rigidity of timber means that wood structures exhibit a greater proportion of deformation due to shear than structures fabricated with isotropic materials such as steel. For example, the components of deflection at the centre of a simply supported beam, such as that shown in figure 3, are as follows:

Deflection due to bending

$$\Delta_B = PL^3/(48EI) \tag{1}$$

Deflection due to shear for solid members

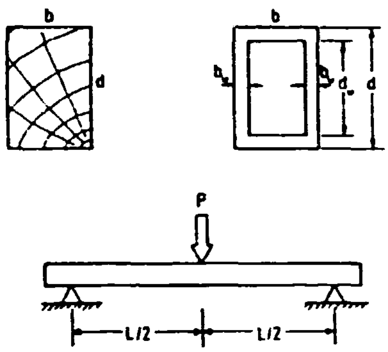
$$\Delta_S = 1.5\ PL/(4Gbd) \tag{2}$$

Deflection due to shear for a box beam

$$\Delta_S = PL/(4Gd_w\Sigma b_w) \tag{3}$$

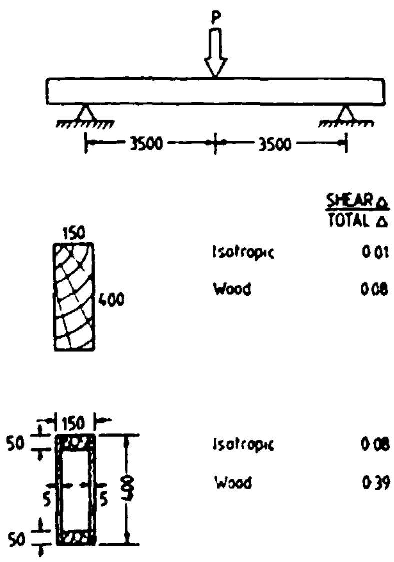
The notation used here is indicated in the figure.

Figure 3. Notation used in deflection computation



For the typical examples of the solid timber and the box beam shown in figure 4, the computed deflections due to shear are 8 per cent and 39 per cent, respectively, of the total deflection. In the computation, the shear modulus, G, has been taken to be 0.06 E_L.

Figure 4. Examples of shear deflections



Thus, in contrast to the case for orthotropic materials, the shear deformation of beams is significant. It should be noted that in most design codes, the design value of the specified modulus of elasticity for structural timber includes an allowance for shear deformations.

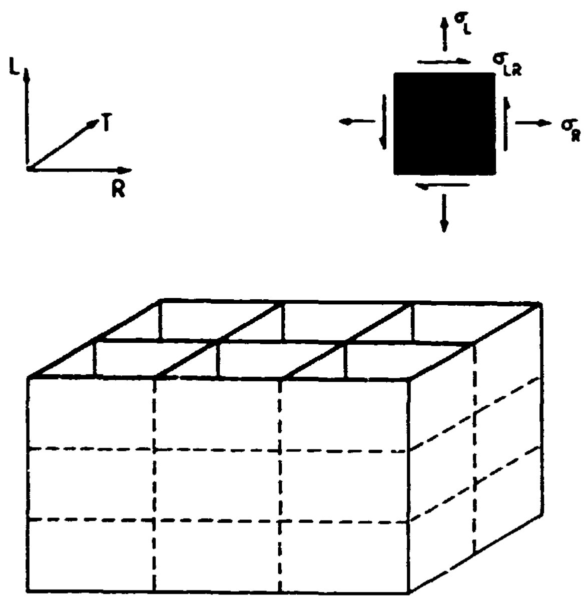
C. Combined stresses for clear wood

Some ideas on a theory of wood strength can be derived by considering an idealized cellular structure aligned with the principal axes, as shown in figure 5. If it is assumed that the individual plates in this structure obey the von Mises failure criterion, then it can be shown that the failure criterion for the wood structure is given by three equations of the following type [1]:

$$(\sigma_L/F_L)^2 + (\sigma_R/F_R)^2 - (\sigma_L/F_L)(\sigma_R/F_R) + (\sigma_{LR}/F_{LR})^2 \leq 1 \qquad (4)$$

where σ_L, σ_R and σ_{LR} denote the applied stresses relative to the R and L axes and F_L, F_R and F_{LR} are the corresponding values of these stresses that would cause failure if each of these stresses were acting on their own.

Figure 5. Idealized cellular structure



An approximation to equation (4) that has been found to fit the limited experimental data equally well is the following:

$$(\sigma_L/F_L)^2 + (\sigma_R/F_R)^2 + (\sigma_{LR}/F_{LR})^2 \leq 1 \qquad (5)$$

Some typical relative values of the ultimate strength parameters are

$F_L = 3.0\ F_c$ in tension $= 1.0\ F_c$ in compression

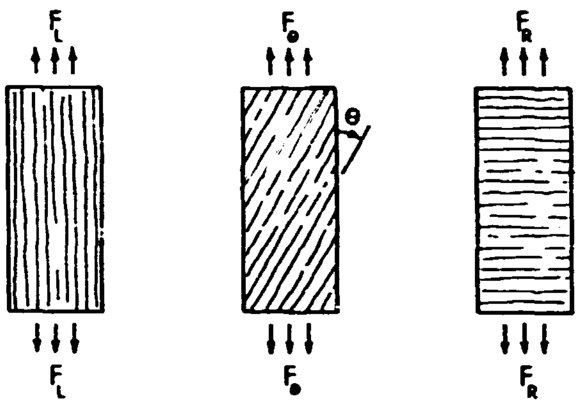
$F_R = 0.07\ F^c$ in tension $= 0.10\ F_c$ in compression

$F_{RL} = 0.20\ F_c$

D. Hankinson's formula

Equation (4) may be used for deriving the strength of timber when the load is applied at an angle to the grain. However, it is considerably simpler, and usually sufficiently accurate for practical purposes, to use Hankinson's formula, illustrated in figure 6.

Figure 6. Illustration of Hankinson's formula



HANKINSON'S FORMULA

$$F_{\theta} = \frac{F_L F_R}{F_L \sin^2\theta + F_R \cos^2\theta}$$

According to Hankinson's formula, the strength of wood at an angle θ to the direction of the grain will be denoted by F_{θ} and is given by

$$F_{\theta} = F_L F_R/(F_L \sin^2\theta + F_R \cos^2\theta) \tag{6}$$

Hankinson's formula has been found useful as a general method of interpolation to obtain estimates of a structural property at an angle to the wood grain. For example, it is usually applied to structural connectors.

E. Simple models of bending strength

Bending strength involves a complex interaction between the tension and compression properties of wood. Some insight into the characteristics of bending strength may be obtained by considering the idealized stress-strain relationship shown in figure 7. For both clear wood and structural timber, the tension strength is taken to be brittle and the compression strength to be elasto-plastic. The tension strength is larger than the compression strength in the case of clear wood and smaller in the case of structural timber. With these assumptions, the stress distribution at failure shown in figure 8 is obtained.

Figure 7. Idealized characteristics of wood in the direction of the grain

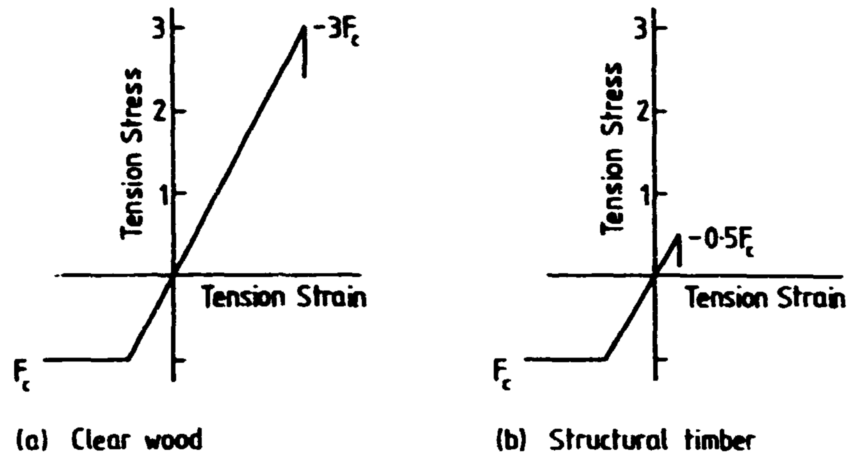
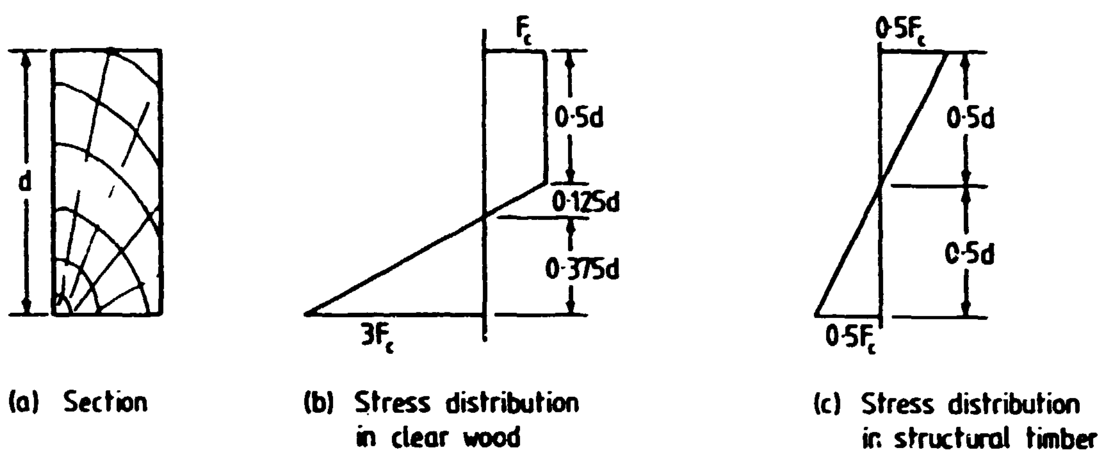


Figure 8. Stress-strain distribution at failure for rectangular members



Conventionally, the bending strength of timber beams is stated in terms of a modulus of rupture, MOR, defined by

$$MOR = M_{ult} y_{max}/I \quad (7)$$

where M_{ult} is the applied bending moment at failure, I is the moment of inertia of the cross-section and y_{max} is the maximum distance from the neutral axis to the edge.

With this definition, the following MOR values are derived:

Section	MOR for clear wood	MOR for structural timber
Square	2.00 F_c	1.00 F_t
Round	2.15 F_c	1.00 F_t
I-beam	1.42 F_c	1.00 F_t

The webs and flanges of the I-beam are taken to be 0.1 of the beam width and depths, respectively.

A form factor, FF, is defined as follows:

$$FF = MOR/MOR_{square} \tag{8}$$

where MOR is the modulus of rupture of the cross-section in question and MOR_{square} is the value obtained for a square cross-section.

The application of equations (6) and (7) leads to the following values for the form factor:

Section	FF for clear wood	FF for structural timber
Square	1.00	1.00
Round	1.07	1.00
I-beam	0.71	1.00

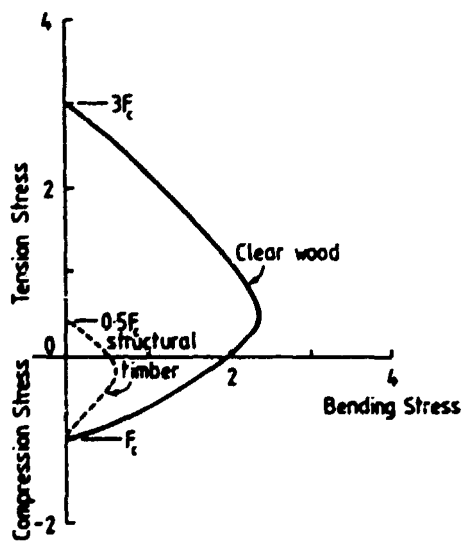
F. Combined bending and axial load

A reasonable picture of the strength of timber members subjected to combined bending and axial load can be obtained by use of the idealized stress-strain curves shown in figure 7. The resulting interaction curves are shown in figure 9. The true relationship is difficult to measure experimentally. Useful data for the case of tension and bending loads on structural timber has been given by Senft and Suddarth [2] and Senft [3]. One practical aspect noted in these studies was that the axial tension force significantly modified the bending moment. A rough estimate of the effective bending moment, M_{eff}, is as follows:

$$M_{eff} = M_{nom} - (2/3)T\Delta_{nom} \tag{9}$$

where T is the applied axial tension force, M_{nom} is the nominal value of the applied bending moment and Δ_{nom} is the value of the deflection at the centre of the beam, computed assuming T = 0.

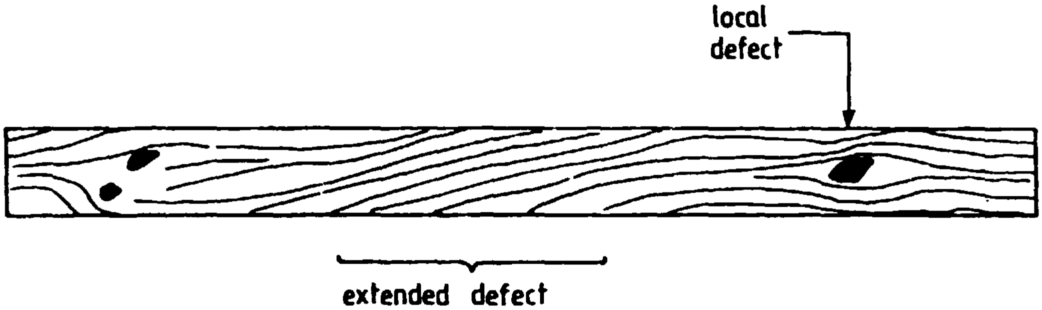
Figure 9. Interaction curves for combined bending and axial load



G. Effect of non-homogeneity

Unlike the properties of many conventional structural materials, such as steel, the properties of timber can vary considerably within a single element. For example, a stick of lumber contains both strong, clear wood and weak defects (figure 10). This type of marked non-homogeneity has a significant effect on the characteristics of nominal strength.

Figure 10. Dispersion of defects in a typical stick of timber



A simple structural model of the nominal strength R of a particular structural element can be written

$$R = R_1 R_2 \qquad (10)$$

$$\bar{R}_2 = 1.0 \qquad (11)$$

where R_1 is a constant for any given structural element but varies from one element to another, and R_2 is a parameter that varies from location to location within an element.

The conventional measure coefficient of variation of the structural element V_R is given by

$$V_R^2 = V_1^2 + V_2^2 \qquad (12)$$

where V_1 and V_2 are the coefficients of variation of R_1 and R_2, respectively. The simplest method of measuring V_2 is to cut pairs of test samples from each structural element and then to measure the correlation coefficient r of the strength for these pairs of test samples. The coefficient of variation V_2 is then given by

$$V_2^2 = V_R^2(1 - r) \qquad (13)$$

Details on the method of assessing the effects of non-homogeneity are given in annex I. Some simple. practical examples will be given here.

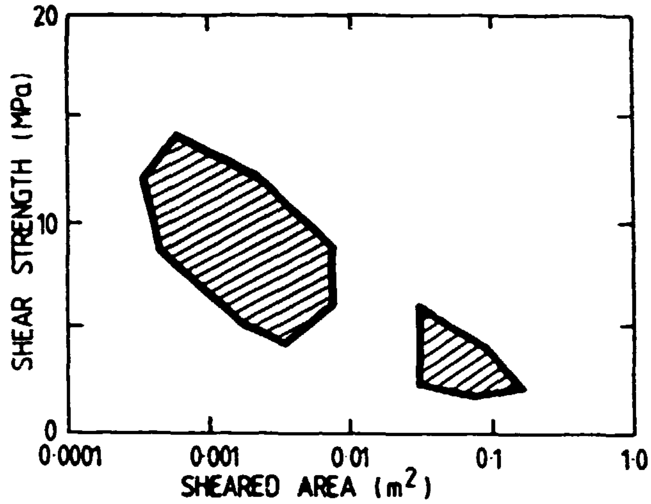
For the case of geometrically similar elements that are brittle, so that failure eventuates if any internal flaw fails, the strength is given by

$$\bar{R} = A_0^{-V_2} \qquad (14)$$

where A_0 is a material and configuration constant and ¢ denotes the volume of stressed material.

Equation (14) is a typical "weakest link" relationship of the type first studied by Weibull [4]. Similar relationships can be derived for the case where the failure is related to the weakest area or to the weakest cross-section of a beam. Figure 11 shows a size effect measured for the shear strength of timber elements. Other useful examples have been given by Foschi and Barrett [5] and Bohannon [6].

Figure 11. Effect of size on the shear strength of timber beams and glued joints



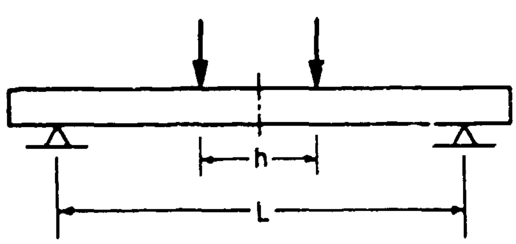
Source: F. J. Keenan, "Sheer strength of wood beams", Forest Products Journal, vol. 24, No. 9 (1974).

Another example of practical value is the symmetrically loaded beam shown in figure 12. The strength of this beam is given by

$$\bar{R} = A_{00}\{L[1 + (h/V_2L)]\}^{-V_2} \qquad (15)$$

where A_{00} is a material constant.

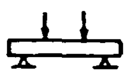
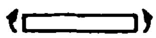
Figure 12. Notation for symmetrically loaded beam



$$\bar{R} = \frac{A_{00}}{L[1+\frac{h}{V_2 L}]^{V_2}}$$

For clear wood and structural timber, typical values of V_2 are 0.1 and 0.25, respectively. Hence, according to equation (15), doubling the span leads to the corresponding factors of 0.93 and 0.84 on strength. The use of equation (15) to assess the effect of loading configuration is illustrated in figure 13. The fact that the effects are different in structural timber than in clear wood is to be noted.

Figure 13. Effect of load configuration on strength

LOADING CONFIGURATION	RELATIVE STRENGTH	
	Clear Wood (V_z=0·10)	Structural Timber (V_z=0·25)
	1·00	1·00
	0·86	0·81
	0·79	0·67

The above theory has been stated in terms of mean nominal strength, but it also gives the same answers when applied to characteristic values such as five-percentile strength values.

Finally, mention should be made of a configuration factor that is peculiar to beams. Beam strength will vary depending on whether the edge placed in tension is randomly chosen or is deliberately selected to be the weakest edge. If the two values of nominal strength are denoted by R_{rand} and R_{weak}, respectively, then for the weaker pieces of timber where defects are visually discernible, the relationship between these strengths is given roughly by

$$Pr(R_{weak} < x) \cong 1.5\ Pr(R_{rand} < x) \tag{16}$$

If it is assumed that both R_{weak} and R_{rand} have the same types of distributions, then equation (16) leads to the following ratios of the five-percentile values of R^0_{weak} and R^0_{rand}:

Coefficient of variation, V_R	R^0_{weak}/R^0_{rand}
0.1	0.97
0.2	0.93
0.3	0.89

In the context of the above discussion it is of interest to note the methods that have been used in various countries to measure bending strength:

(a) United States of America/Canada: random location of defects, random edge placed in tension;

(b) United Kingdom: weakest defect at maximum stress section, random edge placed in tension;

(c) Australia: weakest defect at maximum stress section, weakest edge placed in tension.

H. Effect of natural defects

Natural defects introduce zones of weakness and sometimes zones of flexibility into structural timber. Some of the effects of knots and sloping grain are illustrated in figures 14 and 15. Other defects that have been studied include splits, checks and kino veins, local pockets of decay, compression shakes and bark inclusions.

Figure 14. Effect of slope of grain on the structural characteristics of timber

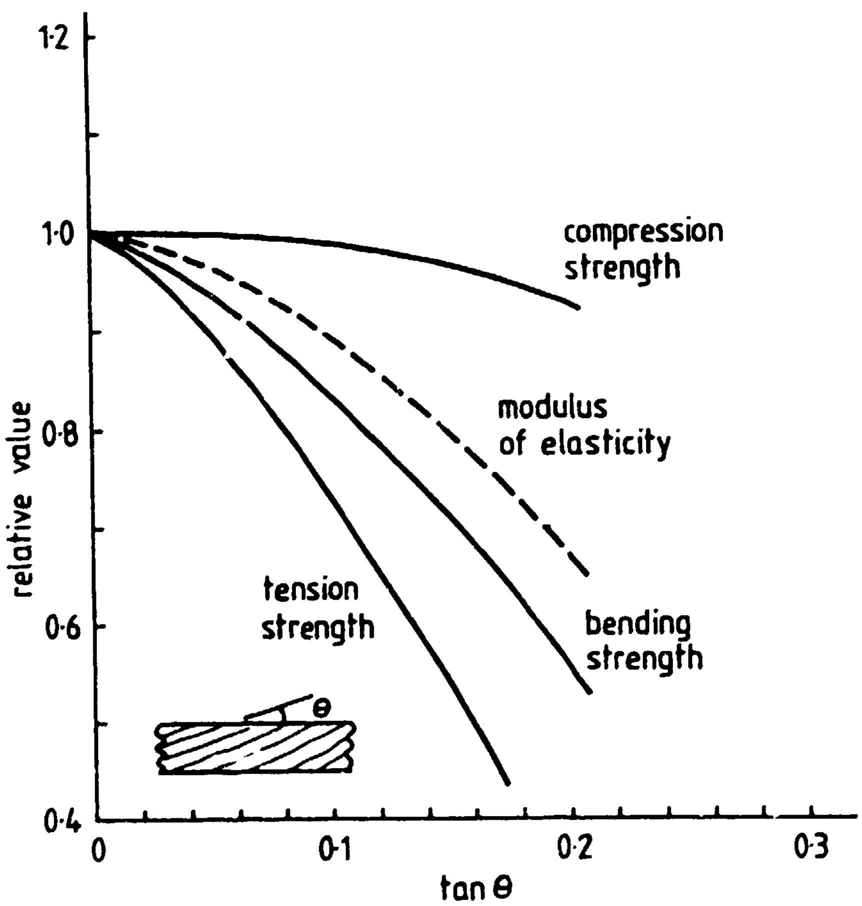
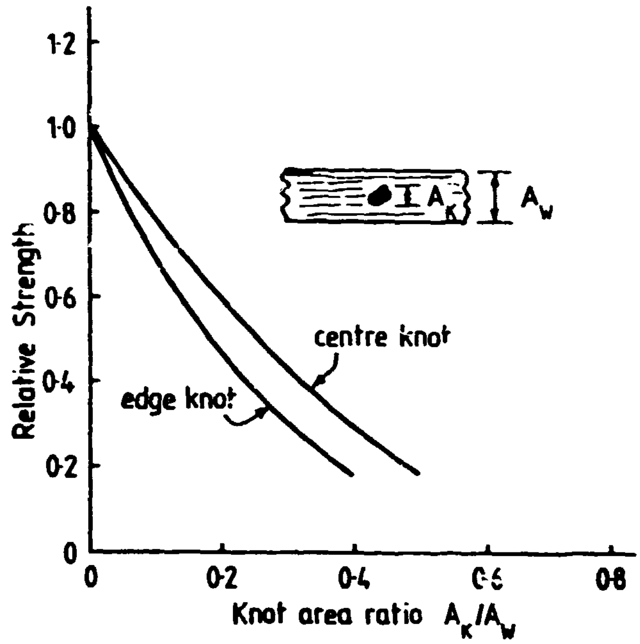


Figure 15. Effect of knot size on tension strength



Source: P. S. Dawe, "The effect of knot size on the tensile strength of European redwood", Wood (November 1964).

I. Characteristics of graded timber

Structural timber is sorted into grades by limiting the sizes of visual defects or some other grading parameters such as local stiffness. The structural characteristics of graded timber can differ quite markedly from those of clear wood, so it must be studied as a separate material. There are some excellent sources of published data on the properties of graded timber in three regions:

(a) Southern pines of the United States: Doyle and Markwardt [7, 8];

(b) Timber of western Canada: McGowan et al. [9], Littleford [10] and Littleford and Abbott [11];

(c) Imported United Kingdom timber: Curry and Tory [12], Curry and Fewell [13] and Fewell [14].

The data of Doyle and Markwardt [7, 8] on Southern pines are summarized, in table form, in annex II. The information in the annex also includes some additional data on clear wood properties that were obtained by extrapolating data from other sources. To normalize the data, the structural grades are quantified in terms of a grade ratio GR, defined as follows:

$$GR = R_{kg}/R_{kc} \tag{17}$$

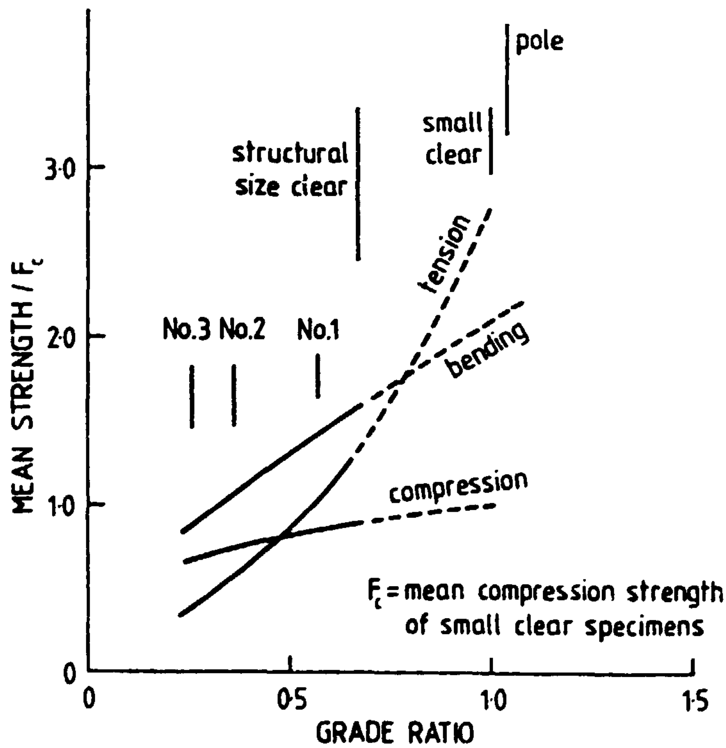
where R_{kg} denotes the characteristic bending strength of graded material and R_{kc} denotes the characteristic bending strength of small, clear wood specimens. The characteristic values are taken to be the five-percentile values.

For the data on Southern pines given in annex II, equation (17) leads to the following grade ratios:

Timber	Grade ratio
Small clears	1.00
Structural clears	0.72
No. 1 grade	0.63
No. 2 grade	0.39
No. 3 grade	0.28

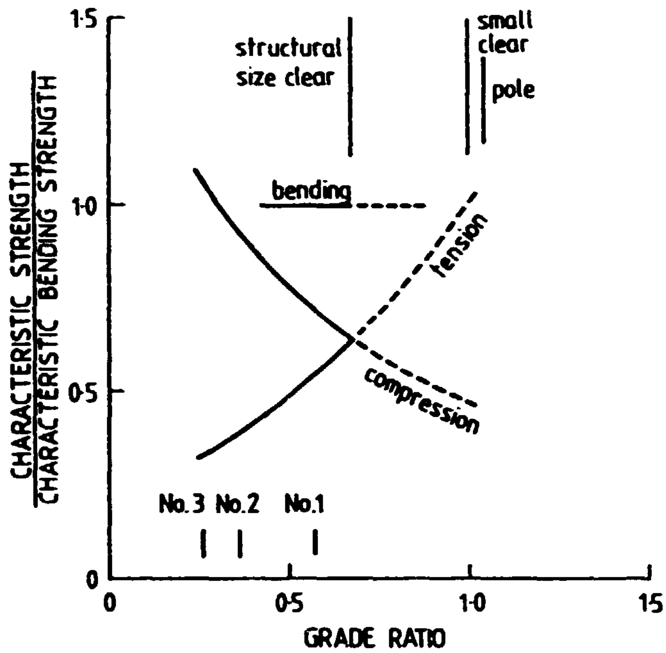
Figures 16-18 show the effect of grade ratio on some of the properties of Southern pine. It is important to note that the properties of small, clear pieces of wood do not provide an accurate picture of the characteristics of structural-size timber.

Figure 16. Effect of grade ratio on the mean strength of Southern pine



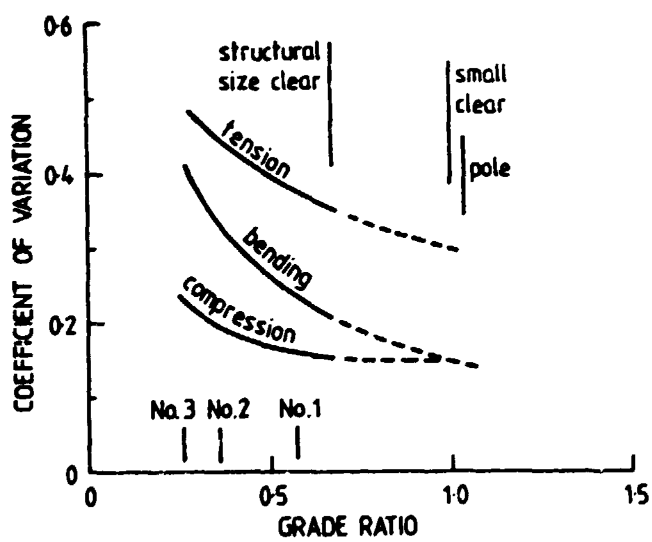
Source: D. V. Doyle and L. J. Markwardt, "Properties of Southern pine in relation to strength grading of dimension lumber, United States Forest Service Research Paper FPL 64 (July 1966) and "Tension parallel-to-grain properties of Southern pine dimension lumber", United States Forest Research Paper FPL 84 (December 1967).

Figure 17. Effect of grade ratio on the characteristic strength ratio of Southern pine



Source: D. V. Doyle and L. J. Markwardt, "Properties of Southern pine in relation to strength grading of dimension lumber, United States Forest Service Research Paper FPL 64 (July 1966) and "Tension parallel-to-grain properties of Southern pine dimension lumber", United States Forest Research Paper FPL 84 (December 1967).

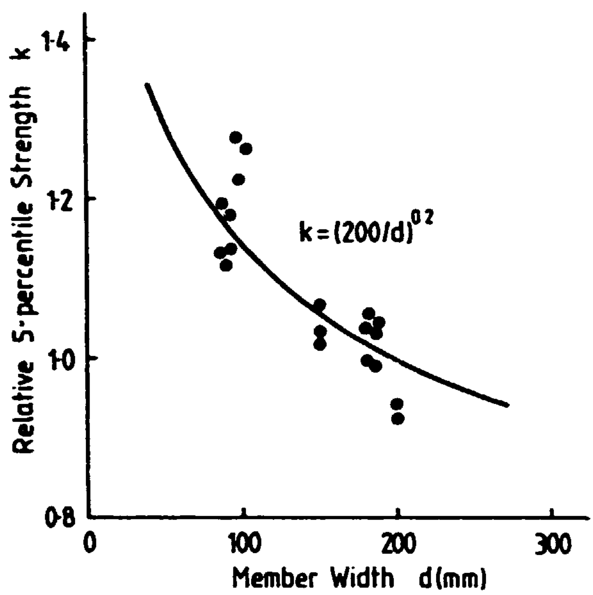
Figure 18. Effect of grade ratio on the coefficient of variation of Southern pine



Source: D. V. Doyle and L. J. Markwardt, "Properties of Southern pine in relation to strength grading of dimension lumber, United States Forest Service Research Paper FPL 64 (July 1966) and "Tension parallel-to-grain properties of Southern pine dimension lumber", United States Forest Research Paper FPL 84 (December 1967).

One troublesome aspect of visually graded timber is that it exhibits a size effect. For sticks of timber with the same visually estimated bending strength ratio but of different size, there is a loss of strength as the stick becomes larger. As an example, for sticks of the same grade of softwood timber, the characteristic strength values, R_k, have the following form: compression strength, $R_k = A_1$; bending strength, $R_k = A_2/d^{0.4}$; and tension strength, $R_k = A_3/d^{0.2}$. Here, A_1, A_2 and A_3 are constants and d is the width of the member. The size effect for tension strength is illustrated in figure 19. This size effect is probably related to the size effect of fracture that will be discussed elsewhere.

Figure 19. Effect of size on tension strength



Source: W. T. Curry and A. R. Fewell, "Grade stress values for timber", Paper prepared for Economic Commission for Europe, Timber Committee, Geneva, June 1981.

Finally, mention should be made of the correlation between structural properties. This correlation is of importance in grading procedures. Higher correlation coefficients lead to more efficient grading. The following are some typical values of correlation coefficients between various parameters and bending strength:

Predictor for bending strength	Correlation coefficient, r
Density	0.50
Average modulus of elasticity	0.50
Visual defect size	0.65
Local modulus of elasticity	0.85
Visual defect size plus density	0.80
Local modulus of elasticity plus average modulus of elasticity	0.90

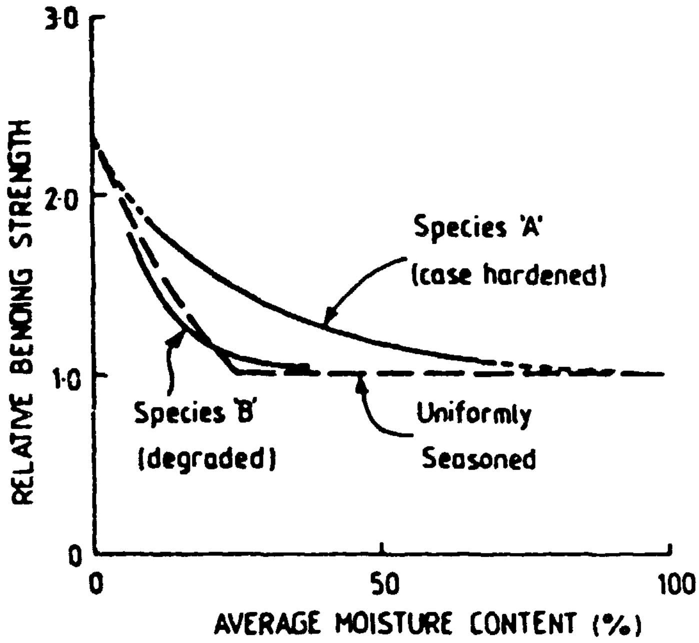
These correlation coefficients are poorer than the values that are obtained with clear wood. A correlation coefficient of less than 0.6 is of little practical value for grading purposes. It should be noted that many factors affect the value of the correlation coefficient and the above values should be regarded as indicative only. For example, Anton [15] has found the correlation coefficient with local modulus of elasticity can be as low as 0.35 if the timber is cut from very young trees.

J. Factors affecting strength

1. Seasoning effects

Figure 20 illustrates the characteristics of strength during drying. The strength value obtained depends on whether the timber can case-harden without degradation. For many species, sizes that are thicker than 150 mm in cross-section are difficult to season without degradation.

Figure 20. Strength during drying

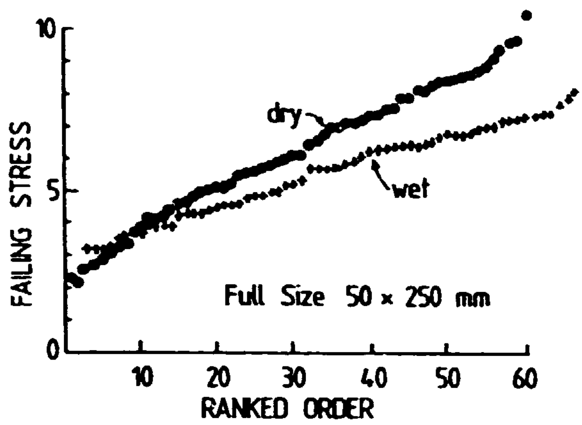


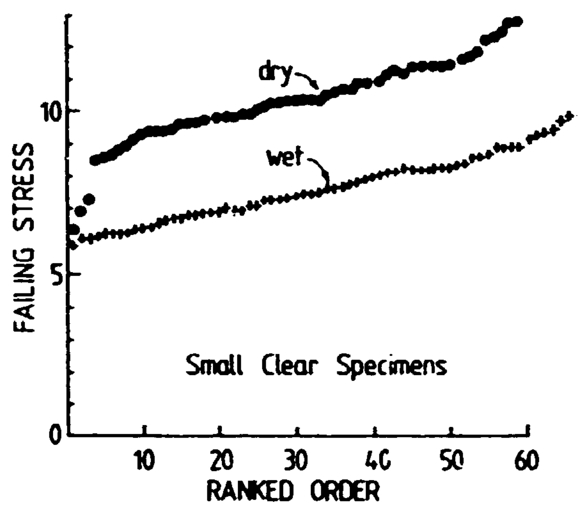
Source: T.R.C. Wilson, T. A. Carlson and R. F. Luxford, "The effect of partial seasoning on the strength of wood", Proceedings of the American Wood Preservers Association (January 1930).

2. Effect of moisture content

The effect of moisture content on strength is illustrated by the example given in figure 21. While there is an increase in strength on drying for small, clear pieces of wood, there is little improvement in strength for the weaker pieces of structural timber, which determine the strength values that are recommended for design purposes.

Figure 21. Effect of moisture content on the bending strength of Douglas fir



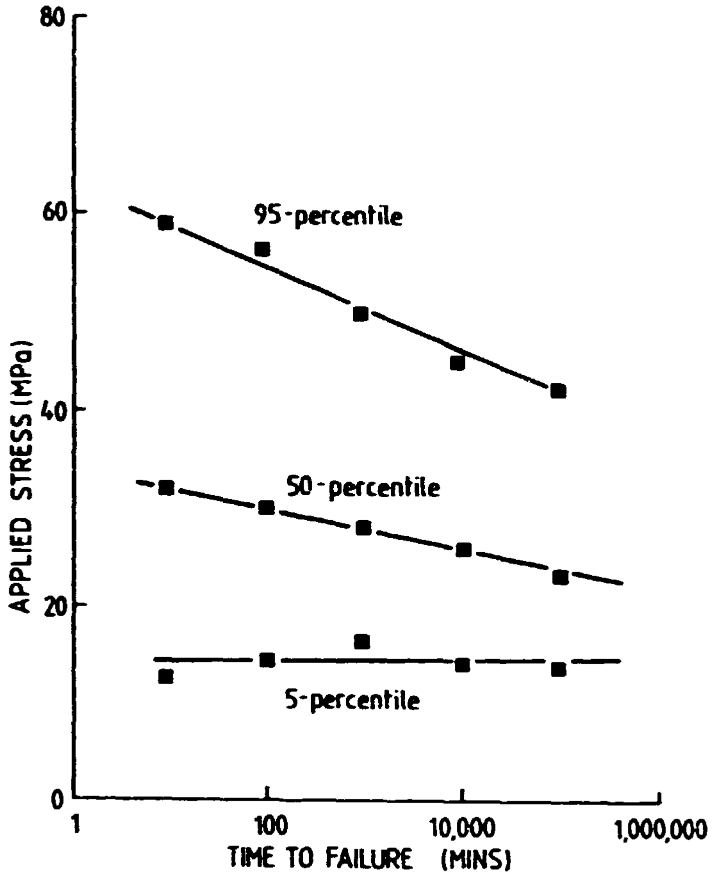


Source: B. Madsen, "Duration of load test for wet lumber in bending", Structural Reserch Series Report No. 4 (University of British Columbia, Department of Civil Engineering. 1972).

3. Effect of duration of load

The effect of duration of load is illustrated by the example shown in figure 22. While the average piece of structural timber exhibits a marked decrease in strength with load duration, only a limited effect is observed for the weaker sticks of timber, which determine the strength values that are recommended for design purposes.

Figure 22. Bending strength of dry hem-fir subjected to ramp loading



4. Fatigue strength

The excellent fatigue strength of clear wood is well known, but it is not generally appreciated that structural timber containing natural defects can deteriorate as a result of fatigue.

In one set of tests on radiata pine, it was found that the average strength was reduced by 20 per cent when subjected to 500 cycles of repeated loading. In other tests, it has been found that if a load is applied such that it causes 5 per cent of a population of structural timber to fail, then on the second application of the same load typically a further 0.5 per cent will fail, i.e. some timber is susceptible to one-cycle fatigue [16].

K. Pole timber

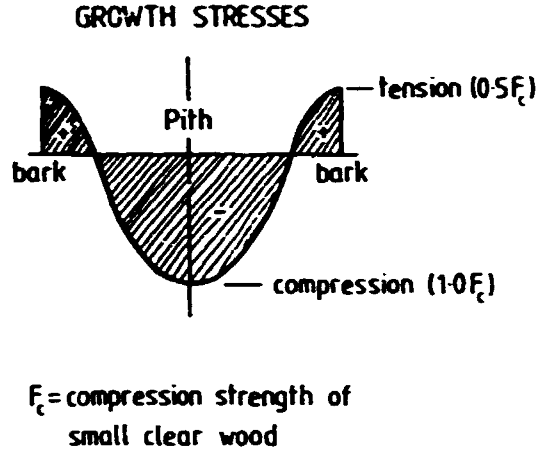
The characteristics of pole timbers differ from those of sawn structural timber for several reasons, including the following:

(a) The wood grain of pole timbers is continuous along edges and around knots, thereby providing a high tension strength;

(b) The strength of pole timbers is enhanced by growth stresses such as those shown in figure 23;

(c) The structural properties of pole timbers are enhanced by the fact that the better timber is on the outside, where it is more effective.

Figure 23. Growth stresses in a typical hardwood



In an extensive series of tests on poles, Boyd found the following typical properties of pole timbers [17]:

$$\text{Mean } (F_b) = 1.1 \text{ mean } (F_{bsc}) \quad (18a)$$
$$\text{Mean } (E) = 1.2 \text{ mean } (E_{sc}) \quad (18b)$$
$$\text{COV } (F_b) = 0.13 \quad (18c)$$

where F_b and E denote the bending strength and modulus of elasticity of the pole timbers in green moisture content conditions and F_{bsc} and E_{sc} are the corresponding values measured on green, small, clear specimens. The term COV () denotes the coefficient of variation. From the above, the following relationship is obtained:

$$\text{Grade ratio} = 1.1 \quad (19)$$

A comparison of this grade ratio with those given earlier indicates that the effective grade of pole timbers is more than 50 per cent better than that of sawn timber.

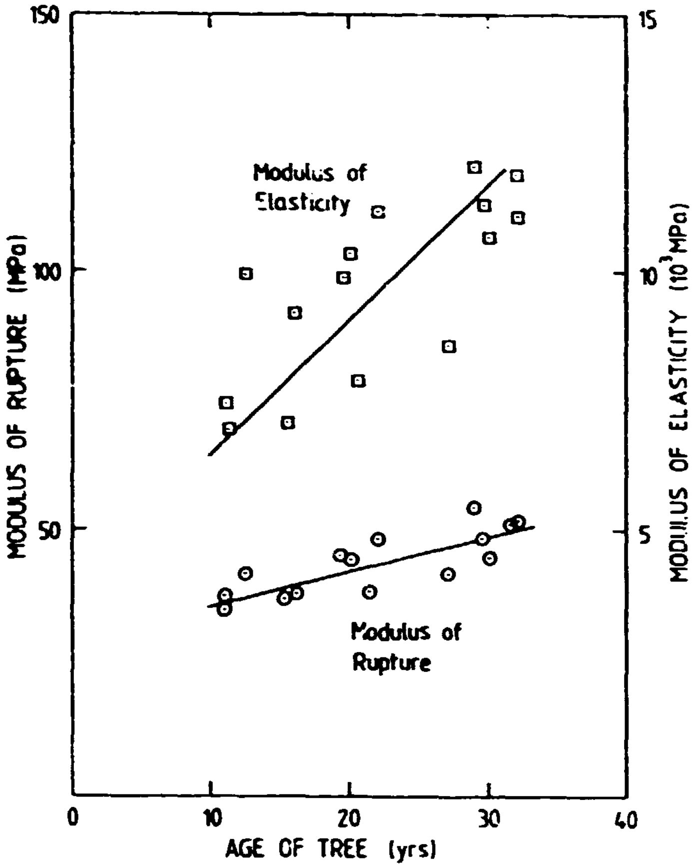
Other matters of practical importance that were noted by Boyd include the following:

(a) The age of the tree can have a significant influence on the structural properties, as illustrated in figure 24;

(b) A regime of seasoning and resoaking, as would occur in the preservative treatment of poles, may reduce the strength by up to 15 per cent;

(c) Shaving poles to improve their aesthetics could lead to a loss of some 10 per cent in strength. This is due to the interruption in the continuity of the wood grain.

Figure 24. Effect of tree age on the pole properties of radiata pine

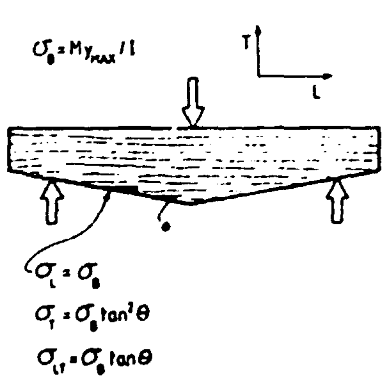


Source: J. D. Boyd, "The strength of Australian pole timbers: IV. Radiata pine poles", Division of Forest Products Technological Paper No. 32 (1964).

L. Tapered beams

For reasons of economy, beams are sometimes cut to a taper. The stresses that arise in this situation have been discussed by Macki and Kuenzi [18]. In addition to the normal checks on strength, an additional check should be made for strength to resist the combined stresses acting along the edge of the beam, as shown in figure 25. In this check, the failure criterion given by equation (5) is to be used.

Figure 25. Stresses along the edge of a tapered beam



M. Conclusions

The preceding discussion has provided information on the structural characteristics of timber that are relevant to the formulation of design recommendations. These include the following:

(a) Both the orthotropic and the non-homogeneous nature of timber must be considered;

(b) While the properties of small specimens of clear wood are useful as index properties for structural timber, it can be quite misleading to use the characteristics of these specimens to derive the characteristics of structural timber;

(c) Design strength is related to the five-percentile values of strength, which may not have the same structural characteristics as the mean strength.

Annex I

STRENGTH THEORY FOR NON-HOMOGENEOUS MATERIALS

Only the essential aspects of this theory will be considered. First it is necessary to recall that for a Weibull distribution of strength R, the probability that R will be greater than x is given by

$$Pr(R > x) = \exp[-\alpha(x/\bar{R})^m] \tag{20}$$

where the material parameters α and m are given by

$$\alpha \cong 0.555 + 0.445V_R$$

$$m \cong \bar{V}_R^{1.09}$$

and V_R is the coefficient of variation of R.

Now consider a structural unit made up of N elements having strengths R_1, R_2, ... R_N and subjected to the stresses σg_1, σg_2, ... σg_N, respectively, when the nominal stress level in the unit is denoted by σ. The probability of survival by the unit for this stress condition, denoted by $Pr(R_{unit} > \sigma)$, is given by

$$Pr(R_{unit} > \sigma) = Pr(R_1 > \sigma g_1),\ Pr(R_2 > \sigma g_2) \ldots Pr(R_N > \sigma g_N) \tag{21}$$

If it is assumed that all elements have a strength defined by equation (20), then equation (21) can be written

$$Pr(R_{unit} > \sigma) = \exp[-(\alpha\sigma^m/\bar{R}_m)(g_1^m + g_2^m + \ldots g_N^m)] \tag{22}$$

A continuous model of equation (22) is

$$Pr(R_{unit} > \sigma) = \exp[-(\alpha\sigma^m/\bar{R}^m) \int g^m(x,y,z)d\phi] \tag{23}$$

where the parameter ϕ denotes volume and g(x,y,z) is a function of the coordinates x, y and z such that σ g(x,y,z) denotes the stress at the point (x,y,z), where σ is the nominal stress level in the member.

By comparison with equation (20), equation (23) can be written in terms of R_{unit}, the mean nominal strength of the structural unit, as follows:

$$Pr(R_{unit} > \sigma) = \exp(-\alpha\sigma^m/\bar{R}^m{}_{unit}) \tag{24}$$

Comparison of equations (4) and (5) leads to

$$\bar{R}^m_{unit} = \bar{R}^m / \int g^m(x,y,z)\ d\phi \tag{25}$$

Thus a computation of the integral $\int g^m(x,y,z)\ d\phi$ for various structural configurations leads to comparative values of the mean strength. It should be noted that since the exponent m remains unchanged, R and R_{unit} are both Weibull distributions with the same coefficient of variation.

Annex II

PROPERTIES OF 100 x 50 mm DRY SOUTHERN PINE

Property/material	Mean value (MPa)	Fifth percentile (MPa)	Coefficient of variation
Compression parallel to the grain			
Small clear	43	33	0.15
Structural clear	40	31	0.15
No. 1 grade	37	28	0.15
No. 2 grade	33	23	0.19
No. 3 grade	29	19	0.23
Bending parallel to the grain			
Small clear	90	70	0.15
Structural clear	67	47	0.20
No. 1 grade	62	40	0.24
No. 2 grade	45	25	0.32
No. 3 grade	39	18	0.45
Tension parallel to the grain			
Small clear	120	71	0.30
Structural clear	54	29	0.35
No. 1 grade	39	19	0.39
No. 2 grade	25	11	0.44
No. 3 grade	16	7	0.49
Modulus of elasticity parallel to the grain			
Small clear	12 500	8 090	0.25
Structural clear	12 500	8 090	0.25
No. 1 grade	12 400	8 310	0.23
No. 2 grade	10 400	7 280	0.19
No. 3 grade	9 500	5 610	0.29
Modulus of rigidity (G_{LT}, G_{LR})			
Small clear	-	-	-
Structural clear	-	-	-
No. 1 grade	900	770	0.15
No. 2 grade	910	660	0.17
No. 3 grade	920	710	0.14
Shear strength parallel to the grain			
Small clear	9.1	-	-
Compression strength perpendicular to the grain (2.5 mm deformation)			
Small clear	11.5	-	-

Sources: D. V. Doyle and L. J. Markwardt, "Properties of Southern pine in relation to strength grading of dimension lumber", Research Paper FPL 64 (Madison, Wisconsin, United States Department of Agriculture, Forest Products Laboratory, July 1966; D. V. Doyle and L. J. Markwardt, "Tension parallel-to-grain properties of Southern pine dimension lumber", Research Paper FPL 84 (Madison, Wisconsin, United States Department of Agriculture, Forest Products Laboratory, December 1967).

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II. STRUCTURAL GRADING OF TIMBER

William G. Keating*

Introduction

Sound timber grading practice, for either structural or other purposes, lies not in the selection of perfect timber: on the contrary, it permits the inclusion in each grade of as many defects as is possible without detracting from the suitability of the timber for the purpose for which it is required. Structural timber grading therefore aims to provide the engineer with a degree of confidence so that when a particular grade of a certain species or species group is specified, a consistent minimum quality, irrespective of the origin of the material, can be expected.

The process of structural grading, also known as stress grading, may be defined as the technique by which timber is sorted according to its ability to carry loads. This process must have a set of rules or criteria for making the sort and a procedure for assigning design properties to each of the grades.

Stress grading is carried out by either visual or mechanical techniques. These may be further subdivided depending on the particular approach.

A. Visual stress grading

Visual grading is the oldest stress grading method in use. It is also the simplest and the most widely used method. It is based on the premise that the mechanical properties of timber containing certain visible characteristics (defects) differ from those of timber that is entirely free of such defects.

In the development of the basic working stresses appropriate to each grade, two different approaches are used. It is worthwhile explaining the procedure in each approach to illustrate the cautious attitude adopted.

1. Small clears approach

Extensive laboratory testing, mainly in the United States during the 1920s, indicated that defects of a given size and location cause approximately the same proportional reduction in strength irrespective of species. This fact has permitted the preparation of strength grading rules for timber in which the visible imperfections are limited so that the weakest piece in any grade should have a certain percentage of the strength of a similar piece of clear timber. This is sometimes called the strength ratio. For example, a joist with a strength ratio of 60 per cent extreme fibre stress in bending would be expected to have at least 60 per cent of the strength of a corresponding clear piece.

As a first step in the process of developing basic working stresses, a comparatively large number of small, clear specimens of a particular species need to be tested in bending, shear and compression parallel to the grain. The methods of testing, sampling and recording results have been laid down in internationally accepted standards such as that of the American Society for Testing Materials, ASTM D143 [1]. A large number of specimens must be tested for each property to obtain some indication of the species variability, which

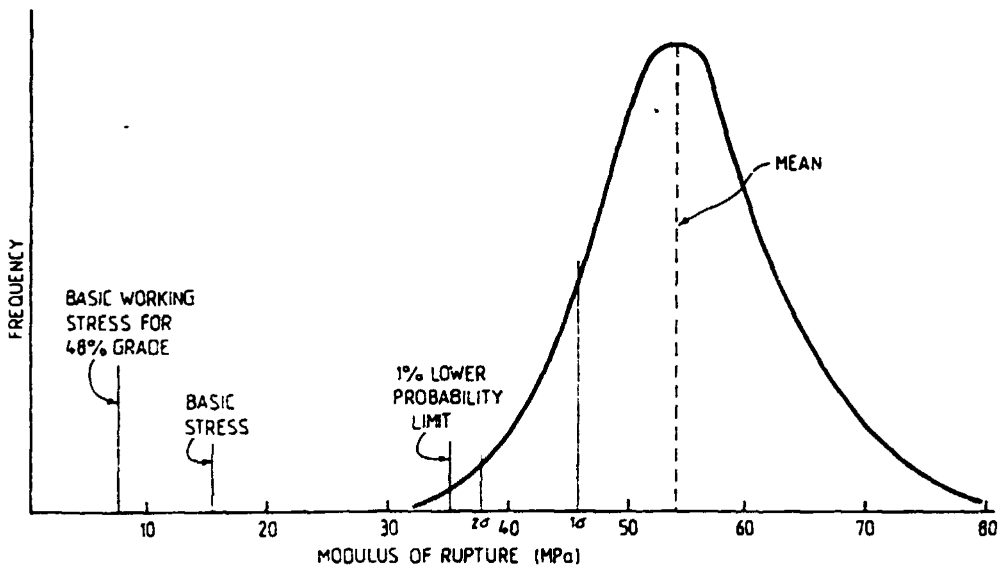
*An officer of CSIRO, Division of Chemical and Wood Technology, Melbourne.

is then interpreted by normal statistical methods. From the distribution curve obtained, the visual practice is to determine firstly the 1 per cent lower probability limit. Taking this limit means that the probability is that 99 per cent of the values will fall above it. Figure 26 illustrates the procedure and indicates that, on average, the 1 per cent lower probability figure is approximately 70 per cent of the mean, or, in statistical terms, $\bar{f}_b - 2.33\sigma$, where $\bar{f}_b$ is the mean bending strength and σ, the standard deviation, is equal to

$$\sigma \sqrt{\{[\Sigma(x - \bar{x})^2]/n\}}$$

where x is the individual test value, $\bar{x}$ is the mean of the test values and n is the number of tests.

Figure 26. Derivation of basic working stress in bending for a 48 per cent grade of a typical species



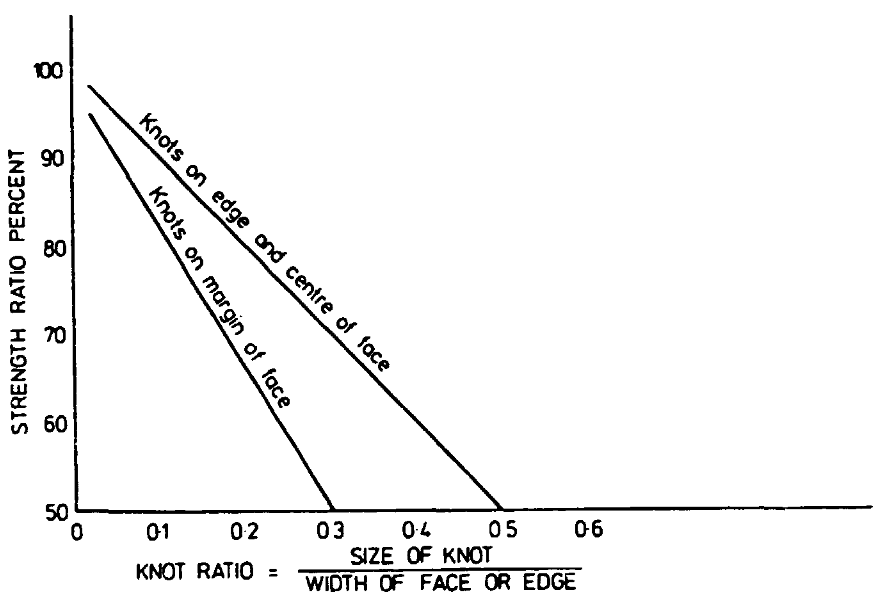
To obtain the basic stress from the 1 per cent probability figure, a reduction factor, k_r, is introduced to allow for long-duration loading, accidental overloading, shape and size of specimens and several other intangibles. Experience over many years has indicated that these may be conveniently combined into one figure. However, it is not necessarily the same figure for every strength property. The general formula for basic stress in bending is $(\bar{f}_b - 2.33\sigma)/k_r$. For bending stress $k_r = 2.22$, it is $(\bar{f}_b - 2.33\sigma)/2.22$.

This, then, is the basic stress for clear timber free of all strength-reducing defects of the particular species tested. To arrive at the basic working stress or grade stress, i.e. a stress that can be assigned to structurally graded material for design purposes, it is necessary to apply the strength ratio factor to allow for the loss of strength caused by slope of grain, knots, shakes, checks or other strength-reducing features as detailed in the appropriate visual grading rule. In figure 26, a strength ratio of 0.48 has been assumed. The foregoing process may be summarized as follows for the particular example:

Mean	$= \bar{f}_b$	= 54.0 MPa
↓		
1 per cent lower probability limit	$= \bar{f}_b - 2.33$	= 35.0 MPa
↓		
Basic stress	$= \bar{f}_b - 2.33\ )/2.22$	= 15.8 MPa
↓		
Basic working stress (grade stress)	$= [(\bar{f}_b - 2.33\ )/2.22]0.48$	= 7.58 MPa

The influence of defects such as knots on the strength ratio is illustrated in figure 27. Similar diagrams may be drawn for the other strength-reducing defects. All relevant strength ratios are set out in tabular form in ASTM D245 for the different types and numbers of defects likely to be encountered in structural-size timber during visual grading. This particular standard has been used around the world for many years.

Figure 27. Effect of knot ratio on strength



Source: L. G. Booth and P. O. Reece, The Structural Use of Timber: A Commentary on the British Code Standard of Practice CP112 (London, Spon Ltd., 1967), p. 287.

In Australia and many other countries, the set of basic working stresses for the various grades has been determined based on a limited number of groups of species rather than individually for a large number of single species. This process is described in another paper.

The visual grading rules lay down limits for the various defects appropriate to each grade.

2. Structural size approach

Extensive laboratory testing has indicated that the derivation of basic working stresses based on testing small, clear specimens and visual grading may result in misleading recommendations for design strengths. The reason is that design strengths must be based on the weaker pieces at the lower tail of structural populations, typically the weakest 1 per cent or 5 per cent; and while the structural characteristics of the average stick of timber are related to the properties of clear wood, the structural characteristics of the timber at the lower end of the distribution are determined by the natural defects occurring in this material.

This approach does not necessarily imply a change in the existing visual grading method, but it does indicate that a change in the basic working stresses may be warranted for a particular grade or grades for a certain species or species group.

In any derivation of basic working stress, it is necessary to define the reference population, and it is probable that the population in this case will refer to a commercially supplied specific grade of timber obtained from a specific source. From this reference population a representative sample is chosen, and following a comparatively large number of standard tests the five-percentile values are determined for strength and stiffness. From these values, the basic working stresses are obtained by dividing by a load factor that reflects the relationship between three-minute loading and long-term loading, together with a factor that is chosen to match existing practice (factor of safety).

The above technique is, in effect, a refinement process to produce more efficient designs than those resulting from the small clears approach, which must of necessity be conservative. This grade verification process, or in-grade testing, is expensive and time-consuming and is warranted only when the species or species group comprises a sufficiently large proportion of the total use. (The most recent Australian draft standard for in-grade testing was discussed at length in a paper by R. H. Leicester [2]. Material on an earlier draft standard, presented at the Timber Engineering Workshop, has been deleted because it is obsolete.)

B. Machine stress grading*

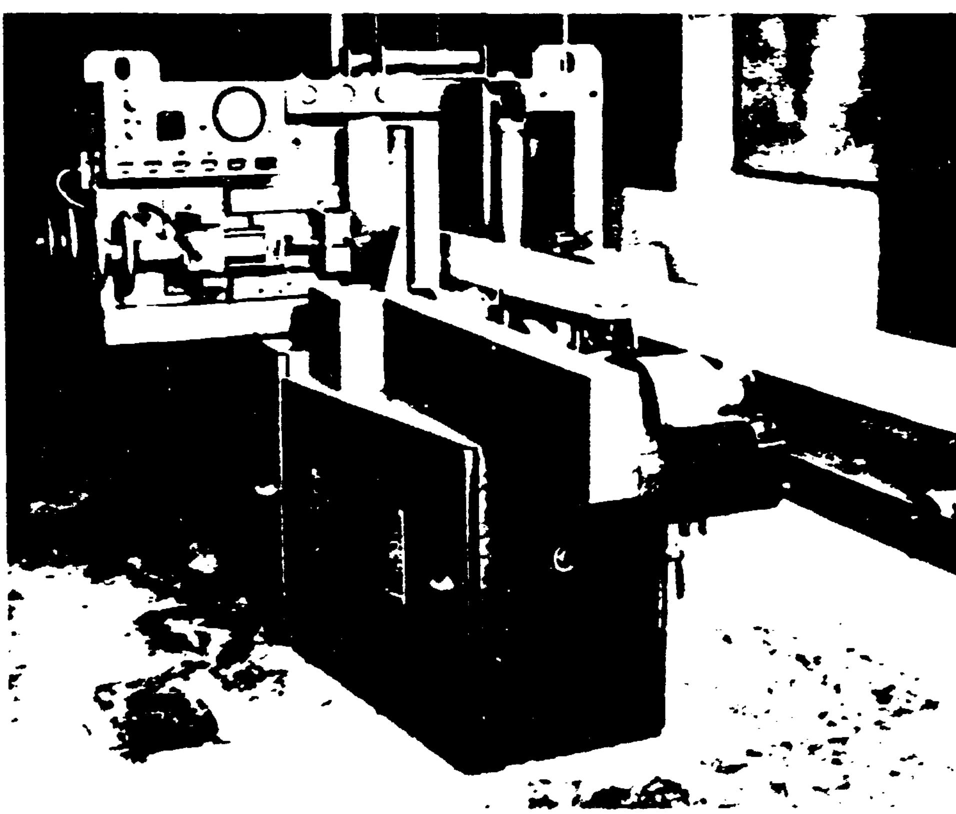
Although visual grading results in greater efficiency in the structural use of timber than if no grading were performed, it suffers nevertheless from a number of disadvantages. Each piece of timber has to be handled and examined on four surfaces. Moreover, once the defects have been measured, it is assumed that pieces of the same species containing defects of the same size are approximately equal in strength. Both these factors lead to high costs. The assumption that pieces containing similar defects are approximately equalin strength implies that all pieces of clear timber of the same species are of equal strength. In fact, apparently similar specimens may differ widely in strength due to various factors, one of which is density. The presence of knots or sloping grain increases the difficulty of estimating strength, since a piece with high inherent strength containing knots may be

*For a further discussion of machine stress grading, see C. J. Mettem, "The principles involved in stress grading, with special reference to its application in developing countries", Paper presented at UNIDO Expert Group Meeting on Timber Stress Grading and Strength Grouping, Vienna, Austria, 14-17 December 1981 (ID/WG.359/3, 1982).

stronger than a clear piece with low inherent strength. For these reasons, visual grading is a somewhat inefficient method and the associated assignment of working stresses must be conservative.

A more direct and accurate approach is to use mechanical grading. This system is based on the fact that there is a direct correlation between the stiffness and ultimate strength of a piece of timber containing defects. This relationship has not always been recognized. For many years, textbooks stated categorically that while defects affected strength generally in proportion to their size or severity, they had no significant effect on stiffness. Pioneer work at the former CSIRO Division of Forest Products on radiata pine showed that such a relationship did exist. Further work at CSIRO, at the Division of Wood Technology of the New South Wales Forestry Commission and at some overseas laboratories resulted in the development of machines that could utilize this relationship to test timber in a non-destructive fashion to predict its load-carrying capacity. Figure 28 shows the current version of the machine developed by the Division of Wood Technology.

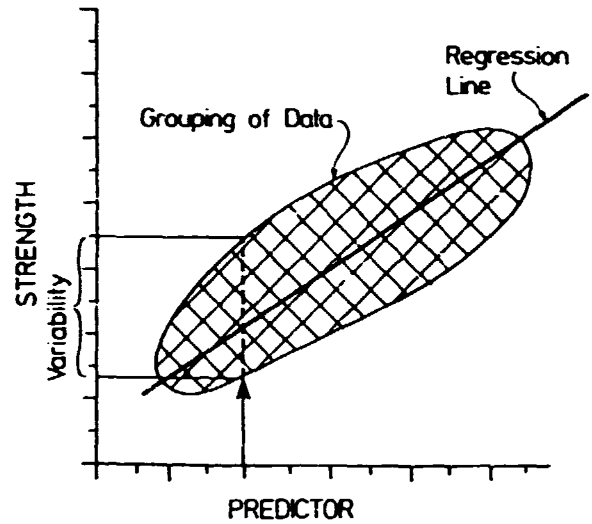
Figure 28. Computermatic stress grading machine



Source: Forestry Commission of New South Wales.

All grading systems are based on the use of predictions to estimate strength properties. In visual grading, for example, the size of visible defects such as knots is used to predict strength. In machine stress grading, the stiffness is used as a predictor. The relationship between the predictor and the mechanical properties of interest is shown by a statistical technique known as a regression. Figure 29 illustrates the use of a regression and the effect of the variability in data on the accuracy of the prediction [3]. Clearly, the tighter the data group around the regression line, the lower the variability and the better the prediction of the strength.

Figure 29. Typical relationship between a predictor and the strength of timber



In mechanical grading, a non-destructive test is made on each piece by passing it through a machine that continuously applies a small load to it over successive short lengths of the piece. The maximum deflection obtained is recorded, and from the known relationship between the modulus of elasticity and the modulus of rupture, the ultimate strength of the piece can be predicted.

To illustrate how the system operates, the following example is taken from work carried out in the United Kingdom by Sunley, Curry and Hudson and reported in 1962 and 1964 [4]. A typical relationship between the modulus of elasticity and modulus of rupture is shown in figure 30. The variability of the material is then taken into account by calculating confidence lines above and below the regression lines such that 98 per cent of the results lie between these lines. The lower confidence line shown in figure 30 is such that only 1 in 100 results will be below the line. The confidence lines are determined by the usual statistical technique of drawing lines parallel to the regression line and at a vertical distance of +2.33 times the standard error of estimate of the modulus of rupture. By applying factors to allow for long-term loading and a factor of safety, the recommended grade (working) stress line is shown.

Table 1 shows the results of visual and mechanical grading for Baltic redwood (Pinus sylvestris L.) and whitewood (Picea abies (L.) Karst.) as obtained by Sunley and Hudson [5]. Thus, compared to visual methods, mechanical grading gave either a larger yield of material in the higher grades or higher working stresses for similar grade yields.

Figure 30. Regression, lower 1 per cent and grade stress lines: European redwood and whitewood

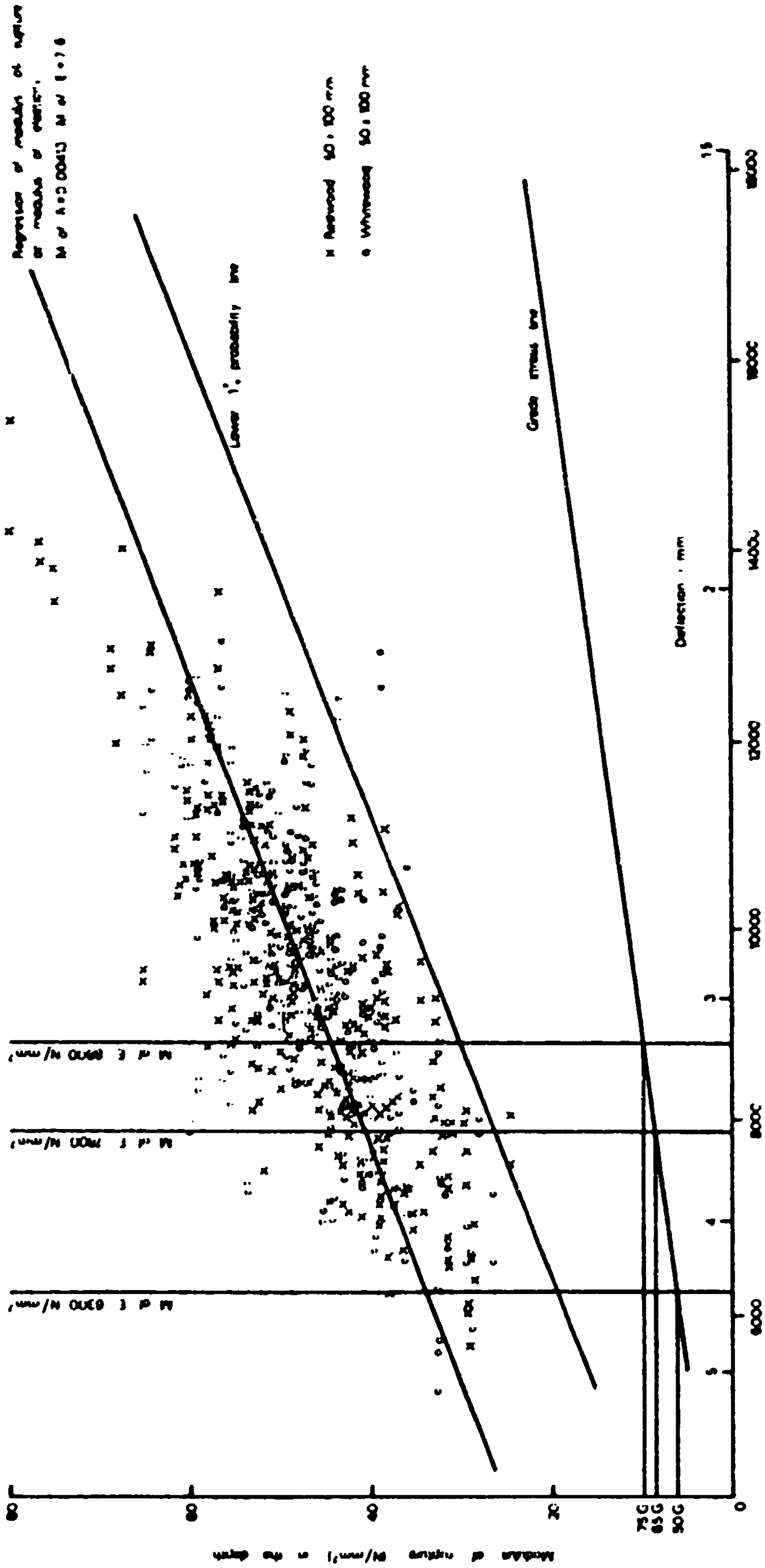


Table 1. A comparison of the results of visually and mechanically graded 2 x 4 in. Baltic redwood and whitewood

Visual grade	Visual grade stress a/ (lbf/in.^{2})	Visual grade yield (%)	Yield of mechanical grades for stresses in column 2 (%)	Grade stress for mechanically graded timber to give same yields as column 3 (lbf/in.^{2})
1	2	3	4	5
75	1 450	35	68.0	1 700
65	1 250	33	18.5	1 450
50	950	23	12.0	1 150
Reject		9	1.5	

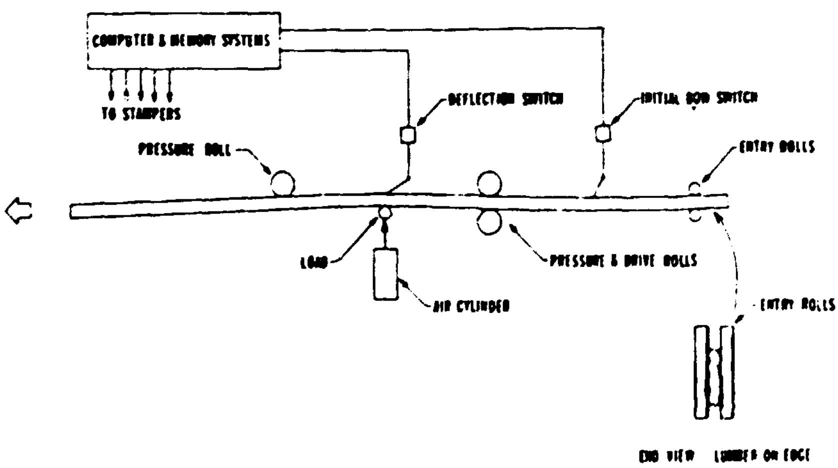
Source: L. G. Booth and P. O. Reece, The Structural Use of Timber: A Commentary on the British Standard Code of Practice CP112 (London, E. and F. N. Spon, 1967), p. 287.

a/ Moisture content, 18 per cent.

1. Computermatic

The machine produced and available in Australia is marketed under the trade name Computermatic. Pieces of timber are fed into the machine on edge in a longitudinal direction. The individual piece is continually deflected in its narrow dimension by a given load. The amount of deflection caused by this load is measured at 150 mm intervals throughout the length of the piece. Measurements are grouped into one of five classes. A colour mark corresponding to the grade class can be sprayed on the pieces at each 150 mm interval. As the piece leaves the machine, the lowest grade rating is computed and a paint spray corresponding to that low point rating is sprayed. Figure 31 shows the operation of the machine.

Figure 31. Operation of the Computermatic machine



The development of this machine was the result of a co-operative venture between the New South Wales Forestry Commission and the Plessey Company. It has met with considerable success in Europe and, for smaller mills, in the United States [6].

There are several other machines on the market. Their operation, as summarized by Mettem [7], is described next.

2. Cook Bolinder SG-AF stress grading machine

The Cook Bolinder machine, which was introduced in 1979, receives the timber on edge and deflects it laterally over a 0.9 m span, in the same configuration as the Computermatic. A high level of accuracy is claimed, probably justly, by use of the principle of constant deflection with load sensing. This is the reverse of the Computermatic method, which loads a given timber and size to a constant stress and senses the ensuing deflections, which vary along the piece.

The timber is deflected in opposite directions in two separate passes through the machine. A computerized sensing mechanism reads off the variable force caused by the fixed deflection, the force being detected by a load cell. During the second pass, the computer provides a continuous read-out of the mean values. Dye marks and stamp marks are applied to indicate grade.

The use of separate passes eliminates interference effects associated with double bending. Constant deflection makes it possible to avoid errors in the load-deflection measurement caused by timber vibrations.

3. Raute Timgrader stress grading machine

The technical literature describing the Raute Timgrader machine dates from 1978, the approximate date of its introduction, although work on prototypes had started earlier.

Timber is deflected horizontally as it passes through on edge, in common with both machines described above. Like the Cook Bolinder, the Timgrader is a fixed-deflection, variable-load machine. A difference in principle is that in this machine the method of allowing for bow is to bend the timber successively in opposite directions in a single pass. The variable forces required to provide fixed deflections are sensed and averaged electronically. The measuring frequency is synchronized with the feed speed of the timber at about 100 mm intervals along the length passing through.

The machine possesses a substantial electronic system, including a process control microcomputer and a unit that can transmit grading information to a sorting machine.

In deriving settings for this machine, it is necessary to calculate allowances for interaction between a low stiffness portion of the length (due, perhaps, to a gross defect) passing one deflection station and material of a different quality (another part of the length) passing the second station.

4. Sontrin timber selector

The Sontrin timber selector has not been approved for general-purpose stress grading to British Standard 4978 under the Kitemark system because of its insensitivity to localized gross defects, particularly away from the centre of the span. These defects may be of importance in structural members with built-in or partially fixed ends, such as rafters or trussed rafters. The timber selector is useful, however, for grading simply supported bending members such as joists. It may be used either as an adjunct to visual grading, taking advantage of the machine's ability to eliminate pieces of low gross stiffness, or in conjunction with a setting procedure that has been devised to weight the required apparent modulus in the settings according to the distance of the suspected defect from the lateral centre line of the piece.

The machine subjects a piece of timber to a predetermined central load and senses the ensuring deflection, giving a simple pass/fail indication for a particular setting. The material is deflected in the depth-wise direction, not laterally as in the machines described above. Variable spans are used, these being related to the intended end use of the piece and ranging from 2.4 m to 5.4 m.

5. Plan-Sell/Innotec Finnograder device

Since the Plan-Sell/Innotec Finnograder, a third-generation device, uses non-contacting measurement methods, it is not correct to refer to it as a machine. The manufacturing company was established in 1973 and was taken over as a subsidiary of Plan-Sell in 1978. The Finnograder has been assessed at the Technical Research Centre of Finland.

The Innotec instruments, which also include an on-line moisture meter and an automatic control system for edging, perform measurements by various kinds of electromagnetic radiation. The Finnograder is claimed to detect density, knottiness, slope of grain and moisture content. Having established relations between these features and tested strengths, performance can be predicted in bending about either axis or in tension.

6. ISO-GreComat grader

The principle of the ISO-GreComat grader, which was assessed at the Otto-Graf Institute at Stuttgart in 1980 and 1981, is to grade timber by measuring its density through the use of isotopes. Knot area ratios are assessed by determining ratios between local and general density, and allowances are made for the influence of moisture content and dimensional variations.

In Canada, according to Kennedy [8], machine-stress-rated (MSR) lumber is produced exclusively by passing kiln-dried lumber through a Continuous Lumber Tester, or CLT-1, marketed by Metriguard Inc. of Pullman, Washington. The machine deflects lumber over a series of 4-ft spans in both vertical directions as it passes through the machine at up to 1,200 ft/min. The load required to achieve a given deflection is recorded in the memory of the machine and used to calculate average and minimum E values, which in turn are correlated with F_b values through previously established regression equations. The tentatively machine-rated lumber is still visually graded and subjected to a visual override, which culls those pieces with edge knots, which would severely limit the performance of the lumber when used on edge but which would not be detected as the lumber moved through the machine on its wide face.

Mills producing MSR lumber must have the approval of the Canadian Lumber Standards Accreditation Board, to whom a written plant standard must be submitted. The standard describes the operation of the mill's grading machine and the qualifications of its operators and quality control personnel. The trade association responsible for grade marking grants initial approval for MSR production after sampling and independent testing of grade-marked material to ensure accuracy of the grading machine. Continuous quality control involves daily mill tests of production, cumulative record-keeping, maintenance of grading and static testing machines, and periodic visits by agency supervisors. Product requirements and producer responsibilities are outlined in a specification.

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III. PROOF GRADING OF TIMBER

Robert H. Leicester*

A. Need for an alternative method of stress grading

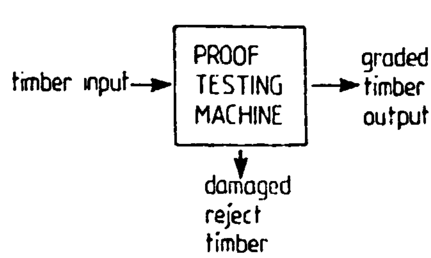
In most areas of Australia, it is mandatory for all timber sold for structural purposes, including that to be used in house framing, to be stress graded. This ruling has created difficulties for the supply of timber, because the two methods that are available for the grading of timber are not entirely suitable for use in the small mills that produce much of the structural material.

One of the two available methods of grading, the visual grading method, is based on the correlation between the visual appearance of a timber and its strength and is very difficult to apply in the confused and untidy environment of the typical small mill. The other method of grading is machine stress grading, which is based on the correlation between local stiffness in a timber and its strength. For the small mill, the drawbacks of this method include the cost of the machine and ancillary equipment, usually in excess of $US 80,000, and the requirement for sophisticated technological support. It was to overcome these difficulties that the concept of proof grading, a method of grading by proof testing, was proposed.

B. Concept of proof grading

The essence of proof grading is to allow any given stick of timber to qualify for a particular stress grade if it first demonstrates the ability to sustain a proof bending stress specified for that particular stress grade. The proof bending stress is applied by a proof testing machine. A schematic of this procedure in its simplest form is shown in figure 32.

Figure 32. Simple proof grading procedure for a small mill



There are several features of proof grading that make it well suited for application in a small mill. The method does not require a high level of technology and the machine to do the grading is both simple and inexpensive. It has been estimated that a machine for continuous on-line testing should cost considerably less than $US 10,000.

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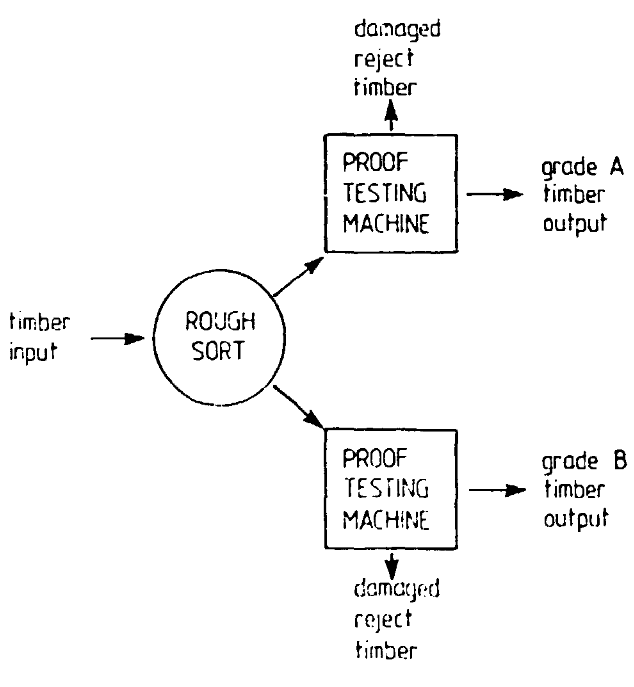
*An officer of CSIRO, Division of Building Research, Melbourne. The author is indebted to A. Hill (Hilleng, Queensland), C. Mackenzie (TRADAC, Queensland) and W. G. Poynter (Foxwood, Queensland) for some of the information provided herein.

However, even when applied outside the small mill situation, proof grading has some advantages over other grading methods. In contrast to those other methods, it can assess the strength of very localized defects, such as those associated with compression shakes or fabricated finger joints. It can also be applied to mills that handle mixed, unidentified species, a situation that occurs in many areas of Australia. Finally, by changing the proportion of reject timber produced by changing the proof stress applied, the stress grade of a specific population of timber can be varied, allowing the output of a mill to be adapted to meet varying market situations.

A commercial disadvantage in the application of proof grading to the output of a large mill is the fact that the output of a proof testing machine is timber of only one stress grade. To some extent this disadvantage can be minimized and more than one stress grade can be produced if a rough initial pre-sorting is carried out, perhaps by visual means or perhaps by the use of a simple stiffness sorting machine. An outline of such a scheme is shown in figure 33. The pre-sorting of timber into grades is undertaken in a variety of ways depending on the operational procedures used at a particular mill. A visual assessment is usually part of the sorting procedure and can prove highly effective, particularly as the proof testing machine provides feedback on the accuracy of the assessment method. The use of stiffness, averaged along the length of a stick, has yet to be tried, but in theory this can be a method for obtaining timber tailored to meet a specified set of strength and stiffness criteria.

Figure 33. Proof grading procedure with rough sort for a large mill



To investigate the feasibility of proof grading, and in particular to study the numerous commercial ramifications of its application as outlined above, extensive laboratory and field studies have been made. So as to cover a representative range of Australian timbers and market situations, these studies have been carried out in co-operation with the Radiata Pine Association of Australia, based at Adelaide; the Queensland Department of Forestry and the Timber Research and Development Advisory Council, both based at Brisbane; and the Timber Promotion Council, based at Melbourne.

Laboratory and mill studies have been undertaken to investigate the following aspects of proof grading timber:

(a) The damage caused by proof testing;

(b) The effect of operator error in single-pass grading;*

(c) The relationship between the proof load in bending and the tension strength and modulus of elasticity of the graded material;

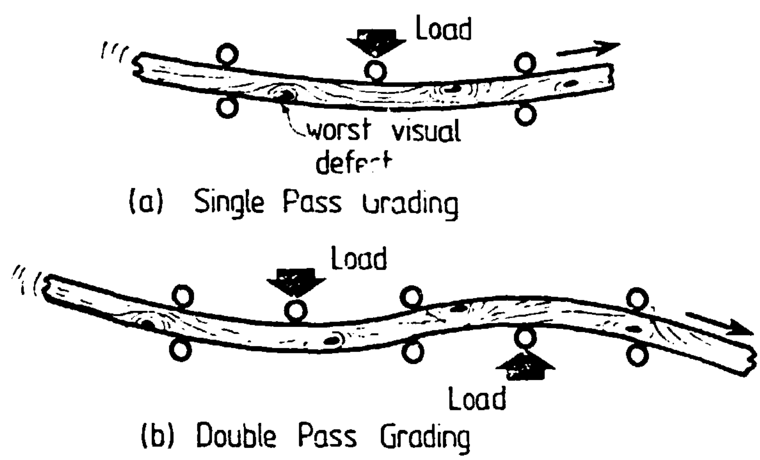
(d) The structural reliability required in the basic design stresses and modulus of elasticity of proof-graded material;

(e) Quality control of grading operations;

(f) The calibration of grading machines;

(g) The commercial viability of proof grading.

Figure 34. Schematic illustration of single- and double-pass grading



As can be seen from the foregoing discussion, there are several ways of grading structural timber, and the number is likely to increase.

C. Continuous proof testing machines

Introduction

With the advent in Australia of commercial proof grading of structural timber [1, 2], experience has been gained in the use of continuous proof testing machines. As a result, it has become apparent that these machines have a potential far beyond that originally envisaged when they were first mooted.

*Differences between machines with single-pass and double-pass grading are shown schematically in figure 34.

Commercial models of continuous proof testing machines are now available. Accordingly, it is opportune to discuss their potential use. The following are four important areas of application: proof grading, the detection of special defects, the in-grade evaluation of structural timber and hybrid grading systems.

1. The commercial machine

The Australian commerical machine was designed by TRADAC, of Queensland, and is constructed by Hilleng, of Brisbane. It applies a continuous bending moment to the timber passing through (figure 35). The loading configuration is shown schematically in figure 36. The timber is loaded on edge, i.e. it is stressed about its major axis. The machine comes with two interchangeable rams, which have load capacities of 20 and 36 kN. The rams are activated by an 800 kPa air pressure source. The feed rate through the machine is about 15 m/min. A limit switch removes the load if the timber breaks or deflects excessively.

Figure 35. The Hilleng proof grading machine

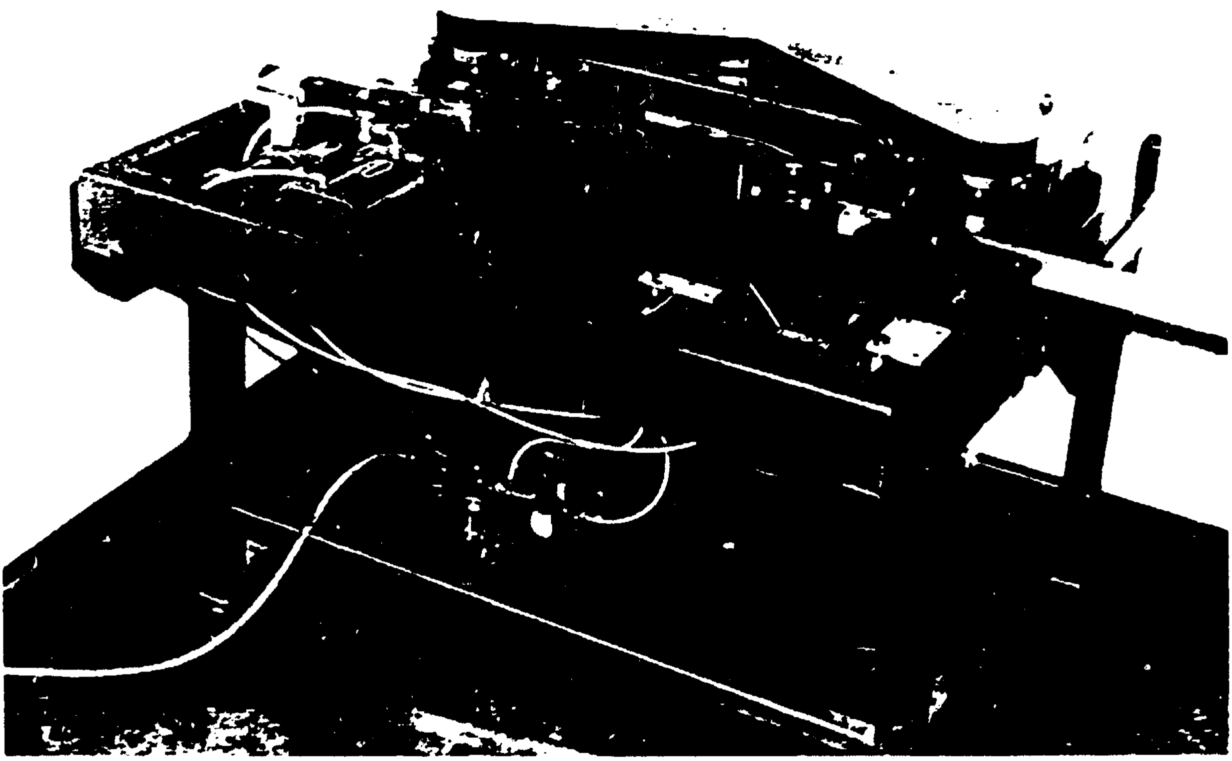
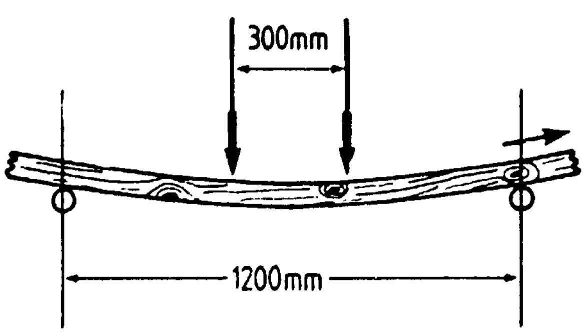


Figure 36. Loading configuration for the Hilleng machine



The machine is sold in two basic models. In the simplest model, an air pressure gauge is used for setting the load. The accuracy of this model under dynamic conditions is about 5 per cent. The cost in 1983 was around $US 12,000. The more sophisticated model uses an electronic load cell to continuously monitor the applied load. Its load measurement accuracy is about 1 per cent and its cost then was around $US 14,000. A read-out device for the load cell, if required, cost a further $2,000. The simple model is suitable for proof grading, while the more sophisticated model is suitable for laboratory work such as in-grade evaluation of structural timber.

The dimensions of the assembled machine are about 2 x 2 x 1.5 m, and its weight is less than 1,000 kg. It is sufficiently small that it can be placed on a truck and transported anywhere in Australia. In fact, the prototype model has been used for demonstrations more than a thousand kilometres away from its base at Brisbane.

2. Advantages of proof grading

Proof grading, details of which are available elsewhere [3], is considerably more reliable than visual grading. It avoids the difficulties in visual grading associated with slope of grain and species identification. The problems associated with species identification are particularly acute where hybrids are involved or where mills are supplied with numerous species. This happens, for example, in northern Queensland, where the mills are typically supplied with more than a hundred species.

In comparison with mechanical stress grading, the advantages of proof grading are its lower initial cost, its low technology and the shorter lead-time for the commencement of a commercial application. A mechanical stress grading system requires extensive laboratory investigations to establish a correlation between strength and stiffness prior to commencing commercial applications; also, the difficulties of accurately measuring stiffness necessitate a tight quality control. By contrast, the assessment procedure and the quality control measures required for proof grading can be carried out fairly rapidly by the proof testing machines themselves.

3. Detection of special defects

Because most methods of grading for strength, such as the visual and mechanical stress grading methods, are based on a correlation between strength and some other parameter, they are not capable of detecting many kinds of highly localized defects, such as poorly glued finger joints, compression shakes, internal decay and internal borer attack. Since a continuous proof testing machine stresses all the timber passing through it, it will fail defective timber and thereby detect any excessively weak material.

In this type of operation, a proof testing machine is used as a detector of defects rather than as a grading machine. The applied loads do not need to be highly accurate for this purpose, so a much simpler machine than the one produced by Hilleng will be adequate. Such a machine can usually be fabricated from parts costing only a few hundred dollars.

It is of interest to note that the first continuous proof testing machine in Australia [3] and possibly also the first one in the United States were developed to detect faulty finger joints.

4. In-grade evaluation of structural timber

It is now generally accepted that it is unreliable to base design recommendations on the properties of small, clear timber specimens unless penalties in the form of large safety factors are used to compensate for the uncertainties involved [4, 5]. Ideally, design strength properties should be based on the five-percentile strength of graded structural-size timber.

Unfortunately, in-grade measurements for this require the testing of large samples of timber. Typically, a sample of some 10,000 sticks may be required to fully evaluate a single species or species mixture. Under normal circumstances this task would take about a year in a typical forest products laboratory. Obviously, then, this form of timber evaluation is not feasible for places such as northern Queensland, where hundreds of species are utilized.

However, in-grade testing for these situations can be made quite feasible through the use of continuous proof testing machines such as the Hilleng machine, described earlier. For this purpose, the proof load is set so as to fail between 1 and 5 per cent of the material passing through. The actual failure load for any particular stick will be indicated by the read-out from the load cell. Even with the relatively slow Hilleng machine, it is possible to test 1,000-2,000 sticks a day in this way.

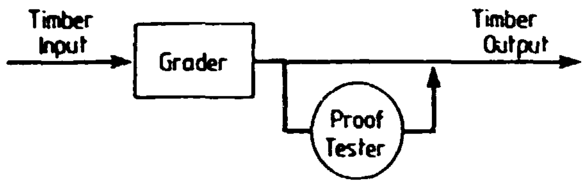
Even where there is only one major species involved, it is desirable to have access to a continuous proof testing machine. This is because the strength properties of the single species will change from time to time owing to changes in the age of the timber cut, changes in silvicultural practices, for instance in pruning or fertilization programmes, and changes in the locations from which timber is cut.

In general terms it may be stated that continuous proof testing machines are invaluable where frequent changes are likely to occur in the structural properties of the timber utilized. Regions or countries that utilize imported timber are particularly prone to rapid changes because of changes in marketing factors beyond their control. Sources of change noted in Australia include the introduction of timber from regrowth forests caused by forest fires, timber from fire-burnt forests, timber from juvenile plantations, timber from borer-infested forests and timber cut from a new species. In such instances, the low cost and short lead-time to utilization associated with a continuous proof tester would make the difference between utilization and non-utilization. For this reason alone, the forestry commission or research laboratory in every region containing production forestry land should own or have access to a laboratory model of a continuous proof testing machine.

5. Hybrid grading systems

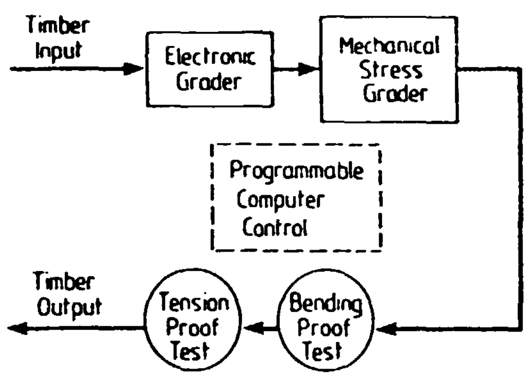
In looking to the future of the use of continuous proof testing machines, one of the most exciting possibilities is that of coupling these machines with other grading machines to form a hybrid grading system [6]. As mentioned earlier, major drawbacks to the use of conventional grading systems such as visual or mechanical stress grading include the long lead-time required to establish the correct grading parameters and the inability of these systems to cope with frequent changes in material characteristics. However, if a proof tester is coupled on-line so as to sample and test the output of a grader, as shown schematically in figure 37, then grading parameters can be devised and checked instantaneously.

Figure 37. A basic hybrid grading system



Possibly the most sophisticated hybrid system that is technically within the bounds of current technology would be one that is formed by coupling a high-speed electronic grader [7] and a high-speed mechanical stress grader [8] with on-line bending and tension proof testing machines, as shown in figure 38. This system would be able to work with complex grading parameters, rapidly evaluating their effectiveness.

Figure 38. An interactive hybrid grading system



A programable computer would be used to interpret the grading parameters measured and to make appropriate decisions concerning the grade of the timber and the type of proof-testing required. The computer control program need not necessarily be pre-set; it could be "learned" by techniques such as those associated with artificial intelligence systems [9].

A hybrid system of this type would represent the ultimate in efficiency and flexibility that is currently possible. It should be able to cope at high speed with that most difficult of grading problems, the grading of mixtures of unidentified, unsorted and untested species.

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IV. MODEL OF THE TIMBER GRADING PROCESS

Robert H. Leicester*

Introduction

In the future, both the quantity and quality of stress-graded lumber may be expected to increase. In addition, there will be a need for a great variety of grading procedures because of differences between mills in number of species processed, volume throughput and access to finance and technology. In order to provide a rational basis for developing suitable grading schemes for the future, a model for evaluating the cost-effectiveness of such schemes is presented. Some prognostications for the future of grading are given.

Stress grading may be defined as the act of sorting lumber into groups, each having a set of specified structural design properties. These design properties may be conveniently subdivided into strength and stiffness properties. To appreciate the following discussion, it is important to realize the significance of reliability in structural design, particularly with respect to strength. Because of this, it is incorrect to associate design properties with individual sticks of timber. Rather, they must be associated with the statistically defined parameters of all lumber contained within a given stress grade.

Stress grading is usually specified as mandatory for timber to be used in engineered structures, e.g. roof trusses. Stress grading is also frequently specified for semi-engineered construction, e.g. timber-framed houses. The attitude of the timber industry to stress grading may be best described as one of mixed enthusiasm, with some segments maintaining a high level of grading and quality control and others virtually ignoring mandatory grading specifications. However, the indications are that grading is finding increasing favour, and future grading will probably be completely different from that of the present.

There are many reasons why both the quantity and quality of stress grading may be expected to increase. The most obvious reason is the growing appreciation of the economies to be obtained through grading. For example, in Australia the strongest grades of timber are about 10 times as strong as the weakest ones [1, 2]. Obviously, in such a situation there is a strong economic incentive to differentiate between the various structural grades of timber, even when the end-use is semi-engineered house construction. The same problem of utilizing a wide range of species exists in most countries with tropical forests [3, 4, 5], and so the use of grading may be expected to become more widespread as timber production in these countries increases.

The advantages to be obtained from stress grading are considerably enhanced if it is used within a strength grouping format [6]. In this format, the design properties of all grades of timber, perhaps numbering several hundred in any one particular country, are replaced by a small set of hypothetical grades. Obviously, this will simplify the design of timber structures and will be particularly useful in the utilization of lesser-known and less-common species. Further benefits are obtained if there is international

*An officer of CSIRO, Division of Building Research, Melbourne.

acceptance of a common strength grouping system. Such an acceptance will assist not only in the trade of structural timber but also in the transfer of technology software, such as standard designs, from one country to another.

With the increased acceptance of stress grading, it may be expected that there will be a corresponding increase in the types of situations in which it is used. For example, the timber throughput, range of sizes and numbers of species handled will vary considerably from one mill to another. Similarly, there will be large differences between mills with respect to the financial capital and technology available. Because of this range of situations, it can be expected that a great variety of grading methods will be employed to maximize the benefits obtainable through stress grading.

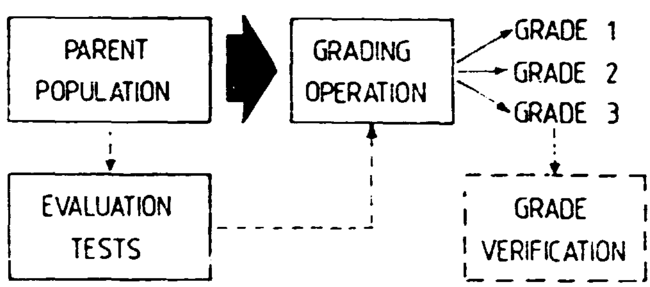
In order to discuss, compare and devise grading methods, it is essential to have universal models of the grading procedure. Suitable models for this do not exist at present, and their development will probably be a focus of research in the near future. The following discussion is based on a simple model that contains the essential features of grading methods.

A. A model of stress grading

1. Concepts

Figure 39 illustrates schematically the basic elements of grading. The essential feature of the procedure is the sorting of a parent population into various grades, such as the grades 1, 2 and 3 shown.

Figure 39. Basic elements of grading



The parent population may be chosen in any way desired. However, once chosen, it must be rigorously defined and observed. Typically, it is a specified segment of the timber fed into the mills in a suitably chosen geographic region over a period of about 10 years. The population may be an identified species, a defined mixture of three to six species or even an unspecified mixture of many unidentified species.

Evaluation tests are used to provide information for setting the parameters in the chosen grading procedure. The statistical characteristics of the structural lumber used for these tests must be representative of those of the parent population. There is a wide range in the types of evaluation tests that are undertaken. As the amount of testing increases, so too does the reliability of the knowledge on lumber properties, and the grading can be made more efficient. The following are typical examples of evaluation tests, presented in order of increasing efficiency and cost, that may be undertaken for a population that has not previously been stress-graded:

(a) No tests;

(b) Measurement of population density only [7];

(c) Measurement of bending and compression strength of 10-50 small, clear timber specimens [7];

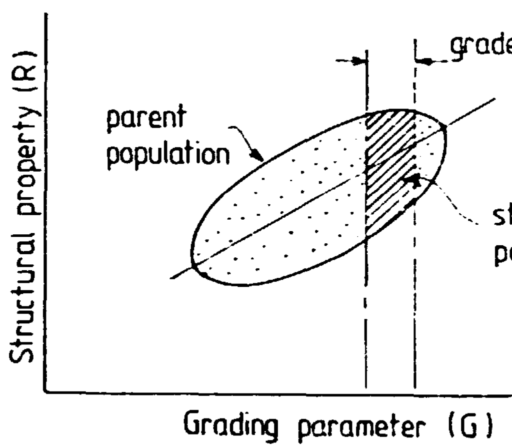
(d) Measurement of bending strength and stiffness of 30-200 sticks of structural lumber [8];

(e) Measurement of bending strength, tension strength and stiffness of 1,000-10,000 sticks of structural lumber.

Because the grade verification process, sometimes referred to as in-grade testing [9], is expensive and time-consuming, it is not always applied. It is most commonly applied to resolve doubts that a particular grading procedure is producing the stress grade that is claimed. Thus, the process helps to mitigate the conflicts inherent in marketing competing species or competing grading systems.

Many structural properties (strength is one) cannot be measured directly during grading. Hence the grading operation, illustrated in figure 40, is based on the correlation that exists between the structural property of interest, denoted by R, and a grading parameter, denoted by G.

Figure 40. The grading operation

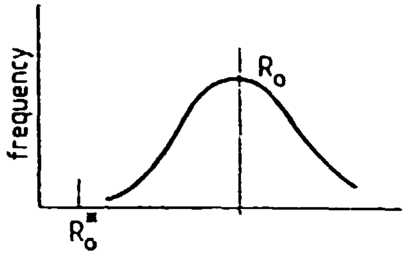


2. Statistical characteristics of grading

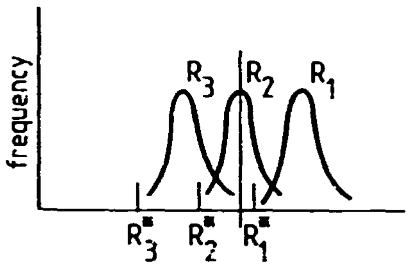
Because of the great range of practical situations encountered, no single grading system can be recommended for universal application. In order to choose the most cost-effective system for a particular situation, a mathematical model of grading is required that assesses the influence of various parameters. The following discussion is based on the use of such a model.

In this model of stress grading, shown in figure 41, the statistical properties of the parent population, denoted by R_0, are distributed among three populations, denoted by R_1, R_2 and R_3, which comprise roughly three quarters of the total parent population. The design values of R for the three stress grades, R_1*, R_{2*} and R_3*, are also shown in the figure.

Figure 41. Distribution of a structural property



(a) Parent Population



(b) Graded Population

The ratio between the design strengths of these populations, which is of some relevance to marketing the structural lumber, is roughly given by

$$R_1^*/R_2^* \cong R_2^*/R_3^* \cong \exp(0.6rV) \tag{1}$$

where r is the correlation coefficient between G and R_0 and V is the coefficient of variation of R_0.

A measure of the average effective increase in design property due to the grading operation is given by the ratio R_2^*/R_o^*, which may be roughly computed from

$$R_2^*/R_o^* \cong \exp[0.6\beta V(1 - \sqrt{1 - 0.95r^2}) - (V/\sqrt{N})\sqrt{1 + 0.2\beta^2}] \tag{2}$$

where R_2^* is the design value of R_2 based on an evaluation with a sample of size N, R_o^* is the nominal ideal design value of the parent population based on an evaluation with an infinite sample and β is a safety index [10, 11]. Typical parameters for strength properties are V = 0.5, β = 3 and for stiffness properties, V = 0.2, β = 1. The effects of the correlation coefficient r and sample size N are shown in figures 42, 43 and 44.

Figure 42. Separation of grade properties

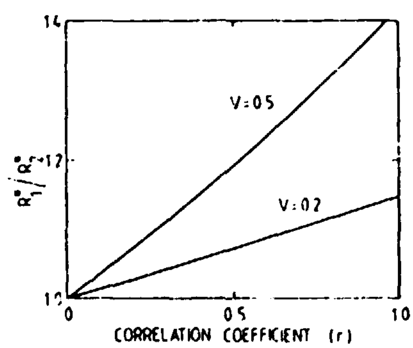


Figure 43. Increase in design values

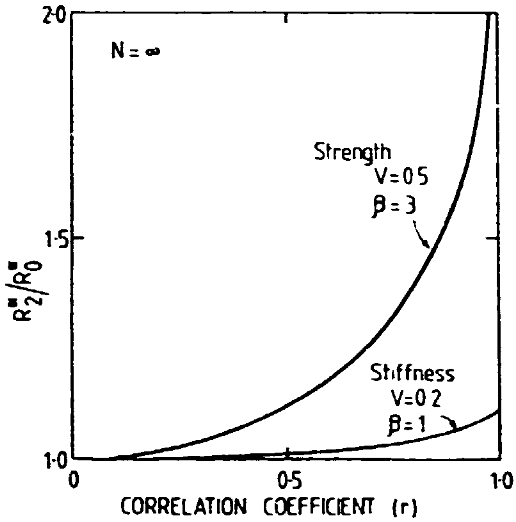
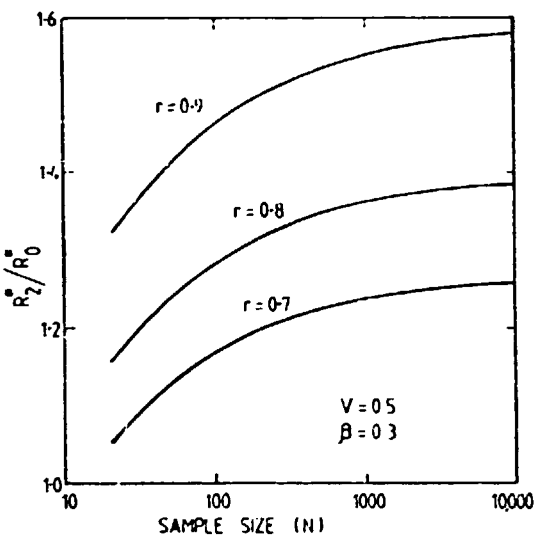


Figure 44. Effect of sample size in evaluating tests



After evaluating R_2*/R_o* from equation (2), the cost C_G of building with graded timber relative to C_U, the nominal cost of building with well-sampled but ungraded timber, may be obtained from

$$C_G/C_U \cong (1 + C_g)(R_0*/R_2*)^{0.8} \tag{3}$$

where C_g denotes the cost of grading relative to the initial cost of ungraded timber.

For practical purposes, two refinements of equation (2) may be required. To account for errors in judgement by the grader, the parameter r is replaced by rr_e, where r_e denotes the correlation coefficient between the true grading parameter and the apparent value measure by the grader. In addition, a major modification to equation (2) is required if the evaluation procedure does not include tests on structural lumber of the parent population concerned.

In this situation, the grading operation is based in part on tests with structural lumber of other populations. As a result, it can be shown that the ratio R_2^*/R_c^* depends also on V_A, the coefficient of variation of A, and r_A, the correlation coefficient of A with the grading parameter G, where A is a population made up of the mean values of R from various parent populations.

B. Summary of grading methods

1. Visual grading

Visual grading is the oldest and most widespread grading method in use. The grading parameter is the size of visible defects, used either on its own or combined with some other parameter such as clear material strength. The quantification of this parameter is specified in a variety of ways. Some are relatively complex, such as the knot area ratio method [12], which notionally involves an X-ray picture of the defects. Others can be very simple, such as that specified for heart-in studs of *Pinus radiata* [13], which considers only the defect size along the edge of the lumber.

2. Mechanical stress grading

The grading parameter used in this method is the modulus of elasticity. The particular modulus measured may be E_{min}, the minimum local modulus along the length of the stick of lumber; E_{av}, the modulus averaged along the stick; or a combination of the two [14].

Most of the available commercial machines make static measurements of E_{min}. Some, such as the Plessey Computermatic [15], are expensive, continuous-measurement machines capable of high throughput rates. Others, such as the TRU Timber Grader [16], are low-cost, low-throughput machines that make spot measurements of the modulus. Machines that make dynamic measurements of the modulus usually measure E_{av} [17, 18].

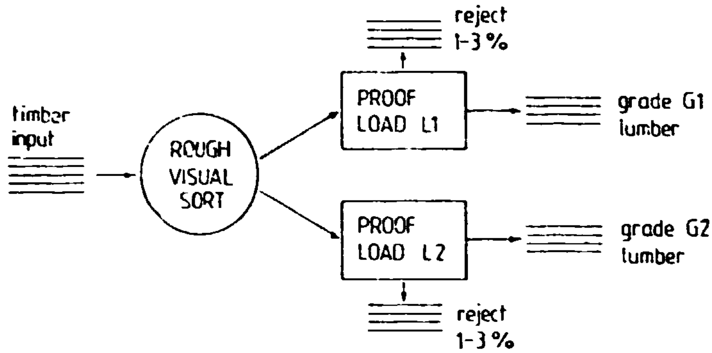
3. Electronic grading

High-speed, non-contacting electronic graders have recently become available. One example is the Finnograder [19]. These measure simultaneously several lumber characteristics such as density, moisture content, knot size and location, and slope of grain. Through the use of microcomputers, very complex and hence potentially very efficient grading parameters may be employed.

4. Proof grading

Proof grading [20, 21], recently introduced in Australia, is illustrated schematically in figure 45. The lumber population is first sorted roughly, usually by rapid visual scanning, into two or more grades. Each grade of lumber is then passed through a continuous proof testing machine that culls out exceptionally weak pieces of timber. In a typical grading operation, the proof load is chosen so that 1-3 per cent of the timber fails and is rejected. Thus the grading parameter in this procedure is the bending strength at the tail of the graded population of lumber.

Figure 45. The proof grading procedure



The proof grading system was developed for application in a situation where there is little available capital and technology.

C. Cost of grading

Apart from one example for a high-speed mechanical stress grader [22], there is little detailed published information on the cost of grading timber. The cost of a grading machine with a throughput rate of 200 m/min is about $US 100,000 and that of one with a throughput rate of 20 m/min is about $US 10,000. Typically, two men are required to run a grading operation.

Table 2 gives a rough estimate of some cost components for machine grading in Australia. The costs associated with visual grading are similar to those for a machine with a throughput of 20 m/min. To compute realistically the added cost of grading, additional information is required on matters such as the amortization period, discounted cash flow and inflation rates and the number of evaluation tests proposed. Typical vales of the relative grading cost parameter C_g are 1-10 per cent, with the smaller costs being associated with higher throughput grading machines.

Table 2. Some cost components of machine grading

	(Cost x 100)/(annual retail value for one year's throughput of timber graded by the machine)		
Machine throughput rate (m/min)	Initial machine cost	Annual running costs	Evaluation tests on 10,000 sticks
200	0.6	0.6	0.6
20	0.6	3.0	6.0

D. Choice of grading method

It is outside the scope of this paper to discuss the application of grading schemes for the many possible mill situations. However, the matters that need to be considered in the choice of a scheme will be discussed.

Equations (2) and (3) provide a useful basis for decisions related to grading timber. To use these equations, the coefficients V and r of the parent population must be estimated. There is enough information available in the published literature to provide rough estimates of these parameters. Some of this information is given in the literature cited at the end of this paper [14, 17, 23-27]. For strength properties V = 0.3-0.6, typical values of r for various grading parameters are as follows:

Visual strength ratio	r = 0.6
Electronic strength ratio	r = 0.7
Modulus E_{min}	r = 0.85
Modulus E_{av}	r = 0.5

The above are only indicative, and true values of r can vary considerably from one parent population to another. For example, for *Pinus radiata*, a value of r = 0.80 has been measured for lumber cut from mature forests [26], while a value of r = 0.35 has been obtained for immature forests [28]. Finally, some estimate must be made of r_e, the measure of error in the operational measurement of the grading parameter G. Only limited information is available on this aspect [21, 29]. Once the most suitable grading parameter G has been found, a machine is chosen to match the throughput rate of the mill, and N, the number of evaluation tests to be undertaken, can then be optimized.

Although equation (3) provides a useful starting point for assessing the economic feasibility of a grading scheme, several other matters frequently dominate the choice of grading method. One is the existence of grades that are already available on the market in wood of a competing species. This will produce a bias in the market value of specific grades. Another aspect is the severe constraints on choice imposed by limitations on the availability of technology and financing.

Finally, a factor that is sometimes not anticipated is the effect of start-up time. For example, a typical small timber laboratory with three operating personnel will complete evaluation tests on some 10,000 sticks of structural lumber per year. With this restriction on the evaluation rate, any grading procedure that is tied to extensive structural lumber testing will not be feasible for mills that process several hundred species.

E. The long-term future

As mentioned earlier, the prognostication for the immediate future is for an increase in the quantity of timber to be graded and for a greater variety of situations in which grading will be undertaken. There will probably also be a more systematic approach to the assessment and development of grading methods by the use of models such as the one sketched in this paper. Over the longer term, it is probable that the grade verification procedure will be highly codified and rationalized to ensure that the design strength characteristics of timber have the same reliability as those of other structural materials. Furthermore, because of their importance, the various grade verification procedures are likely to be co-ordinated internationally by the International Organization for Standardization. There is a possibility that this will be done within a strength grouping format.

Because there will be an increased demand for grading under a wide range of resource and marketing situations, there will also be a wide range of grading systems. Based on the mathematical model discussed in section A.2, the following predictions may be made of the probable characteristics of future grading systems for three interesting situations:

(a) Where few species and large volumes are involved, such as occurs with plantation-grown material, the grading systems will be undertaken by electromechanical methods characterized by high throughput rates and complex but efficient grading parameters. Extensive evaluation testing will be undertaken, perhaps even on an annual basis, to ensure that the grading parameters used are the best ones possible;

(b) Where multiple-species forests are the source of raw material for mills that have easy access to finance and highly developed technology, the grading system will again involve high throughput machines with the ability to operate with complex grading parameters. However, because of the numerous species involved, many of which will be unidentified or available only in small volumes, even a modest amount of evaluation testing for each species may not be economically feasible. Consequently, the grading parameters will probably be evaluated by on-line proof testing facilities. These facilities will need to be capable of applying bending and tension loads and will need to have rapid response characteristics so that the applied load may be changed from one stick of lumber to the next;

(c) Where multiple-species forests are associated with mills that have poor access to finance and technology, the most appropriate grading system will be proof testing based on a simple bending test. Such mills will produce a very limited set of grades, possibly only one for each group of similar species.

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V. VISUAL GRADING OF TIMBER

J. Hay*

Introduction

It is important with any visual grading process that the method of measuring and assessing defects is consistent. This means not only that a single grader should be able to obtain almost the same result on re-grading, but also that any difference between two graders is marginal. To achieve this requires training to explain the grading terms, the methods of measurement, the relative importance of the various defects and the way in which the results are recorded.

A. Training material

The following figures and accompanying descriptions have been used with considerable success in the training of graders at Victoria. They are taken from the Visual Stress Grading Manual of the Timber Promotion Council, by permission of the Council.

1. Timbers in general use

The three main varieties of timber used in the Australian building industry are eucalyptus, radiata pine and Oregon (Douglas fir). Eucalyptus is the generic or family name for a large number of Australian timber species. As these species usually occur in random mixtures, depending on geographical location, timber produced for the building industry from them is generally not classified into individual species. Radiata pine and Oregon are ordered and produced according to species.

Table 3 lists, by species, the various timbers used throughout Australia and the strength and stress characteristics determined for each. These definitions relate to various standards in use throughout Australia, which are subject to occasional amendment. Data obtained from the table must not be considered final. Always refer to the prevailing local standard.

2. Description of a log cross-section

The cross-section of a typical log is shown in figure 46. The amount of sapwood, truewood or heart in a piece of timber depends on the size of the original log and the position within the log from which the piece was cut. A piece of timber may be cut from a log by backsawing or quartersawing.

Sapwood is composed of living wood tissue. It forms the outer layers of a tree trunk and stores and conveys food material within the tree. Truewood is wood tissue from which food material has been removed or converted to hard tissue. It constitutes the bulk of the tree trunk and lies under the sapwood. Heart is the material in the centre of the trunk. It is composed of brittle dead wood tissue and is surrounded by the truewood.

*Co-ordination of Timber Industry Courses, Dandenong College of Technical and Further Education, Victoria.

Table 3. Relationship between visual structural grades and stress grades for common structural timbers a/

Species	Australian Standard	Strength group: Unseasoned	Strength group: Seasoned	Visual Grade b/ — Unseasoned: Str. No. 2	Unseasoned: Str. No. 3	Seasoned: Str. No. 2	Seasoned: Str. No. 3	Seasoned: Str. No. 5	Lyctus susceptibility c/
				Stress grade					
Unidentified Australian hardwoods from Victoria, Tasmania and New South Wales highlands d/	2082	S4	SD4	F11	F8	F17	F14		S
Ash, alpine	2082	S4	SD4	F11	F8	F17	F14		R
Ash, mountain	2082	S4	SD3	F11	F8	F22	F17		R
Ash, silvertop	2082	S3	SD3	F14	F11	F22	F17		R
Box, brush	2082	S3	SD3	F14	F11	F22	F17		R
Box, grey	2082	S2	SD2	F17	F14	F27	F22		R
Gum, blue, southern	2082	S3	SD2	F14	F11	F27	F22		S
Gum, grey, mountain	2082	S3	SD2	F14	F11	F27	F22		S
Gum, Manna	2082	S4	SD4	F11	F8	F17	F14		S
Gum, red, river	2082	S5	SD6	F8	F7	F11	F8		S
Gum, shining	2082	S4	SD4	F11	F8	F17	F14		S
Gum, spotted	2082	S2	SD2	F17	F14	F27	F22		S
Ironbark, red	2082	S1	SD3	F22	F17	F22	F17		S
Jarrah	2082	S4	SD4	F11	F8	F17	F14		R
Mahogany, southern	2082	S2	SD3	F17	F14	F22	F17		R
Messmate	2082	S3	SD3	F14	F11	F22	F17		S
Peppermints, var.	2082	S4	SD4	F11	F8	F17	F14		S
Pine, radiata e/	1490		SD6				F8	F5	R
Stringybark, brown	2082	S3	SD3	F14	F11	F22	F17		R
Stringybark, red	2082	S3	SD4	F14	F11	F17	F14		S
Stringybark, white	2082	S3	SD3	F14	F11	F22	F17		R
Stringybark, yellow	2082	S3	SD3	F14	F11	F22	F17		R
Tallowwood	2082	S2	SD2	F17	F14	F27	F22		S

continued

Table 3 (continued)

Species	Australian Standard	Strength group Unseasoned	Strength group Seasoned	Visual Grade b/ Unseasoned Str. No. 2	Unseasoned Str. No. 3	Seasoned Str. No. 2	Seasoned Str. No. 3	Seasoned Str. No. 5	Lyctus susceptibility c/
				Stress grade					
Kapur	2082	S3	SD3	F14	F11	F22	F17		R
Kwila (Merbau)	2082	S2	SD2	F17	F14	F27	F22		S
Fir, Douglas	2440	S5	SD5	F11	F8	F7	F5		R
(Oregon) f/				Sel. eng.	Std. eng.	Sel. mer.	Mer. mer.		
Hemlock, western	2440	S6	SD6	F8	F7	F5	F4		R
(hem-fir etc.)				Sel. eng.	Std. eng.	Sel. mer.	Mer.		

a/ Stress grade is a strength classification for structural timbers, derived by means of either visual or mechanical grading in accordance with the relevant Australian Standard. The current standards for visual grading of softwoods use differing systems of grade classification. For simplicity, the structural grades of softwoods have been classified here in the same way as hardwoods. The structural grade No. 5 shown is a theoretical grade only and has been included to assist this classification. The grade names for Oregon and western hemlock have also been included.

b/ The following abbreviations are used: Str. No., structural grade; Sel. eng., select engineering grade; Sel. mer., select merchantable grade; Std. eng., standard engineering grade; Mer., merchantable grade.

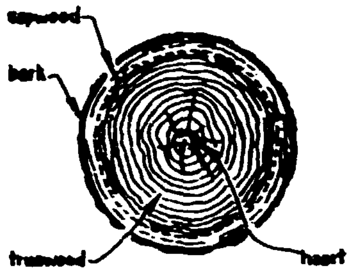
c/ Lyctus susceptibility: R = resistant, S = susceptible.

d/ Hardwoods are usually marketed as a mixed species and only in special circumstances can particular species be obtained. Supply availability should be determined before design documentation or specification.

e/ In seasoned condition, as required by AS 1490 (grades apply to treated and untreated radiata pine).

f/ Grading applies to Oregon from the West Coast of North America.

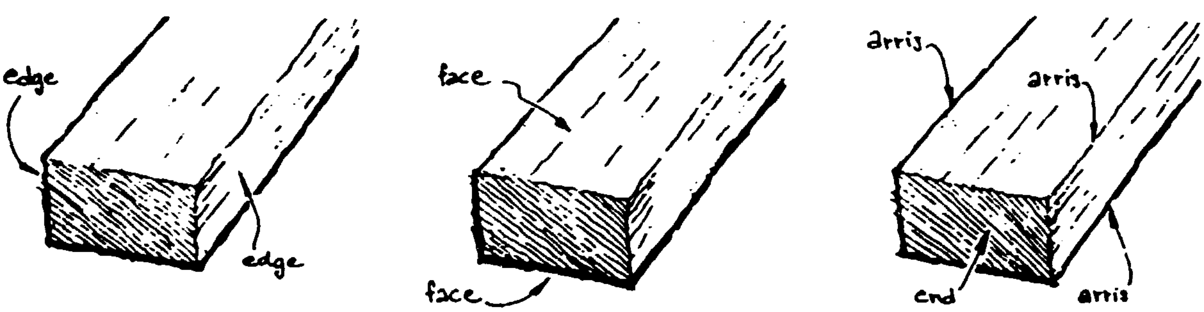
Figure 46. Log cross-section



3. Timber terminology

Each piece of timber has two faces, two edges and two ends (figure 47). Its cross-sectional area is obtained by multiplying the width of one edge by the width of one face. The edges are the two narrow surfaces of one piece of timber. They extend throughout its length. The faces are the two wide surfaces of the piece of timber. They extend throughout its length. A piece of timber may contain sapwood, truewood or heart, or combinations of these. The ends are the two remaining surfaces. They extend across the width of the piece of timber. An arris is the boundary between an edge and a face. Each piece of timber has four arrises.

Figure 47. Edges, faces and ends



4. Visual stress grading

Visual stress grading is a technique for grading timber to meet building standards, without the use of mechanical equipment. Success depends on careful visual inspection of each piece of sawn timber, recognition of any defects within that piece and evaluation of their significance with reference to the appropriate standard.

The grading of timber cannot be considered an exact science because it is based on a visual inspection of each piece and thus relies to an extent on the judgement of the grader. These rules provide for a 5 per cent variation between graders. While definite and complete descriptions of timber quality are usually given, it is not feasible or practicable to be fully explicit in every respect, but defects capable of physical measurement are judged solely on the basis of their dimensions.

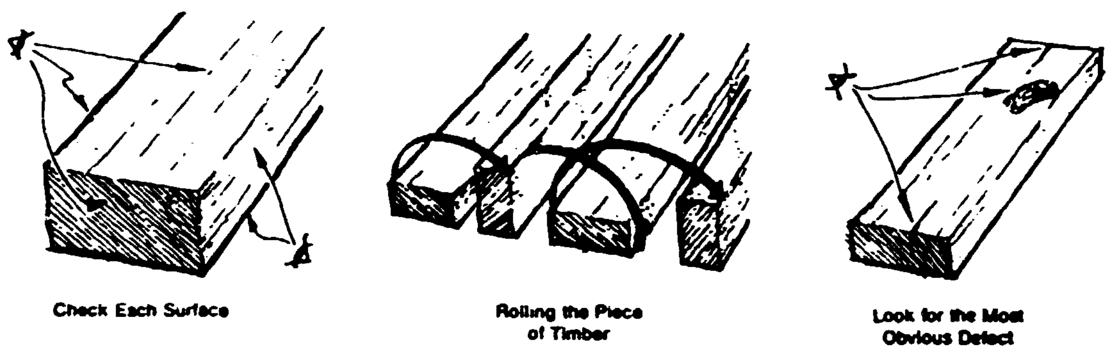
Grade descriptions based upon the poorest pieces allowed in each grade are desirable in the interest of keeping the rules simple and facilitating the work of the grader. As a result, the average quality in any grade is far better than the minimum described.

The stress grader needs a straight edge, a steel tape (calibrated in millimetres) and a strong pocket knife.

Handling the piece of timber

Both faces, both edges and each end of the piece of timber must be inspected. This usually involves inspection from one end and rolling of the piece to allow the length of each edge and face to be examined (figure 48).

Figure 48. Steps in the visual stress grading process



Inspection

Inspection of a piece of timber should be thorough, but for reasons of economy it must be performed as rapidly as possible. The inspector must roll the piece, check the edge and face widths, then look for the most obvious defect. If this is sufficient to reject the piece, any other defects present are ignored.

Rework of many rejected pieces is possible. This is noted in accordance with mill practice and the next piece is inspected without delay.

When in doubt, the piece is stored in a separate stack for future re-evaluation. The supervisor may be asked for assistance during this re-evaluation exercise.

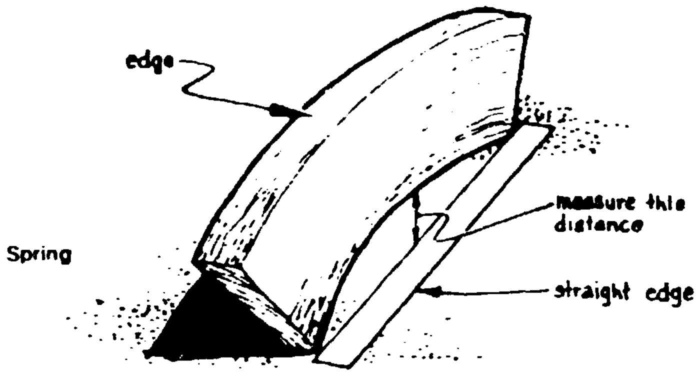
5. Defects

Many of the defects described require measurements to be taken during evaluation. When the grader has had some practical experience it should be possible to judge most defects accurately by eye without measurement. The attainment of this ability should be the goal of the trainee.

Spring

Spring is curvature along the edge of a piece of timber (figure 49). To measure, lay the concave edge of the timber against a straight edge, measure the greatest distance between the straight edge and the face with a tape. When related to the dimensions of the piece, this measurement will indicate acceptability of the spring. With practice, spring may be accurately checked, without measurement, by sighting along the concave arris.

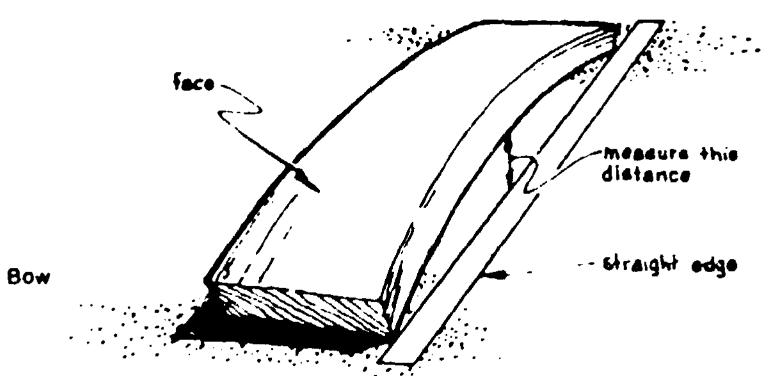
Figure 49. Spring in a piece of timber



Bow

Bow is curvature along the faces of a piece of timber (figure 50). It is measured in the same way as spring and, with practice, may also be accurately checked, without measurement, by sighting along the concave arris.

Figure 50. Bow in a piece of timber



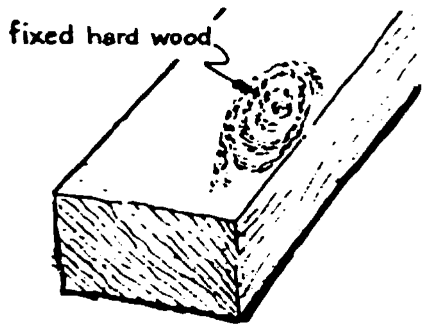
Knots

Knots are inclusions of branch growth sections in timber pieces. There are several types of knot, and the method of evaluation of each type differs. It is important that the timber grader is able to recognize each type. The acceptability of each type of knot is calculated on the basis that the knot area is a loss of area of the piece, regardless of the condition of the knot. That is, the knot is treated as if it were a hole. The acceptability of knots may vary between the three general species - eucalyptus, radiata pine and Oregon. The timber grader should check the standard for the species before grading is commenced.

Sound, tight knots

The material contained within a sound, tight knot (figure 51) is hard, free from decay and firmly fixed within the piece of timber.

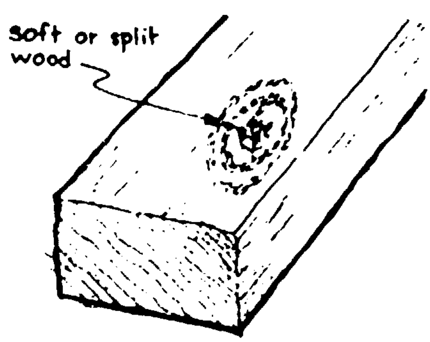
Figure 51. Sound tight knot



Unsound knots

The material contained within an unsound knot (figure 52) may be decayed or softer than the surrounding material of the piece. It may also be chipped, split or loose.

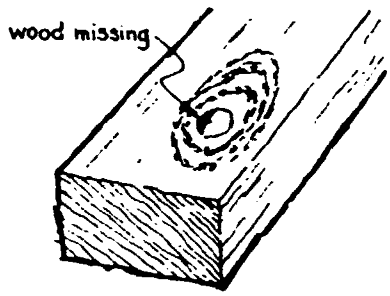
Figure 52. Unsound knot



Knot-holes

Knot-holes occur where the material originally contained within the knot has been dislodged from the piece of timber (figure 53). Sloping grain surrounding a knot may be ignored. The effect of this is taken into consideration when the standards for knot acceptability are determined.

Figure 53. Knot-hole



Arris knots

Arris knots (figure 54) are knots that extend across a face and an edge of a piece of timber. They include a section of the arris in their area. Depending on species, arris knots are measured differently (figure 55). In hardwood, they are measured by determining their width as for face or edge knots but on the face or edge of least effect. Where they occur in Oregon or radiata pine, the width of the arris knot is measured on the face and the edge. These two measurements are then multiplied together and the figure obtained related to the cross-sectional area of the piece to determine acceptability.

Figure 54. Arris knot

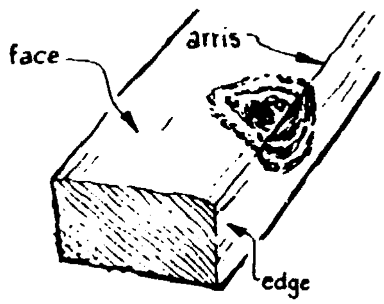
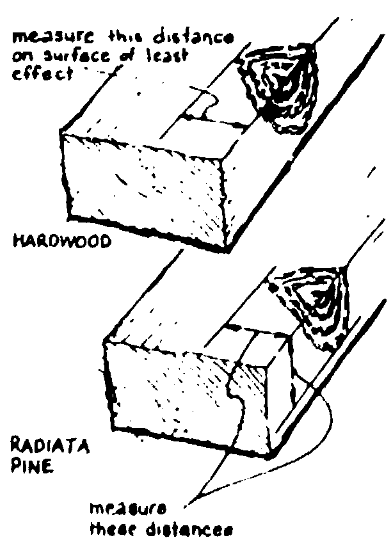


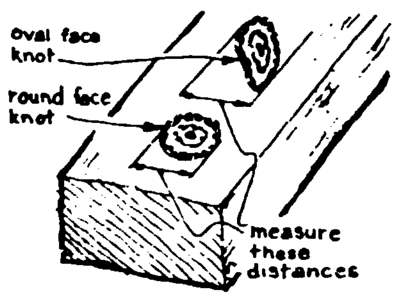
Figure 55. Measurement of arris knots



Face knots (round or oval)

Face knots are knots or knot-holes that occur on the face of a piece of timber. Measure face knots by noting their greatest width across the face of the piece (figure 56). This measurement, when related to the face width, determines the acceptability of the knot.

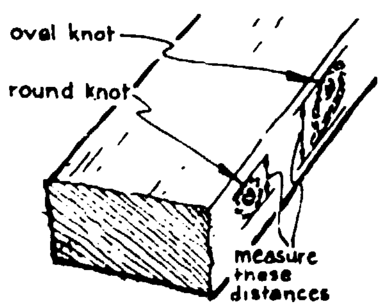
Figure 56. Measurement of face knots



Edge knots (round or oval)

Edge knots are similar to face knots. They occur on the edge of a piece of timber. They are measured in the same way as face knots, and their width in relation to the width of the edge determines their acceptability (figure 57).

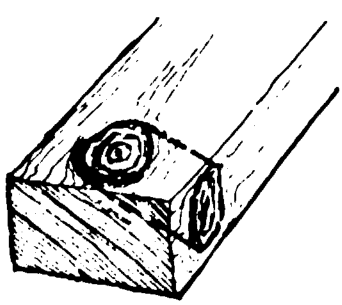
Figure 57. Measurement of edge knots



Through knots

Through knots are knots extending through a piece of timber between an edge and a face (figure 58). The diameter of the knot is measured on the face or edge of greatest effect.

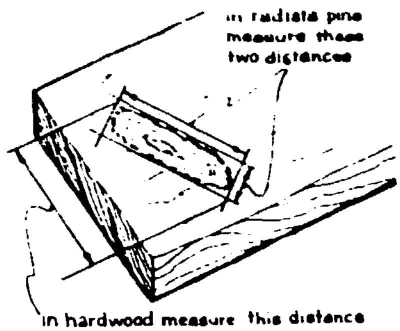
Figure 58. Through knots



Spike knots

Spike knots (figure 59) are elongated knots with a spike-shaped area. In hardwood, the width between the arrises is measured. In radiata pine, their size is determined as follows: (length + width)/2. This size is then related to the width of the face or edge where the knot occurs to determine acceptability.

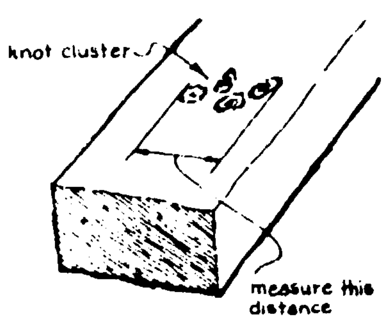
Figure 59. Measurement of spike knot



Knot cluster

A knot cluster (figure 60) contains two or more knots within a single area of deflected grain. The overall width of the cluster is measured and related to the width of the face or edge where it occurs to determine acceptability. Knot clusters are usually only found in radiata pine or Oregon.

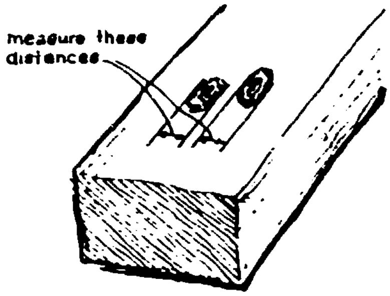
Figure 60. Measurement of knot cluster



Knot groups

A knot group (figure 61) is formed by two or more closely spaced individual knots, each knot being bounded by deflected grain. The aggregate width of the knots within the group is related to the width of the face or edge on which it occurs to determine acceptability. Knot groups are usually only noted in radiata pine or Oregon.

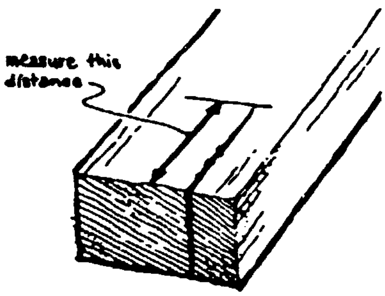
Figure 61. Knot group measurement



End splits

End splits are open cracks between the faces or edges of the piece (figure 62). They are caused by separation of the wood fibres and extend along the piece from the ends. Inspect each end of the piece for end splits and measure the length of any splits.

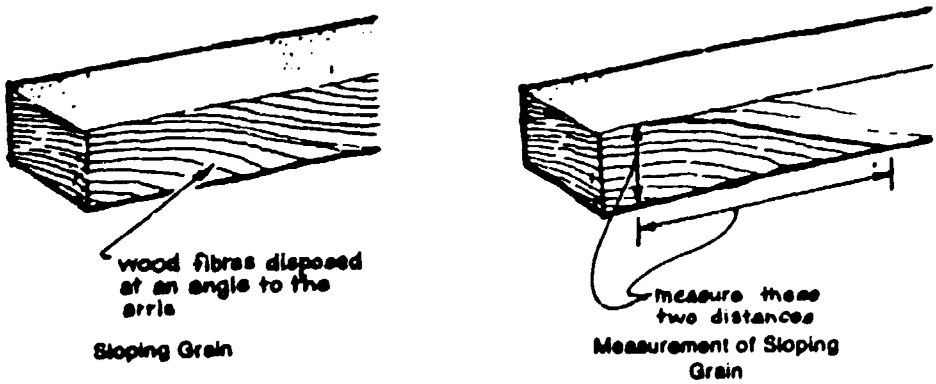
Figure 62. End split



Sloping grain

Sloping grain is the presence of wood fibres at an angle to the arris (figure 63). The general slope of the fibres is important, and local deviations are disregarded. It is measured and limited at that point in the length that shows the greatest slope and is expressed as a ratio such as "slope of grain 1 in 15".

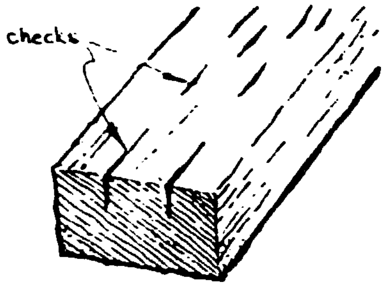
Figure 63. Sloping grain and the measurement thereof



Surface checks

Checks are shallow cracks extending along the grain on a face or edge (figure 64). Checks are usually short in length and do not extend between the faces or edges.

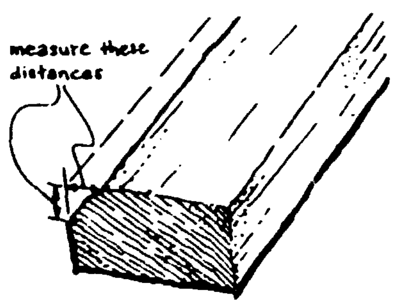
Figure 64. Checks



Want

Want describes wood missing from an arris, edge or face of a piece of timber (figure 65). It may be caused by a piece splitting off, or it may result from a sawing defect. Want is measured as a face and edge width deficiency.

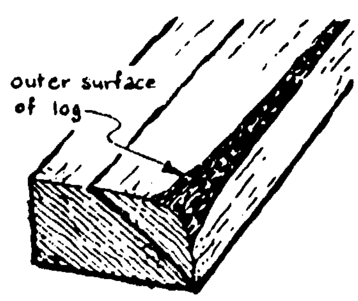
Figure 65. Want



Wane

Wane is the natural absence of wood from an arris or surface produced by cutting the piece from the outer surface of a log (figure 66). The wane area includes a portion of the log's outer surface and possibly also some bark. Wane is measured in the same manner as want. Wane in a piece of hardwood indicates the presence of sapwood. The sapwood must also be evaluated.

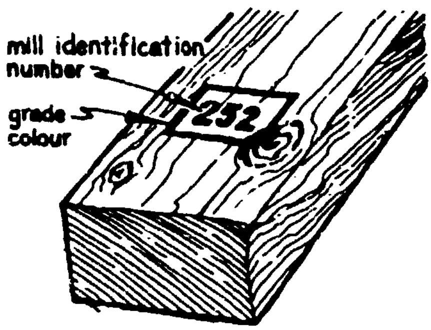
Figure 66. Wane



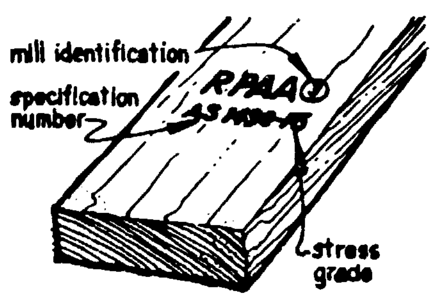
6. Marking

Timber that has been visually or mechanically stress graded must be marked, usually by colour, with an indication of its stress grade. Colour coding for stress grade is as follows: F4, red; F5, black; F7, blue; F8, green; F11, purple; F14, orange, F17, yellow; and F22, white. The marking is usually made on a face near one end of the piece. The particular mark to be applied is dictated by the prevailing standards. It may include other information to indicate species and a mill identification code number (figure 67). These marks are noted during examination of building frames by building inspectors.

Figure 67. Typical marks



Hardwood Marking

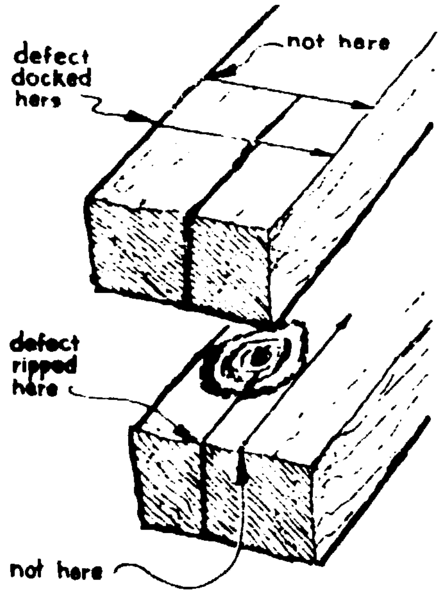


Radiata Pine Marking

7. Reworking

Timber that fails to meet requirements during stress grading must be reworked whenever possible. The grader should indicate what type of rework is required and note this in accordance with mill practice. When considering rework, the grader must have a clear understanding of the permissible extent of a defect within the grade. For example, in the case of an end split that exceeds the length limit imposed by the prevailing standard, it would be wasteful to dock the entire portion of the length of timber affected by the end split. Only that length of the split that exceeds the grade limit should be docked. With practice, a competent grader is able to economically specify rework of all defects in this manner and keep timber wastage to an absolute minimum (figure 68).

Figure 68. Rework within tolerance

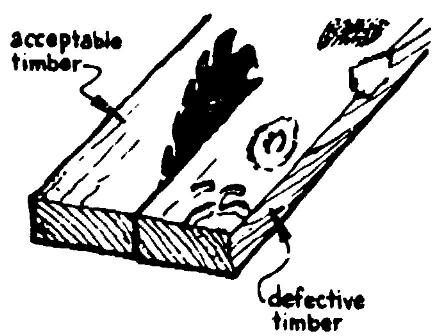


There are two main types of rework: ripping and docking.

Ripping

Where a defect such as sapwood extends along a major portion of the length of a piece but only across part of its width, it may be possible to rip the length of timber, remove the defective portion, and produce a narrower piece that meets grade requirements (figure 69). To judge if ripping is feasible, the grader must decide whether the sound portion of the piece can be ripped to a standard size.

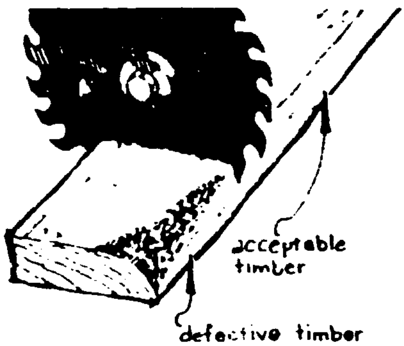
Figure 69. Ripping



Docking

Where a defect affects only a portion of the length of a reject piece of timber, for example an end split, gum pocket or knot, it may be possible to dock the defective portion to produce one or more shorter lengths of timber that meet grade requirements (figure 70). Shorter lengths may be utilized as noggins or finger-jointed where permitted.

Figure 70. Docking



B. Principal timber defects

1. Gum

Gum is a common defect in Australian hardwoods. It is formed in the growing tree when some part is injured by fire, insects or mechanical agency. It is deposited like a shield to cover the injury. Gum attributable to fire may form around the growth ring or a substantial part of the ring. Gum attributable to insects and mechanical agency may be limited to a cone around the affected position. Gum rings may be evident on the cross-cut end of a log, but there may be few other external signs. In sawn and hewn timber, gum becomes evident as veins on quarter-cut faces and as smears, streaks or splashes on backsawn faces, and gum pockets may occur at random. Veins may be distinguishable as "tight", in which case the gum is bridged at close intervals by woody tissue, or "loose", in which case there is evident separation of the wood elements.

Gum is more noticeable in pale timbers than in dark timbers and is often considered unsightly in its predominating occurrences. Tight gum veins do not adversely affect structural timber. Gum pockets and loose veins are described and limited by width and length. It is important to remember that gum veins and pockets are measured radially for width, that is at right angles to the growth rings.

2. Knots

Knots are a section of the original branch of the tree. They are referred to by various names denoting the condition of the branch at the time of conversion, the shape of the knot or the condition of the knot after sawing and planing. Although knots do not occur in Australian hardwoods as often as they do in softwoods, they are an important defect to be considered.

The effect of a knot on the strength of wood in tension and bending is twofold. Firstly, the knot itself makes no contribution to the overall strength of the wood and therefore represents a loss of cross-section, that is the knot acts as if it were a hole. Secondly, and more commonly, every knot causes a swirl or deviation of the grain acting as cross-grain, particularly if the knot is on an edge. The cross-grain can be extremely severe and its reduction of strength very great. In the case of hardwood building scantling, two types of knot are of concern: the face knot, which occurs on the wide face, and the arris knot, which breaks the arris and occurs on the face of the edge or on the edge only. Sizes of knots are judged on diameter, with the distance between lines touching their edges drawn parallel to the edge of the piece.

3. Sloping grain

When timber is used for load-bearing purposes, advantage is taken of the fact that it is stronger along the grain than across it. Where the load is applied at some angle to the grain, a lesser strength must be allowed for. Sawmillers have to accept practically the forest-run of logs, so that in many pieces of sawn timber the direction of grain is not parallel with the length of the piece. Some account must be taken of the angle which the grain makes with the length. Where the angle is small, the effect on strength may be ignored. Where it is large, the timber should not be used for structural purposes.

For many practical purposes it is desirable to set limits on the permissible slope of the grain. Various grading rules impose limits consistent with the structural sufficiency of the grade. In ladder rungs, sporting goods and tool handles, the limit may be 1 in 20. Straight grain is important in barrel staves and in timber that is to be bent after steaming.

Sloping grain is not always easily detected. Growth rings are not reliable indicators of the grain slope. In fact, timber may easily be sawn from a spiral grain log in such a way that the growth ring pattern on each face appears to be parallel with the length of the piece.

For detecting sloping grain. there are three principal methods:

(a) Observe the direction of the seasoning checks that are usually evident in sawn timber; these checks follow the grain, and its direction can be found accurately by examination;

(b) Split a portion off the piece (splits follow the grain) or lift a small sliver from the piece with a knife or chisel and note the direction in which the wood splinters strip;

(c) Use a scribe.

Trade Circular No. 48 of the CSIRO, Division of Building Research, "Sloping grain in timber", gives further information on this subject and includes instructions for measuring. The above methods are for detecting sloping grain if the grader is in doubt. Much of the sloping grain encountered is recognizable to people used to handling timber. Sloping grain associated with knots does not have to be considered as a separate defect: its effect is taken into account by the permissible size for knots.

4. Insect attack

Timber both green and dry is subject to attack by a number of insects. The insect physically consumes a portion of the timber and the attack becomes apparent in sawn timber as holes of various sizes and shapes. Experience with a particular species of timber usually provides a reliable index of what can be tolerated from a particular type of insect attack, and most specifications permit minor insect damage. Holes are limited usually by diameter and by number permissible in a stated area. In some instances, severe insect attack may accompany fungal damage, so care should be exercised in assessing insect damage.

Pinhole borers

Pinhole borers attack some damaged or suppressed trees and some freshly felled logs. The attacks are common in Australian hardwoods. Fortunately, the effect on strength is negligible except in rare cases. Eggs are laid by adult beetles in the green log in the forest. The larvae tunnel into the log, forming generally straight holes about 1.5 mm in diameter. Attack can only occur when the wood is green and ceases as soon as the timber is partly seasoned. The attacks are generally well scattered, but clusters may occur as galleries, and where they do, particular care must be taken to ensure that they have not weakened the piece of timber.

Pinhole borers are known also as ambrosia beetles since the larvae feed on a fungus called ambrosia, which grows on the walls of the tunnels in the wood. This fungal growth ceases as soon as the moisture content is reduced by drying. Staining around the holes is sometimes noticed, but this stained wood is not weakened.

Where the holes are scattered. the effect of pinhole attack on strength and stiffness is negligible, and in most cases appearance is not seriously affected. Where extreme clustering occurs, strength will be reduced and specifications provide against clustering likely to impair strength.

Grub holes

Other forms of insect attack that may be noted in sawn or hewn timber may be caused by longicorn, bostrychid, jewel or other beetles. The weakening effect is roughly proportional to the degree of attack; affected timber is graded accordingly. Holes larger than 6 mm are generally well scattered and are considered by the grading rules to be the same as knots.

Lyctus attack

The lyctus beetle attacks the sapwood only of timber that contains pores and certain loadings of starch. Its effect is to completely destroy the sapwood in susceptible species, so that where the sapwood is wide, lyctus may cause substantial weakening. For this reason, the amount of susceptible sapwood is limited in house framing timbers.

With most timbers there is little risk that structural pieces of large dimensions will lose strength. Where the cross-section involved is small, such as in studs, braces, tiling battens or small mouldings, the major portion could be vulnerable. The introduction of the Timber Framing Code may have caused problems with regard to house framing timbers, as smaller sections are permitted. Sapwood can be immunized against lyctus, and such immunization is required by law in New South Wales and Queensland. Treated sapwood is in every way equivalent in use to normal truewood.

Termite attack

Termite attack may range from insignificant surface runs that have no appreciable weakening effect to complete destruction of the interior of a piece of wood, leading to failure. The termites preferentially attack near the pith and may be present for some years before the attack in large sections is detrimental or before a pole is seriously weakened. For No. 3 Structural Grade, insignificant surface runs are permitted, but care must be taken to ensure that the attack is on the surface only.

5. Brittle heart

This defect is common in several eucalypts and is commonly believed to come from growth stresses within the growing tree. It may, however, be fungal in origin. It is confined to the heart and the immediately surrounding area. The affected area may be relatively small, as is the case in most eucalypts, or fairly extensive, as is the case in large, over-mature trees. Brittle heart timber is usually not detectable by simple visual examination as it may appear similar to sound material in every ay. In some cases, brittle heart may be detected by breaking a small piece of timber and noting whether the fracture is carrot-y, or by prizing up with the point of a penknife small splinters from the surface. Brittle heart timber snaps over the point, whereas sound timber tends to lift off in a long splinter.

Tests show that although the impact strength of brittle heart is very low, its static strength properties are not appreciably less than those of adjacent, unaffected wood unless there is obvious decay. Brittle heart is low in shock resistance and durability. For framing timbers, brittle heart is completely excluded by the grading rules for all timbers with an end-section of less than 175 x 175 mm.

6. Decay

Decayed wood has very little strength and is particularly low in toughness. As the full extent of the decay is difficult to estimate or define and as the decay may, under adverse conditions, spread even after the timber has been put in place, timber showing decay should not be used for purposes where strength is required.

7. Checks, shakes and splits

Checks show as crevices, while shakes occur around or across the growth rings. Checks are caused by unequal shrinkage in the radial, tangential or longitudinal directions. When the outside of a piece dries much more rapidly than the inside, the faces contract while the core remains unaltered, causing checks to appear on the face. As the interior dries, the checks may close and become invisible, but the wood does not re-unite and remains weakened.

End checking is due to local drying at the ends of logs or timber, where stresses set up splits along the medullary rays, which are naturally weak. They, too, may close as the timber dries, but the fibres may continue to be separated and strength reduced.

Ring and star shakes often occur in the ends of hardwood logs after cross-cutting. They r y be due to drying of the ends, but the principal cause is the relief of growth stresses in the tree by cross-cutting.

Splits may be eliminated or reduced in the course of sawmilling.

8. Warping

Distortions may develop in timber for a variety of reasons, and the reputation of timber as a commercial material is adversely affected by these distortions. The principal kinds are bow, cup, spring and twist. They are generally more evident in dry timber than in green timber. However, bow and spring often occur in the course of sawmilling, and some occurrences can be attributed to the uneven distribution of stress in logs. They can increase the problems of controlling dimensions during sawmilling; severe spring, in particular, causes economic losses. They can be accentuated during seasoning but can also be controlled or reduced by suitably placing pieces in seasoning stacks and carrying out the seasoning under weights and restraints associated with steaming treatments. Spring is particularly critical in studs used in light timber framing.

Cup and twist develop mainly during seasoning as a result of differential shrinkage in the several directions relative to the growth rings.

Warp can be limited to stated amounts per unit of length for bow and spring, per unit of width for cup, and per unit of both length and width for twist.

9. Wane

Imperfect sawing sometimes results in the retention on part of the corner, or part of the face or the edge, of a portion of the original log surface. This is called wane. It is accepted on structural pieces when the section meets the strength requirements or the fixing requirements for the piece.

B. Acceptability of defects

When the grading rules for Australian timbers were drafted, the significant characteristics of timber were realistically appreciated. The seriousness of any irregularity, imperfection, blemish, fault or defect is judged primarily with respect to its influence upon appearance and strength. The intended use determines the relative importance of the one with respect to the other.

Some defects are unsightly, some reduce strength and some affect other aspects of serviceability. As timber is used for many different purposes, there are many criteria of acceptability, and the significance of a defect of a certain type and size in some uses may not be the same in other uses. The influence of the above features or defects on appearance is regarded as a matter of opinion. The influence of defects on strength is a technical question that needs to be answered by controlled tests.

There is nothing highly technical involved in deciding limits for grades based on appearance factors. A buyer requires the highest standard possible. The seller points out that, with the best of intentions and deliberate effort to eliminate certain faults, timber will not be perfect. While there may be difficulty in attaining unanimous agreement on the significance of an irregularity, consensus, or at least compromise between the conflicting attitudes of producers and users, must eventually become basic to the preparation of a specification for grading rules. For panelling, furniture and other woodwork likely to be finished with clear coatings, irregularities that contrast unduly with the main appearance of the wood may be regarded as defects. Thus, black stains on a pale background, white streaks on a dark surface, holes, broken or decayed knots, torn grain and other defects may so consistently be regarded as faults by many users that they are unacceptable in timber used for such exacting purposes.

The influence of defects on strength has been investigated in considerable detail. It has been found that sloping grain, knots, splits, decay and holes are the principal causes of loss of strength. The effect of these defects in various sizes and positions has been determined. From the investigations, it is possible to say, for instance, that provided the grain does not exceed a specified slope, or provided the size of a knot does not exceed a certain diameter, or provided the wane does not result in more than a certain loss of section, the piece will have a strength not less than a stated percentage of the strength of, respectively, a straight-grain timber, a knot-free timber or a complete rectangular section.

Accordingly, from a knowledge of the relationships between defects and strength it becomes possible to state the limits for type and size that will ensure that pieces sorted to these limits will not be lower in strength than a specific percentage of the strength of defect-free timber. Thus, the basis of grading rules for structural timber emerges.

The effects of malformations and damage to trees, revealed as defects in products, are more objectionable in some uses than in others. What may be considered a defect for structural purposes may be accepted as a feature where appearance is the only consideration. Usually, if the defect is small in size and isolated, it is accepted without question. However, what is small in the opinion of a seller may not be regarded as small by a purchaser. There needs to be some clarification of terms or some specific descriptions. It would be especially desirable for any statement to have only one meaning for everyone concerned.

For these reasons, there have been conscientious efforts to write grading rules and specifications precisely. The Standards Association of Australia has attended to this matter, bringing together producers, consumers and other interested parties in order to air different points of view before the standards were set. The Association is the publisher of the standards that have been endorsed by the interested parties. Some of the grading rules are reproduced here as tables 4-8. These are extracted from four sources:

(a) AS 2082-1979, Visually Stress Graded Hardwood for Structural Purposes;

(b) AS 1490-1973, Visually Stress Graded Radiata Pine for Structural Purposes;

(c) AS 2440-1981, Sawn Douglas Fir (Oregon) and Sawn Western Hemlock (Canada Pine);

(d) "Sloping grain in timber", CSIRO Trade Circular No. 48.

Table 4. Grading rules for hardwood F8

GREEN IS THE PRESCRIBED COLOR FOR THE BRAND MARK TO DENOTE F8 STRESS GRADE TIMBER

SIMPLIFIED INFORMATION FOR CONSTRUCTION TIMBER (structural) BASED ON AS2082-1979.

General: Each piece of timber of structural grade No. 3 shall be free from compression failures and other fractures.

Size: Each piece shall be sized with the maximum tolerance of 2mm for this season timber.

NOTE: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS		MAXIMUM PERMISSIBLE LIMITS
CHECKS	Internal	Not to exceed ½ thickness
	Surface	Unlimited
KNOTS	Sound	
	Oval	Not to exceed 1/3 width
	Round	Not to exceed 1/3 thickness
	Arris Spike	Not to exceed 1/3 on least face or edge
	Loose Knots - Holes	Same as a Sound Knot
BORERS HOLES	Over 3mm	Over 3mm or where distance between is less than 3mm assess as Sound Knot
	Under 3mm	Unlimited, provided distance between is minimum of twice width of hole
GUM VEINS	Tight	Unlimited
	Loose	
Loose Gum and Shakes	Width	Not to exceed 3mm 1/4 length aggregate
	Length	Not to go through one surface to another
GUM POCKETS	Length	Not to exceed 3 times width or 300mm maximum
	Width	
	- 1 surface	1/2 width or 25mm maximum
	- through	1/3 width or 20mm maximum
BOW		35mm in a piece of 3m long x 38mm thick. (refer to tables)
SPRING		14mm in a piece of 3m long x 100mm wide. (refer to tables)
TWIST		1mm per 10mm of width in pieces. 3m long x 38 mm thick (refer to tables)
CUPPING		1mm in 50mm of width
SLOPE OF GRAIN		1 in 8
DECAY/TERMITES		On surface only and slight
WANT/WANT/SAPWOOD		1/4 of cross section (aggregate) 1/3 thickness except for 1/3 length where it may extend to the full width of the thickness
END SPLITS		Aggregate length at each end. 1½ times width or 150mm maximum
HEART		Not permitted in section under 175mm x 175mm

This information sheet has been prepared as a ready reference guide. For complete details of grading rules, reference should be made to AS2082-1979.

Table 5. Grading rules for radiata pine F5

BLACK IS THE PRESCRIBED COLOR FOR THE BRAND MARK TO DENOTE F5 STRESS GRADE TIMBER

SIMPLIFIED INFORMATION FOR CONSTRUCTION TIMBER (structural) BASED ON AS 1490-1973.

General: Each piece of timber shall be free from decay, shakes, splits and fractures.

Size: Each piece shall be of actual dimensions with a maximum tolerance of 3mm OVERSIZE only.

Note: Each parcel of timber to this grade shall contain a reasonable distrubution of timber ranging from the lower limits of this grade to just short of the next highest grade.

IMPERFECTIONS		MAXIMUM PERMISSIBLE LIMITS
CHECKS		Maximum 600mm long
KNOTS	Face	
	- Centre Margin	4/5 (80%) of face width
	- Face Knot	3/5 (60%) of face width
	Margin	
	- Margin	(10%) of face width
	- Knot	7/16 (43%) of face width
	Edge	2/3 (66%) of thickness
	Arris	7/16 (43%) area of section
	Holes	Measure as a knot
RESIN POCKETS		
	Length	Maximum 150mm
	Width	Maximum 12mm
STAIN		Unlimited
HEART/PITH		1. Only in sizes 240mm and over 2. Must be in middle 1/3
SLOPING GRAIN		1 in 5
WANE/WANT		1. 1/4 (25%) of area of section 2. Maximum 1/4 of any surface
BOW		40mm in a piece 3m in length and 35mm thick (see tables)
SPRING		16mm in a piece 3m in length and 90mm wide (see tables)
TWIST		3mm for every 10mm of surface width (see tables) e.g. 27mm in a piece 90 x 35mm. 3m in length.

This information sheet has been prepared as a ready reference guide. For complete detail of grading rules, reference should be made to AS 1490-1973.

Table 6. Grading rules for Oregon (Douglas fir) F5

Black is the prescribed colour for the brand mark to denote F5 stress-graded timber ●

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4 mm under the nominal size.

The following MAXIMUM imperfections shall be permitted on the worst face and edge:

Note: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	Permitted
FACE KNOTS	5/8 W on Centre Line Elsewhere Proportional (See note over)
MARGIN KNOTS	3/8 W
THROUGH KNOTS	As Face or Margin Knots (Measure only on the face of the piece)
EDGE KNOTS	3/5 T
ARRIS KNOTS	X Sectional Area = 3/8 W x T Useful working guide: 1/2 x 3/4 = 3/8 Therefore maximum permissible arris knot size would be: 1/2 width of face by 3/4 thickness of edge or 3/4 width of face by 1/2 thickness of edge
HOLES AND LOOSE KNOTS	Measure as sound knots in same position
RESIN STREAKS	Permitted
RESIN POCKETS	Permitted
RESIN BLISTERS	Permitted
WANE AND/OR WANT	$\frac{W + T}{3}$ (MAX $\frac{W}{2}$, $\frac{T}{4}$, $\frac{1}{3}$ agg. L)
SAPWOOD	Permitted
END SPLITS	Length: Twice width of piece (at both ends) 2 x W
SHAKES	Centre face 5/8 T Elsewhere 3/4 T
STAIN/DISCOLORATION	Permitted
SOUND HEART	Permitted
SLOPE OF GRAIN	1 in 6
SPRING	25 mm in pieces 3.6 m long x 100 mm wide OR equivalent curvature in other lengths and widths. (Refer 'Spring and Bow' Tables)
BOW	50 mm in pieces 3.6 m long x 50 mm thick OR equivalent curvature in other lengths and thicknesses. (Refer 'Spring and Bow' Tables)
TWIST	1 mm per 10 mm of width in pieces 3.6 m long x 50 mm thick OR equivalent in other lengths and thicknesses.

This information sheet has been prepared as a ready-reference guide. For complete details of the Grading Rules, reference should be made to Australian Standard

Table 7. Grading rules for Oregon (Douglas fir) F7

Blue is the prescribed colour for the brand mark to denote F7 stress-graded timber ●

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4 mm under the nominal size.

The following MAXIMUM imperfections shall be permitted on the worst face and edge:

Note: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	Permitted
FACE KNOTS	1/2 W on Centre Line Elsewhere Proportional
MARGIN KNOTS	1/3 W
THROUGH KNOTS	Measure only on the face of the piece
EDGE KNOTS	1/2 T
ARRIS KNOTS	X Sectional Area = 1/3 W x T Useful working guide: 1/2 x 2/3 = 1/3 Therefore maximum permissible arris knot size would be: 1/2 width of face by 2/3 thickness of edge or 2/3 width of face by 1/2 thickness of edge
HOLES & LOOSE KNOTS	Measure as sound knots in same position
RESIN STREAKS	Up to 375 mm and 1/3 Length in agg.
RESIN POCKETS	Up to 375 mm and 1/3 Length in agg.
RESIN BLISTERS	Up to 375 mm and 1/3 Length in agg.
WANE AND/OR WANT	$\frac{W + T}{6}$ (MAX $\frac{W}{4}$, $\frac{T}{4}$, $\frac{1}{4}$ agg. L)
SAPWOOD	Permitted
END SPLITS	Length: Width of piece (at both ends)
SHAKES	Centre Face 1/2 T Elsewhere 3/4 T
STAIN/DISCOLORATION	Permitted
SOUND HEART	Permitted
SLOPE OF GRAIN	1 in 8
SPRING	19 mm in pieces 3.6 m long x 100 mm wide OR equivalent curvature in other lengths and widths. (Refer 'Spring & Bow' Tables)
BOW	38 mm in pieces 3.6 m long x 50 mm thick OR equivalent curvature in other lengths and thicknesses. (Refer 'Spring & Bow' Tables).
TWIST	1 mm per 10 mm of width in pieces 3.6 m long x 50 mm thick OR equivalent in other lengths and thicknesses.

This information sheet has been prepared as a ready-reference guide. For complete details of the Grading Rules, reference should be made to Australian Standard

Table 8. Grading rules for Oregon (Douglas fir) select dressing grade

SELECT DRESSING GRADE

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4mm under the nominal size.

The following MAXIMUM imperfections shall be permitted on the worst face and edge.

Select dressing grade timber is intended to provide material for finishing and joinery purposes where better appearance is the primary consideration. The timber shall be truly sawn and free from decay. The following imperfections shall be permitted:

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	150mm long
FACE KNOTS MARGIN KNOTS THROUGH KNOTS EDGE KNOTS ARRIS KNOTS HOLES OR LOOSE KNOTS	Not more than 3 in 3.6m length x 200mm width. A proportional Number of Knots shall be Permitted on other Lengths and Widths Providing the Knot does not Exceed 1/8W or 1/3T or 25mm, whichever is least. 1/2 Permitted for Sound Tight Knots or 10mm, whichever is least.
RESIN STREAKS RESIN POCKETS RESIN BLISTERS	150mm Long individually 150mm Long individually 150mm Long individually
WANE	Not permitted
SAPWOOD	Permitted
END SPLITS	1/2 W
SHAKES	Not permitted
STAIN	Not Permitted
SOUND HEART (PITH)	Not permitted
SLOPE OF GRAIN	Permitted
SPRING	38mm in 3.6 Length
BOW	38mm in 3.6 Length
TWIST	Not permitted
DISCOLOURATION	Permitted

VI. REVIEW OF TIMBER STRENGTH GROUPING SYSTEMS

William G. Keating*

Introduction

To the casual observer it must seem strange that in some countries, particularly those where forests are an obvious major natural resource, the structural use of timber lags well behind the use level attained by other construction materials. There are probably many reasons, economic, technical and even cultural, why this is the case. One reason surely must be the problem involved in presenting structural data to the end-user in an appropriate fashion whenever there is a multiplicity of species involved. A technique devised to minimize this problem is known as grouping.

To give just a few examples of the magnitude of the problem, Pong Sono [1] has listed approximately 200 species of merchantable timber in Thailand, and Espiloy [2] notes that in the Philippines there are over 3,000 timber species, of which several hundred are probably merchantable. In Australia, while there are about 80 species used extensively, more than 500 species have been classified for structural use [3]. Many of these species are sold in mixtures because of the practical difficulties associated with their identification and segregation, but strength grouping is able to cope with this requirement [4].

A. Basis of grouping

Essentially, grouping for structural purposes means the creation of a preferably small set of hypothetical species so that any timber may be classified within this set and considered as equivalent to one of the hypothetical species.

From a survey of the literature it would appear that many countries have either adopted the Australian system of strength grouping, as described by Pearson [5] and Kloot [6], or have used it as the basis for developing their own systems. Some of the countries are Kenya, United Republic of Tanzania, Nigeria, Papua New Guinea, Fiji, Samoa and Solomon Islands. In addition, UNIDO has used the technique to develop the design of a low-cost, modular, prefabricated wooden bridge. Of course there are many other systems in use, but most of the well-known ones, such as those used in North America, are, in the main, concerned with a comparatively small number of softwood species.

B. Motivation for grouping

The degree of motivation for adopting a classification system based on structural properties varies directly with the number of species that need to be accommodated. Without grouping, the problems involved are most obvious when it comes to publishing design information. Even if data on a large number of species from a particular country are available, it is not often feasible to publish the relevant design information in a convenient form. This is where the grouping techniques make the presentation much easier.

*An officer of CSIRO, Division of Chemical and Wood Technology, Melbourne.

Building regulations is one area where grouping introduces advantages that are of particular value [7]. Besides the obvious simplification, regulations written in terms of groups rather than individual species incorporate tables of design properties that remain fixed. This means that no major change is involved should a new timber be introduced to the market or an existing one be reassessed. In Australia, the Timber Framing Code [8] manages, by means of a limited set of tables, to present spans and sizes for all the timber framing members required in domestic housing construction. The data are applicable to all grades for several hundred species or species mixtures and are set forth in a convenient format.

Even if a single species dominates the timber construction scene, grouping in relation to building regulations is advantageous. Particularly with plantation timbers, the structural properties of populations of timber taken from the same species can vary from one forest location to the next and can also vary with forest age and silvicultural practices. Transferring a species, or the production from one area, from one group to another is not nearly as complicated as promulgating a new or additional set of design stresses [7].

Internationally, an agreed grouping technique could help timber utilization generally and have special relevance to the structural timber trade. UNIDO modular wooden bridge projects are good examples of grouping applied globally as the set of design standards based on eight strength classes can be used for almost any timber in the world. It is not difficult to envisage how other types of technology transfer in the form of timber design codes and manuals would be possible if an agreed or compatible grouping system for structural timber was in general use. The grouping technique has the following advantages:

(a) Building regulations are concerned with only limited sets of design parameters;

(b) The marketing of structural timber is easier as it is carried out in terms of structural properties rather than by nomination of the species and grading methods;

(c) More flexibility is available to the supplier as the range of species is much wider;

(d) The entry of new, lesser-known species onto the market is facilitated;

(e) Domestic and international trade in structural timber is simplified;

(f) Technology transfer in the form of timber design codes and manuals is easier;

(g) It is much less expensive in terms of time and material to place a species in a group than it is to develop individual working stresses;

(h) It is possible to group a species, albeit conservatively, based on density measurements alone.

C. Existing strength grouping systems

Strength grouping systems in a few countries and one region are discussed next.

1. Australia

Strength grouping in Australia has been in operation for more than 50 years. In 1939, Langlands and Thomas [9] proposed for Australian conditions four strength groups. A species was placed in a group according to its species mean values as determined from standard tests on small, clear specimens. These strength groups were established when there was little information available about the properties of most Australian species, and they were used successfully only because the limits had not been closely defined [5].

The impetus at that time to establish strength groups came, as it does now, from the need to cope with a large number of species, many of which are difficult to identify and many of which are also marketed as mixtures.

The original Australian strength grouping system was revised and expanded, as has been explained by Pearson [5] and Kloot [6]. Prior to expanding the strength groups, which was made necessary by new information and new species, Pearson developed a set of working stresses that has become the basis for a strength classification system.

Working backward from the set of working stresses, it was possible to develop the appropriate strength groups. This process is the reverse of the usual procedure for deriving working stresses for an individual species, allowing for duration of load, accidental overloads and estimating the 1 per cent probability point.

In developing this set of stresses, Pearson reported that three decisions were required. Firstly, it was necessary to decide whether the stresses should be in arithmetic or geometric progression. Secondly, a compromise was required on the magnitude of the differences between successive stresses in order to achieve a satisfactory balance between the simplicity associated with having only a few groups and the greater efficiency of having numerous groups. Finally, the actual value of the stresses had to be decided.

Cooper [10] had shown the merits of a geometric series for working stresses, and such a choice had also been recommended by the International Organization for Standardization and the Food and Agriculture Organization of the United Nations (FAO). Accordingly, a geometric progression was chosen, using a preferred number series with adjacent terms chosen in the ratio of 1.25 to 1 for modulus of rupture. This was judged to be the appropriate compromise between simplicity and preciseness. Also, it appeared that the Australian visual grading rules then being developed would probably also have differences between grades of 25 per cent. The range of the values chosen was such that it covered all the species likely to be used structurally in Australia.

Using the set of values decided upon as the basic working stresses in bending, the values of the other properties were determined from regression equations. The values thus determined are presented in table 9, which is the basis of the current Australian strength classification system.

Table 9. Design properties for sawn timber, round poles and plywood (Megapascals)

Stress grade a/	Basic bending strength b/	Basic tension strength	Basic compression strength	Modulus of elasticity
F34	34.5	20.7	26.0	21 500
F27	27.5	16.5	20.5	18 500
F22	22.0	13.2	16.5	16 000
F17	17.0	10.2	13.0	14 000
F14	14.0	8.4	10.2	12 500
F11	11.0	6.6	8.4	10 500
F8	8.6	5.2	6.6	9 100
F7	6.9	4.1	5.2	7 900
F5	5.5	3.3	4.1	6 900
F4	4.3	2.6	3.3	6 100
F3	3.4	2.1	2.6	5 200
F2	2.8	1.7	2.1	4 500

a/ The insertion of the letter F before each value in the table introduces the concept of stress grade. Stress grade is defined as the classification of a piece of timber for structural purposes by means of either visual or mechanical grading to indicate primarily the basic working stress in bending in megapascals for purposes of design and by implication the basic working stresses for other properties normally used in engineering design. For example, a piece of timber with a stress grade of F14 resulting from a certain combination of strength group and visual grade would have a basic working stress in bending of 14 MPa.

b/ These values are the result of the soft metric conversion of a preferred series of values in imperial units, viz. 5,000, 4,000, 3,200, 2,500, 2,000, 1,600, 1,250, 1,000, 800, 630, 500 and 400 p.s.i., readily recognizable as the R10 series.

As described above, the species mean values for clear material for each strength group for the critical properties were developed for green and dry timber and are shown in tables 10 and 11, respectively.

Table 10. Preliminary classification values for unseasoned a/ timber (Megapascals)

Property	Minimum species mean						
	S1	S2	S3	S4	S5	S6	S7
Modulus of rupture	103	86	73	62	52	43	36
Modulus of elasticity	16 300	14 200	12 400	10 700	9 100	7 900	6 900
Maximum crushing strength	52	43	36	31	26	22	18

a/ As measured or estimated at a moisture content above the fibre saturation point.

Table 11. Preliminary classification values for seasoned a/ timber (Megapascals)

Property	Minimum species mean SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Modulus of rupture	150	130	110	94	78	65	55	45
Modulus of elasticity	21 500	18 500	16 000	14 000	12 500	10 500	9 100	7 900
Maximum crushing strength	80	70	61	54	47	41	36	30

a/ As measured or adjusted to a moisture content of 12 per cent.

Tables 10 and 11 allow the strength grouping of every species that had been, or was capable of being, properly sampled and tested by standard methods using small, clear specimens. Once a species has been strength-grouped, commercial pieces of that species can, following visual grading, be allocated a stress grade by reference to tables 12 and 13. The appropriate design parameters may be determined from table 9.

Table 12. Relationship between strength group, visual grade and stress grade for green timber

Visual grade a/ Structural grade	Per cent of strength of clear material	Stress grade b/ S1	S2	S3	S4	S5	S6	S7
No. 1	75	F27	F22	F17	F14	F11	F8	F7
No. 2	60	F22	F17	F14	F11	F8	F7	F5
No. 3	48	F17	F14	F11	F8	F7	F5	F4
No. 4	38	F14	F11	F8	F7	F5	F4	F3

a/ Standards Association of Australia, 2082-1977: Visually Stress-Graded Hardwood for Structural Purposes (Sydney, 1977); and Standards Association of Australia 1648-1974: Visually Stress-Graded Cypress Pine for Structural Purposes (Sydney, 1974).

b/ Note the interlocking effect (diagonal line), which reduces the possible number of stress grades from 28 to 10.

Table 13. Relationship between strength group, visual grade and stress grade for seasoned timber

Visual grade a/		Stress grade							
Structural grade	Per cent of strength of clear material	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
No. 1	75		F34	F27	F22	F17	F14	F11	F8
No. 2	60	F34	F27	F22	F17	F14	F11	F8	F7
No. 3	48	F27	F22	F17	F14	F11	F8	F7	F5
No. 4	38	F22	F17	F14	F11	F8	F7	F5	F4

a/ Standards Association of Australia, 2082-1977: Visually Stress-Graded Hardwood for Structural Purposes (Sydney, 1977); Standards Association of Australia, 2099-1977: Visually Stress-Graded Seasoned Australian Grown Softwood (Conifers) for Structural Purposes (excluding Radiata Pine and Cypress Pine) (Sydney, 1977); Standards Association of Australia, 1490-1973: Visually Stress-Graded Radiata Pine for Structural Purposes (Sydney, 1973); and Standards Association of Australia, 1648-1974: Visually Stress-Graded Cypress Pine for Structural Purposes (Sydney, 1974).

Because there has been international agreement on the standard methods of testing small, clear specimens, it is possible to utilize data from recognized laboratories anywhere in the world to place any species into a strength group. This has been done for 700 African species [11], 190 South American species [12] and 362 South-East Asian species [13].

One decision that must often be made in classifying a species from tables 10 and 11 is what to do when the three properties do not all have the same classification. A conservative approach would be to assign the species to the lowest group indicated from the individual properties. For many combinations, this is the correct approach, but there are several combinations for which the overall species strength group may justifiably be raised one step above the lowest assessment. Table 14 summarizes the procedure that is followed. It shows that more emphasis is placed on modulus of rupture and modulus of elasticity than on compression strength.

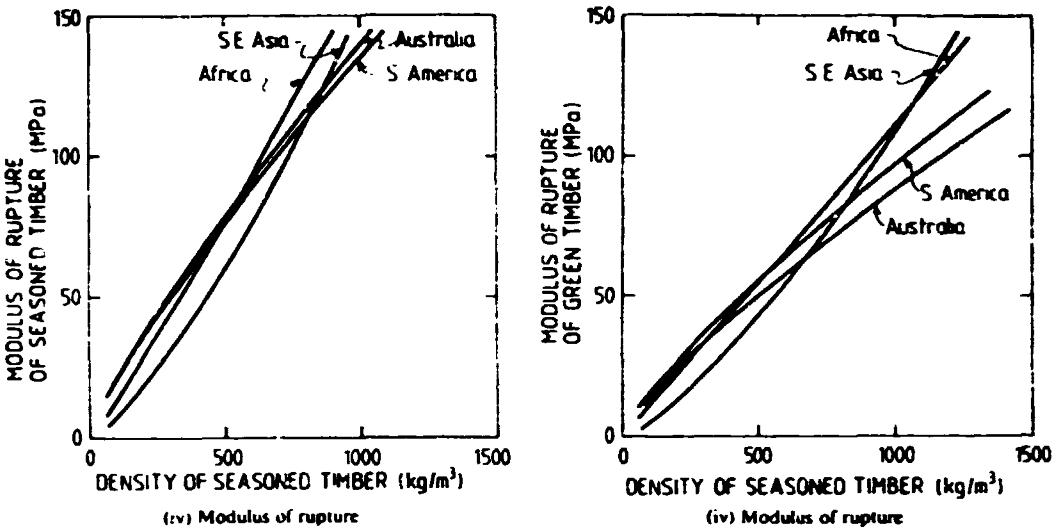
Table 14. Combinations of preliminary classifications that permit the overall strength group assessment to be one step above the lowest in the combination a/

Preliminary classification based on			Assessed
Modulus of rupture	Modulus of elasticity	Maximum crushing strength	S or SD strength group
x	x	x + 1	x
x	x - 2	x - 1	x - 1
x	x + 2	x + 1	x + 1

a/ Strength group x - 1 is stronger than strength group x; e.g. if strength group S4 is denoted by x, then strength group S3 is denoted by x - 1.

This leaves those species for which the strength data are from a sample of less than five trees or for which no data are available. An examination by Leicester and Keating of the relationship between density and modulus of rupture of seasoned timber for 30 species from each of four regions around the world is indicated in figure 71.

Figure 71. Regression lines for modulus of rupture-density of seasoned timber



On the basis of this relationship, table 15 was constructed to permit classification. While it gives a rather conservative assessment, at least it does allow those species with limited data to be entered into the system. In AS MP 45-1979, Report on Strength Grouping of Timbers, species assessed in this fashion are listed with their strength groups in brackets to indicate the provisional nature of the assessment.

Table 15. Minimum air-dry density values from five or more trees for assigning species to strength groups in the absence of adequate strength data

	Air-dry density at 12 per cent moisture content (kg/m^3)							
Strength group (S or SD)	1	2	3	4	5	6	7	8
Unseasoned material	1 180	1 030	900	800	700	600	500	-
Seasoned material	1 200	1 080	960	840	730	620	520	420

Up to this point, discussion has been confined to indirect entry into the strength classification system in table 9 by means of strength grouping combined with visual grading. Direct entry into the system is also possible by machine stress grading and proof grading. Both techniques are described in a later section.

2. United Kingdom

Details of the current system in the United Kingdom of Great Britain and Northern Ireland are given in the British Standard Code of Practice on the Structural Use of Timber CP112:1967 [14]. Briefly, nine strength classes have been proposed, C1 to C9, as shown in table 16.

Table 16. Dry grade stresses and moduli of elasticity for strength classes as proposed for BS 5268: part 2 (Megapascals)

Strength class	Bending	Tension	Compression (parallel)	Modulus of elasticity Mean	Modulus of elasticity Min.
C1	2.8	2.2	3.5	6 800	4 500
C2	4.1	3.2	5.3	8 000	5 000
C3	5.3	4.2	6.8	8 800	5 800
C4	7.5	5.1	7.9	9 900	6 600
C5	10.0	6.0	8.7	10 700	7 100
C6	12.5	7.5	13.6	12 100	9 200
C7	15.6	9.5	15.7	14 900	11 400
C8	19.5	12.0	18.2	17 900	13 800
C9	24.4	15.0	21.3	21 500	16 700

The derivation of these stresses and the allocation of the various grades of those softwood species in common use in the United Kingdom to the appropriate strength classes was based on a testing programme using structural-size timber. The range of species tested in this fashion did not include all those species in use. For those not yet tested, recourse was made to the test data for small, clear wood and a ratio was applied based on the five-percentile results obtained from tests on structural-size timber.

For the softwoods visually graded to British Standard 4978:1973, Timber Grades for Structural Use, the two visual grades GS (General Structural) and SS (Special Structural) cater for most of the imported softwood species and have been allocated to the C3 and C4 strength classes, respectively. The grade ratios, i.e. the comparison with clear strength values, for these two grades are considered to be 0.35 and 0.50, respectively, in bending [15].

For tropical hardwoods that will also be included in British Standard 5268, only one grade, HS (Hardwood Structural), is proposed with a grade ratio in bending of 0.67. As with the softwoods, the stresses were based on a combination of test data from small, clear samples and the five-percentile values obtained from tests on structural-size timber.

3. The Philippines

In the Philippines, a system has been developed that is very similar to the Australian strength grouping system in that it is based on the results of small clear tests and adopts a preferred number progression with an interval of 1.25 between the base numbers [2]. However, it was judged that there was no need to cover the same range, so only five groups have been chosen. According to Espiloy, the advantages of the grouping system are as follows:

(a) Each member species within a class can substitute for another, which helps to overcome the problem of supply;

(b) The traditional bias against less-known species is easily overcome when they are grouped together with more common species. The system helps engineers and architects to familiarize themselves with alternative species by specifying that any timber within a given class, rather than a particular named timber, may be used;

(c) It overcomes the problem that is usually encountered in identifying sawn timber of similar physical and strength characteristics;

(d) It simplifies design and specification procedures and thus facilitates the formulation of a comprehensive building code for structures using solid wood. The grouping scheme forms a rational series that fits closely with timber grades. With this system, only a few sets of working stresses are adequate to cover the proposed strength classes and grades of timber.

The limiting average values for classifying a species into one of the strength classes, C1 to C5, are given in table 17.

Table 17. Minimum strength-class limits for grouping Philippine timber species

Property	Moisture condition	Class of timber				
		C1	C2	C3	C4	C5
Modulus of rupture bending (kg/cm^2) a/	Green, 12% MC	800 1 250	630 1 000	500 800	400 630	315 500
Modulus of elasticity in bending (1,000 kg/cm^2)	Green, 12% MC	130 160	100 120	770 950	600 730	460 560
Compression parallel to grain (kg/cm^2)	Green, 12% MC	400 650	305 500	235 385	185 300	140 230
Compression perpendicular to grain (kg/cm^2)	Green, 12% MC	900 135	560 900	255 580	225 375	140 245
Shear parallel to grain (kg/cm^2)	Green, 12% MC	100 140	80 110	630 850	500 650	400 500
Specific gravity b/	Green, 12% MC	0.670 0.710	0.545 0.580	0.450 0.475	0.365 0.385	0.300 0.315

a/ 1 kg/cm^2 = 0.098 MPa.

b/ Based on weight when oven-dry and volume at test.

A procedure has been developed to cover the case when the property values for a particular species do not all fall within the same strength class.

4. South America

Under the auspices of the Andean Pact, five South American countries - Bolivia, Colombia, Ecuador, Peru and Venezuela - have in recent years undertaken a comprehensive testing programme aimed at developing a set of grade rules and a strength grouping system applicable to the region. This was the subject of a detailed report by Centeno [16], which set out the advantages of a strength grouping system as follows:

(a) It permits a large number of new, little-used species to be introduced to the building industry;

(b) It allows a more homogeneous, balanced and rational exploitation of forest resources;

(c) It limits or eliminates the vices implicit in the selective exploitation of a few precious species;

(d) It drastically simplifies the use and commercialization of wood as a construction material.

As a result of the above testing programme, the five countries have agreed on a single visual grading rule for structural hardwood and a strength grouping system comprising three strength groups. The working stresses derived for each strength group were arrived at after taking cognizance of both the results available from the testing of small, clear samples of 72 species and the testing of approximately 1,500 beams of structural-size timber representing more than 30 species.

The proposed working stresses for the three strength groups are given in table 18. These values are derived by taking the lowest five-percentile value for the group. The minimum modulus of rupture values are then divided by 2.1 to account for accidental overload and the effect of duration of load. A further reduction of 10 per cent is applied to account for a size effect. The modulus of elasticity values are the averages taken directly from the tests without further modification.

For a new species to be classified under the proposed system it is recommended that at least 60 beams be tested in third point bending and that the five-percentile modulus of rupture values, modified as above, and the mean modulus of elasticity values be used to determine the correct strength group by direct comparison with the table. A species may be allowed in a particular group when these parameters are no more than 10 per cent lower than the values indicated.

During the course of the testing programme it was observed that basic density was a good indicator of strength. As a consequence, basic density is now proposed as a method of positioning a species in a group on a preliminary basis. The limits selected, taking a conservative approach, were as given in table 19.

Table 18. Proposed working stresses a/
(Kilograms per square centimetre) b/

Group	F_m	F_c	F_t	F_p	F_v Beams	F_v Joints	Modulus of elasticity $E_{0.5}$	Modulus of elasticity $E_{0.05}$
A	120	170	160	60	20	25	140	110
B	170	130	120	45	16	20	120	95
C	130	100	90	30	12	15	90	70

Source: J. C. Centeno, Andean Grading System for Structural Hardwood Timber: Working Stress-Strength Groups, 32nd Annual Meeting (Atlanta, Georgia, United States of America, Forest Products Research Society, 1978).

a/ F_m is flexure; F_c, compression parallel to the grain; F_t, tension parallel to the grain; F_p, compression perpendicular to the grain; and F_v, shear.

b/ 1 kg/cm^2 = 0.098 MPa.

Table 19. Limits for basic density for each strength group
(Grams per square centimetre)

Group	Basic density a/
A	0.76 and above
B	0.60-0.75
C	0.44-0.59

a/ Basic density is the density of the timber calculated from the green (or fully saturated) volume and the mass when oven-dry.

An interesting approach taken in the development of the single visual grading rule was that the limits set on the size and location of defects should permit an average mill to produce 50-60 per cent of acceptable structural material. The remainder of the mill output would normally be suitable for non-structural applications in housing, such as sheathing and joinery.

Once the above system had been accepted, a timber construction manual was produced. The industry has since grown, as is evidenced by the establishment of factories producing prefabricated houses in several countries and by the construction of wood/cement panel plants. It is noteworthy that the various Governments support the rules and are incorporating them into the relevant building codes.*

*As was reported to the UNIDO Expert Group Meeting on Timber Stress Grading and Strength Grouping, Vienna, Austria, 14-17 December 1981.

The incentive for the Andean Pact countries to develop a stress grading and grouping system was threefold: it would help to overcome serious housing shortages, it would utilize a valuable resource and it would create employment.

5. Mexico

In Mexico [17], development of a simplified set of grading rules is close to being finalized. For convenience sake, the 50 *Pinus* species in use throughout the country have been treated as a single species. A large in-grade testing programme (5,000 full-size pieces) is in progress to determine the appropriate working stresses for the two grades of structural timber considered necessary. North American grading rules have been used, but their validity has been questioned, prompting the above testing programme.

The proposed grading rules have been framed so that, on average, mill output would be 30 per cent top grade and 40 per cent second grade, with the remainder going into non-structural applications. If this breakdown can be reflected throughout the country, there will be sufficient production to fulfil the needs of the local market.

The tentative design values based on the tests to date for the two suggested grades of pine are given in table 20.

Table 20. Tentative design values for Mexican pine
(Kilograms per square centimetre) a/

Grade	Bending: Single members	Bending: Load-sharing members	Mean modulus of elasticity x 10^3
A	140	160	115
B	80	90	90

a/ 1 kg/cm^2 = 0.098 MPa.

Investigations are also under way in an attempt to obviate the need for visual grading.

Also being examined is the indication that in Mexico the within-mill variation is larger than the regional variation. If this is the case, sampling will be much simpler and less expensive than previously thought. This could be of interest to countries with extensive pine resources.

Mexican research workers feel that the lack of a suitable timber grading system is the main reason why housing using timber framing is still not commonly accepted. Timber frame construction could help to make such housing less expensive than housing built with traditional masonry materials and so would make home ownership possible for a larger proportion of the population.

6. Other countries

There are several other countries, particularly those with hardwood resources, that have strength grouping systems similar to or based on the Australian system. In Canada and the United States, grouping techniques are used for the comparatively small number of species used structurally, but the degree of refinement attempted there appears to have little relevance to the problems of developing countries. However, in the broader sense, much of the research work emanating from both Canada and the United States has important implications for other countries, although, of course, most of the work is on softwood species. Of particular interest are the results of in-grade testing research programmes and the detailed studies being undertaken to determine the duration-of-load effect.

7. Summary

While the strength grouping/stress grading systems described so far vary somewhat in the ways they have been developed and are being utilized, they still have much in common. Firstly, they all use a small number of groups to cater for a comparatively large number of individual species. Secondly, they all aim to estimate the influence of defects on strength and stiffness. Thirdly, they are all based on visual grading. Their commonality becomes more obvious when working stresses are developed for some well-known species using the different systems. Indeed, despite the lack of accuracy inherent in the concepts as a whole, the end results are very similar. It is when attempts are made to bestow on the systems a degree of precision that is not really there, nor warranted, that discrepancies become apparent.

D. Extension of the grouping technique

1. Joints

It has been found that grouping is also a very useful technique in developing the basic loads applicable to metal fasteners [18]. When revised, the Australian Timber Engineering Code will use the classification system based on basic and air-dry density in table 21.

Table 21. Proposed minimum density for joint strength groups (Kilograms per cubic metre)

Green timber		Seasoned timber	
Group	Basic density	Group	Air-dry density a/
J1	750	JD1	940
J2	600	JD2	750
J3	475	JD3	600
J4	380	JD4	475

a/ Density at 12 per cent moisture content after reconditioning.

An example of its application to nailed joints is given in table 22.

Table 22. Proposed minimum properties of nailed joints a/

Green timber				Seasoned timber b/			
	Minimum value				Minimum value		
Group	Maximum load (N/nail)	Load at 0.4 mm displacement (N/nail)	Stiffness modulus	Group	Maximum load (N/nail)	Load at 0.4 mm displacement (N/nail)	Stiffness modulus
J1	2 170	685	1 220	JD1	1 920	925	1 420
J2	1 710	505	895	JD2	1 490	700	1 110
J3	1 330	365	650	JD3	1 170	530	875
J4	1 050	270	480	JD4	905	395	680

a/ Loads are for 2.8 mm diameter nails in single shear.

b/ Approximately 12 per cent moisture content.

2. Poles

From tables 12 and 13 it can be seen that if the product under consideration had only one visual grade and one moisture condition, then a greatly simplified new table would be possible. This is the case with poles if they are graded to the Australian standard.

On the basis of a large pole-testing programme carried out by CSIRO [19], poles from mature trees are considered to be in a single grade, that just above the 75 per cent grade. As poles are normally regarded as unseasoned, table 12 can be simplified to the form shown in table 23.

Table 23. Correspondence between strength group and stress grade for round timbers graded to AS 2209-1979 a/

Strength group	Stress grade
S1	F34
S2	F27
S3	F22
S4	F17
S5	F14
S6	F11
S7	F8

a/ The equivalence expressed is based on the assumption that the poles or logs are from mature trees.

3. Plywood

A similar grouping technique is possible with plywood. In Australian Standard 2269-1979 for structural plywood [20], visual grading rules for plywood veneer are specified so that its strength is roughly 60 per cent of the clear wood strength when the maximum permissible defects occur. With this prerequisite satisfied, the stress grade for a plywood may be derived from any one of the following three parameters:

(a) The strength group of the timber veneer;

(b) The density of veneers:

(c) The stiffness of the plywood sheets.

Table 24 shows the relationship between these parameters and the plywood stress grade. In this table, the modulus of elasticity is the value of stiffness of solid wood parallel to the grain that must be used in computing the stiffness of the plywood sheet.

Table 24. Grading parameters for plywood stress grade

	Grading parameter a/		
Plywood stress grade	Timber strength group	Modulus of elasticity of plywood sheet (MPa)	Minimum air-dry density (kg/m^3)
F34	SD1	21 500	1 200
F27	SD2	18 500	1 080
F22	SD3	16 000	960
F17	SD4	14 000	840
F14	SD5	12 200	730
F11	SD6	10 500	620
F8	SD7	9 100	520
F7	SD8	7 900	420

a/ Only one of the three grading parameters needs to be used.

4. Non-structural properties

There are several properties that may be classed as non-structural, e.g. durability and shrinkage, but that are still of critical importance to the engineer. Suggested methods for classifying these have been made by Keating [21].

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VII. PROPERTIES AND END-USES OF A RANGE OF WOOD-BASED PANEL PRODUCTS

Kevin J. Lyngcoln**

Introduction

The range of wood-based panel products has grown dramatically during the last 20 years. There have been many new wood-based panel products and, through technical innovation, modifications to existing wood-based panel products since the World Consultation on Wood-based Panels held at New Delhi in 1975 [1]. In 1970, there were three basic types of wood panels: plywood, particle board and hardboard. In 1982, the basic types of wood-based panels and variations within each type include oriented strandboard (OSB), medium-density fibreboard (MDF), structural flakeboards and waferboards, composite panels as well as innovations within the conventional plywood, particle board and hardboard products.

The objectives of this paper are as follows:

(a) To describe the main types of wood-based panel products in terms of wood material, geometry and adhesive binder;

(b) To compare the main structural properties of each product type;

(c) To relate the physical and mechanical properties to the ability of each product to perform satisfactorily in a range of end uses, mainly in construction, under the climatic conditions experienced in the Asian-Pacific region.

It became obvious when studing the references that to meet the above objectives it would be necessary to comment in detail on the various approaches to the structural application of wood-based panels. The approaches range from traditional prescription standards to straight performance standards to combinations of these.

It was decided to segregate the panels into groups, structural and non-structural, based on the long-term durability of the adhesive binder. This paper will deal mainly with structural wood-based panels.

Finally, it was decided that before any of the above objectives could be dealt with, some of the reasons for the development of the new wood-based panels, particularly those aimed at structural applications, should be considered.

Based on the literature reviewed, it must be stated at the outset that this paper does not provide detailed answers to all of the questions posed in the broad objectives. A great deal of additional research, in some cases fundamental research, will be required to obtain accurate answers. The paper

*This paper was first presented as a lecture at the FAO Technical Consultation on Wood-based Panels held from 13 to 17 January 1983 at New Delhi and is reproduced with the permission of FAO.

**Engineer, Plywood Association of Australia.

provides a review and shows additional areas that need researching if an Asian-Pacific country is to optimize its production of wood-based panels to suit its requirements relative to technical ability, availability of capital for investment and human and forest resources.

A. Reasons for the development of alternative wood-based panels

Since the mid-1970s, there have been considerable efforts, particularly in North America but also in Europe, to develop structural wood-based panel products to replace plywood in construction applications. It is suggested [2] that, as far as the North American industry is concerned, this trend can be attributed to developments in four areas: technology, raw materials, markets and politics.

Although in North America the technology to produce high-quality, non-veneer wood-based panels on a laboratory scale has existed for many years, it is only recently that this level of quality has been achievable in commercial quantities. Process development has been rapid as a result of the following raw material, market and political developments.

The raw material pressures to change come from the increased cost and poorer availability of high-quality peeler logs. Peeler log prices continued to inflate for the first time in history through the North American recession of the early 1980s. Simultaneously there was an availability of low-cost, previously ignored secondary species such as trembling aspen in Canada and spruce and light hardwoods in the upper Midwest of the United States. The density and flaking characteristics of these secondary species made them suitable for the new breed of structural wood-based panels.

Because lower cost plantation softwood species were available in Australia and New Zealand and high quality raw material was available in Malaysia and Indonesia, these trends towards the development of other structural wood panels, including fibre-aligned particle board, have been minimal in those countries.

The high price of peeler logs and their reduced availability to the North American plywood industry has caused a higher priced, lower quality plywood to appear in the marketplace. In traditional uses such as roof and wall sheathing and sub-flooring, plywood no longer holds the dominant competitive edge that it held for decades. So as not to lose these large-volume markets to competitive non-wood panels, traditional plywood-producing companies developed lower cost wood-based alternatives such as oriented strandboard, structural flakeboard, waferboard and Comply.

Finally, in North America there has been a philosophical change in the type of standard used for wood panels. The traditional prescription approach, as detailed in Product Standard PSI-74 [3] and ANSI AS 208.1 [4], has been supplemented by the acceptance of American Plywood Association performance-rated panels. Performance standards for specified applications facilitate the speedy acceptance into building codes of a wide range of panel types and production techniques.

Performance standards must relate closely to end-use and to the climate in which the panels are to be used. The development of performance standards in North America will be discussed in detail later in this paper.

It must be stressed here that although it can be shown that the above reasons for the development of structural wood-based alternatives to plywood are legitimate and logical in North America and Europe, before an Asian or Pacific country embarks on the production of similar wood panels, it would be well advised to ensure that the same reasons exist.

The new wood-based panels generally require a higher cost, higher technology plant and have a lower labour component. They are no better structurally than plywood and, so far, do not have the proven properties and range of applications, both external and structural, of plywood.

Discussions with machinery suppliers in Australia have revealed capital investment per cubic metre of panel produced in 24 hours and manpower (number of workers) required per cubic metre of panel produced in 24 hours (table 25).

Table 25. Capital investment and manpower requirements

Product	Capacity output/ 24 hr (m^3)	Capital investment/m^3/ 24 hours ($US)	Workers/ m^3/24 hr
Plywood	120	33 000	0.42
Particle board	100	95 000	-
	300	53 000	0.20
Medium-density fibreboard	300	64 000	0.20
Oriented strandboard	300	64 000	0.20
Waferboard	300	72 000	0.20

B. Types of wood-based panels

The long-term durability of a wood-based panel is closely related to the durability of the bond, provided the wood component is protected from biological attacks either by a hazard-free end use, e.g. dry interior, or by preservative treatment if a hazard exists. It has been shown [5] that the bond durability is related closely to the chemical type of the adhesive and the conditions of exposure. It is therefore reasonable to classify wood-based panels into non-structural, semi-structural and structural, based on the long-term durability of the adhesive type and thus the ability to continue to carry a sustained load under conditions of semi-exposure or cyclic humidity changes as experienced in the Asian-Pacific region.

A short-term physical test for bond durability related to long-term exposure, such as the 72-hour boil A bond test in AS 2098 [6] or the vacuum pressure soak in PSI-74 [3], also needs to be carried out to verify the performance of a particular resin formulation within a broad chemical grouping. Before a panel can be classified as structural, its physical and mechanical properties must be established. The establishment of these properties will be discussed later in this paper.

However, bond durability under the conditions of use is a prime consideration. Table 26 groups wood-based panels into the above-mentioned classifications under the climatic conditions that prevail in the Asian-Pacific region. The basis for the structural rating is the research work on adhesives durability carried out at the Forest Products Laboratory, Princes Risborough, United Kingdom [5].

Table 26. Classification of wood-based panels a/

Panel type	Adhesive/binder b/	Classification
Plywood	Phenolic, natural or synthetic	Structural
	Melamine urea formaldehyde	Semi-structural
	Urea formaldehyde	Non-structural
Particle board random-oriented, layered or homogeneous, including flakeboard	Phenolic, natural or synthetic	Structural
	Melamine urea formaldehyde	Semi-structural
	Urea formaldehyde	Non-structural
Hardboard (fibreboard)	Naturally occuring Polyphenolic chemicals e.g. lignin	Structural
Medium-density fibreboard	Phenolic, natural or synthetic	Structural
	Urea formaldehyde	Non-structural
Oriented strandboard	Phenolic, natural or synthetic	Structural
Waferboard	Phenolic, natural or synthetic	Structural
Comply	Phenolic, natural or synthetic	Structural
	Isocyanate c/	-

a/ For the further treatment of the suitability of wood-based panels for developing countries, see "Guidelines for the selection of options in establishing wood-based panel industries in developing countries", presented at the Seminar on Wood-based Panels and Furniture Industries, Beijing, 23 March-7 April 1981 (ID/WG.335/16).

b/ The adhesive/binder types relative to board type were established from a great number of the cited references.

c/ Lack of information on the long-term durability of isocyanate binders [7] prevents the classification of this panel at this time.

6. Plywood

Plywood is defined [8] as "a panel consisting of an assembly of veneers bonded together with the grain in most of the alternative plies at right angles". Each ply consists of a rotaryor sliced-cut veneer. Plywood is manufactured from indigenous or plantation pines, e.g. Shorea spp., or eucalypts, e.g. karri, and from combinations of species. Although a typical density range is 500-750 kg/m^3, plywoods are manufactured with densities as low as 400 and as high as 1,200 kg/m^3.

Individual veneer thicknesses for rotary-peeled veneer can range from 0.8 to 4.5 mm. In practice, however, the range is likely to be 1.0-3.2 mm. The panels are hot-pressed in flat presses and, depending on end-use, are bonded with phenolic, melamine, ureas or copolymers.

2. Particle board, including flakeboard

The variables in particle board are the raw materials available, chip geometry and method of chipping, particle alignment, homogenity through the thickness, the adhesive used, density and thickness. With this number of variables, it is impossible to describe particle board in anything but broad terms. Particle board can be platen-pressed or extruded. The production method also affects properties. Each variable is dealt with below in general terms [9].

Originally, particle board utilized wood waste such as planer shavings, chipped lumber and hammer-milled veneer that resulted from other wood processing or fabrication lines. Although particle board still has a waste component, much is now produced from forest thinnings in the form of roundwood and secondary species that are unsuitable for processing into timber or plywood [1, 9].

Chip geometry plays an important part in the basic physical and mechanical properties of particle board. Geimer et al. [9] explain, for example, that "dimensional stability was closely related to flake or particle geometry and quality with best thickness stability associated with small particles and best linear stability associated with large flakes". Particles range in size from 50 mm x 50 mm x 0.5 mm (in the case of ring-flaked particles from 50 mm chips) to 15 mm x 1 mm x 0.1 mm (in the case of planer shavings hammer-milled without screen).

The particles may be randomly oriented, which provides about equal properties in both directions, or aligned to improve stability and strength in the direction of the fibre alignment. The boards may be homogeneous through the thickness or multi-layered, usually three-layered, to give improved surface characteristics and mechanical properties [9].

Particle board is usually manufactured in thicknesses from 3 to 25 mm with the platen-pressed, mat-formed process; from 25 to 75 mm with the extrusion process; and from 2 to 9 mm with the continuous process.

The density range for what are considered medium-density particle boards is 650-810 kg/m^3. Densities above 810 kg/m^3 are considered high-density particle board; those below 650 kg/m^3 are considered low-density [4].

Adhesive binders are similar to those used for plywood, i.e. they can be phenolic, melamine/urea, copolymers or urea formaldehydes, depending on the intended end-use of the board.

3. Hardboard (fibreboard)

Hardboard is a panel manufactured from inter-felted ligno-cellulosic fibres that are consolidated under heat and pressure in a hot press to a density of 500 kg/m^3 or greater. Other materials may be added to improve properties such as stiffness, hardness and resistance to abrasion and moisture, as well as to increase strength, durability and utility [10].

Standard hardboard is usually between 800 and 1,000 kg/m^3 density, while oil-tempered hardboard is slightly denser, between 950 and 1,200 kg/m^3. Both panel types are usually manufactured in thicknesses from 3 to 13 mm [1].

Panels fabricated by the wet form method, which is used in Australia, are smooth on one side only and have a screened back. No binders to improve adhesive properties are used in this process.

4. Medium-density fibreboard

Medium-density fibreboard (MDF) is a panel product manufactured from ligno-cellulosic fibres combined with synthetic resin or other suitable binders. The panels are manufactured to a density of 500-800 kg/m^3 by the application of heat and pressure; in this process, the inter-fibre bond is substantially created by the added binder. Other materials can be added during manufacturing to improve certain properties [11]. With the drying process, which is normal in Australia and New Zealand, densities are between 600 and 800 kg/m^3 and thicknesses are between 3 and 50 mm. The adhesive binder is usually urea formaldehyde, which renders the bond unsuitable for structural applications. Researchers suggest that by adding phenolic- or tannin-based binders, a durable structural panel can be achieved [12].

5. Oriented strandboard

Oriented strandboard (OSB) is a specialized particle board product. It is manufactured from aligned strands of wood that are much longer than they are wide. Typical dimensions for an individual strand are 40 mm in length by 6 mm in width and 0.4 mm thick. OSB is usually a three-layer board in which the top and bottom layers are oriented in the direction of the panel length while the inside layer is aligned in the direction of the panel width. In essence, OSB configuration mimics that of plywood.

Usually the two outside layers each contain 25 per cent of wood particle and the centre layer contains 50 per cent.

OSB uses liquid phenolic resin at 5-6 per cent on a dry solids weight for weight basis. An average density value for OSB is 680 kg/m^3. The commonly available thicknesses for OSB range from 8 to 20 mm [2, 13].

6. Waferboard

As previously reported, waferboard is generally manufactured from low-cost secondary species available throughout North America. The most common species is trembling aspen; however, spruce and some lower density hardwoods are also used. The ability of the species to "flake" properly, as does trembling aspen, is a prerequisite for suitability in waferboard production. Radiata pine possesses fair flaking characteristics and may be significant for the Asian-Pacific region.

The particles used in waferboard production are generally larger than those used for other particle board types. Particles 40-75 mm square by 0.775-1 mm thick are standard. The boards can have the wafers randomly oriented or aligned depending on the properties of the board that are required. Normal waferboard has a thickness range from 12 to 25 mm and an average density of 660 kg/m^3. Waferboards are usually manufactured commercially using powdered phenolic resins [2, 14, 15].

7. Comply

Comply is the trade mark for a range of panels consisting of, basically, a phenolic-bonded particle board overlaid on both sides with veneer. The core may be randomly oriented particle board, OSB or waferboard. The fibres in the core may be aligned at right angles to the direction of the face veneer to give improved dimensional stability in the cross direction.

The panels can be made in a two-pass operation in which first the core is manufactured and then the overlay veneers are glued. One United States manufacturer is using a one-pass operation by pressing the veneers and particle mat at the same time. This company is using isocyanate adhesives. The density of Comply depends on veneer species and core type. From the literature [2], 630 kg/m^3 appears to be an average value for material produced from Southern pine. Presently, the thickness range in Comply appears to be between 12 and 25 mm.

C. Standards

Before comparing the physical and mechanical properties and applications of the wood-based panels described in the preceding section, it is necessary to comment on the types of standards under which wood-based panels have been manufactured and marketed and on current changes in the rationale for these standards. The reasons for change are an important issue that is tied to structural properties and application.

Until relatively recently, all wood-based panels were manufactured and marketed universally to prescription-type standards. Prescription standards are founded on the rationale that if the ingredients and manufacturing processes and methods are rigidly enforced, then the product will have known performance. United States and Australian standards for plywood [16, 10] and particle board [11, 17] are representative of this type of standard.

Prescription or manufacturing standards have been used reasonably successfully for structural timber and plywood in countries where a small number of species are used, e.g. the United States, Canada and Finland. In the Asian-Pacific region, owing to the multitude of available species (90 species are available to the Australian plywood industry, for example), precription standards are inherently conservative and restrictive. This conservatism arises out of the need to group species [18] so that unit stresses need to be applied to only a few species rather than a multitude of species. For example, AS 2269 [16] has unit stresses for eight stress grades rather than for the 90 individual available species. The grouping method leads to a lowest-common-denominator approach based on the properties of the weakest species in the group and is restrictive because every new available species must be tested so that it can be assigned to a group.

AS 2269 has endeavoured to overcome the restrictive and conservative nature of the prescription approach by including a performance section. The performance test is used to establish the stiffness (modulus of elasticity) of any plywood panel manufactured from any species or group of species.

Then, because of the well-established relationship between the modulus of elasticity and other basic strength properties, a grade mark can be applied. The grading mark has associated unit stresses prescribed in the standard. A stress grading machine has been developed to carry out the stiffness sorting. Although machine grading has reduced the conservative and restrictive nature of the prescription approach to applying unit stresses to Australian plywood, it must not be overlooked that its use is based on the relationship between modulus of elasticity and the other strength properties, which was established by a great number of tests.

Prescription standards were a reasonable basis for the standardization of reconstituted wood products as long as the panels were used for non-structural applications such as door skins or furniture, where the main consideration was finish, not structural performance. For the reasons substantiated previously in this paper, particle board, OSB, waferboard, MDF and Comply now are being aimed at structural markets, as alternatives to plywood, particularly in light framed construction. To facilitate the acceptance of these products into building regulations and to avoid being over-conservative or restrictive in their application, there is a school of thought in North America, Australia and New Zealand [19, 20, 21, 22, 23] that favours performance standards over the traditional prescription approach.

A performance standard is defined by the American Plywood Association [19] as follows:

> A performance standard is oriented towards the end use of the product and does not prescribe by what means the product will be manufactured. The overall objective is to assure, <u>for a particular end use</u>, that the product will satisfy the requirements of the application for which it is intended. To do this the performance criteria must address the critical attributes of the product that will assure successful performance in the market place. This necessitated the development of new and innovative test methods, each linked to field requirements. Therefore, under performance testing the qualification process correlates the attributes of the product to the market place.

To qualify under performance testing, a product is evaluated to ascertain its compliance with a series of performance requirements relative to a specific end use. The product must demonstrate, for example, the structural adequacy, stability and durability necessary to perform in service. Simultaneously, properties such as density, internal bond and strength retention are also measured. A product specification is written based on these latter properties, which can be easily measured on a sustained basis. Minimum values are set and a mill must, by means of internal quality control, make sure these values are achieved. The mill is independently audited on a regular basis to ensure, firstly, that its quality control system is operational and, secondly, that a sample of the product when tested still meets the desired performance criteria.

The American Plywood Association has performance standards for roof and wall sheathing and single-layer flooring. The Australian particle board industry has performance standard AS 1860 for single-layer flooring [23].

It must be emphasized that performance standards must be tied closely to a specific end-use and do not allow for the application of unit stresses.

There is no doubt, however, that it is possible to assign unit stresses to reconstituted wood-based panels using the prescription method or the machine-grading methods described above [24]. The problems confronting the end-use of this traditional approach are the large range of products available; the volume of testing that needs to be done by the industry; large coefficients of variation (C_v) across an industry, leading to low unit working stresses; the time required to carry out testing and statistically evaluate the results; and the continual introduction of new products.

If reconstituted wood products are to be truly structural products, unit stresses must be applied. One solution is to set up standard test methods and statistical bases for applying unit stresses, which can then be applied to specific products by individual companies [2]. This approach seems sensible. The following quotation sums up the thinking in North America and the structural use of reconstituted panels generally [2]:

> It is unclear whether dissemination of engineering design values for the new panels will be handled more expeditiously via the performance or the prescription route. The former will require each manufacturer to state and support design values for his product, while the latter could be managed so that single values could be used to represent the products of all manufacturers producing like products. Considerable work has been done towards the derivation of allowable stress values for panel products in support of the latter approach.

D. Structural properties

It became apparent from studying the references that unit stress values, sometimes termed basic working stresses, were available for plywood only. It therefore became necessary to compare the structural properties of the panel types listed in table 26 using mean values for modulus of elasticity and modulus of rigidity and average ultimate values for the remaining structural characteristics.

Table 27 gives an average or range of average values for modulus of rupture, modulus of elasticity, average ultimate tension, compression and panel shear strengths and shear modulus (modulus of rigidity). The values given in table 27 were extracted from many of the sources in the reference list. Where no information could be found, a dash is used. The values in the table are ultimate average mean values and are for purposes of comparison only. They must not be used for engineering design purposes.

It is interesting to note that internal bond strengths were not included, as the literature showed that the values published by FAO [1] needed no modification.

The values given in table 27 for fibre-aligned boards or plywood are parallel to the grain or direction of fibre alignment. The values for other random-oriented particle boards or fibreboards are parallel to the length of the panel.

Table 27. Comparison of structural properties
(Megapascals)

Panel type	Modulus of rupture	Modulus of elasticity	Average ultimate tensile strength	Average ultimate compressive strength	Average ultimate shear strength (panel shear)	Modulus of rigidity
Plywood a/						
Australian low-density (F8)	28.4	9 100	22.7	21.5	5.3	525
Australian medium-density (F14)	46.2	12 500	36.3	34.6	6.8	625
Australian high-density (F22)	72.5	16 000	56.0	54.4	7.6	800
United States West Coast	29.0	9 600	-	41.0	6.9	590
United States Gulf Coast	27.5	9 600	-	41.0	6.9	590
Particle board, random-oriented						
United States high-density	-	-	14-34	17-36	-	-
United States medium-density b/	17-22	2 750-4 000	8-16	9-21	6-10	1 200-1 600
United States low-density	-	-	6-8	3-10	-	-
European medium-density	14.6-28.5	2 500-4 000	7.3-12.7	10.2-16.5	5.8-7.6	970-1 230
Aligned, three-layer	25.4-55.8	9 500-11 000	11.7-21.3	15.2-26.0	6.5-11.2	-
Hardboard						
Tempered	45-75	5 000-6 500	26-54	26-41	16-20	-
Standard	30-65	3 000-5 000	21-41	12-41	12-16	-
Medium-density fibreboard	32-34	3 300-3650	17.2-21.4	20.0-24.8	-	-
Oriented strandboard (OSB)	34.4	9 600	c/	c/	c/	-
Waferboard	17.2	3 400-4 800		11.0-12.4	8.3-9.6	1 500
Comply	31.0	9 600	-	-	-	-

a/ The F designations refer to stress grade, as detailed in Standards Association of Australia (<u>Australian Standard 2269: Structural Plywood</u> (Sydney, 1979).

b/ Medium-density particle board in the density range 650-800 kg/m^3 is the most widely used in the United States.

c/ It is believed these properties would be similar to those of fibre-aligned three-layer particle board.

A ratio that is significant from the viewpoint of transport costs and ease of handling on site, which in turn is important in the building of residences and in form-work applications, is the structural properties of the materials per unit weight. As flexural properties are usually important criteria in wood-based panel utilization, table 28 details the strength-weight and stiffness-weight ratios* of the panels under consideration.

Table 28. Strength and stiffness to weight ratios

Product	Average density (kg/m^3)	Strength-weight ratio	Stiffness-weight ratio
Plywood			
Low-density	520	0.055	17.5
Medium-density	730	0.063	17.1
High-density	960	0.075	16.7
Particle board			
Random-oriented, medium	720	0.028	4.7
Fibre-aligned, medium	720	0.055	14.2
Hardboard			
Tempered	1 075	0.047	5.3
Standard	900	0.052	4.4
Medium-density fibreboard	700	0.047	4.8
Oriented strandboard	660	0.051	14.5
Waferboard	670	0.026	6.1
Comply	620	0.050	15.4

On the basis of structural properties per unit weight, it can be seen that plywood, aligned particle board, OSB and Comply have similar characteristics. Although waferboard has poor ratios for both strength and stiffness, it is popular because of its low cost: it is made from waste, and manufacturing installations in the United States and Canada are close to high population densities rather than near the forests, which reduces transportation costs.

Another major consideration in the structural utilization of any timber or wood-based panel is creep. Creep is the ability of a material to resist increases in deflection additional to the initial deflection while it is under sustained dead load. Steel does not creep, whereas all timber and wood-based materials do. The creep characteristics of timber and plywood are well established; in fact, a multiplying factor of 2 is applied to the short-term deflection for dry timber and plywood products carrying sustained loads [25].

*Strength-weight ratio = (modulus of rupture)/(average density); stiffness-weight ratio = (modulus of elasticity)/(average density).

Creep in wood is affected by conditions of exposure and cyclic humidity. It must be allowed for in any application of wood products involving sustained loads. Domestic floors that carry heavy refrigerators or roof sheathing that carries hot water services are examples of applications requiring the consideration of creep.

From the literature reviewed, it appeared that there was not a great amount of information available on the creep characteristics of reconstituted wood products. One source [26] does, however, compare the creep characteristics of three-layer particle boards having random core and face flakes aligned in the direction of the panel with that of Douglas fir and Southern pine plywood.

Because the particle boards were manufactured at Forest Products Laboratory, Madison, Wisconsin, it is assumed they were bonded with phenolic adhesive with 5-6 per cent phenolic used on a dry weight basis. Creep specimens were loaded for 90 days under both constant humidity, 65 per cent relative humidity at 32° C, and cyclic humidity, 25-85 per cent relative humidity at 26° C. The results are given in table 29.

Table 29. Creep behaviour of panel materials after 90 days loading

Panel type	Density (kg/m^3)	Creep deflection (additional percentage deflection) Constant humidity	Cyclic humidity
Aspen	745	62.2	228.6
Red oak	745	68.9	275.6
Red oak	795	61.5	248.2
Douglas fir plywood	-	43.8	191.2
Southern pine plywood	-	43.6	145.3

The table shows that although the plywoods have better creep properties than the particle boards, it is possible to evaluate the creep characteristics of reconstituted wood products by comparing them to a material, such as plywood, with well-defined creep characteristics.

E. Physical properties

The dimensional stability of wood-based panels under changes in moisture content, sometimes referred to as hygroscopic movement, is an important physical property because of its relevance to almost every application. The linear expansion and thickness swelling of each product should be known so that installation methods can be devised that allow for hygroscopic movement.

A good basis for comparison and design purposes is per cent movement for every 1 per cent change in panel moisture content. Values for a range of wood panels are given in table 30.

Table 30. Hygroscopic movement of various wood panels

Product	Movement (percentage per 1 per cent change in moisture content) Length	Width	Thickness
Softwood plywood, 3-ply a/	0.014	0.020	0.29
Softwood plywood, 9-ply	0.017	0.016	0.27
Mixed hardwood plywood, 3-ply	0.016	0.016	0.29
Mixed hardwood plywood, 9-ply	0.024	0.022	0.29
Australian particle board b/	0.039-0.055	0.039-0.055	0.38-0.61
United States particle board (random oriented) c/	0.046	-	0.28-0.75
United States particle board (aligned fibres) c/	0.015	0.056	0.80
Oriented strandboard d/	0.015	-	0.80
Waferboard d/	0.022	-	1.60
Comply d/	0.022	-	0.50

a/ CSIRO, Division of Building Research, Unpublished data (Melbourne, 1982).

b/ Random three-layered particle board, 650 kg/m^3 average of phenolic and urea bonded.

c/ From D. R. Countryman, "American Plywood Association composite panel research - an update", Proceedings of the 12th Particleboard Symposium (Washington State University, 1979); J. D. McNatt, "Properties of particleboards at various humidity conditions", Research Paper, FPL 225 (Madison, Wisconsin, United States Department of Agriculture, Forest Products Laboratory, 1974); and H. M. Montrey, Engineering Structural Panels for Light Framed Construction (Tacoma, Washington, United States of America, Weyerhauser, 1982).

d/ From J. D. McNatt, "Properties of particleboards at various humidity conditions", Research Paper, FPL 225 (Madison, Wisconsin, United States Department of Agriculture, Forest Products Laboratory, 1974).

No comparable data on the hygroscopic movement of hardboard and medium-density fibreboard could be found in the literature. However, movement at between 30 and 90 per cent relative humidity is reported by FAO [1]. In many applications in which wood-based panels are used, such as wall and roof sheathing and flooring, the thermal conductivity, k, of the panel is an important property. As heating and cooling costs become greater due to increased energy costs, construction techniques that utilize products providing both structural performance and insulation become more attractive. Wood-based panels can provide both of these characteristics.

Thermal conductivities for a range of wood-based panels are given in table 31.

It is believed that waferboard, OSB and Comply would have characteristics similar to those shown for medium-density particle board in table 31. For purposes of comparison, asbestos cement sheeting, a common building panel, has a thermal conductivity of 22 W/m^2/°C per 25 mm thickness.

Table 31. Thermal conductivities of wood-based panels

Product	Thermal conductivity, k ($W/m^2/°C$ per 25 mm thickness)
Plywood, softwood	4.5
Plywood, hardwood	6.4
Particle board, low-density	3.1
Particle board, medium-density	5.3
Particle board, high-density	6.7
Hardboard, standard	4.7
Hardboard, tempered	5.7
Fibreboard, medium-density	4.1

Source: United States Department of Agriculture, Forest Products Laboratory, "Thermal insulation", USDA Handbook No. 72 (Washington, D.C., 1974), chap. 20.

F. End-uses

It is intended finally to examine the suitability of each product for a range of end-uses and climate conditions experienced in the Asian-Pacific region. To do this, the author has selected climate conditions familiar to him [30] but which he believes cover the range of most Asian and Pacific climates. The information given in this section is based on the sources already cited and the author's experience in developing markets for structural plywood [31], [32], particularly in applications in domestic dwellings up to two storeys high. Table 32 details the four climatic conditions considered in the exercise.

Table 32. Climates prevalent in the Asian-Pacific region

Type	Australian example	Humidity	Average temperature (°C) Summer	Winter	Average annual rainfall (mm)
Tropical (cyclonic)	Darwin and Cairns	High all year	35	25	864 in 3 months
Subtropical (cyclonic)	Brisbane	High summer	25	16	1 132
Temperate (warm)	Sydney summer	High	26	8	1 200
Temperate (cool)	Melbourne winter	High	26	6	1 200

The author's opinions on the ability of each wood-based panel product to perform under the range of climates given in table 32 for applications in residential buildings are given in table 33.

Some products have been excluded from an application or limited in use to particular climates.

1. Internal fitments, wall panelling, door skins, furniture

Urea formaldehyde bonded panels are excluded in severe tropical environments because the bond is not durable under prolonged humid conditions [5, 33]. Particle boards with random fibre orientation are excluded under tropical conditions owing to their poor linear expansion and thickness swelling characteristics [27, 28, 29, 2].

2. Flooring

Urea bonded products are excluded from this structural application owing to the poor durability of the bond. Melamine-fortified urea formaldehyde bonded boards are excluded from tropical and subtropical climates owing to the lack of long-term durability under damp or humid conditions. Phenolic bonded particle boards are restricted from use in tropical climates because they are unstable under moisture content changes. OSB, waferboard and Comply are excluded from use in severe tropical climates because it is suspected that the American Plywood Association performance standards [19] are not stringent enough to apply to these climates; however, subsequent testing may yet show that these three products can be used under tropical conditions.

3. Flooring (wet areas)

Wet areas in a building include bathrooms and laundries. Because of the likelihood of continued wetting, boards bonded with other than phenolic adhesives are excluded regardless of climate on the basis that their bonding lacks durability under wet conditions. Under humid conditions, sub-floor ventilation must be increased to allow panels to dry out. Alternatively, plywoods or other wood-based panels used in wet areas should be treated against fungal attack. Phenolic particle board, OSB, waferboard and Comply are excluded for the same reasons they were excluded for normal flooring.

4. Wall sheathing (bracing)

Because wall sheathing is used to brace the house frame against horizontal forces set up by cyclonic winds or earthquakes, long-term structural integrity is essential. Panels with other than phenolic bonds are therefore excluded from this application regardless of climate. Standard hardboard is excluded from use in tropical and subtropical climates owing to moisture uptake and subsequent lack of panel durability and dimensional stability [13].

Medium-density phenolic bonded fibreboard was limited in use for the same reason. OSB, waferboard and Comply were excluded from use in tropical environments for the same reasons they were excluded from the flooring application.

5. Webbed beams

Plywood is the only material included for webbed beams because it is the only material for which unit stresses could be found [16]. To design an engineered fabricated product such as a web beam, unit stresses are essential.

Table 33. Suitability of wood-based panels for end-uses in residential buildings under the four climates detailed in table 32

		Application a/					
Product	Adhesive/binder	Internal fitments, furniture, door skins, wall panelling	Flooring	Flooring (wet areas)	Wall sheathing	Webbed beams	Cladding
Plywood	Urea formaldehyde	2, 3, 4	N/S	N/S	N/S	N/S	N/S
	Melamine urea formaldehyde	1, 2, 3, 4	3, 4	N/S	N/S	N/S	N/S
	Phenol formaldehyde	1, 2, 3, 4	1, 2, 3, 4	1, 2, 4	1, 2, 3, 4	1, 2, 3, 4	1, 2, 3, 4
Particle board	Urea formaldehyde	2, 3, 4	N/S	N/S	N/S	N/S	N/S
	Melamine urea formaldehyde	2, 3, 4	3, 4	N/S	N/S	N/S	N/S
	Phenol formaldehyde	2, 3, 4	2, 3, 4	N/S	2, 3, 4	N/S	3, 4
Hardboard	Standard	2, 3, 4	N/A	N/A	3, 4	N/S	N/S
	Tempered	1, 2, 3, 4	N/A	N/A	1, 2, 3, 4	N/S	1, 2, 3, 4
MDF	Urea formaldehyde	2, 3, 4	N/S	N/S	N/S	N/S	N/S
	Phenol formaldehyde	2, 3, 4	3, 4	N/S	3, 4	N/S	N/S
OSB	Phenol formaldehyde	1, 2, 3, 4	2, 3, 4	2, 3, 4	2, 3, 4	N/S	N/S
Waferboard	Phenol formaldehyde	1, 2, 3, 4	2, 3, 4	2, 3, 4	2, 3, 4	N/S	N/S
Comply	Phenol formaldehyde	1, 2, 3, 4	2, 3, 4	2, 3, 4	2, 3, 4	N/S	N/S

a/ N/S = not suitable; N/A = not applicable; 1 = tropical (cyclonic); 2 = subtropical (cyclonic); 3 = temperate (warm); and 4 = temperate (cool).

6. Cladding

The cladding application has been restricted to phenolic bonded plywood and tempered hardboard for reasons of durability. In both cases the panels would need to be treated against fungal attack with higher preservative loadings, and better penetration patterns would be required as conditions become more severe. It is recommended that plywood be treated with rot preventatives regardless of climate; such treatment is essential, however, in subtropical and tropical environments. In Northern Australia, plywood cladding is also usually treated against termites [34]. The author has observed the successful use of particle board cladding in temperate climates in South Africa and under test at the New South Wales Forestry Commission, Division of Wood Technology, at Sydney.

It is significant that although structural wood-based panels and their application in residential building result mainly from applied research efforts in developed countries such as the United Kingdom, United States, Canada and, more recently, Australia, there are cases where the technology has been transferred to developing countries with great success.

An example of this type of technology transfer occurred after Cyclone Melli caused extensive property damage and life loss in Fiji in 1979. With winds gusting to more than 200 km/hr, damage to traditional housing was extensive. In an effort to minimize future damage to housing, the Fiji Housing Authority, in close cooperation with a local timber and plywood manufacturer, developed a low-cost, cyclone-resistant dwelling along traditional lines using locally made plywood and timber. The structural design was based on specifications for cyclone housing in Australia, prepared by the Plywood Association of Australia [31].

The low-cost dwellings comprised a timber sub-floor, wall frame and roof frame. Phenolic bonded structural plywood was used as flooring. The walls were clad with 7 mm thick structural plywood that had been treated with copper-chrome-arsenic. The plywood provided wind bracing and hold-down resistance and also fulfilled its traditional function as cladding to keep the weather out. Plywood-webbed, timber-flanged box and channel beams were used over window and door openings to resist wind uplift, and treated plywood storm shutters were used over windows to protect occupants from flying debris.

In March 1980, the cost, including assembly, of an 8 x 5 m two-room dwelling was approximately $US 3,100. According to the Fiji Times of 14 March 1980, 98.5 per cent of the material from which the dwelling was built was made in Fiji; 572 man-hours were needed to produce the timber and plywood for a single dwelling and to erect it. Thus the dwelling under discussion used developed-country technology in a developing country to provide structurally improved, low-cost dwellings that used a very high percentage of local material and labour. Similar developments have occurred in other Pacific countries, for instance in Tonga after Cyclone Isaac.

7. Concrete form-work

An important end-use for plywood in Australia is in concrete form-work. Thirty per cent of Australian plywood production is used for this application. The design of plywood form-work is based presently on the availability of unit stresses and standardized section properties [35]. It would be possible to devise performance tests for this application, and this may allow other wood-based panels, particularly OSB, waferboard and Comply, to be used as plywood alternatives. The American Plywood Association was considering the development of such a performance standard for form-work in 1980 [36]. The use of wood panels for form-work would be independent of climatic conditions.

8. Other uses

Finally, there is a wide range of engineered applications in which plywood is already used [37], such as portal frames, containers, industrial beams, pallets, crates and industrial flooring, which may in the long term be open to the other structural wood-based panels described in this paper. These applications will only become available if unit stresses are applied to the products or if performance standards, tied to the specific end-use and climate in which the panels are to be used, are developed.

G. Conclusions

Several conclusions may be drawn from the above discussions:

(a) The continued development of new wood-based panel products will be aimed at structural applications;

(b) New types of products such as OSB, waferboard and Comply have been developed as plywood substitutes to utilize hitherto unused and therefore cheaper wood resources, e.g. aspen in Canada and spruce and low-density hardwoods in the United States;

(c) The technology exists to produce panels with characteristics similar to those of plywood. However, plant costs are higher than for plywood production and manpower requirements are lower. These are believed to be the primary considerations in Asian-Pacific decision-making;

(d) The types of panels produced in a country should depend on the cost and availability of raw materials, the technology available, the climate and the existence of building codes;

(e) Future structural applications of wood-based panels require either the development of unit stresses or the development of performance standards related specifically to one application (which should be related also to the climate in which the panel is to be used) or both;

(f) Research is required into the durability of new adhesives such as isocyanates and the performance of new formulations of existing adhesives such as phenolics, tannins and copolymers;

(g) Medium-density fibreboard will be used to an even greater extent in furniture production.

This paper has described the properties and applications of a wide range of panel products and has sought to give guidance on their application in the climates of the Asian-Pacific region. The end-use array was biased towards use in residential buildings because this use is believed to offer the best potential for utilizing forest resources to the advantage of the majority of people. It is hoped that the paper will provide a basis for decision-making relative to the best utilization of the different types of forest resource in the region, keeping in mind the available technology and the requirements of manpower utilization.

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VIII. STRUCTURAL PLYWOOD

Lam Pham and Robert H. Leicester*

A. Structural properties and design criteria for plywood structural components

Introduction

Structural plywood is usually fabricated from rotary-peeled veneers. The veneers are normally glued together so that the grains of adjacent layers are placed at right angles to one another and the peeler checks lie on the inner side of each layer. Plywood panels may consist of plies of different thicknesses, species and grades of wood. The veneers in plywood have slightly different structural properties from those of the original wood because (a) plywood veneer contains numerous peeler checks, which are partly filled with adhesive during the fabrication process; (b) the veneers are dried to a lower moisture content prior to fabrication; and (c) natural timber defects are dispersed and consequently are less detrimental to strength. Because the alternate veneers are laid with their grains at right angles, plywood sheets are much more nearly equal in strength in the longitudinal and perpendicular directions than solid timber. Naturally, this also means that in any direction, plywood is not as strong as solid timber stressed parallel to the grain. The other advantages of plywood are (a) its greater resistance to checking and splitting; (b) its higher shear strength in planes perpendicular to the veneers; and (c) its greater dimensional stability.

In a temperate climate, the equilibrium moisture content for a typical solid wood under normal indoor use is about 12 per cent and the moisture content of the wood in outdoor service may be higher than this, possibly around 15 per cent. However, plywood made from the same material would have an initial moisture content of only 10 per cent. As a result, plywood in service tends to be at a lower moisture content than solid wood and its structural properties are correspondingly improved. The effect of moisture content on the properties of plywood have been found to be very similar to that for solid wood.

The first section of this paper discusses the structural properties of plywood and the design criteria for structural components using plywood.

1. Structural properties of plywood

Basic strength and stiffness

The nature of a strip of plywood is such that the strength and stiffness of the plies that have their grain perpendicular to the direction of stress contribute little to the strength and stiffness of the panel. A good approximation to the structural properties can be achieved by considering parallel plies only. The structural properties of plywood can therefore be computed once the properties of the individual veneers are known. With this approach,

*Officers of CSIRO, Division of Building Research, Melbourne.

plywood can be considered as a collection of pieces of solid timber stressed parallel to the grain. Thus, it is appropriate to use the same basic stresses for plywood as are used for solid timber.

When the stress is at 45° to the grain of the face ply and when shear stress is considered, the parallel-plies-only approach is not appropriate; the calculation of stress is most conveniently based on the full cross-section and the relevant basic stresses are derived directly from experimental data on plywood.

Table 34 summarizes the methods used to calculate structural properties of plywood using the above approach. The following points are noted:

(a) Tension and compression strength. For the case of plywood with the grain at 45° to the applied stress, experimental data indicate that the tension strength is about 1/6 and the compression strength is about 1/3 of the strength of the same timber stressed parallel to the grain, compared with an expected value of 1/2.5. The low tension strength is probably due to fracture caused by the differential straining of adjacent veneers;

(b) Bending strength. It is difficult to calculate the bending strength of plywood because of complexities introduced not only by the layered nature of the material but also by the non-linear stress-strain characteristics of the wood in the compression zone. Consequently, it is usual in engineering codes to specify that the bending strength, M, of a plywood strip be computed according to some simple procedure such as the equation presented in table 34. The parallel-plies-only approximation is at its worst when three-ply plywood is used in bending with the grain of the face veneers perpendicular to the span;

(c) Shear strength. For plywood there are two shear strength properties that are important in structural design. One is the in-plane, or panel, shear strength and the other is the interlayer, or rolling, shear strength. The panel shear strength is reduced by the presence of peeler checks. This effect is greater for thicker plies and is compensated to some extent by glue infill into the checks. If a veneer contains splits wider than 1.6 mm, it is ineffective in panel shear. An approximate formula for the panel shear strength of plywood without splits is as follows [1]:

$$F_{sp} = (1.17 - t_v/6.35)F_s + 4.03(n - 1)/nt_v \qquad (1)$$

where F_{sp} is the panel shear strength of the plywood (MPa), F_s is the shear block strength of solid wood (MPa) and n and t_v are the number and thickness of the veneers (mm). A description of shear in the plane of the plies for various types of construction is given in table 35 and figures 72-74. The rolling shear strength of most structural plywoods is approximately the inter-fibre shear strength of the solid wood, but there is a slight increase when the veneers are less than 1.6 mm thick due to glueline effects;

(d) Stiffness. The effect of peeler checks in veneer is to reduce the panel shear stiffness to approximately 80 per cent of that of solid wood and the tension stiffness perpendicular to the grain to approximately 70 per cent of that of solid wood. The glueline makes a small contribution to stiffness only for plywood that is less than 1.2 mm thick [1].

Table 34. Structural properties of plywood

Property	Stress direction with respect to grain direction in face plies	Portion of cross-section to be considered in computing area for moment of inertia	Strength and stiffness in terms of basic values a/
Tension	Parallel or perpendicular	Parallel plies only b/	Basic stress for tension parallel to grain
	± 45°	Full cross-sectional area	0.17 x basic stress for tension parallel to grain
Compression	Parallel or perpendicular	Parallel plies only b/	Basic stress in compression parallel to grain
	± 45°	Full cross-sectional area	0.34 x basic stress in compression parallel to grain
Deformation in compression or tension	Parallel or perpendicular	Parallel plies only b/	Basic value for modulus of elasticity
	± 45°	Full cross-sectional area	0.17 x basic value for modulus of elasticity
Shear through thickness	Parallel or perpendicular	Full cross-sectional area	Basic shear stress
	± 45°	Full cross-sectional area	1.5 x basic shear stress
Compression perpendicular to face of plywood	-	Loaded area	Basic stress in compression perpendicular to grain

continued

Table 34 (continued)

Property	Stress direction with respect to grain direction in face plies	Portion of cross-section to be considered in computing area for moment of inertia	Strength and stiffness in terms of basic values a/
Strength in bending	Parallel or perpendicular	The bending moment M is computed as follows: $M = g_{19} F'_b (I/y_{max})$ where g_{19} = 1.20 for 3-ply plywood having the grain of the outer plies perpendicular to the span = 0.85 for all other plywood F'_b = Basic stress for extreme fibre in bending I = Moment of inertia computed on basis of parallel plies only y_{max} = Distance from neutral axis to outermost ply having its grain in the direction of the span	Basic stress for extreme fibre in bending
Deflection in bending	Parallel or perpendicular	Deflection may be calculated by the usual formulae, taking as the moment of inertia that of the parallel plies + 0.03 times that of the perpendicular plies	Basic value for modulus of elasticity
Shear deformation in plane of sheet	Parallel or perpendicular	Full cross-sectional area	Basic value for modulus of rigidity
Shear deformation through the thickness	Parallel or perpendicular	Full cross-sectional area	Basic value for modulus of rigidity

a/ For stress computation, basic values can either be ultimate or working values, depending on the design method.

b/ "Parallel plies" means those plies whose grain direction is parallel to the direction of principal stress.

Table 35. Shear in plane of plies

Type of construction	Position of shear	Area to be considered	Stress direction with respect to grain direction in face plies	Permissible stress a/
Plywood beams	Longitudinal shear between plies (figure 72)	Full shear area	Parallel or perpendicular	0.38 x basic shear stress
Box beams and I-beams with plywood webs	Shear between plies of web or between web and flange (figure 73)	Area of contact between plywood and flange	Parallel or perpendicular	0.19 x basic shear stress
			± 45°	0.19 x basic shear stress
Panels with plywood covers stressed in compression or tension or both	Shear between plies or between cover and framing members, when (a) depth of member exceeds twice its width and end nogging is used or (b) depth of member is not more than twice its width and no end nogging is used (figure 74)	Area of contact between plywood and framing member		
		Interior members	Parallel or perpendicular	0.38 x basic shear stress
			± 45°	Basic shear stress
		Edge members	Parallel or perpendicular	0.19 x basic shear stress
			± 45°	0.19 x basic shear stress

a/ In terms of basic allowable stress.

Figure 72. Longitudinal shear between plies

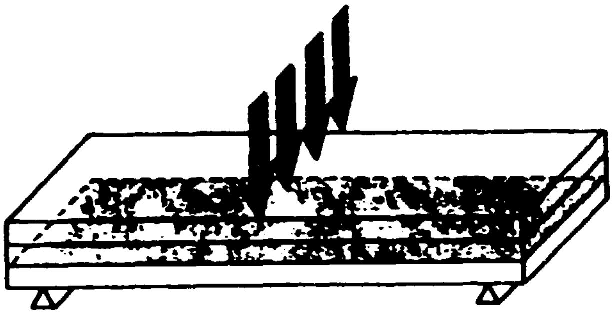


Figure 73. Shear between plies of web and between web and flange

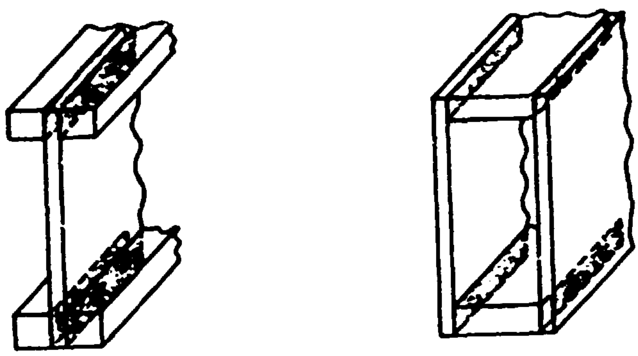
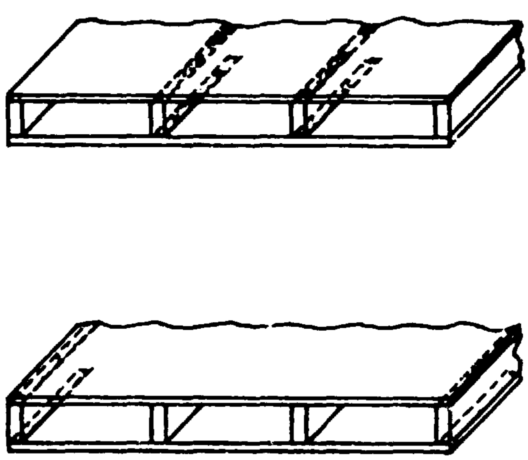


Figure 74. Shear between plies or between cover and framing members



The above presentation of the design information is the traditional North American and Australian method. The Europeans use a different approach, which gives working stresses related to the full cross-section of the plywood (e.g. British Standard 5268, Part II, 1988). With the full cross-section method, the calculation of geometrical properties is simpler but the working stresses differ for each thickness of plywood and different lay-up of veneers. Consequently, a greater number of tables must be used to present the design information.

Both methods, parallel plies only and full cross-section, are frequently used not only in design codes of practice but also in the presentation of experimental data. The reader is therefore advised to take note of the method used, particularly when comparing information from different sources.

Buckling strength

An important factor that has to be considered in the design of plywood components, particularly in engineered structural components, is the buckling of the panels under stress. The design procedure based on small deformation theory involves the use of the elastic buckling load. A good approximation for this load may be obtained from the use of simple variational methods, such as those described by Timoshenko and Gere [2]. These methods have been applied, with the plywood panel considered as an orthotropic plate by Lekhnitskii [3] and March *et al.* [4] to solve a variety of problems of plywood buckling. The plywood plate equations and their applications to buckling problems are summarized in a later part of this paper.

Experimental data indicate that the theoretical elastic buckling load is a good indicator of the start of the growth of large lateral deflection. This is not immediatly followed by failure. For most cases, such as in compression and shear, the panel is able to sustain loads much greater than the buckling value. Thus, in design practice, the allowable loads must be kept under the buckling loads to prevent excessive deformation rather than for strength reasons.

A convenient method of normalizing the information on buckling strength is to define two parameters, a stability factor K_u and a slenderness factor S_o, as follows:

$$K_u = F_u/F_{u(s)} \tag{2}$$

and

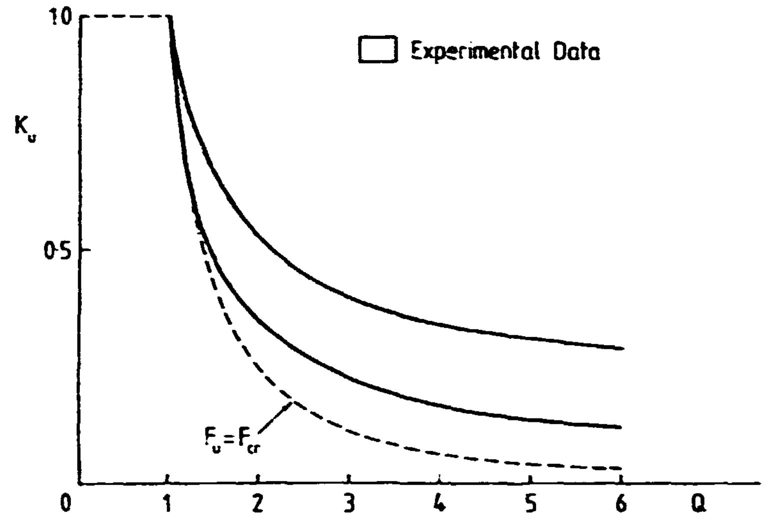
$$S_o = \sqrt{F_{u(s)}/F_{crit}} \tag{3}$$

where F_u is the ultimate value of the applied stress, $F_{u(s)}$ is the value of F_u for a completely stable system and F_{crit} is the elastic buckling stress of the system.

A typical example for plywood under compression is presented in figure 75. The additional strength above the elastic buckling load that may be carried by plywood panel elements is amply demonstrated in the figure. This additional strength can only develop with large deformations and can only be made use of if large deformations are acceptable. No theoretical estimate of post buckling strength is available except for the case of plywood web in shear and plywood web in compression. These estimates are given in section C of this chapter.

The derivation of allowable strength for plywood liable to buckling is identical in approach to the derivation of design buckling strength for solid timber components, which has been described in another paper.

Figure 75. Buckling of plywood plate under compression



2. Design criteria for structural components using plywood

Plywood as sheathing materials

Floors and roofs

Floors and roofs are usually designed for the most adverse effects from a specified concentrated load or a specified uniformly distributed load. The three main design criteria are strength, deformation and vibration. Strength is rarely the critical design criterion for plywood floors and roofs. The current Australian floor design criterion is based on the floor stiffness under a concentrated load. A stiffness of 0.17 mm per 100 N is considered adequate for domestic construction from both the deformation and vibration standpoints [5]. A comprehensive report on plywood composite panels for floors and roofs has been compiled by the American Plywood Association [6].

Shear walls

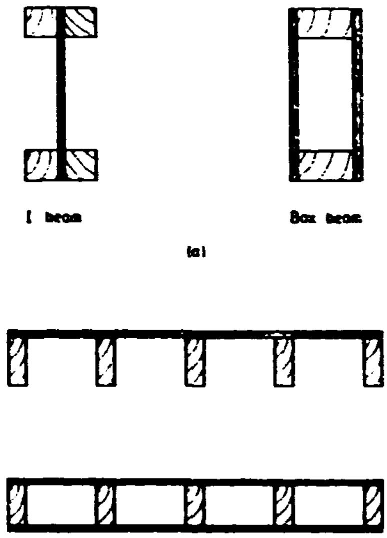
Plywood sheets, when attached to wall frames, form effective diaphragms to provide shear resistance. The common method of connecting plywood to the wall frame is by nailing. The racking resistance of this form of construction is mostly controlled by the lateral resistance of the nails. For tall panels, the racking deflection may control the design (usually limited to about 8 mm). For large panels with few studs in between, the buckling load of the panel may be the controlling parameter. As mentioned earlier, the design load should be kept below the buckling load to control the growth of lateral deflections on walls, which might cause serviceability problems.

Plywood in engineered structural components

Plyweb beams

For both I-beams and box beams, plywood is frequently used as the web material. It is connected with glue or mechanical fasteners to timber flanges and vertical timber stiffeners at supports and at intervals along the beam span (figure 76A).

Figure 76. Plywood in engineered structural components



(b) Single and double stress skin panels

Glued connections, if carried out properly, usually have adequate shear resistance at the interface to ensure complete interaction between the solid timber flange and the plywood web. For the normal design range, buckling of the plywood web under shear is not likely to be a problem. The shear capacity of the plyweb beam is in any case much larger than the buckling shear capacity of the web alone owing to the diagonal tension action of the web in the buckling range and the Vierendeel truss action of the beam flanges and stiffeners. For nailed construction, the load-deformation characteristics of the nail introduce another design criterion for both strength and deflection. This creates no problem for the estimate of strength, but deflection is a much more difficult problem. Booth [7] and Fageiri and Booth [8] have outlined methods to account for the flange-web joint displacement on the deflection of a plywood beam. In computing the deflection of plyweb beams, it is essential that the shear deflection be taken into account.

Stressed-skin panels

Plywood stressed-skin panels consist of plywood sheets glued or mechanically fastened to the top or to both top and bottom surfaces of longitudinal timber members. The assembly acts as an integral section to resist bending (figure 76B). The strength and deflection of a stressed-skin panel are dependent on the strength and rigidity of the joints between the plywood and the longitudinal members. Two special design considerations need to be taken into account: (a) the effect of shear lag on bending stresses and deflection and (b) the buckling of the plywood compression skin. The shear lag problem is only significant for relatively short span panels that have a span to clear spacing ratio of less than 8. The problem has been analysed by Foschi [9, 10], who proposed a correction factor to be applied to the bending stresses and deflection calculated from basic engineering formulae applied to the full section.

B. Plywood specifications

1. Material specifications

Plywood adhesives

The adhesives used in the manufacture of plywood are based on synthetic resins and are all thermosetting, including those used for cold-pressed plywood, in which application they are modified to cure at room temperature. As the name implies, the adhesives are cured or set by the action of heat and, once cured, are not replasticized by any subsequent heating. It is in this way that plywood adhesives differ from most conventional woodworking adhesives (polyvinyl alcohols, polyvinyl chlorides, contact adhesives and animal glues), which are thermoplastic. One cold-setting adhesive commonly used in the boatbuilding industry is resorcinol formaldehyde. This is a thermosetting adhesive.

The principal difference between the adhesives used in plywood manufacture is the degree to which they are waterproof. To ascertain the degree to which a glueline is waterproof, the Standards Association of Australia, under the close direction of the plywood industry and CSIRO, has defined the following bond tests, from Type A to Type D in descending order of performance:

Type of bond	Description	Required adhesive
A	72 hr boil (or 7 hr at 200 kPa steam pressure)	Resorcinol/phenol formaldehyde Phenol formaldehyde Tannin formaldehyde
B	6 hr boil	Urea/melamine formaldehyde
C	3 hr at 70° C	Low extended urea formaldehyde
D	16-20 hr cold soak, 20° C	High-extension urea formaldehyde

After soaking as prescribed above, the glueline is opened with a testing chisel and visually examined for wood failure. The basic requirement is that 50 per cent of the failure must occur in the wood for the sample to pass. If this standard is reached, the result is a plywood in which the glueline is always as strong as the parent wood.

Type A bond

Plywood manufactured to Type A bond has a glueline that will not deteriorate under the action of water or extremes of heat and cold. It will withstand long-term stresses without degrading in any way. Plywood manufactured to Type A bond therefore has a permanent, fully waterproof glueline that can be used under long-term stress in exposed conditions. It is readily recognized by the black colour of the glueline and the "Testing PAA Plywood" mark of the Plywood Association of Australia. Marine, exterior and structural plywoods have this Type A bond.

Type B bond

Type B bond plywood is incorporated within the exterior standard. However, owing to the adhesives used, this type of glueline will in time break down under the action of water or when placed under long-term stress. The glueline therefore cannot be termed fully permanent. For example, the standard suggests that Type B bond plywood can be used for concrete form-work with limited life expectancy.

Type C and D bonds

Plywoods manufactured to Type C and D bonds are for interior use only. Products made with this bond must not be recommended for exterior use or for structural applications involving long-term stresses, even in interior applications. The glueline can be readily recognized by its light colour and the "PAA Approved Interior Plywood" mark.

Plywood products

Some of the following standards directly cover the products made by the Australian plywood industry; others are related standards that are of interest to the consumers of plywood in Australia.

There are seven important publications of the Standards Association of Australia that refer directly to plywood and that are used in the industry.

AS 2269, Structural Plywood

AS 2269 specifies requirements for the construction, manufacture, grading and finishing of stress and surface grades of structural plywood. It specifies veneer qualities, bond quality, joints, dimensional tolerances, moisture content and basic working stresses.

The standard prescribes three different methods for the determination of stress grades for structural plywood:

(a) Species identification;

(b) Density determination;

(c) Mechanical stress-grading of the finished sheet of plywood.

Two surface grades, based on veneer quality of the face and back veneers, and one bond quality, Type A bond, are prescribed.

Appendices describe sampling, testing and acceptance, stress grading of plywood sheets, physical and mechanical data of structural plywood, stress grades for veneer of individual species and information to be supplied with enquiries and orders.

AS 2272-1979, Marine Plywood

AS 2272-1979 applies to plywood manufactured for use in the construction of marine craft. Permissible timber species used in the manufacture of this plywood are specified and details are given of quality of veneers, scarf joints and finger joints and manufacturing tolerances. The type of glueline is specified as Type A.

AS 2271-1979, Plywood and Blockboard for Exterior Use

AS 2271-1979 sets out requirements for the construction, manufacture, grading and finishing of plywood and blockboard intended for uses where it is exposed to the weather or damp conditions. It specifies veneer and core strip qualities, bond quality, joints, dimensional tolerances and moisture content. Two bond qualities are specified, Types A and B.

Exterior-grade plywood and blockboard may be grooved, prefinished, overlaid or preservative-treated and/or scarf-jointed by agreement between the purchaser and the vendor.

Appendices A, B and C of the standard describe, respectively, sampling, testing and acceptance; information required for enquiries and ordering exterior plywood and blockboard; and good practice for handling and storage of plywood and blockboard.

AS 2270-1979, Plywood and Blockboard for Interior Use

AS 2270-1979 specifies requirements for the construction, manufacture, grading and finishing of plywood and blockboard intended for non-structural uses where the material is fully protected from the weather or wet or damp conditions. It specifies veneer and core strip qualities, bond quality joints, dimensional tolerances and moisture content. Two bond qualities, Types C and D, are specified.

Interior-grade plywood and blockboard may be grooved, prefinished, overlaid or preservative-treated and/or scarf-jointed to suit individual needs.

Appendices A, B and C of the standard describe, respectively, sampling, testing and acceptance; information required for enquiries and ordering interior plywood and blockboard; and good practice for handling and storage of plywood and blockboard.

AS 2098, Methods of Test for Veneer and Plywood

AS 2098 describes in detail the various tests carried out on veneer and plywood. Methods for the determination of moisture content, bond quality, resistance of glueline to attack by micro-organisms, quality of scarf joints and depth of peeler checks are given.

AS 2097-1977, Methods for Sampling Veneer and Plywood

AS 2097-1977 describes procedures for the sampling of veneers and plywood and specifies the number of samples required.*

*Sampling for properties of preservative-treated plywood and veneer is set out in Standards Association of Australia, Australian Standard 1605-1974: Methods for Sampling and Analysis of Wood Preservatives and Preservative-Treated Wood (Sydney, 1974).

AS 2289-1979, Glossary of Terms Used in the Plywood Industry

AS 2289-1979 standard is a comprehensive reference manual on all terms used in the plywood industry.

Other Australian standards used by the plywood industry are as follows:

AS 01-1964, Glossary of Terms Used in the Timber Standards

AS 01-1964 defines technical and descriptive terms used or likely to be used in the Australian timber standards. This includes terms used in plywood standards.

AS 02-1965, Nomenclature of Australian Timbers

AS 02-1965 lists the timbers of Australia in alphabetical order of their standard trade names, gives their corresponding botanical name, the state in which they occur and other common names. This standard is valuable as there always is a certain amount of confusion over names of species.

AS 1604, Preservative Treated Sawn Timber, Veneer and Plywood

AS 1604 defines the hazard and gives the required penetration patterns and loadings of preservative retentions required for timber and plywood if they are to survive the hazard. This standard also covers the treatment of veneer and plywood to immunize it against lyctus attack.

2. Design specifications

The design rules for plywood are given in AS 1720-1975, Timber Engineering Code. The parallel-plies-only approach is adopted in this code. A working stress format is used and the basic values of stress are given as basic working stresses. The appendices allow a quick estimate of the permissible bending moment and stiffness of typical structural plywood as well as the buckling strength of plywood diaphragms. The theoretical bases of the buckling formulae in the code are given in sections C and D of this paper.

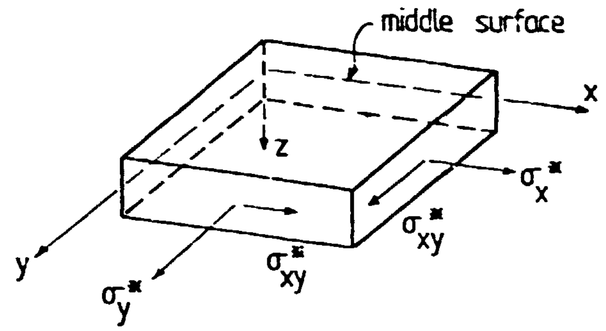
C. Plywood plate equations and examples of their application

Introduction

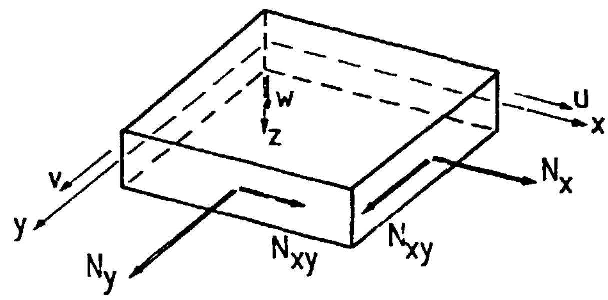
Lateral deflections and buckling strengths are usually the properties of plywood elements that are of most importance in structural design. In this section some variational equations that are useful for obtaining approximate estimates of these properties are derived. For simplicity, only plywoods with lay-ups that are symmetrical about the middle surface are considered.

The notation and sign convention used herein is shown in figure 77. The Cartesian coordinates are taken to lie in the middle surface of the plate along the axes of elastic symmetry, and the coordinate z is taken normal to this surface. The displacement of the middle surface in the x, y and z directions will be denoted by u, v and w. The forces acting on a plywood element of unit width and length are the membrane forces N_x, N_y and N_{xy}, the moments M_x, M_y and M_{xy}, the shears V_x and V_y and the lateral load, p.

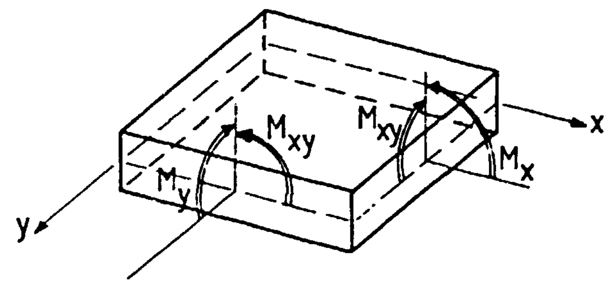
Figure 77. Sign convention for axes, displacement, forces and moments



(a) Coordinate system and stresses



(b) Displacements and membrane forces



(c) Bending and twisting moments

1. Plate forces and moments

It is shown in standard texts on elasticity that to the degree of approximation required, the strains e_x^*, e_y^* and e_{xy}^* at any point in the plywood plate are related to the displacements u, v and w by

$$e_x^* = \frac{\partial u}{\partial x} + \frac{1}{2}\left(\frac{\partial w}{\partial x}\right)^2 - z\frac{\partial^2 w}{\partial x^2}$$

$$e_y^* = \frac{\partial v}{\partial y} + \frac{1}{2}\left(\frac{\partial w}{\partial y}\right)^2 - z\frac{\partial^2 w}{\partial y^2} \qquad (4)$$

$$e_{xy}^* = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} + \left(\frac{\partial w}{\partial x}\right)\left(\frac{\partial w}{\partial y}\right) - 2z\frac{\partial^2 w}{\partial x \partial y}$$

and the corresponding stresses σ_x^*, σ_y^* and σ_{xy}^* are

$$\sigma_x^* = \frac{E_x^*}{\lambda}(e_x^* + \mu_{yx}^* e_y^*)$$

$$\sigma_y^* = \frac{E_y^*}{\lambda}(e_y^* + \mu_{xy}^* e_x^*) \tag{5}$$

$$\sigma_{xy}^* = G_{LT} e_{xy}^*$$

where

$$\lambda = 1 - \mu_{TL}\mu_{LT} \tag{6}$$

F_x^* and E_y^* are the Young's moduli in the x and y directions, respectively, and have the value E_L or E_T. E_L and E_T are the Young's moduli of elasticity parallel and perpendicular to the grain of the plies, respectively. μ_{xy}^* and μ_{yx}^* are the Poisson ratios for the plies in the x and y directions, respectively, and have the value of μ_{TL} or μ_{LT}. μ_{TL} and μ_{LT} are the Poisson ratios for the plies relative to the grain direction and are defined such that

$$E_L\mu_{TL} = E_T\mu_{LT} \tag{7}$$

The membrane forces and moments are defined by the following equations:

$$N_x = \int_{-h/2}^{h/2} \sigma_x^* \, dz$$

$$N_y = \int_{-h/2}^{h/2} \sigma_y^* \, dz \tag{8}$$

$$N_{xy} = \int_{-h/2}^{h/2} \sigma_{xy}^* \, dz$$

$$M_x = \int_{-h/2}^{h/2} \sigma_x^* z \, dz$$

$$M_y = \int_{-h/2}^{h/2} \sigma_y^* z \, dz \tag{9}$$

$$M_{xy} = \int_{-h/2}^{h/2} \sigma_{xy}^* z \, dz$$

where h is the thickness of the plywood sheet. Substitution of equations (2) into (5) leads to

$$N_x = H_1 e_x + g e_y$$

$$N_y = H_2 e_y + g e_x \tag{10}$$

$$N_{xy} = H_{12} e_{xy}$$

where

$$H_1 = \int_{-h/2}^{h/2} (E_x^*/\lambda)\, dz$$

$$H_2 = \int_{-h/2}^{h/2} (E_y^*/\lambda)\, dz \tag{11}$$

$$H_{12} = hG_{LT}$$

$$g = hE_L \mu_{TL}/\lambda$$

and e_x, e_y and e_{xy} are the middle surface strains obtained from equations (4), i.e.

$$e_x = \frac{\partial u}{\partial x} + \frac{1}{2}\left(\frac{\partial w}{\partial x}\right)^2$$

$$e_y = \frac{\partial v}{\partial y} + \frac{1}{2}\left(\frac{\partial w}{\partial y}\right)^2 \tag{12}$$

$$e_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} + \left(\frac{\partial w}{\partial x}\right)\left(\frac{\partial w}{\partial y}\right)$$

In plate analysis, it is convenient to use the parameters E_A, E_B. A_1, A_2 and A_3, defined by the following equations:

$$E_A = \left(\frac{\lambda}{h}\right) H_1 \tag{13}$$

$$E_B = \left(\frac{\lambda}{h}\right) H_2$$

$$A_1 = \frac{H_1}{H_1 H_2 - g^2}$$

$$A_2 = \frac{H_2}{H_1 H_2 - g^2} \tag{14}$$

$$A_3 = \frac{1}{2H_{12}} - \frac{g}{H_1 H_2 - g^2}$$

The substitution of equations (5) into (9) leads to

$$M_x = -D_1 \frac{\partial^2 w}{\partial x^2} - \alpha \frac{\partial^2 w}{\partial x \partial y}$$

$$M_y = -D_1 \frac{\partial^2 w}{\partial y^2} - \alpha \frac{\partial^2 w}{\partial x \partial y} \tag{15}$$

$$M_{xy} = -C \frac{\partial^2 w}{\partial x \partial y}$$

where

$$D_1 = \int_{-h/2}^{h/2} (E_x z^2/\lambda)\, dz$$

$$D_2 = \int_{-h/2}^{h/2} (E_y z^2/\lambda)\, dz \tag{16}$$

$$C = G_{LT} h^3/6$$

$$\alpha = E_L \mu_{TL} h^3/12\lambda$$

In plate analysis, it is convenient to use additional parameters E_1, E_2, D_3 and β, defined by the following equations:

$$D_1 = E_1 h^3/12\lambda$$

$$D_2 = E_2 h^3/12\lambda$$

$$D_3 = \alpha + C \tag{17}$$

$$\beta = D_3/\sqrt{D_1 D_2}$$

From equations (16) and (17),

$$D_3 = \frac{2\lambda G_{LT} - E_L \mu_{TL}}{\sqrt{E_1 E_2}} \tag{18}$$

2. Typical elastic parameters

Typical elastic parameters for plywood sheets are given in tables 36 and 37. The elastic constants used in the computation of these parameters were chosen to conform with the recommendations of AS 1720-1975 and are as follows: $E_T = E_L/30$, $G_{LT} = E_L/20$, $\mu_{TL} = 0.02$ and $\lambda = 0.988$.

Table 36. Elastic parameters for plywood with equal plies lay-up

No. of plies	E_A/E_L	E_B/E_L	E_1/E_L	E_2/E_L	β
3	0.6667	0.3555	0.9642	0.0691	0.4603
5	0.6133	0.4200	0.7989	0.2344	0.2745
7	0.5857	0.4476	0.7210	0.3123	0.2504
9	0.5704	0.4630	0.6765	0.3568	0.2418
11	0.5606	0.4727	0.6478	0.3855	0.2377
Ω	0.5167	0.5167	0.5167	0.2300	0.5167

Table 37. Elastic parameters for plywood with balanced plies lay-up

No. of plies	E_A/E_L	E_B/E_L	E_1/E_L	E_2/E_L	β
3	0.5167	0.5167	0.8792	0.1541	0.3228
5	0.5167	0.5167	0.6073	0.4260	0.2336
7	0.5167	0.5167	0.5569	0.4764	0.2306
9	0.5167	0.5167	0.5393	0.4940	0.2302
11	0.5167	0.5167	0.5312	0.2300	0.2300
Ω	0.5167	0.5167	0.5167	0.5167	0.2300

3. Field equations

For translational equilibrium in the x, y and z direction of the element shown in figure 77.

$$\frac{\partial N_x}{\partial x} + \frac{\partial N_{xy}}{\partial y} = 0 \tag{19}$$

$$\frac{\partial N_{xy}}{\partial x} + \frac{\partial N_y}{\partial y} = 0 \tag{20}$$

$$p + \frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} + N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} = 0 \tag{21}$$

and for rotational equilibrium about the x and y axes,

$$V_x = \frac{\partial M_x}{\partial x} + \frac{\partial M_{xy}}{\partial y} \tag{22}$$

$$V_y = \frac{\partial M_{xy}}{\partial x} + \frac{\partial M_y}{\partial y} \tag{23}$$

Equations (15), (17), (21), (22) and (23) lead to the following useful field equation:

$$D_1 \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = p + N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} \tag{24}$$

From equations (9), the following equation of compatibility is obtained:

$$\frac{\partial^2 e_x}{\partial y^2} + \frac{\partial^2 e_y}{\partial x^2} - \frac{\partial^2 e_{xy}}{\partial x \partial y} = \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 - \left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) \tag{25}$$

For the analysis of plate problems, it is convenient to introduce an Airy stress function, Φ defined as follows:

$$\frac{\partial^2 \Phi}{\partial x^2} = N_y$$

$$\frac{\partial^2 \Phi}{\partial y^2} = N_x \tag{26}$$

$$\frac{\partial^2 \Phi}{\partial x \partial y} = -N_{xy}$$

It is apparent that membrane stresses derived from this stress function will always automatically satisfy the equations of equilibrium (19) and (20). Substitution of equations (26) into equations (24) and (25) leads to the following field equations that are required for the solution of plate problems according to large deformation theory:

$$D_1 \frac{\partial^4 w}{\partial w^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = \tag{27}$$

$$p + \left(\frac{\partial^2 \Phi}{\partial y^2}\right)\left(\frac{\partial^2 w}{\partial x^2}\right) + \left(\frac{\partial^2 \Phi}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) - 2\left(\frac{\partial^2 \Phi}{\partial x \partial y}\right)\left(\frac{\partial^2 w}{\partial x \partial y}\right)$$

$$A_1 = \frac{\partial^4 \Phi}{\partial x^4} + 2A_3 \frac{\partial^4 \Phi}{\partial x^2 \partial y^2} + A_2 \frac{\partial^4 \Phi}{\partial y^4} = \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 - \left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) \tag{28}$$

In small deflection theory, the right-hand side of equation (28) is taken to be zero.

4. Boundary conditions

The mathematical formulation of boundary conditions that correspond to various practical plate edge conditions are to be found in texts on elastic plate theory. For a straight edge lying along the line $y = 0$, some typical boundary conditions associated with membrane forces are as follows:

$$N_x = N_{xy} = 0 \text{ (free edge)} \tag{29}$$

$$u = v = 0 \text{ (fixed edge)} \tag{30}$$

and typical boundary conditions associated with the bending moments are as follows:

$$w = \frac{\partial^2 w}{\partial x^2} = 0 \text{ (free edge)} \tag{31}$$

$$M_x = w = 0 \text{ (simply supported edge)} \tag{32}$$

$$M_x = V_x - \frac{\partial M_{xy}}{\partial y} = 0 \text{ (fixed edge)} \tag{33}$$

5. Potential energy equation

The strain energy of a deformed plywood plate, denoted by V_p, is defined by

$$V_P = \frac{1}{2}\int_{area}\int_{-h/2}^{h/2} [\partial_x{}^*e_x{}^* + \partial_y{}^*e_y{}^* + \partial_{xy}{}^*e_{xy}{}^*] \, dz \, dA \tag{34}$$

where dA denotes an elemental area of the middle surface. Substitution of equations (4) and (5) into (34) leads to

$$V_p = V_b + V_m \tag{35}$$

where

$$V_b = \frac{1}{2}\int_{area}\left[D_1\left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2\left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2\left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) + 2C\left(\frac{\partial^2 w}{\partial x \partial y}\right)\right] dA \tag{36}$$

$$V_m = \frac{1}{2}\int_{area}\left[H_1 e_x{}^2 + H_2 e_y{}^2 + 2g e_x e_y + H_{12} e_{xy}{}^2\right] dA \tag{37}$$

The potential energy, Ω_p, of the lateral load p, and potential energy, Ω_m, of the boundary membrane forces are as follows:

$$\Omega_p = -\int_{area} pw \, dA \tag{38}$$

$$\Omega_m = \oint [(N_y v + N_{xy} u)dx - (N_x u + N_{xy} v) \, dy] \tag{39}$$

If it is assumed that there is no stretching of the middle surface during bending, i.e. $e_x = e_y = e_{xy} = 0$, then

$$V_m = 0 \tag{40}$$

$$\Omega_m = - \;\text{area}\; \left[N_x \frac{\partial w}{\partial x}^2 + N_y \frac{\partial w}{\partial y}^2 + 2N_{xy} \frac{\partial w}{\partial x} \frac{\partial w}{\partial y}\right] dA \tag{41}$$

The method for the drivation of equation (41) from equation (39) is described in detail by Timoshenko and Gere [2].

6. Solution of the plate equations

The plate equations may be solved with the aid of computers by conventional numerical techniques, such as the finite difference or finite element methods. However, for sheets of simple geometrical shape, particularly rectangles, solutions may be more easily obtained by methods associated with mathematical series. These are discussed in detail in texts on elasticity. The following are examples of the use of variational methods to obtain approximate solutions.

Lateral deflections of plywood plates

For a plate loaded only by lateral loads, the membrane forces are negligible at small deflections, and from equation (21) the field equation is as follows:

$$D_1 = \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = p \tag{42}$$

and the appropriate boundary conditions are given by equations (31)-(33).

From equations (35)-(39), the correponding potential energy functional V is as follows:

$$V = \frac{1}{2} \int_{area} \left[D_1 \left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2 \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2\left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) + 2C\left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 \right] dA - \int_{area} pw \; dA \tag{43}$$

An approximate estimate of lateral defections can be obtained by choosing a reasonable deflection shape for w, substituting into equation (43) and then minimizing the functional V.

Elastic buckling of plywood plates

Only the small deformation (critical elastic) buckling of initially flat plates subjected to membrane forces applied at the boundry will be considered. From equation (24), the equilibrium equation for this is as follows:

$$D_1 \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} \tag{44}$$

and from equations (35) to (41), the corresponding increment in the potential energy functional V during lateral deformations is as follows:

$$\Delta V = \Delta V_b + \Delta \Omega_m \tag{45}$$

where

$$\Delta V_b = \frac{1}{2} \int_{area} \left[D_1 \left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2 \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2\left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) \alpha + 2C\left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 \right] dA \tag{46}$$

$$\Delta \Omega_m = -\frac{1}{2} \int_{area} \left[N_x \left(\frac{\partial w}{\partial x}\right)^2 + N_y \left(\frac{\partial w}{\partial y}\right)^2 + 2N_{xy} \left(\frac{\partial w}{\partial w}\right)\left(\frac{\partial w}{\partial y}\right) \right] dA \tag{47}$$

where w is the lateral deflection during buckling, and N_x, N_y, and N_{xy} are the membrane forces that are present just prior to buckling. The buckling condition is given by

$$\delta V = 0 \tag{48}$$

7. Examples of application

Deflection of rectangular plywood plates

As a first example, the variational method will be applied to compute the deflection of a rectangular plate laterally loaded by a concentrated load P at the point x_o, y_o. The plate, shown in figure 78a, has sides of length a and b. The deflection pattern of the plate will be taken to be approximated by

$$w = A_o \sin\left(\frac{\pi x}{a}\right) \sin\left(\frac{\pi y}{b}\right) \tag{49}$$

where A_o is a constant to be determined. The substitution of equation (49) into (43) leads to

$$V = A_o^2 \frac{\pi^4 ab}{8} \left(\frac{D_1}{a^4} + \frac{D_2}{b^4} + \frac{2D_3}{a^2 b^2}\right) - PA_o \sin\left(\frac{\pi x_o}{a}\right) \sin\left(\frac{\pi y_o}{b}\right) \tag{50}$$

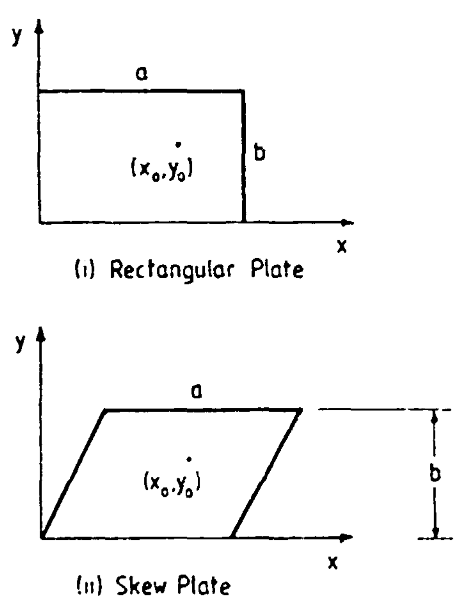
Hence the minimization

$$\frac{\partial V}{\partial A_o} = 0 \tag{51}$$

gives

$$A_o = \frac{4P}{\pi^4 ab} \frac{\sin\left(\frac{\pi x}{a}\right) \sin\left(\frac{\pi y}{b}\right)}{\left[\frac{D_1}{a^4} + 2\left(\frac{D_3}{a^2 b^2}\right) + \frac{D_2}{b^4}\right]} \tag{52}$$

Figure 78. Notation for plates



Single-term variational solutions of this type typically underestimate the true deflection by 5-20 per cent. However, they are simple to obtain and are extremely versatile. For example, if the plate is skewed to an angle θ as indicated in figure 78b, then an appropriate assumption for the deflected shape is as follows:

$$w = A_o \sin\left[\frac{\pi}{a}(x - \delta y)\right]\sin\left[\frac{\pi y}{b}\right] \tag{53}$$

where $\delta = \cot\theta$. This leads to the solution

$$A_o = \frac{4P\sin\left[\frac{\pi}{a}(x_o - \delta y_o)\right]\sin\left(\frac{\pi y_o}{b}\right)}{ab\chi\pi^4 \sin(\theta)} \tag{54}$$

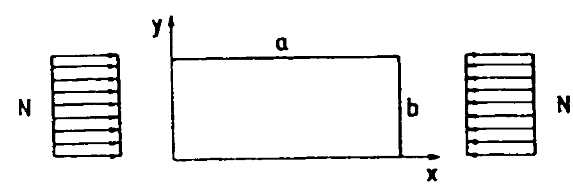
where

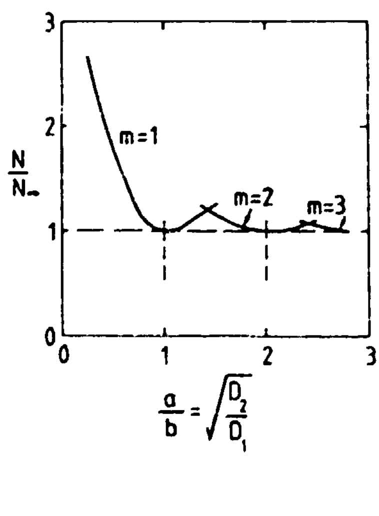
$$\chi = \frac{D_1}{a^4} + 2D_3\left[\frac{1}{a^2}\left(\frac{1}{b^2} + \frac{\delta^2}{a^2}\right)\right] + D_2\left[\left(\frac{1}{b^2} + \frac{\delta^2}{a^2}\right)^2 + \frac{4\delta^2}{b^2a^2}\right] \tag{55}$$

Uniform end compression

The variational method will be used to compute the buckling load of a rectangular plywood plate under uniform compression in one direction. A simply supported rectangular plywood plate of length a and width b is subjected to a uniform membrane compression N in the direction of the x-axis, as shown in figure 79.

Figure 79. Buckling of plywood plate under uniform compression





A suitable approximation for the deflection shape is

$$w = A_o \sin(\frac{m\pi x}{a}) \sin(\frac{n\pi y}{b}) \tag{56}$$

where A_o is a constant and m and n are integers to be determined. The substitution of $N_x = N$, $N_y = N_{xy} = 0$ and equation (56) into equations (45) to (47) leads to

$$\Delta V = \frac{abA_o^2}{8}\left[D_1(\frac{m\pi}{a})^4 + D_2(\frac{n\pi}{b}) + \frac{2\beta m^2 n^2 \pi^4}{a^2 b^2} - N(\frac{m\pi}{a})^2\right] \tag{57}$$

where $\beta = D_3/\sqrt{D_1 D_2}$. Hence the condition $\Delta V = 0$ gives

$$N = \frac{2\sqrt{D_1 D_2}}{b^2}(\gamma + 2\beta n_2 + \frac{n^4}{\gamma}) \tag{58}$$

where

$$\gamma = \sqrt{\frac{D_1}{D_2}}(\frac{mb}{2})^2 \tag{59}$$

Obviously the minimum value of N is given for the case m = 1, i.e.

$$N = \frac{2\sqrt{D_1 D_2}}{b_2}(\gamma + 2\beta + \frac{1}{\gamma}) \tag{60}$$

This equation is plotted in figure 79. A strip infinitely long in the direction of the x-axis, denoted by N_∞, is obtained from the condition $\delta N/\delta\gamma = 0$, which gives

$$\gamma = 1 \tag{61}$$

$$N_\infty = \frac{2\sqrt{D_1 D_2}}{b^2}(1 + \beta) \tag{62}$$

It is apparent from equations (59) and (60) that for an infinitely long strip, the plate buckles into panels of length a_{ch}, given by

$$a_{ch} = (D_1 D_2)^{0.25} b \tag{63}$$

This length will be called the characteristic buckling length.

For the case of a panel with the sides y = 0 and y = b clamped, and the sides x - 0, simply supported, a suitable deflection shape is

$$w = A_o \sin(\frac{m\pi x}{a})[1 - \cos(\frac{2n\pi y}{b})] \tag{64}$$

This leads to the following solution for an infinitely long strip:

$$N_\infty = \frac{8\pi\sqrt[4]{D_1D_2}}{b^2}\left(\frac{1}{\sqrt{3}} + \frac{\beta}{3}\right) \qquad (65)$$

$$a_{ch} = (16D_2/3D_1)^{0.25}b \qquad (66)$$

Single-term variational solutions of the type derived above typically overestimate the buckling strength by 1-10 per cent.

D. Strength of plywood plates

Introduction

This section summarizes the information relevant to the design of plywood in engineered structural components. It includes a summary of the formulae for elastic buckling loads of plywood plates under compression, bending and shear and a discussion on the ultimate strength of buckled plywood plates.

1. Formulae for elastic buckling loads

The definitions and notations contained in section C apply also here. In that section, the parameters E_1, E_2, D_3 and β were defined as follows:

$$D_1 = E_1h^3/12\lambda$$

$$D_2 = E_2h^3/12\lambda$$

$$D_3 = G_{LT}h^3/6 + E_L\mu_{TL}h^3/12\lambda$$

$$\beta = D_3/\sqrt{D_1D_2}$$

Uniform end compression

A simply supported rectangular plywood plate of length a and width b is subjected to a uniform membrane compression N in the direction of the x-axis, as shown in figure 79. The elastic buckling value of N is given by

$$N_{crit} = (\pi^2\sqrt{D_1D_2}/b^2)[\gamma + 2\beta + (1/\gamma)] \qquad (67)$$

where

$$\gamma = \sqrt{D_1/D_2}(mb/a)^2 \qquad (m = 1, 2, 3...) \qquad (68)$$

For a strip infinitely long in the direction of the x-axis, the elastic buckling load, N_∞, is given by

$$N_\infty = (2\pi^2\sqrt{D_1D_2}/b^2)(1 + \beta) \qquad (69)$$

It is apparent from equations (67) and (68) that for an infinitely long strip the plate buckles into panels of length a_{ch}, given by

$$a_{ch} = (D_1/D_2)^{0.25} b \qquad (70)$$

This length is called the characteristic buckling length. For the case of a panel with sides y = 0 and y = b clamped, the buckling load for an infinitely long strip is as follows:

$$N_{\infty} = (8 \cdot {}^{4}\sqrt{D_1 D_2}/b^2)(0.577 + \beta/3) \qquad (71)$$

and the characteristic buckling length is as follows:

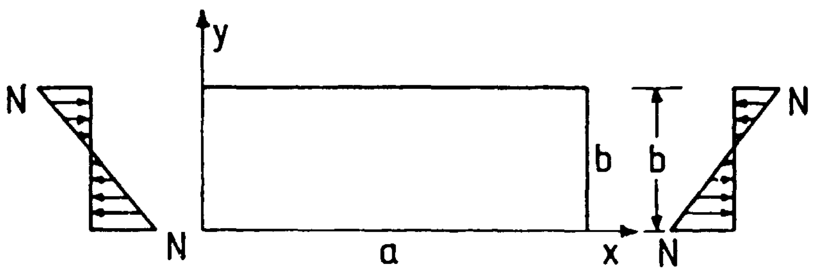
$$a_{ch} = (16D_2/3D_1)^{0.25} b \qquad (72)$$

Edgewise bending

A simply supported rectangular plate of length a and width b in (figure 80) is loaded in edgewise bending so that the membrane stresses prior to buckling are given by

$$\begin{aligned} N_x &= N[(2y/b) - 1] \\ N_y &= N_{xy} = 0 \end{aligned} \qquad (73)$$

Figure 80. Plate under edgewise bending



The value of the buckling force parameter N is as follows:

$$N_{crit} = (9 \cdot {}^{4}/32)(\sqrt{D_1 D_2}/b^2)\sqrt{[\gamma + 2\beta + (1/\gamma)][\gamma + 8\beta + (16/\gamma)]} \qquad (74)$$

where

$$\gamma = (mb/a)^2 \sqrt{D_1/D_2} \qquad (75)$$

The solution for an infinitely long plate is as follows:

$$N_{crit} = \frac{9\ {}^{4}}{32b^2} D_1 D_2 \sqrt{25 + 40\beta + 16\beta^2} \qquad (76)$$

and the characteristics buckling length is as follows:

$$a_{ch} = (D_1/4D_2)^{0.25} b \tag{77}$$

Edge shear

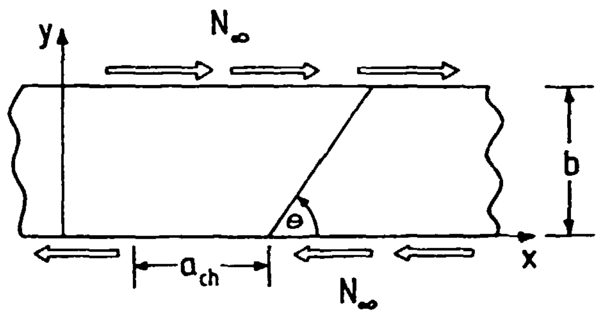
An infinitely long, simply supported strip of width b is subjected to edge shear force N_∞, as shown in figure 81. For these conditions, the plate buckles roughly in skew-shaped panels with the buckling force

$$N_{\infty crit} \cong (\pi^2 \sqrt{D_1 D_2}/b^2)(D_2/D_1)^{0.25}(3.66 + 2.0\beta) \tag{78}$$

and the characteristic buckling length

$$a_{ch} \cong (D_1/D_2)^{0.25}(1.0 + 0.22\beta)b \tag{79}$$

Figure 81. Plate under shear



Combined edge loads

For an infinitely long, simply supported strip of plywood subjected to a uniform end compression N_x, an edgewise bending with maximum value N_{bx} and an edge shear N_{xy}, an appropriate criterion for the onset of buckling obtained from the result of numerical analysis is

$$(N_x/N_{xo}) + (N_{xy}/N_{xyo})^2 + (N_{bx}/N_{bxo})^2 = 1 \tag{80}$$

where N_{xo}, N_{xyo} and N_{bxo} would be the elastic buckling values of N_x, N_{xy} and N_{bx}, respectively, if these forces were acting alone.

2. Ultimate strength of buckled plywood plates

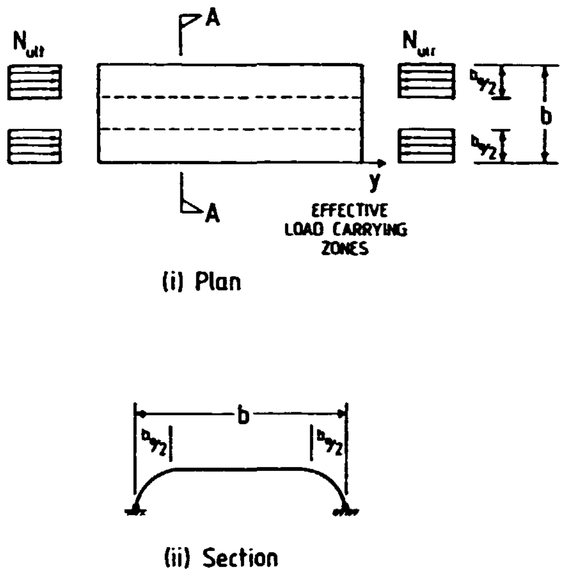
If large deformations are acceptable in a structural design, it is often very advantageous to make use of the additional strength above the elastic buckling load that may be carried by plywood plates. The necessary large deformation theory for this is outside the scope of this paper: however, two particular structural cases will be discussed to provide some insight on large deformation behaviour.

Strength of plywood plate in end compression

For the thin plate loaded in end compression, as shown in figure 82, large deformation theory shows that with increasing deformations the load tends to be carried by two edge strilps of width denoted $b_o/2$. If it is assumed that just prior to failure the membrance forces on the edge strip are $N_{u(s)}$, the maximum that would be possible for completely stable members, then the average membrane edge force, N_u, is given by

$$N_u = N_{u(s)}(b_o/b) \tag{81}$$

Figure 82. Ultimate strength of plate under compression



For the particular plywood sheet, the elastic buckling load is as follows:

$$N_{cr} = (\pi^2 \sqrt{D_1D_2}/b^2)k_o \tag{82}$$

Furthermore, if it is assumed that the plate acts effectively as a strip of width b_o in elastic buckling, then

$$N_u = (\pi^2 \sqrt{D_1D_2}/{b_o}^2)k_o \tag{83}$$

Hence

$$N_u/N_{u(s)} = b_o/b = \sqrt{N_{cr}/N_{u(s)}} \tag{84}$$

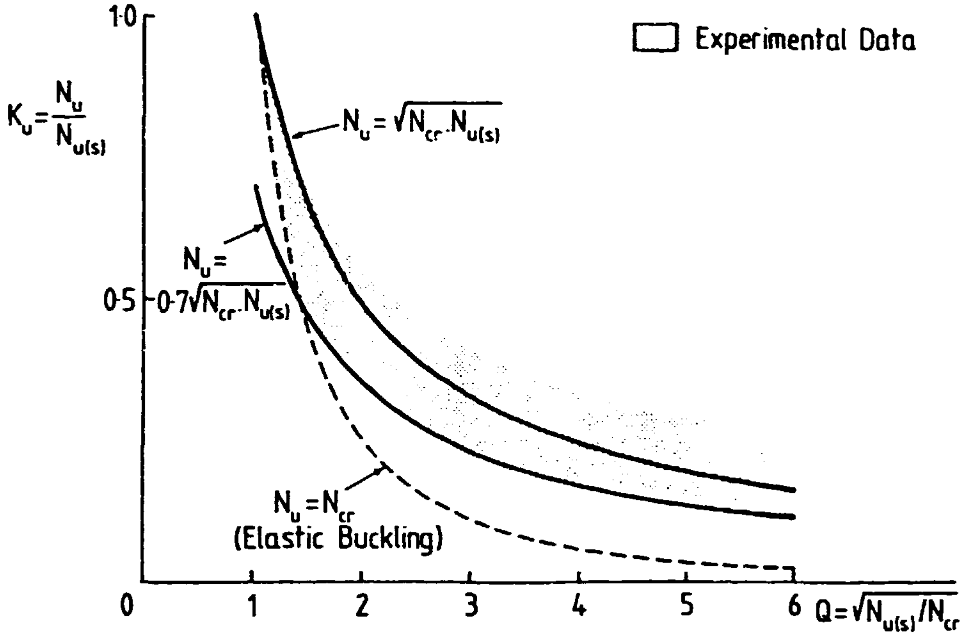
or

$$N_u = \sqrt{N_{cr}N_{u(s)}} \tag{85}$$

Equation (85) must be considered to be merely an estimate of the panel strength, not only because of the heuristic arguments used to arrive at it, but also because the magnitudes of initial imperfections and other important strength parameters have not been considered. Figure 83 is a plot of the results of a series of tests on plywood sheets by March et al. [4] It indicates that equation (85) is valid for extremely thin plates ($N_{cr}/N_{u(s)} < 0.1$), but that a more suitable formula for design is

$$N_u = 0.7\sqrt{N_{cr}N_{u(s)}} \tag{86}$$

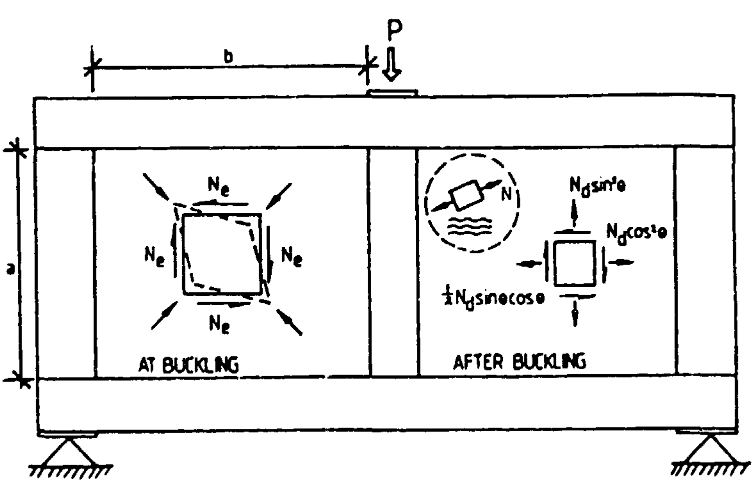
Figure 83. Experimental data for ultimate strength of plate under compression



Strength of plywood web in shear

Consider a slender plywood panel loaded in shear (figure 84). Prior to buckling, the web panel develops both tensile and direct compressive stresses. After buckling has developed, the web has no further capacity to carry extra compressive stresses, but further laod can still be carried by diagonal tension. Many models for this diagonal tension have been developed to predict the post-buckling load capacity but are not suitable for application to plywood web beams. For example, the partial diagonal tension model currently being used for steel plate girders has been found to be unsuitable because plywood webs do not display the required ductile yield plateau and because timber flanges are, relatively speaking, too thick to form plastic hinges. Of the various models available, the complete diagonal tension model appears to best fit the experimental behaviour observed in tests on plywood web beams, and this model will be used in this paper for the ultimate shear strength of plywood webs.

Figure 84. Ultimate strength of plate under shear



The ultimate strength, V_u, of a plywood web beam will be taken to be the lesser of the following:

$$V_{u1} = V_e + V_d + V_f \tag{87}$$

$$V_{u2} = V_s + V_f$$

where V_e is the elastic shear buckling load of a plywood web, V_d is the shear carrying capacity resulting from the diagonal tension strength of the plywood web, V_f is the contribution of the flanges and web stiffeners to the shear carrying capacity of the beam and V_s is the panel shear strength of a stable plywood web.

The method used to obtain the elastic buckling load V_e was described in section C. The stable shear strength V_s, which is the ultimate panel shear strength in the absence of buckling, was described in section A.

The interaction between the web and the flanges is complex, particularly in the post-buckling region and will not be discussed in detail herein. Roughly, the contribution of the flanges to the shear carrying capacity for the test beams, denoted by V_f, is estimated by assuming that the flanges act as simply supported beams with deflection matching those of the plywood web under shear.

Finally, the shear force V_d carried by a diagonal tension field N_d inclined at an angle θ to the flanges is given by

$$V_d = 0.5a.N_d \sin\theta \cos\theta \tag{88}$$

where N_d is the tension membrane force per unit length (figure 84).

The tension membrane force N_d has a maximum possible value given by

$$N_d = N_{t\theta} - N_{tE} \tag{89}$$

where $N_{t\theta}$ is the membrane force of the plywood in the direction θ and N_{te} is the membrane buckling force N_E resolved in the direction θ, i.e.

$$N_{tE} = 2N_E \sin\theta \cos\theta \tag{90}$$

The membrane force $N_{t\theta}$ can be obtained by fitting a curve to the following three points of known tensile strength (figure 85).

$$\begin{aligned} \theta &= 0 & N_{t\theta} &= kt_w f_{tu} \\ \theta &= \pi/4 & N_{t\theta} &= t_w F_{tu}/6 \\ \theta &= \pi/2 & N_{t\theta} &= (1 - k)t_w F_{tu} \end{aligned} \tag{91}$$

where F_{tu} is the ultimate tensile stress of the timber along the grain, k is the ratio of areas of effective plies to the gross area and t_w is the plywood web thickness.

The expression for V_d is taken so as to give the highest possible shear capacity. This is achieved when θ is approximately equal to 15°. For this case, equations (88), (89) and (90) lead to

$$V_d = 0.25a(N_{t\theta} - N_e/2) \tag{92}$$

Figure 85. Variation of strength with direction of tensile stress for plywood plate

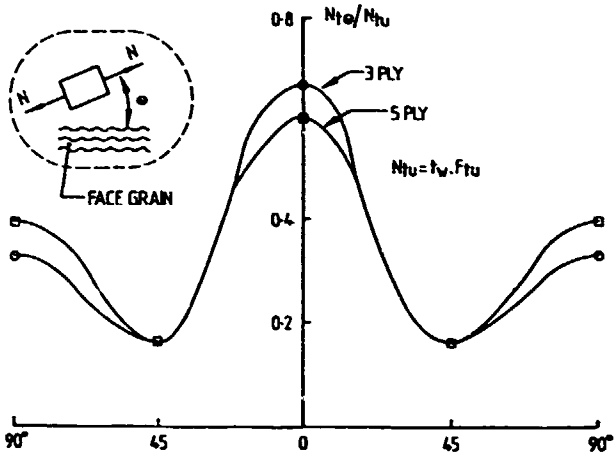
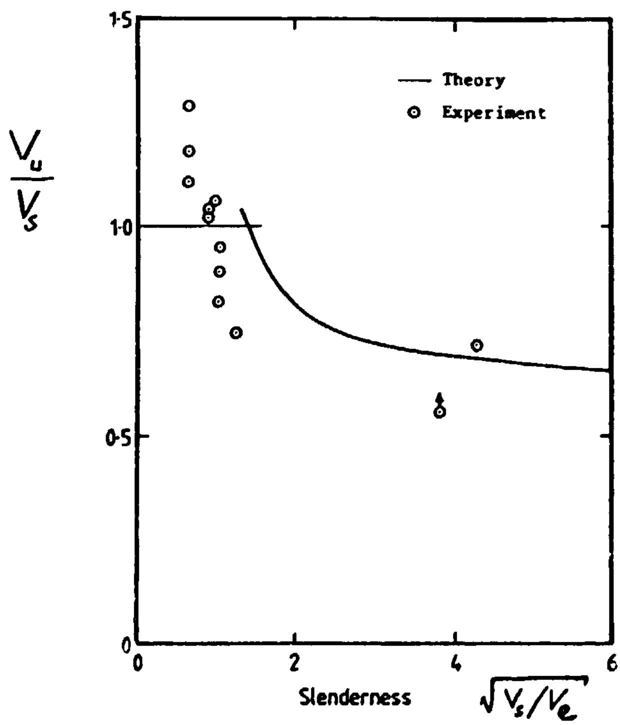


Figure 86 is a graph of the results of a series of tests on plywood web beams in shear. It indicates that the above method of estimating plywood web ultimate shear strength agrees fairly well with the available experimental data.

Figure 86. Ultimate strength of stiffened plywood webs



Source: R. H. Leicester and L. Pham, "Ultimate strength of plywood webs", Paper No. 145C, Proceedings of Pacific Timber Engineering Conference, Auckland, New Zealand, May 1984.

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IX. GLUED LAMINATED TIMBER

Robert H. Leicester*

Introduction

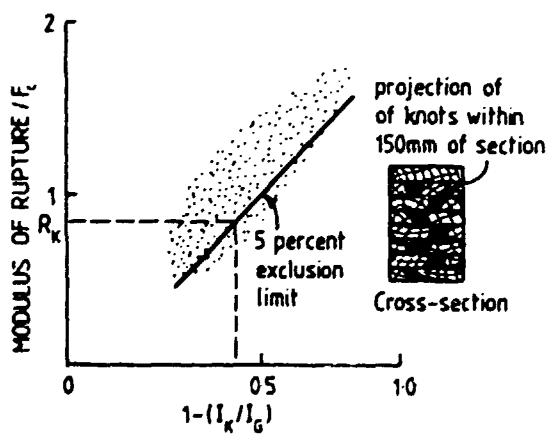
Glued laminated timber, or glulam, as it is often referred to, is formed by gluing together then laminae to form structural members, often of large size. In Australia, a great variety of species, including both softwoods and hardwoods, have been used for glulam. Each species has its own particular value in this regard. Softwood glulam tends to be easier to fabricate and therefore will often be the most economical type to use, while hardwood glulam is often favoured because of the specific properties of its timber, such as its appearance or durability.

The glulam laminae used commercially range in thickness from 50 mm down to veneer thicknesses of 5 mm or even less. In order to form laminae of sufficient length, particularly for large members, it is usually necessary to end-joint planks of timber. Because glulam members are fabricated from small elements, they can be of any size or shape. Members longer than 30 m and deeper than 2 m have been fabricated. Most commonly, members are rectangular in cross-section and are either straight or have a uniform curvature.

A. Strength theories for glulam

Early theories of glulam strength were related to the I_K/I_G concept illustrated in figure 87. The term I_K denotes the moment of inertia of the projected area of all knots within 150 mm of the cross-section under crutiny; the term I_G denotes the moment of inertia of the gross cross-section. The I_K/I_G concept is based on finding the correlation of strength with the parameter $1 - I_K/I_G$. The 5 per cent exclusion limit is then used to predict the characteristic strength R_K. By running surveys of knot sizes in timber boards, it is possible to predict the statistics of I_K/I_G values that would be expected for various glulam lay-ups.

Figure 87. Illustration of the knot ratio concept



*An officer of CSIRO, Division of Building Research, Melbourne.

The I_K/I_G concept has been used extensively [1] but is not particularly effective for covering a wide range of parameters [2]. Accordingly, other strength theories have been proposed. The following will contain a description of th theory used for AS 1720 [3]. This theory is based on the concept that the strength of glulam is equal to the strength of solid wood, with some slight enhancement for one of the following two reasons:

(a) There is a local reinforcement effect, wherein the weakness of a local defect is overcome to some extent by the assistance of clear wood in the laminae on each side of the defect;

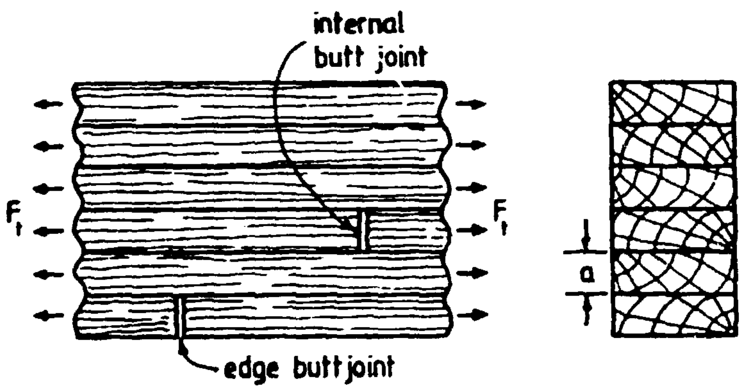
(b) There is a load sharing effect, wherein the five-percentile or characteristic strength of glulam is increased because the probability of occurrence of several weak laminae is less than with a single piece of solid wood of comparable weakness.

B. Theory of local reinforcement

1. Strength of butt-jointed glulam

A butt joint, illustrated in figure 88, is essentially the absence of an end-joint between pieces of timber that form a lamina. This type of joint is useful for illustrating the characteristics of local reinforcement, as without such reinforcement this type of joint would have no strength at all.

Figure 88. Butt joints in glulam



The strength of butt joints was discussed earlier. The applied load is stated in terms of a stress intensity factor D_I, which is given by

$$K_I = f_t(\pi a)^{0.5} \tag{1}$$

for internal butt joints. The applied nominal tension stress f_t is stated in MPa units and the lamina thickness a is stated in millimetres. For edge butt joint, the intensity factor is about 40 per cent greater than the value given by equation (1).

The critical stress intensity factor can be conservatively estimated from

$$K_{IC} = 0.15\rho \tag{2}$$

where ρ is the density of air dry timber in kg/m_3.

Equations (1) and (2) lead to the following value of stress to cause fracture:

$$f_{t(ult)} = 0.15c/(\pi a)^{0.5} \qquad (3)$$

If there are N butt joints in the zone of maximum stress, then a weakest-link situation applies, and the stress to cause fracture of the member is reduced by a factor N^{α}, where α is the coefficient of variation of the butt joint strength. Typically, $\alpha = 0.2$, so equation (3) should be modified to read

$$f_{t(ult)} = 0.15c/(\pi^{0.5}a^{0.5}N^{0.2}) \qquad (4)$$

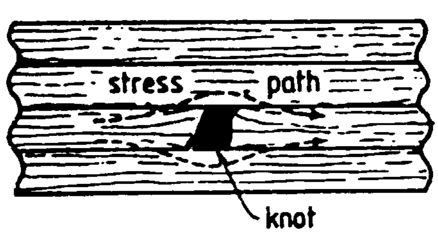
Equation (4) indicates that the nominal tension stress to cause failure increases as the lamina thickness a decreases. For a lamina thickness of a few millimetres, the tension strength is close to that of structural-grade timber; however, for lamina thicknesses in the range that is most frequently used in most fabrication, say 20-40 mm, the fracture strength may be less than one fourth of the value of the tension strength of structural timber.

As would be expected from the above, glulam fabricated from veneer is extremely strong [4, 5, 6, 7]. Such glulam is often referred to as "microlam" or "laminated veneer lumber".

2. Strength of continuous laminae

Figure 89 illustrates schematically the method whereby the stress in a lamina, interrupted by the discontinuity of a defect, may be carried by the adjacent laminae, provided these are straight-grained material that is defect-free. This is the same action as that associated with a butt joint; however, in the case of a natural defect, the reinforcement effect is more efficient as the defect is not as sharply discontinuous as a butt joint.

Figure 89. Local reinforcement of a defect

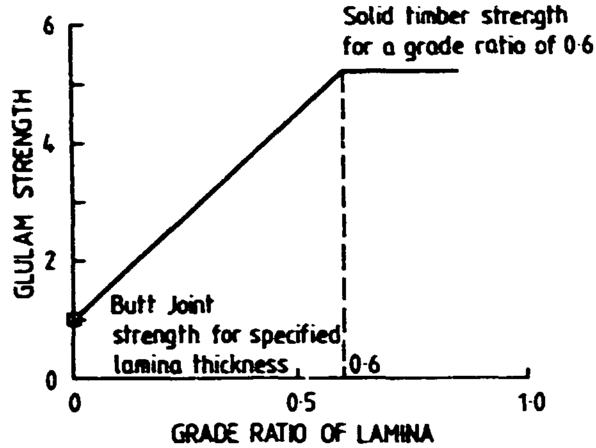


The method for assessing the strength of glulam is illustrated in figure 90. This evaluation is made in terms of the grade ratio of the timber used for the laminae. The grade ratio, GR, is defined as follows:

$$GR = R_{kg}/R_{kc} \qquad (5)$$

where R_{kg} is the five-percentile characteristic value of bending strength of the graded timber and R_{kc} is the characteristic bending strength of small, clear wood specimens.

Figure 90. Method of establishing glulam strength



Glulam strength is evaluated in the following steps:

(a) The strength of butt-jointed glulam is computed and taken to be the value relevant to a grade ratio of zero;

(b) The strength of clear structural timber is determined and taken to be relevant to a grade ratio of 0.6;

(c) The glulam strength for the grade ratio relevant to the laminae used is obtained by interpolation between the values for grade ratios of zero and 0.6.

The above applies to bending and tension strength. It is assumed that the reinforcement effect on compression strength is negligible.

Examples of the reinformcement factor specified in AS 1720 are given in table 38.

Table 38. Factor for local reinforcement

	Reinforcement factor a/			
Lamina thickness (mm)	Select engineering grade	Standard engineering grade	Select building grade	Standard building grade
50	1.00	1.10	1.15	1.20
20	1.00	1.15	1.25	1.40
5	1.00	1.25	1.45	1.70

Source: Standards Association of Australia, Australian Standard 1720-1975; SAA Timber Engineering Code (Sydney, 1975).

a/ Applicabale only to straight-grained softwoods.

C. The load-sharing effect

The load-sharing effect of laminae constrained to deform together will be discussed elsewhere. It is a function of the interaction of the load-deformation characteristics of randomly selected groups of laminae. The enhancement of the five-percentile strength depends on the species considered, and the following is a typical set of values:

Number of laminae	Load-sharing factor
1	1.00
2	1.14
5	1.26
10	1.33

In AS 1720 [3] the effective number of laminae for load-sharing estimates is specified as follows: tension member, N; column, 0.5N; and beam, 0.25N, where N is the total number of laminae in the member.

D. Stiffness of Glulam

It may be assumed that laminating does not affect the stiffness properties of the laminae. If the laminae are of mixed species or grades, then a conventional analysis using the transformed section technique may be used to evaluate the deflections of a glulam beam or the extension of a glulam tie.

E. Effect of curvature on bending strength

1. Effect on longitudinal stresses

The effect of longitudinal stress on strength is described in terms of a curvature factor, CF, and defined as follows:

$$CF = \frac{\text{Modulus of rupture of curved member}}{\text{Modulus of rupture of straight member}} \qquad (6)$$

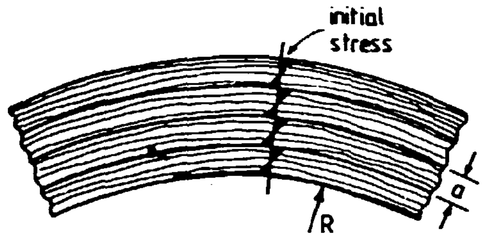
The effect of this curvature factor is illustrated in figure 91 and is based on the reports by Finnorn and Rapari [8] Hudson [9] Kostukevich and Wangaard [10] and Wangaard *et al.* [11].

For a radius-to-thickness ratio of 100, the bending during fabrication can introduce an initial stress of up to 40 per cent of the ultimate strength of the laminae. The effect of this is to give a curvature factor, as defined by equation (6), of about 0.8 immediately after fabrication; this relaxes to a value of 0.9 after a year or so.

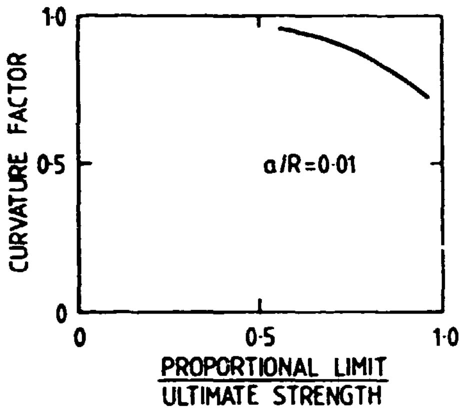
2. Effect on transverse stresses

As discussed elsewhere, the application of a bending moment to a curved member can introduce a transverse tension stress, which can lead to failure by fracture of longitudinal splits or checks. An examination of some typical member dimensions indicates that an effective curvature factor of about 0.8 would be obtained through such failures if the members were to contain through cracks of 20-30 mm length.

Figure 91. Effects of curvature on longitudinal strength



(a) Initial stresses due to fabrication



(b) Measured values of the curvature factor

F. End joints in laminae

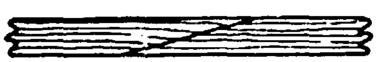
1. Types of joints

The three most commonly used end joints in commercial practice are the butt, finger and scarf joints illustrated in figure 92.

Figure 92. Types of end joints

(a) Butt Joint

(b) Finger Joint



(c) Scarf Joint

2. Butt joints

The characteristcs of butt joints have been discussed elsewhere. It has been found that in general it is uneconomical to place butt joints in tension members or within the bottom third of beams, because to do so would necessitate increased member sizes due to the poor strength of butt joints.

3. Finger joints

Currently this is the most successful type of end joint for commercial application. The machinery available for fabricating finger joints is quite effective and reliable. Typically the joints have a strength equal to that of structural timber having a grade ratio of about 0.4 and for design purposes may be considered to be equivalent to such timber.

If a finger joint fails to glue effectively, it acts as a butt joint, with a consequent dramatic drop in strength. There would appear to be no quality control technique that will guarantee that the quantity of defective finger joints is a negligible proportion, such as less than 1 in 10,000. Consequently, when finger-jointed laminae are used for critical structural members, it is recommended that every fabricated joint should be proof-tested to ensure its structural integrity.

4. Scarf joints

Correctly fabricated scarf joints have a strength close to that of clear wood, and these joints used to be strongly favoured by the aircraft industry. However, in the rough environments of factories that fabricate building products, it is difficult to fabricate satisfactory scarf joints have a scarf slope of less than 1 in 10. Furthermore, ineffective scarf joints act like butt joints, and the proportion of such joints tends to be larger than that found in the fabrication of finger joints. Consequently, the use of scarf joints is not generally recommended for the fabrication of critical structural members.

5. Spacing of end joints

The spacing of butt joints has a significant effect on the strength of glulam. This effect can be predicted accurately through the use of fracture mechanics theory and has been discussed elsewhere.

The spacing of scarf joints, if carefully fabricated, does not appear to affect the tension strength of glulam [12]. In theory, the spacing of finger joints should also have a negligible effect. However, in one experimental study, Stickler and Pellerin [13] found that the vertical stacking of finger joints lead to a loss in strength of 15 per cent. This was probably due to the fact that commercial finger-jointing machines cannot produce sufficiently perfect joints so as to completely prevent stress discontinuities across joints.

G. Fabrication of glulam

1. Fabrication methods

Glulam is fabricated with a great range of timber species. In general, the difficulties of successful gluing increase with the density of the timber. Cold-setting glues are the most commonly used glues. Three popular types are urea formaldehyde, casein and resorcinol. Urea glues are cheap but deteriorate in high-temperature and high-humidity environments, and some ureas have a questionable long-term integrity. Casein glues are excellent in dry environments, but in humid environments they are prone to attack by fungi. Resorcinol glues have the best all-round characteristics, including stability in exposed environments; however, they are also the most expensive of the three cold-setting glues mentioned.

Hot-setting glues such as the phenolics have excellent stability in exposed environments. They are usually cured by radio-frequency techniques.

Many timber additives, such as preservatives and fire retardants, create difficulties in gluing, and special fabrication techniques are necessary if these additives are impregnated prior to the laminating process.

2. Quality control

Notionally, quality control should comprise the following two components:

(a) Laboratory tests to assess the timber, the glue and the operating limits for successful fabrication;

(b) On-line checks of the fabrication process to ensure that the acceptable range of the operating parameters is not exceeded.

Parameters of significance include the species density, surface condition and moisture content of the timber; the mix temperature and pot life of the glue; and the temperature, pressure and time associated with the pressing operation. Many of these parameters are difficult to monitor continuously, so it is usual to also specify quality control checks on samples of the finished product.

H. Costs of glulam

The following example of costs, assessed by one particular manufacturer, may be taken as indicative of the distribution of costs associated with the fabrication of glulam.

Item	Relative cost
Basic costs	
Timber	55
Urea formaldehyde glue	5
Labour	25
Capital	15
	100
Extras	
End jointing	25
Resorcinol glue	15
Copper chrome arsenate treatment	15
Sealer and primer	5
Patching	20
Wrapping	5
	85

I. Durability of exposed structure

Since glulam is frequently used for large exposed structures, its performance in exposed conditions is a major concern. Experience has shown that even well-glued members can delaminate alarmingly.

Probably the most useful information on this matter is to be found in the paper by Huggins and Aplin [14] who surveyed 57 Canadian bridges fabricated from softwoods and up to 30 years old. Results of the survey incl... the following:

(a) Crecsote pressure treatment at a retention rate of 30 kg/m^3 proved to be completely satisfactory;

(b) Severe checking occurred if a moisture content differential of 5 per cent occurred between the surface and a depth of 60 mm;

(c) There was negligible checking on surfaces protected from wind and rain. A north-south orientation led to much worse deterioration than an east-west orientation;

(d) Copper chromearsenate treatment was associated with severe checking.

From the above, it would appear that the best protection against delamination and/or severe checking in exposed locations is obtained by impregnating the timber with an oil to deter the two-way movement of moisture through the surface of the glulam. Oil-based additives can be impregnated either by conventional pressure-treatment processes or by hot-and-cold bath methods. The latter process requires only the construction of a shallow, heatable bath and is particularly suitable for members of awkward shape that may not fit into commercial pressure-treatment cylinders.

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X. ADHESIVES FOR TIMBER

R. E. Palmer*

Introduction

Adhesive bonding (gluing) is but one of the processes used in the manufacture of certain structural components. Bonds can be categorized according to service requirements as structural, temporary structural or non-structural. A structural bond has the strength and durability of the wood being bonded. Thus, a structural design incorporating bonded components can be based on wood strengths, and the bond does not constitute a fault or weakness.

Bond testing is based on a comparison of bond strength with wood strength. Sample joints are subjected to a specified water soak treatment and then broken apart. Bond quality is expressed in terms of the area of broken fibre as a proportion of the total bonded area. The load at break is only of secondary importance. Sometimes the water soak treatment simply weakens the joint; in its more severe forms, it functions as an accelerated aging regime.

This paper presents what is essentially an Australian philosophy on structural wood gluing. The philosophy was developed over many years because of the need to bond a large number of hardwoods and softwoods of widely varying densities, primarily in plywood manufacture but also in glulam manufacture, where solid woods are glued. Such material can be put into service in a wide range of climates from Cairns to Alice Springs to Hobart. This need contrasts with waht in the developed countries of the northern hemisphere, where bonded structural components are manufactured from a rather smaller number of relatively gluing characteristics. The successful gluer in Australian industry tends to be somewhat of an experimenter who must be able to cope with a wide range of production variables, only some of which are under his control. Clearly, this does not so much apply to particle board production, where the process is more of a mass production process. Also, particle board finds only limited structural application, in its use as domestic flooring.

A. The adhesives

An adhesive is a substance capable of holding materials together by surface attachment. The materials held together by an adhesive are termed adherends. For many centuries the principal wood adhesive was animal glue, so that the terms gluing and adhesive bonding of wood have become synonymous.

The wood adhesives of greatest commercial importance are those based on formaldehyde, i.e. urea formaldehyde (UF), melamine formaldehyde (MF), phenol formaldehyde (PF), tamin formaldehyde (TF) and resorcinol formaldehyde (RF). Also of importance are those based on polyvinyl acetate (PVA), which have largely replaced UF and animal glue in furniture and interior joinery applications. Elastomerics are used primarily for interior panel and trim attachment but are likely to become more important in enhancing the stiffness of structural assemblies. Epoxies are expensive and can vary widely in performance because of the large number of possible formulations; they tend to be used in

*An officer of CSIRO, Division of Building Research, Melbourne.

special applications where cost is not a major consideration. These wood adhesives are dealt with in United States Department of Agriculture publications [1], [2].

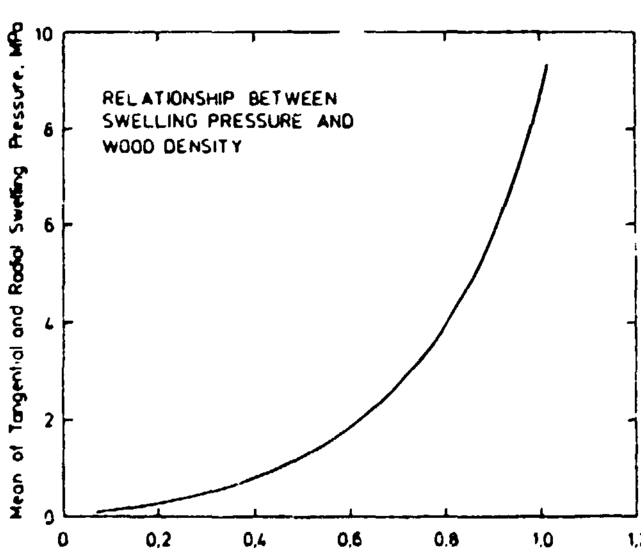
B. Wood as an adherend

Certain characteristics of wood make it a unique adherend. These have been discussed in some detail in previous papers and so are only briefly summarized below.

Owing to the orientation of its constituent fibres, wood has properties that are highly directional. It is hygroscopic, i.e. its moisture content depends on the temperature and relative humidity of the surrounding air. Both its dimensions and its propensity to absorb water are dependent on its moisture content.

Wood is porous. The art of gluing lies in being able to control the flow of glue into the first few layers of cells just prior to cure. Because of its porosity, wood offers the glueline little protection from the surrounding environment. Wood density varies because the degree of porosity varies. Indeed, it can vary by a factor of about 10. Strength properties generally increase with density. The swelling force, i.e. the force needed to prevent the expansion of wood during a moisture content increase, also increases with wood density (figure 93). The glueline constitutes a link in a mechanical system. The external forces capable of being applied to the glueline and the internal forces generated as a result of wood moisture content fluctuations both increase with wood density. Since porosity decreases with an increase in density, it becomes more difficult as density increases to produce bond strengths that match wood strengths increases.

Figure 93. Relationship between swelling pressure and wood density



Source: R.S.T. Kingston and T. Perkitny, "On the relationship between the active swelling pressure of wood and passive compressibility by external forces", *Holz als Rohstoff und Werkstoff*, vol. 30, No. 1 (1972), pp. 18-28.

C. Durability considerations

The sue of adhesives in building construction depends on a thorough knowledge of their long-term durability. Exposure trials carried out by Knight [3] over a period of 35 years have provided what still constitutes the most useful and extensive body of data available on wood adhesive durabilities. These data are summarized in table 39.

Table 39. Performance of various types of wood adhesive in terms of end-use environment and actual or estimated life

Chemical type of adhesive	Exposure and performance		
	Full exterior	Semi-exterior and damp interior	Dry interior
Resorcinol and phenol resorcinol formaldehyde	Expected life 25 years	Indefinitely long	Indefinitely long
Melamine-fortified urea formaldehyde	Fails in 5-10 yr	Estimated life of 10-20 yr	Indefinitely long
Urea formaldehyde	Fails in 2-5 yr	Fails in 5-10 yr	Indefinitely long
Casein	Fails in 1-2 yr	Fails in 2-5 yr	Indefinitely long
Animal	Fails in a few months	Fails in 1 yr	Indefinitely long

In these trials, a range of test specimens was used, with plywood being the predominant type. The effective life of the bond was taken as the time to reach a certain degree of delamination. While this end point is obviously beyond what is acceptable structurally, the data indicate that durability is relatable to the chemical nature of the adhesive and the environment to which the bond is exposed. On the basis of these and other data, there is only one type of chemical adhesive that can be used in any structural application where it is possible to use wood. This is the phenolic type.

Two other adhesive types can be used in the unlikely event that it is possible to guarantee that the bond will not ever in the life of the structure be subjected to high or widely flucturating relative humidity. The first is casein which has an excellent reputation for structural integrity in interior conditions in temperate climates. It can be used in wooden aircraft, but there are strict design requirements that all joints must be self-draining and that the aircraft must be kept in a hangar when not in use. When the casein is saturated with water, its bond strength can fall to as low as 20 per cent of initial dry strength. This is largely recoverable provided that drying takes place within a short time. Prolonged high humidity conditions lead to breakdown by biological attack. While attempts have been made, with varying success, to reduce the susceptibility of casein adhesives to biological breakdown by adding various preservatives, the service restrictions remain. The other type of adhesive is that based on melamine. Straight melamine formeldehyde adhesives have limited application because they have a very short shelf-life and are less durable than straight phenol formaldehyde adhesives, even though they are similar in cost and have similar high-temperature curing requirements. Their use in Austialia is largely confined to the production of Class I particle board. A more common form is a melamine-fortified urea formaldehyde adhesive, which can have a long shelf-life, is much cheaper than the equivalent phenolic type and can be cured under ambient conditions. Its breakdown mechanism is by hydrolysis, probably accelerated by the fluctuating stresses associated with normal in-service changes in moisture content.

Since large variations in formulation are possible within any chemical type, how then does one select a potentially durable formulation and know that it is being used so as to produce a durable bond? In practice there are two types of test. The first is a simple strength test, usually in shear or cleavage. The second is a strength test after some accelerated aging regime that has been shown from long-term durability trials to reduce strength in a way related to the reduction in strength that would occur in service. The test requirements for plywood adhesives recommended for the Australian standard are given in table 40. The test for plywood is a cleavage test in which bond strength is compared with wood strength. On cleavage, the break must occur predominantly in the wood, regardless of wood density. The ability of other adhesives to pass these tests is no guarantee of their long-term durability. The lack of a similar set of tests for other adhesives such as epoxies, polyurethanes, cross-linked, polyvinyl acetates,, acrylics etc. rules out their use in structural applications at the present time.

D. Phenolic adhesives

In view of the fact that only adhesives of the phenolic type can be used in unrestricted structural applications, the remainder of this paper will be largely confined to this type, with some mentioned of the closely related melamine type. The traditional phenolic adhesives are the straight alkali-catalyzed phenolic formaldehydes for use in hot press applications such as plywood or particle board manufacture. Acid-catalyzed phenolics can be cured at room temperature, but the residual acid continues to degrade the wood and can lead to early failure. Resorcinol is closely related chemically to phenol but reacts much more rapidly with formaldehyde. Resorcinol formaldehyde adhesives can thus be cured at ambient temperatures and so be used for the manufacture of glulam, finger-jointed material and various fabricated components. Wattle tannin is a natural polyphenol, and tannin formaldehydes can be used where straight phenol formaldehydes are used. Resorcinol is an expensive chemical and is often partly replaced by tannin or phenol. However, phenol resorcinol formaldehyde adhesives (PRFs) require a higher curing temperature than straight resorcinol formaldehydes. Of the formulations available in Australia, the minimum glueline temperature for the cure of resorcinal formaldehydes is taken as 20° C; for a PRF, it is 40° C. The formulation of adhesives is a science in itself and well outside the scope of most end users.

E. Testing and durability predication

Experience with phenolics over many years has shown that bonds prepared under the same conditions as test pieces that exhibit high wood failures both initially and after a 72-hr boil can be expected to remain structurally sound for a least 25 years even under more adverse conditions, in which case the durability of the wood itself is more likely to be the limiting factor. These three requirements - chemical type, 72-hr boil and high wood failure - are the basis for long-term durability prediction. In the case of plywood, the 72-hr

Table 40. Australian plywood bond type classification

Bond type	Service condition	Adhesive type b/	Water soak treatment (prior to testing to Rule 7.2 of AS 2098.[illegible]) Temperature (°C)	Time
A a/	Type A bonded plywood panels can be used in applications involving extreme long-term exposure to weather or wet or damp conditions and/or long-term structural performance requirements without glueline breakdown or glueline creep. Examples: structural plywood flooring, highway signs and marine plywood.	Synthetic phenol formaldehyde, natural polyphenol formaldehyde or mixtures thereof	100	$72 \, {}^{+1}_{-0}$ hr or 6 hr at 200 hPa steam pressure
B	Type B bonded plywood panels can be used in applications involving short-term (not more than two years) exposure to weather or wet or damp conditions and/or short-term structural performance without glueline breakdown or glueline creep. They can also be used for applications involving long-term exposure to protected exterior non-structural environments. Examples: concrete form-work and exterior door-skins.	Melamine urea formaldehyde	100	$6 \text{ h} \, {}^{+5}_{-0}$ min
C	Type C bonded interior plywood panels can be used in applications involving full protection from the weather or wet or damp conditions. They can be used in applications involving long-term exposure to generally high humidity and short-term exposure to extremely high humidity. Examples: interior panelling in geographically locations that have prolonged periods of high humidity, and panelling in bathrooms.	Urea formaldehyde	70 ± 1	$3 \text{ h} \, {}^{+5}_{-0}$ min

continued

Table 4C (continued)

Bond type	Service condition	Adhesive type b/	Water soak treatment (prior to testing to Rule 7.2 of AS 2098.2)	
			Temperature (°C)	Time
D	Type D bonded plywood panels can be used in interior applications fully protected from the weather or wet or damp conditions. They can be used for interior applications involving long-term exposure to medium humidity with occasional exposure to high humidity. Examples: furniture and interior wall panelling.	Extended urea formaldehyde	15-20	16-24 hr

a/ Experience has shown that Type A bonded plywood will withstand complete exposure to the weather for 20 years or maintain its integrity in structural situations for 50 years without glueline breakdown or glue-line creep.

b/ With adhesives of other chemical types, compliance with the test requirements alone would not indicate equal durability, and confirmation by actual service trials would be required.

boil test is used both as a quality control test and an accelerated aging test. In glulam or finger joint manufacture (and in plywood in the United States), a vacuum-pressure soak treatment (VPS) is applied to the test specimen to provide a slightly more severe intial or quality control test. In a VPS treatment, the specimen is immersed in water in a pressure vessel. A vacuum is applied to withdraw the air from within the wood and then a pressure is applied in order to bring about saturation. Times and pressures vary with different standards. This has the effect of placing some stress on the glueline and possibly eliminating hydrogen bonding effects. The 72-hr boil test is not a routine test but used only to establish that the particular adhesive formulation will have the required durability when used to bond the species to be used in actual manufacture. Some formulations are capable of producing bonds only with low-density species. Also, the presence of preservatives and some wood extractives can interfere either with surface wetting characteristics or with the curing reaction of the adhesive. In the case of woods such as teak, which have poor wetting characteristics, low wood failures are obtained when specimens are broken either in the dry state or after a VPS. Where the problem is one of interference with the curing reaction, low wood failures show up after a 72-hr boil. This is known to occur with some eucalyptus because of the presence of hydrolysable tannins. It also occurs where copper-chrome-arsenate preservative treatments have been applied. In general, timbers treated with copper-chrome-arsenate cannot be glued consistently on a commercial basis.

In many glulam and finger-jointing standards, the bond is assessed by determining the amount of delamination visible after a standardized VPS and drying régime. Australian experience has shown this to be quite inadequate. It can be demonstrated that where delamination occurs in such a test, there is zero wood failure if the bond is cleaved in the wet condition after a VPS. Hardwoods with a density of up to 900 kg/m^3 ar glued commercially in Australia. Of these, the beams that have delaminated in service are the ones that were accepted on the basis of high initial shear strength values in the absence of high wood failures.

A complicating factor in the testing of hardwood joints is the low permeability of some hardwoods to liquid water. In some Australian hardwoods, the penetration of liquid water under a standard VPS treatment can be of the order of millimetres and so little stress is placed upon the glueline. The test specimen proposed in 1983 for the Australian glulam standard was a 25 mm slice of beam with a saw cut extending 10 mm into theglueline on the cutface. After VPS treatment, the gluelines are cleaved wet and the wood failure estimated after dyring.

While the gluing considerations in plywood and glulam production are thought to be reasonably well understood, this is not the case for finger-jointing. Finger joints are generally tested in bending, and test results are usually highly variable. There tends to be an inverse relationship between load to failure and amount of wood failure. Those specimens that break at a load consistent with the average bending strength for the species tend to exhibit quite low wood failure, particularly in the higher density species. A suitable test specimen has not yet been adopted that allows for the low water permeability of some hardwoods. Where laminated beams are tested to destruction the break frequently begins in a finger joint if there is one in the lower laminate. Given that all the wood bonds lose strength to some extent with time and that thos exhibiting lowwood failure do so more rapidly, there must be some doubt about the durability of finger joints, particularly in critical tension situations. These considerations have led some Australian glulam manufacturers to either use scarf joints in theother laminates or to regard finger-jointing purely as a materials handling aid and to design beams on thea basis that all joints act as butt joints. The factthat there have been no known failures of beams in service due to finger-joint failure probably reflects a tendency to overdesign rather than good gluing practice. Clearly some species can be finger-jointed to a consistently high quality. At present the main outlet for finger-jointed material in Australia is for wall studs and bottom plates. The quality of production should be thoroughly investigated before considering the use of finger-jointed material in situations any more critical than these.

While scarf-jointing does not lend itself to automation in the same way as does finger-jointing, it is possible, if care is taken, to consistently produce high-strength, high-wood-failure joints.

F. Specification

Durability considerations allow structural design incorporating bonded components to be based on wood strengths. These considerations also simplify test methods in that the wood has already been selected on the basis of strength. Proof testing procides virtually no information on bond quality and the only meaningful tests are destructive ones. This cana be shown by considering a finger joint in timber gradesas F17. A reasonable proof load for such a joint would be 2.5 times the basic working stress, or 42.5 MPa. The species average for clear material is likely to be in excess of 100 MPa. The modulus of rupture at the joint would need to be in the region of 80 MPa in order to exhibit significant wood failure. This is well in excess of any proof load.

Except in the case of plywood, where a simple wetting treatment can be used to show up areas of delimination, bonded components are essentially untested. The specifying engineer must therefore rely on the skill and integrity of the component manufacturer. In choosing a reliable manufacturer, national standards can be of some assistance provided that they include a relevant set of test methods and minimum personnel and equipment requirements for manufacturers. Many countries have an industry organization, such as the Plywood Association of Australia, that runs a quality control scheme to ensure that any product bearing its stamp will conform to the standards to which it is produced. Such an organization must be prepared to withdraw the stamp of a manufacturer that consistently fails to meet the requirements of the standard. Where no such organization exists, it is necessary for the engineer to satisfy himself that a given manufacturer is capable of consistently producing components to the rigorous standards required for structural applications. Clearly, the engineer cannot be expected to be an expert in adhesives, wood technology and production engineering. This is the province of the manufacturer.

In assessing the capabilities of a manufacturer, the first step is to determine whether it has the appropriate personnel and production and testing equipment for the purpose. Canadian Standard 0177, Qualification Code for Manufacturers of Structural Glued-Laminated Timber, provides a useful checklist for such an assessment. Production should then be sampled on a regular basis and tested to destruction according to the principles previously outlined.

Finally, successful structural bonding depends on the close control of a range of variables such as time, temperature, pressure, wood quality and moisture content. The required level of control is unlikely to be attainable on a building site. Thus, structural bonding will continue for the forseeable future to be part of the manufacturing process rather than of the construction process.

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