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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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THE USE OF TIMBER IN CONSTRUCTION- ANALYSIS OF BUILDING CODES*

Prepared by

Amamtino Ramos de Freitas **

* This document has not been edited.

** Director, Wood Division, Technical Research Institute (IPT), Sao Paulo, Brasil.

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ABSTRACT

This paper presents a survey of eight timber codes: National Design Specification (USA); Australian Timber Structures Code; Malaysian Code of Practice for the Structural use of Timbers; Philippine Building Code, Chapter 3 - Wood; ABNT Brazilian Standard NBR 7190 - 1982; Japanese Building Code for Wood Construction; Andean Pact (JUNAC), Design Handbook for Wood Construction, and the CIB Structural Timber Design Code.

Comments and suggestions are made regarding the application of the information generated in this survey to timber codes for developing countries.

PART I - COMMENTS AND SUGGESTIONS

1. INTRODUCTION

During the First Consultation on the Wood and Wood Products Industry, which took place in Helsinki, in 1983, under UNIDO and FAO sponsorship (39), recommendations were made with respect to the promotion of wood in construction, with special emphasis on developing countries.

One of the issues discussed during the Consultation was the contribution that wood could make to mitigate the housing shortage in developing countries. However, it was recognized that in some of these countries the use of wood in construction was hindered by building codes and regulations.

A specfic recommendation was made by the Consultation regarding the possibility of UNIDO developing, in cooperation with other international organizations, an internationally accepted strength-grouping system for timber from developing countries used for structural purposes, and of stress-grading rules. This specific topic had been the main theme of an Expert Group Meeting organized by UNIDO in 1981 (40), and was again discussed in the Expert Group Meeting on Timber Construction held in 1985 (38).

In light of the considerations above, the main objective of this paper is to make a survey of the timber codes of selected countries and to suggest some conceptual guidelines which could assist developing countries in establishing new codes or up-dating existing ones. Although most of the codes reviewed also include the use in construction of other wood products, such as plywood and laminated wood, the present review is restricted to aspects related only to the utilization of solid lumber.

2. DETERMINISTIC AND PROBABILISTIC APPROACHES TO STRUCTURAL DESIGN

The basic objective of structural design is to obtain safe and economical structures. From the early days of civilization until Galileo's work "The Two New Sciences", published in 1638, structural design was exclusively an art based on trial and error (5).

The experience gained in building successful structures was transferred from generation to generation, in the form of empirical rules for proportioning structural members and their joints under various loading conditions. Since there was no rational method to evaluate the load-carrying capacity of these members, structural safety was a matter of judgment based on occasional failures which were accepted as inevitable learning experiences.

After Euler had published his formula for the buckling strength of long columns in 1757 and Coulomb had correctly interpreted the bending resistance of beams in 1773, it was possible to determine the cross-sectional dimensions of structural members according to rational procedures (30). Consequently, an indication of the degree of safety associated with a given structure could be established by comparing load effects to structural resistance.

2.1. Factors of safety

The term "factor of safety" was defined in Rankine's "A Manual of Civil Engineering", published in 1883, as the ratio of the breaking load to the working load. The effects of the quality of materials and workmanship, and of the type of load, be it static or dynamic, on the structural safety were recognized by Rankine. Different factors of safety were recommended for different degrees of control during construction and also for live loads as compared to dead loads. For good ordinary timber construction, the recommended factors of safety were 4 to 5 for dead loads and 8 to 10 for live loads. The wider variability of wood as compared to metals was probably recognized, since for metals the recommended factors of safety were 3 for dead loads and 4 for live loads (30).

The need to specify different factors of safety for different materials or different degrees of control during construction was an indication that the factor of safety alone, as defined by Rankine, did not represent an adequate measure of safety. Figure 1 shows two cases of load and resistance combination with the same central safety factor, (defined as the ratio between the mean resistance and the mean load) but with two completely different probabilities of failure.

2.2. Deterministic approach

In the conventional deterministic approach, safety is assured by designing for minimum strength values and maximum load values, according to the guidelines established by experience.

Since both loads and resistances are assumed to be deterministic values, failure is to be prevented by using an adequate factor of safety in design. In the deterministic approach, the factor of safety is the parameter that quantifies structural safety. Structures designed with the same factor of safety are assumed to be associated with the same degree of safety, irrespective of materials variability, quality of workmanship or load characteristics. In real life, however, structural loads and resistances are random variables, rather than fixed constants. By assuming an adequate factor of safety in design, the risk of failure can be reduced to very small levels but cannot be completely eliminated. According to Freudenthal (17), for a single application of the design load, steel structures designed under the conventional deterministic structural codes have a probability of failure between 10^{-4} and 10^{-6}, and concrete structures between 10^{-3} and 10^{-5}.

Once it has been recognized that structural safety is related to random variables, the logical way to deal with such safety is through probabilistic methods.

2.3. Probabilistic approach

Statistical concepts in structural safety were first introduced by Max Meyer in 1926. After World War II, this subject received renewed interest, especially with the advancement of the aircraft industry and the development of the space programs (6).

In the probabilistic approach to structural design, the degree of safety associated with a given structure is expressed by its probability of survival or, conversely, by its probability of failure. In the most general case, both loads and resistances are considered to be random variables associated with time, S(t) and R(t). Consequently, the safe life of the structure is also represented by a random variable, I (17).

For a given structure, the basic problem is to determine the probability of survival (reliability), I(t), or its complement, the probability of failure, p_f (t):

$$I(t) = 1 - p_f(t) = P\,[T > t] = P\,[R(\tau) > S(\tau) \mid 0 \leq \tau \leq t]$$

The reliability of a structure normally decreases with time because of wear, cumulative damage, and increased chance of occurrence of heavier loads. However, the problem of computing the reliability function, as stated above, has not been satisfactorily solved.

2.4. Semi-probabilistic method

For time-invariant problems, i.e., if R (t) = R and S(t) = S, and when both R and S are random variables, the problem of evaluating safety is reduced to the classical theory of structural reliability; the probability of failure is the probability of S being greater than R.

When the density functions f_R (r) and f_S (s) can be approximated from experimental data and assumed to be independent, the probality density function for R/S or R-S can be computed. The probability of failure is defined by the probability of R/S < 1 or R - S < 0.

According to the distribution assumed for R/S or R-S, it may be possible to account separately for the influence of the variability of loads and resistances on the probability of failure. This leads to the establishment of separate factors for loads (load factor) and resistances (performance or material factor), as if the conventional factor of safety used in the deterministic approach had been separated into two components. This method is called the semi-probabilistic or Level I Method. It uses the load and resistance factors obtained from probabilistic considerations, but keeps the overall format of the deterministic method.

Because it does not significantly depart from the conventional design approach, this method, also known as Load and Resistance Factor Design - LRFD, has gained considerable support. The CIB timber code and Eurocode-5 are based on this format (12,13).

The load factor, which multiplies the load values, reflects the degree of variability of the loads; conversely, the material factor, by which the strength values are divided, reflects the variability of the material strength properties.

Of the eight timber codes reviewed in this paper only the CIB code adopts a semi-probabilistic approach to design. Although a number of the other codes are based on characteristic strength values, they do not use the material and load factor approach, nor any other probabilistic method, thus being classified as deterministic codes.

2.5. Stress grades

The ability of a given piece of lumber to adequately support load depends basically on the inherent strength of wood species it is made from, and on the presence of strength-reducing defects. In visual grading, the reduction in strength is evaluated by visual inspection; in machine grading, this reduction is evaluated by measuring a strength predictor, usually the modulus of elasticity in bending. The ratio between the strength of a piece having defects and that of the same piece with no defects is defined as "strength ratio".

Other important factors that influence the load-carrying capacity of a structural member made with wood are moisture content, load duration, size and geometry of the piece, chemical treatment, etc.

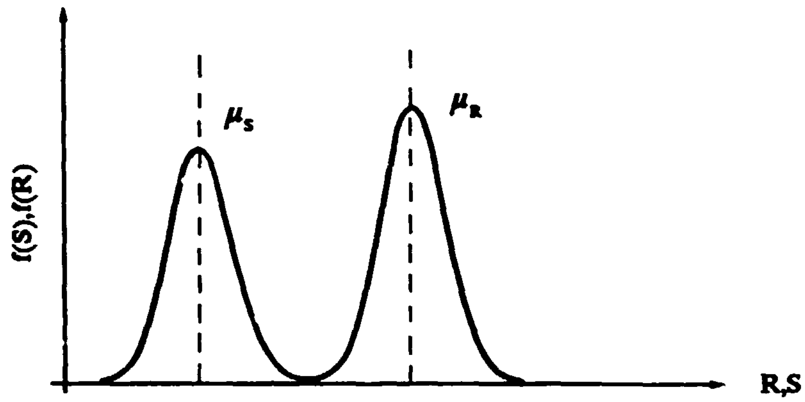
The objective of stress grading is to classify timber, of the same species or not, into groups of uniform guaranteed minimum strength. As described in the following paragraphs, the codes reviewed in this paper take different approaches to the establishment of stress grades.

The grading rules of the American code (NDS) have nine structural grades and present individual design values for each grade and for about 50 species, or group of species, in a total of over 1100 choices of combinations. The Japanese grading rules (JAS) have seven structural grades for softwoods and consider two groups of species. The Australian code is based in a stress grading system that has only 12 grades. This relatively small number of grades results from the fact that all species are grouped in seven strength classes for green timber (S1 to S7) and eight strength classes for dry timber (SD1 to SD8), one class being 25% stronger in bending than the one that immediately follows it, and also because the four visual grades are based on the same geometric progression regarding strength ratios.

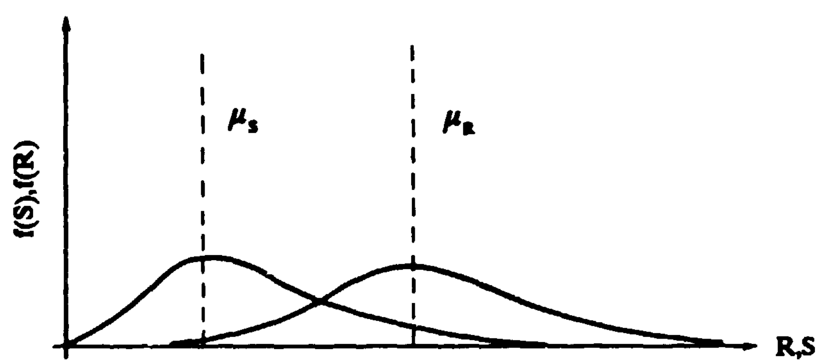
The Malaysian code presents individual species values for the working stresses of four visual grades, but also contemplates grouping the species into four strength classes in order to simplify procurement of structural timber.

The Philippine code has three structural grades and presents individual working stresses for each species or group of species.

The Brazilian standard for the design and construction of wood structures is completely based on results obtained from testing small clear specimens, without any reference to stress grades.



a) Very good control of loads and strength



b) Poor control of loads and strength

Figure 1 - Effect of dispersion of loads and strenghts on probability of failure for constant central factor of safety

Source: MacGregor (22)

The JUNAC handbook for the design of wood structures is based on a single visual grade having a strength ratio of 0.80 and on three strength classes, for which working stresses are given for the main mechanical properties.

Finally, the CIB code has 13 standard strength classes; characteristic values are given for each one of them for the main properties. The common ratio between the characteristic strength in bending of one class and that of the class immediate below is 1.25. This code also considers five standard density classes based on characteristic values that also keep a common ratio of 1.25 between them.

2.6. Joints

A common feature to most of the codes analyzed is that the design of joints is somewhat independent from the stress grade under consideration. It is usually based on some selected properties such as shear and compression perpendicular to grain which keep a close correlation with density. The American code, for example, gives design values for lateral loads of nails and spikes in single shear for different specific gravity classes. The Australian code groups the species into six joint groups for dry timber and six joint groups for green timber. The Malaysian code classifies species into five joint groups and the Philippine code into three. Although the CIB code recommends that the characteristic load-carrying capacity of joints be determined by appropriate testing, the CIB formulas for estimating it, are based on the density of the wood.

3. CONCLUSIONS AND RECOMMENDATIONS

Developing countries that want to utilize their tropical wood resources in construction, face problems that are different from those found in temperate zones. Among other factors, the multiplicity of species, the size of the logs, the type of strength-reducing defects present in the wood, and the lack of adequate material and human resources call for creative solutions that will bring a more rational utilization of wood as a construction material. To reach this objective, and on the basis of the present review of timber codes, the following suggestions are made:

a) timber codes for developing countries should be based on design rules valid for a small number of strength classes and not for individual species;

b) similar to the Australian code, strength ratios and strength classes should be based on geometric progressions having the same common ratio, so that the number of stress grades is kept to a minimum;

c) timber codes should have provisions for including new species based only on the determination of their specific gravity and a few simple strength tests. As these species become better known, detailed information on their strength properties will allow placing them in higher strength classes;

d) although a deterministic approach to timber design for developing countries would be more direct and easier to implement, the recommendations made by the CIB code with respect to the determination of characteristic values should be followed in order to allow for compatibilization of design procedures in the future;

e) finally, developing countries should be assisted by proper international organizations in establishing or upgrading their wood technology laboratories and in the development of testing programs aimed at the generation of technical information that would allow the adequate utilization of their wood resources in construction.

PART II - REVIEW OF TIMBER CODES

1. U.S. NATIONAL DESIGN SPECIFICATION FOR WOOD CONSTRUCTION

1.1. General considerations

The U.S. National Design Specification for Wood Construction (NDS) is published by the National Forest Products Association (26). It follows a deterministic approach to design, using allowable stresses that are published under a separate bulletin (27) for different grades of about 50 species of wood or groups of species, mostly softwoods.

Besides an average value for the modulus of elasticity E, the following allowable stresses are given:

- Extreme fiber in bending .... F_b
- Tension parallel to grain.. F_t
- Horizontal shear .... F_v
- Compression perpendicular to grain .... $F_{c\perp}$
- Compression parallel to grain .... F_c

The National Design Specification Supplement lists seven regional agencies that are authorized by the American Lumber Standards Committe (ALSC), to write grading rules for structural lumber (and also to certify grades) and 14 other agencies accredited to inspect and certify structural grade lumber.

Dressed dimension lumber, i.e., lumber with nominal thickness of 2 to 4 inches, bearing an ALSC - certified symbol, is defined by a common set of grade names and descriptions, called "The National Grading Rule" (NGR). This rule is presented in the grading rules book published by each of the ALSC-certified grading agencies.

Table 1-1 lists the NGR grades of visually-graded lumber, and the corresponding bending strength ratios.

In addition to visual grades, the NGR rules also contemplate machine stress grades, for lumber two inches thick or less, described by a pair of numbers, the first indicating the allowable stress in bending and the second the modulus of elasticity; 1500f-1.4E, for example, refers to structural lumber having an allowable bending stress of 1500 psi and modulus of elasticity in bending of 1.4 x 10^6 psi. Two sets of pairs (f,E) are recognized: the first, containing higher values of F_b, to be used mainly for trussed rafters and other engineered constructions; the second, containing higher values of E for equivalent levels of F_b, is intended for floor joists and other applications where deflection governs design.

TABLE 1-1

NGR* STRUCTURAL GRADES OF VISUALLY-GRADED DIMENSION LUMBER

Grade name	Bending strength ratio
Structural light framing	
2-4" Thick, 2-4"	Wide
Select structural	67%
1	55%
2	45%
3	26%
Structural joists & planks	
2-4" Thick, 5" & Wider	
Select structural	65%
1	55%
2	45%
3	26%

Grade Name	Bending Strength ratio
Light framing	
2-4" Thick, 2-4"	Wide
Construction	34%
Standard	19%
Utility	9%
Studs	
2-4" Thick, 2-6" Wide, 10' & Shorter	
Stud	26%
Appearance framing	
2-4" Thick, 2" & Wider	
Appearance	55%

*NGR: National Grading Rule.

Source: Southern Pine Inspection Bureau Grading Rules Book (33)

The tables presented in the National Design Specification Supplement offer possibilities of more than 1100 choices of visually-graded and 28 of machine-graded structural lumber.

1.2. Determination of allowable stresses

1.2.1. Clear wood strength

The first step in establishing the allowable stresses given by the National Design Specification is the determination of the clear wood strength, by means of testing small clear specimens (ASTM D 2555-81, D 143-83). On the basis of these test results, clear wood strength can be described by the average value of E and $F_{c\perp}$ and the 5% lower exclusion limit (5% LEL) for F_b, F_c, and F_v.

Clear wood strength properties for which results from small clear specimen tests are usually not available, such as tension parallel to grain, F_t, tension perpendicular to grain, $F_{t\perp}$, and modulus of rigidity, G, can be estimated on the basis of other properties. For example, F_t is assumed to be equal to F_b, $F_{t\perp}$, is equal to 0.33 times F_v, and G is equal to 0.069 times the modulus of elasticity E.

1.2.2. Adjustment factor

In order to account for the duration-of-load effect and for uncertainties related to manufacture, use, stress concentration, etc., the 5% LEL for each strength property is divided by a factor as given in Table 1-2.

1.2.3. Size factor

The bending stress for the correct depth d in inches of the beam shall be adjusted by multiplying the F_b value by $F = (2/d)^{1/9}$. This is necessary because the bending strength obtained for actual 2.0 by 2.0 in. clear specimens tends to be higher than that observed for full-size members.

1.2.4. Strength ratio

The values obtained in line with the above factors are called "basic stresses" and shall be multiplied by the appropriate strength ratios, expressed as decimals, according to the criteria below.

A strength ratio of 100% is assumed for compression perpendicular, $F_{c\perp}$, for all stress grades;

A strength ratio of 55% of the bending strength ratio is assumed for tension parallel to grain, F_t ;

A quality factor of 1.0 is assumed for the modulus of elasticity, E , in bending strength ratios of 55% or higher. For strength ratios of 45%-54%, E has to be multiplied by a quality factor of 0.90 and below 45%, by 0.80;

Higher strength ratios can be assigned to special density classes of some softwood species, such as Douglas-fir and southern pine. For example, an increase of 5% in E and of 17% in F_b, F_t, F_c, and $F_{c\perp}$, allowable is assigned to the dense grades of these two species.

The strength ratios associated with the presence of knots and cross-grain in compression and bending members, and those associated with splits, checks, and shakes for horizontal shear in bending are given by ASTM D 245-81 (2).

1.2.5. Drying

The above steps lead to the establishment of allowable properties for a given stress grade of green lumber. Since wood becomes stronger when it dries, an increase in these values becomes necessary if the wood is to be used at a lower moisture content. This factor is taken into account in the design values published in the National Design Specifications Supplement.

TABLE 1-2

ADJUSTMENT FACTORS

Strenght property		Softwoods	Hardwoods
Bending	F_b	2.1	2.3
Modulus of elasticity (bending)	E	0.94	0.94
Tension parallel to grain	F_t	2.1	2.3
Compression parallel to grain	F_c	1.9	2.1
Horizontal shear	F_v	4.1	4.5
Compression perpendicular (proportional limit and stress at a 1 mm deformation)	$F_{c\perp}$	1.67	1.67

TABLE 1-3

REDUCTION OF ALLOWABLE STRESS WHEN MOISTURE CONTENT EXCEEDS 19%

Allowable stresses		Lumber thickness 2" to 4"	5" and above
Extreme fiber in bending	F_b	0.86	1.00
Tension parallel to grain	F_t	0.84	1.00
Horizontal shear	F_v	0.97	1.00
Compression perpendicular	$F_{c\perp}$	0.67	0.67
Compression parallel	F_c	0.70	0.91
Modulus of elasticity	E	0.97	1.00

Source: NDS (27)

1.3. Modification of stresses and loads

1.3.1. Moisture conditions

Except for kiln dried southern pine and Virginia pine-pond pine dimension lumber, whose stresses are given for 15% maximum moisture content, all other values specified in the NDS Supplement refer to lumber used at 19% maximum moisture content.

When use conditions are such that moisture content exceeds 19%, the allowable stresses must be multiplied by reduction factors given in Table 1-2.

1.3.2. Temperature

The fact that the strength of wood is affected by temperature is recognized. Modification factors, expressed in percent decrease (or increase) of design values for each 1°F above (below) 68°F are given for three levels of moisture content (0%, 12%, 24%) for the temperatures ranging from -100°F to + 150°F.

1.3.3. Preservative and fire retardant treatments

For all purposes, pressure-treated lumber (except pieces treated with high salt retentions for marine use for which increased impact loads, item 1.3.4.d below, do not apply) is assigned the same design values as untreated wood.

One of the appendices of the NDS presents special procedures for assigning design values to lumber pressure impregnated with fire retardant chemicals, namely for F_t, F_c, F_v, F_b and E. The effects of fire retardant treatment are determined on the basis of tests conducted on matched samples of clear, straight-grain material.

1.3.4. Duration of load

The NDS design values are for normal duration of loading, which means fully stressing a member to the allowable stress by the application of the full maximum normal design load for a duration of approximately 10 years (either continuously or cumulatively) and/or the application of 90% of this full maximum normal load continuously throughout the remainder of the life of the structure, without encroaching on the factor of safety.

For durations of load differing from normal loading, the design values are adjusted in order to account for the behaviour of wood under such conditions.

If the member is fully stressed to the design value by the application of the full maximum load permanently, or for periods longer than 10 years continuously or cumulatively, the allowable stresses shall be decreased by 10%.

Conversely, for short time loading the design values are multiplied by the following factors:

a) 1.15 for two months duration
b) 1.25 for seven days duration
c) 1.33 for wind or earthquake
d) 2.00 for impact

1.3.5. Flexure (F_b)

The design values for extreme fiber in bending can be modified in the following cases:

1.3.5.1. Slender beams

When the depth d of a beam is greater than its breadth b, lateral supports are required at the points of bearing to prevent rotation, and the design value at extreme fiber in bending (F_b) shall be modified depending on the slenderness factor of the beam C_s. This factor which must not exceed 50, is a function of the unsupported length (l_u), depth to breadth ratio and load conditions:

$$C_s = \left(\frac{l_e d}{b^2}\right)^{1/2}$$

Examples of l_e (effective length)

$l_e = 1.37\, l_u + 3d$ for single span, center load

$l_e = 1.63\, l_u + 3d$ for single span, uniform load

Beams with slenderness factor $C_s = 10$ or less, classified as short beams, may be designed using the full value F_b.

Beams with slenderness factor C_s greater than 10, but less than

$$C_k = 0.811 \left(\frac{E}{F_b}\right)^{1/2}$$

classified as intermediate beams, are designed using design stress at extreme fiber F_b', lower than F_b, given by

$$F_b' = F_b \left[1 - \frac{1}{3} \left(\frac{C_s}{C_k}\right)^4 \right]$$

Long beams, i.e. those having $C_s > C_k$, but not greater than 50, are to be designed with extreme fiber value F_b' given by:

$$F_b' = \frac{0.438\, E}{(C_s)^2}$$

1.3.5.2 Form and size factors

The form factor assigns the same design load to round section beams (form factor = 1.18) and diamond section beams (form factor = 1.414), as that of a square bending member of same cross- sectional area.

The size factor $C_F = (12/d)^{1/9}$ adjusts the value F_b for the retangular sawn-wood beams having depth greater than 12 inches. It does not apply to machine stress-rated lumber or visually-graded lumber 2 to 4 inches thick.

1.3.6. Shear (F_v)

An increase of 50% on the design value for shear, F_v, is allowed in some joint details, when the joint is located at least five times the depth of the member from its end.

1.4. General design procedures

1.4.1. Bending members

1.4.1.1. Flexure stresses

The design of members subjected to flexural loads is carried out according to the traditional theory of elasticity. Safety is assured by limiting the extreme fiber stress, f_b, and the maximum horizontal shear, f_v, to design values F_b and F_v respectively. Special consideration is given to the design of slender beams (as seen under 1.3.5.1.), beams with notches, beams supported by fasteners and shear at joints.

1.4.1.2. Deflection

Deflection due to bending is calculated according to the theory of elasticity, using the design value for E given in the NDS Supplement. However, inelastic deformation due to permanent loads (creep) is accounted for by multiplying the initial deflection by a factor of 1.5, for seasoned lumber, or 2.0, in the case of unseasoned wood. The deflection caused by the short-term or normal component of the design load is added to this calculated initial deflection due to permanent loads.

In cases where deflection may be critical to the behavior of the structure, the designer may choose to use a reduced value for the modulus of elasticity. This procedure may be based on the selection of a lower exclusion limit (e.g., 5% or 16%) of the population of E values, considered as having normal distribution and a coefficient of variation of 0.25 for visually-graded lumber and 0.11 for machine stress-rated lumber.

1.4.2. Compression members

Consideration is given to the design of simple solid wood columns and of spaced columns. The allowable stress F_c' in compression depends on the slenderness ratio l_e/d of the column, which shall not exceed 50 for solid columns.

The effective column length l_e is a function of the conditions of end fixicity and of lateral support of the column. Except for the rotation-free/translation-fixed condition, recommendations are made to increase the theoretical value of l_e by 10% to 30%.

1.4.2.1. Short columns

When the ratio l_e/d for solid columns is of 11 or less:

$$F_c' = F_c$$

1.4.2.2. Intermediate columns

For solid colums having an l_e/d ratio greater than 11, but less than K, where:

$$K = 0.671\, (E/F_c)^{1/2}$$

$$F_c' = F_c \left[1 - \frac{1}{3} \left(\frac{l_e/d}{K}\right)^4 \right]$$

1.4.2.3. Long columns

Solid columns with an l_e/d ratio of K or greater:

$$F_c' = \frac{0.30\, E}{(l_e/d)^2}$$

1.4.2.4. Spaced columns

Recommendations regarding the design of spaced columns include details on limitations of l/d ratios, end-fixicity classes, location and dimension of spacer blocks, and load capacity of connectors.

The formulas for calculating F_c' for spaced columns are basically the same for solid columns, except that the modulus of elasticity E is multiplied by a factor C_x, which has a value of 2.5 or 3.0 depending on the arrangement of the connectors in the end block.

1.4.3. Tension members

The unit stress in axial tension f_t, determined on the basis of the net area, shall not exceed the design values in tension parallel to grain F_t.

Due to the low resistance of wood, design situations that induce tension perpendicular to grain stresses should be avoided. Whenever this type of stress is present, it should be fully absorbed by adequate mechanic reinforcement. A particular example is the case of loads hanging below the neutral axis of a beam.

1.4.4. Combined loads

Members subjected to combined loads should be designed in such way that the extreme fiber stress, either in tension or compression, never exceeds the design value. Eccentric-loaded columns and truss compression chords receive special consideration.

1.4.4.1. Flexure and axial tension

$$\frac{f_t}{F_t} + \frac{f_b}{F_b} \leq 1 \quad \text{and} \quad \frac{f_b - f_t}{F_b'} \leq 1$$

1.4.4.2. Flexure and axial compression

a) General case:

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b' - Jf_c} \leq 1$$

$$\text{where:} \quad J = \frac{(l_e/d) - 11}{K - 11}, \qquad 0 \leq J \leq 1$$

b) Short columns: $\left(\frac{l_e}{d} < 11, J = 0\right)$

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b'} \leq 1$$

c) Long columns: $\left(\frac{l_e}{d} > 11, J = 1\right)$

$$\frac{f_c}{F_c'} + \frac{f_b}{F_b' - f_c} \leq 1$$

1.4.5. Bearing on end grain

NDS provides specific design values F_g for 48 species or groups of species for end grain bearing, for two types of service conditions: wet (MC > 19%) and dry (MC ≤ 19%). For sawn lumber used under dry conditions, pieces 4 inch thick or less have design values that are about 36% higher than thicker pieces.

1.4.6. Bearing perpendicular to grain

Except for bearings less than 6 inches in length and not closer than 3 inches to the end of the member, the induced unit stress in compression perpendicular to grain, $f_{c\perp}$, shall not exceed the correspondent design value, $F_{c\perp}$.

Bearings with length l_b measured along the grain less than 6 inches may have the design value $F_{c\perp}$ multiplied by a factors of:

$$\frac{l_b + 0.375}{l_b}$$

1.4.7. Stresses and loads at angle to grain

Design values in compression on surfaces inclined to the grain shall be obtained by the Hankinson formula:

a) End bearing:

$$F_n = \frac{F_g\, F_{c\perp}}{F_g \sin^2\theta + F_{c\perp} \cos^2\theta}$$

b) Connectors:

$$N = \frac{PQ}{P \sin^2\theta + Q \cos^2\theta}$$

1.4.8. Wood fastenings

Consideration is given to the various types of fastenings used in wood construction: timber connectors, bolts, lag screws, drift bolts, nails, spikes, wood screws, metal-plate connectors and spike grids. The design values presented for these fastenings apply to all grades of the wood species listed in respective tables.

Provisions are made so that the following factors are duly taken into account in the calculation of the design value of joints: moisture conditions, number and location of fastenings, thickness and number of pieces joined together, joint geometry, etc.

1.4.8.1. Bolted joints (double shear)

Design values, both for loads parallel and perpendicular to grain, are given for each type of bolt, length varying from 3 to 17 inches and diameter from 0.5 to 1.5 inches, for 12 different species or group of species.

A total of 1368 load design values are presented (2 load directions x 12 species x 57 types of bolts).

1.4.8.2. Other types of joints

For other types of joints, design loads are specified according to the specific gravity of the different wood species, which for this purpose are classified into five groups.

In the particular case of withdrawal of nails and spikes, the loads are given per inch of penetration into side grain of member holding point, for each individual value of species specific gravity, in a total of 1050 design values. The same disposition applies to withdrawal loads of wood screws, with 275 design values specified.

Design values for lateral loads of nails and spikes in single shear are given for each specific gravity class, but a minimum penetration (expressed in nail diameters) is requeired for each class.

2. AUSTRALIAN STANDARD 1720.1-1988. SAA TIMBER STRUCTURES CODE. PART 1 - DESIGN METHODS

2.1. General considerations

According to Keating (21), the present system of strength grouping, in which the Australian Timber Code (32) is based, originated in 1939 when four strength groups were proposed. The criterion used to place a given species in a strength group was its mean strength value obtained from standard tests on small clear specimens. Later on, before the strength groups were expanded in order to accommodate new information on a broader range of species, a set of working stresses was developed and became the basis for the Australian classification system.

This set of working stresses was established using a preferred number series with adjacent terms chosen in the ratio of 1.25 to 1.0 for modulus of rupture. The values of the other properties were determined from regression equations (V.Table 2-1).

The fact that visual stress grades in Australia are also based on steps of 25% reduction in bending strength brings the advantage of reducing the total possible number of structural grades from 32 to 12, as shown in Tables 2-2 and 2-3.

The Australian structural code has seven strength classes (S1-S7) for unseasoned wood and eight classes for wood to be used at 12% moisture content (SD1-SD8). In addition, for the purpose of establishing basic design loads for joints, species are further classified into six joint groups if used unseasoned (J1-J6) and six if used seasoned (JD1-JD6).

Working back from the set of working stresses, the species mean values for each strength group for the critical properties were developed for green and dry wood as shown in Tables 2-2 and 2-3. In case the actual mean values for a given species do not perfectly match the minimum limits shown in this table for a single strength class, the species can be classified one step above the lowest class, as shown in Table 2-6.

Another important feature of the Australian grouping system is that the relationship between density and modulus of rupture of seasoned timber provides the basis for a preliminary classification based on density alone, as shown is Tables 2-7 and 2-8.

The Australian code recommends that all stresses shall be calculated on the basis of elastic theory in order that its requirements regarding permissible stresses may be satisfied with regard to load effects at any particular location. For complex structures or structural elements the Australian code makes provision for accepting experimentally based design.

TABLE 2-1

DESIGN PROPERTIES FOR SAWN TIMBER, ROUND POLES AND PLYWOOD

Stress grade	Basic bending strength (MPa)	Basic tension strength (MPa)	Basic compression strength (MPa)	Modulus of elasticity (MPa)	Modulus of rigidity (MPa)
F34	34.5	20.7	26.0	21.500	1.430
F27	27.5	16.5	20.5	18.500	1.230
F22	22.0	13.2	16.5	16.000	1.070
F17	17.0	10.2	13.0	14.000	930
F14	14.0	8.4	10.2	12.500	800
F11	11.0	6.6	8.4	10.500	700
F8	8.6	5.2	6.6	9.100	610
F7	6.9	4.1	5.2	7.900	530
F5	5.5	3.3	4.1	6.900	460
F4	4.3	2.6	3.3	6.100	410
F3	3.4	2.1	2.6	5.200	350
F2	2.7	1.6	2.1	4.500	300

Source: Keating (21), AS 1720.1 - 1988 (32)

TABLE 2-2

STRESS GRADES SPECIFIED IN THE AUSTRALIAN TIMBER CODE** FOR UNSEASONED WOOD

Visual grade		Strength group						
Nomenclature	% Strength of clear material	S1	S2	S3	S4	S5	S6	S7
Structural grade nº 1	75	F27*	F22	F17	F14	F11	F8	F7
Structural grade nº 2	60	F22	F17	F14	F11	F8	F7	F5
Structural grade nº 3	48	F17	F14	F11	F8	F7	F5	F4
Structural grade nº 4	38	F14	F11	F8	F7	F5	F4	F3

* F27 designates a stress grade having basic working stress in bending of 27.5 MPa as shown in Table 2-1.
** Australian Standard AS 1720.1-1988 (32)

Source: Keating (21)

TABLE 2-3

STRESS GRADES SPECIFIED IN THE AUSTRALIAN TIMBER CODE** FOR DRY WOOD

Visual grade		Strength group							
Nomenclature	% Strength of clear material	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Structural grade nº 1	75		F34*	F27	F22	F17	F14	F11	F8
Structural grade nº 2	60	F34	F27	F22	F17	F14	F11	F8	F7
Structural grade nº 3	48	F27	F22	F17	F14	F11	F8	F7	F5
Structural grade nº 4	38	F22	F17	F14	F11	F8	F7	F5	F4

* F34 designates a stress grade having basic working stresses in bending of 34,5 MPa as shown in Table 2-1.
** Australian Standard AS 1720.1-1988.

Source: Keating (21)

TABLE 2-4

PRELIMINARY CLASSIFICATION VALUES FOR UNSEASONED* TIMBER

Property	Minimum species mean						
	S1	S2	S3	S4	S5	S6	S7
Modulus of rupture (MPa)	103	86	73	62	52	43	36
Modulus of elasticity (10^3 MPa)	16.3	14.2	12.4	10.7	9.1	7.9	6.9
Maximum crushing strength (MPa)	52	43	36	31	26	22	18

* As measured or estimated at a moisture content above saturation point.

Source: Keating (21)

TABLE 2-5

PRELIMINARY CLASSIFICATION VALUES FOR SEASONED* TIMBER

Property	Minimum species mean							
	S1	S2	S3	S4	S5	S6	S7	S8
Modulus of rupture (MPa)	150	130	110	94	78	65	55	45
Modulus of elasticity (10^3MPa)	21.5	18.5	16.0	14.0	12.5	10.5	9.1	7.9
Maximum crushing strength (MPa)	80	70	61	54	47	41	36	30

* As measured or adjusted to a moisture content of 12 percent.

Source: Keating (21)

TABLE 2-6

COMBINATIONS OF PRELIMINARY CLASSIFICATIONS THAT PERMIT THE OVERALL STRENGTH GROUP ASSESSMENT TO BE ONE STEP ABOVE THE LOWEST IN THE COMBINATION

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

Note: Strength group x-1 is stronger than strength group x.

Source: Keating (21)

TABLE 2-7

MINIMUM AIR-DRY DENSITY VALUES FROM 5 OR MORE TREES FOR ASSIGNING SPECIES TO STRENGTH GROUPS IN THE ABSENCE OF ADEQUATE STRENGTH DATA

(a) Unseasoned material								
Strength group	S1	S2	S3	S4	S5	S6	S7	
Air-dry density at 12 percent moisture content (kg/m^3)	1180	1030	900	800	700	600	500	
(b) Seasoned material								
Strength group	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Air-dry density at 12 percent moisture content (kg/m^3)	1200	1080	960	840	730	620	520	420

Source: Keating (21)

TABLE 2-8

PROPOSED MINIMUM DENSITY FOR JOINT STRENGTH GROUPS

Group	Green timber Basic density (kg/m^3)	Group	Seasoned timber* Air-dry density (kg/m^3)
J1	750	JD1	940
J2	600	JD2	750
J3	475	JD3	600
J4	380	JD4	475

* Density at 12% moisture content after reconditioning.

Source: Keating (21)

TABLE 2-9

BASIC WORKING STRESSES FOR COMPRESSION PERPENDICULAR TO GRAIN AND SHEAR AT JOINTS

Strength group		Basic working stress, MPa	
Unseasoned	Seasoned	Compression perpendicular to grain	Shear at joints details
--	SD1	10.4	4.15
--	SD2	9.0	3.45
--	SD3	7.8	2.95
S1	SD4	6.6	2.45
S2	SD5	5.2	2.05
S3	SD6	4.1	1.70
S4	SD7	3.3	1.45
S5	SD8	2.6	1.25
S6	--	2.1	1.05
S7	--	1.7	0.85

Source: Australian Standard 1720. 1-1988 (32)

Permissible stresses for each particular case are based on basic stresses modified by specific factors appropriate to the service conditions for which the structural member is being designed. Basic working stress is defined as stress appropriate to an arbitrarily chosen, but constant, basic reference set of conditions. It is derived from the known strength properties of a given species, after due allowance is made for such factors as material variability, long-duration load, grade of timber, and safety factor.

While basic working stresses for bending, tension and compression, as well as design values of modulus of elasticity and rigidity, are specified for each stress grade, as shown in Table 2-1, the working stresses for compression perpendicular to grain and shear at joints depend only on the strength group, as shown in Table 2-9.

2.2. Modification factors

Permissible stresses are obtained by multiplying the basic working stresses and the design values for modulus of elasticity , E, and rigidity, G, by modification factors, in order to take into account specific conditions of use. These modification factors are the following:

j_2 – duration of load factor for deflection (bending, compression and shear members)

j_3 – duration of load factor for deflection (tension members)

k_1 – duration of load factor for strength

k_4 – partial seasoning factor (unseasoned timber partly dry before use)

k_5 – for seasoned timber used where moisture content may exceed 15%

k_6 – for timber used in warmer regions of Australia

k_7 – length of bearing factor

k_8 – load sharing factor for parallel support systems

k_9 – load sharing factor for grid systems

k_{11} – size factors for flexural and tension members

k_{12} – stability factor for slender members

g_{13} – effective length factor for column design

2.3. General design procedures

The design procedures given by the Australian timber code are based on the traditional methods of the theory of elasticity. Members are proportioned with the objective of reaching calculated stresses (and deflections or deformations) that do not exceed the permissible stress (or specified deflections or deformations).

2.3.1. Beam design

Calculated stresses for unnotched beams shall not exceed the following permissible stress:

a) Bending

$$F_b = k_1\, k_4\, k_5\, k_6\, k_8\, k_{11}\, k_{12}\, F_b'$$

b) Shear

$$F_s = k_1\, k_4\, k_5\, k_6\, F_s'$$

c) Compression perpendicular to grain

$$F_p = k_1\, k_4\, k_5\, k_6\, k_7\, F_p'$$

Special consideration is given to notched beams and to slender beams. The stability factor k_{12} for modification of the basic working stress in bending depends on the geometry of the beam, expressed by the slenderness coefficient S, which is defined about the major axis of the beam (S_1) and minor axis ($S_2 = 0$ in all cases where the beam can bend only about the minor axis), and also on the material constant for beams, ρ , which is related to the stress grade.

This material constant is higher for unseasoned lumber (e.g. for F34, 1.32 dry vs. 1.23 green); for the same moisture conditions, it decreases from higher to lower stress grades (e.g. 1.23 for F34 vs 0.78 for F2).

– Values of k_{12}:

For $\rho S \leq 10$ $\quad k_{12} = 1.0$

For $10 \leq \rho S \leq 20$ $\quad k_{12} = 1.5 - 0.05\,\rho S$

For $\rho S > 20$ $\quad k_{12} = \dfrac{200}{(\rho S)^2}$

2.3.2. Column design

Calculated stress for unnotched columns shall not exceed the following permissible stress:

$$F_c = k_1\, k_4\, k_5\, k_6\, k_8\, k_{12}\, F_c'$$

The stability factor k_{12} for columns is the product of the material factor ρ for columns and the slenderness factor S for columns, similar to the case of beams. The slenderness factor is again defined for bending about the major and the minor axis of the column, and takes into consideration the geometry of the cross section, the distance between points where lateral movement is restricted and the fixicity condition of the column ends (expressed by modification factor g_{13}).

2.3.3. Design of tension members

Calculated stresses for unnotched tension members shall not exceed the following permissible stresses:

$$F_t = k_1\, k_4\, k_5\, k_6\, k_{11}\, F_t'$$

2.3.4. Combined bending and axial stresses

a) Bending and compression:

$$\left(\frac{f_{bx}}{F_{bx}}\right)^2 + \frac{f_c}{F_{cy}} \leq 1 \text{ and } \frac{f_{bx}}{F_{bx}} + \frac{f_c}{F_{cx}} \leq 1$$

where

$$f_{bx} = \frac{M_x}{Z_x}$$ nominal bending stress about the major axis

F_{bx} = pemissible design value of f_{bx}

$$f_c = \frac{P}{A}$$ nominal compression stress acting on column

F_{cx} and F_{cy} are permissible design values of the compression stress (f_c) if the members were used as a column that could buckle only about its major or minor axis respectively.

b) Bending and tension

The nominal bending stress f_{bx} and axial stress f_t of a member subject to combined bending and axial tension shall be given by:

$$0.6\, f_{bx} + f_t \leq F_t$$

$$f_{bx} - f_t \leq F_{bx}$$

where F_t and F_b are the permissible tension and bending stress for the member used as a tie or a beam respectively.

2.3.5. Connections

As dicussed earlier, for the purpose of joint design the Australian code classifies all wood species into six joint goups for unseasoned timber (J1 - J6) and six for seasoned timber (JD1 - JD6). Joint design is based on the assumption that there are no strength-reducing defects in the joint, thus dispensing with any consideration of the stress grade of the lumber.

The following mechanical fasteners are given consideration in the design of joints according to AS 1720.1-1988: nails, wood screws, bolts, coach screws, split-ring connectors and shear-plate connectors.

2.3.6. Nailed joints

a) Lateral loads

Basic working loads are given for all joint groups for single shear of common nails with diameter ranging from 2.5 to 5.6 mm. Permissible loads are obtained by multiplying these loads by the following modification factors:

k_1 = duration of load factor

k_{13} = 1.0 for nails in side grain
= 0.6 for nails in end grain

k_{14} = 1.0 for nails in single shear
= 2.0 for nails in double shear

k_{16} = 1.2 for nails in metal plates
= 1.1 for plywood gussets
= 1.0 otherwise

k_{17} = factor for multiple nailed joints

The procedures for establishing permissible lateral loads also give consideration to nail spacing, edge and end distance, depth of nail penetration and pre-boring to avoid splitting.

b) Withdrawal loads

Basic working loads in withdrawal for common nails driven into side grain are given per mm of penetration for all joint groups for diameters varying fron 2.5 to 5.6 mm. Permissible loads are taken to be identical to these working loads.

2.3.7. Screwed joints

The recommendations for screwed joints are very similar to those of nailed joints, as described in the following paragraphs.

a) Lateral loads

Basic working loads for wood screws in single shear are given for all joint groups, for shank diameters varying from 2.74 to 7.72 mm. Permissible loads are obtained by multiplying the basic loads by modification factors:

$$Q = k_1\; k_{13}\; k_{16}\; k_{17}\; Q'$$

The only difference is that k_{16} has only two values

k_{16} = 1.2 for close fitted screws driven though metal sideplates

= 1.0 otherwise

Considerations are made about spacing, edge and end distance, screw length and lumber thickness, and pre-boring.

b) Withdrawal loads

Basic working loads for plain wood screws for withdrawal from side grain are given for all joint groups, for shank diameters varying from 2.74 to 7.72 mm.

The permissible load is given by:

$Q = k_{13}\, Q'$ where

k_{13} = 1.0 for screws in side grain

= 0.6 for screws in end grain

Recommendation is made regarding the fact that the permissible withdrawal load cannot be higher than the permissible load for the screw, which is given for three different types of materials: steel, brass and bronze, and aluminum alloy.

2.3.8. Bolted joints

a) Lateral loads

In contrast with the recommendations presented for nails and screws, which did not consider grain orientation, for bolts the basic working values are given separately for loads acting parallel to the grain (Q'_a) and perpendicular to the grain (Q'_p). These values for working loads are given for all joint groups, and timber thickness ranging from 25 to 200 mm, for nine bolt diameters (M6 - M8 - M10 - M12 - M16 - M20 - M24 - M30 - M36 according to specification AS 1111). For loads acting on an angle with grain, the Hankinson formula applies:

$$Q'_\theta = \frac{Q'_a \; Q'_p}{Q'_a \, sen^2 \theta + Q'_p \cos^2 \theta}$$

The basic working load for a bolted-joint system is establisled according to the thickness of joined members, in the same way that was used for nails and screws; the pemissible load is obtained by multplying the basic working load by modification factors:

$Q_\theta = k_1 \; k_{16} \; k_{17} \; Q'_\theta$ where

$k_{16} =$ 1.2 for bolts transferring loads through metal plates

$=$ 1.0 otherwise

Considerations are given to spacing, edge and end distances, and washers.

b) Axial loads

The basic working load of bolts loaded axially shall be taken as the lesser of the axial strength of the bolt, and the bearing strengh of the wood under the washer, when loaded perpendicular to the grain. Design values for these parameters, i.e., axial-bolt strength and diameter of washer, are provided.

2.3.9. Other types of joints

The calculation of permissible loads for coach screws, split ring connector and shear plate connectors follow the same lines presented for nails, screws and bolts. They are obtained by multiplying the given basic loads by modification factors that taken into account the various use conditions. In the same way, considerations are presented regarding spacing, edge and end distance for each one of them.

3. MALAYSIAN CODE OF PRATICE FOR THE STRUCTURAL USE OF TIMBERS

3.1. General considerations

The Malaysian timber code is described in the Malaysian Standard MS 544:1978, (31), published by the Standard and Industrial Research Institute of Malaysia (SIRIM), Selangor.

This code is also based on basic and permissible stresses although they are defined somewhat differently from the Australian code. Basic stress is here defined as the stress which can safely be permanently sustained by timber containing no strength-reducing characteristics. This definition is equivalent to that of clear wood strength as presented by the American code, NDS (21). Permissible stress is the stress which can safely be sustained by a structural component under the particular condition of service and loading.

3.1.1. Structural grades

Appendix A of MS 544:1978 describes sizes and grades of Malaysian structural timbers, whose main features are:

a) about 75 species or species groups are classified into five shrinkage groups according to established limits for tangential shrinkage from green to 19 percent. These shrinkage values must be taken into consideration for determining the green cross-sectional dimension when cutting any given nominal size at 19% MC;

b) preferred cross-sectional dimensions, ranging in thickness and width from 1 to 12 inches including half-inch size, in a total of 91, are given for green wood and wood at 19% MC;

c) three grades are specified:
- select structural: to be used for special applications where a maximum strength/weight ratio of timber is required;
- standard structural: to be specified for normal purposes;
- common building: where the timber does not perform any important structural function, or is not designed by means of engineering calculations.

For each particular grade, limits are set on defects that reduce load-bearing capacity such as: shakes and checks, slope of grain, spiral grain, wane, borer and pin holes, sapwood (when not treated), curvature (bow and side bending), knots, decay, brittle heart, etc. Pieces are supposed to be individually inspected and visually graded according to the amount of defects presented.

3.1.2. Density classes

About 50 species or group of species are grouped into four classes according to their densities at 19% MC: heavy hardwoods, medium hardwoods, light hardwoods and one softwood (*Agathis spp*).

Permeability to pressure treatment at 25% MC, expressed in gallons-per-cubic foot is presented for the heartwood of each of these species or group of species.

3.1.3. Grade stresses and strength groups

For a given strength property the grade stress is the product of the respective basic stress by the grade strength ratio.

Although the Malaysian Standard MS 544: 1978 does not explicitly mention the strength ratios corresponding to the various properties and grades, the data presented on basic stresses indicate the approximate values shown in Table 3-1.

Basic properties and grade stresses, as well as mean and minimum modulus elasticity, are presented for green and dry timber of 56 Malaysian species, the great majority of them hardwoods. In order to simplify design and lumber procurement, the Malaysian Timber Code also groups these 56 species into four strength classes. Grade stresses for each group are taken as those representing the weakest species in the group. Table 3-2 and 3-3 present grade stresses for green and dry (MC $\leq$ 19%) timber respectively.

3.1.4. Permissible stresses and modification factors

Permissible stress is defined as the product of the grade stress and the appropriate modification factor for the particular conditions of use and loading under consideration.

The modification factors of the Malaysian code, to be used in the same fashion as the Australian code, are the following:

k_1 = duration of loading for beams;

k_2 = length and position of bearing for beams and ties;

k_{31} = shear for notched beams (under side);

k_{32} = shear for notched beams (upper side);

k_4 = form factor for beams with cross section other than rectangular;

k_5 = form factor for slender beams;

k_6 = factor for different limiting values of slenderness ratio and load duration for columns;

k_7 = effective length of spaced columns;

k_8 = duration of load effect for nails, screws, bolts and connectors;Table 3-1k_9 = duration of load effect for split-rings and sheer plates;

k_{10} = moisture content effect for connector joints;

k_{11} = factor for below standard end distance, edge distance and spacing of connectors;

k_{26} = modification factor for maximum design loads, when design is carried out by experimental testing;

load sharing = factor of 1.1 that multiplies the grade stress of repetitive members, four or more.

3.2. General design procedures

3.2.1. Flexural members

Members subject to flexure shall be designed according to basic principles of engineering. However, the following observations are made:

3.2.1.1. Effective span

Should be taken as the distance between the centers of bearings;

3.2.1.2. Stiffness and deflection

Members should be designed so that excessive deflections for a given particular use condition are avoided. For example, deflection of floors when fully loaded should not exceed 0.003 of the span. Repetitive member should be designed with mean value of the modulus of elasticity, E, while main components should be proportioned taking the minimum value of E;

3.2.1.3. Lateral support

Depth to breath ratio f rectangular beams are restrited to maximum values according to the degree of lateral support;

3.2.1.4. Notched members

The effective depth should be taken as the minimum depth of the net cross section;

3.2.1.5. Built-up members

Should be provided with web stiffeners to ensure stability at all points of concentrated loads.

3.2.2. Compression members

The following points are brought under attention in the design of compression members:

3.2.2.1. Effective lengths

To be taken according to end fixicity and lateral support conditions;

3.2.2.2. Maximum slenderness ratio

180 for loads resulting from dead weights and superimposed loads and 250 otherwise;

3.2.2.3. Compression members subject to bending

Total calculated compression stress shall not exceed the permissible stress in compression. Special consid-

TABLE 3-1

APPROXIMATE STREGTH RATIOS OF THE STRESS GRADES SPECIFIED IN THE MALAYSIAN STANDARD MS544:1978

Property	Stress grade (%)		
	Select	Standard	Common
1. Bending and tension parallel	80	63	50
2. Compression parallel	80	63	50
3. Compression perpendicular	85	80	75
4. Shear parallel	70	55	45

TABLE 3-2

GREEN* STRESSES AND MODULI OF ELASTICITY FOR STRENGTH GROUPS (PSI)

Strength group	Grade	Bending and tension parallel to grain	Compression parallel to grain	Compression perpendicular to grain	Shear parallel to grain	Modulus of elasticity 10^3 Mean	Modulus of elasticity 10^3 Minimum
A	Basic	3,000	2,500	250	400	2,000	1,250
	Select	2,400	2,000	210	280		
	Standard	1,850	1,550	200	220		
	Common	1,500	1,250	180	180		
B	Basic	2,500	2,000	150	300	1,600	900
	Select	2,000	1,600	120	210		
	Standard	1,500	1,250	120	160		
	Common	1,250	1,000	110	130		
C	Basic	1,800	1,400	100	200	1,300	750
	Select	1,400	1,100	85	140		
	Standard	1,100	850	80	110		
	Common	900	700	75	90		
D	Basic	1,100	950	60	200	830	430
	Select	850	750	50	140		
	Standard	650	550	45	110		
	Common	550	470	40	90		

* MC > 19%.

Source: Malaysian Standard MS 544 : 1978 (31)

TABLE 3-3

DRY* STRESS AND MODULI OF ELASTICITY FOR STRENGTH GROUPS (PSI)

Strength group	Grade	Bending and tension parallel to grain	Compression parallel to grain	Compression perpendicular to grain	Shear parallel to grain	Modulus of elasticity Mean (10^3)	Modulus of elasticity Minimum (10^3)
A	Basic	3,660	3,230	280	470	2,140	1,400
	Select	2,900	2,550	230	330		
	Standard	2,300	2,000	220	260		
	Common	1,800	1,600	210	210		
B	Basic	2,880	2,330	180	310	1,700	950
	Select	2,300	1,850	150	220		
	Standard	1,800	1,450	140	170		
	Common	1,400	1,150	130	130		
C	Basic	2,100	1,600	110	210	1,350	800
	Select	1,650	1,250	90	150		
	Standard	1,300	1,000	80	110		
	Common	1,050	800	80	90		
D	Basic	1,400	1,200	90	200	950	450
	Select	1,100	950	75	140		
	Standard	800	70	70	110		
	Common	700	600	65	90		

* MC ≤ 19%.

Source: Malaysian Standard MS 544 : 1978 (31)

eration is given to slender members, in a similar way as described in the codes already reviewed in preceding paragraphs;

3.2.2.4. Notching and drilling

Allowances for the holes should be made in the design;

3.2.2.5. Spaced columns

Consideration is given as to limit the space between individual shafts and to guarantee that the rigidity of the system is appropriate. Recommendations are made on how to calculate permissible loads of spaced columns;

3.2.2.6. Compression members in triangulated frameworks

Recomendations are presented for determining the slenderness ratio according to the conditions of end fixicity and lateral support for continuous an non-continuous members.

3.2.3. Tension members

3.2.3.1. Effective cross-section

Allowance must be made for the reduction in area caused by sinking, holes, notches.

3.2.3.2. Combined tension/bending

Members subject to bending and axial loading simultaneously should be designed so that the total tension stress does not exceed the permissible stress.

3.2.4. Joints

In order to simplify design, the Malaysian code, like the Australian, provides for classifying all commercial species in joint strength groups, which are designated by J1 to J5. However, contrary to the Australian code, there is no explicit reference to these groups being based on the density of the species.

In the same fashion as codes previously reviewed, the following considerations are made:

3.2.4.1. Basic lateral loads

For green and dry wood, perpendicular and parallel to grain, given for each joint group with respect to nails, screws, and bolts of various diameters.

3.2.4.2. Details about spacing

Edge and end distance, duration of loading, etc., are also given.

3.2.4.3. Other types of joints

Dry basic loads are given for all joint groups for commercially-available split-ring and shear-plate connection, as well details on their spacing, end and edge distance, etc.

3.2.5. Design by experimental testing

The Malaysian code makes provision for accepting experimental testing as an alternative to calculation of timber structures, where circumstances require, but always by agreement between the parties concerned.

Consideration is given to many aspects of the test method such as: pre-loading, deflection test, strength test, number of components to be tested, etc. Acceptance is conditioned to the lowest ultimate load recorded, which must be at least 2.5 times the design load when only one structure is tested, or 2.0 times the design load if five similar components are tested.

4. PHILIPPINE TIMBER CODE

4.1. General considerations

The Philippine timber code (29) also follows the traditional engineering methods based on the theory of elasticity. Members should be proportioned so that calculated stresses will never exceed the allowable unit stresses specified for the respective species and grade used.

4.1.1. Stress grades and working stresses

The code contains the description of three visual stress grades, based on strength ratios of 80%, 67% and 56%, applied on bending strength.

Working stresses, for use in dry conditions, are given for 21 tropical hardwoods for the following properties:

- bending and tension parallel to grain
- modulus of elasticity in bending
- compression parallel to grain
- compression perpendicular to grain
- shear parallel to grain.

4.1.2. Adjustments of working stresses

The Philippine code makes provisions for the following adjustments:

a) increase of 10% in the modulus of elasticity of lumber that is surface seasoned before loading to the maximum allowable load;

b) decrease of 10% for the compression parallel to grain and 33 1/3% for the compression perpendicular to grain for use in continuously wet conditions;

c) decrease of 10% in all working stresses when the member is fully stressed to maximum allowable stress for more than 10 years, under the condition of maximum design load;

d) increase in the allowable units stresses when the full maximum load is of short duration, as follows:
15% for 2-month duration
25% for 7-day duration
33 1/3% for one day duration
100% for impact loads

e) adjustment for shear stresses near the support of beams, for loads closer than three times the depth of the beam from the support;

f) increase in compression perpendicular to grain for bearings shorter than 150 mm and located 75 mm or more from the end of a timber.;

g) increase of 150 % for horizontal shear in joint details.

4.2. General design procedures

4.2.1. Columns

a) Solid columns

The calculated stress shall not exceed the working stress in compression parallel, C,

$$\frac{P}{A} = \frac{3.619\,E}{(L/r)^2} \leq C$$

columns shall be limited to L = 50d, except for length of components of spaced columns, which can go up to L = 80d;

b) Spaced columns: shall be designed according to principles accepted to the building official.

4.2.2. Joints

In order to make the design of joints more convenient, the Philippine code establishes three strength groups according to specific gravity, as follows:

Group I	0.87 - 0.65 specific gravity
Group II	0.48 - 0.62
Group III	0.35 - 0.44

a) Bolts

For each one these groups the code gives allowable loads, parallel and perpendicular to grain, for bolts in double shear with diameters varying from 12 mm to 32 mm, and length from 40 mm to 305 mm. Loads at angle to grain shall be calculated using Hankingson's formula. Details are given regarding spacing, edge and distance of bolts.

b) Shear plate, toothed-ring and split-ring

Allowable loads are given for one shear-plate unit and bolt in single shear for the three joint groups, for loading parallel and perpendicular to grain, for two sizes of plate and bolt in single shear for the three joint groups, for loading parallel and perpendicular to grain, for two sizes of plate and bolt and various combinations of lumber thickness.

The same information is provided for one toothed-ring unit and bolt and one split-ring with bolt in single shear. Details on spacing and end distance for loading perpendicular to grain are given for the three types of connector.

c) Wood screws and nails

Design loads referring to the three joint groups are provided for wood screws and nails, both for withdrawal load from side penetration, for each 25 mm of penetration, and for lateral load in side grain.

5. BRAZILIAN SPECIFICATION FOR THE DESIGN AND CONSTRUCTION OF WOODEN STRUCTURES: ABNT NBR 7190 - 1982

5.1. General considerations

In Brazil, the design and construction of wood structures is governed by Standard NB-7190, issued in 1982 by the Brazilian Association – ABNT (4).

This standard follows the conventional approach to design. Loads and resistances are specified as deterministic values, and the design procedures are based on the theory of elasticity. All the allowable stresses are derived from mean ultimate values obtained for small clear specimens tested in green condition according to Method ABNT MB 26-1953.

5.2. Design procedures

5.2.1. Bending members

The allowable stress in bending, F_b, is 15% of the average value of the modulus of rupture of small clear specimens. The allowable shear stress, F_v, is 10% of the average ultimate horizontal shear value obtained from small clear specimens.

5.2.2. Compression members

For columns having a slenderness index, λ, of 40 or less, the allowable stress in compression parallel to grain, F_c, is 20% of the average crushing strength, obtained from small clear specimens.

The slenderness index, λ, is defined as the ratio between the buckling length, l, (pin-end conditions) and the radius of giration, i:

$$\lambda = \frac{l}{i} \cdot$$

For $40 < \lambda < \lambda_o$

$$F'_c = F_c \left[1 - \frac{1}{3} \frac{\lambda - 40}{\lambda_o - 40}\right]$$

For $\lambda > \lambda_o$ (long columns)

$$F_c' = \frac{2}{3} \left(\frac{\lambda_o}{\lambda}\right)^2 F_c$$

where

$$\lambda_o = \left(\frac{3\pi^2 E}{8 F_c}\right)^{1/2}$$

5.2.3. Tension members

The allowable stress in tension parallel to grain F_t, is the same as the allowable stress in bending, F_b, i.e., 15% of the average value of the modulus of rupture.

5.2.4. Combined loads

The total tensile stress resulting from the combined flexural and axial loads shall not exceed the allowed stress in tension, F_t.

When $\lambda < \lambda_o$, the compression stress shall not exceed the value given by the following expression:

$$F_{bc} = F'_c + (F_b - F'_c)\, e$$

where

$$e = \frac{F_b}{F_{bc}} = \frac{|M|/W}{|M|/W + N/A}$$

when $\lambda > \lambda_o$ and $e > e_o$

$$F_{bc} = \frac{2}{3} F_c + (F_b - \frac{2}{3} F_c)(e - e_o)$$

where

$$e_o = 1 - (\frac{\lambda}{\lambda_o})^2$$

and |M| is the absolute value of the bending moment, W is the section modulus, N is the axial compression force, and A is the cross-sectional area.

When $\lambda > \lambda_o$ and $e < e_o$, the bending stress should be disregarded and the column calculated as indicated by the second equation in item 5.2.2. (long columns).

5.2.5. Compression perpendicular to grain stresses

The allowable stress for compression perpendicular to grain is 6% of the crushing strength obtained from small clear specimens. This value can be increased up to 100% if the bearing area is at least 15 cm (6 in.) away from the end of the piece.

5.2.6. Joints

The NBR-7190 Standard does not specify any working load for joints, leaving them to the design engineer. However, some recommendations are made with respect to joints, which were initially intended for the construction of heavy structures:

a) minimum bolt diameter shall be 16mm for main structural members and 9mm for secondary members. The same requirement for thickness apply to metal plates;

b) washers shall have a bearing area that is large enough to exert the full axial load of the bolt without exceeding the allowable stress of the wood in compression perpendicular to grain;

c) all joints should be properly centered; no joint should depend on one bolt only;

d) nailed joints in main structural members should have their allowable load determined through experimental testing;

e) finally, details are given with regard to edge and end distance and spacing of bolts and connectors.

6. JAPANESE BUILDING CODES FOR WOOD CONSTRUCTION

The information presented in this chapter comes from the English translation of two documents: "The Establishment of the Technical Standards for Ensuring Structural Safety of Wood Frame Construction." (8), and the JAS standard "Structural Lumber for Wood Frame Construction" (9).

6.1. General considerations

About 60% of new houses in Japan are built with wood, mostly softwoods imported from North America and USSR, and hardwoods from Southeast Asia. The traditional Japanese house construction is of the post-and-beam type, with preferred species for each type of structural and non-structural members.

In this type of construction some structural members remain apparent, thus also emphasizing clear face grades.

In the last decade there has been a slow but steady interest in plataform frame construction following the North American building system. The Japanese Agricultural Standard - JAS, specifications for dressed softwood structure lumber for wood frame construction consider two species groups, SI and SII, which include Japanese as well as North American species, and two product categories: Framing A lumber and Framing B lumber.

Framing A lumber is intended for use where members require high bending strength and stiffness. It contains four grades:

- special (similar to NDS Select Structural)
- grade 1 (≃ NGR nº 1)
- grade 2 (≃ NGR nº 2)
- grade 3 (≃ NGR nº 3 and Stud grade)

Framing B is mostly used as compressional structural members and corresponds to the NGR light framing grades. It contains three grades:

- construction (≃ NGR construction grade)
- standard (≃ NGR standard grade)
- utility (≃ NGR utility grade)

Cross-sectional dimensions include nominal 2 inches (40 mm green, 38 mm dry) thickness by 3,4,6,8,10 and 12 inches (65,90,143,190,241 and 292 mm green; 64,89,140,184,235 and 286 mm dry) in width and 4 inch square columns (90 x 90 mm green, 89 x 89 mm dry). Lumber with moisture content not exceeding 19% is considered dry.

According to a paper by Briggs and Dickens (7) JAS grades are more strict than the NGR requirements, specially with regard to knot displacement and wane.

6.2. Design procedures

Unfortunately the information presented in the documents available did not contain many details on the design of wooden structures, except for those given below.

6.2.1 General requirements

a) provisions shall not apply to tea ceremony houses, summer garden house or other similar buildings, nor to storerooms, barns or similar buildings with a total floor area less than 10 square meters;

b) timber to be used in principal structural members shall be free of defects affecting wood strength.

6.2.2 Columns requirements (main ones)

The ratio of the smallest cross-sectional dimension of a column and the vertical distance between two horizontal members is limited to a range of 1/20 to 1/30, depending on the type of building, story considered and space between columns:

a) the smallest cross-sectional dimension of a column with structural responsibilities in buildings having three or more stories shall not be less than 13.5 cm;

b) notches: cutting one-third or more of the column cross section requires reinforcement;

c) corner columns of buildings having two or more stories must be continuous or otherwise be reinforced to perform as such;

d) slenderness ratio of main columns shall not exceed 150.

6.2.3 Beams

a) notches are not allowed in the lower side of beams in the vicinity of their middle portion.

6.2.4 Braces

a) braces acting in tension shall have a minimum cross-sectional dimensions of 1.5 cm x 9.0 cm;

b) braces acting in compression shall be at least 3 cm in thickness and 9 cm in width;

c) notching is not allowed in braces.

7. ANDEAN PACT HANDBOOK FOR DESIGN OF WOOD STRUCTURES

7.1. General Considerations

Five Andean countries (Bolivia, Colombia, Equador, Peru and Venezuela) have developed, under the JUNAC - Cartagena Agreement, a joint project aiming at the technological development of their tropical forest resources. This project, entitled PADT-REFORT, has produced a wood design handbook that was published in 1982 (20).

The design procedures described in this handbook were partly based on the experimental findings resulting from an extensive testing program involving the five countries.

Physical and mechanical properties of 104 tropical woods of commercial importance, including two softwoods, were determined for small clear specimens and for full-size beams.

On the basis of these experimental results, a strength grouping system and a single visual grade system were proposed, together with a preferred series of lumber cross-sectional dimensions.

7.2. General design procedures

The design procedures recommended by the JUNAC handbook are based on accepted engineering methods of structural analysis for linear elastic behaviour of structures. Each member is to be designed so that calculated stresses do not exceed allowable stresses, and that deflections remain within prescribed acceptable limits.

Allowable stresses, as presented in Table 7-1, are based on strength properties resulting from the testing of small clear specimens, modified by factors given in Table 7-2.

$$\text{Allowable stress} = \frac{FC}{FS} \cdot \frac{FT}{FDC} \cdot \text{maximum stress}$$

FC = factor of strength reduction due to quality (strength ratio)

FT = size factor

FS = safety and serviceability factor

FDC = duration of load effect

TABLE 7-1

ALLOWABLE STRESSES (kg/cm²) PRESCRIBED BY THE JUNAC WOOD DESIGN HANDBOOK*

Species strength group	Bending f_m	Tension parallel f_t	Compression parallel f_c	Compression perpendicular $f_{c\perp}$	Horizontal shear f_v	Modulus of Elasticity $E_{0.05}$	E_{mean}
A	210	145	145	40	15	95000	130000
B	150	105	110	28	12	75000	100000
C	100	75	80	15	8	55000	90000

* Values for green wood, which may be used for dry wood.

Source: JUNAC (20)

TABLE 7-2

REDUCTION FACTORS ADOPTED BY THE JUNAC DESIGN HANDBOOK

Factor	Bending	Compression Parallel	Horizontal Shear	Compression Perpendicular
FC	0.80	–	–	–
FT	0.90	–	–	–
FS	2.00	1.60	4.00	1.60
FDC	1.15	1.25	–	–

7.2.1. Reduction factors

7.2.1.1. Strength reduction factor - FC

This factor was established by comparing small clear test results with those obtained in testing visually-graded beams 4 cm x 14 cm (2 in. x 6 in. commercial size), with spans between 2,600 m and 3,20 m.

A reduction factor of 0,80 is adopted for all strength groups.

7.2.1.2. Safety and serviceability factor - FS

This factor has the objective of making allowances for material variability, type and consequences of failure, uncertainties due to manufacture, etc. It varies between 1.6 and 4.0, depending on the property considered, as shown in the table above.

7.2.1.3. Size factor - FT

As mentioned in some of the codes previously analysed in this report, the size factor takes into account the difference in depth of the beam tested in laboratory as compared to the depth of full size beams. Tension members also show strength reduction due to size.

$$FT = \left(\frac{5}{h}\right)^{1/9}$$

h = depth of the beam in cm.

7.2.1.4. Duration of load effect - FDC

Although it is recognised that the maximum stress sustained by wood decreases with duration of load, the argument is made that such time effect is less pronounced at the actual stress level present in strutural members in real life. The proposed values for FDC, which apply only to bending and compression parallel to grain, are lower than those prescribed in other codes.

7.3. Design of bending members

7.3.1. Deflections

For houses, calculated deflections are limited to a fraction of the span varying between 1/250 and 1/350, depending on the loading condition and ceiling material; other buildings may have less strict limits.

Deflections should be calculated using $E_{0.05}$; for deflections caused by long-term loads, the calculated values should be multiplied by 1.2 or 1.8, for dry and green wood respectively.

7.3.2. Allowable loads

Bending members shall be designed using traditional elastic theory for stress analysis. The calculated extreme fiber stresses, horizontal shear stresses and compression perpendicular to grain stresses (bearings), shall not exceed the values given in Table 7-1.

In addition to extreme fiber stress, horizontal shear stresses and stresses at the supports, there are also recommendations about lateral stability of slender beams.

Extensive design-aid material is given to illustrate the use of beams of preferred cross sectional dimensions under various loading conditions.

7.4. Design of compression members

Similar to other codes, columns are classified according to their slenderness ratio, λ, in three types:

- short columns $\lambda < 10$
- intermediate columns $10 \leq \lambda \leq C_k$

$$C_k = 0.7025 \left(\frac{E}{f_c}\right)^{1/2} \text{ (rectangular sections)}$$

- long columns $C_k < \lambda < 50$

Columns having λ greater than 50 shall not be allowed.

7.4.1. Allowable loads

7.4.1.1. Columns in axial compression only

Calculated stresses due to specified loads should not exceed the allowable stress f_c given in Table 7.1.

The value of E to be used in the equations presented below must be $E_{0.05}$.

$$N_{adm} = f_c \; A$$

A is the cross-sectional area

– Intermediate columns

$$N_{adm} = f_c A \left[1 - \frac{1}{3} \left(\frac{\lambda}{C_k} \right)^4 \right]$$

– Long columns

$$N_{adm} = 0{,}329 \; \frac{EA}{(\lambda)^2}$$

7.4.1.2. Combined axial compression and bending

$$\frac{N}{N_{adm}} + \frac{k_m |M|}{Z f_m} < 1$$

where

$$k_m = \frac{1}{1 - 1.5\, N/N_{cri}}$$

k_m = moment amplification factor due to axial compression

$|M|$ = absolute value of maximum moment

Z = section modulus

N = applied axial load

N_{cri} = Euler critical load $\left(\frac{\pi^2 EI}{l_e^2}\right)$

As it was the case for the design of beams, the JUNAC handbook also presents abundant material illustrating the design of columns and multiple-member structures under compression. In addition, a full-chapter is dedicated to the design of bracing walls and another to the design of light frames, specially trusses.

7.4.1.3. Combined axial tension and bending

$$\frac{N}{A f_t} + \frac{|M|}{Z f_m} < 1$$

N = tension load

$|M|$ = absolute value of maximum moment

A = cross-sectional area

Z = section modulus

f_t = allowable stress in axial tension

7.5. Joints

Detailed instructions are given for the design and construction of bolted and nailed joints with special emphasis on spacing, end and edge distance, load direction, etc. Allowable joint loads, for compression parallel and perpendicular, are given for bolts with diameters ranging form 1/4 to 3/4 inches and lengths of 2 to 9 cm, for the three strength groups, using green wood. Reduction factors are given for joints having more than one bolt along the same line parallel to the load direction.

Allowable loads for nails in single shear are given for the three strength classes, with nail diameters varying from 2.4 mm to 4.9 mm and lengths from 51 mm to 102 mm, green wood. Pre-boring is recommended for species belonging to strength group A. Modification factors to take into account situations other than single shear are given; for example nails in double shear are allowed 67% higher loads.

8. CIB STRUCTURAL TIMBER DESIGN CODE

8.1. General considerations

This code (12) has been prepared by CIB Working Group W18 - Timber Structures and served as the basis for the preparation of Eurocode-5, which is the EEC recommended code for the design of wood structures.

The Eurocodes, which are available for other materials such steel and concrete, are aimed at the harmonization of design rules as a means of promoting the integration and the competitiveness of the construction industry within the European Common Market.

Although Eurocode-5 is based on a semi-probabilistic approach to design, using the partial factor method, the CIB code is also applicable to deterministic methods, provided material properties are derived from characteristic values and suitable safety factors for strength and stiffness are available for the design calculations. However, it does not specify safety factors nor partial load coefficients and load combinations. They are left to the responsibility of national public authorities.

The design procedures recommended by the CIB code are based on the principle of structural mechanics, engineering design, and experimental data, interpreted statistically as far as possible. Design methods or materials other than those recommended by the CIB code may be used as long as their validity con be substantiated by analytical and engineering principles, reliable test data, or both.

8.2. Basic assumptions and design procedures

Mechanical properties of wood and wood-based materials are given as characteristics values for strength and stiffness. A characteristic value is defined as the population lower 5-percentile value directly applicable to a load duration of 3 to 5 minutes, at a temperature of 20° ± 2° C and a relative humidity of 0.65 ± 0.05, and estimated with a confidence level of 0.75. These conditions are also valid for the definition of mean values of some elastic properties given by the code.

In the case of tensile strength perpendicular to grain, the characteristic values are also related to a volume of 0.02 m^3. The characteristic relative density given for a species or species group is defined as the lower 5-percentile value with mass at a moisture content $w = 0$, and volume at a temperature of 20 ± 2°C and relative humidity of 0.65 ± 0.05.

8.2.1. Moisture classes

The CIB code specifies three moisture classes for structures dependent on moisture content :

Moisture class 1: when the material has a moisture content corresponding to a temperature of 20 ± 2°C and to a relative humidity only occasionally over 0.65;

Moisture class 2: same as class 1, but with relative humidity occasionally over 0.85;

Moisture class 3: all other climatic conditions.

In the conditions defined above, the equilibrium moisture content of most softwoods will not exceed 0.12 in Moisture Class 1 and 0.18 in Moisture Class 2.

8.2.2. Load-duration classes

As in the codes previously reviewed in this paper, the CIB code also takes into account the different behaviour of wood under loads of different durations. Five classes of load duration are recognized:

- long term: over 10^5 hours (>10 years)
- medium-term: 10^4 hours (one year)
- short-term: 10^2 hours (one week)
- very short-term: under 10 hours
- instantaneous.

8.2.3. Limit states

The objective of design is to supply specifications and recommendations for the construction of safe and economical structures. Structures should be designed so that there is a prescribed safety against reaching a limit state, be it an ultimate limit state or a serviceability limit state.

A limit state is defined as a state in which one of the criteria relating to the load-bearing capacity of the structure or to its conditions of service is no longer valid.

An ultimate limit states correspond to the maximum load-carrying capacity or to complete unserviceability.

On the other hand, serviceability limit states refer to criteria governing normal use of the structure such as: excessive deflections, excessive vibrations, local damages, etc.

8.2.4. Verification of design

The general condition of safety, i.e., the condition for the actual limit state not being reached is expressed as:

$$\theta\,(F, f, a, \mu, C) > 0$$

and the design criterion will be:

$$\theta\,(F_d, f_d, a_d, \mu_d, C) > 0$$

where

F — represents actions

f — represents material properties

a — represents geometrical parameters

μ — are quantities covering the uncertainties of the calculation model

C — are constants including preselected design constraints

θ ( ) — represents the limit state function

d — denotes design value (subscript)

The CIB code recommends that the values for actions, partial coefficients and load combinations to be taken into account in design should be prescribed by national public authorities. However, among the various Eurocodes sponsored by EEC, one code shall deal specifically with actions acting on structures (13).

The following values should be given for actions:

F_k characteristic value

$\Psi_o\, F_k$ combination value

Based on those characteristic values, the design value is obtained by multiplying them by a partial load coefficient γ_f

$$F_d = \gamma_f F_k$$

or

$$F_d = \gamma_f\ \Psi_o\ F_k$$

and the design load combination given as

$$\sum_{i=1}^{n} \gamma_{fi}\, F_{ki} + \sum_{j=1}^{m} \gamma_{fj}\, \Psi_{oj}\, F_{kj}$$

The design values of strength parameters should be obtained from the characteristic values, modified according to climate class and load-duration class, by division by a partial material coefficient γ_m.

$$f_d = f_k / \gamma_m$$

$$E_d = E_k / \gamma_m$$

For serviceability limit states $\gamma_m = 1.0$ and deflections are calculated with the mean values of the elastic properties; for ultimate limit states γ_m should be prescribed by the concerned public authority.

Usually the geometric parameters are assumed to be those specified in the design, a. In case deviation from this value may have a significant effect on the structure, then:

$a_d = a + \Delta a$

or

$a_d = a - \Delta a$

where a is the characteristic value and Δ a takes into account the importance of variations in a and the given tolerance limits for a.

Finally, ultimate limit states may be calculated by using elastic or plastic theories, according to the response of the structure to the actions. However, the characteristic values given in the item 8.2.4 are those derived from test loads by the theory of linear elasticity, which therefore should also be used in the design of individual members. Serviceability limit states are, in general, calculated according to elastic theory. When design is made by testing full-size structures, the procedures should follow the appropriate RILEM-CIB Standard.

8.3. Structural timber

Timber to be used as structural members should be graded in accordance with rules ensuring that the required mechanical properties of the timber are satisfactory. These strength grading rules may be based on visual grading or machine grading.

8.3.1. Standard strength and density classes

The CIB code considers 13 standard strength classes, with the characteristic values of bending strength ranging from 5.0 to 75 MPa and modulus of elasticity from 2300 to 12200 MPa. The ratio between the bending strength of one class and that of the class preceding it is about 1.25; for modulus of elasticity this ratio is about 1.15. The ratio between tension parallel to grain strength and bending strength ranges from approximately 0.55 for the lower strength classes to 0.70 for the stronger classes. As shown in Table 8-1, the target characteristic values are given for the principal properties.

In order to be assigned to one of the strength classes, a given grade must have characteristic values of bending strength, tension strength and modulus of elasticity not less than the target value, and the values of compression strength and shear strength should exceed the requirements of the nearest lower class.

TABLE 8-1

STANDARD STRENGTH CLASSES. CHARACTERISTIC VALUES IN MPa.

Property		SC5	SC6	SC8	SC10	SC12	SC15	SC19
Bending	f_m	5.0	6.0	7.5	9.5	12	15	19
Tension parallel to grain	$f_{t,0}$	2.5	3.2	4.1	5.4	7.0	9.1	11.8
Compress. parallel to grain	$f_{c,0}$	5.8	7.0	8.4	10.0	12.0	14.5	17.5
Shear parallel to grain	f_v	1.0	1.1	1.3	1.5	1.7	2.0	2.2
Modulus of elasticity	E_0	2300	2600	3000	3500	4000	4600	5300

TABLE 8-1 (continued)

Property		SC24	SC30	SC38	SC48	SC60	SC75
Bending	f_m	24	30	38	48	60	75
Tension parallel to grain	$f_{t,0}$	15.5	20	25	34	44	54
Compress. parallel to grain	$f_{c,0}$	21	25	30	36	43	52
Shear parallel to grain	f_v	2.6	3.0	3.5	4.0	4.6	5.2
Modulus of elasticity	E_0	6100	7000	8100	9300	10600	12200

Th^ density classes established by the CIB code are given in Table 8-2.

The ratio between the minimum characteristic values of one density class and that of the class preceding it is aproximately 1.25.

For those calculations where mean values of modulus of elasticity and modulus of rigidity are required, they should be taken as

$E_{mean} = 1.4\, E_{o}$

$G_{mean} = 0.095\, E_{o}$

Finally, two strength properties, tension perpendicular to grain and compression perpendicular to grain, have their characteristic values expressed according to standard density classes, as presented in Table 8-2.

8.4. Design of basic members

8.4.1. General considerations

The design strength values to be applied in the equations used to define the sizes of structural members, or to verify their conditions with relation to limit states, shall be those obtained by:

- multiplying the characteristic values or mean elastic moduli by a modification factor k_{mod} shown in table 8-3, taking into accout the influence of moisture content and loading time, and
- dividing by the partial coefficient γ_m which is always $\gamma_m = 1.0$ for serviceability limit state; for ultimate limit state should be specified the relevant public authority, as state earlier.

8.4.2. Design of tension members

Stresses in tension members should satisfy the following conditions:

a) Tension parallel to grain

$\sigma_{t,d} \le f_{t,d}$

or using simplified notation,

$\sigma_t \le f_t$

b) Tension perpendicular to grain

$\sigma_t \le k_{vol,90}\, f_{t,90}$ or $\sigma_{t,d} \le k_{vol,90}\, f_{t,90,d}$

TABLE 8-2

STANDARD DENSITY CLASSES, CMINIMUM CHARACTERISTIC RELATIVE DENSITIES AND CHARACTERISTIC STRESSES

Property	Standard density class				
	D300	D400	D500	D600	D800
Minimum, characteristic relative density	0.32	0.40	0.50	0.63	0.78
Tension perpendicular (MPa) $f_{t,90}$	0.40	0.50	0.65	0.85	1.10
Compression perpendicular $f_{c,90}$	2.00	3.00	4.50	6.80	10.1

TABLE 8-3

MODIFICATION FACTOR k_{mod} TO CHARACTERISTIC AND MEAN VALUES

Load duration class	Values for strength calculations			Values for deformation calculations		
Moisture class	1 and	2	3	1	2	3
Long-term	0,55*	(0,35)	0,45 (0,30)	0,7	0,6	0,4
Medium-term	0,70	(0,50)	0,60 (0,40)	1	0,8	0,7
Short-term	0,80	(0,70)	0,70 (0,60)	1	0,8	0,7
Very short-term	0,95	(0,90)	0,80 (0,75)	1	0,8	0,7
Instantaneous	1,1	(1,1)	0,95 (0,95)	–	–	–

* Values in parentheses refer to tension perpendicular to grain.

where $k_{vol,90}$ takes into account the volume V uniforly loaded under tension perpendicular to the grain, if larger than 0.02 m^3

$$k_{vol,90} = 1 \quad \text{for} \quad V \leq 0.02m^3$$

$$k_{vol,90} = \frac{0.45}{V^{0.2}} \quad \text{for} \quad V > 0.02m^3$$

8.4.3. Compression without column effect

When the direction of the load forms an angle α with the grain (i.e. when the surface where the load is applied is at an angle $\theta = 90° - \alpha$ with the grain) the stresses in compression should satisfy the condition:

$$\sigma_c \leq f_{c,90} - (f_{c,0} - f_{c,90}) \sin\alpha$$

This condition makes sure that the compressive stress directly under the load is acceptable, but does not guarantee that an element in compression can carry the load in question.

For bearings on the side grain ($\alpha = 90°$),

$$\sigma_c \leq k_{c,90}\, f_{c,90}$$

where $k_{c,90}$ takes into account the higher loading capacity of narrow bearings, as recognized by most of the codes previously reviewed. For bearing less than 150 mm long and distant fron the end at least 100 mm and from the next loaded area at least 150 mm, $k_{c,90}$ may assume values given by:

$$k_{c,90} = \left(\frac{150}{l}\right)^{1/4} \qquad 1.0 \leq k_{c,90} \leq 1.8$$

where l is the length of the area loaded in compression perpendicular to grain.

8.4.4. Design of flexural members

Bending stresses should satisfy the following condition:

$$\sigma_m \leq k_{inst}\, f_m$$

where k_{inst} (≤ 1) is a modification factor that takes into account lateral buckling of slender beams. The reduction in strength due to lateral buckling can be disregarded, i.e. $k_{inst} = 1.0$, if the beam is laterally supported at the ends and if

$$\sigma_m = \left(\frac{f_m}{\sigma_{m,crit}}\right)^{1/2} \leq 0.75$$

where λ_m is the slenderness ratio for bending, and $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability.

When the initial curvature (deviation from straghtness) of the beam is less than $l/200$, k_{inst} may be calculated as

$\lambda_m < 0.75$	$k_{inst} = 1$
$0.75 < \lambda_m < 1.4$	$k_{inst} = 1.56 - 0.75\,\lambda_m$
$1.4 < \lambda_m$	$k_{inst} = \dfrac{1}{\lambda_m^{\,2}}$

For a beam with lateral support on the compression side throughout its length and torsion prevented at its supports k_{inst} may also be taken as 1.0. For beams with rectangular cross-section of depth d and breath b,

$$\lambda_m = \left[\frac{l_e h}{\pi b^2}\ \frac{f_{m,d}}{E_{0,d}}\ \left(\frac{E_{0,mean}}{G_{mean}}\right)^{1/2}\right]^{1/2}$$

TABLE 8-4

RELATIVE EFFECTIVE BEAM LENGTH l_e/l

Type of beam and load	l_e/l
Simply supported, uniform load or equal end moment	1.00
Simply supported, concentrated load at center	0.85
Cantilever, uniform load	0.60
Cantilever, concentrated end load	0.85
Cantilever end moment	1.00

where l_e is the beam efective length, which depends on the conditions of loading, lateral support and end fixicity, as given in Table 8-4.

8.4.5. Shear

Shear stresses should satisfy the general condition $\tau < f_v$

When vertical loads are applied near the supports on the top of beams of height h supported in the bottom, shear forces acting in a distance 2h or less from support can be calculated according to a reduced influence line, similar to provisions encountered in previously reviewed codes. Instructions are also given for the calculation of notched beams, specially for notches in the bottom condition. Notches are limited to half of the beam.

8.4.6. Torsion

Torsional stresses should satisfy the condition

$$\tau_{tor} \leq k_{tor} f_v$$

where k_{tor} is usually taken as 1.0.

8.4.7. Combined stresses

In addition to approximate empirical or semi-empirical expressions for calculating stresses in tapered beams, which are beyond the scope of this review, the CIB code focuses on combined stresses in a similar manner to that of codes previously analysed.

8.4.7.1. Tension and bending

The stresses should satisfy:

$$\frac{\sigma_t}{f_{t,o}} + \frac{\sigma_m}{f_m} \leq 1,$$

And in cross-sections where

$$\sigma_t + \sigma_m \leq 0 \quad , \quad \text{also}$$

$$|f_m| - \sigma_t \leq f_m$$

8.4.7.2. Compression and bending without column effect

In cross-sections where

$$\sigma_m + \sigma_c \leq 0 \quad , \quad \frac{|\sigma_c|}{f_{c,o}} + \frac{|\sigma_m|}{f_m} \leq 1$$

In cross-sections where

$$\sigma_m + \sigma_c \geq 0 \quad , \quad \sigma_m + \sigma_c \leq f_m$$

Slender beams under compression, or columns subject to lateral loads are analysed under item 8.4.7.4.

8.4.7.3. Torsion and shear

$$\left(\frac{\tau}{f_v}\right)^2 + \frac{\tau_{tor}}{k_{tor} f_v} \leq 1$$

8.4.7.4. Columns

For columns subjected to lateral loads, besides the conditions analysed under 8.4.7.2., the bending stresses resulting from initial curvature and from deflections must be taken into consideration.

The general condition is satisfied if:

$$\frac{|\sigma_c|}{k_c f_{c,o}} + \frac{|\sigma_m|}{f_m} \frac{1}{1 - \frac{k_c}{k_E} \frac{|\sigma_c|}{f_{c,o}}} \leq 1$$

where

σ_m are the bending stresses calculated without any regard to initial curvature or deflections

$$k_E = \frac{\sigma_E}{f_{c,o}} = \frac{\pi^2 E_o}{f_{c,o} \lambda^2} \qquad (\sigma_E = \text{Euler stress})$$

$k_c = 0.5\,(A - B)$, where

$$A = [1 + (1 + \eta\lambda \frac{f_{c,o}}{f_m})k_E), \text{ and}$$

$$B = [(1 + (1 + \eta\lambda \frac{f_{c,o}}{f_m})\, k_E)^2 - 4k_E)]^{1/2}$$

and

$e = \eta r \lambda$ e = maximum eccentricity of axial force

r = core radius

λ = slenderness ratio

8.5. Joints

8.5.1. General considerations

In contrast with some of the codes analyzed so far, the CIB code does not present tables listing the recommended values (allowable or characteristic) of joint loads for the different types of connectors.

The general approach is to base the determination of characteristic load-carrying capacity on tests carried out in conformity with standard methods. Another distinct point is that the CIB code makes mention of slip values, not considered by the other codes reviewed.

Consideration is given to multiple-fastener joints, joints with more than one type of fastener, arrangement of fasteners in the joint, spacings and distances to the ends and edges, etc.

8.5.2. Nails and staples

Nails in end grain should be considered incapable of transmitting force. Each joint should have at least two nails. For laterally-loaded nails, timber-to-timber joints, the characteristic load-carrying capacity in Newtons per shear plane can be determined by:

$F_k = k\, d^{\beta}$

where

d = nail diameter

k, β = factors that depend on nail type and its yield moment,wood species and grade (density), conditions of driving (pre-boring), etc., and which must be determined by testing.

For more than 10 nails in line, the load-carrying capacity of the extreme nails should be reduced by one-third.

For a load F not exceeding one third of the characteristic load-carrying capacity F_k, the corresponding joint slip u is given by

$u = 0.5\, d\, (F/F_k)^{1.5}$

For steel-to-timber joints the load carrying capacity, as determined for timber-to-timber joints, may be multiplied by 1.25; staggering is not required.

When nails driven across the grain are subject to axial loads, the characteristic withdrawal resistance F is the minimum of the three values given by:

$f_1 d$

$f_1 dh + f_2 d^2$ (smooth nails)

$f_2 d^2$ (threaded nails)

where f_1 and f_2, which depends mainly on the type of nail, timber species and grade (especially density), must be determined by testing.

Staples driven across the grain may be interpreted as two slender nails each, if the angle formed by the crown and grain direction is greater than 30°; if this angle is smaller, the load-carrying capacity should be multiplied by 0.7.

8.5.3. Bolts and dowels

For timber-to-timber Joints, the characteristic load-carrying capacity F in Newtons per shear plane for bolts with a yeld strength of at least 240 MPa is the smallest of values given by:

a) $18\, \rho\, (k_{\alpha,1} t_1 + k_{\alpha,2} t_2) d$ (only for 2-member joints)

b) $35\, \rho\, k_{\alpha,2} t_2 d$ (only for 3-member joints)

c) $70\, \rho\, k_{\alpha,1} t_1 d$

d) $75\, d^2 \rho\, [(\dfrac{k_{\alpha,1} + k_{\alpha,2}}{2} \cdot \dfrac{f_y}{240})]^{1/2}$

where ρ is the relative density of the wood,

t_1 and t_2 are timber thickness in mm. In a 3-member joint subscript 1 denotes the side member and subscript 2 the middle member. In 2-member joints the subscripts are chosen so that $k_{\alpha,1} t_1 \leq k_{\alpha,2} t_2$.

d is bolt diameter in mm.

$k_{\alpha,1}$ $k_{\alpha,2}$ are factors taking into consideration the influence of the angle between force at the joint and the direction of the grain:

$$k_{\alpha,1} \text{ (or } k_{\alpha,2}) = \frac{k_{90}}{k_{90} \cos^2\alpha + \text{sen}^2\alpha} \quad k_{90} = 0.45 + 8d^{-1.5} \; (d_{\text{in mm}})$$

Similar to the case with nails, joints with more than 4 bolts in line must have the load-carrying capacity of the extra bolts reduced by one-third.

The shear force V produced by bolts or dowels must not exceed two thirds of the shear strength of the wood

$$V \leq \frac{2}{3} f_v\, b_e\, t$$

f_v = design value for horizontal shear

b_e = distance from the loaded edge to the furthest bolt

t = thickness of the member

For steel-to-timber joints, when the side plates are of steel, t_1 and t_2 are taken as the thickness of the wood member. If the steel plate is the middle memeber, the formula given under 8.5.1.2.b is omitted and the values of formula 8.5.1.2.d should be multiplied by 1.4.

The rules given for bolts also apply to dowels, which are smooth steel rods, i.e., bolts without head, but load-carrying capacity should be multiplied by 1.25.

8.5.4. Wood and lag screws

For timber-to-timber joints, the load-carrying capacity, expressed in Newtons, of laterally-loaded screws with a yield strength of at least 240 MPa, screwed perpendicularly to grain, is the smallest of the values given by:

a) $70\, \rho\, k_{\alpha,1}\, t\, d$

b) $75\, d^2 [\rho\, \dfrac{k_{\alpha,1} + k_{\alpha,2}}{2} \quad \dfrac{f_y}{240}]^{1/2}$

where

t = thickness in mm. of the timber

d = diameter in mm. of the screw, mensured on the smooth shank

$k_{\alpha,1}$ $k_{\alpha,2}$ factors, similar to those given for nails, that take into account the influence of the angle between the force int the joint and the direction of the grain in the member under the screw head ($k_{\alpha,1}$) and the member receiving the point ($k_{\alpha,2}$).

Screws in end grain should normally be considered incapable of transmitting force. Similar to bolts and dowels, screws should not not generate shear stresses greater than two-thirds the shear strength of the wood at the joints.

For steel-to-timber joints, only equations b under 8.5.4. applies and the resulting characteristic load shall be multiplied by 1.25.

When screws driven perpendicularly to grain are subject to axial loading, the characteristic withdrawal, expressed in Newtons, is given by:

$F=f_3(l_t-d)d$

where

d = diameter in mm. measured on the smooth shank

l_t = threaded length in mm. of the member receiving the screw

f_3 = factor that depends on the shape of the screw, timber species and grade

8.5.5. Timber connectors

Joints made with timber connectors should have their characteristic load-carrying capacity and deformation characteristics determined by testing.

The testing procedures should give consideration to:

- angle between force direction and the direction of grain
- diameter of bolts and screws
- dimensions or joined members
- spacing and distances to the ends and the edges
- manufacturing conditions.

Finally, the CIB code recommends that the load-carrying capacity of nail plates should be derived from standardized testing according to RILEM/CIB-3TT. In addition to the sections regarding the design of structural members and joints, the CIB code has specific sections focussing on design of components and special structures, construction, and fire resistance.

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