CIB-W18/17-14-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIMPLIFIED STATIC ANALYSIS AND DIMENSIONING OF TRUSSED RAFTERS

by

H Riberholt

Technical University of Denmark
Denmark

RAPPERSWIL

SWITZERLAND

MAY 1984

INDHOLDSFORTEGNELSE

1. INTRODUCTION

In [Riberholt, 1982] guidelines for static models of trussed rafters are given, and these are mainly thought applied to frame models. In this paper a simple calculation method is forwarded for the static analysis and dimensioning of trussed rafters.

It has been an object to forward a calculation method which is as simple as possible and still taking into account the essential effects.

Due to lack of time only W-trusses have been investigated, but it is expected that later on other types of trusses will be dealt with in a similar manner.

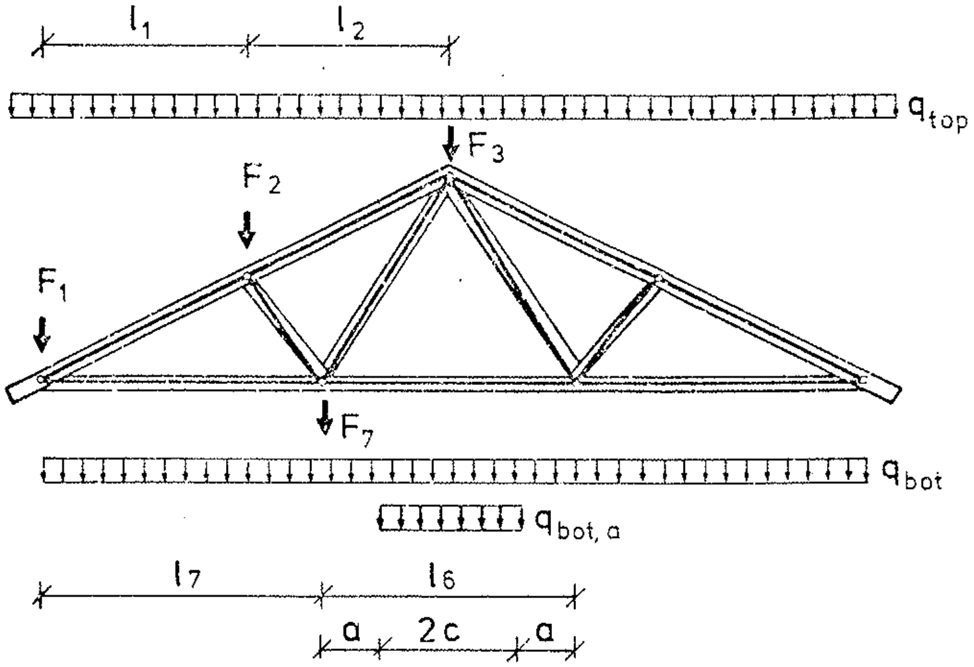
2. DESCRIPTION OF THE MOMENT COEFFICIENT METHOD

The moment coefficient method is applicable to trusses supported close to the heel joint. Cantilevered W-trusses may be designed according to [Feldborg & Johansen, 1981, chap. 8] or as described in section 4.2.

The method takes into account that the straight chords are continuous. If they contain splices, these must be placed, where the moments are close to zero.

Normal forces

The normal forces can be found under the assumption that all the connections in the truss act as pinned joints loaded with nodal forces. These are found by means of some approximate equilibrium requirements for the adjacent chord bays. Figure 2.1 gives some examples.



Examples: $F_2 = 1/2(l_1 + l_2)q_{top}$

$F_7 = 1/2(l_6 + l_7)q_{bot} + c\ q_{bot,a}$

Figure 2.1 Lattice model of a W-truss and nodal forces.

Moments

The moments in the chords from distributed line loads with constant intensity over each top chord or over the entire bottom chord can be found from

$$M = k_{mom} \cdot q_{90} \, l_{bay}^2 \qquad (2.1)$$

where

k_{mom} Moment coefficient

q_{90} Distributed line load perpendicular to the chord

l_{bay} Bay length as defined in figure 2.2.

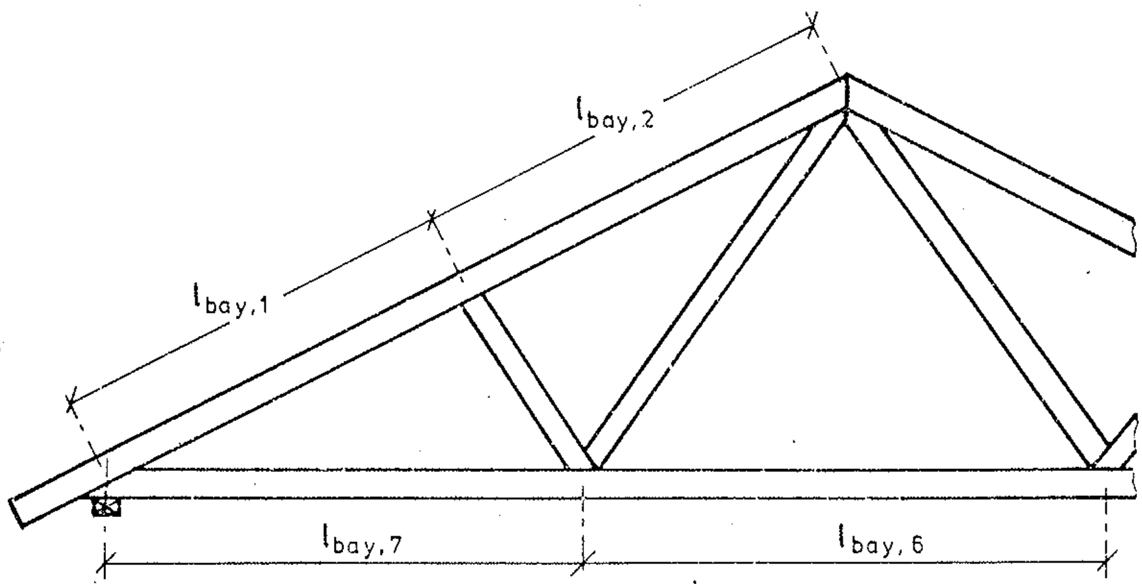


Figure 2.2 Bay lengths in a W-truss.

The bay lengths are defined as the lengths between the centre of the nodes or the middle of the supports.

If the previous distributed line loads cause moments of the same magnitude as the moments from concentrated forces or distributed loads of limited distribution these can be found under the assumption that the chord is a continuous beam over several bays. See section 3.3.7 and 4.1.

For example, the moments in the bottom chord in figure 2.1 arising from $q_{bot,a}$ can be calculated as if the bottom chord was a continuous beam supported by four simple supports.

The moment coefficients are determined by calibration with frame models as described later in this paper.

Internal eccentricities in the connections between chords and lattice must be taken into account. The eccentricity moments can either be incorporated into the moment coefficients or they can be calculated separately.

A more detailed description of how the moments are calculated is given in section 4.

Dimensioning

The dimensioning of the timber cross sections muse be done as described in chapter 2.6 in [Riberholt, 1982].

3. CALIBRATION OF MOMENT COEFFICIENTS

3.1. Regarded Phenomena, review

By analysing a W-truss it is found that the moment coefficients depend on the following. The are sensitive to some of the parameters and relatively insensitive to others.

I: Slope of the roof. The following have been analysed, 1:4, 1:3, 1:2.

II: Slip in the connections. There has been calculated with the following values, which are chosen from those given in [Feldborg & Johansen, 1981] for long term loaded trusses.

TABLE 3.1

Connections	Slip in mm Used for the stress analysis				
	No slip	A	B	C	D
Heel joint, slip in the direction of the top chord	0	1	2	4	6
Splice in the middle of the the bottom chord.	0	0.5	1.5	3	6
Connection at the ridge, slip in the direction of the thop chord.	0	0.5	1.5	3	6

A corresponds to nail plates with long teeth, B to nails driven directly through thin steel plate gussets and C to nailed plywood gussets.

III: Eccentricities in the connections between the diagonals and the chords. Two cases have been regarded, they are shown in figure 3.1. The ridge connection remained the same.

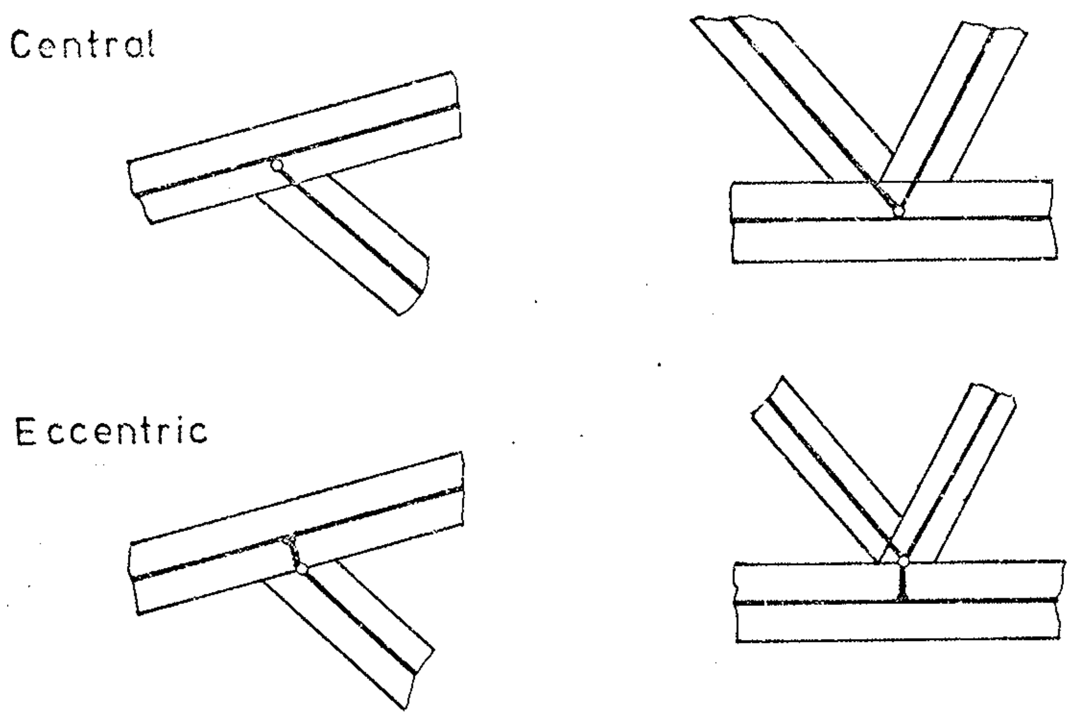


Figure 3.1 Central and eccentric connections, partly between the top chord and the diagonal, partly the k-joint in the bottom chord.

IV Eccentric support of the heel joint.

V The dimensions of the bottom and top chord.

VI The span of the truss.

VII The load on the top chord in proportion to the load on the bottom chord, e.g. heavy or light roof.

VIII Symmetric or asymmetric load on the top chord.

IX Attic load over a part of the bottom chord.

3.2. Method of calibration

The moment coefficients were calculated by means of a frame program which was especially developed for the task. Topologi, geometry, and loadings were generated automatically and so were the moment coefficients by means of equation (2.1), which gives

$$k_{mom} = M_{frame\ model}/(q_{90}\ l_{bay}^2) \qquad (3.1)$$

In the frame analysis the timber parts were assumed linear elastic. The connections were modelled as shown in figure 3.1 and 3.2, and the slip in a connection of the real truss was modelled by a prescribed slip between the members. There have been calculated with slip (mutual movement) values as given in table 3.1, which are assumed to be typical for different types of connections. It is namely expected that the dimensioning of the connections will result in that the fasteners are stressed approximately equally, independently of the magnitude of the force or the size of the connection.

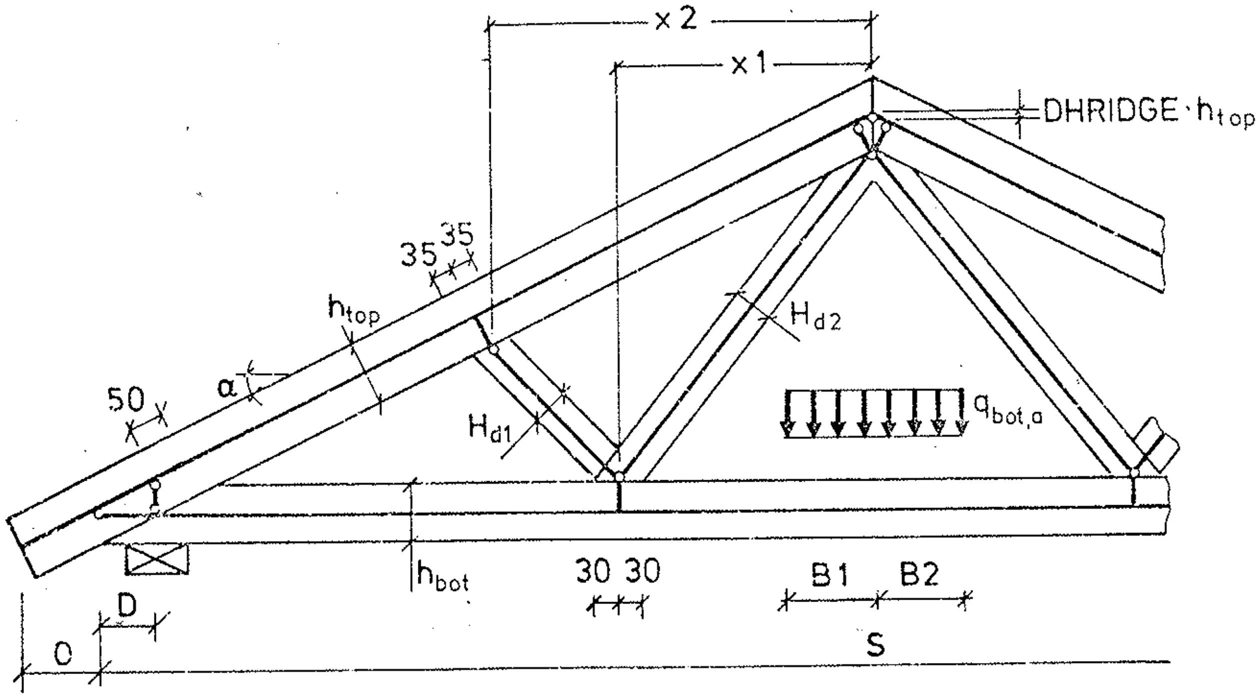


Figure 3.2 Frame model and geometry determining parameters. The pinned joint at the ridge can be shifted downwards in order to model that the compression force between the top chords is transferred in the lower part of the joint.

In the frame program the prescribed slip is treated as described in annex A.

Moment coefficients have been calculated for the maximum moment in the bay and at the nodes. It has been taken into account that the moment curve at the node in reality does not end up in a peak, but instead turn of due to contact pressure or forces in the connectors. As peak values there have been emploied the moments in a certain little distance of the node point corresponding to points within the connection domain, see [Riberholt, 1982]. Figure 3.2 gives some examples of the node distances in mm.

The lengths X1 and X2 are assigned values so that the lengths in the top and bottom chords approximately are equal, see also figure 2.2.

The dimensions of the timber parts have been chosen so that they correspond to what is given in a recently issued Danish span table [Wood Trussed Rafters, 1983].

3.3. Results of the calibration

The effects of the phenoma listed in section 3.1 have been investigated, either by varying one parameter or several parameters simultaneous corresponding to the following headings.

There has been used at reference truss with parameters as given in section 3.3.1, with a roof slope of 1:3 and with a slightly eccentric ridge connection DHRIDGE = 0.167.

3.3.1. Roof Slope, Connection Slip, Ridge Model

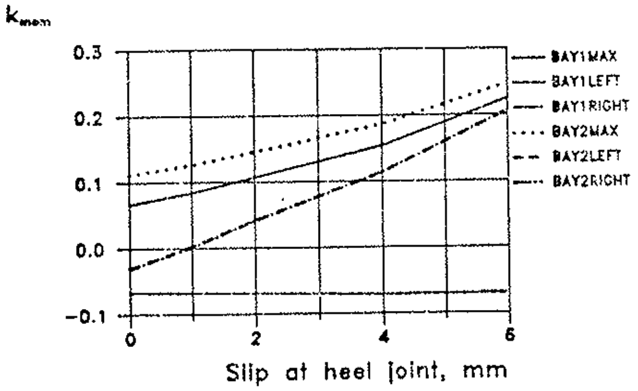
The other parameters were set to:

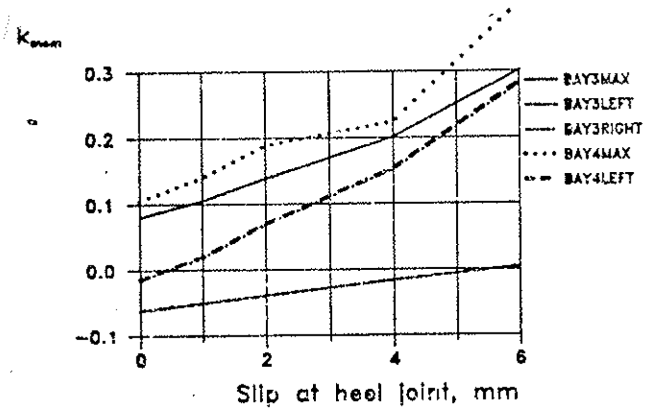
III Centric connections
IV Little support eccentricity, D = 70 mm
V H_t = 150 mm, H_b = 125 mm, Width = 50 mm
VI S = 8000 mm
VII Heavy roof q_{top} = 1.61 N/mm
VIII Symmetric top chord load
IX No partial attic load

Overhang	$0 =$ 500 mm	E_{moduli},	top & bot.	8400 MPa
X1	= 1310 mm		diag.	7200 MPa
X2	= 1950 mm			

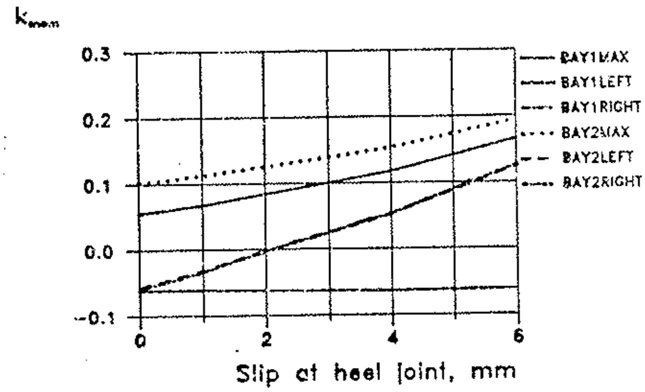
Figure 3.3 contains the results for a central ridge connection, and figure 3.4 for an eccentric ridge connection, where the eccentricity is determined as DHRIDGE $\cdot\ h_{top} = 0.167 \cdot 150$ mm. This value is expected to reflect that the contact between the two top chords occurs in the lower part of the joint due to rotations of the chord ends. In figure 3.5 is given k_{mom} for DHRIDGE = 1/4, but this is more due to completeness than to reality.

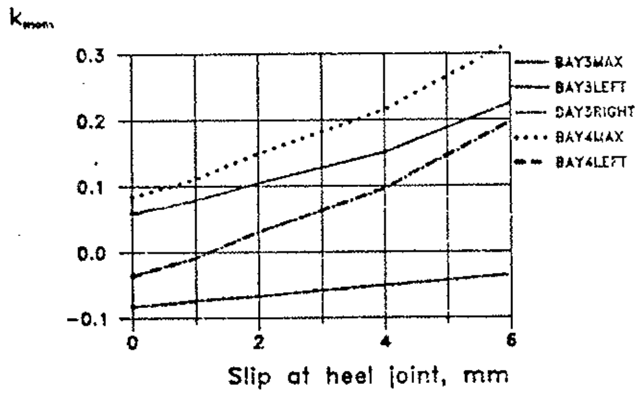
Roof slope 1:4



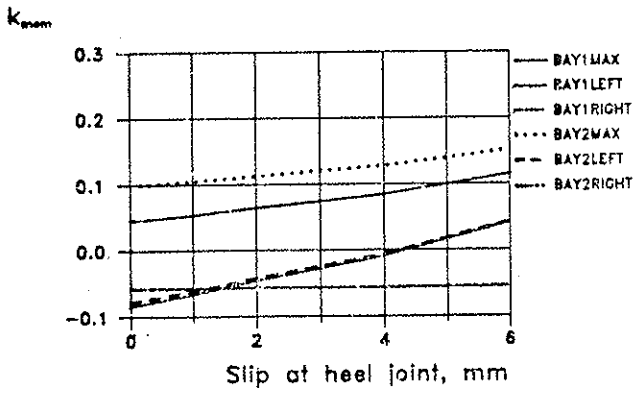


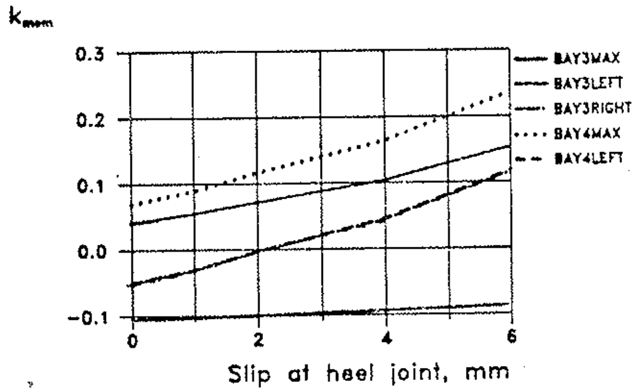
Roof slope 1:3





Roof slope 1:2





Bay members

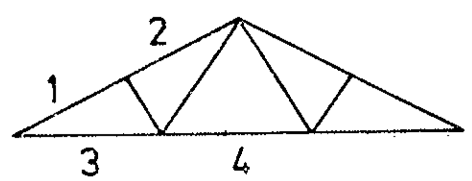
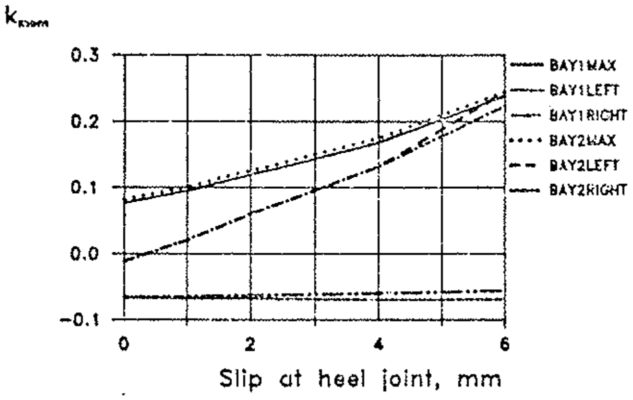
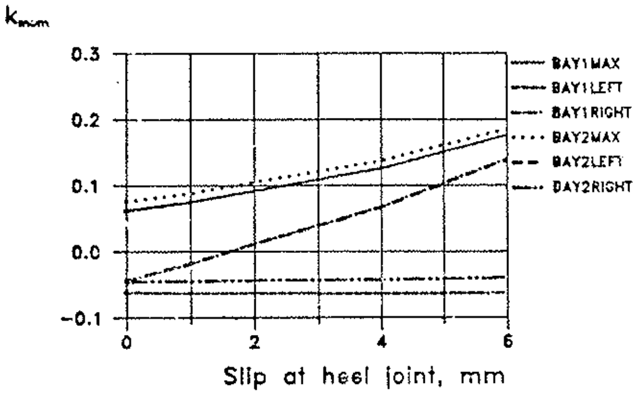


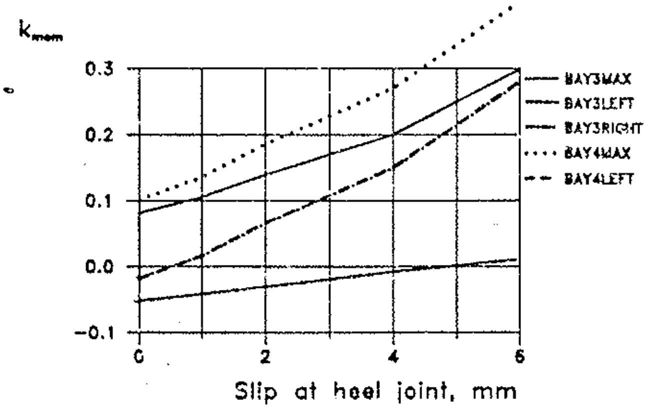
Figure 3.3 Moment coefficients for different roof slopes and slips in the connections. Central Ridge Connection, DHRIDGE = 0.

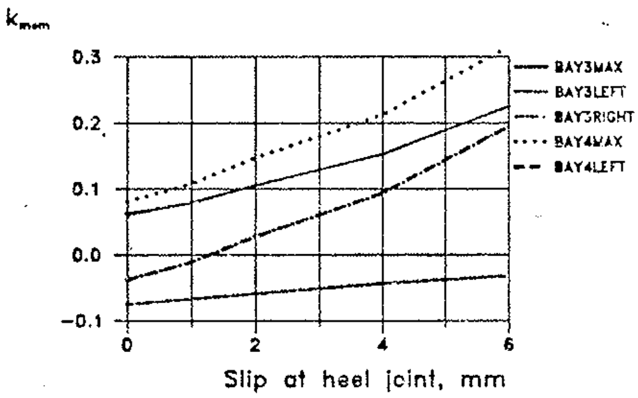
Roof slope 1:4



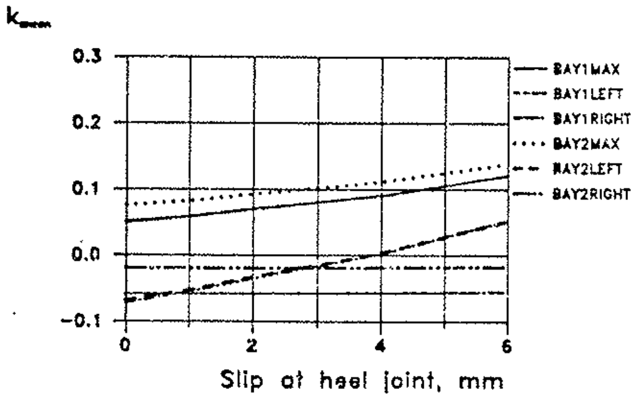
Roof slope 1:3

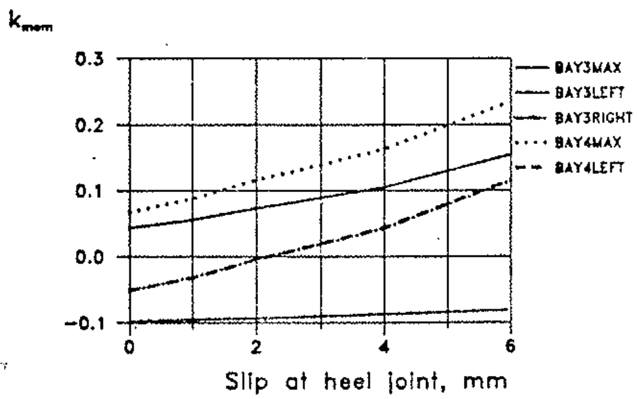






Roof slope 1:2





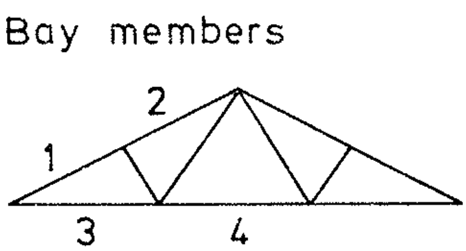


Figure 3.4 Moment coefficients for different roof slopes and slips in the connections. Eccentric Ridge Connection, DHRIDGE = 0.167 ~ 1/6.

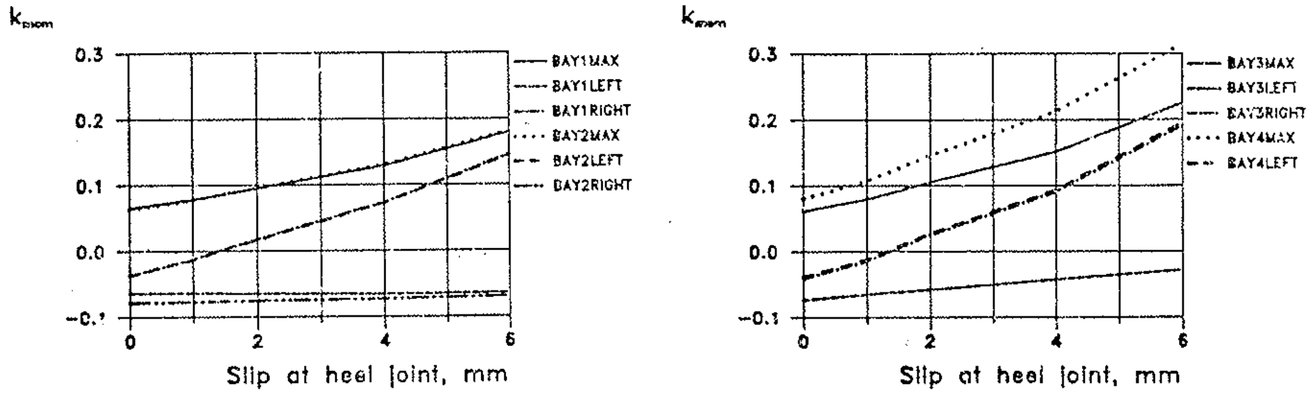


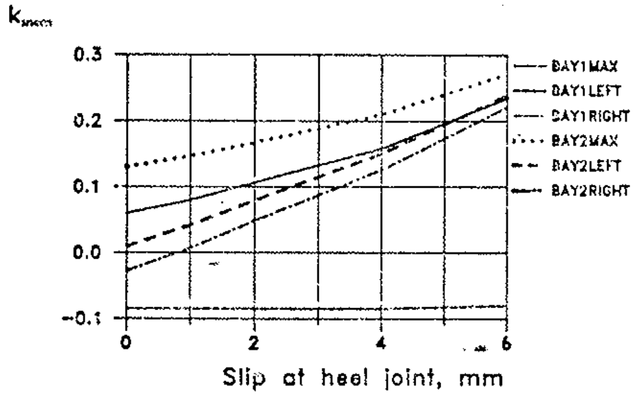
Figure 3.5 Moment coefficients for a roof slope of 1:3 and for different slips in the connections. Large eccentricity in the Ridge Connection, DHRIDGE = 0.25 = 1/4.

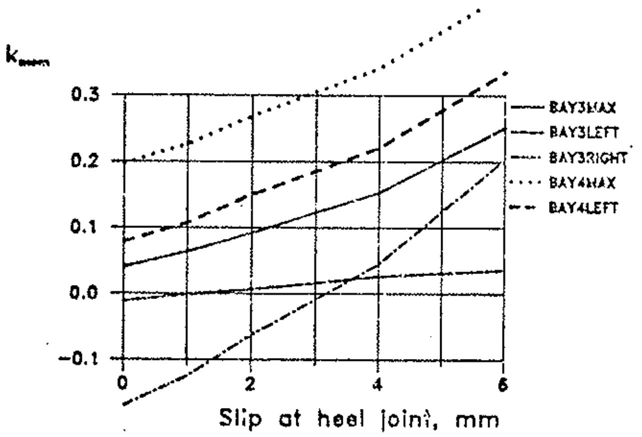
3.3.2. Roof Slope, Connection Slip, Ridge Model

The other parameters were set to the same values as in section 3.3.1 except for:

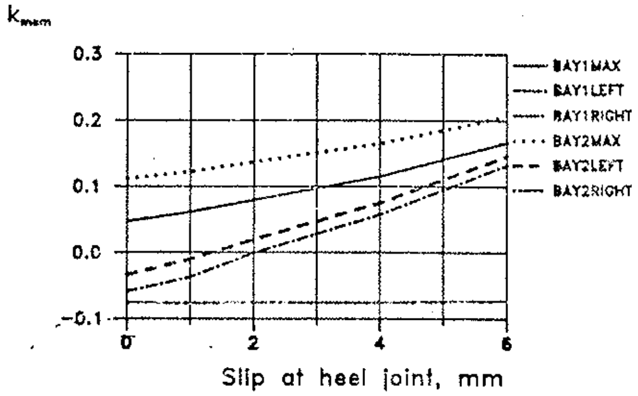
III Eccentric connections. The diagonals are connected to the chords at the inner periphery, see figure 3.1 lowermost.

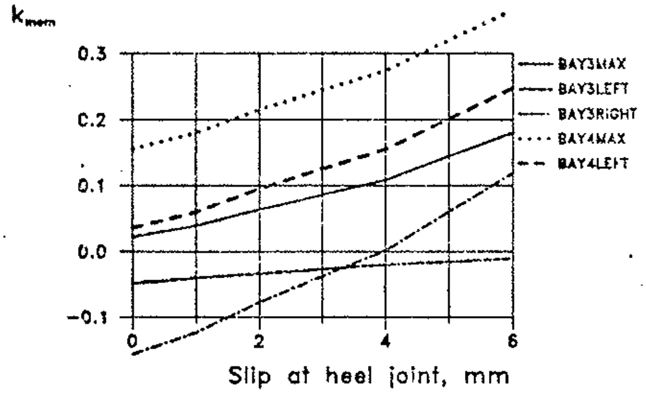
Roof slope 1:4



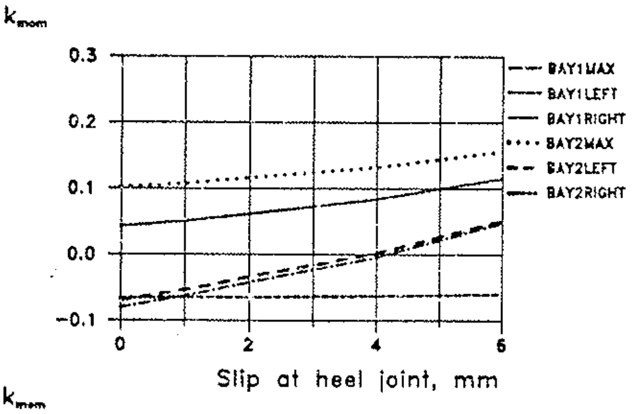


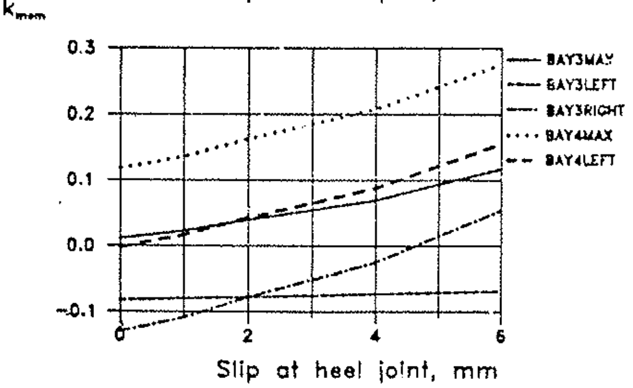
Roof slope 1:3





Roof slope 1:2





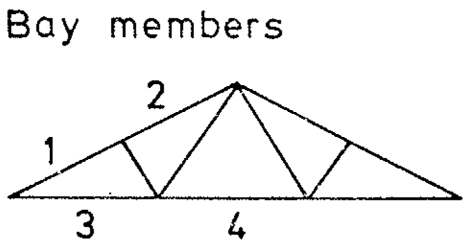
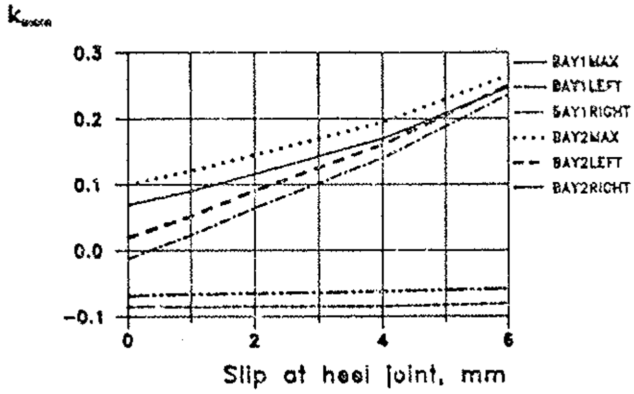
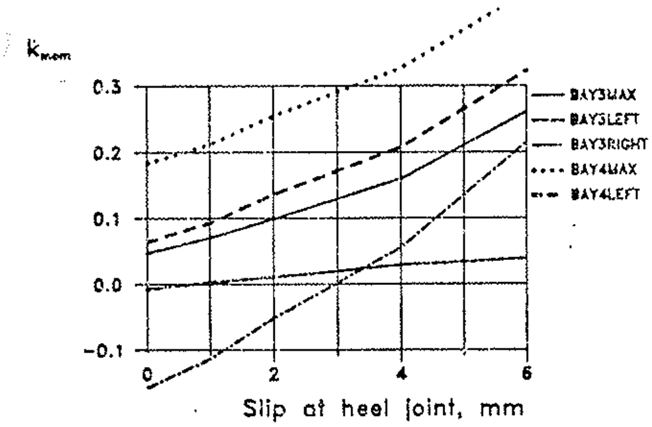


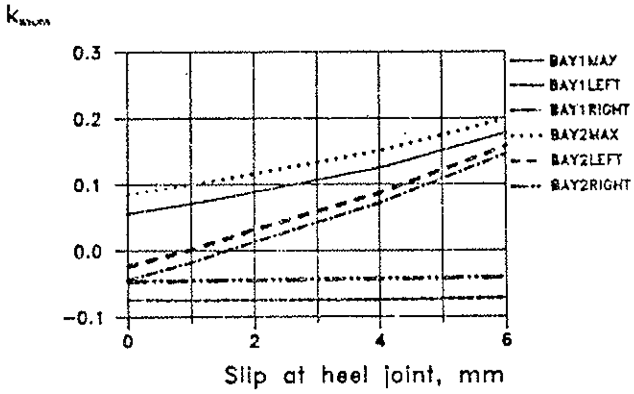
Figure 3.6 Moment coefficients for different roof slopes and slips in the connections. Central Ridge Connection, DHRIDGE = 0.

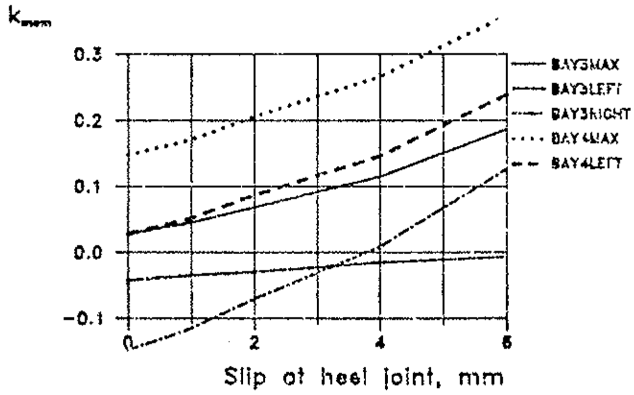
Roof slope 1:4



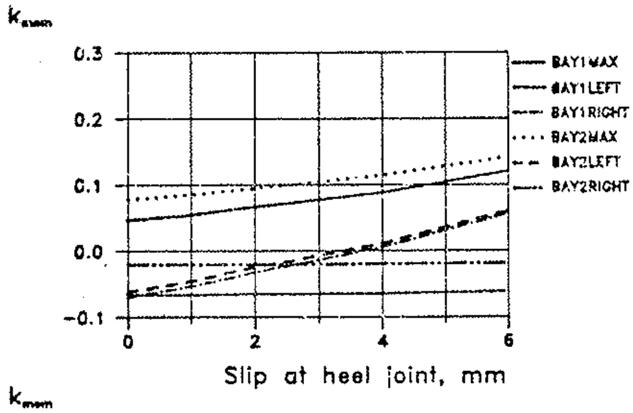


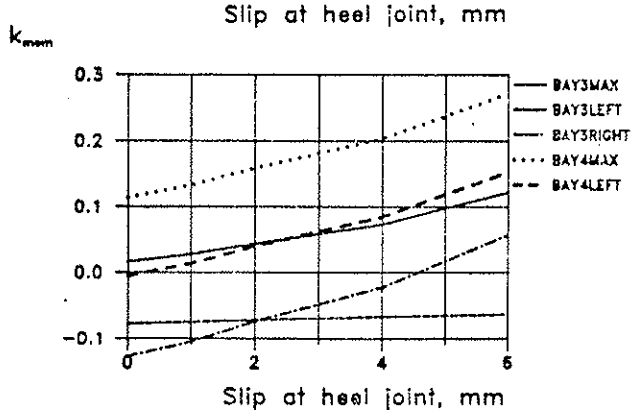
Roof slope 1:3





Roof slope 1:2





Bay members

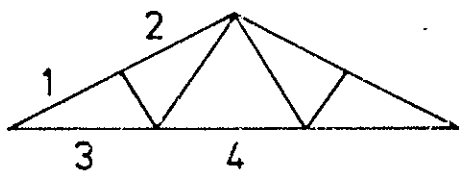
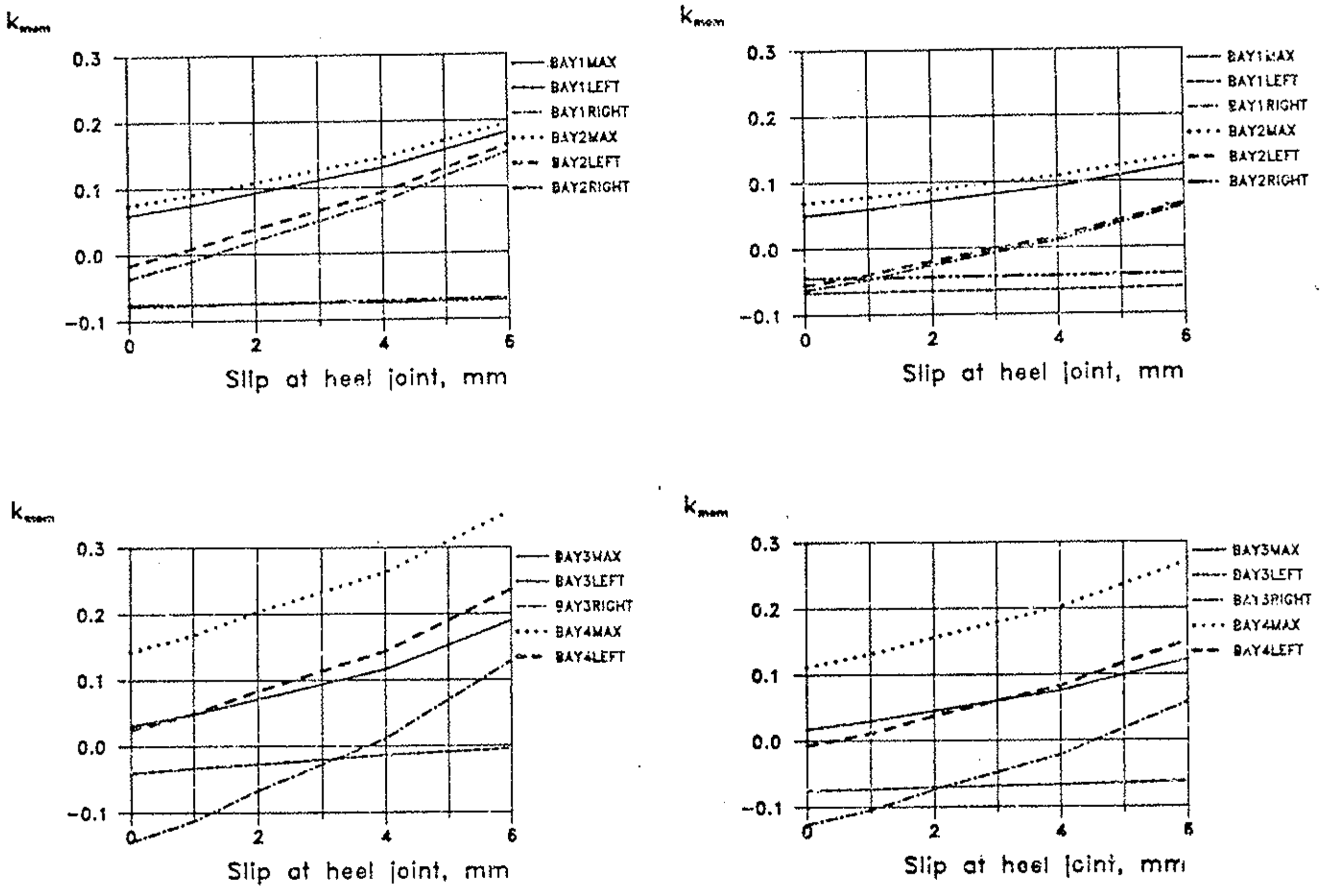


Figure 3.7 Moment coefficients for different roof slopes and slips in the connections. Eccentric Ridge Connection, DHRIDGE = 0.167 ~ 1/6.

Roof slope 1:3

Roof slope 1:2



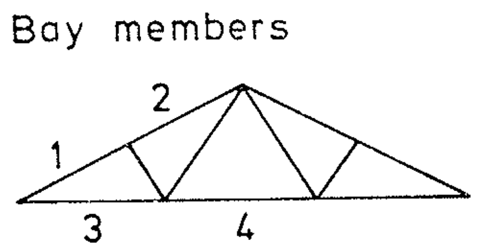


Figure 3.8 Moment coefficients for different roof slopes and slips in the connections. Large eccentricity in the Ridge Connection, DHRIDGE = 0.25 = 1/4.

By comparing the results in figure 3.4 and 3.7 it can be seen that the values for k_{mom} differ, especially at the K-joint in the bottom chord (Bay 3 right and Bay 4 left). By analysing the differences in the moments at the nodes, it was found that the moments in the truss with eccentric connections could be determined from those in the truss with central connections

$$M_{ecc.} = M_{central} + k_M \, |M_e| \qquad (3.2)$$

$$= k_{mom,central} \, q_{90} \, l_{bay}^2 + 1/2 \, k_M \, |F_{par}| \, h_{chord}$$

where

$M_{central}$ node moment in truss with central connections

M_e Eccentricity moment = $1/2 \, F_{par} h_{chord}$

F_{par} The chord parallel component of the force from the diagonals to the chord.

k_M A factor depending of truss type and node. For W-trusses with a roof slope of 1:3 the following values were found.

T-connection in the middle of the top chord, approximate values.

Bay 1: $k_M = 0.2$
Bay 2: $k_M = 1.2$

K-connection in the bottom chord, exact digits.

Bay 3: $k_M = -0.63$
Bay 4: $K_M = 0.37$

It can be added that the moment contributions at the T-connection are of less importance than those at the K-connection. If the bottom chord was idealized to a continuous beam over 3 bays and simply supported the factor k_M could be calculated to -0.6 and 0.4 instead of the figures -.63 and 0.37.

3.3.3. Support eccentricity

From figure 3.9 it is seen that the suppport eccentricity influences the momentdistribution in the chords and thereby the moment coefficients, especially the adjacent bays number 1 and 3.

For small support eccentricities $0 < D <$ approx. 200 mm it is reasonable to include the eccentricity effect in the moment coefficients. But for larger eccentricities the moment at the heel joint is dominating and it determines the dimensions. It should therefore be treated separately, for example as described in section 4.2.

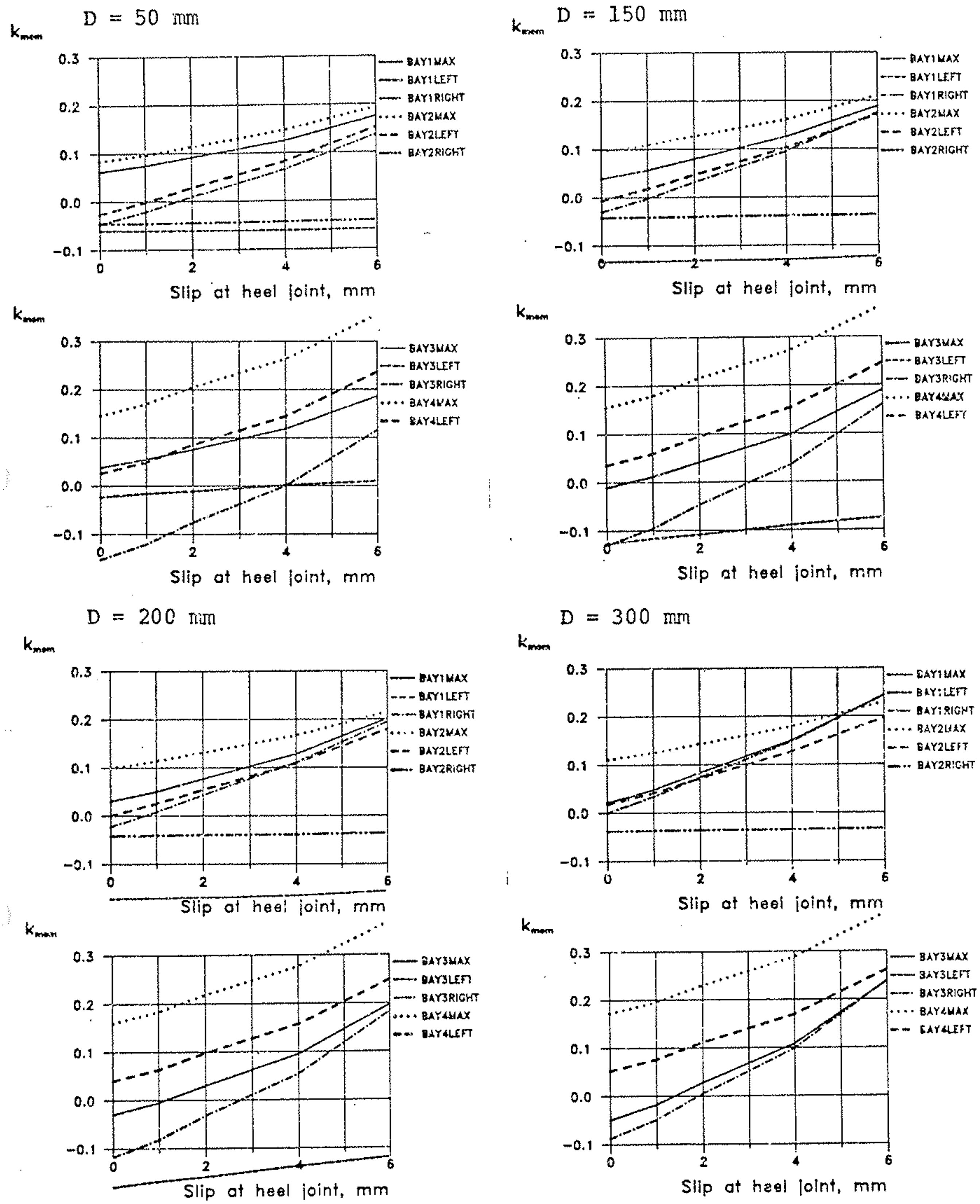


Figure 3.9 Moment coefficients for a roof slope of 1:3, span = 8.0 m and different suppport eccentricities D. Eccentric connections.

3.3.4. Dimensions and E-moduli of the chords

Only the height of the chords influences the moment distribution substantially. In figure 3.10 moment coefficients for trusses are given where the heights of both the chords are 25 smaller than those used for the figures 3.4 and 3.7 topmost to the right.

Central connections Eccentric connections

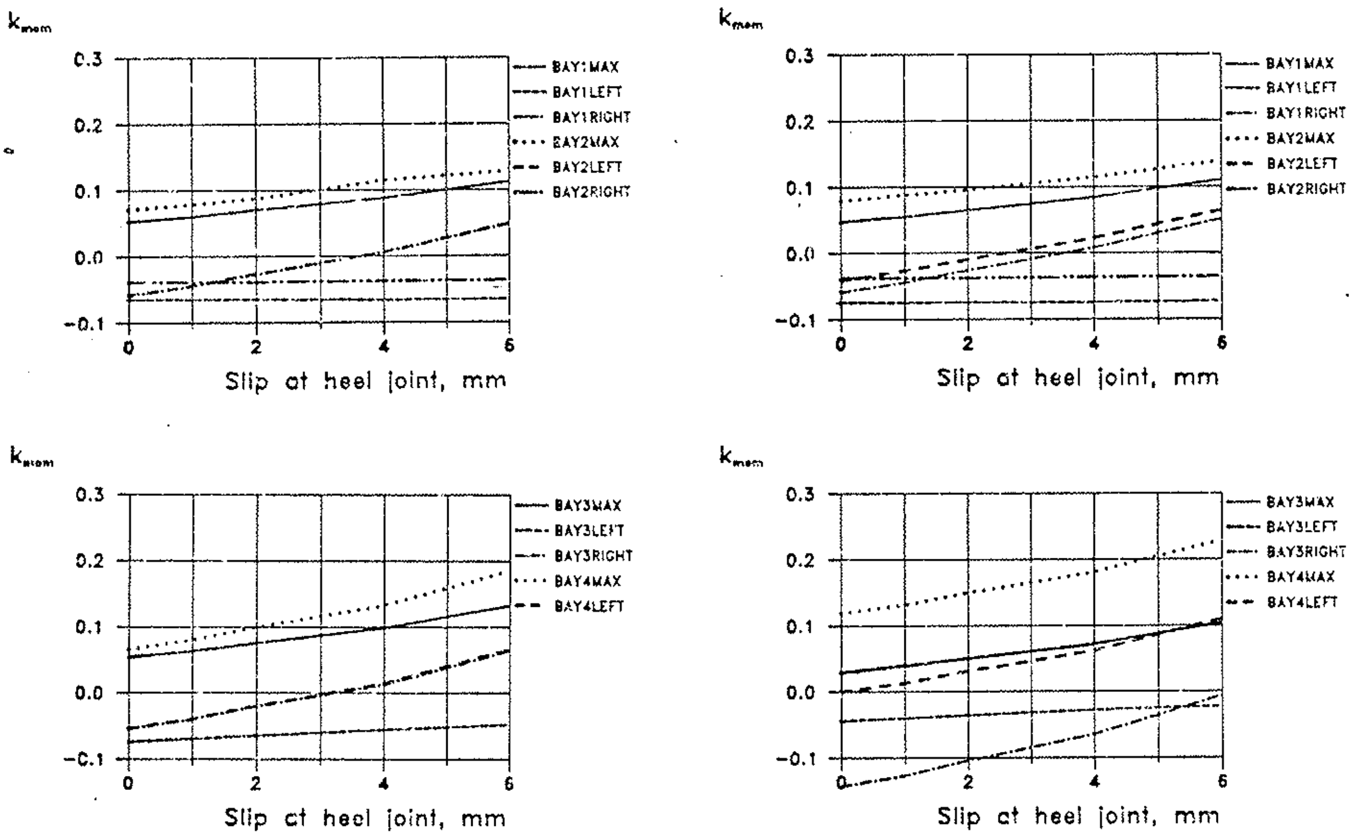


Figure 3.10 Moment coefficients for a roof slope of 1.3. Heights of the chords h_{top} = 125 mm and h_{bot} = 100 mm

Compared with figure 3.4 and 3.7 one finds, in general, smaller numerical values of k_{mom}. This is due to the fact that the eccentricity moments at the nodes are proportional to the height of the chords, and that the prescribed slips in the joints cause larger positive chord moments in the stiff (high) chords.

Calculations have shown that a change of 25% in the E-moduli only caused negligible alterations in the moment coefficients.

3.3.5. The span of the truss

In figure 3.11 k_{mom} is shown for trusses similar to those used for the figures 3.4 and 3.7 topmost to the right. The only differences are that here Span = S = 1200 mm and chord height = $h_{top} = h_{bot}$ = 175 mm is used. The chord dimensions are selected from [Wood Trussed Rafters, 1983].

Central connections

Eccentric connections

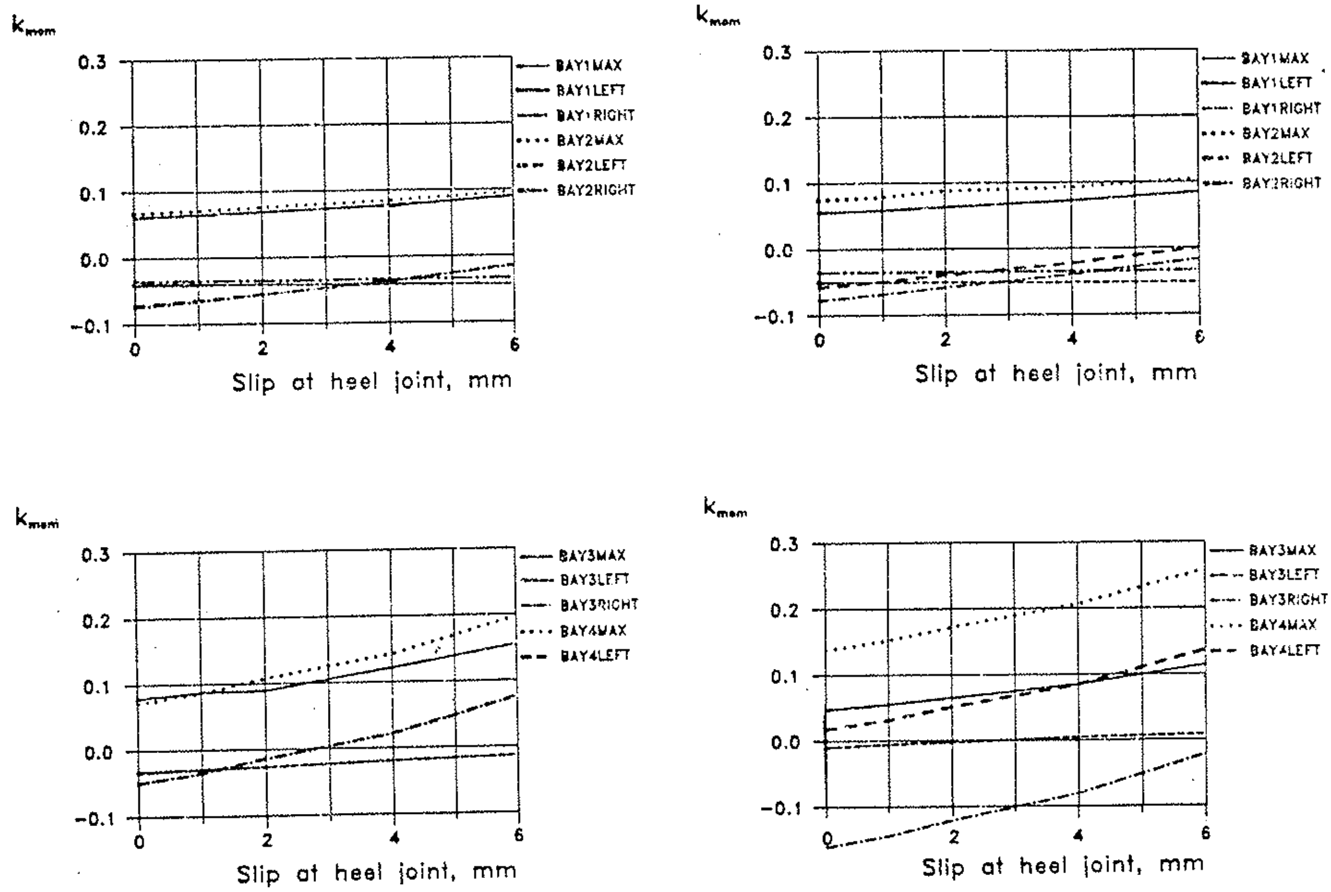


Figure 3.11 Moment coefficients for a roof slope of 1:3.

Compared with figure 3.4 and 3.7 one finds that for small slips in the joints k_{mom} values are of approximate the same size. But it is seen that for the larger span k_{mom} is not so sensitive to the slip.

3.3.6. Heavy or light roof, symmetric or asymmetric load on the top chord

For a reference truss moment coefficients were calculated both for central and eccentric connections.

For both trusses it was found that when the load on the top chord was changed from 1.61 kN/m to 1.31 kN/m (heavy to light roof) the values of the moment coefficients were altered less than 10 percent, except for some calculated for larger slips.

It was further found that the moment coefficients did not depend on whether the load on the top chord was symmetric or asymmetric (top chord loads: 1.61 and 1.12 kN/m). The differences were negligible.

3.3.7. Partial Attic Load

A reference truss was analysed with an extra distributed line load over 1.2 m of the middle bay of the bottom chord, see figure 3.2 with B1 = B2 = 0.6 m. The extra load intensity was 1.05 kN/m.

It has been investigated whether the moments caused by the partial attic load $q_{bot,a}$ corresponds to what would have been found in a continuous beam over 3 spans. This means that the resulting moments could be found from

$$M_{res} = k_{mom}\, q_{bot}\, l^2 + M_{qbota,cont.beam} \qquad (3.3)$$

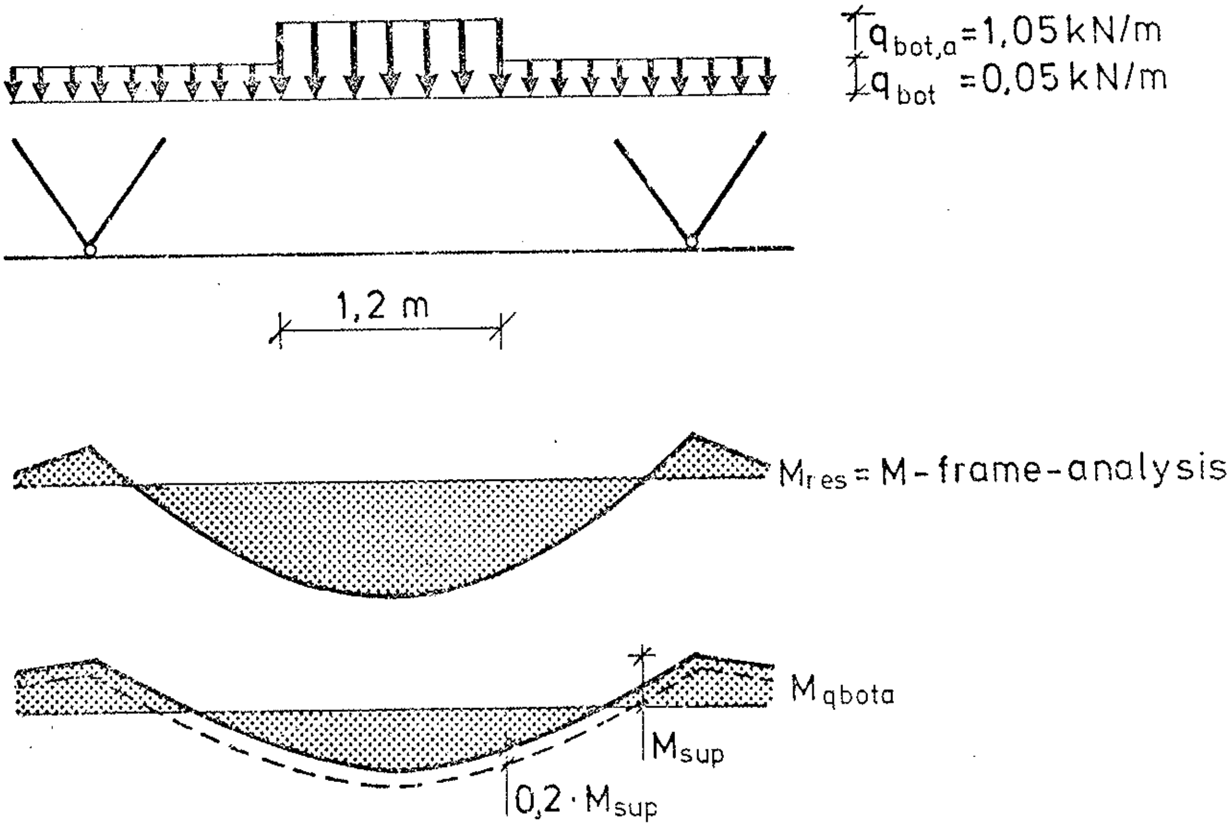


Figure 3.12 Distributed loads on the bottom chord. Resulting moment distribution and moment distribution caused by $q_{bot,a}$ on a similar continuous beam over 3 spans.

Meanwhile it was found that if (3.3) was solved for k_{mom} it gave values which differed considerably from those given in the figures 3.4 and 3.7.

Instead of this a "Swedish rule" was used. It states that the moment distribution due to the partial attic load can be found as the moment distribution in a continuous beam over 3 spans, but with the moments at the supports reduced by a factor of 0.8. This moment distribution is shown dotted in figure 3.12. The moment coefficients found in this way by solving (3.3) is given in figure 3.13. In figure 3.14 they are shown for a span of 12.0 m and all other parameters as the reference truss.

Central connections | Eccentric connections

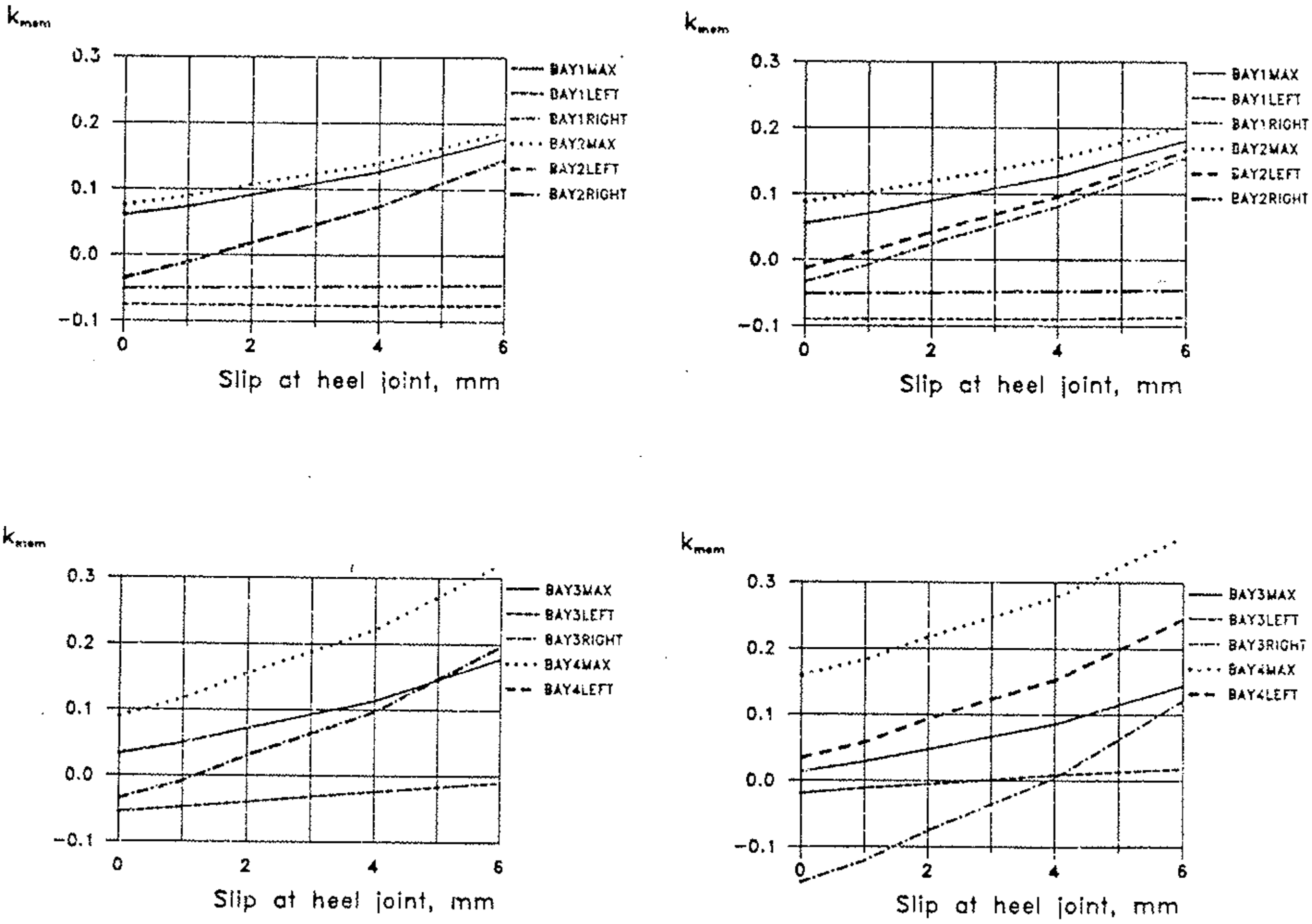


Figure 3.13 Moment coefficients for a reference truss with a roof slope of 1:3 and a span of 8.0 m, and loaded with a partial attic load.

Central connections Eccentric connections

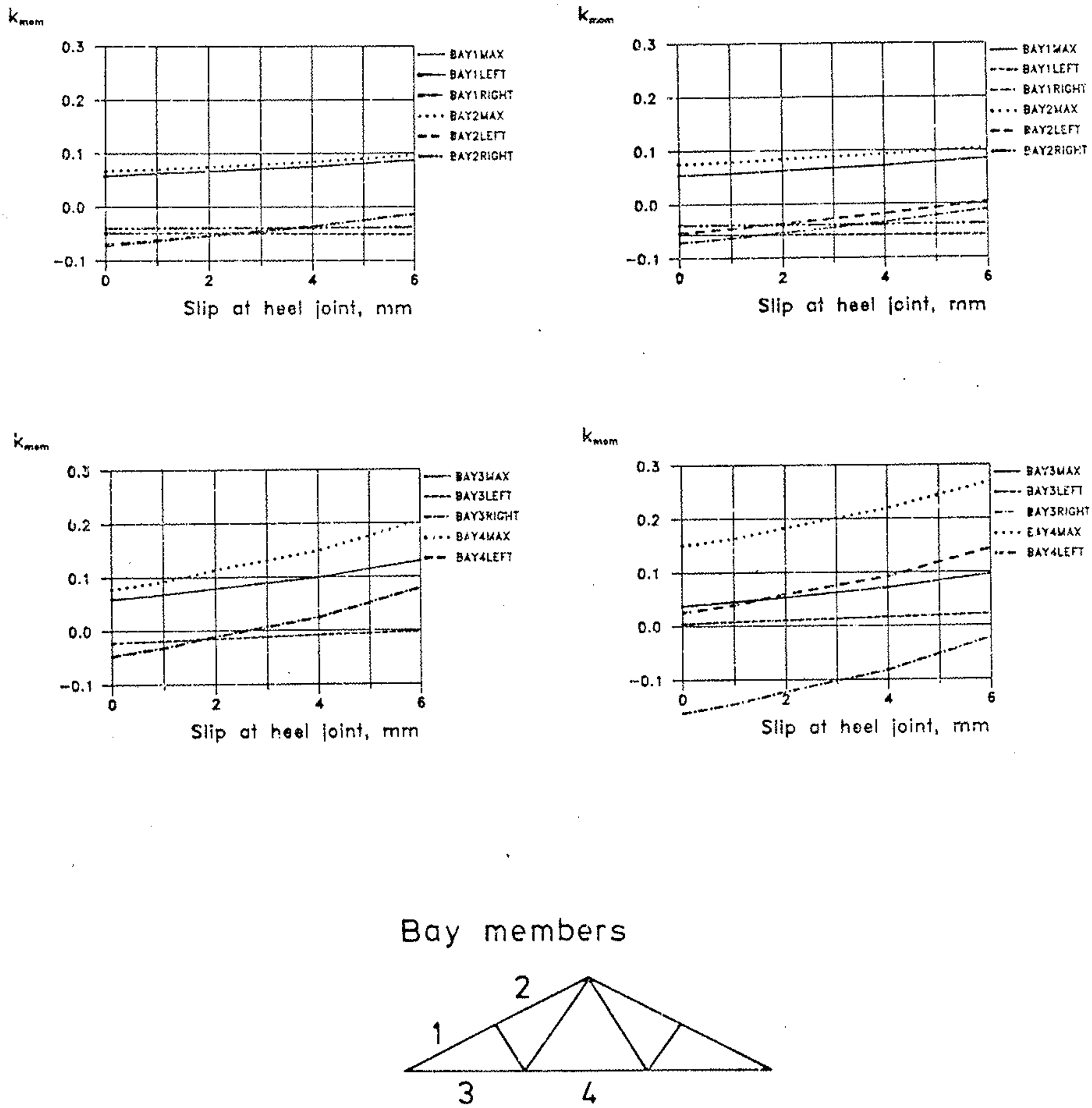


Figure 3.14 Moment coefficients for a roof slope of 1:3 and a span of 12.0 m, and loaded with a partial attic load.

4. A SIMPLE STRESS ANALYSIS FOR A W-TRUSS

The normal forces are calculated as described in section 2. The moments are calculated as described in either section 4.1 or 4.2.

4.1. W-truss with a little Support Eccentricity

The following moment coefficients can be used for w-trusses provided that

The support eccentricity D is less than:

200 mm
2 · bottom chord height

The slope of the roof is larger than 1:4

The moment coefficients depend on the slip in the joints, and values are given for the following three cases. See also table 3.1.

A Nail plates with long teeths

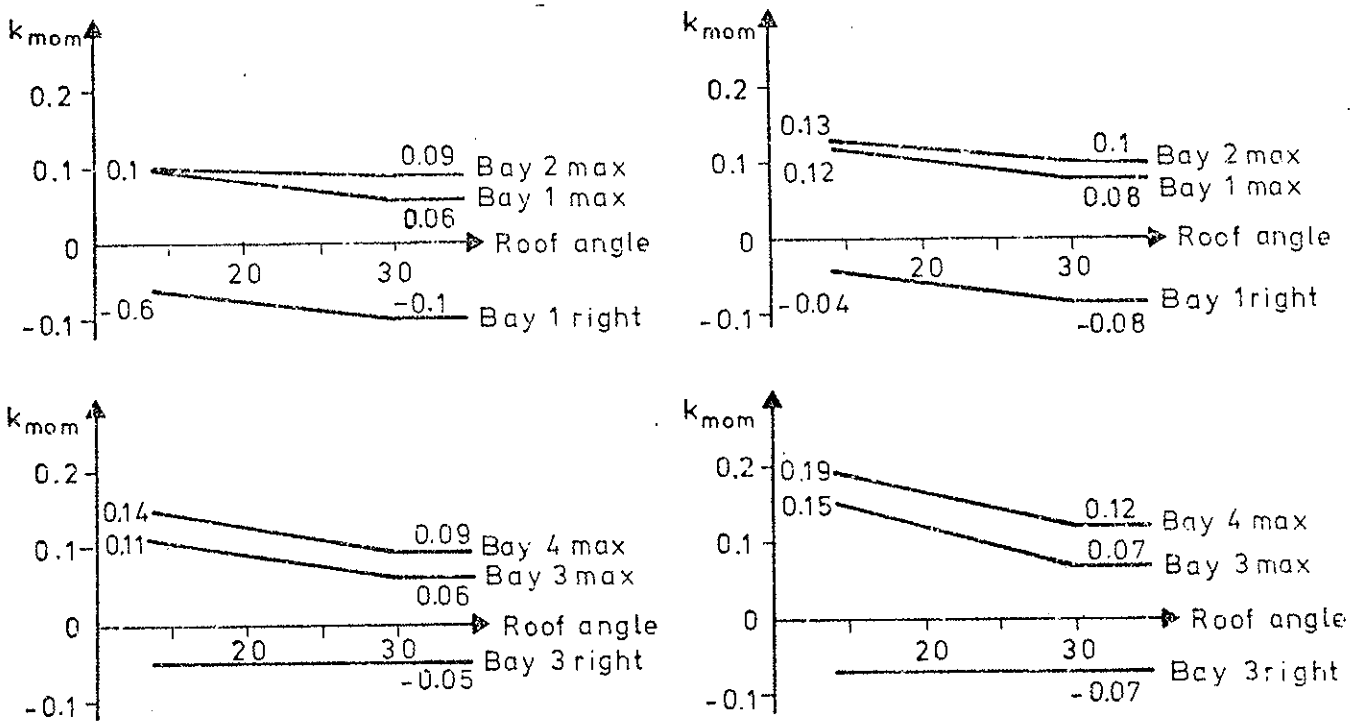
B Nails driven directly through thin steel plate gussets

C Nailed plywood or wood gussets.
Connectors with teeth pressed into the wood.

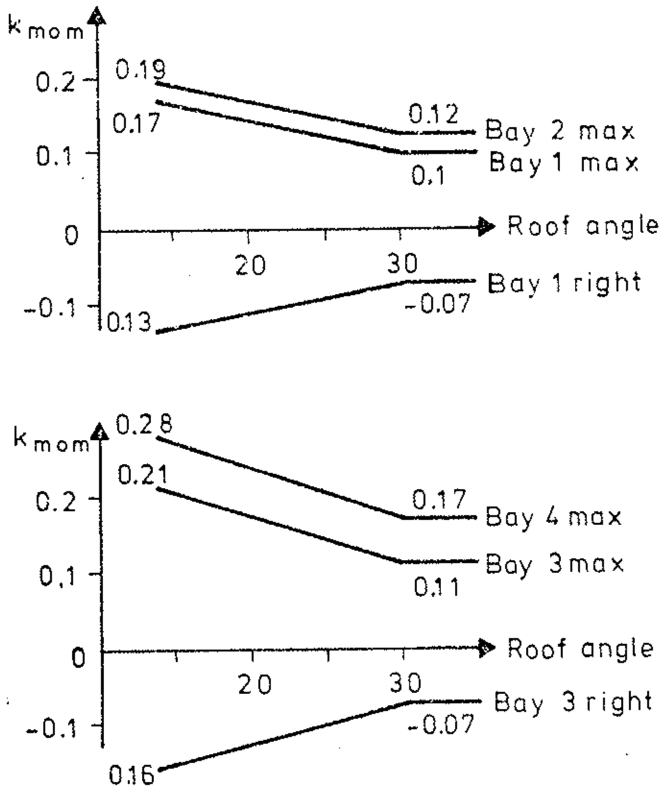
Provided the moments are calculated from equation (2.1) or the first term of (4.1) the values of k_{mom} given in figure 4.1 and 4.2 can be used.

Case A

Case B



Case C



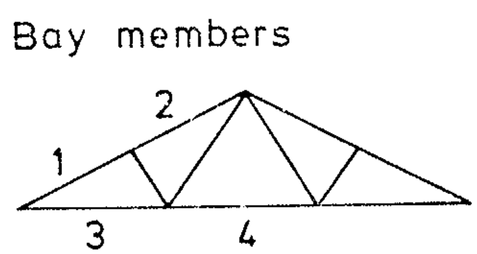
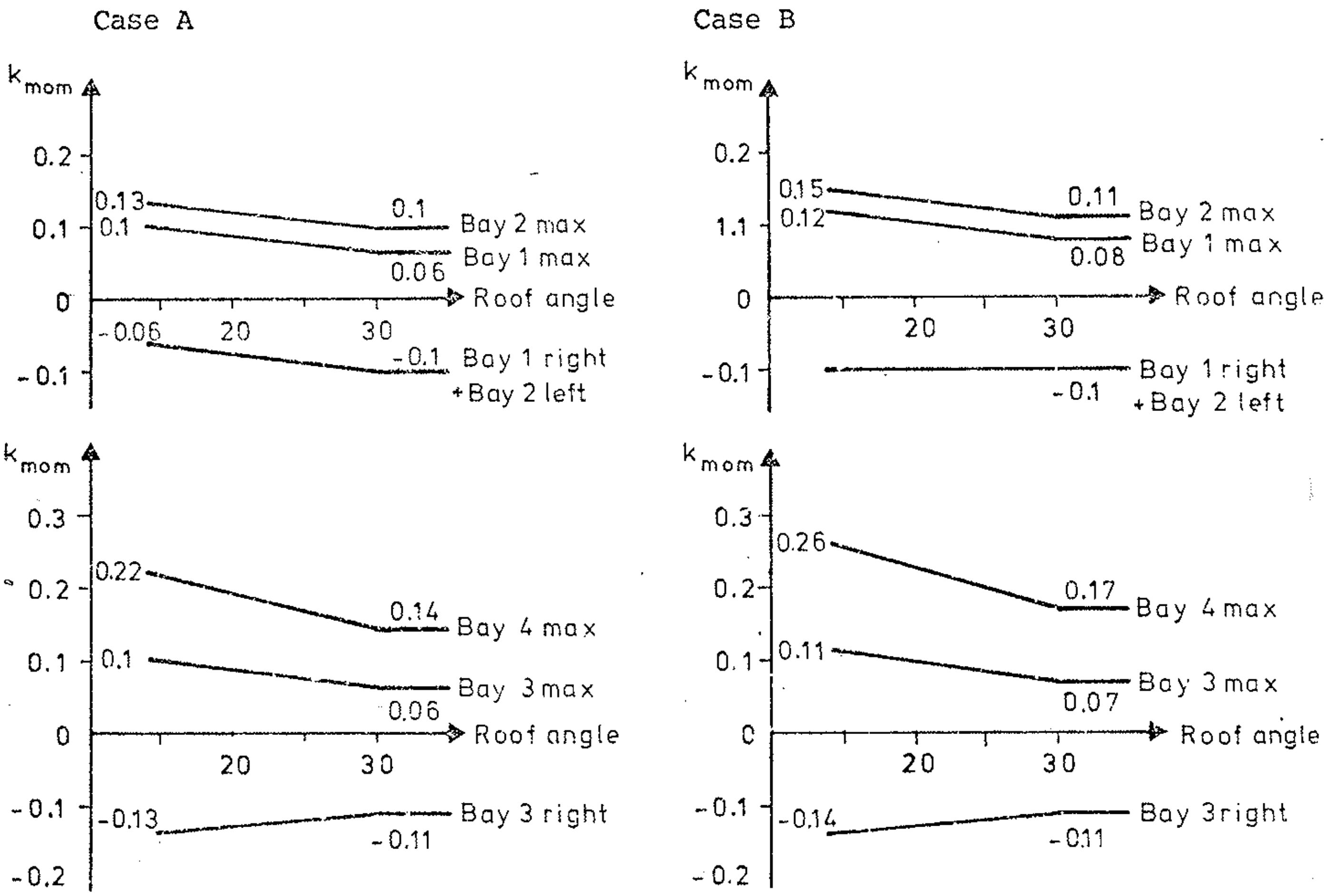
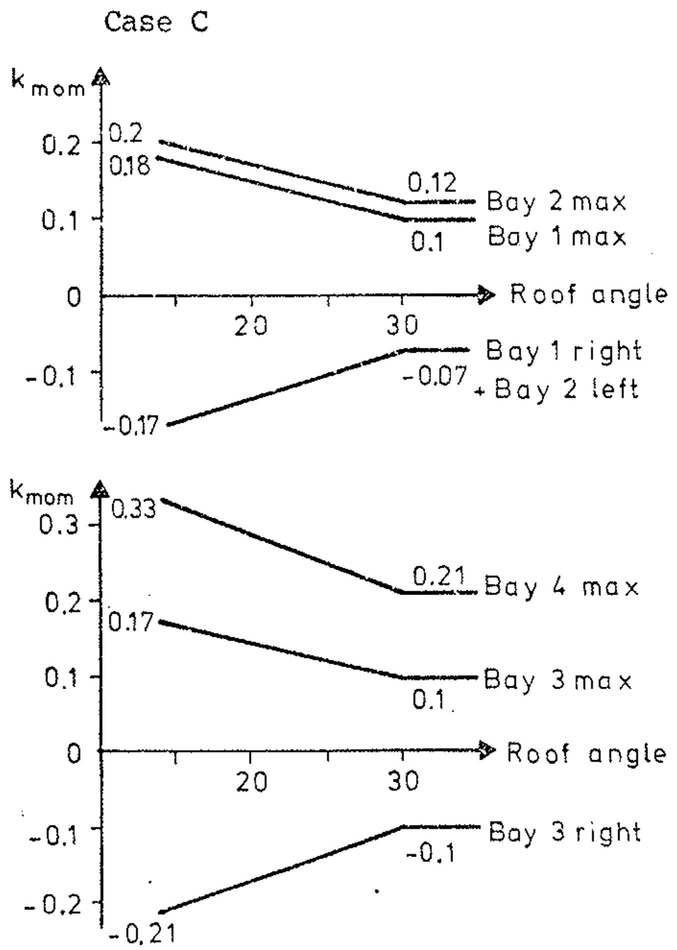


Figure 4.1 $k_{mom,central}$ for central connections between chords and diagonals.





Bay members

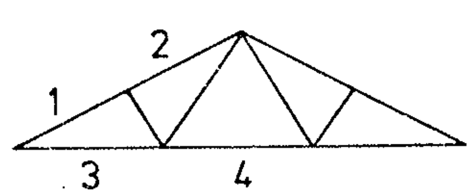


Figure 4.2 $k_{mom,ecc}$ for eccentric connections where the diagonals are connected to the inner side of the chords, see figure 3.1 bottom.

If the eccentricities in the chord-diagonal connections are smaller than those assumed for figure 4.2 then k_{mom} can be found by interpolation between the values in figure 4.1 and 4.2.

If the eccentricity moments from the chord-diagonal connections are taken into account seperately, it can be done at the nodes by

$$M = k_{mom,central}\ q_{90}\ l_{bay}^2 + k_M \cdot 1/2\ |F_{par}|\ h_{chord} \qquad (4.1)$$

where

$k_{mom,central}$ Moment coefficients at the nodes, see figure 4.1.

q_{90} Distributed load perpendicular to the chord.

l_{bay} Length of the bay

k_M A factor which reflects the partition of the eccentricity moment. The following values can be used.

For the T-connection in the middle of the top chord.

Bay 1: $k_M = 0.2$
Bay 2: $k_M = 1.2$

For the K-connection in the bottom chord.

Bay 3: $k_M = -0.63$
Bay 4: $k_M = 0.37$

F_{par} The chord parallel component of the force the diagonals to the chord.

h_{chord} Height of the cross section of the chord.

The effect of the eccentricity moments in the joints must also be taken into account in the bays. This can approximately be done by

$$M = k_{mom,central}\, q_{90}\, l_{bay}^2 + \frac{1}{4} h_{chord}[(k_M F_{par})_{left} + (k_M F_{par})_{right}] \quad (4.2)$$

where $k_{mom,central}$ is assigned a value corresponding to M_{max} in the bay and the values in the brackets are evaluated at each node.

If the chords are subjected to distributed load of limited extension or concentrated forces, then the moment contribution can be determined in the following way. First one find the moments at the supports of a continuous beam which corresponds to the continuous chord that is with the same number and length of bays. Then the moments at the supports are reduced by a factor 0.8 and with these the moments in the bays are calculated. See figure 3.12.

4.2. W-truss with a large Support Eccentricity

It is assumed that the support eccentricity is so large that it is decisive for the dimensions of the chords. The critical cross sections are thus just above the support. At this point the normal forces and moments can be found from figure 4.3 and the following formulas

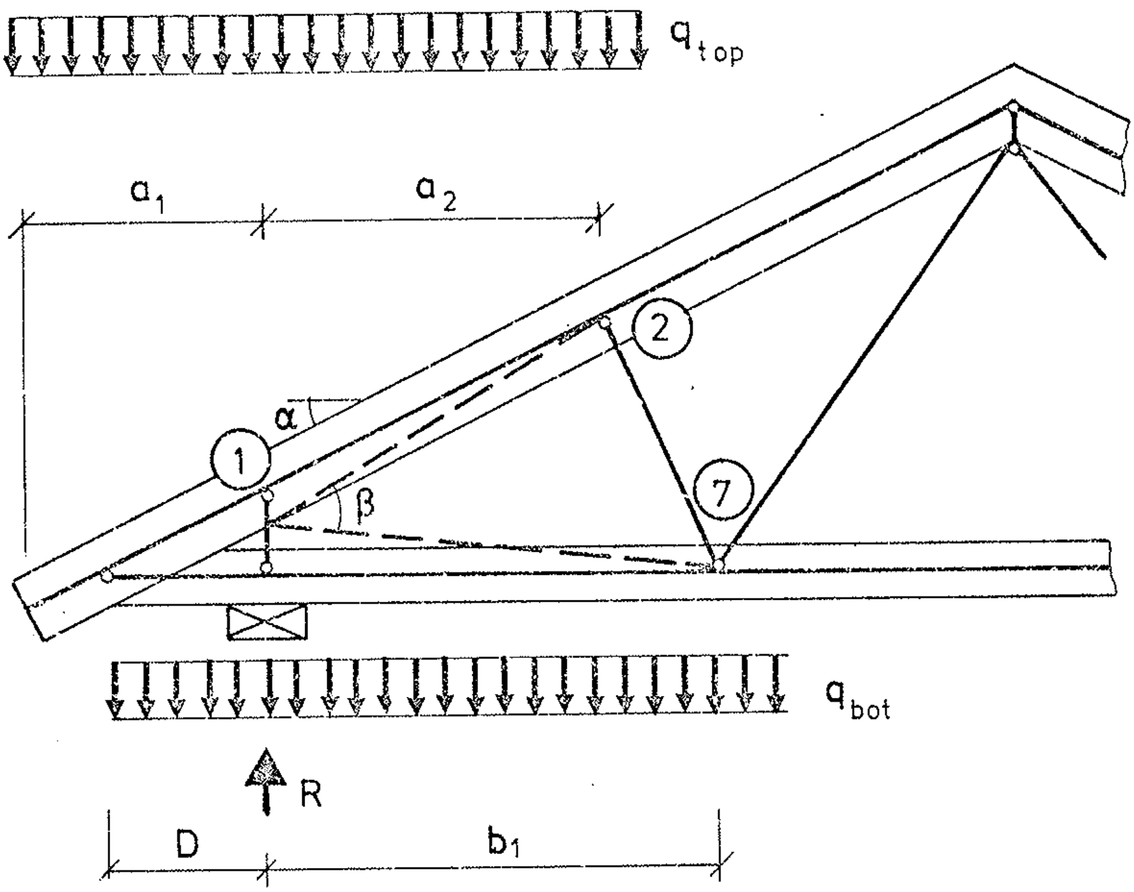


Figure 4.3 Cantilevered truss. Geometry and design modification of the truss.

Provided that there is a wedge in the space between the top and bottom chord so that the whole support area is covered, then the following calculation method can be used.

The total reaction is R and the nodal forces F_i are

$$F_1 = q_{top}\ 1/2\ (a_1 + a_2)^2/a_2 + q_{bot}\ 1/2(D + b_1)^2/b_1 \qquad (4.3)$$

$$F_2 = q_{top}\ 1/2\ (a_1 + a_2)(a_2 - a_1)/a_2 + \text{Force from top bay} \qquad (4.4)$$

$$F_7 = q_{bot}\ 1/2\ (D + b_1)(b_1 - D)/b_1 + \text{Force from middle bay} \qquad (4.5)$$

The normal forces are approximately

$$N_{bot} = (R - F_1)\cot\beta \qquad (4.6)$$

$$N_{top} = (R - F_1)/\sin\beta \qquad (4.7)$$

The sum of moments M_{tot} in the two cross sections is

$$M_{tot} = -N_{bot} \cdot D\ \mathrm{tg}\,\alpha - 1/2\ q_{top}\ a_1^2 - 1/2\ q_{bot}\ D^2 \qquad (4.8)$$

As suggested in [Feldborg & Johansen, 1981] M_{tot} is partitioned between the two cross sections proportional to their section moduli. The argument is that this reflects the non-linear behavior of the members and the heel joints at the ultimate state.

$$M_{top} = M_{tot}\ \frac{b_{top} h_{top}^2}{b_{top} h_{top}^2 + b_{bot} h_{bot}^2} \qquad (4.9)$$

$$M_{bot} = M_{tot}\ \frac{b_{bot} h_{bot}^2}{b_{top} h_{top}^2 + b_{bot} h_{bot}^2} \qquad (4.10)$$

In [Riberholt, 1980] it is explained how it can be taken into account that there is a smaller likelyhood for the fact that a big growth defect (knot) occurs at a moment peak than at the moment maximum in a bay. This effect is especially pronounced for the large moment peak at the support.

If the strength control of the cross sections just over the support is carried out as

$$\text{Bottom chord} \qquad \frac{\sigma_t}{f_{t,o}} + \frac{\sigma_m}{k_{fm} f_m} \leq 1$$

$$\text{Top chord} \qquad \frac{\sigma_t}{f_{c,o}} + \frac{\sigma_m}{k_{fm} f_m} \leq 1 \qquad (4.11)$$

then the bending strength increasing factor k_{fm} can be assigned the following values, and still the probability of failure will be the prescribed 5%.

$k_{fm} = 1.4$ for u/k - SC 19

$k_{fm} = 1.3$ for T24 - SC 24

It must be emphasized that the evaluation is done for Nordic spruce and pine of the qualities mentioned. But since it has been done relatively conservative and if the density and distribution of growth defects do not differ too much, then the values should also be applicable to the CIB strength classes.

ANNEX A. Description of how prescribed deformations at the end of a beam element can be incorporated into a frame program.

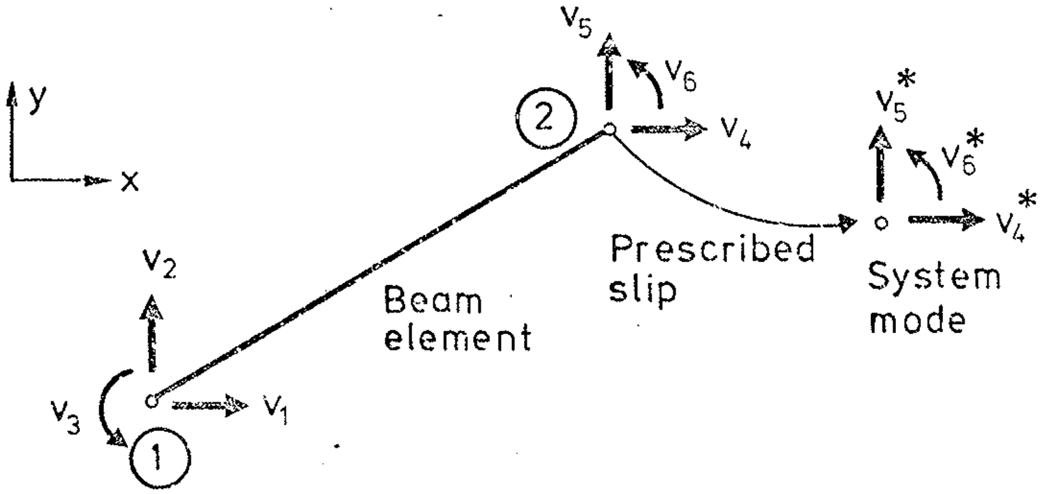


Figure A1. Degress of freedom for a plane beam element. The prescribed slip is calculated positive from the beam node to the system node.

Equilibrium of the element gives

$$\{q\} = [k]\{v\} + \{q_o\} \tag{A.1}$$

where $\{q\}$ Nodal forces

$[k]$ Stiffness matrix of the beam element

$\{v\}$ Deformations of the beam nodes

$\{q_o\}$ Nodal forces for $\{v\} = \{0\}$

The slip $\{g\}$ between the beam node and the system node is defined as

$$\{g\} = \{v^*\} - \{v\} \tag{A.2}$$

For beam node 2 it can be written as

$$\begin{Bmatrix} v_1^* \\ v_2^* \\ v_3^* \\ v_4^* \\ v_5^* \\ v_6^* \end{Bmatrix} - \begin{Bmatrix} v_1 \\ v_2 \\ v_3 \\ v_4 \\ v_5 \\ v_6 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ g_4 \\ g_5 \\ g_6 \end{Bmatrix} \tag{A.3}$$

Solving (A.2) for $\{v\}$ and substituting it into (A.1) one find

$$\begin{aligned} \{q\} &= [k](\{v^*\} - \{g\}) + \{q_o\} \\ &= [k]\{v^*\} + (\{q_o\} - [k]\{g\}) \\ &= [k]\{v^*\} + \{q_o^*\} \end{aligned} \tag{A.4}$$

where $\{q_o^*\}$ can be interpreted as the nodal forces for $\{v^*\} = \{0\}$. The equation (A.4) mentioned above corresponds to the normal equilibrium equation (A.1), the only modification is that the right hand side is changed to

$$\{r\} = -\{q_o\} + [k]\{g\} \tag{A.5}$$

The forces $\{s\}$ and moments can be calculated from

$$\{s\} = [S]\{v\} + \{s_o\} \tag{A.6}$$

$$= [S](\{v^*\} - \{g\}) + \{s_o\}$$

where $[S]$ Stress matrix
$\{s_o\}$ Forces and moments for $\{v\} = \{0\}$

References

Feldborg, Th. & Johansen, M., 1981. Wood Trussed Rafter Design. SBI-Rapport 118.

Riberholt, H., 1980. Strength Distribution of Timber Structures. IUFRO Wood Engineering Group Meeting, Oxford UK 1980.

Riberholt, H., 1982. Guidelines for Static Models of Trussed Rafters. CIB W.18/15-14-1. Karlsruhe, June 1982.

Wood Trussed Rafters, 1983. Træspærfag, TRÆ 28. Træbranchens Oplysningsråd (Wood promotion Council).

CIB-W18/17-14-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIMPLIFIED CALCULATION METHOD FOR W-TRUSSES

by

B Källsner

Swedish Forest Products Research Laboratory
Sweden

RAPPERSWIL

SWITZERLAND

MAY 1984

Introduction

This paper presents a simplified calculation method for W-trusses. The method is applicable to heel joint constructions with or without wedges. The proposed construction of the heel joint with a wedge has some advantages over other heel joint constructions since the magnitude of the bending moments in the heel joint and their distribution between upper and lower chords may be influenced by altering the length of the wedge.

The greatest difficulty in the calculation of W-trusses is to take into consideration the influence on the moment distribution of the displacements of the joints. Consequently the report concentrates on the determination of the bending moments while the calculation of the axial forces has been left out. A more complete report will be published later in which both W-trusses and WW-trusses will be treated.

Assumptions

The method is based on the static model presented in figure 1. The upper and lower chords are assumed to be continuous beams which are hinged to each other. The fictitious bars in the heel joints and the diagonals are assumed to be hinged to the chords. As far as the chords are concerned, the system lines are placed in the centre of gravity of the timber. In the diagonals, the system lines have been placed so that no eccentricities occur in their connections to the upper and lower chords. In the same way it is assumed that the imaginary hinges between the fictitious bars and the chords are placed on the system lines of the chords.

The angle between the fictitious bar in the heel joint and the upper chord shall not be less than 14°, and the point of intersection of the fictitious bar with the system line of the upper chord shall be 50 mm from the end of the wedge along the upper chord, see figure 2. Finally, the fictitious bar shall be placed at least 50 mm from the point of intersection of the underside of the wedge and the upper side of the lower chord.

The roof truss is assumed to be symmetrical with regard to the positions of the chords and the diagonals. The eccentricity of the supports in the left and right heels need not be the same. The notations introduced are presented in figure 3.

The external distributed loads acting on the roof truss are divided into three parts, according to figure 4, where q_{u1} is a symmetrical upper chord load, q_{u2} is an asymmetrical upper chord load and q_{ℓ} is a symmetrical lower chord load.

To calculate the displacements of the joints in figure 1 and bending moments hereby introduced, the unit-load method has been used.

Calculation procedure

1. Calculate the support reactions R_1 and R_2 (figure 3) taking into consideration all external loads. The roof truss is subsequently regarded as being supported at the points of intersection between the system lines of the upper and lower chords, so that R_1 and R_2 are also treated as external loads.

2. Remove the fictitious bars in the left and the right heel joints and consider the chords to be supported on rigid supports in the joints 1 - 7. Apply the external distributed loads on the roof truss and calculate the moment distribution.

3. Consider the influence of the support displacements caused by the external distributed loads by adding to the support moments the following moments, (the joints are numbered as in figure 1):

$$\Delta M_2 = (0.548\ q_{u1} + 0.208\ q_{u2} + 0.519\ q_{\ell})\ \frac{b^2{}_u}{\sin^2\alpha}$$

$$\Delta M_4 = (0.548\ q_{u1} - 0.208\ q_{u2} + 0.519\ q_{\ell})\ \frac{b^2{}_u}{\sin^2\alpha}$$

$$\Delta M_6 = (0.276\ q_{u1} - 0.188\ q_{u2} + 0.290\ q_{\ell})\ \frac{b^2{}_{\ell}}{\sin^2\alpha}$$

$$\Delta M_7 = (0.276\ q_{u1} + 0.188\ q_{u2} + 0.290\ q_{\ell})\ \frac{b^2{}_{\ell}}{\sin^2\alpha}$$

where b_u = the width of the upper chord (figure 2)

b_{ℓ} = the width of the lower chord (figure 2)

α = the slope of the roof

4. In item 2, the influence of the fictitious bars in the left and the right heel joints was disregarded. In fact, the external distributed loads give rise to axial forces in the fictitious bars. To calculate the verti cal force component X_i (see figure 5), the following expression can be used:

$$X_i = \frac{\frac{q_u L}{E_u I_u \cos\alpha}\,\psi_u{}^3 \left[1 - 3\left(\frac{\xi_i}{\psi_u}\right)^2 + 2\left(\frac{\xi_i}{\psi_u}\right)^3\right] - \frac{q_{\ell} L}{E_{\ell} I_{\ell}}\,\psi_{\ell}{}^3 \left[1 - 3\left(\frac{\eta_i}{\psi_{\ell}}\right)^2 + 2\left(\frac{\eta_i}{\psi_{\ell}}\right)^3\right]}{16\eta_i \left[\frac{\psi_u}{E_u I_u \cos\alpha}\left(1 - \frac{\xi_i}{\psi_u}\right)^2 \left(1 - \frac{1}{8}\left(1 \pm \frac{\xi_i}{\psi_u}\right)^2\right) + \frac{\psi_{\ell}}{E_{\ell} I_{\ell}}\left(1 - \frac{\eta_i}{\psi_{\ell}}\right)^2 \left(1 - \frac{1}{8}\left(1 + \frac{\eta_i}{\psi_{\ell}}\right)^2\right)\right]}$$

where q_u = the resulting distributed upper chord load acting on the appropriate half of the roof truss.

$E_u I_u$ = bending rigidity of the upper chord.

$E_{\ell} I_{\ell}$ = bending rigidity of the lower chord.

i = 1 for the left heel joint and i = 2 for the right heel joint.

The influence of the force X_i on the bending moment distribution of the roof truss is considered in item 6.

5. Determine how the support forces R_1 and R_2 are distributed on the upper and lower chords via the fictitious bars in the left and right heel joints. Assuming that the upper and lower chords are placed on rigid supports, the forces X_{ui} and $X_{\ell i}$ (figure 6) can be solved from the equations

$$\frac{X_{u1}}{X_{\ell 1}} = \frac{E_u I_u}{E_\ell I_\ell} \frac{\psi_\ell}{\psi_u} \cos\alpha \frac{\left(1 - \frac{\eta_1}{\psi_\ell}\right)^2 - \frac{\psi_\ell}{7/2 - 3\psi_\ell}\left(1 - \left(\frac{\eta_1}{\psi_\ell}\right)^2\right)^2}{\left(1 - \frac{\xi_1}{\psi_u}\right)^2 - \frac{1}{2}\psi_u\left(1 - \left(\frac{\xi_1}{\psi_u}\right)^2\right)}$$

$$X_{u1} + X_{\ell 1} = R_1$$

where X_{ui} = the vertical component of the force acting on the upper chord.

$X_{\ell i}$ = the vertical component of the force acting on the lower chord.

6. Add the force component X_i to X_{ui} and subtract the force component X_i from $X_{\ell i}$. Calculate the bending moment distribution in the upper and lower chords assuming that the supports are rigid.

7. Consider the yield of the supports by adding to the support moments the additional moments:

$$\Delta M_2 = -\left(1.17X_{u1}\frac{\eta_1}{\psi_u} + 0.97X_{\ell 1}\frac{\eta_1}{\psi_\ell} + 0.34X_{u2}\frac{\eta_2}{\psi_u} + 0.45X_{\ell 2}\frac{\eta_2}{\psi_\ell}\right)\frac{b_u^2}{L\sin^2\alpha}$$

$$\Delta M_4 = -\left(0.34X_{u1}\frac{\eta_1}{\psi_u} + 0.45X_{\ell 1}\frac{\eta_1}{\psi_\ell} + 1.17X_{u2}\frac{\eta_2}{\psi_u} + 0.97X_{\ell 2}\frac{\eta_2}{\psi_\ell}\right)\frac{b_u^2}{L\sin^2\alpha}$$

$$\Delta M_6 = -\left(0.01X_{u1}\frac{\eta_1}{\psi_u} + 0.08X_{\ell 1}\frac{\eta_1}{\psi_\ell} + 0.77X_{u2}\frac{\eta_2}{\psi_u} + 0.80X_{\ell 2}\frac{\eta_2}{\psi_\ell}\right)\frac{b_\ell^2}{L\sin^2\alpha}$$

$$\Delta M_7 = -\left(0.77X_{u1}\frac{\eta_1}{\psi_u} + 0.80X_{\ell 1}\frac{\eta_1}{\psi_\ell} + 0.01X_{u2}\frac{\eta_2}{\psi_u} + 0.08X_{\ell 2}\frac{\eta_2}{\psi_\ell}\right)\frac{b_\ell^2}{L\sin^2\alpha}$$

where X_{ui} and $X_{\ell i}$ are the force components corrected for the force X_i as in item 6.

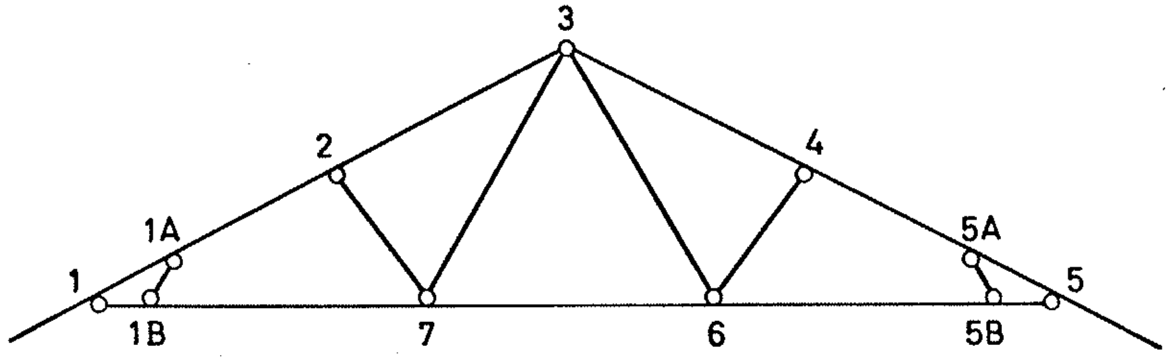


Figure 1

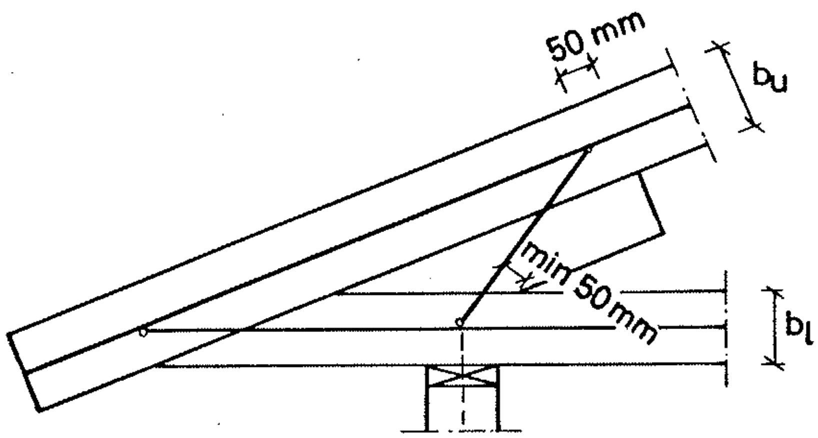


Figure 2

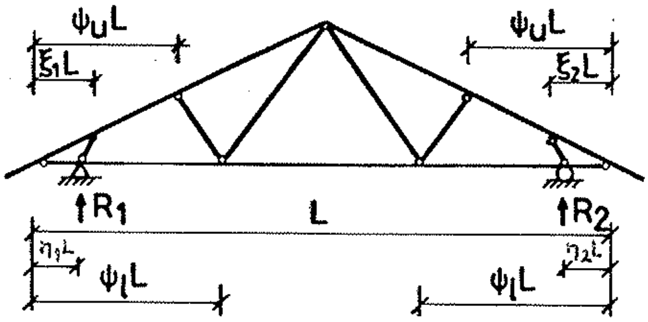


Figure 3

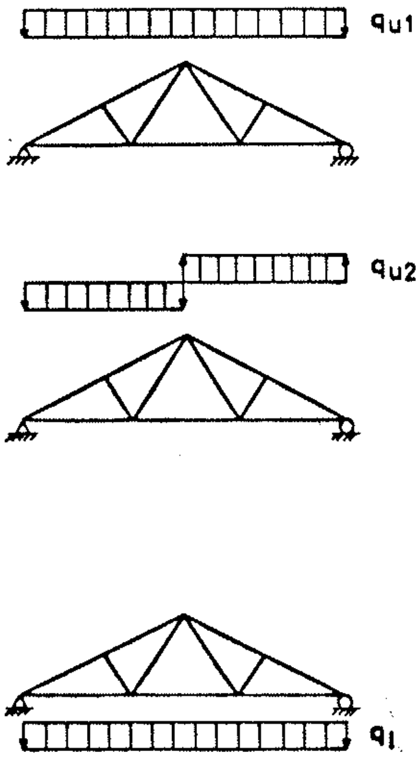


Figure 4

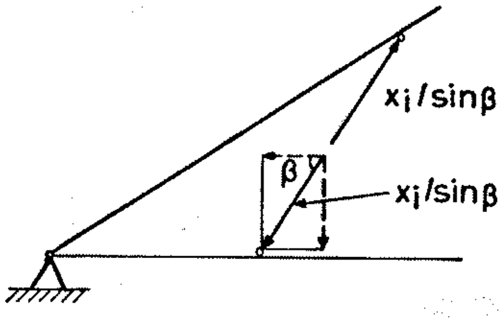


Figure 5

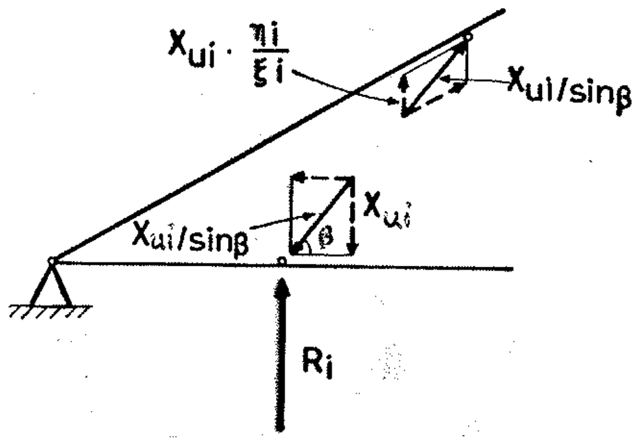


Figure 6

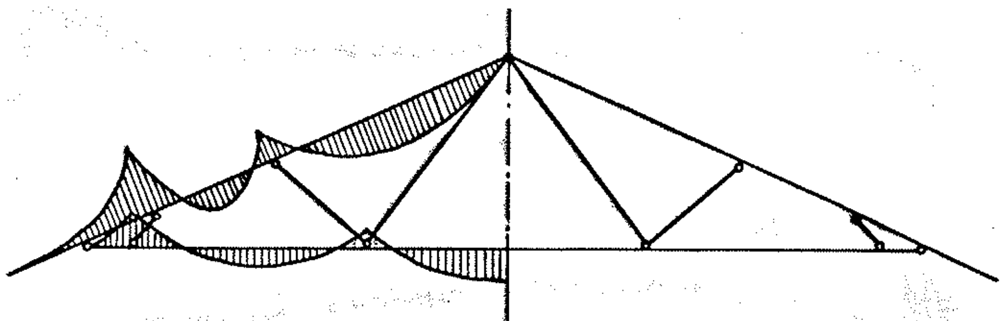


Figure 7

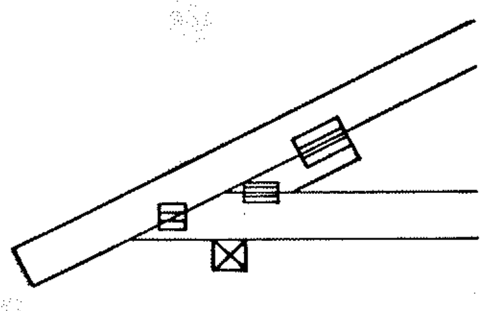


Figure 8

CIB-W18/17-15-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PROPOSAL FOR CHAPTER 7.4 BRACING

by

H Brüninghoff

CEC-Bois, Femib, Glulam, Ulm
Federal Republic of Germany

RAPPERSWIL

SWITZERLAND

MAY 1984

7.4 Bracing

7.4.1 General

This section applies to compression and bending members, which should be hold to avoid unsuitable lateral deflections. This can be done by staying against fixed supports or bracing members.

7.4.2 Single supports

Supports of compression and bending members have to be designed for a load of

$$F_q = \frac{F_N}{\eta_1}$$

where F_N is the force of a compression member or the force of the compression part of a beam.

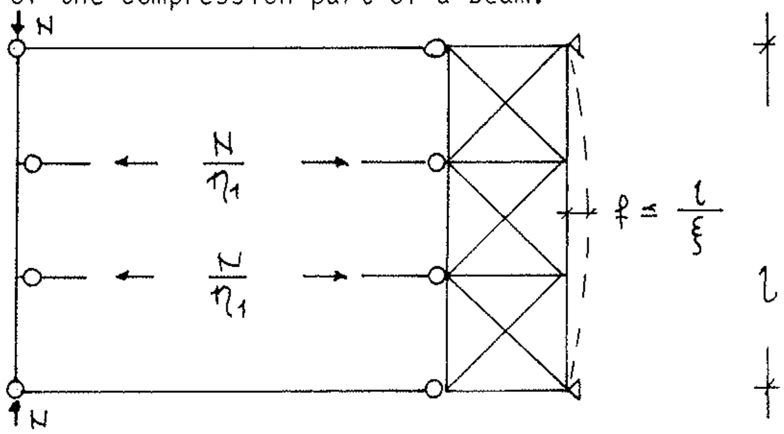


Fig. 7.4 a Single supports

7.4.3 Continous bracing

7.4.3.1 Compression members

Compression chords of trusses have to be supported against bracing members, which are to be designed for an uniform load of

$$q = \frac{m \cdot F_N}{\eta_2 \cdot l}$$

where

m is the number of chords,

F_N the average compression force,

l the span of the bracing member.

The deformation of the bracing members shall not exceed the value $f = l/\xi$.

The procedure can be used approximately for the compression flanges of beams with T- or I- cross sections.

7.4.3.2 Beams with rectangular cross sections

Beams with rectangular cross sections have to be supported against bracing menbers, which are to be designed for an uniform load of

$$q = \frac{m \cdot M}{\eta_3 \cdot l \cdot b}$$

where

m is the number of beams,

M the maximum bending moment of the beam under vertical loading,

l the span of the bracing member,

b the width of the cross section.

The bracing members should support the compression chords of the beams.

The deformation of the bracing members shall not exceed the value $f = l/\xi$.

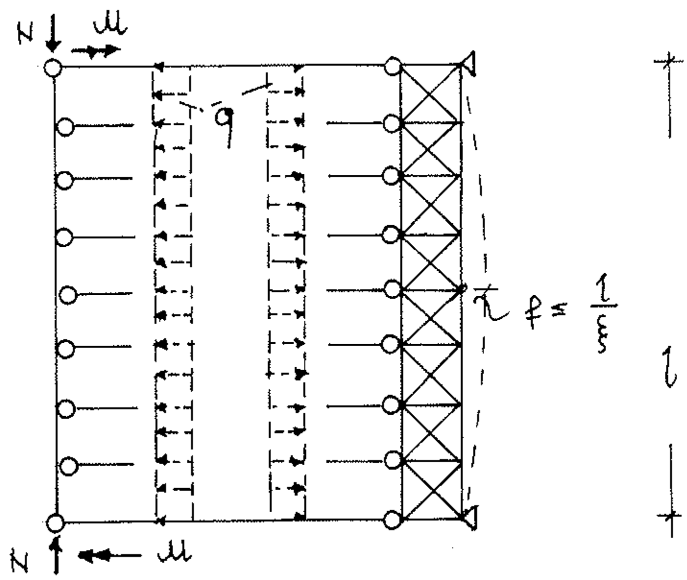


Fig. 7.4 b Continous bracing

Proposal for the values η_i and ξ

η_1 = 100
η_2 = 30
η_3 = 350
ξ = 500

(to be confirmed; note ANNEX 75)

Remarks

The coefficients η_i depend on the initial deflections without loads of the structural members. In addition to it the mode and the line of attack in the cross section of the exterior loads have some influence.

Comments are given by ANNEX 75 (under preparation).

CIB-W18/17-15-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SEISMIC DESIGN OF SMALL WOOD FRAMED HOUSES

by

K F Hansen

Danish Building Research Institute
Denmark

RAPPERSWIL

SWITZERLAND

MAY 1984

CONTENTS

1. INTRODUCTION

It is to be expected that European manufacturers of one-family houses to a growing extent will seek new markets abroad and so in many cases will face the requirements that the houses must be able to withstand earthquakes. A particularly suitable type for earthquake areas is timber framed houses with a sheating of for example plywood, particle boards, fiber boards, plasterboards or similar materials.

In this paper first a qualitative description is given of the forces induced in a house during an earthquake. The general rules for the design of earthquake-proof houses are listed and a method for taking up the forces in panelled timber framed houses based on vertical shear walls and horizontal diaphragms is described. The last chapters of the report deal with the general requirements to be met when designing the structural elements and joints of the house and what steps to take to ensure that the statically secondary elements of the house can withstand seismic vibrations.

Supplementary to the general advice and instructions of the report the Appendix contains guidelines on design of panels subjected to forces in their own plane.

2. QUALITATIVE DESCRIPTION OF THE CONSEQUENCES OF AN EARTHQUAKE ON A HOUSE

The direct consequences of an earthquake on an area are that the ground trembles and that faults and failure lines are formed on the ground surface. By seismic design is to be to be understood design against the consequences of earth termors. It is not possible to give design rules for a situation where the failure line is formed directly under the house. This risk should be minimized through the choice of building site on the basis of data about the geological character of the area.

During an earthquake a building will be exposed through its foundation to a set of vertical and horizontal accelerations of varying intensity and direction. As a result ALL parts of the building connected to the foundation will be exposed to inertia forces, with directions corresponding to the direction of the accelerations. The magnitude of the forces is proportional to the intensity of the accelerations and the weight (mass) of the seperate parts. Thus the loads on a house during an earthquake are of a dynamic nature and design against earthquakes should therefore, ideally be based on a dynamic analysis, where the loading is a set of time dependent displacements of the foundation.

However, a dynamic seismic analysis for 1-2 storey houses is made only in very special cases. This is due partly to the fact that the procedure is rather laborious and subject to considerable uncertainty particularly as regards what damping properties should be attributed to the structural elements and joints, and partly to the fact that the eigenfrequency of such buildings normally is such that the forces can be predicted without performing an actual analysis given the overall damping properties.

3. SEISMIC STANDARDS

As a result of the reasons stated in the above chapter the seismic standard of most countries subtitutes the dynamic analysis with a static for ordinary houses

Normally the seismic design requirements are that the house must be able to withstand a set of horizontal, static forces in two directions at right angles to each other, the size of the force being proportional to the weight (mass) of the building parts. The proportionality factor is stated in the standard dependent on the actual building area.

The precise definition of these forces and their lines of action differ from seismic standard to seismic standard, but the procedure outlined in the following cover most cases.

4. GENERAL RULES FOR THE DESIGN OF HOUSES IN EARTHQUAKE AREAS

The following principles should be followed wherever possible, when planning houses for construction in earthquake areas:

1) Minimize the weight of all parts of the house.
2) Choose a simple, compact plan design.
3) Aim at symmetry. Ideally, symmetry about two vertical planes at right angles to each other.
4) Choose the same layout of walls in both stories of 2-storey houses.

Principle (1) is a direct consequence of the fact that the forces on a house during an earthquake are proportional to the weight of the building parts. Especially the choice of roof covering may have a great influence on the vertical forces on walls and the foundation.

The other principles all aim at the finished building becoming as simple and statically clear as possible, and to avoid torsional forces.

Investigations of damages caused by earthquakes have shown that apperently the simplest structures have the greatest chance of escaping undamaged from an earthquake. This is probably due to the fact that it is much easier to predict the critical spots in a simple structure.

5. STATIC MODEL OF TIMBER FRAMED HOUSES

5.1. Code defined seismic loads.

The code defined seismic loads that a house should be able to withstand are shown on Fig. 1

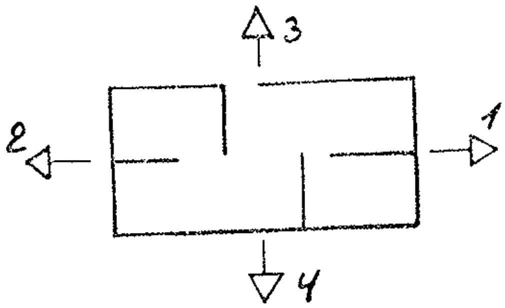


Figure 1: Horizontal section of house.
Lines of action for the seismic loads.

In each direction the total horizontal force is $K = \alpha \cdot V$, where α is a product of several factors to be found in the actual seismic code and V is the weight of the house (including live load).

In highly threatened earthquake areas the horizontal force on the building would typically correspond to 15% of the weight of the building (α multiplied by the partial coefficient for seismic load). The vertical distribution of K can vary somewhat depending on the actual code, but it can generally be assumed that K is distributed on each element according to its weight.

5.2. Timber Framed Houses Conceived as a Shear Wall Structure.

There is general agreement in the earthquake literature that the most effective way for houses to withstand the effects of an earthquake is by shear action in walls and partitions.

The follwong requirements must be met in order that this shear action is effective:

1) Walls, horizontal partitions and roof planes must be able to withstand forces acting in their own plane.

2) The connection between two structural elements must be able to transmit the shear forces.

3) The connection between walls and the foundation must be able to transmit shear forces as well as normal forces.

4) There must be 3 preferably 4 walls that are neither parallel intersect along a common line.

5) All diaphragms must be able to transmit forces perpendicular to their own plane to their supporting structure.

Generally all of these 5 requirements can be met by a timber framed house.

5.3. Determining Forces in Panels and Joints.

When the loads have been determined on the basis of a seismic standard and the structural system has been chosen in accordance with the rules listed in Chapter 4, the following simple procedure can be used to determine the forces in a structure.

A house with a rectangular plan and pitched roof is considered. On Fig. 2 are shown the forces that the house must be able to transmit to the foundation, when the earthquake is assumed to act in the longitudinal direction of the house.

First it is assumed that only the facades of the house can transmit horizontal forces to the foundation.

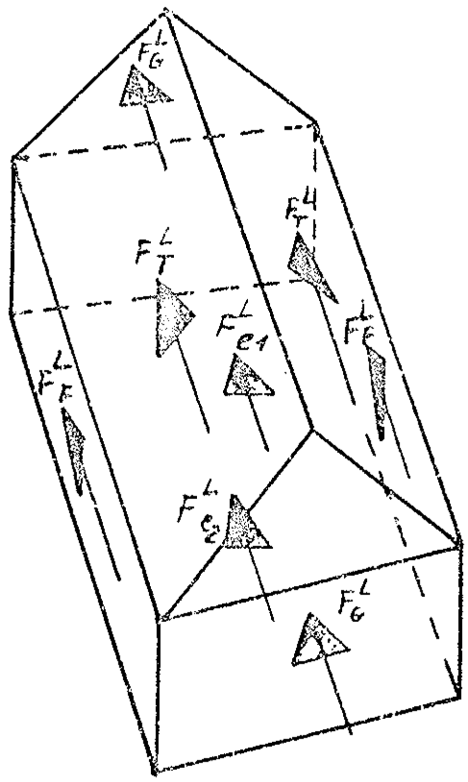


Fig. 2: Seismic forces in the longitudinal direction of the house

F_G^L: Force on a gable

F_F^L: Force on a facade

F_T^L: Force on a roof plane

F_{e1}^L: Force on a horizontal, first floor partition

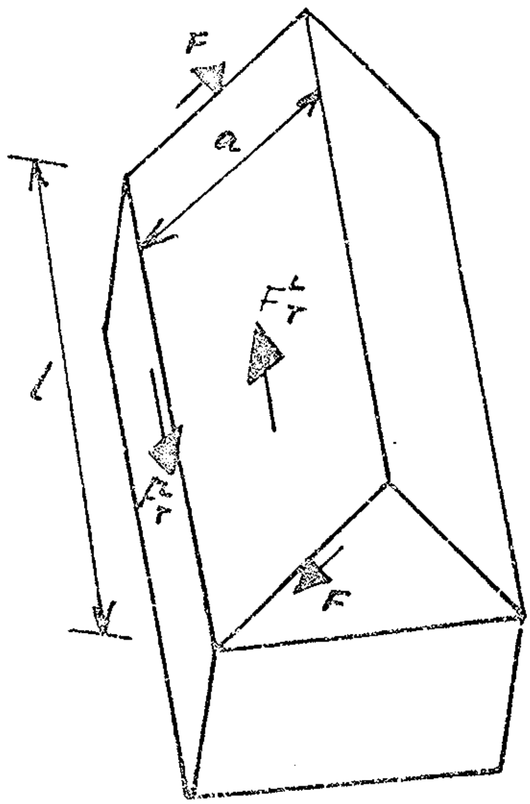
F_{e2}^L: Force on horizontal, ground floor partition

When determining the forces on the basis of

$$F = \alpha \cdot V$$

where V is the weight of the part in question, the live load on the horizontal partitions must be included in V together with the weight of any non-bearing walls.

All the shear forces in a horizontal section just above the foundation must be transmitted in the connection between the facade and the foundation. This connection must also be able to transmit the total overturning moment acting on the house except for the overturning moment from the forces F in the gables.



Length of house: L
Width of roof plane: a

$$F_T^L \cdot \frac{a}{2} = F \cdot L$$

Fig. 3: Transmission of forces from roof to facade and gables.

The forces F are a result of a shifting of the forces in the roof plane.

The forces that the joints between the various walls and partitions must be able to transmit can be calculated from Figs. 2 and 3 by simple equilibrium considerations.

When the earthquake acts in the transverse direction of the house, then both the resulting shear forces on the house and the total overturning moment are transmitted through the connection between the gables and the foundation.

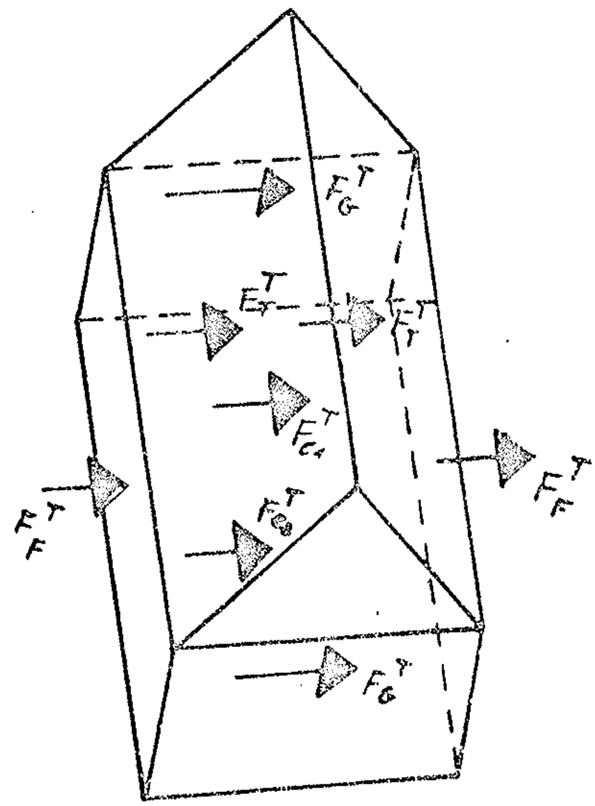


Fig. 4: Earthquake forces in the transverse direction of the house

In the above it was assumed that the horizontal forces were transmitted to the foundation through the gables and the facades. If a house also contains load bearing partition walls that are directly connected to the foundation, then these walls are comparable with the facades and gables as regards the absorption of the horizontal forces. Naturally the division of the forces between inner and outer walls depends on their rigidity as well as the rigidity of the horizontal partitions. If the rigidity of the inner wall is comparable to that of the outer walls, then the horizontal seismic forces deriving from the horizontal partition can be distributed in proportion to how much of the vertical load from the horizontal partitions each wall supports.

6. STRENGTH AND DUCTILITY REQUIREMENTS FOR STRUCTURAL ELEMENTS AND JOINTS

When designing against earthquake forces it is not only prudent to avoid brittle failures but it is a requirements that must be met, if the house is to survive a major earthquake.

During an earthquake the house receives a certain amount of kinetic energy through the movements of its foundations. The house must be able to absorb this energy. If the house behaves elastically until a brittle failure occurs, the situation can be described in a simplified manner as shown on Fig. 5.

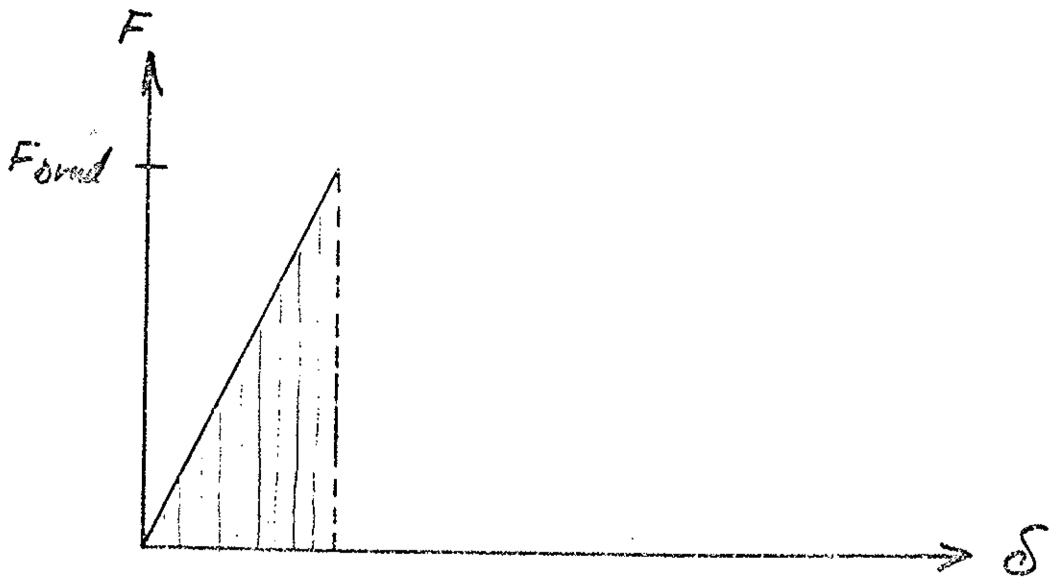


Fig. 5: Stress-strain curve for a house with brittle failure

The hatched section corresponds to the amount of energy that the house can absorb, and this must be larger than the amount of energy received by the house during the earthquake.

If the house behaves in an elastic-plastic way the situation can be described in a simplified manner as shown on Fig. 6.

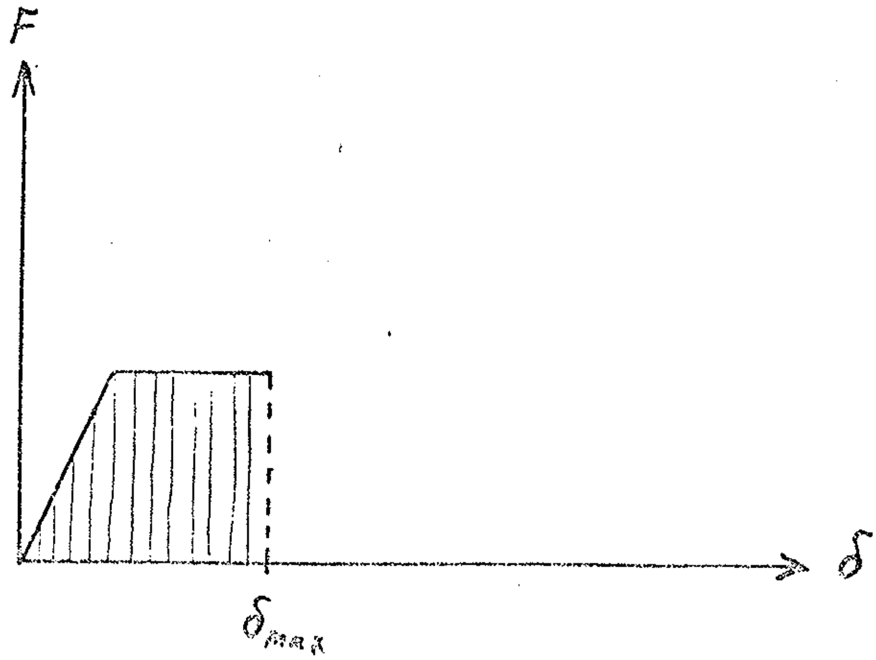


Fig. 6: Stress-strain curve for a house with plastic failure.

The hatched section corresponds to the amount of energy that can be absorbed by the house.

It is evident that the forces generated by an earthquake in a house with a plastic behaviour are smaller than the forces generated in a house with brittle failure behaviour.

The forces given in various seismic standards all assume that part of the seismic energy will be absorbed by plastic deformations, and therefore, the requirement for avoiding brittle failure must be met.

7. REQUIREMENTS FOR NON-BEARING STRUCTURES AND SECONDARY STRUCTURES

If a building is designed and built as a shear wall structure, the relative movements between the various parts of the building are small and the damage from these realtive movements on secondary structures, non-bearing walls, windows, doors etc. will therefore, be relatively small.

In spite of this, details by windows, etc. should be made in such a way that it is possible for a wall to deflect without the window getting a similar deflection in order to avoid broken window panes. In (1) a gap of at least 1/8" $\sim$ 3 mm between wall and window is recommended. For non-bearing walls it would be prudent to build in a certain gap, too.

Even if the shear wall structure reduces the relative deflections among the bearing structural parts, it cannot of course, avoid that all parts within the house are exposed to the earth tremors.

The following rules serve to reduce the unfortunate consequences of these tremors.

1) Cupboard, like book cases, closets etc. ought to be screwed (not nailed to the walls. Closets should be provided with a positive locking device.
2) Brick chimneys and fire places as well as heavy partitions should be avoided.
3) Stoves, boilers and similar, heavy objects must be properly secured to floors and walls, and they should be placed as low as possible.
4) Any gas installation must have an easily accessible closing valve.

APPENDIX

The directions in this paper is of a general character and should be supplemented with a procedure for designing panelled timber frames as well as directions for designing the joints between these.

As will appear from this paper, the action of the dynamic seismic load on houses can be calculated as traditional, static loads (short-term load). This, however, imposes certain limitations on the choice of designs with the result that designs leading to brittle failure should not be used.

This means that the traditionally used design principles as well as a large part of the traditional designs can be employed.

Reference is made to SBI Publication 140: Trækonstruktioner, forbindelser (wooden structures, joints), 1984 regarding the design of the joints.

Regarding panelled timber frames reference is made to H.J. Larsen and Riberholdt: Stabiliserende skiver af træplader (stabilizing wooden panels), F95, Department of Structural engineering, The Technical University of Denmark, where a design procedure based on Danish standards may be found.

As Danish load requirements do not include seismic forces of any importance, the content should be supplemented as follows:

1) All sheatings should be joined by track joints.

2) Nail-glueing should not be used.

3) The nailing between panels and timber frame should be done with few nails to avoid that an overload causes a failure of stability and not a slip in the nailing.

CIB-W18/17-17-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAMPLING OF WOOD FOR JOINT TESTS ON THE BASIS OF DENSITY

by

I Smith and L R J Whale

Timber Research and Development Association
United Kingdom

RAPPERSWIL

SWITZERLAND

MAY 1984

Sampling of wood for joint tests on the basis of density.

1. Introduction

In this paper are described two methods used by TRADA to sample wood for mechanical joint specimens on the basis of density. These methods are employed in the following circumstances:

Method 1

It is required to produce a large number of sets of nominally identical specimens matched on the basis of density and it is not possible to cut a replicate for each set from a common piece of wood.

Method 2

It is required to produce sets of different types of joints matched on the basis of density and it is not possible to cut a replicate for each set from a common piece of wood or the sets have different numbers of replicates.

It is possible to use method 2 to select wood for specimens in a single set to match a prespecified density distribution.

The objective of any density matching procedure is to produce identical distributions of specimen density for each set. It is not sufficient to merely select specimens so that sets have identical means and standard deviations for specimen density. As an illustration Table 1 shows an example of incorrectly matched sets of specimens. In what follows it is shown how matched density distributions between sets have been attained.

Incorrectly matched density distributions. TABLE 1.

Density values (kg/m^3)

Replicate	Set 1	Set 2	Set 3
1	375.0	395.4	414.6
2	425.0	395.4	414.6
3	475.0	500.0	414.6
4	525.0	500.0	585.4
5	575.0	604.6	585.4
6	625.0	604.6	585.4
Mean	500.0	500.0	500.0
S.D.	85.4	85.4	85.4

2. Method 1 - Matching sets of nominally identical specimens

2.1 Theory (Three piece symmetric joint specimens)

Consider the matching of sets of three piece symmetric joint specimens in which all three members are of the same timber species. Within an individual specimen the three members are intended to have identical densities.

Let: n = number of sets of nominally identical specimens
m = number of replicates per set.

There are in total $n \times m$ specimens containing $3 \times n \times m$ pieces of wood ($2 \times n \times m$ nominally identical side members and $n \times m$ nominally identical centre members).

Considering for the purpose of demonstration a case where the cross-sectional area of each side member is half the cross-sectional area of the centre member. The average density for the j^{th} replicate of the i^{th} set is given by:

$$D_{i,j} = (DL_{i,j} + 2DC_{i,j} + DR_{i,j})/4$$
$$i = 1, n \text{ and } j = 1, m \qquad (1)$$

where: $DL_{i,j}$ = density of left hand member
$DC_{i,j}$ = density of centre member
$DR_{i,j}$ = density of right hand member

To be able to perform the matching process it is necessary to determine the density of each of the $3n \times m$ pieces of wood and then to rank the densities of the side members from 1 to $2 \times n \times m$ and rank the densities of the centre members from 1 to $n \times m$.
(Densities ranked in ascending magnitude).

Each piece of wood is assigned to a given set and replicate according to the rules in Table 2.

<u>Ranks of joint member densities</u> TABLE 2

Replicate	Rank of side member density	
	left hand member	right hand member
1, 3, 5, ---	2 (n(j-1) + i) - 1	2 (n(j-1) + i)
2, 4, 6, ---	2 (nj - i) + 1	2 (nj - i + 1)
	Rank of centre member density	
1,3,5, ---	nj + 1 - i	
2,4,6, ---	n (j - 1) + i	

i signifies set number, j signifies replicate number.

The scheme underlying the method of Table 2 is demonstrated in Table 3 for a case where n = 4 and m = 6.

<u>Rank of joint member densities.</u> TABLE 3

Replicate		Set i											
		1			2			3			4		
		L	C	R	L	C	R	L	C	R	L	C	R
	1	1	4	2	3	3	4	5	2	6	7	1	8
	2	15	5	16	13	6	14	11	7	12	9	8	10
j	3	17	12	18	19	11	20	21	10	22	23	9	24
	4	31	13	32	29	14	30	27	15	28	25	16	26
	5	33	20	34	35	19	36	37	18	38	39	17	40
	6	47	21	48	45	22	46	43	23	44	41	24	42

L signifies left hand member, C signifies centre member, R signifies right hand member.

The method described could easily be modified for the matching of joints with other numbers of wooden members. It should however be realised that the quality of the matching achieved by this method will reduce with any reduction in the number of members per specimen. This is because $D_{i,j}$ is averaged over a number of members, equation (1). In general the quality of the matching will improve with any increases in the number of replicates per set.

2.2 Example (Method 1)

TRADA recently undertook a series of longer term tests on three piece symmetric dry European redwood bolted joints subjected to lateral loadings. The cross-sectional area of each side member was half the cross-sectional area of the centre member. It was required to produce 14 matched sets of nominally identical specimens with 6 replicates per set. Applying the theory in Section 2.1 of this paper the results in Table 4 were obtained.

It can be seen from Table 4 that even with as few as 6 replicates per set extremely good matching of average specimen densities was obtained.

3. Method 2 - Matching sets of different types of specimens

3.1 Theory

As mentioned in the introduction the objective of a matching procedure is to produce identical distributions of specimen density for each set of joint specimens. This is achieved when for specimens ranked on the basis of ascending density the i^{th} ranked specimens from each set have a common density. Exact matching of density distributions cannot be achieved economically and there is need for a matching method which gives an optimal selection of wood for joint specimens in situations where there is a limited choice of densities. A method for an optimal selection of wood is described below.

Example 1 - Di,j values (kg/m^3)

TABLE 4

Replicate		Set i															
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	Mean	S.D.
j	1	350.0	349.5	349.5	353.3	357.0	356.0	357.3	355.0	355.5	355.5	356.5	355.5	357.8	358.0	354.7	2.9
	2	372.0	372.0	370.5	370.0	371.0	367.0	367.0	359.5	370.5	366.5	369.3	369.0	369.5	369.5	368.8	3.1
	3	391.5	391.5	391.5	392.0	392.0	387.5	387.5	388.0	388.0	387.5	387.5	387.5	387.5	378.5	388.4	3.4
	4	403.0	405.0	405.0	404.8	404.5	406.0	402.3	406.0	404.3	404.0	402.0	401.5	401.0	401.0	403.6	1.7
	5	411.0	411.0	411.8	412.5	412.5	414.0	414.3	414.5	417.5	426.0	426.0	426.5	426.5	428.0	418.0	6.6
	6	443.0	441.8	440.5	441.0	441.0	439.5	438.0	436.3	433.0	433.0	432.8	446.5	445.8	444.5	439.8	4.5
Mean		395.1	395.1	394.8	395.6	396.3	395.0	394.4	393.2	394.8	395.4	395.8	397.8	398.0	396.7	395.6	
S.D.		29.4	29.3	29.3	28.5	27.5	28.4	27.5	29.1	26.6	28.6	27.8	31.4	30.7	31.2	28.8	

26

The method can only be successfully applied if the number of pieces of wood from which specimens of any given type can be cut is greater than the number of specimens required. (Strictly speaking the requirement is that there must be a greater number of wood densities to choose from than there are replicates). For a good matching, available, densities from which a selection is made should cover the entire target density range.

Method 2 consists of two steps:

Step 1: Calculating target densities for n ranked specimens.

The target density for i^{th} ranked specimen is taken to be the density corresponding to the most likely cumulative frequency associated with the i^{th} highest of n observations. The most likely cumulative frequencies for i = 1, n are assumed to be given by order statistic medians m_i for a uniform distribution on the interval 0 to 1. Using Filliben's algorithm 1/:

$$m_i = \begin{cases} 1 - m_n & , i = 1 \\ (i - 0.3175) / (n + 0.365) & , i = 2, 3, \ldots, n - 1 \\ 0.5^{(1/n)} & , i = n \end{cases} \qquad (2)$$

Assuming that the target density distribution is normally distributed, the target density values are assigned to each of the n specimens in a set according to:

$$\begin{aligned} d_i &= \bar{D} + \phi_N (m_i) \sigma_D \\ &= \bar{D} \ (1 + \phi_N (m_i) V_D) \end{aligned} \qquad (3)$$

where:

d_i = target density for i^{th} ranked replicate, i = 1, n

$\overline{D}$ = mean density for the target density distribution,

σ_D = standard deviation for the target density distribution,

V_D = coefficient of variation for the target density distribution = $\sigma_D / \overline{D}$,

$\phi_N(m_i)$ = percent point function of a standard normal distribution at m_i

= number of standard deviations from mean associated with the m_i level of exclusion.

Step 2: Selection of the best combination of available densities.

Let us assume that there is a transportation cost associated with assigning each available density to each target density and that cost is:

$$C_{ji} = (d_i - x_j)^2 \qquad (4)$$

where: x_j = observed density of j^{th} ranked piece of wood available for specimen preparation, j = 1, m.

The optimum combination of available densities is that combination which minimises the total transportation cost. In any solution account has to be taken of any conditions limiting the supply of wood for each of the available densities. The combination of available densities that minimises the total transportation cost can be found manually on a trial and error basis or on an automated basis using the 'transportation algorithm' from operations research /2/3/. Experience at TRADA has shown that a mixture of manual and automated approaches gives a rapid solution. This mixed approach consists of generating a matrix of transportation costs, making an initial assignment on the basis of judgement and then, using the transportation algorithm, checking for attainment of an optimal solution before if necessary updating the solution iteratively until the optimal solution is found.

The basic principle described can be used to match density distributions for sets of joints with different numbers of replicates.

3.2 Example (Method 2)

In a recent series of tests at TRADA it was required to optimise the selection, on the basis of density, of 20 from 32 pieces of dry European redwood available for the cutting of embedment specimens with nails bearing parallel to grain. The available densities in kg/m^3 were:

392	392	407	424	424	424	427	427
430	430	432	446	448	458	458	470
470	480	485	485	485	485	520	545
545	545	545	602	606	606	606	606

The target density distribution was characterised by a mean of 488.6 kg/m^3 and a standard deviation of 53.5 kg/m^3

Table 5 shows the values for m_i and d_i calculated in accordance with equations (2) and (3). Also shown in Table 5 are the rank, j, and the value, x_j, of the available density assigned to each d_i together with the ratio of x_j to d_i for each i.

With only 32 pieces of wood to select from, a relatively good match to the target distribution was attained. In general the quality of the match will improve with any increase in the number of available densities.

4. Conclusion

The two methods described demonstrate that through use of relevant rational sampling strategies good matching of density distributions for wood in joint specimens can be achieved economically.

REFERENCES

1/ FILLIBEN, J.J. The probability plot correlation coefficient test for normality. Technometrics, Vol.17, No.1, February 1975.

2/ TAHA, H.A. Operations research: an introduction. London, Collier MacMillan Publishers. Third edition, 1982.

3/ BRONSON, R. Schaum 's outline of theory and problems of operations research. McGraw-Hill Inc. 1982.

IS/PI.
30.3.84.

Example 2 - Results

TABLE 5

i	m_i	$\phi_N(m_i)$	d_i (kg/m^3)	j	x_j (kg/m^3)	$\frac{x_j}{d_i}$
1	0.03406	-1.824	391.0	1	393.0	1.0026
2	0.08262	-1.388	414.3	3	407.0	0.9824
3	0.13172	-1.116	428.9	9	430.0	1.0026
4	0.18082	-0.912	439.8	12	446.0	1.0141
5	0.22993	-0.739	449.1	13	448.0	0.9976
6	0.27903	-0.586	457.2	14	458.0	1.0017
7	0.32814	-0.445	464.8	15	458.0	0.9854
8	0.37724	-0.313	471.9	16	470.0	0.9960
9	0.42634	-0.186	478.6	17	470.0	0.9820
10	0.47545	-0.062	485.3	18	480.0	0.9891
11	0.52455	+0.062	491.9	19	485.0	0.9860
12	0.57366	+0.186	498.6	20	485.0	0.9727
13	0.62276	+0.313	505.3	21	485.0	0.9598
14	0.67186	+0.445	512.4	22	485.0	0.9465
15	0.72097	+0.586	520.0	23	520.0	1.0000
16	0.77007	+0.739	528.1	24	545.0	1.0320
17	0.81918	+0.912	537.4	25	545.0	1.0141
18	0.86828	+1.116	548.3	26	545.0	0.9940
19	0.91738	+1.388	562.9	27	545.0	0.9682
20	0.96594	+1.824	586.2	28	602.0	1.0270
					Mean	0.9927
					S.D.	0.0207

i = rank of target density value, i = 1, 20.

m_i = order statistic median for i^{th} ranked replicate.

d_i = target density for i^{th} ranked replicate.

j = rank of available density, j = 1, 32 .

x_j = value of j^{th} ranked available density.

CIB-W18/17-17-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAMPLING STRATEGY FOR PHYSICAL AND MECHANICAL PROPERTIES OF IRISH GROWN SITKA SPRUCE

by

V Picardo

Institute for Industrial Research & Standards
Ireland

RAPPERSWIL

SWITZERLAND

MAY 1984

SAMPLING STRATEGY FOR PHYSICAL AND MECHANICAL PROPERTIES OF IRISH GROWN SITKA SPRUCE

1. Introduction

Of the softwood species grown in Ireland, Sitka Spruce constitutes approximately 45% of the total area covered by forests, and in 1982 it was estimated that about 60% of all newly planted trees in state forests were Sitka Spruce. It is also forecast that by the turn of the century, Ireland will have more than enough timber to satisfy its own needs. However, to date there has not been a systematic study of the quality of the timber, particularly from the structural usage point of view. For this reason, it was decided to carry out a three year project with the objective of obtaining:

(i) basic physical and strength properties of Irish Sitka Spruce that will be available in the near future

(ii) a data base upon which future values could be evaluated from a limited test programme

(iii) relevant data for stress grading.

2. Background

In order to understand the basis for the sampling strategy described in the following sections, it is necessary to explain a little about the terms used in Irish forestry.

A 'forest' is the name for the total area of a plantation which consists of several 'compartments'.

A 'compartment' within a forest is an area, the boundary of which is clearly identifiable such as by a river, road, edge of farmland or county boundary. A 'compartment' would be further divided into 'sub-compartments' which are also sometimes referred to as 'stands'. A 'sub-compartment' has no clearly defined demarcation lines. In general a 'sub-compartment' would be an area consisting of trees of a homogenous species and of the same age. There are, however, some 'sub-compartments' that may have mixed species but these are few.

A 'plot' is an area temporarily marked out in a 'sub-compartment' for the purpose of monitoring the growth or some other characteristics of that 'sub-compartment'.

A growth classification known as 'yield class' (abbreviated to Y.C.) is used to classify a 'sub-compartment' or a whole forest if it is a homogeneous unit. The Y.C. is a measure of the mean annual volume increment. Thus Y.C. 16 means that the crop (whatever the species) will produce or is capable of producing a mean annual increment which reaches a maximum of 16 cu. metres per hectare per annum. The Y.C. system is based upon the total volume production to date divided by age. The primary means by which Y.C. is determined are height and age. Curves have been developed relating height and age to Y.C. Y.C.'s vary from 8 to 28. 'Thinning' is the process of cutting a certain number of trees in a stand in order to influence the growth of the remaining trees. The type and intensity of thinning and thinning cycle depends on the forester and have the greatest influence on the profitability of the stand. Thinning may or may not be carried out right up to the 'maturity' age of the stand.

'Maturity' age is the age when the maximum growth potential of a stand has been reached and is therefore ready for clearfelling. In the case of Sitka Spruce, the maturity age is about 45 years.

3. Sampling Outline

It was decided at the very beginning that three different section sizes would be tested. It was also decided that samples would be obtained from a range of yield classes and five was considered a reasonable number. These would be chosen from the most common Y.C.'s viz. 12, 16, 20, 24 and 28.

Two limitations that had to be considered in determining the actual numbers that would be tested were:

(a) only 300 planks could be 'processed' through the laboratory in 1 year, and

(b) the project duration was scheduled for 3 years.

As a result, a unit of 60 planks per Y.C. per size was adopted, as shown in the table below.

Y.C.	Size 1	Size 2	Size 3	Total per Y.C.
12	60	60	60	180
16	60	60	60	180
20	60	60	60	180
24	60	60	60	180
28	60	60	60	180
Total per size	300	300	300	900

The next problem was how to go about a random selection and this fell into 3 stages:

(a) Selection of 'stands' for each Y.C.

(b) Selection of trees within each 'stand'

(c) Selection of planks.

The selection process adopted was such that, in the case of (a) the probability of selecting a 'stand' increased with the area of the stand, i.e. the larger the area of the stand, the more chance of it being selected; in the case of (b) the probability of selecting a tree of a particular diameter increased with the frequency or number of that particular diameter present in the selected stand and in the case of (c) every plank had an equal chance of being picked.

The problem now was whether to select 1 plank of each size from 60 different stands, 60 planks from a single stand or something intermediate. The first option would be impractical; the second would be too biased. The third option offered the possibility of sampling from all over the country. It was decided once again to opt for 5 stands per yield class which would mean selecting 12 planks of each size from each stand.

4. Selection of Stands

Since the primary objective was to obtain data on the timber that would be available in the market in the near future, only those 'stands' marked for clearfelling in 1983 were considered. As all state forests are well documented and monitored, this information was easily obtained from the Inventories Section of the Forest and Wildlife Service in the form of a computer print-out.

Of the stands that were available, any stands that had either (a) no thinning carried out or (b) were severely understocked, were excluded from the selection process. It was then assumed that all the remaining stands were more or less similarly managed silviculturally.

The areas of each stand were then tabulated against the stand number and a third column containing cumulative areas was added. The cumulative area was multiplied by a factor of 10 to obtain whole numbers as shown in the example below.

Stand No.	Area (ha)	Cumulative Area x 10
1	0.4	4
2	3.6	40
3	2.8	68
4	0.4	72

Numbers between 1 and the maximum number in column 3 above were then selected from random number tables. Thus in the example above, if the number selected was between 41 and 68, stand no. 3 would be selected. It can be seen, therefore, that the bigger the area, the bigger the chance of selection. It was also possible that two stands from the same forest could be selected.

By this method, a total of 15 stands were selected although only 5 were required. This was done to cover the possibility that the stand selected (a) had already been felled; (b) was wind blown; (c) was not the required Y.C. when checked.

In respect to (c) above, it was decided that where a Y.C. measurement of the selected stand did not reach the required value then the nearest adjacent stand of the same age and required Y.C. would be taken. This had to be done in order to expedite the selection process.

It should be mentioned, however, that experience so far has shown that there was an equal likelihood of a higher Y.C. as of a lower Y.C. and therefore on a national basis an assumption was made that the changes in Y.C. upwards and downwards would balance out.

5. Selection of Trees

Once a stand was selected the area map of the stand was divided by a grid into plots of 0.04 ha each. The plots were numbered and then randomly selected from random number tables at the rate of 1 per hectare with a minimum number of 2 plots being selected.

The distributions of diameters at breast height (DBH) of each of the selected plots was then carried out. It was thus possible to obtain an average distribution of DBH for the stand and also the average number of trees (N) per 0.04 ha.

For the purpose of this project, it was decided to cut N trees according to the frequency distribution of DBH. The method adopted in selecting the trees is best described by an example. In the table below is shown a distribution of DBH.

DBH	Tally	No.	Designated Tree Number
10	11	2	1, 2
11	1111	4	3, 4, 5, 6
12	1	1	7
13	~~1111~~	5	8, 9, 10, 11, 12
14	111	3	13, 14, 15

Total = 15

A number was designated to each tree as shown in column 4. These numbers were then drawn one by one out of a bag. Thus if tree no. 9 was drawn out first, then the first tree selected would be of DBH 13. If tree no. 1 was drawn out next, then the second tree selected would be of DBH 10 and so on.

The way this was transferred to the site was as follows. The site map of the stand was crossed by parallel lines in an arbitrary direction approximately equally spaced. In the example above,15 points were then marked approximately equidistant apart along the lines with the first tree being located along the first line an arbitrary distance from one end of the line. Obviously, a tree of the required DBH would not be found at the exact point. So the nearest tree of the required DBH in the vicinity of the point would be selected. The person doing the selection would thus walk up and down the parallel lines until all the trees were selected.

6. Selection of Planks

The selected trees will be felled and cut into bolts of 5 m length each. Bolts with a top diameter less than 17 cm will be excluded. The bolts will be transported to the nearest of 2 State sawmills and converted into planks of the required sizes. The planks obtained will be numbered with the tree number and the section (i.e. whether butt or top).

The planks will then be kiln dried to 18% moisture content and the first 12 planks of each size from each forest will be selected using random number tables. However, any plank with excessive distortions (twist, bow, cup) or wane will be rejected.

7. Analyses

It is anticipated that enough information will be obtained on which the following variations may be ascertained:

(a) Variation of strength between Y.C.'s - which if significant will be used for adjusting the strength properties depending on the mix of Y.C.'s that can be expected in any one year.

(b) Variation of strength between forests within a Y.C. (i.e. geographical location) - which if significant will be used to adjust values for each Y.C.

(c) Variation of rate of rejection (due to distortion) between Y.C.'s. This would be important for forestry management and future planting policy.

Some of the problems that will need to be resolved in the analyses is that of bias due to

(a) Area of selected stands and

(b) Number of planks obtained for each size.

(c) It should be mentioned here that effects due to milling will not be considered.

8. Concluding Comment

Although the data obtained will be used for other objectives (other than those already stated) and the analytical problems from the statistical viewpoint will have to be resolved, it is hoped the results obtained will be a significant step towards forming a methodology for creating data banks for timber properties here in Ireland.

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB - W18

MEETING SEVENTEEN

RAPPERSWIL

SWITZERLAND

MAY 1984

CONTENTS

1. LIST OF DELEGATES

CANADA

T A Eldridge	Don Mills
C K A Stieda	COFI of BC, North Vancouver

DENMARK

A Egerup	Charlottenlund
M Johansen	Statens Byggeforskningsinstitut, Hørsholm
H Riberholt	Technical University of Denmark, Lyngby

FEDERAL REPUBLIC OF GERMANY

H Brüninghoff	CEI-Bois, Femib, Glulam, Ulm
J Ehlbeck	Universität Karlsruhe, Karlsruhe
A Epple	FMPA, Stuttgart
P Glos	Universität München, München
G Steck	Universität Karlsruhe, Karlsruhe

FINLAND

J Kangas	Technical Research Centre of Finland, Espoo
E K Leppavuori	Technical Research Centre of Finland, Espoo
T Poutanen	Tampere
U Saareleinen	Technical Research Centre of Finland, Espoo

ISRAEL

U Korin	Building Research Station, Haifa

NETHERLANDS

J Kuipers	Delft University of Technology, Delft

NORWAY

E Aasheim	Norsk Treteknisk Institutt, Oslo
R Lackner	Norsk Treteknisk Institutt, Oslo
P H Leirtun	Norges Byggstandardiseringsråd, Oslo
T Ø Ramstad	Norges Byggforskningsinstitutt, Oslo

SWEDEN

B Edlund	Chalmers University of Technology, Göteborg
U Girhammer	Fort F-Research Department, Märsta
B Källsner	Svenska Träforskningsinstitutet, Stockholm
B Norén	Svenska Träforskningsinstitutet, Stockholm
B Thunell	Royal Inst. of Techn., Wood Techn. + Processing, Stockholm

SWITZERLAND

E Gehri	Eidgenössische Technische Hochschule, Zürich
U A Meierhofer	EMPA, Dübendorf
J Natterer	Ecole Polytechnique Fédérale, Lausanne
W Winter	Ecole Polytechnique Fédérale, Lausanne

UNITED KINGDOM

L G Booth	Imperial College of Science and Technology, London
H J Burgess	TRADA, High Wycombe
A Fewell	Princes Risborough Lab., Princes Risborough
R Marsh	Ove Arup and Partners, London
J Smith	TRADA, High Wycombe
J G Sunley	TRADA, High Wycombe

UNITED STATES OF AMERICA

D H Brown	American Plywood Association, Tacoma
E G Stern	Virginia Polytechnic Institute & State University, Blacksburg

ZIMBABWE

D Cresswell	University of Zimbabwe, Salisbury

2. CHAIRMAN'S INTRODUCTION

MR. SUNLEY said that after acting as chairman for 17 meetings he would now like to hand over to somebody else. DR. STIEDA had accepted the position of vice-chairman subject to endorsement by the meeting, and MR. SUNLEY said he would now write to CIB Headquarters to propose that DR. STIEDA would take over as chairman at the next meeting, which would be held in Israel at the invitation of DR. KORIN.

3. COOPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: Reporting in the absence of MR. LARSEN at a meeting in New Zealand MR. SUNLEY said he had expressed the hope that matters to be raised in ISO/TC 165, due to meet in Corsica the following week, would be discussed by CIB-W18 in advance. It was felt that strength grouping could well be considered if referred back by ISO. The plywood testing document would be discussed in ISO/TC 165 and also TC 139 before going out for letter ballot as a Draft International Standard. The meeting supported the proposal for public comment and early publication of the document.

RILEM: PROFESSOR KUIPERS said that in his meeting the previous day there had been no further discussion of joints testing. The test method for nails would be published in Materials and Structures in June. No further comments had been received on the one for staples, which had been included in the Lillehammer proceedings and would be sent for publication shortly.

There had been a long consideration of the method of test for structures and there would be much to change. The question of fixing the load factor value was referred to CIB-W18 and could perhaps give rise to a W18 paper. A third draft would be produced and placed before W18 for comment.

After further discussion the plywood test method had been confirmed as a basis for other sheet materials also, and the chipboard document being prepared would be similar to that for plywood. The comments of CIB-W18 were sought on RILEM procedures and draft proposals, and particularly their help in the evaluation of test results; DR. NORÉN thought a paper on Nordic methods should be produced on the last topic.

A discussion of delamination tests by the CIB-W18 meeting concluded that those considered by ISO/TC 165 could be considered as quality control tests and that the RILEM group should separately take up a study of more fundamental tests related to the properties of the material and the glue-line.

CEI-Bois/FEMIB: MR. RIBERHOLT said an international working group would produce proposals for the qualification of glulam at the end of this year, and would then go on to consider production control. At the request of the chairman he undertook to prepare a report for the CIB-W18 proceedings.

IUFRO S5.02: MR. AASHEIM said there would be a meeting of the timber engineering group in Mexico in December, and PROFESSOR EHLBECK was asked to report the proceedings for CIB-W18.

IABSE: PROFESSOR EDLUND reported that this year's IABSE conference in Vancouver in September would include a half-day seminar on wood structures, with eight papers appearing in the proceedings. A number of CIB-W18 members would be attending.

EEC EUROCODES: MR. SUNLEY said the Commission for the European Community would be appointing a number of W18 and CEI-Bois/FEMIB members to prepare a draft for Eurocode 5 on timber. He thought the working group might be formed and active by the next W18 meeting.

4. AFRICAN, CARIBBEAN AND LATIN-AMERICAN SUB-GROUP

MR. CRESSWELL reported on behalf of MR. BECKETT, who was presenting a paper at the Pacific Area Standards Congress meeting in New Zealand. He said Zimbabwe was the centre for the East African region, which hoped to reinforce contacts with the Pacific and build up relations with South America in cooperation with DR. de FREITAS. A report for the proceedings was being prepared by MR. BECKETT. MR. AASHEIM said Norwegian aid was being provided to establish a laboratory in Harare.

5. TRUSSED RAFTER SUB-GROUP

The chairman said it had been hoped that a draft trussed rafter Annex would be produced by Denmark. MR. RIBERHOLT thought an approach could be drafted from two papers before the present meeting and it was agreed that he should undertake this and produce a draft design chapter and Annex for discussion at the next meeting if possible.

In connection with nail-plate design MR. AASHEIM said a meeting was being arranged in Oslo shortly to revive the work.

6. SAMPLING SUB-GROUP

PROFESSOR GLOS said no Annex had yet been prepared but he hoped to produce it for the next meeting. He named the members of the sub-group and said he had written for information to determine differences in regional practice. Replies had been received but the analysis was not yet complete. He gave an illustrated description of the principles being pursued and undertook to supply a paper for the next meeting.

7. TIMBER FRAME HOUSING SUB-GROUP

MR. SUNLEY said he had not been successful in finding a chairman for the sub-group and still wanted a volunteer or suggestions, possibly for someone outside W18. The work would help towards Eurocode 8 on seismic design. DR. NORÉN thought the Nordic group could perhaps make a suggestion. MR. SUNLEY said terms of reference had already been drawn up and he would discuss further developments with DR. NORÉN.

8. COLUMNS

MR. POUTANEN introduced his paper CIB-W18/17-2-1 'Model for Timber Strength under Axial Load and Moment'. In the subsequent discussion MR. RIBERHOLT commented that the CIB formula for columns did not allow for possible out-of-plane deflection. MR. POUTANEN pointed out that the formula for combined tension and bending was inaccurate and asked for further test results to compare with his model.

9. TRUSSED RAFTERS

In the absence of the author, MR. V. PICARDO, note was taken of paper CIB-W18/17-14-1 'Data from Full Scale Tests on Prefabricated Trussed Rafters'. DR. EGERUP reminded the meeting that W18 had asked specially for this large volume of test results which represented a great deal of work and provided very useful data for study by others.

Introducing his paper CIB-W18/17-14-2 'Simplified Static Analysis and Dimensioning of Trussed Rafters', MR. RIBERHOLT said the object was to put forward a method as simple as possible. The Nordic group had agreed on how connections should be modelled in a frame analysis leading to the moment coefficients given in the paper, which was proposed as a basis for an Annex on simplified trussed rafter design.

DR. KÄLLSNER presented his paper CIB-W18/17-14-3, 'Simplified Calculation Method for W-trusses'. The chairman said the proposed analytical method seemed complex compared with methods of fifteen years ago. MR. RIBERHOLT and DR. NORÉN thought the industry favoured methods of medium sophistication, but a simple model could be provided in addition. There was discussion of the possible application of plasticity theory to improve the agreement of models with experimental results.

It was concluded that guidelines had been presented by the two papers, and in response to a question by the chairman MR. RIBERHOLT said he would try his best to develop an Annex on trussed rafter design for the next meeting.

10. PLYWOOD

PROFESSOR EHLBECK presented paper 17-4-1 'Determination of panel shear strength and panel shear modulus of beech plywood in structural sizes' by himself and MR. F. COLLING which described improvements to the RILEM method. He said some comments from DR. POST were now to be checked and discussed.

The chairman and DR. BOOTH suggested it was important that the RILEM proposals should go quickly for public comment, at which stage the comments from Germany could propose the modifications described. DR. NORÉN asked whether the test configuration might be valid for nail plate shear tests and PROFESSOR KUIPERS said it should be useful also for other sheet materials.

Paper 17-4-2 'Ultimate strength of plywood webs' by DR. LEICESTER and DR. PHAM was introduced by MR. RIBERHOLT who raised some questions he felt should be put to the authors. The chairman asked MR. RIBERHOLT to write to DR. LEICESTER thanking him for his contribution and proposing that the first part of the paper might be appropriate as Code material if rewritten suitably.

11. CIB STRUCTURAL TIMBER DESIGN CODE

MR. FEWELL drew attention to the last sentence of Clause 4.1.0 requiring test specimens to be orientated at random in the testing machine. There was a discussion of this requirement compared with placing the worst defect in the test span, which underestimates the characteristic strength in comparison with other materials.

It was concluded that MR. FEWELL, MR. RIBERHOLT and DR. NORÉN would consider the topic and produce a short note giving recommendations. On the related question in joint testing, PROFESSOR KUIPERS said this was not catered for in the RILEM proposals but was to be covered when making recommendations for sampling. The chairman said DR. SMITH would give thought to the matter and make suggestions.

12. JOINTS

Paper 17-7-1, 'Influence of nail properties on nailed joint behaviour' by DR. I. SMITH and others was introduced by the principal author. There followed a discussion of the method of determining the yield strength of nails and the possibility of quality control. The chairman pointed out that the CIB Code indicated a characteristic tensile nail strength varying with diameter, and MR. RIBERHOLT said this had been explained by strain hardening in drawing wire down to the smaller diameters.

Presenting his paper 17-7-2, 'Notes on the effective number of dowels and nails in timber joints', DR. STECK said that it gave formulae for defining joint efficiency in a way better than currently provided in the CIB Code. An extension of the work to non-linear behaviour was proposed by MR. RIBERHOLT with support from the chairman and DR. BOOTH, and DR. SMITH said he had done theoretical work bringing in non-linearity and variability which might assist.

13. BRACING

Paper 17-15-1, 'Proposal for Chapter 7.4 BRACING' by DR. BRÜNINGHOFF was introduced by the author, who said the proposed chapter and Annex would give easy rules to cover the great majority of cases and background information to help with more difficult cases. There was a discussion of the varying bracing requirements in different countries, with a suggestion by PROF.GLOS that options could be provided depending on initial straightness. DR. EGERUP said a typical example would be of interest and DR. BRÜNINGHOFF said this could be provided. Finally the draft chapter was accepted subject to production of an Annex giving the recommended coefficients and an example of the application of the method.

14. TIMBER STRESSES AND GROUPING

Following a request at the previous meeting, MR. FEWELL presented his paper 17-6-1 'The determination of grade stresses from characteristic stresses for BS 5268: Part 2' together with the background paper 17-6-2 'The determination of softwood strength properties for grades, strength classes and laminated timber for BS 5268: Part 2'.

The need for an Annex 44 was accepted after discussion. It was felt that it should not be limited to the practice of a single country but that other countries should be encouraged to present descriptions of their own methods. MR. ELDRIDGE said he would produce a paper explaining sampling and stress derivation in Canada.

15. SAMPLING

Paper 17-17-1 'Sampling of wood for joint tests on the basis of density' by I. SMITH and L.R.J. WHALE was described with illustrations by DR. SMITH. Answering a question by PROF.GLOS he said the approach was different from the ISO method but he felt it was needed for research purposes. DR. NORÉN said the ISO method was based on Nordic recommendations; the new approach might be better and this could be discussed.

Paper 17-17-2 'Sampling strategy for physical and mechanical properties of Irish grown sitka spruce' by V. PICARDO was received in the author's absence. Earlier in the meeting PROF. GLOS had emphasised the importance of describing exactly how the sampling was done, and the paper providing this information for the Irish study was discussed in relation to the choice of sampling method.

16. STRUCTURAL SAFETY

Paper 17-102-1 'Safety principles' by H.J. LARSEN and H. RIBERHOLT was described by the latter as an explanation of principles in the Nordic codes leading to recommendations for CIB-W18. After a discussion of the paper, PROFESSOR KUIPERS gave an illustrated description of studies in Holland apparently indicating that a higher factor of safety than currently applied would need to be adopted for timber to achieve the same level of safety as in steel and concrete structures. The meeting agreed that the conservatism in deriving design stresses for timber should be emphasised, and that the topic should form a major subject for discussion at future meetings.

Paper 17-102-2 'Partial coefficients limit states design codes for structural timberwork' by I. SMITH was accepted as having been adequately considered by the discussion that had already taken place.

17. TIMBER FRAME HOUSING

Paper 17-15-2 'Seismic design of small wood framed houses' by K.F. HANSEN was considered, the chairman noting that it would be a useful paper in connection with developments in Eurocodes.

18. OTHER BUSINESS

PROF. GLOS asked why the harmonisation of grading rules was undertaken by the ECE rather than ISO. The chairman said the difference between North American and European practice was a difficulty, but North American machine grading was likely to be accepted for the United Kingdom. DR. NORÉN said work had been started towards international acceptance of machine grading and the ECE might take it up at a later stage. The chairman suggested that a United Kingdom paper on machine grading might be prepared for the next meeting.

MR. SUNLEY expressed the appreciation of those attending for the organisation and facilities provided by MR. MEIERHOFER and the host country.

19. NEXT MEETING

The next meeting will take place at Beit Oren, near Haifa, Israel, on the 3rd-7th June 1985.

The chairmen of sub-groups are invited to progress the proceedings of their sub-groups well in advance of the main meetings.

20. PAPERS PRESENTED AT THE MEETING

CIB-W18/17-2-1	Model for Timber Strength under Axial Load and Moment - T Poutanen
CIB-W18/17-4-1	Determination of Panel Shear Strength and Panel Shear Modulus of Beech-Plywood in Structural Sizes - J Ehlbeck and F Colling
CIB-W18/17-4-2	Ultimate Strength of Plywood Webs - R H Leicester and L Pham
CIB-W18/17-6-1	The Determination of Grade Stresses from Characteristic Stresses for BS 5268 : Part 2 - A R Fewell
CIB-W18/17-6-2	The Determination of Softwood Strength Properties for Grades, Strength Classes and Laminated Timber for BS 5268: Part 2 - A R Fewell
CIB-W18/17-7-1	Mechanical Properties of Nails and their Influence on Mechanical Properties of Nailed Timber Joints Subjected to Lateral Loads - I Smith, L R J Whale, C Anderson and L Held
CIB-W18/17-7-2	Notes on the Effective Number of Dowels and Nails in Timber Joints - G Steck
CIB-W18/17-9-1	On the Long-Term Carrying Capacity of Wood Structures - Y M Ivanov and Y Y Slavik
CIB-W18/17-14-1	Data from Full Scale Tests on Prefabricated Trussed Rafters - V Picardo
CIB-W18/17-14-2	Simplified Static Analysis and Dimensioning of Trussed Rafters - H Riberholt
CIB-W18/17-14-3	Simplified Calculation Method for W-Trusses - B Källsner

CIB-W18/17-15-1	Proposal for Chapter 7.4 Bracing - H Brüninghoff
CIB-W18/17-15-2	Seismic Design of Small Wood Framed Houses - K F Hansen
CIB-W18/17-17-1	Sampling of Wood for Joint Tests on the Basis of Density - I Smith, L R J Whale
CIB-W18/17-17-2	Sampling Strategy for Physical and Mechanical Properties of Irish Grown Sitka Spruce - V Picardo
CIB-W18/17-102-1	Safety Principles - H J Larsen and H Riberholt
CIB-W18/17-102-2	Partial Coefficients Limit States Design Codes for Structural Timberwork - I Smith

21. CURRENT LIST OF CIB-W18 PAPERS

Technical papers presented to CIB-W18 are identified by a code CIB-W18/a-b-c, where:

a denotes the meeting at which the paper was presented. Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden; February/March 1977
8 Brussels, Belgium; October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979
13 Otaniemi, Finland; June 1980
14 Warsaw, Poland; May 1981
15 Karlsruhe, Federal Republic of Germany; June 1982
16 Lillehammer, Norway; May/June 1983
17 Rapperswil, Switzerland; May 1984

b denotes the subject:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing
9 Duration of Load
10 Timber Beams
11 Environmental Conditions
12 Laminated Members
13 Particle and Fibre Building Boards
14 Trussed Rafters
15 Structural Stability
16 Fire
17 Statistics and Data Analysis
100 CIB Timber Code
101 Loading Codes
102 Structural Design Codes
103 International Standards Organisation
104 Joint Committee on Structural Safety
105 CIB Programme, Policy and Meetings
106 International Union of Forestry Research Organisations

c is simply a number given to the papers in the order in which they appear:

Example: CIB-W18/4-102-5 refers to paper 5 on subject 102 presented at the fourth meeting of W18.

Listed below, by subjects, are all papers that have to date been presented to W18. When appropriate some papers are listed under more than one subject heading.

LIMIT STATE DESIGN

1-1-1	Limit State Design - H J Larsen
1-1-2	The Use of Partial Safety Factors in the New Norwegian Design Code for Timber Structures - O Brynildsen
1-1-3	Swedish Code Revision Concerning Timber Structures - B Norén
1-1-4	Working Stresses Report to British Standards Institution Committee BLCP/17/2
6-1-1	On the Application of the Uncertainty Theoretical Methods for the Definition of the Fundamental Concepts of Structural Safety - K Skov and O Ditlevsen
11-1-1	Safety Design of Timber Structures - H J Larsen

12

TIMBER COLUMNS

2-2-1	The Design of Solid Timber Columns - H J Larsen
3-2-1	The Design of Built-up Timber Columns - H J Larsen
4-2-1	Tests with Centrally Loaded Timber Columns - H J Larsen and S S Pedersen
4-2-2	Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns - B Johansson
5-9-1	Strength of a Wood Column in Combined Compression and Bending with Respect to Creep - B Källsner and B Norén
5-100-1	Design of Solid Timber Columns (First Draft) - H J Larsen
6-100-1	Comments on Document 5-100-1, Design of Solid Timber Columns - H J Larsen and E Theilgaard
6-2-1	Lattice Columns - H J Larsen
6-2-2	A Mathematical Basis for Design Aids for Timber Columns - H J Burgess

6-2-3 Comparison of Larsen an Perry Formulas for Solid Timber Columns - H J Burgess

7-2-1 Lateral Bracing of Timber Struts - J A Simon

8-15-1 Laterally Loaded Timber Columns: Tests and Theory - H J Larsen

17-2-1 Model for Timber Strength under Axial Load and Moment - T Poutanen

SYMBOLS

3-3-1 Symbols for Structural Timber Design - J Kuipers and B Norên

4-3-1 Symbols for Timber Structure Design - J Kuipers and B Norên

1 Symbols for Use in Structural Timber Design

PLYWOOD

2-4-1 The Presentation of Structural Design Data for Plywood - L G Booth

3-4-1 Standard Methods of Testing for the Determination of Mechanical Properties of Plywood - J Kuipers

3-4-2 Bending Strength and Stiffness of Multiple Species Plywood - C K A Stieda

4-4-4 Standard Methods of Testing for the Determination of Mechanical Properties of Plywood - Council of Forest Industries, B.C.

5-4-1 The Determination of Design Stresses for Plywood in the Revision of CP 112 - L G Booth

5-4-2 Veneer Plywood for Construction - Quality Specifications -ISO/TC 139. Plywood, Working Group 6

6-4-1 The Determination of the Mechanical Properties of Plywood Containing Defects - L G Booth

6-4-2 Comparsion of the Size and Type of Specimen and Type of Test on Plywood Bending Strength and Stiffness - C R Wilson and P Eng

6-4-3 Buckling Strength of Plywood: Results of Tests and Recommendations for Calculations - J Kuipers and H Ploos van Amstel

7-4-1 Methods of Test for the Determination of Mechanical Properties of Plywood - L G Booth, J Kuipers, B Norên, C R Wilson

7-4-2 Comments Received on Paper 7-4-1

7-4-3 The Effect of Rate of Testing Speed on the Ultimate Tensile Stress of Plywood - C R Wilson and A V Parasin

7-4-4 Comparison of the Effect of Specimen Size on the Flexural Properties of Plywood Using the Pure Moment Test - C R Wilson and A V Parasin

8-4-1 Sampling Plywood and the Evaluation of Test Results - B Norên

9-4-1 Shear and Torsional Rigidity of Plywood - H J Larsen

9-4-2 The Evaluation of Test Data on the Strength Properties of Plywood - L G Booth

9-4-3 The Sampling of Plywood and the Derivation of Strength Values (Second Draft) - B Norên

9-4-4 On the Use of the CIB/RILEM Plywood Plate Twisting Test: a progress report - L G Booth

10-4-1 Buckling Strength of Plywood - J Dekker, J Kuipers and H Ploos van Amstel

11-4-1 Analysis of Plywood Stressed Skin Panels with Rigid or Semi-Rigid Connections - I Smith

11-4-2 A Comparison of Plywood Modulus of Rigidity Determined by the ASTM and RILEM CIB/3-TT Test Methods - C R Wilson and A V Parasin

11-4-3 Sampling of Plywood for Testing Strength - B Norên

12-4-1 Procedures for Analysis of Plywood Test Data and Determination of Characteristic Values Suitable for Code Presentation - C R Wilson

14-4-1 An Introduction to Performance Standards for Wood-base Panel Products - D H Brown

14-4-2 Proposal for Presenting Data on the Properties of Structural Panels - T Schmidt

16-4-1 Planar Shear Capacity of Plywood in Bending - C K A Stieda

17-4-1 Determination of Panel Shear Strength and Panel Shear Modulus of Beech-Plywood in Structural Sizes - J Ehlbeck and F Colling

17-4-2 Ultimate Strength of Plywood Webs - R H Leicester and L Pham

STRESS GRADING

1-5-1	Quality Specifications for Sawn Timber and Precision Timber - Norwegian Standard NS 3080
1-5-2	Specification for Timber Grades for Structural Use - British Standard BS 4978
4-5-1	Draft Proposal for an International Standard for Stress Grading Coniferous Sawn Softwood - ECE Timber Committee
16-5-1	Grading Errors in Practice - B Thunell
16-5-2	On the Effect of Measurement Errors when Grading Structural Timber - L Nordberg and B Thunell

STRESSES FOR SOLID TIMBER

4-6-1	Derivation of Grade Stresses for Timber in the UK - W T Curry
5-6-1	Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes - W T Curry
5-6-2	The Description of Timber Strength Data - J R Tory
5-6-3	Stresses for EC1 and EC2 Stress Grades - J R Tory
6-6-1	Standard Methods of Test for the Determination of some Physical and Mechanical Properties of Timber in Structural Sizes (third draft) - W T Curry
7-6-1	Strength and Long-term Behaviour of Lumber and Glued Laminated Timber under Torsion Loads - K Möhler
9-6-1	Classification of Structural Timber - H J Larsen
9-6-2	Code Rules for Tension Perpendicular to Grain - H J Larsen
9-6-3	Tension at an Angle to the Grain - K Möhler
9-6-4	Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler
11-6-1	Evaluation of Lumber Properties in the Unites States - W L Galligan and J H Haskell
11-6-2	Stresses Perpendicular to Grain - K Möhler
11-6-3	Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4) - K Möhler

12-6-1 Strength Classifications for Timber Engineering Codes - R H Leicester and W G Keating

12-6-2 Strength Classes for British Standard BS 5268 - J R Tory

13-6-1 Strength Classes for the CIB Code - J R Tory

13-6-2 Consideration of Size Effects and Longitudinal Shear Strength for Uncracked Beams - R O Foschi and J D Barrett

13-6-3 Consideration of Shear Strength on End-Cracked Beams - J D Barrett and R O Foschi

15-6-1 Characteristic Strength Values for the ECE Standard for Timber - J G Sunley

16-6-1 Size Factors for Timber Bending and Tension Stresses - A R Fewell

16-6-2 Strength Classes for International Codes - A R Fewell and J G Sunley

17-6-1 The Determination of Grade Stresses from Characteristic Stresses for BS 5268 : Part 2 - A R Fewell

17-6-2 The Determination of Softwood Strength Properties for Grades, Strength Classes and Laminated Timber for BS 5268: Part 2 - A R Fewell

TIMBER JOINTS AND FASTENERS

1-7-1 Mechanical Fasteners and Fastenings in Timber Structures - E J Stern

4-7-1 Proposal for a Basic Test Method for the Evaluation of Structural Timber Joints with Mechanical Fasteners and Connectors - RILEM 3TT Committee

4-7-2 Test Methods for Wood Fasteners - K Möhler

5-7-1 Influence of Loading Procedure on Strength and Slip-Behaviour in Testing Timber Joints - K Möhler

5-7-2 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures - RILEM 3TT Committee

5-7-3 CIB-Recommendations for the Evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures - J Kuipers

6-7-1	Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures (seventh draft) - RILEM 3TT Committee
6-7-2	Proposal for Testing of Integral Nail Plates as Timber Joints - K Möhler
6-7-3	Rules for Evaluation of Values of Strength and Deformation from Test Results - Mechanical Timber Joints - M Johansen, J Kuipers, B Norén
6-7-4	Comments to Rules for Testing Timber Joints and Derivation of Characteristic Values for Rigidity and Strength - B Norén
7-7-1	Testing of Integral Nail Plates as Timber Joints - K Möhler
7-7-2	Long Duration Tests on Timber Joints - J Kuipers
7-7-3	Tests with Mechanically Jointed Beams with a Varying Spacing of Fasteners - K Möhler
7-100-1	CIB-Timber Code Chapter 5.3 Mechanical Fasteners; CIB Timber Standard 06 and 07 - H J Larsen
9-7-1	Design of Truss Plate Joints - F J Keenan
9-7-2	Staples - K Möhler
11-7-1	A Draft Proposal for an International Standard: ISO Document ISO/TC 165N 38E
12-7-1	Load-Carrying Capacity and Deformation Characteristics of Nailed Joints - J Ehlbeck
12-7-2	Design of Bolted Joints - H J Larsen
12-7-3	Design of Joints with Nail Plates - B Norén
13-7-1	Polish Standard BN-80/7159-04:Parts 00-01-02-03-04-05. "Structures from Wood and Wood-based Materials. Methods of Test and Strength Criteria for Joints with Mechanical Fasteners"
13-7-2	Investigation of the Effect of Number of Nails in a Joint on its Load Carrying Ability - W Nozynski
13-7-3	International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber - B Norén
13-7-4	Design of Joints with Nail Plates - Calculation of Slip - B Norén

13-7-5	Design of Joints with Nail Plates - The Heel Joint - B Källsner
13-7-6	Nail Deflection Data for Design - H J Burgess
13-7-7	Test on Bolted Joints - P Vermeyden
13-7-8	Comments to paper CIB-W18/12-7-3 "Design of Joints with Nail Plates" - B Norên
13-7-9	Strength of Finger Joints - H J Larsen
13-100-4	CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim
14-7-1	Design of Joints with Nail Plates (second edition) - B Norên
14-7-2	Method of Testing Nails in Wood (second draft, August 1980) - B Norên
14-7-3	Load-Slip Relationship of Nailed Joints - -J Ehlbeck and H J Larsen
14-7-4	Wood Failure in Joints with Nail Plates - B Norên
14-7-5	The Effect of Support Eccentricity on the Design of W-and WW-Trusses with Nail Plate Connectors - B Källsner
14-7-6	Derivation of the Allowable Load in Case of Nail Plate Joints Perpendicular to Grain - K Möhler
14-7-7	Comments on CIB-W18/14-7-1 - T A C M van der Put
15-7-1	Final Recommendation TT-1A: Testing Methods for Joints with Mechanical Fasteners in Load-Bearing Timber Structures. Annex A Punched Metal Plate Fasteners - Joint Committee RILEM/CIB-3TT
16-7-1	Load-Carrying Capacity of Dowels - E Gehri
16-7-2	Bolted Timber Joints: a Literature Survey - N Harding
16-7-3	Bolted Timber Joints: Practical Aspects of Construction and Design; a Survey - N Harding
16-7-4	Bolted Timber Joints: Draft Experimental Work Plan - Building Research Association of New Zealand
17-7-1	Mechanical Properties of Nails and their Influence on Mechanical Properties of Nailed Timber Joints Subjected to Lateral Loads - I Smith, L R J Whale, C Anderson and L Held

17-7-2 Notes on the Effective Number of Dowels and Nails in Timber Joints - G Steck

LOAD SHARING

3-8-1 Load Sharing - An Investigation on the State of Research and Development of Design Criteria - E Levin

4-8-1 A Review of Load-Sharing in Theory and Practice - E Levin

4-8-2 Load Sharing - B Norén

DURATION OF LOAD

3-9-1 Definitions of Long Term Loading for the Code of Practice - B Norén

4-9-1 Long Term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen

5-9-1 Strength of a Wood Column in Combined Compression and Bending with Respect to Creep - B Källsner and B Norén

6-9-1 Long Term Loading for the Code of Practice (Part 2) - B Norén

6-9-2 Long Term Loading - K Möhler

6-9-3 Deflection of Trussed Rafters under Alternating Loading during a Year - T Feldborg and M Johansen

7-6-1 Strength and Long-Term Behaviour of Lumber and Glued-Laminated Timber under Torsion Loads - K Möhler

7-9-1 Code Rules Concerning Strength and Loading Time - H J Larsen and E Theilgaard

17-9-1 On the Long-Term Carrying Capacity of Wood Structures - Y M Ivanov and Y Y Slavic

TIMBER BEAMS

4-10-1 The Design of Simple Beams - H J Burgess

4-10-2 Calculation of Timber Beams Subjected to Bending and Normal Force - H J Larsen

5-10-1 The Design of Timber Beams - H J Larsen

9-10-1 The Distribution of Shear Stresses in Timber Beams - F J Keenan

9-10-2 Beams Notched at the Ends - K Möhler

11-10-1 Tapered Timber Beams - H Riberholt

13-6-2	Consideration of Size Effects in Longitudinal Shear Strength for Uncracked Beams - R O Foschi and J D Barrett
13-6-3	Consideration of Shear Strength on End-Cracked Beams - J D Barrett and R O Foschi

ENVIRONMENTAL CONDITIONS

5-11-1	Climate Grading for the Code of Practice - B Norên
6-11-1	Climate Grading (2) - B Norên
9-11-1	Climate Classes for Timber Design - F J Keenan

LAMINATED MEMBERS

6-12-1	Directives for the Fabrication of Load-Bearing Structures of Glued Timber - A van der Velden and J Kuipers
8-12-1	Testing of Big Glulam Timber Beams - H Kolb and P Frech
8-12-2	Instruction for the Reinforcement of Apertures in Glulam Beams - H Kolb and P Frech
8-12-3	Glulam Standard Part 1: Glued Timber Structures; Requirements for Timber (Second Draft)
9-12-1	Experiments to Provide for Elevated Forces at the Supports of Wooden Beams with Particular Regard to Shearing Stresses and Long-term Loadings - F Wassipaul and R Lackner
9-12-2	Two Laminated Timber Arch Railway Bridges Built in Perth in 1849 - L G Booth
9-6-4	Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler
11-6-3	Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4)- K Möhler
12-12-1	Glulam Standard Part 2: Glued Timber Structures; Rating (3rd draft)
12-12-2	Glulam Standard Part 3: Glued Timber Structures; Performance (3rd draft)
13-12-1	Glulam Standard Part 3: Glued Timber Structures; Performance (4th draft)
14-12-1	Proposals for CEI-Bois/CIB-W18 Glulam Standards - H J Larsen
14-12-2	Guidelines for the Manufacturing of Glued Load-Bearing Timber Structures - Stevin Laboratory

14-12-3	Double Tapered Curved Glulam Beams - H Riberholt
14-12-4	Comment on CIB-W18/14-12-3 - E Gehri

PARTICLE AND FIBRE BUILDING BOARDS

7-13-1	Fibre Building Boards for CIB Timber Code (First Draft) - O Brynildsen
9-13-1	Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto and J Ehlbeck
9-13-2	The Structural Use of Tempered Hardboard - W W L Chan
11-13-1	Tests on Laminated Beams from Hardboard under Short- and Longterm Load - W Nozynski
11-13-2	Determination of Deformation of Special Densified Hardboard Under Long-term Load and Varying Temperature and Humidity Conditions - W Halfar
11-13-3	Determination of Deformation of Hardboard under Long-term Load in Changing Climate - W Halfar
14-4-1	An Introduction to Performance Standards for Wood-Base Panel Products - D H Brown
14-4-2	Proposal for Presenting Data on the Properties of Structural Panels - T Schmidt
16-13-1	Effect of Test Piece Size on Panel Bending Properties - P W Post

TRUSSED RAFTERS

4-9-1	Long-term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen
6-9-3	Deflection of Trussed Rafters under Alternating Loading During a Year - T Feldborg and M Johansen
7-2-1	Lateral Bracing of Timber Struts - J A Simon
9-14-1	Timber Trusses - Code Related Problems - T F Williams
9-7-1	Design of Truss Plate Joints - F J Keenan
10-14-1	Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon
11-14-1	Design of Metal Plate Connected Wood Trusses - A R Egerup

12-14-1 A Simple Design Method for Standard Trusses - A R Egerup

13-14-1 Truss Design Method for CIB Timber Code - A R Egerup

13-14-2 Trussed Rafters, Static Models - H Riberholt

13-14-3 Comparison of 3 Truss Models Designed by Different Assumptions for Slip and E-Modulus - K Möhler

14-14-1 Wood Trussed Rafter Design - T Feldborg and M Johansen

14-14-2 Truss-Plate Modelling in the Analysis of Trusses - R O Foschi

14-14-3 Cantilevered Timber Trusses - A R Egerup

14-7-5 The Effect of Support Eccentricity on the Design of W- and WW- Trusses with Nail Plate Connectors - B Källsner

15-14-1 Guidelines for Static Models of Trussed Rafters - H Riberholt

15-14-2 The Influence of Various Factors on the Accuracy of the Structural Analysis of Timber Roof Trusses - F R P Pienaar

15-14-3 Bracing Calculations for Trussed Rafter Roofs - H J Burgess

15-14-4 The Design of Continuous Members in Timber Trussed Rafters with Punched Metal Connector Plates - P O Reece

15-14-5 A Rafter Design Method Matching U.K. Test Results for Trussed Rafters - H J Burgess

16-14-1 Full-Scale Tests on Timber Fink Trusses Made from Irish Grown Sitka Spruce - V Picardo

17-14-1 Data from Full Scale Tests on Prefabricated Trussed Rafters - V Picardo

17-14-2 Simplified Static Analysis and Dimensioning of Trussed Rafters - H Riberholt

17-14-3 Simplified Calculation Method for W Trusses - B Källsner

STRUCTURAL STABILITY

8-15-1	Laterally Loaded Timber Columns: Tests and Theory - H J Larsen
13-15-1	Timber and Wood-Based Products Structures. Panels for Roof Coverings. Methods of Testing and Strength Assessment Criteria. Polish Standard BN-78/7159-03
16-15-1	Determination of Bracing Structures for Compression Members and Beams - H Brüninghoff
17-15-1	Proposal for Chapter 7.4 Bracing - H Brüninghoff
17-15-2	Seismic Design of Small Wood Framed Houses - K F Hansen

FIRE

12-16-1	British Standard BS 5268 the Structural Use of Timber: Part 4 Fire Resistance of Timber Structures
13-100-2	CIB Structural Timber Design Code. Chapter 9. Performance in Fire

STATISTICS AND DATA ANALYSIS

13-17-1	On Testing Whether a Prescribed Exclusion Limit is Attained - W G Warren
16-17-1	Notes on Sampling and Strength Prediction of Stress Graded Structural Timber - P Glos
16-17-2	Sampling to Predict by Testing the Capacity of Joints, Components and Structures - B Norén
16-17-3	Discussion of Sampling and Analysis Procedures - P W Post
17-17-1	Sampling of Wood for Joint Tests on the Basis of Density - I Smith, L R J Whale
17-17-2	Sampling Strategy for Physical and Mechanical Properties of Irish Grown Sitka Spruce - V Picardo

CIB TIMBER CODE

13-100-3c	Comments on CIB Structural Timber Design Code - British Standards Institution
13-100-4	CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim
14-103-2	Comments on the CIB Structural Timber Design Code - R H Leicester
15-103-1	Resolutions of TC 165-meeting in Athens 1981-10-12/13

LOADING CODES

4-101-1	Loading Regulations - Nordic Committee for Building Regulations
4-101-2	Comments on the Loading Regulations - Nordic Committee for Building Regulations

STRUCTURAL DESIGN CODES

1-102-1	Survey of Status of Building Codes, Specifications etc., in USA - E G Stern
1-102-2	Australian Codes for Use of Timber in Structures - R H Leicester
1-102-3	Contemporary Concepts for Structural Timber Codes - R H Leicester
1-102-4	Revision of CP 112 - First Draft, July 1972 - British Standards Institution
4-102-1	Comparsion of Codes and Safety Requirements for Timber Structures in EEC Countries - Timber Research and Development Association
4-102-2	Nordic Proposals for Safety Code for Structures and Loading Code for Design of Structures - O A Brynildsen
4-102-3	Propasal for Safety Codes for Load-Carrying Structures - Nordic Committee for Building Regulations
4-102-4	Comments to Proposal for Safety Codes for Load-Carrying Structures - Nordic Committee for Building Regulations
4-102-5	Extract from Norwegian Standard NS 3470 "Timber Structures"
4-102-6	Draft for Revision of CP 112 "The Structural Use of Timber" - W T Curry
8-102-1	Polish Standard PN-73/B-03150: Timber Structures; Statistical Calculations and Designing.

8-102-2	The Russian Timber Code: Summary of Contents
9-102-1	Svensk Byggnorm 1975 (2nd Edition); Chapter 27: Timber Construction
11-102-1	Eurocodes - H J Larsen
13-102-1	Program of Standardisation Work Involving Timber Structures and Wood-Based Products in Poland
17-102-1	Safety Principles - H J Larsen and H Riberholt
17-102-2	Partial Coefficients Limit States Design Codes for Structural Timberwork - I Smith

INTERNATIONAL STANDARDS ORGANISATION

3-103-1	Method for the Preparation of Standards Concerning the Safety of Structures (ISO/DIS 3250) - International Standards Organisation ISO/TC98
4-103-1	A Proposal for Undertaking the Preparation of an International Standard on Timber Structures - International Standards Organisation
5-103-1	Comments on the Report of the Consultion with Member Bodies Concerning ISO/TS/P129 - Timber Structures - Dansk Ingeniorforening
7-103-1	ISO Technical Committees and Membership of ISO/TC 165
8-103-1	Draft Resolutions of ISO/TC 165
12-103-1	ISO/TC 165 Ottawa,September 1979
13-103-1	Report from ISO/TC 165 - A Sørensen
14-103-1	Comments on ISO/TC 165 N52 "Timber Structures; Solid Timber in Structural Sizes; Determination of Some Physical and Mechanical Properties"
14-103-2	Comments on the CIB Structural Timber Design Code - R H Leicester

JOINT COMMITTEE ON STRUCTURAL SAFETY

3-104-1	International System on Unified Standard Codes of Practice for Structures - Comitê Europêen du Bêton (CEB)
7-104-1	Volume 1: Common Unified Rules for Different Types of Construction and Material - CEB

CIB PROGRAMME, POLICY AND MEETINGS

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

CIB-W18/17-2-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

MODEL FOR TIMBER STRENGTH UNDER AXIAL LOAD AND MOMENT

by

T Poutanen
Finland

RAPPERSWIL

SWITZERLAND

MAY 1984

MODEL FOR TIMBER STRENGTH UNDER AXIAL LOAD AND MOMENT

Tuomo Poutanen

Contents

MODEL FOR TIMBER STRENGTH UNDER AXIAL LOAD AND MOMENT

Tuomo Poutanen

SUMMARY

The model is based on bi-linear stress-strain relationship. It is assumed there are two failure reasons. In compression failure occurs when critical strain is reached. In tension failure occurs, when stress exceeds defect strength. It is assumed that defect strength defines completely the tension strength according to the principle of the weakest link of chain or the principle of brittle fracture mechanics. It is assumed that there is only one defect per cross section.

Interaction diagram for axial load and moment is derived. Also equation for size effect is derived. Comparisons to available test results are done.

1. INTRODUCTION

It is known that the generally used linear interaction formula:

$$\frac{N}{N_u} + \frac{M}{M_u} \leq 1 \qquad (1)$$

is conservative.

The author has developed a timber strength model which is different from formula 1.
The author has no resources to make tests to verify the model.
The model is presented here to raise discussion and to get tests results.

2. LIMITATIONS

Due to lack of time and shortage of resources:

a. Only rectangular cross section is handled.
b. Stress-strain relationship in compression is assumed to be bi-linear.
c. Formulae are derived to deal with mean values.
d. Size effect is not included in strength model.
e. The emphasis is on timber which has an equal mean tension and compression strength.
f. It is assumed the moment affects either of the cross section main axes.

3. BASIC ASSUMPTIONS

3.1. Cross section deformations

Cross section planes are assumed to remain planes under load.

3.2. Stress-strain relationship

Stress-strain relationship is shown in figure 1.
It must be especially noted:

a. Behaviour in tension is linear up to failure.
b. Behaviour in compression is bi-linear. After strain has reached a certain value stress does not increase but remains constant, f_c. This assumption is done mainly to simplify calculations. However it is shown / 1,2 / that this is a good approximation for compression behaviour.

Compression bending strength f_{bc} is assumed to be equal to compression strength f_c.

$$f_{cb} = f_c \tag{3}$$

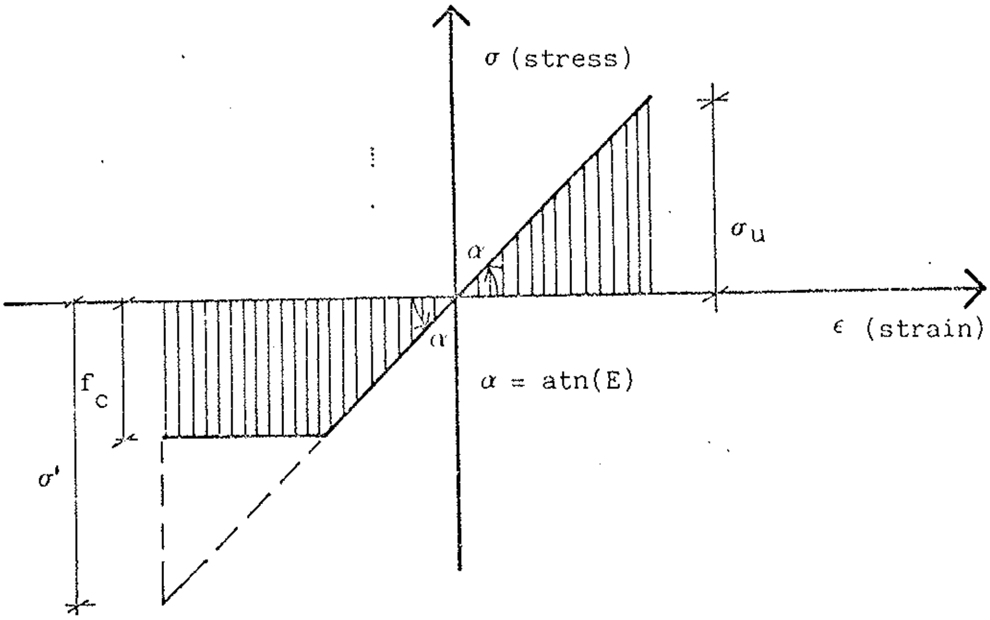


Figure 1, assumed stress-strain relationship

3.3. Cross section failure

There are two reasons for failure:
In compression failure occurs when critical strain is reached, see figure 1.

$$|\sigma'| = n\, f_c \tag{4}$$

In tension failure occurs when stress reaches defect strength, figure 2.

$$\sigma = f_v \tag{5}$$

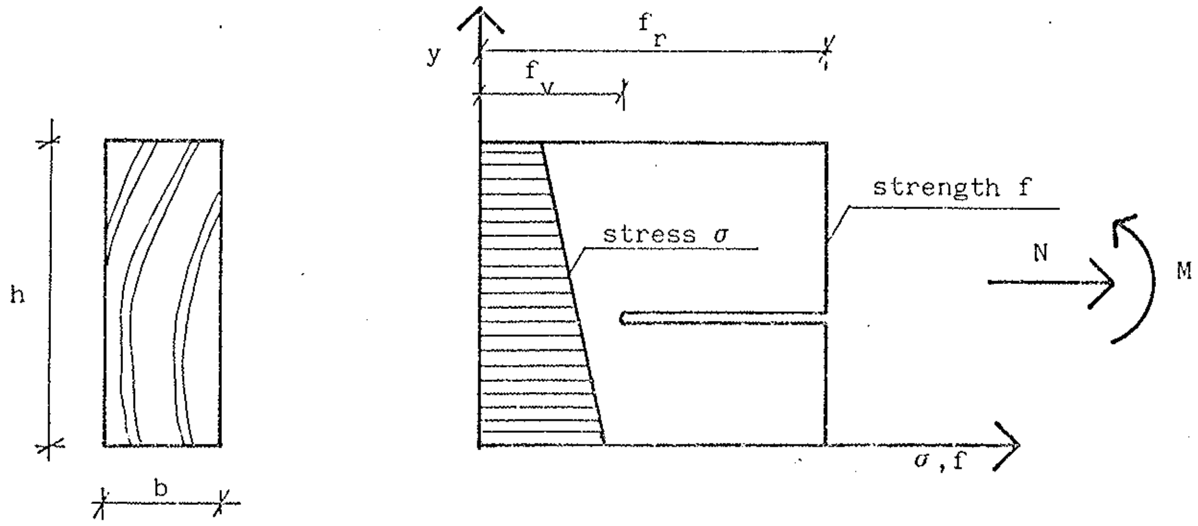


Figure 2, tension failure occurs when stress σ reaches defect strength f_v

This is the key assumption of the model. Defects dominate tension strength totally. Defect free timber has infinite strength, but if it contains a defect and if it fails it leads to complete failure of cross section in spite of the strength of the defect free part of cross section. Defect strength f_v is assumed to be same as tension strength f_t by mean value and distribution if tension test is carried out with a test sample which has one defect.

$$f_v = f_t \tag{6}$$

However, due to practical reasons, f_v must be given different meaning. Test samples have a certain size and contain a different number of defects and fail when stress reaches the weakest defect. So, equation 5 depends on test specimen and f_v is strength of the weakest defect of said specimen.

Effect of shear force on strength is ignored.

3.4. Defect

Defect is assumed to be local strength disturbance e.g. knot or inclined fiber. Thenature of defect is not described and handled further. Anyhow, defect must be relatively small compared to cross section. Also defect has the nature of brittle fracture mechanics. This also means that defect might be more dangerous than hole, because defect can destroy the strength of defect free timber. It is not necessary to make an assumption on distance between defects on timber beam. This distance might possibly be 200 ... 500 mm, but there must be only small number of defects on timber beam.
Size effect is ignored in the strength model, that is why it is assumed there is one defect per tension part of cross section. Timber between defects is defect free and is strong enough to avoid failures. So, failure always occurs in cross sections with defect, if tension stress causes the failure.

3.5. Effect of load duration and moisture

Cross section behaviour can be described by values f_c, f_v, n, E. Load duration and moisture have an effect on cross section through these values. This topic will not be handled further.

3.6. Size and grade effect

The model basically deals with cross section strength with one defect on tension side. In reality a timber beam has several defects. The number of defects and defect strength depend upon beam size and timber grade. However, the strength of the beam is dominated by the worst defect and there is analogy between cross section strength with one defect and beam strength with the worst defect.

Problems arise when size increases. Questions to be answered are: how the defects are located and what is the density. In the strength model these questions may remain open but when the model is used to find out size effects assumptions must be made. This topic will be handled further in section 6.

3.7. Strength deviation

Deviation consists of deviation of f_c, f_v, n and E. To simplify calculations deviation of f_c, n and E is ignored. Ignoring the deviation of f_c and n is justified because it affects results very little. Ignoring the deviation of E is justified because only strength values are handled and E has no effect on them (but has a large influence on diflections). It turns out that main results of the model can be derived ignoring also the deviation of f_v. It also turns out that distribution of the deviation of f_v has no practical influence on results.

4. Cross section strength

4.1. Introduction

According to the basic assumptions there may be two failure reasons: exceeding the compression strain or exceeding the defect strength. Thus cross section has at least two design states. Due to elasto-plastic behaviour of cross section there are altogether four design states. These are listed in the order of decreasing normal force:

a. Cross section under tension, state 1
b. Cross section under elastic compression and tension, state 2
c. Plastic cross section, brittle failure, state 3
d. Plastic cross section, ductile failure, state 4

4.2. Cross section under tension, state 1

When the cross section is completely under tension failure occurs when stress $\sigma(N, M, y)$ reaches the defect strength f_v, figure 3. It is assumed that stress never reaches the defect free timber strength f_r or reaches it so seldom that this case can be ignored.

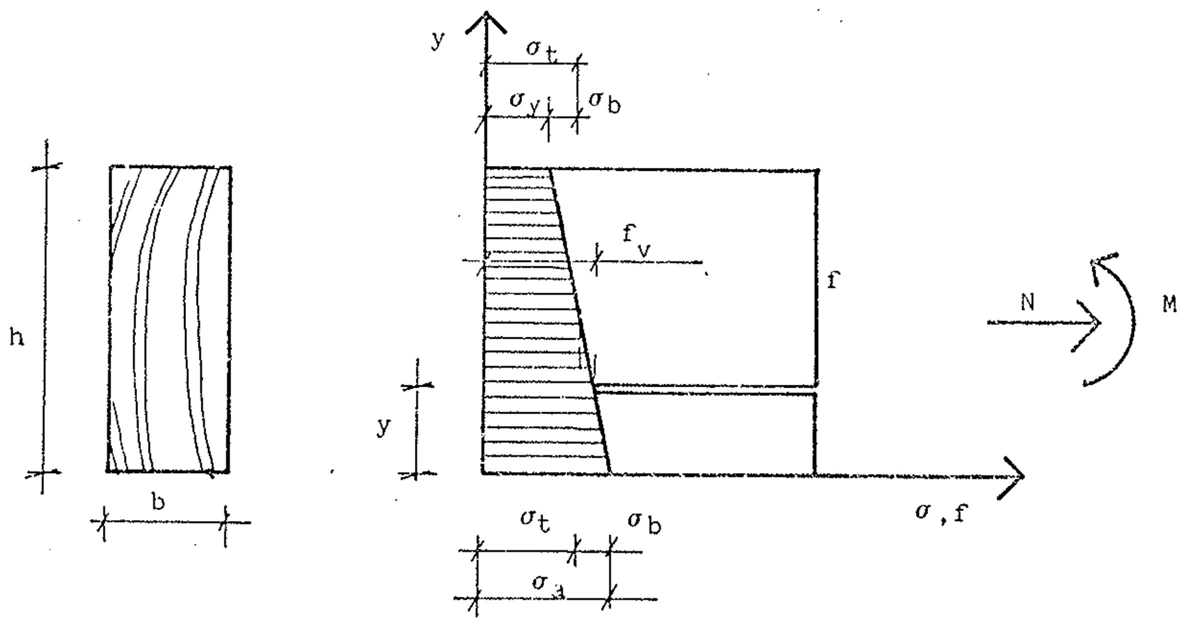


Figure 3, failure situation when cross section is under tension

It can be noted from figure 3 that f_v can be found out by loading the cross section with constant stress. This means that defect strength f_v is equal to timber tension strength f_t. The effect of the location of the defect can be found out by assuming that the probability for any location is equal. Assume that f_t has distribution function f'. Let us consider defect location i. The failure probability is made out of three probabilities:

Firstly, the location has stress σ_i while the mean stress is σ_t:

$$P_1 = \frac{\sigma_i}{\sigma_t} \tag{7}$$

Secondly, location i represents one out of n:

$$P_2 = \frac{1}{n} \tag{8}$$

Thirdly, probability for failure on stress level σ_i is:

$$P_3 = \int_0^{\sigma_i} f' \tag{9}$$

Probability for failure of the location i is:

$$P_i = p_1 \cdot p_2 \cdot p_3 \tag{10}$$

Probability for failure for the whole cross section is:

$$P = \sum_{i=1}^{n} P_i \tag{11}$$

To simplify the case we handle mean strength values and we may write:

$$P = \frac{1}{n} \sum_{i=1}^{n} \frac{\sigma_i}{\sigma_t} \cdot \int_0^{\sigma_i} f' = 0,5 \tag{12}$$

This equation is general and needs the distribution function and its parameters. To simplify the case we assume that deviation is 0 then:

$$\int_0^x f'(x) = 0\,; \qquad x \leq f_t \tag{13}$$

$$\int_0^x f'(x) = 1\,; \qquad x > f_t$$

In this case we will find out that f_t is the value that σ- figure has in its area centroid and f_t can be calculated, figure 3:

$$y \cdot \frac{\sigma_t + \sigma_b + f_t}{2} = \frac{h\,\sigma t}{2} \tag{14}$$

$$\frac{y}{h} = \frac{\sigma_t + \sigma_b - f_t}{2\,\sigma_b} \tag{15}$$

By eliminating y from 14,15 we get design formula:

$$\sigma_b \leq \sqrt{f_t^2 - \sigma_t^2} \tag{16}$$

Using other notation, figure 3:

$$\sigma_a \leq \sqrt{2f_t^2 - \sigma_y^2} \tag{17}$$

Writing the formula in another form:

$$\left(\frac{\sigma_t}{f_t}\right)^2 + \left(\frac{\sigma_b}{f_t}\right)^2 \leq 1 \tag{18}$$

By using notation from formula 1 and considering:

$$N_u = f_t \cdot b \cdot h \tag{19}$$

$$N = \sigma_t \cdot b \cdot h \tag{20}$$

$$M = \sigma_b \cdot \frac{1}{6} \cdot b \cdot h^2 \tag{21}$$

we get:

$$\left(\frac{N}{N_u}\right)^2 + \left(\frac{6M}{hN_u}\right)^2 \leq 1 \tag{22}$$

This formula 22 and also formulae 16,17,18 are new design formulae which correspond to formula 1 in case cross section is completely under tension.

Later in this study timber with equal tension and compression strength will be handled:

$$f_c = f_t = f_v \tag{23}$$

In this case it can be derived from the model that fictitious bending strength (rupture strength) meets the condition:

$$f_b = 1{,}34 \cdot f_t \tag{24}$$

Considering this, formulae 18 and 22 can be written:

$$\left(\frac{N}{N_u}\right)^2 + 1{,}80 \left(\frac{M}{M_u}\right)^2 \leq 1 \tag{25}$$

or:

$$\left(\frac{\sigma_t}{f_t}\right)^2 + 1{,}80 \left(\frac{\sigma_b}{f_b}\right)^2 \leq 1 \tag{26}$$

All these formulae are based on mean strength values ignoring deviation instead of the correct formula 12.

The error which results from ignoring deviation can be found out by making comparison to formula 12.
There has been available tension test material of 250 samples T24 grade Finnish soft wood. From the test tension strength f_t and the worst defect strength f_v have been found out $f_t = f_v$. Normal, log-normal and Weibull distribution has been fitted into the test material, table 2. Because formulae 16 -26 are only based on defect strength, test results may be used to find out the difference from formula 12. This comparison has been made with all combinations of distribution (normal, log-normal and Weibull) and all combinations of N and M. It was found out that the difference is less than 3 % in all cases. The difference is largest with the largest M which means the case $\sigma_y = 0$.
All this means that formulae 16-18, 22-26 can be considered to be practically the same as 12.
the whole cross section is under tension stress if:

$$\frac{N}{N_u} \geq 0{,}71 \tag{27}$$

This is also boundary condition for design state 1.
In this design state failure is brittle due to defect.

4.3. Cross section under elastic compression and tension, state 2

Boundary conditions for this state are: cross section is partly under tension and partly under compression and compression stress is less than compression strength, figure 4:

$$|\sigma_y| < f_c \tag{28}$$

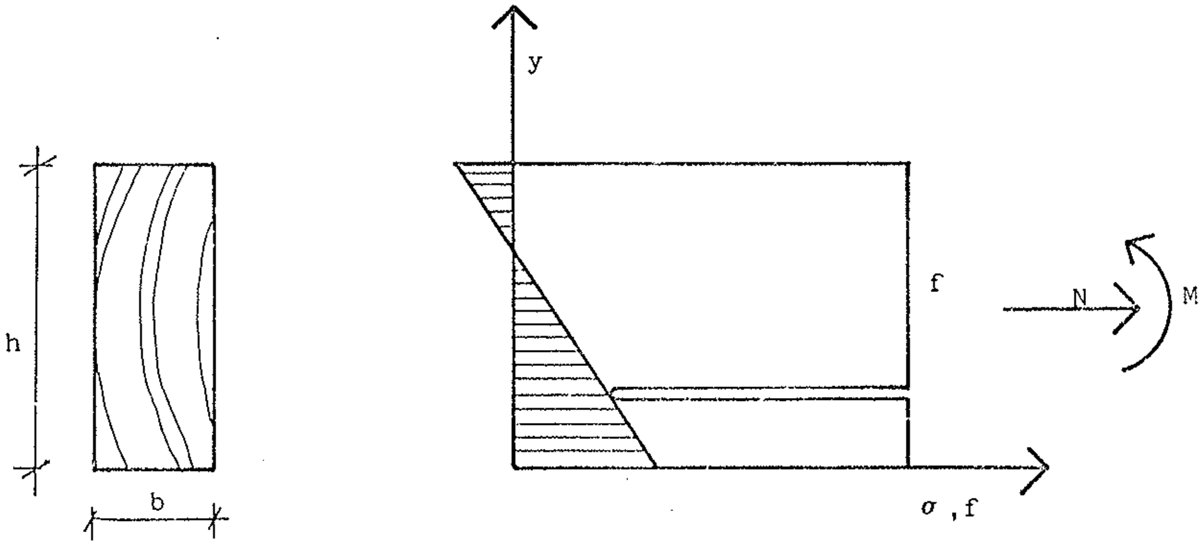


Figure 4, failure situation when cross section is under elastic compression and tension

Design formula can be derived from state 1, assuming that failure never occurs on compression side. Ignoring size effect state 2 is the same as state 1 when cross section is assumed to be only tension part of it.

By notation $\sigma_y = 0$ in formula 17, we get:

$$\sigma_a \leq \sqrt{2}\, f_t \tag{29}$$

This can be written also in the following form:

$$\frac{\sigma_t}{f_t} + \frac{\sigma_b}{f_t} \leq \sqrt{2} \tag{30}$$

or:

$$\frac{N}{N_u} + \frac{6M}{hN_u} \leq \sqrt{2} \tag{31}$$

In special case $f_c = f_t$ formulae 30,31 can be written:

$$0{,}71 \frac{\sigma_t}{f_t} + 0{,}95 \frac{\sigma_b}{f_b} \leq 1 \qquad (32)$$

or:

$$0{,}71 \frac{N}{N_u} + 0{,}95 \frac{M}{M_u} \leq 1 \qquad (33)$$

Boundary condition for state 2 is:

$$0{,}21 \leq \frac{N}{N_u} < 0{,}71 \qquad (34)$$

In design state 2 there is brittle failure on tension side of cross section.

4.4. Plastic cross section, brittle failure, state 3

If compression stress exceeds compression strength but ultimate compression strain is not exceeded the cross section is in design state 3, figure 5.

Boundary conditions are:

$$|\sigma'_y| \geq f_c \qquad (35)$$

$$|\sigma'_y| \leq nf_c \qquad (36)$$

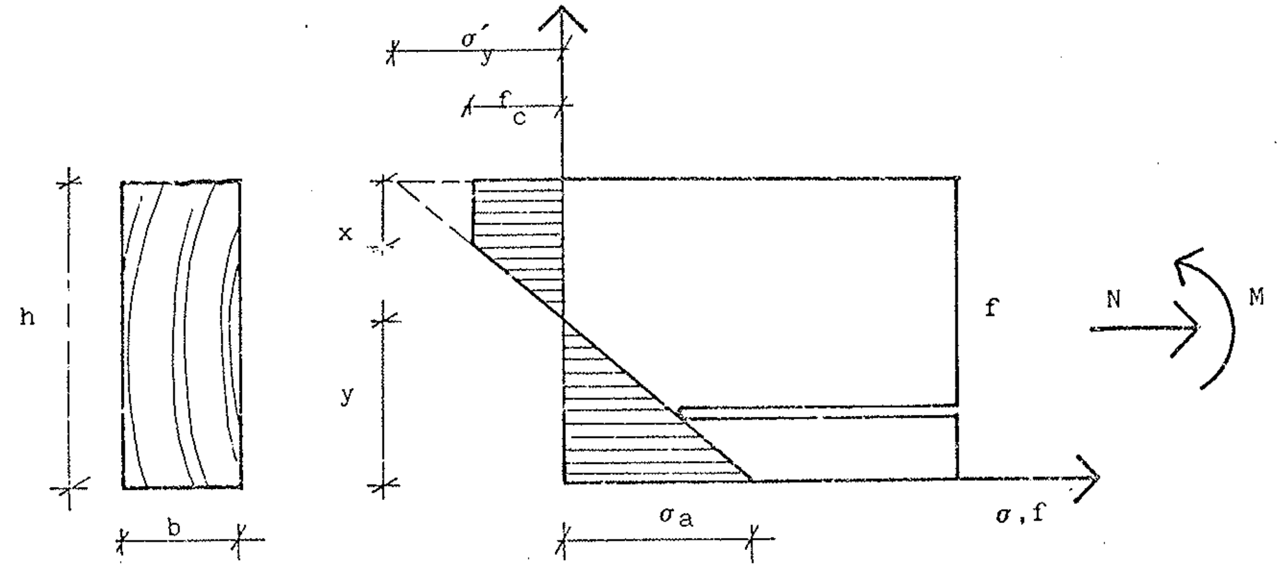


Figure 5, failure situation when cross section is under tension and plastic compression

Failure criterion is

$$\sigma_a \leq \sqrt{2}\, f_t \tag{37}$$

which can be derived in the same way as in design state 2.

In addition to formula 37 we need equilibrium conditions:

$$\int_A \sigma \, dA = N \tag{38}$$

$$\int_A \sigma (y-h/2) \, dA = M \tag{39}$$

and geometric conditions:

$$\frac{x}{h} = \frac{|\sigma'_y| - f_c}{\sigma_a + |\sigma'_y|} \tag{40}$$

$$\frac{y}{h} = \frac{\sigma_a}{\sigma_a + |\sigma'_y|} \tag{41}$$

When N and M are known x, y, σ_y', σ_a can be solved from equations 38 - 41.
It is not possible to solve these equations in closed form.
Therefore numerical solution is used.
In design state 3 failure occurs on tension side after plastic deformation on compression side.

4.5. Plastic cross section, ductile failure, state 4

Boundary conditions are:

$$|\sigma_y'| > f_c \tag{42}$$

$$\sigma_a < \sqrt{2}\, f_v \tag{43}$$

Design criterion is:

$$|\sigma_y'| \leq n\, f_c \tag{44}$$

Otherwise cross section is handled with equations 38 - 41.
Cross section has ductile failure on compression side.

4.6. N, M-interaction

According to the model cross section strength can be derived from f_v, f_c and n without knowing f_b. Results are not sensitive to variations of f_c. That is why assumption of $f_c = f_v$ can be made.
In the appendix N, M-interaction curve has been plotted.

Design states 1-4 have been shown with different lines. In state 4 curves for ultimate strain have been plotted with values $n = 2,3$, which correspond to appr. strain 1, 1,5 %.
Lines according to design formula 1 have also been plotted.
It can be found out that formula 1 is 15...20 % conservative compared to the model if $N, M \neq 0$.

4.7. Bending strength

Bending strength (modulus of rupture) of the model is not constant and needs an explanation. In design state 1 bending strength varies depending upon moment. In design states 2 and 3 bending strength (ultimate tension of cross section) has constant value $\sqrt{2} \cdot f_t$.
It has also been stated $f_b = 1{,}34\ f_t$. This equation is based on $f_t = f_c$ and f_b is fictitious bending strength calculated from linear stress-strain relationship, which is wrong according to the model.

5. Comparisons to test results

5.1. Introduction

According to the model bending strength can be calculated out of tension and compression strength. This property can be used to test the model when the same timber has been tested for tension, compression and bending strength. There are two sets of data of this kind, Finnish timber code and a test with Finnish timber.
Mr. Buchanan has derived a model for N, M-interaction. He has also done large number of tests. This can be used as a comparison.

5.2. Finnish timber code

Short term mean strength values of Finnish timber code /4,5/ in grades T24, T30 are shown in table 1

strength value	grade	
	T24	T30
f_c	28,7	32,6
f_t	28,9	31,5
f_b	38,0	41,9
f_c/f_t	0,99	1,03
f_b/f_c	1,32	1,28
f_b/f_t	1,31	1,33

Table 1, Appr. mean strength values of Finnish T24, T30 timber

It can be found out that tension and compression strengths can be considered to be the same because the difference is -1... +3 %. It is also found out that $f_b/f_{c,t}$ = 1,28...1,33 and on an average 1,31. This value is only 2 % smaller than the value it snould be according to the model 1,34. The difference is small and the model can be considered to fit into Finnish timber code in grades T24, T30.

5.3. Test with Finnish timber

300 tension and 250 bending tests have been made all with same kind of timber, cross section grade, humidity etc. / 5 /. Compression tests were not included, but tests made elsewhere show that $f_c \approx f_t$. So, the assumption $f_c = f_t$ is made.

Into the test material was fitted normal distribution:

$$f(x) = \frac{1}{\sqrt{2\pi}\,\alpha} \cdot \exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\alpha}\right)^2\right) \tag{45}$$

log-normal distribution:

$$f(x) = \frac{1}{x\,\beta\sqrt{2\pi}} \quad \exp\left(-\frac{1}{2}\left(\frac{\log(x)-\gamma}{\beta}\right)^2\right) \tag{46}$$

Weibull distribution:

$$f(x) = \frac{\beta}{\delta}\left(\frac{x-\mu}{\delta}\right)^{\beta-1} \exp\left(-\frac{x-\mu}{\delta}\right)^{\beta} \tag{47}$$

Results for tension tests are shown in table 2 and bending tests in table 3.

distribution	parameters	fractiles 1 %	5 %	50 %
normal	μ = 30,8	12,1	17,6	30,8
	α = 8,01			
Weibull	μ = 10,9	14,9	18,2	30,3
	δ = 22,3			
	β = 2,650			
log-normal	γ = 3,387	15,7	18,9	29,6
	δ = 0,272			

Table 2, tension strength values f_t (MPa)

distribution	parameters	fractiles 1 %	5 %	50 %	f_b/f_t
normal	$\mu = 40,0$ $\alpha = 9,42$	18,0	24,5	40,0	1,30
Weibull	$\mu = 16,2$ $\delta = 26,7$ $\beta = 2,6932$	21,1	25,1	39,5	1,30
log-normal	$\nu = 3,657$ $\beta = 0,241$	22,0	26,0	38,7	1,31

Table 3, bending strength values f_b (MPa)

In table 3, also ratio f_b/f_t is shown. It is found out that this is practically constant 1,30... 1,31 and only 3 % less than expected on the basis of the model 1,34. This means that the model fits into the test results. It also seems that the results of the model do not depend upon the assumed strength distribution.

5.4. Buchanan's tests

Information about Buchanan's tests was received just before writing this. Buchanan has derived a strength model for timber and done a large number of tests, appr. 4000. A part of the model and test results are shown in figures 54, 58 of Buchanan's thesis / 2 /. These figures have been copied into figure 6, adding the author's N, M-interaction with dots (n=2,5). Also test results from figures 54, 58 have been picked up (approximately) and plotted with circles in the figure of the appendix.

It can be found out that the author's and Buchanan's models give closely the same results though they are based on completely different theoretical background. The largest differences lie on the area $N/N_u = 0,7...1,0$. Relative exactness of the author's model is best on this area.

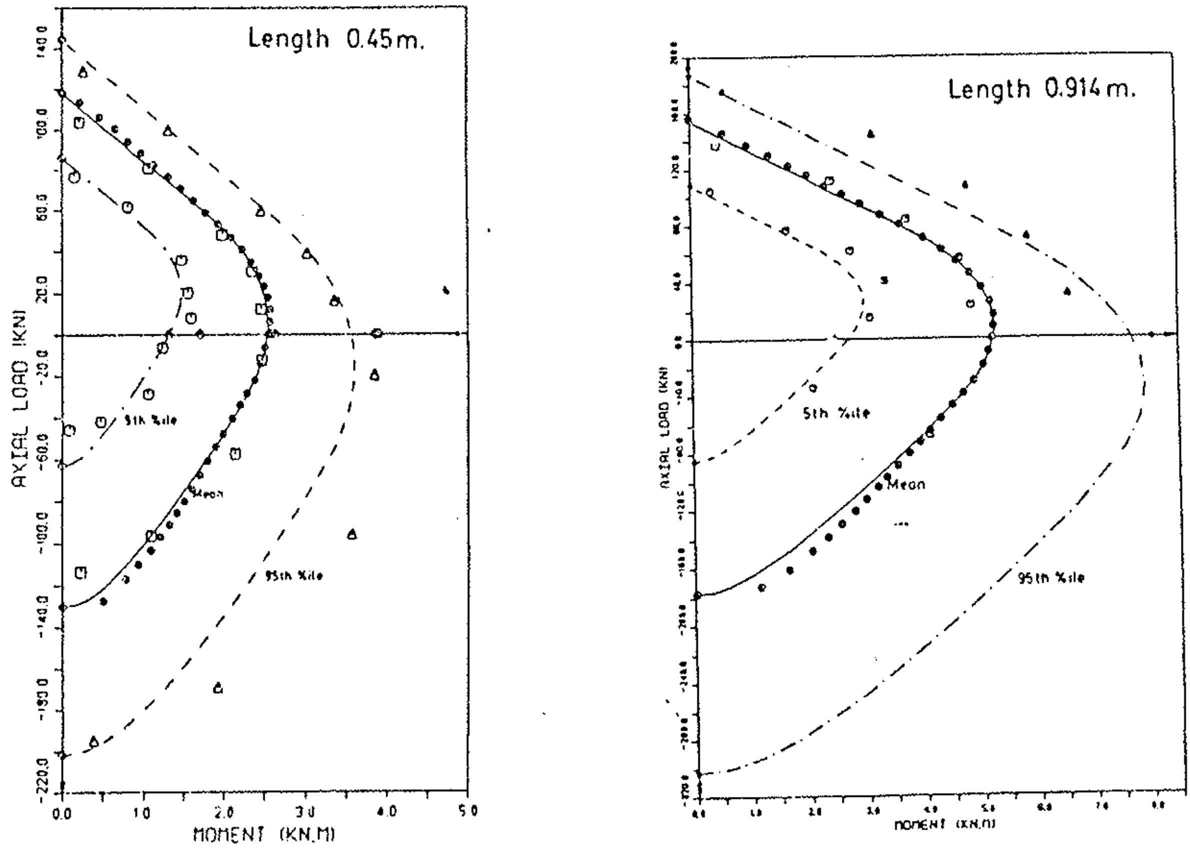


Figure 6 figures 54, 58 from / 2 / added with author´s model with dots n = 2,5.

6. Size effect

Failure criterion in design states 1-3 is completely based on brittle fracture mechanics and it can be compared to the failure of the weakest link of chain.

This principle gives an opportunity to explain the fact that in timber structures increasing size decreases ultimate stress.

Assume that defect has cumulative strength distribution function F. Consider timber size which has n defects. Then probability for no failure on stress level σ_1 is:

$$P_1 = \left(1-F(\sigma_1)\right)^n \tag{48}$$

If size is increased k times defects increase r times. If we examine beam length effect on tension strength we may assume k=r.

New size ~~fails~~ remains unfailed with probability:

$$P_2 = \left(1-F(\sigma_2)\right)^{kn} \tag{49}$$

on stress level σ_2.

We must claim that all sizes fail with equal probability, so $P_2 = P_1$, which also means $\sigma_2 < \sigma_1$. This also means that larger size resist lower stress and it is weaker.

Size effect can be defined as:

$$K = \frac{\sigma_2}{\sigma_1} \tag{50}$$

Writing equations 48, 49 equal we get:

$$1-F\ (\sigma_1) = \left(1-F(\sigma_2)\right)^k \tag{51}$$

Out of this equation size effect can be calculated in all confidence levels. Some numerical calculations made with 50 % and 5 % confidence levels have shown that confidence level has very little effect on results. Thus the simplest confidence level 50 % is used. Assuming F is cumulative distribution function of normal distribution it is found out that the mean value has no effect on results and we select 1 as the mean value and v as the coefficient of variation.

Then we may write:

$$1-F(\sigma_1) = 0,5 = \left(1-F(\sigma_2)\right)^k \tag{52}$$

Using size 1 as reference ($\sigma_1 = 1$) we get:

$$K = F^{-1}(1-0,5^{1/k}) \tag{53}$$

This is an equation which gives the reduction of mean strength value when defects (size) increase k times. In this case F is cumulative distribution function of normal distribution having 1 as mean value and having coefficient of variation same as variation of defect.
In literature size effect is often given as:

$$K = \left(\frac{s_1}{s_2}\right)^p \tag{54}$$

where s_1, s_2 represent size and p is constant. To length effect there has been given different values for p such as: 0,192, 0,167 and 0,25 / 2,3 /. Assume that F is cumulative distribution function of normal distribution, we find out that equations 54 and 53 give very closely the same results if different values of p in equation 54 correspond to different coefficient of variation in equation 53.
If p has values 0,192, 0,167 and 0,25 the needed coefficient of variation is appr. 23 %, 20 % and 29 %.
Literature has not reported variations, but calculated values look reasonable. Nevertheless, it is obvious that increasing variance increases size effect as shown by equation 53.

If we apply this same principle for timber beam with constant span depth ratio subjected to moment load, we find out: when size increases k times we may assume number of defects increase k^2 times because defects increase k times in length and depth direction. In this case equation 53 can be used when k is replaced by k^2. This is not self-evident because moment loaded beam has defects on different stress levels. This means that equations 48, 49 must be divided into different stress levels which also means different confidence levels. Anyhow, it was found out that change of confidence level does not change result and one stress level may be used. In this case literature shows that equation 54 can be used. Fewell / 3 / gives the constant p value 0,4. In this case equations 54 and 53 give closely the same results if coefficient of variation is appr. 24 %. Calculations made with equations 50, 51 on confidence level 5 % give the same result, appr. 24 %. This is a good approximation for the coefficient of variation of Fewell's tests.

7. Conclusions

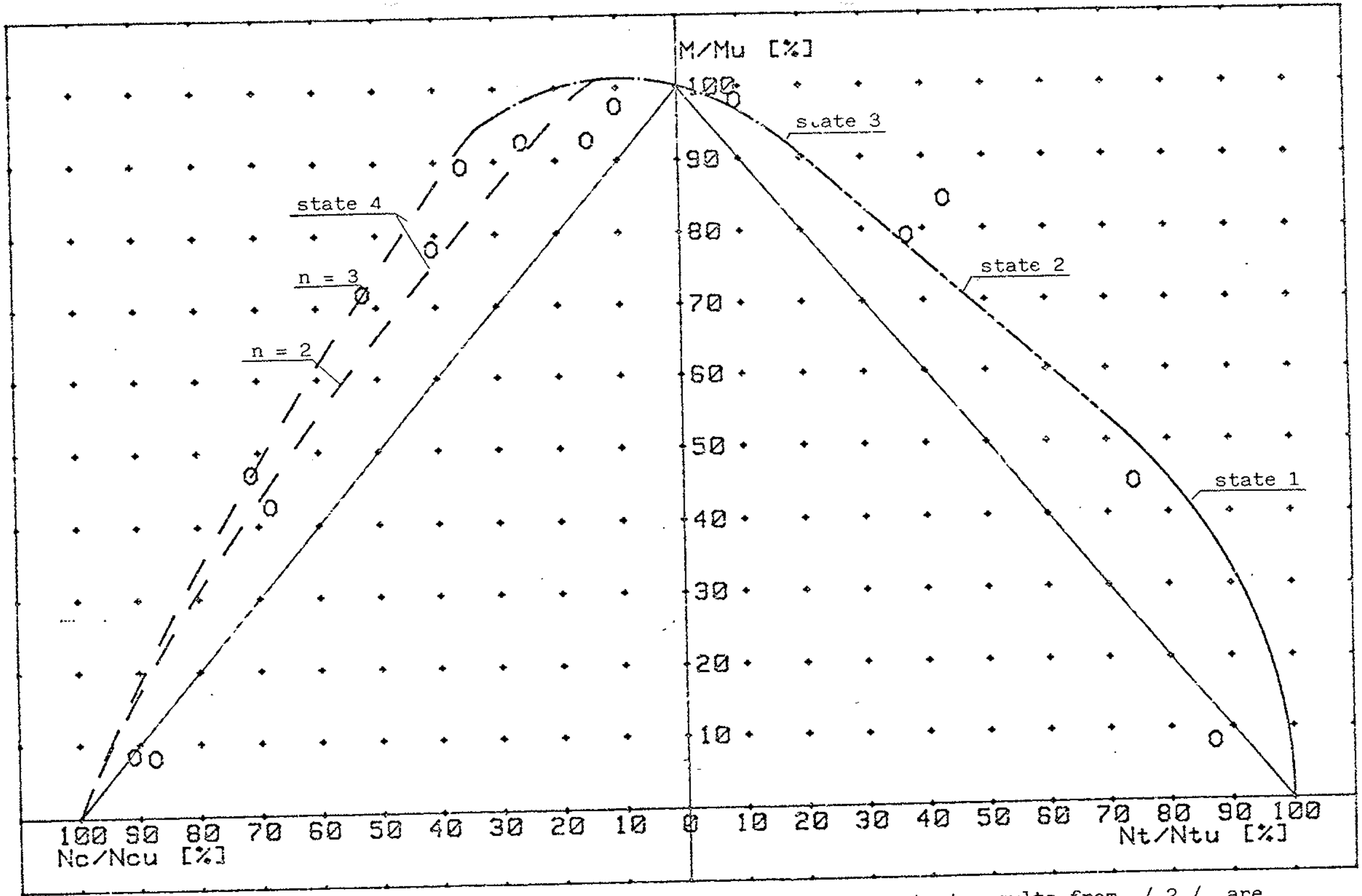
1. According to the model design formula 1 is 15... 20 % conservative when $N, M \neq 0$.
2. Results can be derived from basic assumptions without any calibration to test results.
3. According to the model bending strength can be calculated from compression and tension strength. This calculation fits well with Finnish timber code and a test of 550 samples / 4,5 /.
4. The N, M-interaction curve of the model is closely the same as that derived by Buchanan / 2 / from completely different basic assumptions.
5. It is possible to derive equation for size effect from the model. This equation means that size effect can be practically completely explained by coefficient of variation of defect. Test information on size effect is limited but the model seems to fit into available tests.
6. The model defines clearly cross section deformations. This gives an opportunity to unlinear analysis of timber structures.

Notation

E	modulus of elasticity
K	size factor
N	force
M	moment
	subscript u ultimate
F	cumulative distribution function
b	width of cross section
f	strength
	subscript b bending
	c compression
	r defect free
	t tension
	v defect
f'	distribution function
h	depth of cross section
n	factor for ultimate compression strain
t,p,r	constants
s	size
v	coefficient of variation
x	depth of plastic zone of cross section
y	location of neutral axis or defect of cross section
	stress, subscripts as f
	y upper edge
	a lower edge

Literature cited

1. Andersson J.A., Stess-Strain Relationship for Defect-Free Timber Beams, Wood Science, Vol.14, No.1, 1981
2. Buchanan H.B., Strength model and design methods for bending and axial load interaction in timber members, The University of British Columbia, 1984
3. Fewell A.R., Size factors for timber bending and tension stresses, CIB-W18/16-6-1, Norway, 1983
4. Finnish timber code, RIL 120, 1983
5. Saarelainen U., Unpublished data on tests with Finnish timber, 1984



Interaction diagram of the model, lines presenting formula 1 are shown, test results from / 2 / are shown with circles appr.

CIB-W18/17-4-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DETERMINATION OF PANEL SHEAR STRENGTH AND PANEL SHEAR MODULUS OF BEECH-PLYWOOD IN STRUCTURAL SIZES

by

J Ehlbeck and F Colling

University of Karlsruhe
Federal Republic of Germany

RAPPERSWIL

SWITZERLAND

MAY 1984

Determination of Panel Shear Strength and Panel Shear Modulus of Beech Plywood in Structural Sizes

J. Ehlbeck and F. Colling

Introduction

High quality beech plywood is a usable material in timber engineering, e.g. for reinforcing glulam structures under perpendicular-to-grain tensile stresses or for plywood gussets in nailed or glued trusses. Beech plywood may also be used as a web of I- or box-beams or in similar structural applications.

Therefore, it is necessary - among others - to make available data of the panel shear strength and the panel shear modulus. In a study made in the "Versuchsanstalt für Stahl, Holz und Steine" of the University of Karlsruhe (FRG) such data were collected using shear tests with beech plywood panels of 10 to 40 mm panel thickness. This task gave the opportunity of using the RILEM-recommended test method developed for testing plywood in structural sizes. This RILEM-Recommendation /1/ was approved by CIB-W18 and forwarded to ISO as a basis for a draft ISO Standard. In a joint committee of ISO/TC 139 and 165 this recommendation was discussed. A draft proposal of this ISO Joint Committee will be under discussion at the main meetings of the technical committees TC 139 and TC 165. It was considered useful to perfom tests using high strength plywood panels at this stage in order to propose additional or modified test methods if necessary.

./
/.

Test Specimen

The test specimen and test set-up described in the RILEM-Recommendation are shown in Figs. 1 and 2. A method of measuring the shear deformation is given in ASTM Standard D 2719-76 /2/ and will be proposed also in the ISO draft. This principle is given in Fig. 3. When using this loading apparatus, the external loads act to the test specimen in a manner shown in Fig. 4.

The panel shear strength is

$$f_p = \frac{F_{t,u}}{L \times t} \qquad (1)$$

with $F_{t,u}$ = ultimate applied tensile load of the testing machine; L = length of panel; t = panel thickness.

In former investigations MÖHLER and EHLBECK /3/ used small test specimens and found panel shear strengths for beech plywood in the range of 18 to 20 N/mm² (Mpa). According to that, ultimate tensile loads in the range of 430 to 480 kN had to be expected for test specimens of 600 mm length and 40 mm panel thickness. At the same time shear stresses will occur in the timber rails of 115 x 35 mm cross-section. These stresses amount to

$$f = \frac{H}{2 \times 35 \times 115} \qquad (2)$$

with $H \approx F_t/4$ (see Fig. 4). With $F_{t,u}$ = 480 kN, shear stresses in the rails will figure up to about 15 N/mm² (Mpa) with the failure of the test specimen certainly to occur in the rails instead of shear failure in the plywood panel. These considerations made sure that the proposed timber rails of the RILEM-Recommendation are not practicable for high quality hardwood plywood of a more than 20 mm - panel - thickness. Moreover, it seemed to be too time-consuming to use steel rails instead of timber rails.

So, the timber rails were replaced by beech plywood rails of the same thickness as the test panel thickness. With a rail width of 150 mm instead of 115 mm the shear stress in the rails amounts only to half the shear stress in the test panel:

$$f = \frac{H}{2 \times t \times 150} = \frac{f_p \times t \times 600}{4 \times 2 \times t \times 150} = 0,5 \times f_p \qquad (3)$$

With the plywood rails taken from the same panel under scrutiny, it could be expected that a failure in the rails would never occur. From these considerations the test specimens were modified as shown in Fig. 5. In no case of altogether 72 tests failure occurred in the rails.

Test Set-up

Ultimate test loads up to 500 kN require heavy and unwieldy test equipment when the test set-up according to Fig. 2 is used. The steel bars and pins and yokes as well as the steel link connected to the crosshead of the testing machine are only used to transform the tensile loads of the testing machine to compressive loads acting on the test specimen. Why not imposing compression loads of the testing machine directly to the test specimen?

A stationary test equipment was developed (see Fig. 6). It proved to be time-saving, because only the test specimen had to be moved into or out of the testing apparatus. The test load F, as shown in Fig. 4, acts directly on the test specimen. It has to be taken into account that the panel shear strength in this case amounts to

$$f_p = \frac{F_u \cdot \cos 14°}{L \times t} \qquad (4)$$

with F_u = ultimate compressive load of the testing machine.

Test Procedure

A constant rate of loading was chosen so that the ultimate load was reached within 3 ± 1 minutes. The load-deformation curves were recorded. A typical graph is given in Fig. 7. The shear modulus can be calculated from the straigth line portion of the load-deformation-curve. The elastic range extends to about 40 % of the ultimate load.

Test Results

Shear failure occured always along the weakest cross-section of the test panel. The shear strength achieved was not influenced by the location of the failure section. The coefficient of variation of the panel shear strength was rather low, ranging from 9 to 10 %. Compared with shear strength values obtained from small specimen, a reduced shear strength of about 80 % must be taken into consideration for plywood panels in structural sizes.

./.

The mean panel shear strength of the beech plywood under scrutiny amounts to 11,5 N/mm²; the shear modulus ranges from 700 to 800 N/mm². A detailed research report is in preparation /4/.

Conclusions

The RILEM-Rēcommendations for testing the panel shear strength of plywood in structural sizes may be usable for softwood plywood. If the panel shear strength increases, however, e.g. for hardwood plywood, timber rails in the dimension proposed by the RILEM-Recommendations turn out to be the weakest part of the test specimen. In such cases the rails should be of the same material and of such a cross-section that the shear stresses in the rails do not cause early failure of the test specimen.

The test equipment described in the RILEM-Recommendations becomes heavy and unwieldy when high tensile loads have to be applied. A simple stationary test set-up, easy to handle and of big advantage when testing many replications, proved to be a useful alternative with compressive loads of the testing machine directly applied to the test specimen.

References

/1/ International Union of Testing and Research Laboratories for Research and Testing (RILEM): Testing Methods for Plywood in Structural Grades for Use in Load-Bearing Structures. RILEM-Recommendation TT2, 1st Edition, Paris, 1976.

/2/ American Society for Testing and Materials (ASTM): Standard Methods of Testing. Plywood in Shear Through-The-Thickness. ANSI/ASTM Standard D 2719-76. Philadelphia, Penn., USA, 1976.

/3/ Möhler, K. und J. Ehlbeck: Kurzzeit- und Dauerstandversuche zur Ermittlung der statischen und Dauerstandfestigkeit von Bau-Furnierplatten. Berichte aus der Bauforschung, Heft 92, Holzbau-Versuche V. Teil. W. Ernst u. Sohn, Berlin 1974.

/4/ Ehlbeck, J. und F. Colling: Ermittlung der Scherfestigkeit und des Schubmoduls von Buchenfurnierplatten rechtwinklig zur Plattenebene. HOLZ als Roh- und Werkstoff 42 (1984); in preparation.

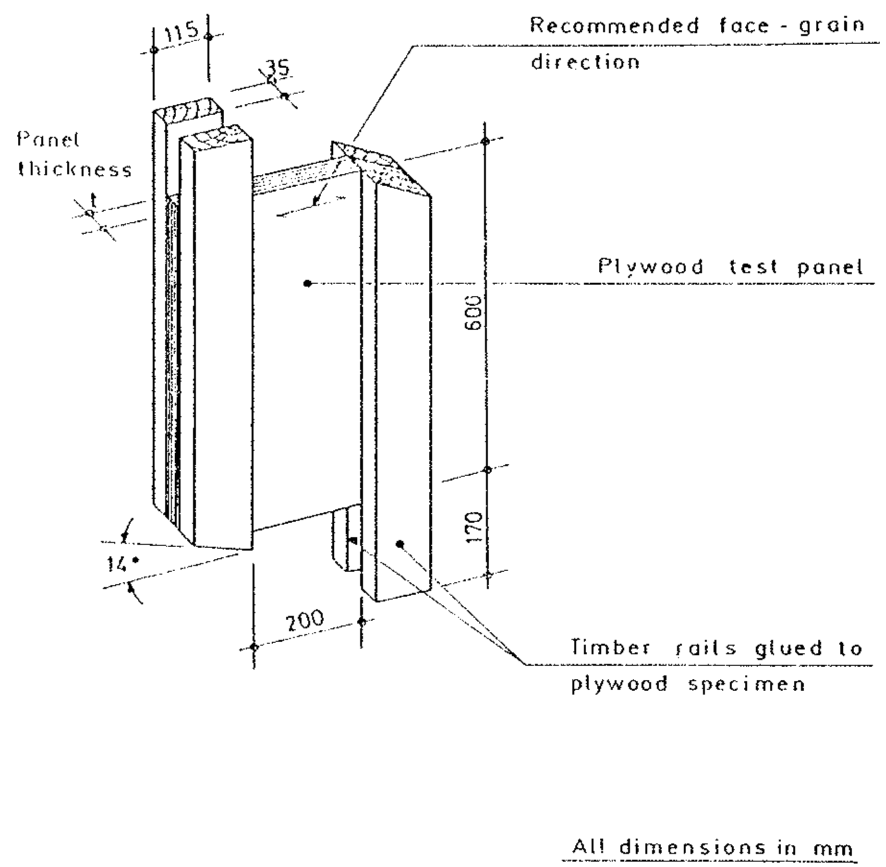


Fig. 1: Test Specimen According to RILEM - Recommendation and ISO - Proposal 1983 (Joint Committee TC 139/165)

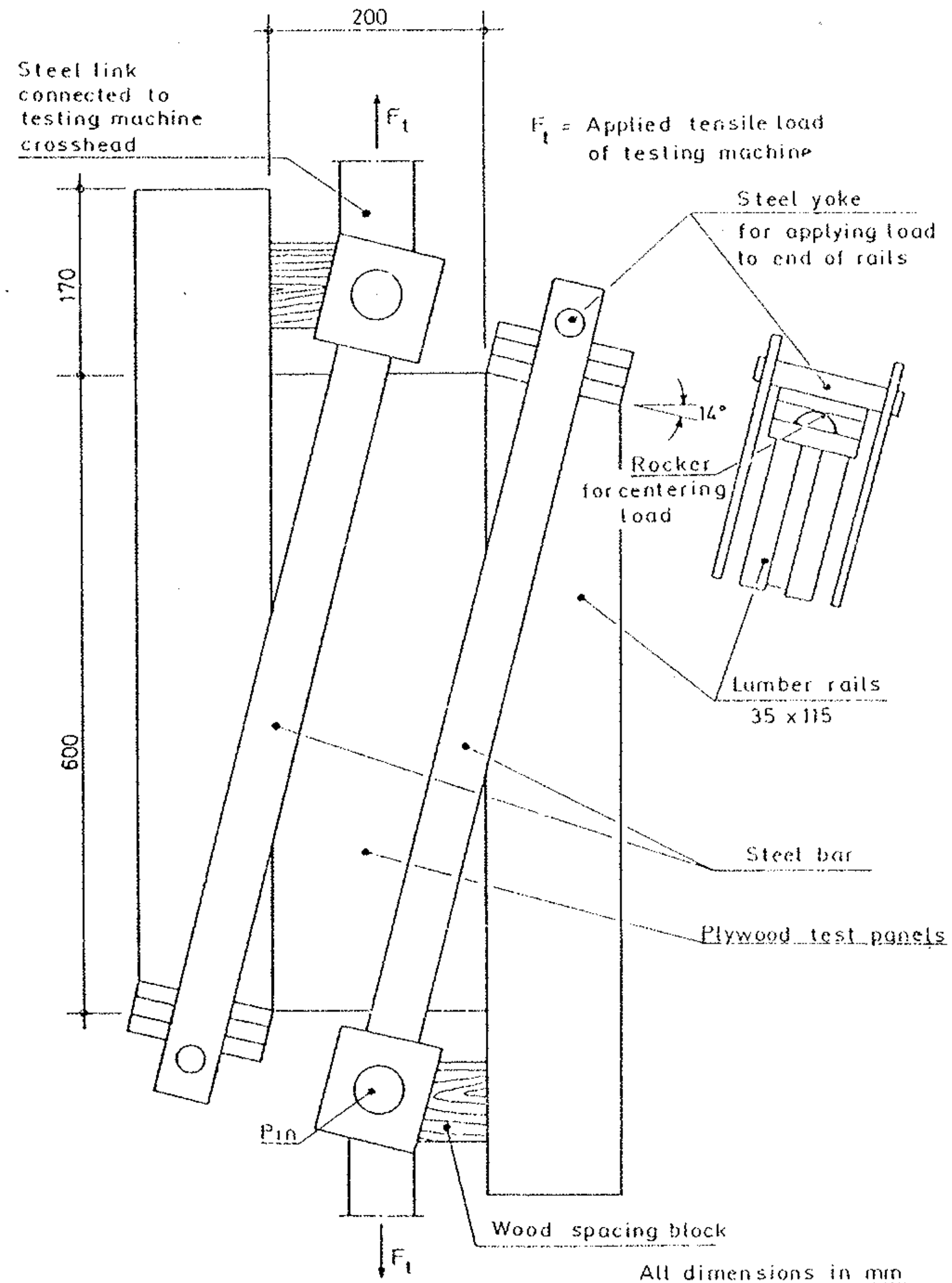
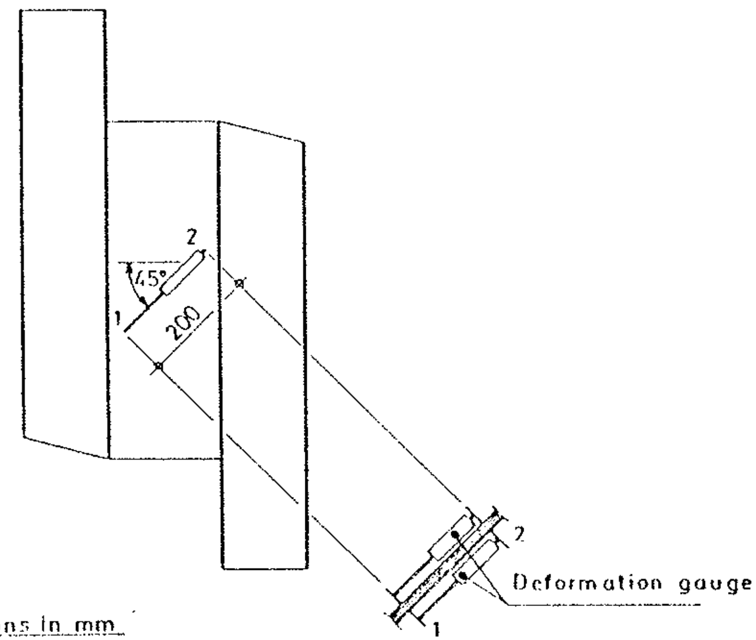
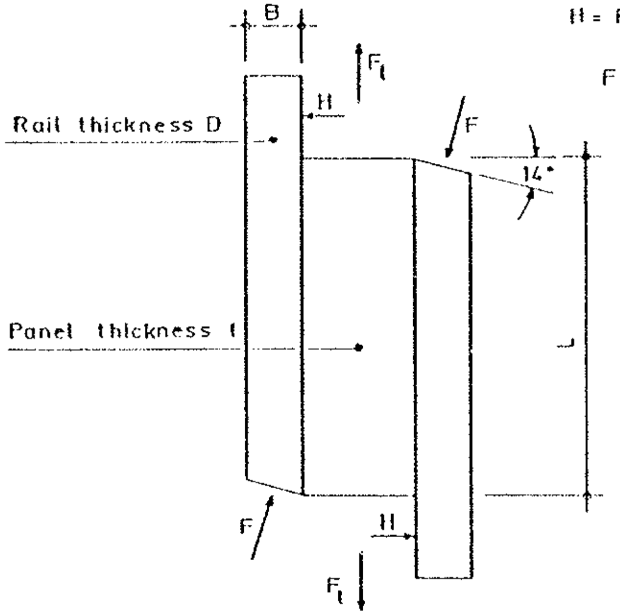


Fig. 2: Test Set - up According to RILEM - Recommendation and ISO - Proposal 1983 (Joint Committee TC 139/165)



All dimensions in mm

Fig. 3: Measuring Device for Determination of Shear Modulus



$F_t \hat{=}$ Applied tensile load

$H = F \times \tan 14° \approx \frac{F_t}{4}$

$F = \frac{F_t}{\cos 14°} = 1.03 \times F_t$

Fig. 4: Loads Acting to Test Specimen

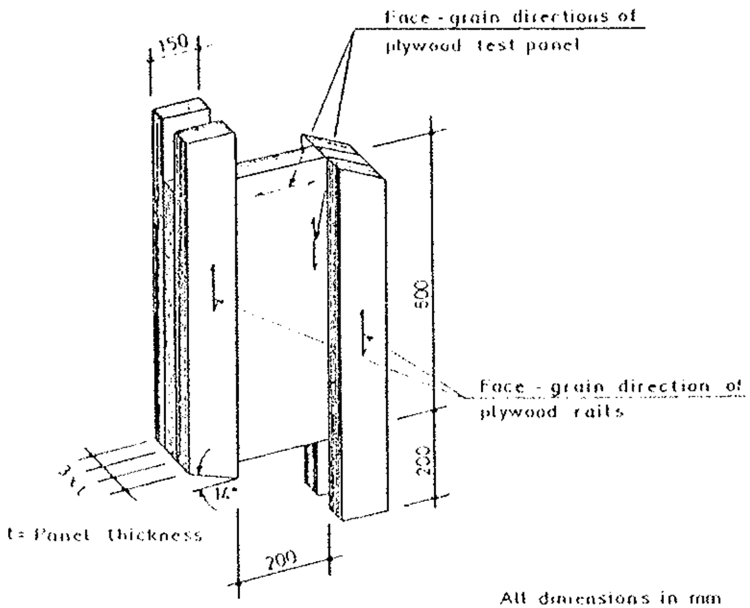


Fig 5 : Modified Test Specimen with Plywood Rails of Same Thickness as Test Panel

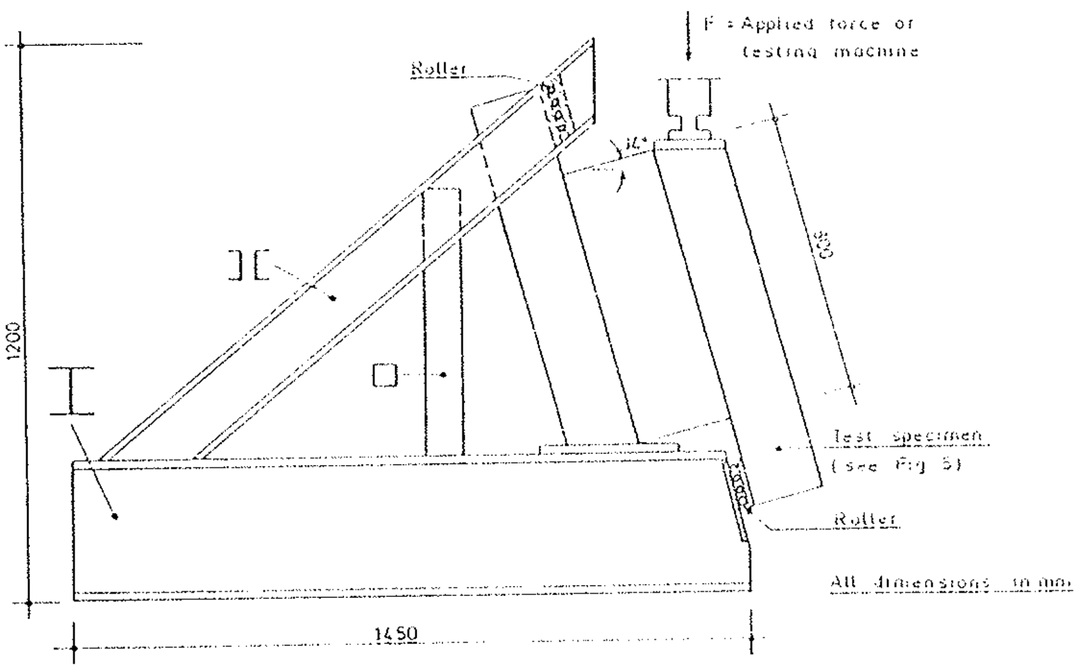


Fig. 6 Test Set-up Using Compressive Applied Loads

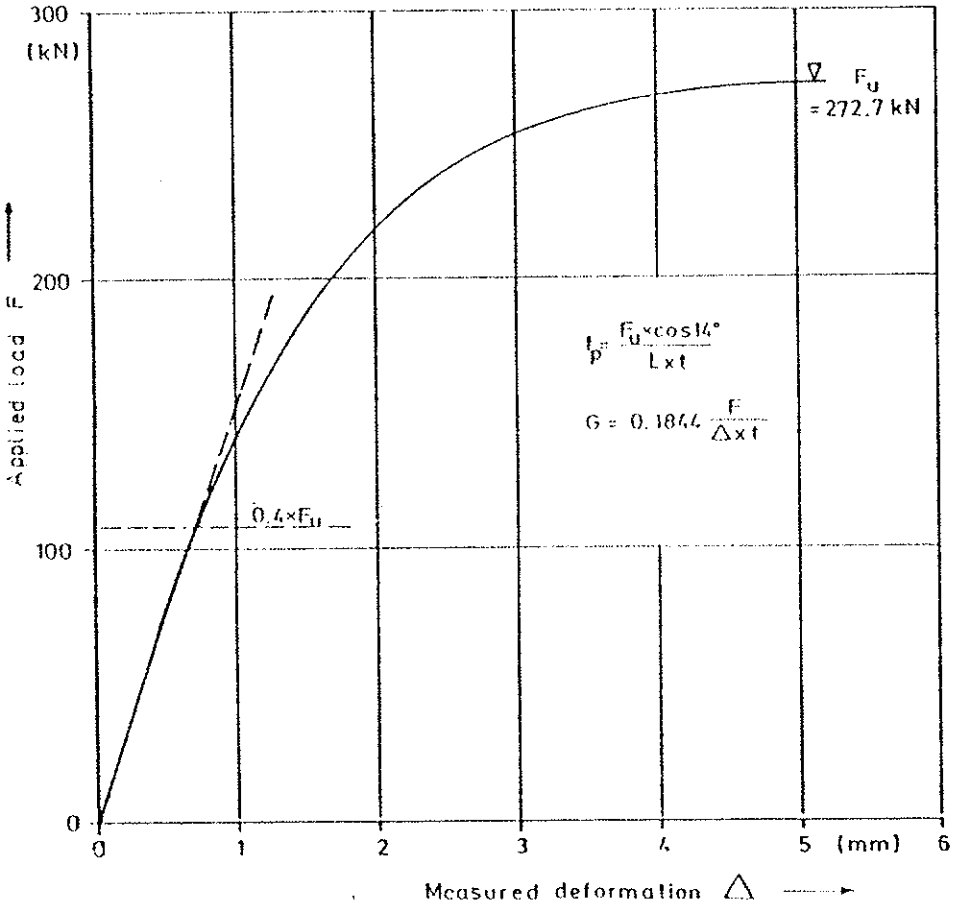


Fig. 7: Load-Deformation-Curve of a Test Specimen which Failed within the Measuring Range, Panel Thickness 40 mm

CIB-W18/17-4-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ULTIMATE STRENGTH OF PLYWOOD WEBS

by

R H Leicester and L Pham

Division of Building Research, Csiro

Australia

RAPPERSWIL

SWITZERLAND

MAY 1984

RHL 16/3/84

ULTIMATE STRENGTH OF PLYWOOD WEBS

BY

R.H. Leicester, Ph.D.*

L Pham, Ph.D.*

PACIFIC TIMBER ENGINEERING CONFERENCE, AUCKLAND, NEW ZEALAND, MAY 1984.

ABSTRACT

Methods are given for computing the strengths of both stiffened and unstiffened plywood webs of glued I-beams. These methods, verified by experiments with radiata pine plywood webs, indicate that the ultimate strength of plywood webs can be considerably greater than the critical elastic load.

1. INTRODUCTION

The strength of plywood webs in I-beams and box beams is rarely a critical design consideration. Indeed, unless glued edge-joints are used, the strength capacity of the web is usually determined by the strength of the fastener system. Nevertheless, in the interests of ensuring adequate reliability, it is useful to have a method for evaluating the ultimate strength of plywood webs.

In the following, the strength of the plywood web of an I-beam will be discussed. The notation to be used is shown in Figure 1. Only webs with the plywood face grain running parallel to, or perpendicular to the beam axis will be considered. The two specific problems investigated are the buckling strength of an unstiffened web supporting a concentrated flange load, and the ultimate shear capacity of a stiffened plywood panel.

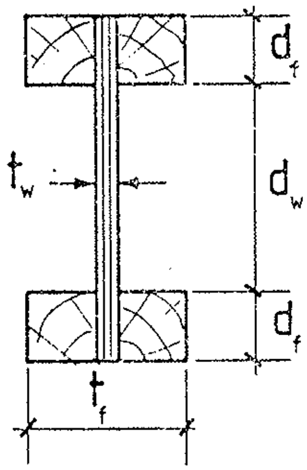


Figure 1 Notation for I-beam

Throughout the discussion, the slenderness of the plywood web will be described by means of a parameter λ which will be defined by an equation of the type

$$\lambda = (\text{squash load/elastic critical load})^{0.5} \qquad (1)$$

---

* CSIRO Division of Building Research

2. NOTATION

2.1 Parameters

a = dimension in the x-direction

B = bearing length of concentrated flange load, Figure 2

d_f, d_w = depth of flange and web, Figure 1

D_x = $\int_{-\delta}^{\delta} (E_x/\gamma)\, z^2 dz$, bending stiffness of plywood in the x-direction

D_y = $\int_{-\delta}^{\delta} (E_y/\gamma)\, z^2 dz$, bending stiffness of plywood in the y-direction

D_{xy} = $G_{LT}\, t_w^3/6$, torsional stiffness of plywood

E = Youngs modulus of elasticity for solid wood

f = strength in terms of stress

f_c, f_s = compression and shear strength

G_{LT} = shear modulus of solid wood

P = concentrated flange load, Figure 2

t_f, t_w = thickness of flange and web, Figure 1

V = shear strength

V_f, V_w = shear strength contributions of flange and web

x, y = cartesian coordinates in the plane of the plywood web, Figures 2 and 6

z = coordinate normal to the plywood web

α = $E_L\, \mu_{TL}\, t_w^3/(12\, \gamma)$

β = $(D_{xy} + \alpha)/(D_x\, D_y)^{0.5}$

γ = $1 - \mu_{TL}\, \mu_{LT}$

δ = $t_w/2$

Δ_z = lateral deformation of the web

θ = angle of face grain relative to the x-axis

λ = slenderness parameter, equation (1)

μ = Poisson's ratio

σ_t = direct tension stress

ϕ = angle of diagonal tension stress relative to the x-axis

Ω = proportional area of plies lying in the direction of the applied compression stress

2.2 Subscripts

c = compression

crit = critical elastic buckling value

f = flange

L = value along the longitudinal direction

s = shear

sc = value measured with small clear specimens of solid wood

sq = ultimate or 'squash' value for stable elemtns

t = tension

T = value transverse to the longitudinal direction

ult = value at ultimate load

x, y = values along x and y directions

3. CONCENTRATED LOADS ON FLANGES

3.1 Structural Geometry

The three configurations to be considered are illustrated in Figure 2. These configurations concern a concentrated load of magnitude P, distributed along a length B of the flange. The plywood web in the vicinity of this load is unstiffened.

3.2 Squash Load

The squash stress $f_{c,sq}$ ~~or~~ of a plywood element in compression is given by

$$f_{c,sq} = \Omega f_{c,sc} \tag{2}$$

where $f_{c,sc}$ denotes the compression strength of small clear specimens of solid wood, and Ω denotes the proportional area of plies lying in the direction of the applied stress.

Because the flanges tend to spread the action of the applied load, the effective bearing length of the concentrated load, denoted by B_{eff}, is greater than B. From a limited set of tests it was found that the squash load P_{sq} could be given by

$$P_{sq} = B_{eff} t_w f_{c,sq} \tag{3}$$

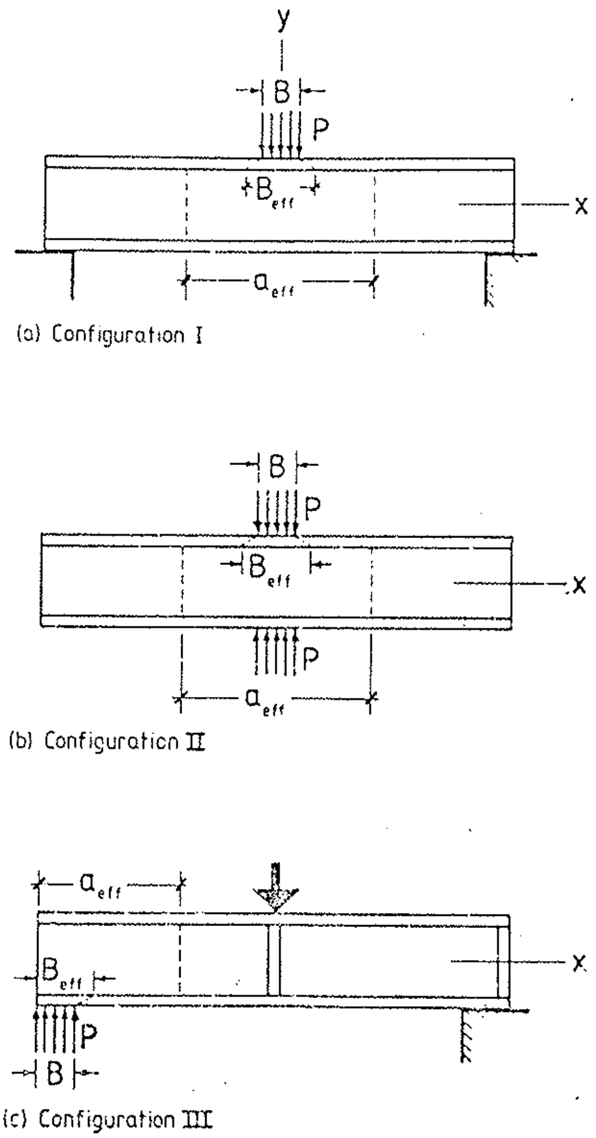


Figure 2 Configurations for concentrated loadings on flanges

where

$$B_{eff} = B + 3.0\ d_f \tag{4a}$$

for configurations I and II, and

$$B_{eff} = B + 0.5\ d_f \tag{4b}$$

for configuration III.

3.3 The Elastic Critical Load

The theoretical elastic critical load P_{crit} was computed by the method of Alfutov and Balabukh [1,2] for several typical cases, and fitted to the following empirical formula

$$P_{crit} = \Pi^2\ a_{eff}\ D_y/d_w^2 \tag{5}$$

where

$$a_{eff} = 2.0\ B_{eff} + 2.5\ d_w\ (D_x/D_y)^{0.25} \tag{6a}$$

for configuration I,

$$a_{eff} = 1.0\ B_{eff} + 1.0\ d_w\ (D_x/D_y)^{0.25} \tag{6b}$$

for configuration II, and

$$a_{eff} = 3.0\ B_{eff} + 0.5\ d_w\ (D_x/D_y)^{0.25} \tag{6c}$$

for configuration III.

In the computations it was assumed that there was a fixed condition between the web and the flange. Comparisons between the empirical equations and the exact theoretical solutions are shown in Figure 3.

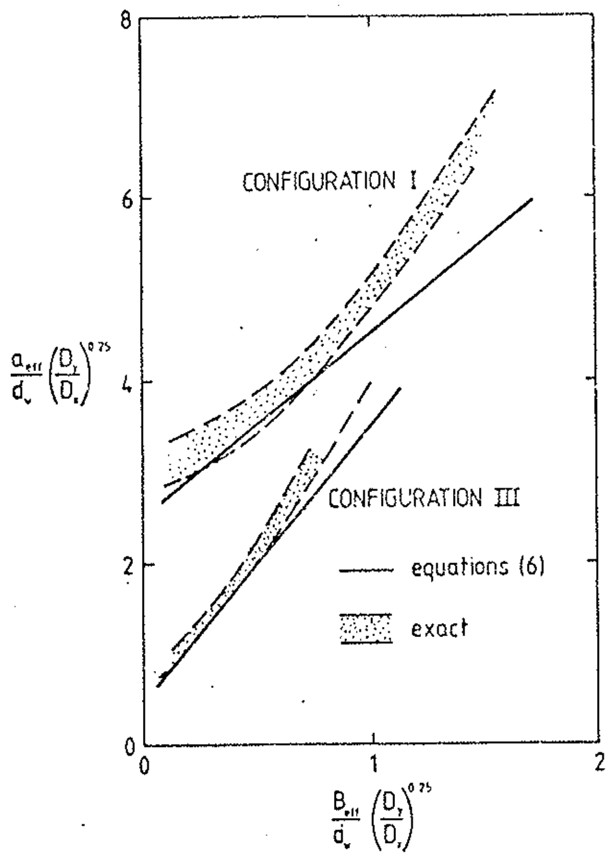


Figure 3 Effectiveness of the approximation of equations (6)

3.4 Ultimate Load

No satisfactory theory to predict the ultimate load P_{ult} was obtained. However, a conservative fit to experimental data was found to be given by the following equation:

$$P_{ult}/P_{sq} = 1.0 - 0.25\ \lambda \tag{7}$$

in which the slenderness parameter λ is defined by

$$\lambda = (P_{sq}/P_{crit})^{0.5} \tag{8}$$

3.5 Experimental Data

Load tests were made on some 50 I-beams fabricated with 3-ply and 7-ply radiata pine plywood webs, and 90 x 40 mm deep flanges of solid radiata pine.

The critical elastic load, measured by means of a Southwell plot, occurred at a lateral deformation $\Delta_{z,crit} \cong t_w$ and is in reasonable agreement with theoretically computed values as shown in Figure 4. In Figure 5, the measured ultimate loads are compared with the predictions of equation (7). In defining the ultimate load a limitation $\Delta_{z,ult} \leq 0.05\ d_w$ was placed on the lateral deformation.

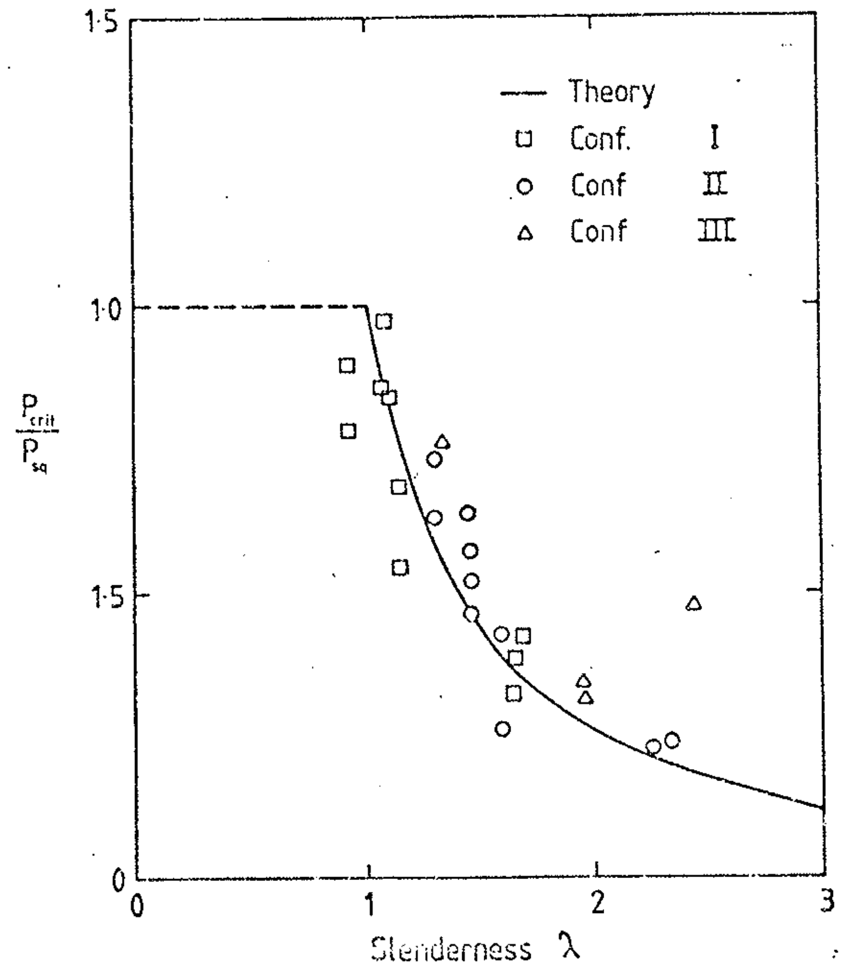


Figure 4 Critical elastic loads for unstiffened plywood webs

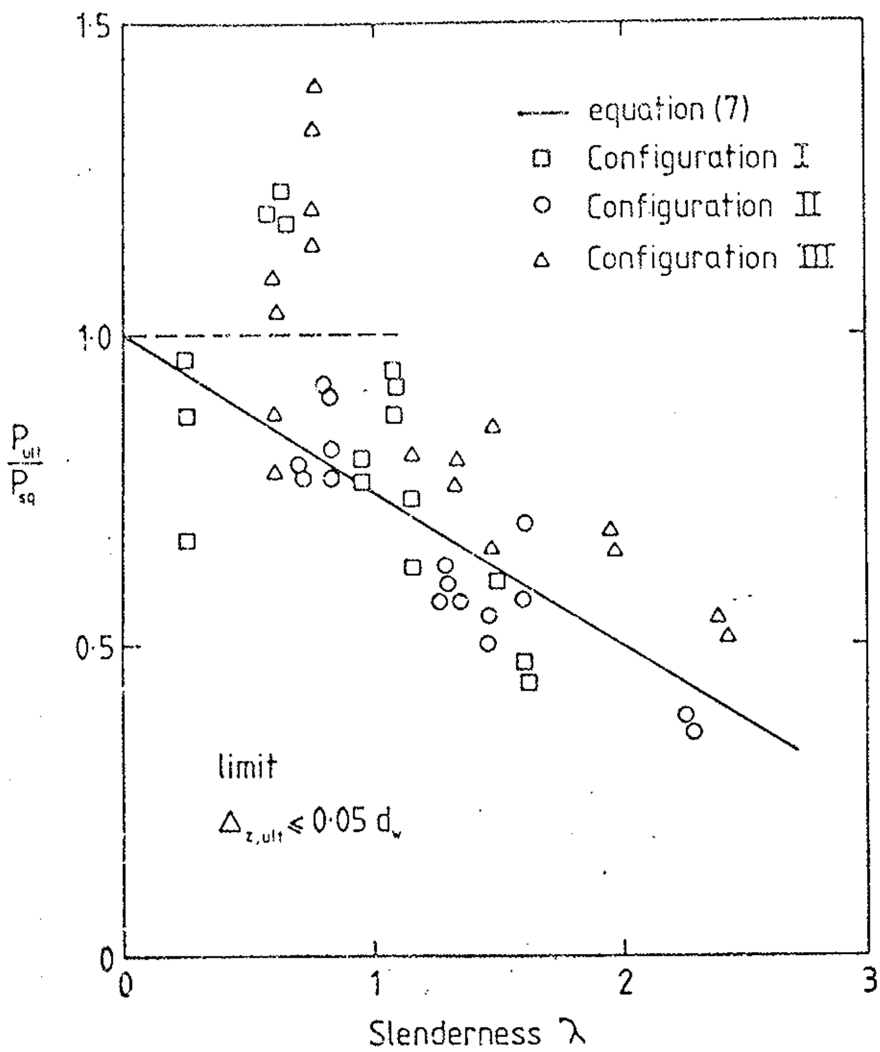


Figure 5 Ultimate loads for unstiffened plywood webs

4. SHEAR STRENGTH OF A STIFFENED WEB

4.1 Structural Geometry

The stiffened plywood web to be considered, illustrated in Figure 6, is glued to both flanges and stiffeners. The applied central load P gives rise to a shear force V = P/2 on each plywood panel.

4.2 Squash Load

From Curry and Hearmon [3], the squash stress of plywood in shear, denoted by $f_{s,sq}$, may be taken to be given by the following equation, stated in N,mm units,

$$f_{s,sq} = [1.17 - 0.16\,(t_w/n)]\, f_{s,sc} + 4\,(n-1)/(t_w) \qquad (9)$$

where $f_{s,sc}$ denotes the shear strength of small clear specimens of solid wood, and n is the total number of plies.

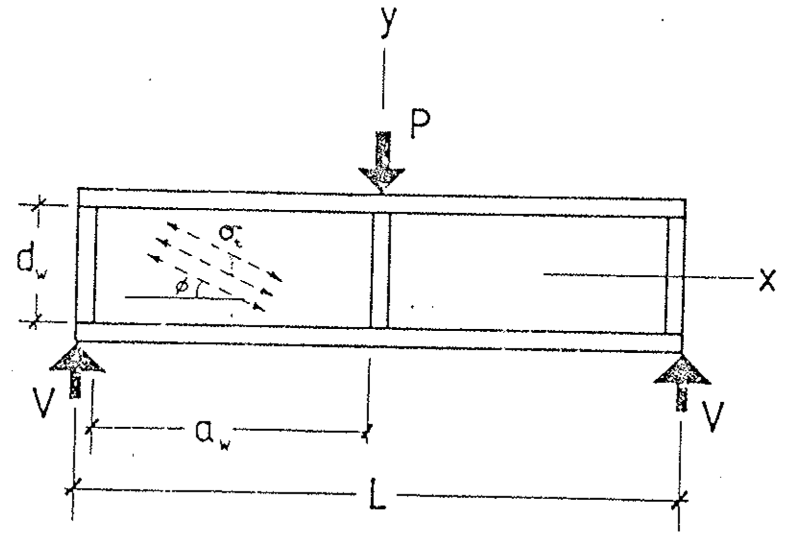


Figure 6 I-beam with stiffened plywood web

The squash load capacity of the plywood web, denoted by $V_{w,sq}$, may be estimated from

$$V_{w,sq} = d_w \, t_w \, f_{s,sq} \tag{10}$$

4.3 The Elastic Critical Load

A good estimate of the theoretical elastic critical shear stress, denoted by $f_{s,crit}$, of a rectangular plywood panel fixed on all four edges, is given by the lesser of the following

$$f_{s,crit} = 1.8 \, (\Pi/d_w)^2 \, (D_x \, D_y^3)^{0.25} \, (3.66 + 2.0 \, \beta) \tag{11a}$$

$$f_{s,crit} = 1.8 \, (\Pi/a_w)^2 \, (D_x^3 \, D_y)^{0.25} \, (3.66 + 2.0 \, \beta) \tag{11b}$$

where the elastic constants D_x, D_y and β are defined in the notation of Section 2. Equations (11) were obtained by minimization of a function derived by Lekhnitskii [4], together with the assumption that for a plywood panel with fixed edges, the critical stress is 80 per cent higher than that of a panel with hinged edges. The elastic shear capacity of the plywood web, denoted by $V_{w,crit}$, is given by

$$V_{w,crit} = d_w \, t_w \, f_{s,crit} \tag{12}$$

4.4 Ultimate Web Capacity

Once a plywood web has buckled, an I-beam will carry additional load through the action of tension membrane stresses, denoted by $\sigma_{t,\phi}$, acting at an angle ϕ to the beam axis as shown in Figure 6. If this stress plus the contribution of the buckling stress is taken to be equal to the ultimate tension strength of the plywood, then

$$\sigma_{t,\phi} = f_{t,\phi} - 2 \, f_{s,crit} \sin\phi \cos\phi \tag{13}$$

where $f_{t,\phi}$ denotes the tension strength of the plywood at an angle ϕ to the beam axis.

Taking into account the shear force carried by the vertical component of the diagonal tension then leads to the following expression for $V_{w,ult}$, the ultimate shear capacity of the plywood web

$$\begin{aligned} V_{w,ult} &= d_w\, t_w\, \{f_{s,crit} + \sigma_{t,\phi} \sin\phi \cos\phi\} \\ &= d_w\, t_w\, \{[1 - 2 \sin^2\phi \cos^2\phi]\, f_{s,crit} + f_{t,\phi} \sin\phi \cos\phi\} \end{aligned} \tag{14}$$

The value of ϕ will be chosen so as to maximize the value of $V_{w,ult}$. For typical plywood webs this leads to value of ϕ that is at an angle of 20^{o} to the strong axis of the plywood. For this direction the tension strength $f_{t,\phi}$ is roughly equal to 2 $f_{s,sq}$. Substitution of these values into equation (14) leads to

$$V_{w,ult}/V_{w,sq} = 0.64 + (0.79/\lambda^2) \tag{15}$$

where

$$\lambda = (V_{w,sq}/V_{w,crit})^{0.5} \tag{16}$$

4.5 Flange Effect

The shear deformation of plywood I-beams contributes a significant proportion fo the total beam deflection. As a result, a significant porportion of the total load may be carried by bending of the flange elements. Thus the total shear load on the beam, denoted by V, may be written

$$V = V_w + V_f \tag{17}$$

where V_w and V_f denote the shear load carried by the web and flange respectively.

For the I-beam shown in Figure 6, the flanges may be considered as a pair of simply supported beams and hence

$$V_f = 4\, E_f\, t_f\, d_f^3\, \Delta_y/L^3 \tag{18}$$

where E_f, t_f and d_f are the Youngs modulus, width and depth of each flange, L is the beam span, and Δ_y is the vertical deflection at the centre of the beam.

At the ultimate load, a lower bound estimate of deflection, denoted by $\Delta_{y,ult}$ may be obtained from consideration of the shear deformation alone, i.e.

$$\Delta_{y,ult} = (V_{w,ult}\, L)/(2\, t_w\, d_w\, G_{LT}) \tag{19}$$

Equations (18) and (19) lead to the following estimte of $V_{f,ult}$, the shear load carried by the flange at failure

$$V_{f,ult} = 2\, V_{w,ult}\, (E_f/G_{LT})\, (t_f/t_w)\, (d_f/d_w)\, (d_f/L)^2 \tag{20}$$

The ultimate shear load capacity of the beams, denoted by V_{ult}, is then given by

$$V_{ult} = V_{w,ult} + V_{f,ult} \tag{21}$$

Of course, a check must be made to ensure that the flanges can accept the deformation $\Delta_{y,ult}$ and can carry the additional vertical loads imposed by the diagonal tension field.

4.6 Experimental Data

Load tests were made on 12 I-beams fabricated with 3-ply and 5-ply radiata pine plywood webs. Some of the results shown in Figures 7 and 8 indicate that both the elastic critical load (measured by a Southwell plot) and the ultimate load capacity of the webs are reasonably well predicted by equations (11) and (15) respectively.

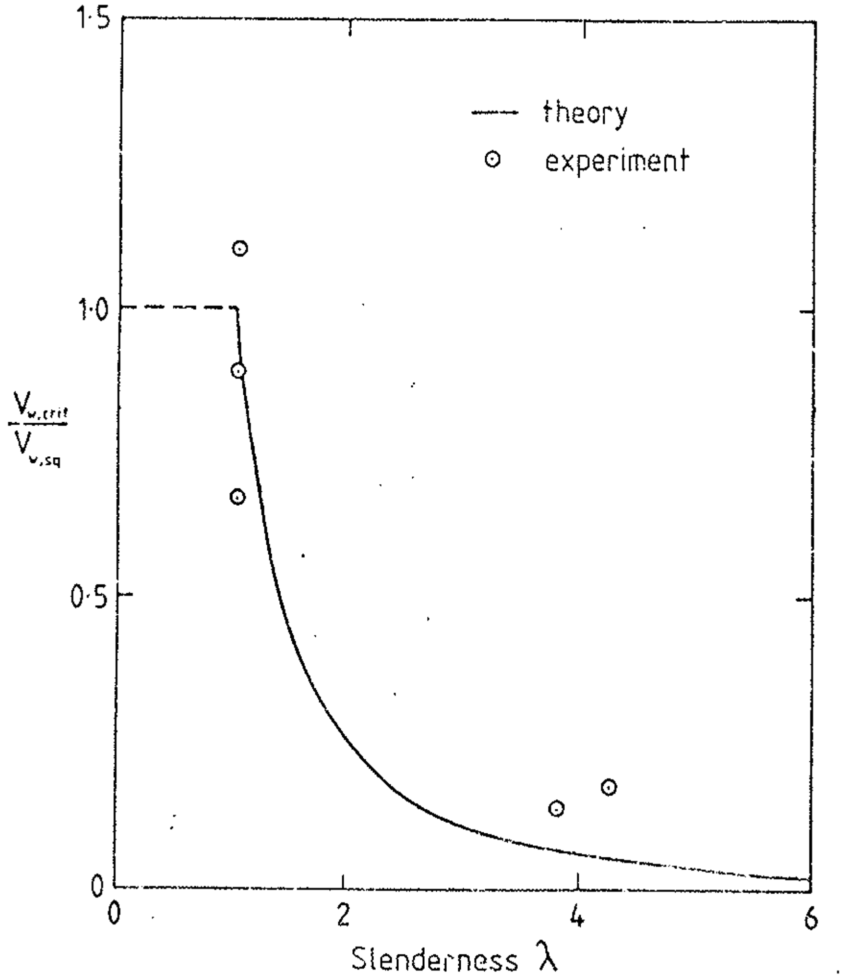


Figure 7 Critical elastic loads for stiffened plywood webs

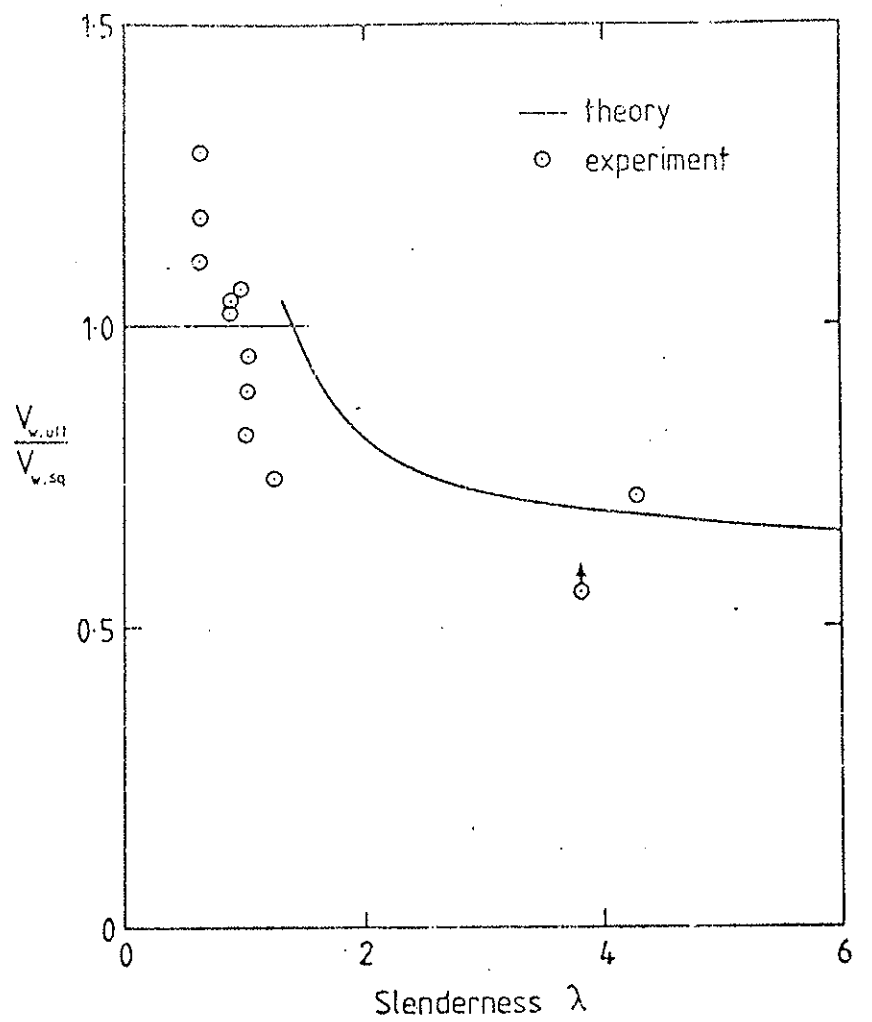


Figure 8   Ultimate strength of stiffened plywood webs

Lower bound estimates according to equations (18) and (19) of the additional shear force carried by the flanges is shown in Figure 9. It is to be noted that the ultimate shear strength of a stiffened plywood web beam can be several times the strength of the web alone.

The lateral deformations of the webs were roughly given by $\Delta_{z,crit} \cong t_w$ at the elastic critical load, and for the ultimate load were bounded by $\Delta_{z,ult} < 0.05\ d_w$.

5. CONCLUDING COMMENT

Simple procedures have been given for estimating the critical elastic loads and the ultimate strength of both stiffened and unstiffened plywood webs for glued I-beams. The methods are obviously relevant to the design of plywood webs of other types of structures.

In practical terms, it is of interest to comment that in the application of these equations, it has been noted that except for extreme designs it is difficult to induce failure in stiffened plywood webs of I-beams; it was also found that for many practical situations it is unnecessary to stiffen a plywood web.

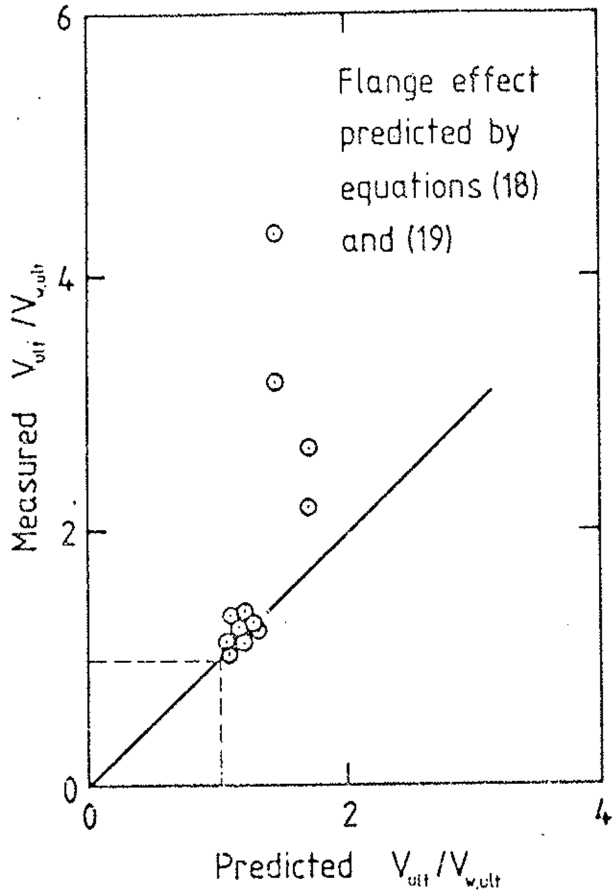


Figure 9 Measured effect of flange stiffness on the strength of stiffened plywood webs

REFERENCES

[1] ALFUTOV, N.A., and BALABUKH, L.I.: "On the possibility of solving plate stability problems without determination of the initial state of stress", Prikladnaia Methematika i Mekhaniki, Vol.31, No.4, 1967, pp.716-722.

[2] LEICESTER, R.H., and PHAM, L.: "Alfutov, Balabukh, Timoshenko and the buckling of layered orthotropic plates". Seventh Australasian Conference on the Mechanics of Structures and Materials, Perth, Western Australia, 12 to 14 May 1980, pp.8-13.

[3] CURRY, W.T., and HEARMON, R.F.S.: "The strength properties of plywood, Part II", Bulletin No. 33, Forest Products Research, Ministry of Technology, HMSO, London, 1980.

[4] LEKHNITSKII, S.G.: "Anisotropic plates". (English translation by Tsai, S.W., and Cheron, T.). Gordon and Breach, New York, 1968, p.472.

CIB-W18/17-6-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE DETERMINATION OF GRADE STRESSES FROM CHARACTERISTIC STRESSES FOR BS 5268 : PART 2

by

A R Fewell

Princes Risborough Laboratory
United Kongdom

RAPPERSWIL

SWITZERLAND

MAY 1984

DEPARTMENT OF THE ENVIRONMENT
BUILDING RESEARCH ESTABLISHMENT
PRINCES RISBOROUGH LABORATORY

THE DETERMINATION OF GRADE STRESSES FROM CHARACTERISTIC STRESSES FOR BS 5268:PART 2

by A R Fewell, March 1984

INTRODUCTION

The writer has been asked to produce a draft Annex 44 to the CIB Code listing the factors used in the UK to derive grade stresses from characteristic stresses. These factors are given in detail by Fewell[1] for visual grades but the slight variations applicable to machine grades are not included.

This paper gives a resumé of the factors for visual grades, states the relevant differences for machine grades and presents a draft Annex 44.

REDUCTION FACTORS

The relevant factors for visual grades, as given by Fewell[1] are as follows:

$$f_g = f_K . K_{LD} . K_{SZ} . K_S$$

where f_g is the grade stress

f_K is the characteristic stress (based on 200 mm depth for bending strength and 200 mm width for tension strength),

K_{LD} is the load duration factor and adjusts the test duration f_K values to long duration and has the value of 0.563 for all stresses.

K_{SZ} is the size factor adjusting the bending and tension stresses to 300 mm depth or width and has the values

0.849 for bending stress

0.925 for tension stress

1.00 for all other stresses.

K_S is a safety factor and has the value

0.724 for bending, tension and compression parallel stresses

In addition to the above values taken from reference 1 a K_s value for compression perpendicular and shear stresses can be deduced from the grade stresses given in BS 5268, the characteristic stresses given in Annex 43 of the CIB Code and the load duration factor $K_{LD} = 0.563$. $\therefore$ $K_s = 0.673$ for compression perpendicular and shear stresses. These reduction factors for compression perpendicular and shear are arbitrary since no characteristic stresses for these properties have been derived in the UK from tests.

It should be noted that when deriving permissible bending or tension stresses from grade stresses the grade stress is multiplied by a factor K_7. Where:

$K_7 = 1.17$ for timber having a depth (or width for tension) of 72 mm or less.
$K_7 = (300/h)^{0.11}$ for timber having a depth or width greater than 72 mm but less than 300 mm.
$K_7 = 0.81\,[(h^2 + 92300)/(h^2 + 56800)]$ for timber having a depth or width greater than 300 mm.

Owing to comparatively few data being available for the tension strength of full size members, and based on the work of Curry and Fewell[2] the grade tension stresses have been given an overriding value of 0.6 times the bending stresses for all grades.

There are no reduction factors for modulus of elasticity.

For machine grades there are two differences from the factors given above. Firstly since the strength variability between machine graded parcels is less than for visual grades the K_s value for bending, tension and compression parallel is 0.80. Secondly, over many years it has been found that the equation which gives the best fit when adjusting samples of bending strength data of different depths to 200 mm depth for determining grading machine settings, is $K_{SZ} = 0.73\,[(h^2 + 92300)/(h^2 + 56800)]$. Therefore the K_{SZ} factor for bending stress is 0.907.

The reduction factors given above are summarised diagramatically in Figure 1.

The factors described above are included in a draft Annex 44 to the CIB Code which is attached at the end of this paper.

REFERENCES

1 Fewell A.R. The determination of softwood strength properties for grades, strength classes and laminated timber for BS 5268:Part 2. BRE report 1984.

2 Curry W.T. and Fewell A.R. The relations between ultimate tension and compression strength of timber and its modulus of elasticity. PRL. BRE. CP 22/77. 1977.

Annex 44

REDUCTION FACTORS FOR DERIVING DESIGN STRESSES FORM CHARACTERISTIC VALUES

44.0 General

This annex provides examples of reduction factors accepted by national codes authorities.

44.1 United Kingdom

The reduction factors listed here are relevant to British Standard BS 5268. The Structural Use of Timber: Part 2 Permissible Stress Design, Materials and Workmanship. In this case the reduction factors are used to determine grade stresses (f_g) from characteristic values (f_K).

The characteristic stresses are relevant to the conditions outlined in Annex 43 clause 43.1. These include bending strength for a cross-section depth of 200 mm and tension strength for a cross-section width of 200 mm.

The grade stresses are relevant to the same conditions as the characteristic values except that bending and tension stresses are for 300 mm depth or width and the duration of load is long term. This results in three factors; K_{SZ} for size, K_{LD} for load duration and K_S for general safety.

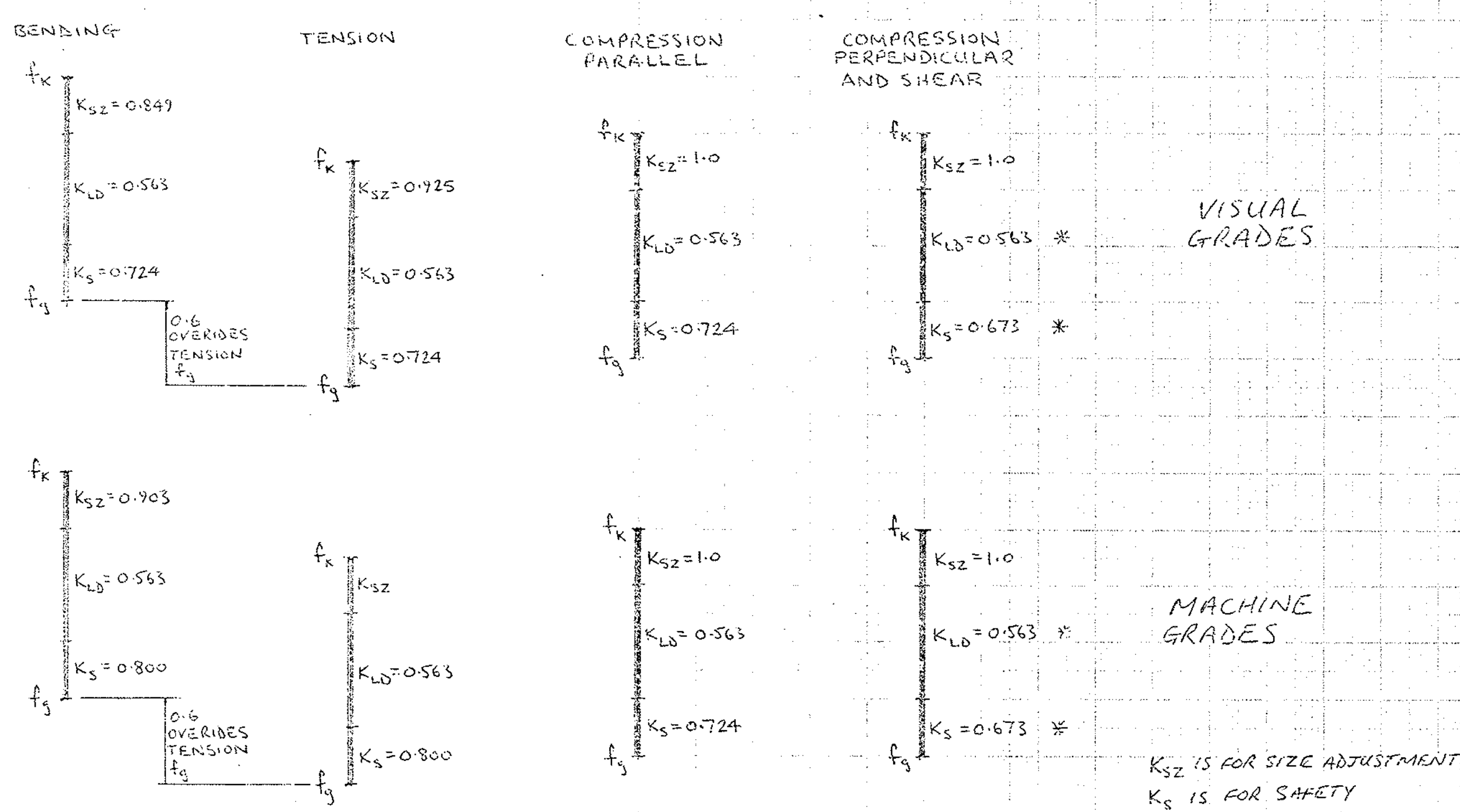
$$f_g = f_K.K_{SZ}.K_{LD}.K_S$$

FACTOR	GRADING METHOD	BENDING	TENSION PARALLEL	COMPRESSION PARALLEL	COMPRESSION PERPENDICULAR AND SHEAR
K_{SZ}	VISUAL MACHINE	0.849 0.903	0.925 -	1.0 1.0	1.0 1.0
K_{LD}	VISUAL MACHINE	0.563 0.563	0.563 0.563	0.563 0.563	0.563 0.563
K_S	VISUAL MACHINE	0.724 0.800	0.724 0.800	0.724 0.724	0.673 0.673

There is an overriding adjustment applied to f_g for tension which must not be greater than 0.6 times f_g for bending.

It should be noted that BS 5268 gives factors by which the f_g values for tension and bending can be adjusted for the size of member used in the design.

The reduction factors for compression perpendicular and shear stresses are somewhat arbitrary since no characteristic values for these properties have been determined in the UK from tests.



K_{SZ} IS FOR SIZE ADJUSTMENT
K_S IS FOR SAFETY
K_{LD} IS FOR LOAD DURATION

NOTE. BENDING AND TENSION f_g VALUES ARE RELEVANT TO 300 mm DEPTH OR WIDTH AND ARE ADJUSTED FOR OTHER SIZES.

* THESE VALUES ARE ARBITRARY SINCE NO f_K VALUES HAVE BEEN DETERMINED FROM TESTS IN THE U.K.

FIG. 1 REDUCTION FACTORS

CIB-W18/17-6-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE DETERMINATION OF SOFTWOOD STRENGTH PROPERTIES FOR GRADES, STRENGTH CLASSES AND LAMINATED TIMBER FOR BS 5268 : PART 2

by

A R Fewell

Princes Risborough Laboratory
United Kingdom

RAPPERSWIL

SWITZERLAND

MAY 1984

THE DETERMINATION OF SOFTWOOD STRENGTH PROPERTIES FOR GRADES, STRENGTH CLASSES AND LAMINATED TIMBER FOR BS 5268:PART 2

by A R Fewell - BRE, PRL. August 1983

INTRODUCTION

The revision of the Code of Practice for timber structural design BS CP 112: Part 2[1] which has now been published as BS 5268:Part 2[2] has taken more than ten years to complete. During this time PRL has continually tested structural size timber specimens of various species to investigate strength properties and it is the use of these data to determine grade stresses for the revised Code that forms a major departure from procedures used in the past. Almost all the grade stresses published in CP 112 were derived from tests on small clear specimens. The shortcomings and inaccuracies inherent in the small clear specimen approach have become increasingly obvious over the last fifteen years and as a result various attempts have been made to incorporate structural size test data into the grade stress determinations for successive drafts of BS 5268.

This report gives the procedure used to determine solid and laminated timber grade stresses for BS 5268 and is based on the final paper by Curry and Fewell[3] but also includes some later alterations made by the British Standards sub-committee responsible for code stresses. Despite the increased reliability of stresses obtained by this procedure, the lack of data still necessitates a somewhat inconsistent approach. For future revisions, more full size specimen test results, improved procedures for sampling and new techniques for calculating lower fifth percentile estimates to a given confidence level should further increase reliability by permitting the use of a more uniform and refined procedure for grade stress determination.

To ease some of the problems associated with specifying and supplying the most economic combinations of species and grade and to minimise the apparently confusing variety of choice, BS 5268 has introduced a system of strength classes which groups together in a number of classes all combinations of species and grade having similar strength. This report also gives the procedure for assigning stress values to the strength classes and the method used to allocate grades and species to each class.

GENERAL PROCEDURE

The procedure for the determination of grade stresses is relevant only to visual grades. For machine grades relations between strength and modulus of elasticity (Ep) are used to derive machine settings for any level of strength within the range of the species concerned. The method of determining these machine settings is to be the subject of a British Standard.

Despite the continual enlargement of PRL's databank there are still many species listed in BS 5268 for which structural size test data do not exist. To account for such species, relativities between structural size test values and mean small clear test values were used which were established for species where both sets of data exist. The overall procedure is summarised below and the nomenclature used is repeated later in the detailed descriptions of the analysis.

(a) Lower 5th percentile characteristic values of bending strength (f_{mk}) and modulus of elasticity (E_k), and mean modulus of elasticity (E) for timber graded to the SS grade of BS 4978[4] were obtained from the results of tests on structural size specimens. It should be noted that a closed visual stress grade, that is one where both the lower and upper limits to the permissible defects are defined would have been preferred to the SS grade as the definitive grade to which all other grades are indexed. However, this was not possible within the UK grading system.

(b) Mean values of bending strength (f_{sc}) and modulus of elasticity (E_{sc}) from tests on small clear specimens were then determined.

(c) The f_{mk}, E_k and E values were expressed as ratios of the corresponding mean stress values, f_{sc} and E_{sc} and these ratios were used to determine characteristic values for species for which their are no structural size test data.

(d) Characteristic values for tension and compression, and for the other grades were obtained by multiplying the values from a, and c, by previously established grade relativity factors.

(e) Grade stress values for use in design were determined by multiplying the lower 5th percentile values by various factors to allow for the effects of duration of load and size, and to include a factor of safety.

(f) Grade stresses for machine grades were determined from a consideration of their relativities with visual grades, their use and the implications of any changes from the current Code.

(g) Stresses for strength classes and the assignment to strength classes of various species and grades were determined.

(h) Laminating grades. Factors were determined to enable permissible stresses for various combinations of grade and number of laminations to be calculated.

MATERIAL AND TESTING

The data used in this analysis was drawn from PRL's databank of structural size test results. An outline of the samples used is given in Table 1. Although the samples of timber that were tested were subject to some pre-selection they nevertheless covered the spectrum of strength likely to be included in commercial samples of graded timber. The species tested were European redwood/whitewood, British grown Douglas fir and Sitka spruce and Canadian hem-fir and spruce-pine-fir.

The timber was conditioned to a moisture content of some 14 to 17 per cent and the tests were carried out to the procedures given in BS 5820:1979[5]. For the bending tests a span to depth ratio of 18 was used, the load was applied at the third points of the span and the duration of a test was generally between 10 to 15 minutes. In all tests the grade determining defect was located within the zone of maximum stress with the tension edge selected at random. Modulus of elasticity was determined from shear free deflections measured within the centre third of the span. The tension tests were made on full cross-section specimens with the major defects clear of the grips and the duration of each test was again 10 to 15 minutes.

To assist with some aspects of this analysis, for example the determination of grade relativity factors, data from tests carried out in Canada were also used. These samples are listed in Table 2 and used a rapid proof loading method to determine the bending and tension strength properties at the lower ends of the distributions of strength for timber conforming to the Canadian NLGA stress grades. The proof load was set to achieve about a 10 per cent breakage rate and the species tested were Douglas fir-larch, hem-fir and spruce-pine-fir. The tests were carried out at the sawmills and the timber covered a wide range in moisture contents, from 10 per cent to green, and was tested at temperatures ranging from −17 to 13°C. For the bending tests a span to depth ratio of 17 was used, the load was applied at the third points of the span and the duration of test was generally less than one minute. No positive action was taken to locate the grade determining defect in each piece of timber within the zone of maximum stress. Centre point deflection relative to the supports was measured for the determination of modulus of elasticity. The tension tests were made on full cross-section specimens and the duration of a test was also generally less than one minute. The full test programme involved some 4000 specimens but the number of specimens in each sample is not known.

ANALYSIS

(a) Characteristic values

The BS 4978 SS grade pieces were selected out from each sample listed in Table 1 and analysed using the 3 parameter Weibull function to determine the lower fifth percentile bending stress and E value and mean E value for each sample given in Table 3.

Subsequently the bending stress values were adjusted to a standard depth of 200 mm using the factor from Fewell and Curry[6], $K = (200/h)^{0.4}$ where h is depth of sample. No adjustments for moisture content were incorporated.

The modulus of elasticity values were adjusted to 18 per cent moisture content using the equation from Covington and Fewell[7] $\log E_1 = \log E_2 + C\ (M_2 - M_1)$ where M is moisture content and the suffixes 1 and 2 denote the lower and higher moisture contents respectively and C has the values:

European redwood/whitewood	6.36×10^{-3}
Hem-fir	4.53×10^{-3}
Spruce-pine-fir	4.67×10^{-3}
Other softwoods	5.01×10^{-3}

For all samples of each species combination, weighted averages of the adjusted lower 5 per cent or mean values were calculated to determine characteristic values f_{mk}, E_k and E. These values are given in Table 4 and except for redwood/whitewood and Sitka spruce were taken from Curry and Fewell[8] for which they were first calculated. For redwood/whitewood the test data were re-sorted to include only specimens from Vth and VI quality parcels in the ratio 5 to 1. This gives samples which are more representative of the material available commercially, than the samples used in the original test programme.

(b) Mean small clear bending strength and E values

Mean bending strength (f_{sc}) and modulus of elasticity (E_{sc}) values at 18 per cent moisture content for small clear specimens were determined from data given by Tory[9]. For the North American species combinations the tests were carried out in North America to ASTM D2555-78a[10] using 2 inch square specimens, whilst for other species the tests were carried out at PRL using 2 cm square specimens and the data are taken from Bulletin 50[11]. Consequently two different approaches were used to determine the mean values required from the North American and PRL data. The test data were for specimens at moisture contents of around 12 per cent and also at fibre saturation point, necessitating adjustments to derive values for 18 per cent. It was decided to assume a fibre saturation point for all species of 27 per cent moisture content.

UK data approach. Test values are given in Table 5 for the two moisture conditions mentioned above. Also given are the values at 18 per cent moisture content determined from the equation

$$f_{sc} = \exp\left[\ln f_{FSP} + (\ln f_D - \ln f_{FSP})\frac{27-18}{27-M_D}\right]$$

where f is bending strength and can be replaced by E throughout,

M_D is the dry test moisture content and the suffixes FSP and D denote strength (or E) at 27 per cent and the dry moisture contents respectively

North American data approach. To adjust the 2 inch values to 2 cm square values the North American data given in Table 6 were multiplied by factors taken from Bulletin 50, ie 1.053 for bending strength and 0.935 for modulus of elasticity. To adjust to 18 per cent moisture content the bending strength and E values for the 27 per cent moisture condition were multiplied by a ratio R given in Table 6. Values of R were determined from

$$R = R_{ASTM}^{0.6}$$

where R_{ASTM} = ratio of 12 per cent moisture content strength or E to values at fibre saturation point taken from ASTM D2555

The f_{sc} and E_{sc} values for a species combination were then taken as the mean of all those values for the individual species and are given in Table 7.

(c) Ratios of structural size test values to small clear test values

Table 8 shows the ratio of structural size test values, f_{mk}, E and E_k from Table 4 to the small clear f_{sc} and E_{sc} values from Tables 5 and 7. It can be seen from Table 8 that except for BC Douglas fir the ratios do not differ greatly between species. It should be noted that test evidence for both British grown and Canadian Douglas fir is available which shows that this species is currently much overrated for strength in the national Codes due to stresses being based on small clear specimen tests. Omitting Douglas fir, mean values of the ratios were calculated and are also given in Table 8. For all species where no structural size test data exist, except for North American Douglas fir, the mean ratios were multiplied by the f_{sc} and E_{sc} values to determine the characteristic values for the SS grade which are listed in Table 9. For completeness Table 9 also includes the values from Table 4.

For Canadian Douglas fir-larch, bending strength and E values for the NLGA SEL grades given in Table 10 from the Canadian test data referred to in Table 2 were used. Because of differences in test methods these stresses may not be directly comparable with the UK stresses but they nevertheless allow relative strengths of the three species groups to be established. The stresses in Table 10 suggest the following strength ratios between Douglas fir-larch and the species

	f_{mk}	E	E_k
Hem-fir	1.077	1.077	1.097
Spruce-pine-fir	1.044	1.137	1.125

Applying these ratios to the SS grade stresses for hem-fir and spruce-pine-fir in Table 9, and averaging the results, gives the stresses and E values for SS grade Canadian Douglas fir-larch included in Table 9.

For the USA species no structural size test data were available and so an indirect approach was used to determine their characteristic values. Because the USA and Canadian species groups (except Southern pine) have similar species compositions it was decided that the stresses should be obtained by factoring the corresponding characteristic values for the Canadian species groups by the ratios of the small clear specimen values for each of the groups. This maintains the strength relativity between the two sources of supply. The resulting stresses and E values are also given in Table 9.

Before stress values for the other grades and properties were determined consideration was given to a number of factors which influenced the values finally specified.

1 Values of characteristic stress are invariably subject to uncertainty, and even for quite large samples the stresses for the SS grade can differ by more than 20 per cent between samples of the same species. Therefore small differences in stress values between species are really of no significance to the safety of structures and to specify stress values with quite small differences is also a practical inconvenience with no real advantage to the efficient use of the material.

2 The characteristic bending stress of spruce-pine-fir is shown from the Canadian tests to be 4 per cent higher than for hem-fir, whilst from the UK tests it is 6 per cent lower. There is some justification for concluding that this difference reflects UK sampling effects which would tend to lower the strength of spruce-pine-fir compared with hem-fir.

3 It was previously agreed by the Code committee that British grown Scots pine and larch should be assigned the same stress values as European redwood/whitewood, and that British grown European spruce should have the same stress values as British grown Sitka spruce.

4 Canadian test results show that the characteristic bending stresses for Douglas fir-larch are higher for the Select grade, but lower for the No 1, 2 and 3 grades, than those for the same grades of Hem-fir and spruce-pine-fir[12].

The four points given above were used to modify some of the bending strength values in Table 9 to produce the assigned values given in Table 11. In general the bending stress of 21.6 N/mm^2 for European redwood/whitewood and for Hem-fir obtained from the test results, was taken as the reference level and those species whose characteristic bending stress was within $\pm$ 5 per cent of this were assigned the same value.

(d) Other properties and grades

The characteristic stress values for bending, modulus of elasticity, tension and compression parallel to grain for the other grades were obtained by multiplying the SS grade values in Table 11 by grade relativity factors. The relativity factors for the BS 4978, NLGA Joist and Plank grades were determined in a previous study by Curry and Fewell[8] and are given in Table 12. The grade relativity factors for No 1 and No 2 grades in tension differed little and in line with the Canadian test evidence the same factor was applied to both. The relativity factors are the same for the Joist and Plank and Structural Light Framing grades and for both the Canadian NLGA and USA NGRDL grades.

The grade relativity factors for the Light Framing and Stud grades were obtained by estimation from their specified grade strength ratios and grade relativity factors established for the Joist and Plank grades. These factors are included in Table 12. With no test evidence available and in line with Canadian practice, no tension stresses are given for the No 3 grades, the Light Framing and the Stud grades. Timber of these grades should not be used in constructions where it would be subject to direct tension forces.

Characteristic stress values determined using the SS grade stresses for the different species in Table 11, and the grade relativity factors in Table 12 are given in Tables 13 to 17.

(e) Grade stress values

Grade stress values were obtained by applying factors to the characteristic stresses given in Tables 13 to 17. The characteristic values are relevant to a short duration load and a section width of 200 mm and must therefore be adjusted to the standards adopted by BS 5268 which are long term load duration and 300 mm section width. The general safety factor incorporates an allowance for the fact that in CP 112[1] stresses were determined from a first percentile estimate instead of the fifth percentile used here. The following summarises the factors for the different properties.

Bending strength. The relevant factors are

section depth	0.849	(From $K = (200/h)^{0.4}$ - Fewell[13])
duration of load	0.563	
general safety	0.724	

Thus characteristic stresses in bending are multiplied by a combined factor of 0.346.

Tension strength. The relevant factors are

section width	0.925	(From $K = (200/h)^{0.192}$ - Fewell[13])
duration of load	0.563	
general safety	0.724	

Thus characteristic stresses in tension are multiplied by a combined factor of 0.377.

Compression (parallel). The relevant factors are

duration of load	0.563
general safety	0.724

Thus characteristic stresses in compression parallel to the grain are multiplied by a combined factor of 0.408.

Modulus of elasticity. There are no factors required but all values have been rounded off to the nearest 500 N/mm^2.

Other properties. In addition to the major strength properties given above the Code will also include grade stresses for shear parallel to grain and for compression perpendicular to grain, ie bearing stress. These have been derived by Tory[9] and are included in this report to complete the stresses which are given in Tables 18 to 20. These tables also include modifications as a result of the following decisions by the Code committee.

1 It was decided by the committee that because of the size effect on bending and tension stresses that these stresses should be allowed to increase for sizes less than 300 mm width or depth reaching a maximum at 72 mm, using the factor K from the equation

$K = (300/h)^{0.11}$ as suggested by Fewell[13]

This size factor is less than the factors used in this report to adjust the test data to derive characteristic stresses. The reasons for this decision were firstly that using larger factors could be unsafe due to the large variability in the test data, secondly it is possible that larger factors would be more unsafe for hardwoods and machine graded timber and thirdly for simplicity one equation could be used for both bending and tension stresses and all grades and species. For bending members with depths greater than 300 mm the grade stress is reduced in line with the factor currently used in CP 112:Part 2.

One effect of the above decision is that bending and tension stresses for the Structural Light Framing, Light Framing and Stud grades which are relevant to a 38 x 89 mm section size are increased. Stresses for other section sizes of these grades are found by multiplying the stresses for the 38 x 89 mm section size by the factors given in Table 21. These factors are retained in line with North American practice despite a lack of evidence to justify their use.

2 The committee also decided that because of the desired equivalence between visual and machine versions of SS and GS grades, tension stresses for BS 4978 grades should be 0.6 times the bending stresses in line with evidence on machine grading[14].

(f) Machine grades

CP 112:Part 2 and BS 4978 include four machine stress grades. These are MSS and MGS which have the same stress values as the corresponding visual grades, and the M75 and M50 grades, which correspond to the numbered visual grades in the Code. Machine stress grading selects timber to specified stress values and any changes in the specified stress levels must be accompanied by corresponding changes in machine selection, ie in the control settings by which machines operate. Therefore a change to the stresses of a visual grade represents a reassessment of the strength of that grade and does not change the timber included within that grade, as would happen if a bending stress for a machine grade was altered. With the same grade designations changes in the stress values of machine grades compared with those previously used could cause problems, and adversely affect the strength or serviceability of structures. The changes in the stress values for the SS and GS grades given in this paper are relatively small and can be accomodated with current grading practice. With the M75 grade, for which there is no corresponding visual grade, a decision must be made as to what stress values to assign. The trussed rafter industry has largely developed around the M75 and M50 grades and it is therefore desirable to maintain for these grades the CP 112 stress values for bending, and to adjust the tension, compression and modulus of elasticity values to maintain the relativity of these properties for particular species. This approach was accepted by the Code committee and the resulting machine grade stresses are included in Table 18.

Comparisons between the grade stresses derived here for BS 5268 and those given in CP 112, for some of the more important species, are given in Figures 1 to 6. Major differences between the CP 112 and BS 5268 methods of derivation of stresses are:

1 the modulus of elasticity values in BS 5268 are shear free;
2 the tension stresses take account of the lower strength evident from structural size tests;
3 the compression stresses include a factor for duration of load which was not adequately allowed for in the CP 112 stresses;

4 by ignoring small and possibly unreal differences a number of widely used species are assigned the same grade stresses for bending, tension and compression parallel to the grain which should simplify specification and supply. The species are Douglas fir-larch, hem-fir and spruce-pine-fir from Canada, Douglas fir-larch and hem-fir from the USA, larch, Scots pine and Corsican pine from the UK and European redwood/whitewood;

5 the grade stresses for British grown and imported Douglas fir have been significantly reduced. This reflects the very strong test evidence, both in the UK and North America, that previously specified stresses overrate the strength of the material now being produced.

(g) Strength classes

The increasing number of stress grades and species available in the UK has meant that the task of specifying timber for structural use has become more complicated. In an attempt to ease this situation the code committee decided to incorporate a strength class system into BS 5268.

To appreciate the major advantage of the strength class system it must be understood that timber of a particular stress grade does not have the same strength regardless of species. Therefore to specify a stress grade without specifying a species is meaningless in terms of strength. In the past the structural engineer has had the problem of having to decide which of the numerous combinations of species and grade will be specified before design calculations can be completed. He cannot hope to accurately anticipate the changes in availability and movements in prices which may occur in the time interval between completing the design and purchasing the timber.

Strength classes are divisions of timber strength into which species/grade combinations of similar strength are allocated. One value of each strength property is assigned to all species/grades in each strength class. Thus a structural engineer can design to and specify a particular strength class and allow availability and price at the time of purchase to determine the actual grade/species combination to be used. Strength classes are therefore seen as a simplification not only for the engineer but also for suppliers who will be able to relate their stocks more easily to specifications received. Of course on occasions it may be necessary for the designer to limit the species, as for example when durability or joint strength is important.

The species grouping system in CP 112 differs from strength classes in that it simply groups together timber species of similar strength. This does not allow high grades of weaker species to be classified with low grades of stronger species even if their stress values permit.

Establishing strength class boundaries on the basis of a mathematical progression results in a system which is inefficient for many of the commonly used species/grades. Because of the wide variety of structural component types, sizes and spans it is also impractical to tie the class boundaries to increments of structural component design. It was therefore decided to adopt the approach previously accepted by the Code committee which was to make the class boundary strength values match the major species/grades used in the UK. Whilst this ensures the most efficient use of these species/grades, inevitably others are penalised. However, for those species which can be machine graded, machine settings can be produced which will enable a more efficient approach by grading directly to the strength class boundaries.

The strength properties given in Tables 18, 19 and 20 for all the species/grades are listed in Table 22, ranked on the basis of bending strength. Taking into account the major species/grades used in the UK, an examination of Table 22 determined bending stress values of 10.0, 7.5, 5.3, 4.1 and 2.8 N/mm^2 for classes SC5 to SC1 respectively. Class values for the other properties were taken from the lowest value in Table 22 for species/grade combinations with bending stress values equal to or greater than the bending value for that class. An exception to this was made where the lowest value of a particular property would have penalised the remaining species/grades in that class too severely. An example of this is the demotion of SS grade British grown spruce to the Sc 2 class because of its low modulus of elasticity values. In line with a previous draft of BS 5268, species/grades were admitted to a strength class if their bending stress values were equal to or greater than the class value and their values for E(minimum), E(mean), shear and compression parallel exceeded 95 per cent of the class values. No account was taken of compression perpendicular to grain in assigning species/grades to strength classes. The strength property values assigned to the classes are given in Table 23 and the allocation of species/grades to these classes is shown in Tables 24, 25, 26 and 27.

The size factors given in Table 21 for various sizes of North American Light Framing, Stud and Structural Light Framing grades were used to calculate stress values for allocating these sizes and grades to the classes shown in Tables 26 and 27.

(h) Laminating grades

In CP 112 permissible stress values for laminated timber were obtained by multiplying the specified basic stresses for the species by factors related to the grade of the timber used, and to the number of laminations in the member. For BS 5268 the SS grade stresses have effectively replaced the basic stresses and it is therefore appropriate to use these for laminated timber. A new set of factors based on the SS grade stresses for redwood/whitewood were calculated to give where possible the same permissible stresses obtained from CP 112 whilst retaining the same relativity in permissible stresses, for sections with different numbers of laminations. These factors for single grade constructions and for combinations of two grades, are given in Tables 28 and 29.

CONCLUSIONS

By publishing the procedures used to determine the Code stress values this paper will assist those charged with the responsibility of producing future revisions.

It is obvious from the content of this paper that improvements can still be made to the procedures by continued research which will increase the data available, increase knowledge of the factors affecting strength and refine the statistical techniques used in the analysis.

REFERENCES

1 British Standard Code of Practice CP 112:Part 2:1971. The structural use of timber. BSI 1971.

2 British Standard Code of Practice BS 5268:Part 2:1983. The structural use of timber - permissible stress design, materials and workmanship. BSI 1983.

3 Curry W T and Fewell A R. Grade stress values for BS 5268 - second revision. CSB 32/2-81/7. PRL 1981.

4 British Standard Specification BS 4978:1973. Timber grades for structural use. BSI 1973.

5 British Standard Specification BS 5820:1979. Methods of test for the determination of certain physical and mechanical properties of timber in structural use. BSI 1979.

6 Fewell A R and Curry W T. Depth factor adjustments in the determination of characteristic bending stresses for visually stress graded timber. The Structural Eng June 83. Vol 61B No 2.

7 Covington S A and Fewell A R. The effect of change in moisture content on the geometrical properties, modulus of elasticity and stiffness of timber. PRL BRE CP21/75 1975.

8 Curry W T and Fewell A R. Canadian and UK strength tests on structural timber and comparison of results. PRL 1981.

9 Tory J R. Grade stresses and strength classes for structural softwoods in BS 5268:Part 2. PRL 1980.

10 ASTM D2555-78a. Standard methods for establishing clear wood strength values. ASTM 1978.

11 Lavers G M. The strength properties of timber. FPRL Bulletin 50 HMSO 1969.

12 Bury K V. Statistical analysis of NLGA bending tests. Univ of BC. 1981.

13 Fewell A R. Size factors for bending and tension stresses. CSB/32/2 82/27. Nov 1982.

14 Curry W T and Fewell A R. The relations between the ultimate tension and compression strength of timber and its modulus of elasticity. PRL BRE CP 22/77 1977.

Table 1 TIMBER SAMPLES FROM UK TESTS

Species	Origin of timber	Nominal size mm	Number of pieces		
			Bending	Tension	Compression
Redwood/whitewood	Sweden	38 x 100		213	
		38 x 150	485	206	
		38 x 200		195	
		50 x 100			104
		50 x 150	443		
		50 x 200	507		
		50 x 170	97		
		70 x 195	93		
	Finland	38 x 100	248		
		38 x 150	245		
		50 x 200	228		
	Poland	38 x 100			199
		50 x 100			218
Hem-fir	Canada	44 x 100	197		
		44 x 150	124		
		44 x 200	121		
		75 x 150	92		
		75 x 250	91		
Spruce-pine-fir(1)	Canada	38 x 88	320	213	213
		38 x 184	324	215	210
Douglas fir	United Kingdom	38 x 100	253		
		47 x 200	218		
Sitka spruce	United Kingdom	47 x 228	81		
		72 x 250	86		
		73 x 154	89		

(1) These samples were selected for destructive testing from a larger sample of 7492 pieces.

Table 2 TIMBER SAMPLES FROM CANADIAN TESTS

SPECIES	NLGA GRADE	SECTION DEPTH MM	NUMBER OF BENDING SAMPLES
Douglas fir-larch	SEL	89 140 184 235	2 2 2 2
	No 1	89 184 235	1 1 1
	No 2	89 140 184 235	2 2 2 2
	No 3	89 140 184 235	1 1 1 1
Hem-fir	SEL	89 140 184 235	1 1 1 1
	No 2	89 140 184 235	1 1 1 1
	No 3	89 140 184 235	1 1 1 1
Spruce-pine-fir	SEL	89 140 184 235	4 4 4 2
	No 1	89 140 184	2 2 2
	No 2	89 140 184 235	4 4 4 2
	No 3	89 140 184 235	1 1 1 1

Table 3 BENDING STRENGTH AND MOE LOWER FIFTH PERCENTILES AND MEAN MOE VALUES FOR INDIVIDUAL SAMPLES GRADED TO THE SS GRADE

Species	Nominal size mm	Bending strength		Modulus of elasticity		
		No of pieces	Lower 5% N/mm^2	No of pieces	Lower 5% N/mm^2	Mean N/mm^2
Redwood/whitewood	38 x 150	126	24.0	126	7143	11164
	50 x 150	158	23.9	158	7434	10834
	50 x 200	132	22.1	132	7610	11302
	58 x 170	85	24.0	85	6981	11192
	70 x 195	91	23.1	91	7817	11900
	38 x 100	54	24.2	54	7043	10373
	38 x 150	62	23.4	62	7425	10782
	50 x 200	66	22.0	66	6604	10540
Hem-fir	74 x 150	83	20.2	83	6832	10903
	74 x 248	85	22.3	85	7559	11689
	44 x 100	149	27.3	149	8676	13289
	44 x 150	112	25.7	112	8243	11508
	44 x 200	110	21.8	110	8920	12757
Spruce-pine-fir	38 x 88	214	28.6	487	7471	11052
	38 x 184	264	20.9	575	6369	10045
Douglas fir	38 x 100	84	21.8	84	6823	11232
	47 x 200	130	18.8	130	7695	11492
Sitka spruce	47 x 228	43	16.6	43	4606	7521
	72 x 250	68	15.5	68	5071	7959
	73 x 154	57	16.7	57	5860	8477

Table 4 WEIGHTED MEAN VALUES OF SS GRADE STRUCTURAL SIZE TEST DATA WITH BENDING STRENGTH ADJUSTED TO 200 mm DPETH AND MOE ADJUSTED TO 18% MOISTURE CONTENT

Species	Characteristic values for SS Grade N/mm^2		
	f_{mk}	E	E_K
European redwood/whitewood	21.6	10561	7021
Canadian hem-fir	21.6	11822	7953
Canadian spruce-pine-fir	20.4	10185	6664
British grown Sitka spruce	16.5	7882	5130
British grown Douglas fir	17.9	11019	7120

Table 5 SMALL CLEAR DATA FROM PRL TESTS

Species	Bending strength N/mm^2				Modulus of elasticity N/mm^2			
	f_{FSP}	f_D	M_D	f_{SC}	E_{FSP}	E_D	M_D	E_{SC}
Parana pine	52	98	12.0	76.1	8700	10400	12.0	9680
Pitch pine	66	107	12.0	88.2	10400	12600	12.0	11670
Western red cedar	38	57	12.0	48.5	6800	7700	12.0	7330
Redwood/whitewood	42	78	13.3	62.5	7500	10100	13.3	9150
British grown Douglas fir	53	91	12.0	73.3	8300	10500	12.0	9560
British grown larch	50	87	12.9	71.2	7300	9100	12.9	8400
British grown Scots pine	46	89	12.0	68.4	7300	10000	12.0	8820
British grown European spruce	36	66	13.4	53.8	6300	8500	13.4	7680
British grown Sitka spruce	34	67	12.0	51.1	5900	8100	12.0	7140
British grown Corsican pine	41	81	12.0	61.7	7000	9200	12.0	8250

Table 6 NORTH AMERICAN SMALL CLEAR TEST DATA

SPECIES COMBINATION	SPECIES	CANADA				USA			
		BENDING STRENGTH N/mm^2		MODULUS OF ELASTICITY N/mm^2		BENDING STRENGTH N/mm^2		MODULUS OF ELASTICITY N/mm^2	
		f_{FSP}	R	E_{FSP}	R	f_{FSP}	R	E_{FSP}	R
Douglas fir-larch	Douglas fir (Coast)	-	-	-	-	52.9	1.336	10756	1.143
	Douglas fir (West)	-	-	-	-	53.2	1.346	10432	1.121
	Douglas fir (North)	-	-	-	-	51.3	1.404	9715	1.154
	Douglas fir (Canada)	52.0	1.375	11122	1.127	-	-	-	-
	Western larch	59.8	1.418	11404	1.149	52.8	1.375	10053	1.160
Hem-fir	California red fir	-	-	-	-	40.1	1.428	8067	1.160
	Grand fir	-	-	-	-	40.3	1.291	8619	1.149
	Noble fir	-	-	-	-	42.5	1.394	9515	1.143
	Amabilis/Pac silver	37.8	1.432	9287	1.127	44.2	1.380	9791	1.138
	White fir	-	-	-	-	40.4	1.360	8005	1.165
	Western hemlock	48.0	1.370	10177	1.121	45.8	1.380	9012	1.143
Spruce-pine-fir and Western Whitewoods	Engleman spruce	39.0	1.413	8626	1.138	32.4	1.507	7095	1.149
	Black spruce	40.5	1.488	9101	1.087	-	-	-	-
	White spruce	35.2	1.413	7929	1.149	-	-	-	-
	Red spruce	40.5	1.404	9136	1.121	-	-	-	-
	Alpine fir	35.6	1.321	8674	1.104	33.8	1.404	7254	1.132
	Balsam fir	36.5	1.326	7784	1.138	-	-	-	-
	Western white pine	-	-	-	-	32.3	1.543	8226	1.127
	Lodgepole pine	39.0	1.493	8784	1.138	37.9	1.375	7419	1.138
	Ponderosa pine	39.3	1.451	7791	1.127	35.4	1.442	6874	1.170
	Sugar pine	-	-	-	-	33.7	1.360	7116	1.093
	Jack pine	43.5	1.418	8046	1.154	41.6	1.346	7364	1.154
	Mountain Hemlock	-	-	-	-	43.2	1.437	7157	1.160
Southern pine	Loblolly pine	-	-	-	-	50.3	1.399	9667	1.160
	Longleaf pine	-	-	-	-	58.9	1.375	10935	1.143
	Shortleaf pine	-	-	-	-	51.3	1.404	9570	1.149
	Slash pine	-	-	-	-	59.9	1.456	10563	1.165

Table 7 MEAN VALUES FROM SMALL CLEAR TESTS (Table 6) FOR THE CANADIAN AND USA SPECIES COMBINATIONS

		Small clear mean values N/mm^2	
Species	Origin	f_{sc}	E_{sc}
Douglas fir-larch	CAN USA	82.4 75.1	11984 10953
Hem-fir	CAN USA	63.1 60.4	10228 9496
Spruce-pine-fir Western whitewoods	CAN USA	58.0 54.2	8915 7800
Southern pine	USA	81.7	10988

Table 8 RATIOS OF SS GRADE STRUCTURAL SIZE TEST VALUES (FROM Table 4) TO MEAN SMALL CLEAR TEST VALUES (FROM Tables 5 AND 7)

	Ratios of SS grade values to mean small clear values		
Species	f_{mk}/f_{sc}	E/E_{sc}	E_k/E_{sc}
European redwood/whitewood	0.35	1.15	0.77
Canadian hem-fir	0.34	1.16	0.78
Canadian spruce-pine-fir	0.35	1.14	0.75
British grown Sitka spruce	0.32	1.10	0.72
British grown Douglas fir	0.24	1.15	0.74
Mean ratios (excluding BG Douglas fir)	0.34	1.14	0.76

Table 9 CHARACTERISTIC VALUES FOR THE SS GRADE

Species	Characteristic values for SS grade N/mm^2		
	f_{mk}	E	E_k
Redwood/whitewood	21.6	10561	7021
Hem-fir (Canada)	21.6	11822	7953
Spruce-pine-fir (Can)	20.4	10185	6664
Sitka spruce (BG)	16.5	7882	5130
Douglas fir (BG)	17.9	11019	7120
Western red cedar	16.5	8356	5571
Corsican pine (BG)	21.0	9405	6270
Parana pine	25.9	11035	7357
Pitch pine	30.0	13304	8869
Douglas fir-larch (Can)	22.4	12160	8110
Douglas fir-larch (USA)	20.4	11115	7413
Hem-fir (USA)	20.7	10976	7384
Western whitewoods (USA)	19.1	8911	5831
Southern pine	27.8	12526	8351

Table 10 CHARACTERISTIC VALUES FOR THE NLGA SELECT GRADE FROM CANADIAN IN-GRADE TEST DATA

Species	Sel grade		
	f_{mk}	E	E_k
Douglas fir-larch	23.9	12620	9380
Hem-fir	22.2	11720	8550
Spruce-pine-fir	22.9	11100	8340

Table 11 CALCULATED AND ASSIGNED VALUES OF STRESS (N/mm^2) FOR THE SS GRADE OF BS 4978, RANKED IN ORDER OF BENDING STRESS

SPECIES	PROPERTY			
	BENDING STRENGTH f_{mk}		MODULUS OF ELASTICITY	
			E	E_k
	calculated	assigned	calculated	calculated
Pitch pine	30.0	30.0	13304	8869
Southern pine (USA)	27.8	27.8	12526	8351
Parana pine	25.9	25.9	11035	7357
Douglas fir-larch (C)	22.4	21.6	12160	8110
Hem-fir (C)	21.6	21.6	11822	7953
Redwood/whitewood, Scots pine (BG), larch (BG)	21.6	21.6	10561	7021
Corsican pine (BG)	21.0	21.6	9405	6270
Hem-fir (USA)	20.7	21.6	10976	7384
Douglas fir - larch (USA)	20.4	21.6	11115	7413
Spruce pine-fir (C)	20.4	21.6	10185	6664
Western whitewoods (USA)	19.1	19.1	8911	5831
Douglas fir (BG)	17.9	17.9	11019	7120
Western red cedar	16.5	16.5	8356	5571
European spruce (BG) Sitka spruce (BG)	16.5	16.5	7832	5130

Table 12 GRADE RELATIVITY FACTORS INDEXED TO THE SS GRADE CHARACTERISTIC BENDING STRESS

PROPERTY	BS 4978		NLGA AND NGRDL							
			JOIST AND PLANK: STRUCTURAL LIGHT FRAMING				LIGHT FRAMING			STUD
	SS	GS	Sel	No 1	No 2	No 3	Const	Std	Util	
Bending	1.00	0.71	1.07	0.75	0.75	0.55	0.63	0.47	0.37	0.55
Tension	0.63	0.51	0.76	0.53	0.53	-	-	-	-	-
Compression (Par)	0.90	0.77	1.00	0.90	0.78	0.60	0.69	0.52	0.38	0.60
Mean E	1.00	0.84	1.01	0.90	0.90	0.87	0.89	0.87	0.87	0.87
Min E	1.00	0.83	1.05	0.90	0.90	0.80	0.90	0.87	0.87	0.85

Table 13 CHARACTERISTIC STRENGTH VALUES FOR THE BS 4978 GRADES

SPECIES	PROPERTY									
	Bending		Tension		Compression parallel		Mean E		Minimum E	
	SS	GS	SS	GS	SS	GS	SS	GS	SS	GS
Pitch pine	30.0	21.3	18.9	15.3	27.0	23.1	13304	11175	8869	7361
Southern pine (USA)	27.8	19.7	17.5	14.2	25.0	21.4	12526	10522	8351	6931
Parana pine	25.9	18.4	16.3	13.2	23.3	19.9	11035	9269	7357	6106
Douglas-fir-larch (C)	21.6	15.3	13.6	11.0	19.4	16.6	12160	10214	8110	6731
Hem-fir (C)	21.6	15.3	13.6	11.0	19.4	16.6	11822	9930	7953	6601
Redwood/whitewood, Scots pine (BG) and larch (BG)	21.6	15.3	13.6	11.0	19.4	16.6	10561	8871	7021	5827
Corsican pine (BG)	21.6	15.3	13.6	11.0	19.4	16.6	9405	7900	6270	5204
Hem-fir (USA)	21.6	15.3	13.6	11.0	19.4	16.6	10976	9220	7384	6129
Douglas fir-larch (USA)	21.6	15.3	13.6	11.0	19.4	16.6	11115	9337	7413	6153
Spruce-pine-fir (C)	21.6	15.3	13.6	11.0	19.4	16.6	10135	8555	6664	5531
Western whitewoods (USA)	19.1	13.6	12.0	9.7	17.2	14.7	8911	7485	5831	4840
Douglas fir (BG)	17.9	12.7	11.3	9.1	16.1	13.8	11019	9256	7120	5910
Western red cedar	16.5	11.7	10.4	8.4	14.9	12.7	8356	7019	5571	4624
European spruce (BG) Sitka spruce (BG)	16.5	11.7	10.4	8.4	14.9	12.7	7882	6621	5130	4258

Table 14 CHARACTERISTIC STRENGTH VALUES FOR THE CANADIAN NLGA JOIST AND PLANK AND STRUCTURAL LIGHT FRAMING GRADES

PROPERTY	GRADE	SPECIES		
		DOUGLAS FIR-LARCH	HEM-FIR	SPRUCE-PINE-FIR
Bending	Sel	23.1	23.1	23.1
	No 1	16.2	16.2	16.2
	No 2	16.2	16.2	16.2
	No 3	11.9	11.9	11.9
Tension	Sel	16.4	16.4	16.4
	No 1	11.5	11.5	11.5
	No 2	11.5	11.5	11.5
	No 3	-	-	-
Compression Parallel	Sel	21.6	21.6	21.6
	No 1	19.4	19.4	19.4
	No 2	16.9	16.9	16.9
	No 3	13.0	13.0	13.0
Mean E	Sel	12282	11940	10287
	No 1	10944	10640	9167
	No 2	10944	10640	9167
	No 3	10579	10285	8861
Min E	Sel	8516	8351	6997
	No 1	7299	7158	5998
	No 2	7299	7158	5998
	No 3	6488	6362	5331

Table 15 CHARACTERISTIC STRENGTH VALUES FOR THE USA NGRDL JOIST AND PLANK AND STRUCTURAL LIGHT FRAMING GRADES

PROPERTY	GRADE	SPECIES			
		Douglas fir-larch	Hem-fir	Western whitewoods	Southern pine
Bending	Sel	23.1	23.1	20.4	29.8
	No 1	16.2	16.2	14.3	20.9
	No 2	16.2	16.2	14.3	20.9
	No 3	11.9	11.9	10.5	15.3
Tension	Sel	16.4	16.4	14.5	21.1
	No 1	11.5	11.5	10.1	14.7
	No 2	11.5	11.5	10.1	14.7
	No 3	–	–	–	–
Compression parallel	Sel	21.6	21.6	19.1	27.8
	No 1	19.4	19.4	17.2	25.0
	No 2	16.9	16.9	14.9	21.7
	No 3	13.0	13.0	11.5	16.7
Mean E	Sel	11226	11086	9000	12651
	No 1	10004	9878	8020	11273
	No 2	10004	9878	8020	11273
	No 3	9670	9549	7753	10893
Min E	Sel	7734	7753	6123	8769
	No 1	6672	6646	5248	7516
	No 2	6672	6646	5248	7516
	No 3	5930	5907	4665	6681

Table 16 CHARACTERISTIC STRENGTH VALUES FOR THE CANADIAN NLGA LIGHT FRAMING GRADES

PROPERTY	GRADE	SPECIES		
		DOUGLAS FIR-LARCH	HEM-FIR	SPRUCE-PINE-FIR
Bending	Const	13.6	13.6	13.6
	Std	10.2	10.2	10.2
	Util	8.0	8.0	8.0
	Stud	11.9	11.9	11.9
Compression parallel	Const	14.9	14.9	14.9
	Std	11.2	11.2	11.2
	Util	8.2	8.2	8.2
	Stud	13.0	13.0	13.0
Mean E	Const	10322	10522	9065
	Std	10579	10285	8861
	Util	10579	10285	8861
	Stud	10579	10285	8861
Min E	Const	7299	7158	5998
	Std	7056	6919	5798
	Util	7056	6919	5798
	Stud	6894	6760	5664

Table 17   CHARACTERISTIC STRENGTH VALUES FOR THE USA NGRDL LIGHT FRAMING GRADES

PROPERTY	GRADE	SPECIES			
		DOUGLAS FIR-LARCH	HEM-FIR	WESTERN WHITEWOODS	SOUTHERN PINE
Bending	Const Std Util Stud	13.6 10.2 8.0 11.9	13.6 10.2 8.0 11.9	12.0 9.0 7.1 10.5	17.5 13.1 10.3 15.3
Compression parallel	Const Std Util Stud	14.9 11.2 8.2 13.0	14.9 11.2 8.2 13.0	13.2 9.9 7.3 11.5	19.2 14.5 10.6 16.7
Mean E	Const Std Util Stud	9892 9670 9670 9670	9769 9549 9549 9549	7931 7753 7753 7753	11143 10898 10898 10898
Min E	Const Std Util Stud	6672 6449 6449 6301	6646 6424 6424 6276	5248 5073 5073 4956	7516 7265 7265 7098

Table 18 Grade stresses for softwoods: graded to BS 4978 rules: for the dry exposure condition

Standard name	Grade	Bending parallel to grain*	Tension parallel to grain*	Compression		Shear parallel to grain	Modulus of elasticity	
				Parallel to grain	Perpendicular to grain**		Mean	Minimum
		N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
Redwood/whitewood (imported and Scots pine, (British-grown)	SS/MSS	7.5	4.5	7.9	2.1	0.82	10500	7000
	GS/MGS	5.3	3.2	6.8	1.8	0.82	9000	6000
	M75	10.0	6.0	8.7	2.4	1.32	11000	7000
	M50	6.6	4.0	7.3	2.1	0.82	9000	6000
Corsican pine (British-grown)	SS/MSS	7.5	4.5	7.9	2.1	0.82	9500	6500
	GS/MGS	5.3	3.2	6.8	1.8	0.82	8000	5000
	M75	10.0	6.0	8.7	2.4	1.33	10500	7000
	M50	6.6	4.0	7.3	2.0	0.83	9000	5500
Sitka spruce and European spruce (British-grown)	SS/MSS	5.7	3.4	6.1	1.6	0.64	8000	5000
	GS/MGS	4.1	2.5	5.2	1.4	0.64	6500	4500
	M75	6.6	4.0	6.4	1.8	1.02	9000	6000
	M50	4.5	2.7	5.5	1.6	0.64	7500	5000
Douglas fir (British-grown)	SS/MSS	6.2	3.7	6.6	2.4	0.88	11000	7000
	GS/MGS	4.4	2.6	5.6	2.1	0.88	9500	6000
	M75	10.0	6.0	8.7	2.9	1.41	11000	7500
	M50	6.6	4.0	7.3	2.4	0.88	9500	6000
Larch (British-grown)	SS	7.5	4.5	7.9	2.1	0.82	10500	7000
	GS	5.3	3.2	6.8	1.8	0.82	9000	6000
Parana pine (Imported)	SS	9.0	5.4	9.5	2.4	1.03	11000	7500
	GS	6.4	3.8	3.1	2.2	1.03	9500	6000
Pitch pine (Caribbean)	SS	10.5	6.3	11.0	3.2	1.16	13500	9000
	GS	7.4	4.4	9.4	2.8	1.16	11000	7500
Western red cedar (Imported)	SS	5.7	3.4	6.1	1.7	0.63	8500	5500
	GS	4.1	2.5	5.2	1.6	0.63	7000	4500
Douglas fir-larch (Canada)	SS	7.5	4.5	7.9	2.4	0.35	11000	7500
	GS	5.3	3.2	6.8	2.2	0.35	10000	6500
Douglas fir-larch (USA)	SS	7.5	4.5	7.9	2.4	0.85	11000	7500
	GS	5.3	3.2	6.8	2.2	0.85	9500	6000
Hem-fir (Canada)	SS/MSS	7.5	4.5	7.9	1.9	0.68	11000	7500
	GS/MGS	5.3	3.2	6.8	1.7	0.68	9000	6000
	M75	10.0	6.0	9.3	2.4	1.13	12000	8000
	M50	6.6	4.0	7.7	2.1	0.71	10500	7000
Hem-fir (USA)	SS	7.5	4.5	7.9	1.9	0.68	11000	7500
	GS	5.3	3.2	6.8	1.7	0.68	9000	6000
Spruce-pine-fir (Canada)	SS/MSS	7.5	4.5	7.9	1.8	0.68	10000	6500
	GS/MGS	5.3	3.2	6.8	1.6	0.68	8500	5500
	M75	9.7	5.8	8.5	2.1	1.10	10500	7000
	M50	6.2	3.7	7.1	1.8	0.68	9000	5500
Western whitewoods (USA)	SS	6.6	4.0	7.0	1.7	0.66	9000	6000
	GS	4.7	2.8	6.0	1.5	0.66	7500	5000
Southern pine (USA)	SS	9.6	5.8	10.2	2.5	0.98	12500	8500
	GS	6.8	4.1	8.7	2.2	0.98	10500	7000

*Stresses applicable to timber 300 mm deep (or wide):

**When the specifications specifically prohibit wane at bearing areas the SS grade compression perpendicular to the grain stress may be multiplied by 1.33 and used for all grades.

Table 19 Grade stresses for Canadian softwoods: graded to NLGA rules: for the dry exposure condition

Standard name	Grade	Bending parallel to grain	Tension parallel to grain	Compression: Parallel to grain	Compression: Perpendicular to grain‡	Shear parallel to grain	Modulus of elasticity: Mean	Modulus of elasticity: Minimum
		N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
Douglas-fir-larch	J & P*							
	Sel	8.0	6.2	8.8	2.7	0.93	12500	8500
	No 1	5.6	4.3	7.9	2.7	0.93	11000	7500
	No 2	5.6	4.3	6.9	2.4	0.93	11000	7500
	No 3	4.1	—	5.3	1.8	0.61	10500	6500
	SLF**							
	Sel	9.8	7.1	8.8	2.7	0.93	12500	8500
	No 1	6.4	4.9	7.9	2.7	0.93	11000	7500
	No 2	6.4	4.9	6.9	2.4	0.93	11000	7500
	No 3	4.7	—	5.3	1.8	0.61	10500	6500
	LF†							
	Const	5.4	—	6.1	2.7	0.93	11000	7500
	Std	4.0	—	4.6	2.4	0.93	10500	7000
	Util	3.2	—	3.4	1.8	0.61	10500	7000
	STUD†	4.7	—	5.3	1.8	0.61	10500	7000
Hem-fir	J & P*							
	Sel	8.0	6.2	8.8	2.1	0.71	12000	8500
	No 1	5.6	4.3	7.9	2.1	0.71	10500	7000
	No 2	5.6	4.3	6.9	1.8	0.71	10500	7000
	No 3	4.1	—	5.3	1.4	0.47	10500	6500
	SLF**							
	Sel	9.1	7.1	8.8	2.1	0.71	12000	8500
	No 1	6.4	4.9	7.9	2.1	0.71	10500	7000
	No 2	6.4	4.9	6.9	1.8	0.71	10500	7000
	No 3	4.7	—	5.3	1.4	0.47	10500	6500
	LF†							
	Const	5.4	—	6.1	2.1	0.71	10500	7000
	Std	4.0	—	4.6	1.9	0.71	10500	7000
	Util	3.2	—	3.4	1.4	0.47	10500	7000
	STUD†	4.7	—	5.3	1.4	0.47	10500	7000
Spruce-pine-fir	J & P*							
	Sel	8.0	6.2	8.8	1.8	0.68	10500	7000
	No 1	5.6	4.3	7.9	1.8	0.68	9000	6000
	No 2	5.6	4.3	6.9	1.6	0.68	9000	6000
	No 3	4.1	—	5.3	1.2	0.45	9000	5500
	SLF**							
	Sel	9.1	7.1	8.8	1.8	0.68	10500	7000
	No 1	6.4	4.9	7.9	1.8	0.68	9000	6000
	No 2	6.4	4.9	6.9	1.6	0.68	9000	6000
	No 3	4.7	—	5.3	1.2	0.45	9000	5500
	LF†							
	Const	5.4	—	6.1	1.8	0.68	9000	6000
	Std	4.0	—	4.6	1.6	0.68	9000	6000
	Util	3.2	—	3.4	1.2	0.45	9000	6000
	STUD†	4.7	—	5.3	1.2	0.45	9000	5500

*J & P. Joist and Plank Grades: stresses applicable to timber with cross-sectional dimensions greater than or equal to 38 mm × 114 mm.

**SLF. Structural Light Framing Grades: stresses applicable to timber of 38 mm × 89 mm cross section: for other section sizes see Table 21.

†LF. Light Framing and Stud Grades: stresses applicable to timber 38 mm × 89 mm cross section: for other section sizes see Table 21.

‡When the specifications specifically prohibit wane at bearing areas the SS grade compression perpendicular to the grain stress may be multiplied by 1.33 and used for all grades.

Table 20 Grade stresses for USA softwoods: graded to NGRDL rules: for the dry exposure condition

Standard name	Grade	Bending parallel to grain	Tension parallel to grain	Compression Parallel to grain	Compression Perpendicular to grain'	Shear parallel to grain	Modulus of elasticity Mean	Modulus of elasticity Minimum
		N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
Douglas-fir-larch	J & P*							
	Sel	8.0	6.2	8.8	2.4	0.85	11000	8000
	No 1	5.6	4.3	7.9	2.4	0.85	10000	6500
	No 2	5.6	4.3	6.9	2.2	0.85	10000	6500
	No 3	4.1	—	5.3	1.6	0.56	9500	6000
	SLF**							
	Sel	9.1	7.1	8.8	2.4	0.85	11000	8000
	No 1	6.4	4.9	7.9	2.4	0.85	10000	6500
	No 2	6.4	4.9	6.9	2.2	0.85	10000	6500
	No 3	4.7	—	5.3	1.6	0.56	9500	6000
	LF'							
	Const	5.4	—	6.1	2.4	0.85	10000	6500
	Std	4.0	—	4.6	2.2	0.85	9500	6500
	Util	3.2	—	3.4	1.6	0.56	9500	6500
	STUD'	4.7	—	5.3	1.6	0.56	9500	6500
Hem-fir	J & P*							
	Sel	8.0	6.2	8.8	1.9	0.68	11000	8000
	No 1	5.6	4.3	7.9	1.9	0.68	10000	6500
	No 2	5.6	4.3	6.9	1.7	0.68	10000	6500
	No 3	4.1	—	5.3	1.3	0.45	9500	6000
	SLF**							
	Sel	9.1	7.1	8.8	1.9	0.68	11000	8000
	No 1	6.4	4.9	7.9	1.9	0.68	10000	6500
	No 2	6.4	4.9	6.9	1.7	0.68	10000	6500
	No 3	4.7	—	5.3	1.3	0.45	9500	6000
	LF'							
	Const	5.4	—	6.1	1.9	0.68	10000	6500
	Std	4.0	—	4.6	1.7	0.68	9500	6500
	Util	3.2	—	3.4	1.3	0.45	9500	6500
	STUD'	4.7	—	5.3	1.3	0.45	9500	6500
Western whitewoods	J & P*							
	Sel	7.1	5.5	7.8	1.7	0.66	9000	6000
	No 1	5.0	3.8	7.0	1.7	0.66	8000	5000
	No 2	5.0	3.8	6.1	1.5	0.66	8000	5000
	No 3	3.6	—	4.7	1.1	0.44	8000	4500
	SLF**							
	Sel	8.1	6.3	7.8	1.7	0.66	9000	6000
	No 1	5.7	4.3	7.0	1.7	0.66	8000	5000
	No 2	5.7	4.3	6.1	1.5	0.66	8000	5000
	No 3	4.1	—	4.7	1.1	0.44	8000	4500
	LF'							
	Const	4.8	—	5.4	1.7	0.66	8000	5000
	Std	3.5	—	4.0	1.5	0.66	8000	5000
	Util	2.8	—	3.0	1.1	0.44	8000	5000
	STUD'	4.1	—	4.7	1.1	0.44	8000	5000
Southern pine	J & P*							
	Sel	10.3	8.0	11.3	2.5	0.98	12500	9000
	No 1	7.2	5.5	10.2	2.5	0.98	11500	7500
	No 2	7.2	5.5	8.9	2.2	0.98	11500	7500
	No 3	5.3	—	6.8	1.7	0.64	11000	6500
	SLF**							
	Sel	11.7	9.1	11.3	2.5	0.98	12500	9000
	No 1	8.2	6.3	10.2	2.5	0.98	11500	7500
	No 2	8.2	6.3	8.9	2.2	0.98	11500	7500
	No 3	6.0	—	6.8	1.7	0.64	11000	6500
	LF'							
	Const	6.9	—	7.8	2.5	0.98	11000	7500
	Std	5.1	—	5.9	2.2	0.98	11000	7500
	Util	4.1	—	4.3	1.7	0.64	11000	7500
	STUD'	6.0	—	6.8	1.7	0.64	11000	7000

*J & P. Joist and Plant Grades: stresses applicable to timber with cross-sectional dimensions greater than or equal to 38 mm × 114 mm.

**SLF. Structural Light Framing Grades: stresses applicable to timber of 38 mm × 89 mm cross section: for other section sizes see Table 21.

'LF. Light Framing and Stud Grades: stresses applicable to timber 38 mm × 89 mm cross section: for other section sizes see Table 21.

'When the specifications specifically prohibit wane at bearing areas the SS grade compression perpendicular to the grain stress may be multiplied by 1.33 and used for all grades.

TABLE 21. MODIFICATION FACTORS BY WHICH THE STRESSES IN TABLES 19 AND 20 SHOULD BE MULTIPLIED FOR SECTIONS OTHER THAN 38 mm x 89 mm

Section size mm	Grade	Bending parallel to grain K_7	Tension parallel to grain K_{14}	Compression parallel to grain K_{10}	Modulus of elasticity K_{11}
38 x 38	SLF				
	Sel	1.10	1.05	1.05	1.0
	No. 1	1.10	1.05	1.20	1.0
	No. 2	1.10	1.05	1.35	1.0
	No. 3	1.10	–	1.55	1.0
	LF				
	Const	0.90	–	1.0	1.0
	Std	0.75	–	1.0	1.0
	Util	0.50	–	0.65	1.0
	STUD	1.10	–	1.55	1.0
38 x 63	SLF				
	Sel	1.05	1.05	1.05	1.0
	No. 1	1.05	1.05	1.20	1.0
	No. 2	1.05	1.05	1.35	1.0
	No. 3	1.05	–	1.55	1.0
	LF				
	Const	0.80	–	1.0	1.0
	Std	0.95	–	1.0	1.0
	Util	0.60	–	0.75	1.0
	STUD	1.05	–	1.55	1.0
63 x 63	SLF				
	Sel	1.05	0.75	1.05	1.0
	No. 1	1.05	0.30	1.20	1.0
	No. 2	1.05	0.30	1.35	1.0
	No. 3	1.05	–	1.55	1.0
	LF				
	Const	0.80	–	1.0	1.0
	Std	0.95	–	1.0	1.0
	Util	0.60	–	0.75	1.0
	STUD	1.05	–	1.55	1.0
63 x 89 and 89 x 89	SLF				
	Sel	0.95	0.75	1.0	1.0
	No. 1	0.40	0.30	1.0	0.80
	No. 2	0.40	0.30	1.0	0.80
	No. 3	0.35	–	1.0	1.0
	STUD	0.35	–	1.0	1.0
38 x 114 and 38 x 140	STUD	0.95	–	1.05	1.0

NOTE. For grades and properties not listed and for 38 mm x 89 mm cross sections the modification factor has the value 1.0; that is, no modification of the tabulated stresses is required.

Table 22 (Sheet 1) SPECIES AND GRADES RANKED IN ORDER OF BENDING STRENGTH

Species	Grade	Bending N/mm^2	Comp Par N/mm^2	Shear N/mm^2	Mean E N/mm^2	Min E N/mm^2
Southern pine (USA)	SEL-SLF	11.7	11.3	0.98	12651	8769
Pitch pine	SS	10.5	11.0	1.16	13304	8869
Southern pine (USA)	SEL-JP	10.3	11.3	0.98	12651	8769
Hem-fir (CAN)	M75	10.0	9.3	1.13	12000	8000
R/W, Scots pine & larch	M75	10.0	8.7	1.32	11000	7000
BG Douglas fir	M75	10.0	8.7	1.41	11000	7500
BG Corsican pine	M75	10.0	8.7	1.33	10500	7000
Douglas fir-larch (CAN)	SEL-SLF	9.8	8.8	0.93	12282	8516
Spruce-pine-fir (CAN)	M75	9.7	8.5	1.10	10500	7000
Southern pine (USA)	SS	9.6	10.2	0.98	12526	8351
Hem-fir (CAN)	SEL-SLF	9.1	8.8	0.71	11940	8351
Spruce-pine-fir (CAN)	SEL-SLF	9.1	8.8	0.68	10287	6997
Douglas fir-larch (USA)	SEL-SLF	9.1	8.8	0.85	11226	7784
Hem-fir (USA)	SEL-SLF	9.1	8.8	0.68	11086	7753
Parana pine	SS	9.0	9.5	1.03	11035	7357
Southern pine (USA)	No 1-SLF	8.2	10.2	0.98	11273	7516
Southern pine (USA)	No 2-SLF	8.2	8.9	0.98	11273	7516
W Whitewood (USA)	SEL-SLF	8.1	7.8	0.66	9000	6123
Douglas fir-larch (CAN)	SEL-JP	8.0	8.8	0.93	12282	8516
Hem-fir (CAN)	SEL-JP	8.0	8.8	0.71	11940	8351
Spruce-pine-fir (CAN)	SEL-JP	8.0	8.8	0.68	10287	6997
Douglas fir-larch (USA)	SEL-JP	8.0	8.8	0.85	11226	7784
Hem-fir (USA)	SEL-JP	8.0	8.8	0.68	11086	7753
Douglas fir-larch (CAN)	SS	7.5	7.9	0.85	12160	8110
Douglas fir-larch (USA)	SS	7.5	7.9	0.85	11115	7413
Hem-fir (CAN)	SS	7.5	7.9	0.68	11822	7953
Hem-fir (USA)	SS	7.5	7.9	0.68	10976	7384
R/W, Scots pine & larch	SS	7.5	7.9	0.82	10561	7021
BG Corsican pine	SS	7.5	7.9	0.82	9405	6270
Spruce-pine-fir (CAN)	SS	7.5	7.9	0.68	10185	6664
Pitch pine	GS	7.4	9.4	1.16	11175	7361
Southern pine (USA)	No 1-JP	7.2	10.2	0.98	11273	7516
Southern pine (USA)	No 2-JP	7.2	8.9	0.98	11273	7516
W Whitewood (USA)	SEL-JP	7.1	7.8	0.66	9000	6123

Table 22 (Sheet 2) SPECIES AND GRADES RANKED IN ORDER OF BENDING STRENGTH

Species	Grade	Bending N/mm^2	Comp Par N/mm^2	Shear N/mm^2	Mean E N/mm^2	Min E N/mm^2
Southern pine (USA)	CONST	6.9	7.8	0.98	11148	7516
Southern pine (USA)	GS	6.8	8.7	0.98	10522	6931
W Whitewood (USA)	SS	6.6	7.0	0.66	8911	5831
R/W, Scots pine & larch	M50	6.6	7.3	0.82	9000	6000
BG Douglas fir	M50	6.6	7.3	0.88	9500	6000
BG Sitka & Euro-Spruce	M75	6.6	6.4	1.02	9000	6000
Hem-fir (CAN)	M50	6.6	7.7	0.71	10500	7000
BG Corsican pine	M50	6.6	7.3	0.83	9000	5500
Parana pine	GS	6.4	8.1	1.03	9269	6106
Douglas fir-larch (CAN)	No 1-SLF	6.4	7.9	0.93	10944	7299
Douglas fir-larch (CAN)	No 2-SLF	6.4	6.9	0.93	10944	7299
Hem-fir (CAN)	No 1-SLF	6.4	7.9	0.71	10640	7158
Hem-fir (CAN)	No 2-SLF	6.4	6.9	0.71	10640	7158
Spruce-pine-fir (CAN)	No 1-SLF	6.4	7.9	0.68	9167	5998
Spruce-pine-fir (CAN)	No 2-SLF	6.4	6.9	0.68	9167	5998
Douglas fir-larch (USA)	No 1-SLF	6.4	7.9	0.85	10004	6672
Douglas fir-larch (USA)	No 2-SLF	6.4	6.9	0.85	10004	6672
Hem-fir (USA)	No 1-SLF	6.4	7.9	0.68	9878	6646
Hem-fir (USA)	No 2-SLF	6.4	6.9	0.68	9878	6646
BG Douglas fir	SS	6.2	6.6	0.88	11019	7120
Spruce-pine-fir (CAN)	M50	6.2	7.1	0.68	9000	5500
Southern pine (USA)	STUD	6.0	6.8	0.64	10898	7098
Southern pine (USA)	No 3-SLF	6.0	6.8	0.64	10898	6681
W red cedar	SS	5.7	6.1	0.63	8356	5571
BG Sitka & Euro-spruce	SS	5.7	6.1	0.64	7882	5130
W Whitewood (USA)	No 1-SLF	5.7	7.0	0.66	8020	5248
W Whitewood (USA)	No 2-SLF	5.7	6.1	0.66	8020	5248
Douglas fir-larch (CAN)	No 1-JP	5.6	7.9	0.93	10944	7299
Douglas fir-larch (CAN)	No 2-JP	5.6	6.9	0.93	10944	7299
Spruce-pine-fir (CAN)	No 1-JP	5.6	7.9	0.68	9167	5998
Spruce-pine-fir (CAN)	No 2-JP	5.6	6.9	0.68	9167	5998
Hem-fir (CAN)	No 1-JP	5.6	7.9	0.71	10640	7158
Hem-fir (CAN)	No 2-JP	5.6	6.9	0.71	10640	7158
Hem-fir (USA)	No 1-JP	5.6	7.9	0.68	9878	6646
Hem-fir (USA)	No 2-JP	5.6	6.9	0.68	9878	6646
Douglas fir-larch (USA)	No 1-JP	5.6	7.9	0.85	10004	6672

Table 22 (Sheet 3) SPECIES AND GRADES RANKED IN ORDER OF BENDING STRENGTH

Species	Grade	Bending N/mm^2	Comp Par N/mm^2	Shear N/mm^2	Mean E N/mm^2	Min E N/mm^2
Douglas fir-larch (USA)	No 2-JP	5.6	6.9	0.85	10004	6672
Douglas fir-larch (USA)	CONST	5.4	6.1	0.85	9892	6672
Hem-fir (USA)	CONST	5.4	6.1	0.68	9769	6646
Spruce-pine-fir (CAN)	CONST	5.4	6.1	0.68	9065	5998
Hem-fir (CAN)	CONST	5.4	6.1	0.71	10522	7158
Douglas fir-larch (CAN)	CONST	5.4	6.1	0.93	10822	7299
Southern pine (USA)	No 3-JP	5.3	6.8	0.64	10898	6681
Spruce-pine-fir (CAN)	GS	5.3	6.8	0.68	8555	5531
R/W, Scots pine & larch	GS	5.3	6.8	0.82	8871	5827
G Corsican pine	GS	5.3	6.8	0.82	7900	5204
Hem-fir (USA)	GS	5.3	6.8	0.68	9220	6129
Hem-fir (CAN)	GS	5.3	6.8	0.68	9930	6601
Douglas fir-larch (USA)	GS	5.3	6.8	0.85	9337	6153
Douglas fir-larch (CAN)	GS	5.3	6.8	0.85	10214	6731
Southern pine (USA)	STAND	5.1	5.9	0.98	10898	7265
W Whitewood (USA)	No 1-JP	5.0	7.0	0.66	8020	5248
W Whitewood (USA)	No 2-JP	5.0	6.1	0.66	8020	5248
W Whitewood (USA)	CONST	4.8	5.4	0.66	7931	5248
W Whitewood (USA)	GS	4.7	6.0	0.66	7485	4840
Hem-fir (USA)	STUD	4.7	5.3	0.45	9549	6276
Douglas fir-larch (USA)	STUD	4.7	5.3	0.56	9670	6301
Spruce-pine-fir (CAN)	STUD	4.7	5.3	0.45	8861	5664
Hem-fir (CAN)	STUD	4.7	5.3	0.47	10285	6760
Douglas fir-larch (CAN)	STUD	4.7	5.3	0.61	10579	6894
Douglas fir-larch (CAN)	No 3-SLF	4.7	5.3	0.61	10579	6488
Hem-fir (CAN)	No 3-SLF	4.7	5.3	0.47	10285	6362
Spruce-pine-fir (CAN)	No 3-SLF	4.7	5.3	0.45	8861	5331
Douglas fir-larch (USA)	No 3-SLF	4.7	5.3	0.56	9670	5930
Hem-fir (USA)	No 3-SLF	4.7	5.3	0.45	9549	5907
BG Sitka & Euro-spruce	M50	4.5	5.5	0.64	7700	5000
BG Douglas fir	GS	4.4	5.6	0.88	9256	5910
Hem-fir (USA)	No 3-JP	4.1	5.3	0.45	9549	5907
Douglas fir-larch (USA)	No 3-JP	4.1	5.3	0.56	9670	5930
BG Sitka & Euro-spruce	GS	4.1	5.2	0.64	6621	4258

Table 22 (Sheet 4) SPECIES AND GRADES RANKED IN ORDER OF BENDING STRENGTH

Species	Grade	Bending N/mm^2	Comp Par N/mm^2	Shear N/mm^2	Mean E N/mm^2	Min E N/mm^2
W red cedar	GS	4.1	5.2	0.63	7019	4624
Spruce-pine-fir (CAN)	No 3-JP	4.1	5.3	0.45	8861	5331
Hem-fir (CAN)	No 3-JP	4.1	5.3	0.47	10285	6362
Douglas fir-larch (CAN)	No 3-JP	4.1	5.3	0.61	10579	6488
Southern pine (USA)	UTIL	4.1	4.3	0.64	10898	7265
W Whitewood (USA)	STUD	4.1	4.7	0.44	7753	4956
W Whitewood (USA)	No 3-SLF	4.1	4.7	0.44	7753	4665
Hem-fir (USA)	STAND	4.0	4.6	0.68	9549	6424
Douglas fir-larch (USA)	STAND	4.0	4.6	0.85	9670	6449
Douglas fir-larch (CAN)	STAND	4.0	4.6	0.93	10579	7056
Hem-fir (CAN)	STAND	4.0	4.6	0.71	10285	6919
Spruce-pine-fir (CAN)	STAND	4.0	4.6	0.68	8861	5798
W Whitewood (USA)	No 3-JP	3.6	4.7	0.44	7753	4665
W Whitewood (USA)	STAND	3.5	4.0	0.66	7753	5073
Hem-fir (USA)	UTIL	3.2	3.4	0.45	9549	6424
Douglas fir-larch (USA)	UTIL	3.2	3.4	0.56	9670	6449
Spruce-pine-fir (CAN)	UTIL	3.2	3.4	0.45	8861	5798
Hem-fir (CAN)	UTIL	3.2	3.4	0.47	10285	6919
Douglas fir-larch (CAN)	UTIL	3.2	3.4	0.61	10579	7056
W Whitewood (USA)	UTIL	2.8	3.0	0.44	7753	5073

TABLE 23 GRADE STRESSES AND MODULI OF ELASTICITY FOR STRENGTH CLASSES: FOR THE DRY EXPOSURE CONDITION

Strength class	Bending parallel to grain	Tension parallel to grain	Compression parallel to grain	Compression perpendicular to grain*		Shear parallel to grain	Modulus of elasticity	
					Grade		Mean	Minimum
	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)
SC1	2.8	2.2 ‡	3.5	2.1	1.2	0.46	6800	4500
SC2	4.1	2.5 ‡	5.3	2.1	1.6	0.66	8000	5000
SC3	5.3	3.2 ‡	6.8	2.2	1.7	0.67	8800	5800
SC4	7.5	4.5 ‡	7.9	2.4	1.9	0.71	9900	6600
SC5	10.0	6.0 ‡	8.7	2.8	2.4	1.00	10700	7100
SC6§	12.5	7.5	12.5	3.8	2.8	1.50	14100	11800
SC7§	15.0	9.0	14.5	4.4	3.3	1.75	16200	13600
SC8§	17.5	10.5	16.5	5.2	3.9	2.00	18700	15600
SC9§	20.5	12.3	19.5	6.1	4.6	2.25	21600	18000

*When the specification specifically prohibits wane at bearing areas the higher values of compression perpendicular to the grain stress may be used, otherwise the lower values apply.

‡Note the Light Framing, Stud, Structural Light Framing No. 3 and Joist and Plank No. 3 grades should not be used for tension members.

§ Classes SC6, SC7, SC8 and SC9 will usually comprise the denser hardwoods.

TABLE 24 SOFTWOOD SPECIES/GRADE COMBINATIONS WHICH SATISFY THE REQUIREMENTS FOR STRENGTH CLASSES: GRADED TO BS 4978

Standard name	Strength class				
	SC1	SC2	SC3	SC4	SC5
Imported					
Parana Pine			GS	SS	
Pitch Pine (Caribbean)			GS		SS
Redwood			GS/M50	SS	M75
Whitewood			GS/M50	SS	M75
Western Red Cedar	GS	SS			
Douglas Fir-Larch (Canada)			GS	SS	
Douglas Fir-Larch (USA)			GS	SS	
Hem-Fir (Canada)			GS/M50	SS	M75
Hem-Fir (USA)			GS	SS	
Spruce-Pine-Fir (Canada)			GS/M50	SS/M75	
Western Whitewoods (USA)	GS		SS		
Southern Pine (USA)			GS	SS	
British grown					
Douglas Fir		GS	M50/SS		M75
Larch			GS	SS	
Scots Pine			GS/M50	SS	M75
Corsican Pine		GS	M50	SS	M75
European Spruce	GS	M50/SS	M75		
Sitka Spruce	GS	M50/SS	M75		

Machine grades MGS and MSS are interchangeable with GS and SS grades respectively.

A species/grade combination from a higher strength class may be used where a lower strength class is specified.

TABLE 25 NORTH AMERICAN SOFTWOOD SPECIES/GRADE COMBINATIONS WHICH SATISFY THE REQUIREMENTS FOR STRENGTH CLASSES: NLGA AND NGRDL JOIST AND PLANK GRADES

Standard name and origin	Strength class				
	SC1	SC2	SC3	SC4	SC5
Douglas Fir-Larch (Canada)	No. 3		No. 1, No. 2	Sel	
Douglas Fir-Larch (USA)	No. 3		No. 1, No. 2	Sel	
Hem-Fir (Canada)	No. 3		No. 1, No. 2	Sel	
Hem-Fir (USA)	No. 3		No. 1, No. 2	Sel	
Spruce-Pine-Fir (Canada)	No. 3		No. 1, No. 2	Sel	
Western Whitewoods (USA)	No. 3	No. 1, No. 2	Sel		
Southern Pine (USA)			No. 1, No. 2, No. 3		Sel

A species/grade combination from a higher strength class may be used where a lower strength class is specified.

These classifications apply only to timber of a size not less than 38 mm x 114 mm.

Note that Joist and Plank No. 3 grade should not be used for tension members.

Table 26 North American softwood species/grades which satisfy the requirements for strength classes: NLGA and NGRDL structural light framing grades

Section size Actual	Standard name and origin	Strength class SC1	SC2	SC3	SC4	SC5
38 mm × 89 mm	Douglas fir-larch (Canada)	No 3	—	No 1, No 2	Sel	—
	Douglas fir-larch (USA)	No 3	—	No 1, No 2	Sel	—
	Hem-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Hem-fir USA	No 3	—	No 1, No 2	Sel	—
	Spruce-pine-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Western Whitewoods (USA)	No 3	No 1, No 2	Sel	—	—
	Southern pine (USA)	—	—	No 1, No 2, No 3	—	Sel
38 mm × 38 mm	Douglas fir-larch (Canada)	No 3	—	No 1, No 2	Sel	—
	Douglas fir-larch (USA)	No 3	—	No 1, No 2	Sel	—
	Hem-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Hem-fir (USA)	No 3	—	No 1, No 2	Sel	—
	Spruce-pine-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Western Whitewoods (USA)	No 3	No 1, No 2	Sel	—	—
	Southern pine (USA)	—	—	No 3	No 1, No 2	Sel
38 mm × 63 mm	Douglas fir-larch (Canada)	No 3	—	No 1, No 2	Sel	—
	Douglas fir-larch (USA)	No 3	—	No 1, No 2	Sel	—
	Hem-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Hem-fir (USA)	No 3	—	No 1, No 2	Sel	—
	Spruce-pine-fir (Canada)	No 3	—	No 1, No 2	Sel	—
	Western Whitewoods (USA)	No 3	No 1, No 2	Sel	—	—
	Southern pine (USA)	—	—	No 3	No 1, No 2	Sel
63 mm × 63 mm	Douglas fir-larch (Canada)	No 3	—	Sel	—	—
	Douglas fir-larch (USA)	No 3	—	Sel	—	—
	Hem-fir (Canada)	No 3	—	Sel	—	—
	Hem-fir (USA)	No 3	—	Sel	—	—
	Spruce-pine-fir (Canada)	No 3	—	Sel	—	—
	Western Whitewoods (USA)	No 3	—	Sel	—	—
	Southern pine (USA)	—	—	No 3	—	Sel
63 mm × 89 mm and 89 mm × 89 mm	Douglas fir-larch (Canada)	—	—	Sel	—	—
	Douglas fir-larch (USA)	—	—	Sel	—	—
	Hem-fir (Canada)	—	—	Sel	—	—
	Hem-fir (USA)	—	—	Sel	—	—
	Spruce-pine-fir (Canada)*	—	—	Sel	—	—
	Western Whitewoods (USA)	—	—	Sel	—	—
	Southern pine (USA)	—	—	—	Sel	—

A species/grade combination from a higher strength class may be used where a lower strength class is specified.

Note that Structural Light Framing No 3 grade should not be used for tension members.

The size modification factors from Table 21 are included in the stresses used to assign these species/grades combinations to the strength classes.

Table 27 North American softwood species/grades which satisfy the requirements for strength classes: NLGA and NGRDL light framing and stud grades

Section size Actual	Standard name and origin	Strength class SC1	SC2	SC3	SC4	SC5
38 mm × 89 mm	Douglas fir-larch (Canada)	Std, Stud, Util	Const	—	—	—
	Douglas fir-larch (USA)	Std, Stud, Util	Const	—	—	—
	Hem-fir (Canada)	Std, Stud, Util	Const	—	—	—
	Hem-fir USA	Std, Stud, Util	Const	—	—	—
	Spruce-pine-fir (Canada)	Std, Stud, Util	Const	—	—	—
	Western Whitewoods (USA)	Std, Stud	Const	—	—	—
	Southern pine (USA)	Util	Std	Const, Stud	—	—
38 mm × 38 mm	Douglas fir-larch (Canada)	Stud	Const	—	—	—
	Douglas fir-larch (USA)	Stud	Const	—	—	—
	Hem-fir (Canada)	Stud	Const	—	—	—
	Hem-fir (USA)	Stud	Const	—	—	—
	Spruce-pine-fir (Canada)	Stud	Const	—	—	—
	Western Whitewoods (USA)	Const, Stud	—	—	—	—
	Southern pine (USA)	Std	—	Const, Stud	—	—
38 mm × 63 mm	Douglas fir-larch (Canada)	Const, Std, Stud	—	—	—	—
	Douglas fir-larch (USA)	Const, Std, Stud	—	—	—	—
	Hem-fir (Canada)	Const, Std, Stud	—	—	—	—
	Hem-fir (USA)	Const, Std, Stud	—	—	—	—
	Spruce-pine-fir (Canada)	Const, Std, Stud	—	—	—	—
	Western Whitewoods (USA)	Const, Std, Stud	—	—	—	—
	Southern pine (USA)	—	Const, Std	Stud	—	—
63 mm × 63 mm	Douglas fir-larch (Canada)	Const, Std, Stud	—	—	—	—
	Douglas fir-larch (USA)	Const, Std, Stud	—	—	—	—
	Hem-fir (Canada)	Const, Std, Stud	—	—	—	—
	Hem-fir (USA)	Const, Std, Stud	—	—	—	—
	Spruce-pine-fir (Canada)	Const, Std, Stud	—	—	—	—
	Western Whitewoods (USA)	Const, Std, Stud	—	—	—	—
	Southern pine (USA)	—	Const, Std	Stud	—	—
63 mm × 89 mm	Douglas fir-larch (Canada)	Std, Util	Const	—	—	—
	Douglas fir-larch (USA)	Std, Util	Const	—	—	—
89 mm × 89 mm	Hem-fir (Canada)	Std, Util	Const	—	—	—
	Hem-fir (USA)	Std, Util	Const	—	—	—
	Spruce-pine-fir (Canada)	Std, Util	Const	—	—	—
	Western Whitewoods (USA)	Std	Const	—	—	—
	Southern pine (USA)	Util	Std	Const	—	—
38 mm × 114 mm*	Douglas fir-larch (Canada)	Stud	—	—	—	—
and	Douglas fir-larch (USA)	Stud	—	—	—	—
38 mm × 140 mm*	Hem-fir (Canada)	Stud	—	—	—	—
	Hem-fir (USA)	Stud	—	—	—	—
	Spruce-pine-fir (Canada)	Stud	—	—	—	—
	Western Whitewoods (USA)	Stud	—	—	—	—
	Southern pine (USA)	—	Stud	—	—	—

A species/grade combination from a higher strength class may be used where a lower strength class is specified.

*Available in Stud grade only.

Note that Light Framing and Stud grades should not be used for tension members.

The size modification factors from Table 21 are included in the stresses used to assign these species/grades combinations to the strength classes.

Table 28 MODIFICATION FACTORS TO BE APPLIED TO SS GRADE STRESSES FOR SINGLE GRADE LAMINATED MEMBERS, AND HORIZONTALLY LAMINATED BEAMS

LAMINATION GRADE	NUMBER OF LAMINATIONS	VALUES OF MODIFICATION FACTORS					
		Bending	Tension	Compression par	Compression perp	Shear	Modulus of Elasticity[1]
LA	4 or more	1.85	1.85	1.15	1.33	2.00	1.00
LB	4	1.26	1.26	1.04	1.33	2.00	0.90
	5	1.34	1.34				
	7	1.39	1.39				
	10	1.43	1.43				
	15	1.48	1.48				
	20 or more	1.52	1.52				
LC	4	0.74	0.74	0.92	1.33	2.00	0.80
	5	0.82	0.82				
	7	0.91	0.91				
	10	0.98	0.98				
	15	1.05	1.05				
	20 or more	1.11	1.11				

[1] applied to the mean value.

Table 29 MODIFICATION FACTORS TO BE APPLIED TO SS GRADE STRESSES FOR COMBINED GRADE LAMINATED MEMBERS, AND HORIZONTALLY LAMINATED BEAMS

GRADE COMBINATION	NUMBER OF LAMINATIONS	VALUES OF MODIFICATION FACTORS					
		Bending	Tension	Compression par	Compression perp	Shear	Modulus of Elasticity[1]
LA/LB/LA	4 or more	1.76	1.76	1.09	1.33	2.00	0.93
LB/LC/LB	4	1.26	1.26	0.97	1.33	2.00	0.81
	8	1.32	1.32				
	12	1.37	1.37				
	16	1.41	1.41				
	20 or more	1.45	1.45				

Note. For these factors to apply the outer laminations in not less than 25 per cent of the cross-section at both edges should be of the higher grade.

[1] applied to the mean value.

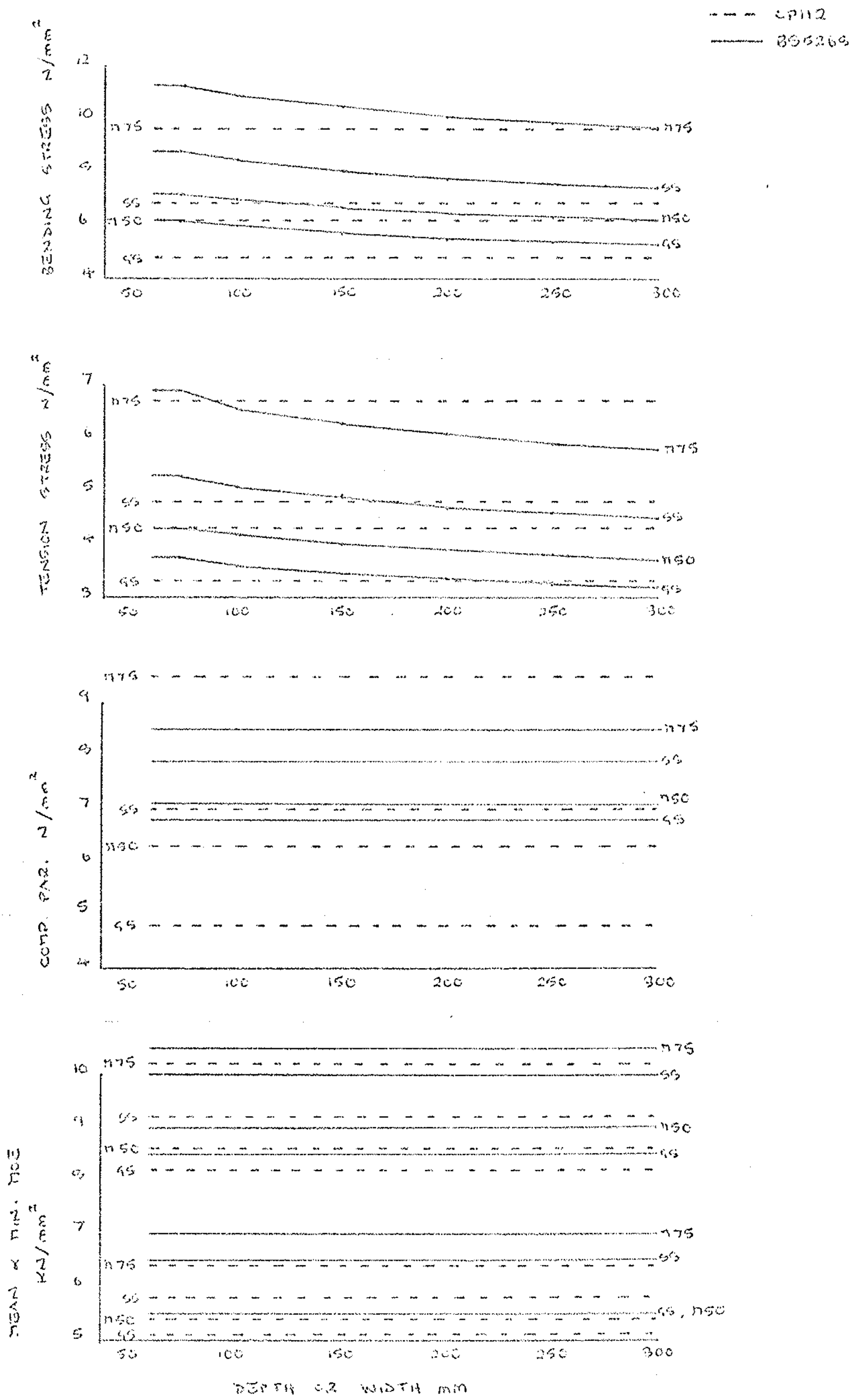


FIG. 1

COMPARISON OF MAJOR STRENGTH PROPERTIES FROM CP112 & BS5268 FOR SPRUCE - PINE - FIR - BS4978 GRADES

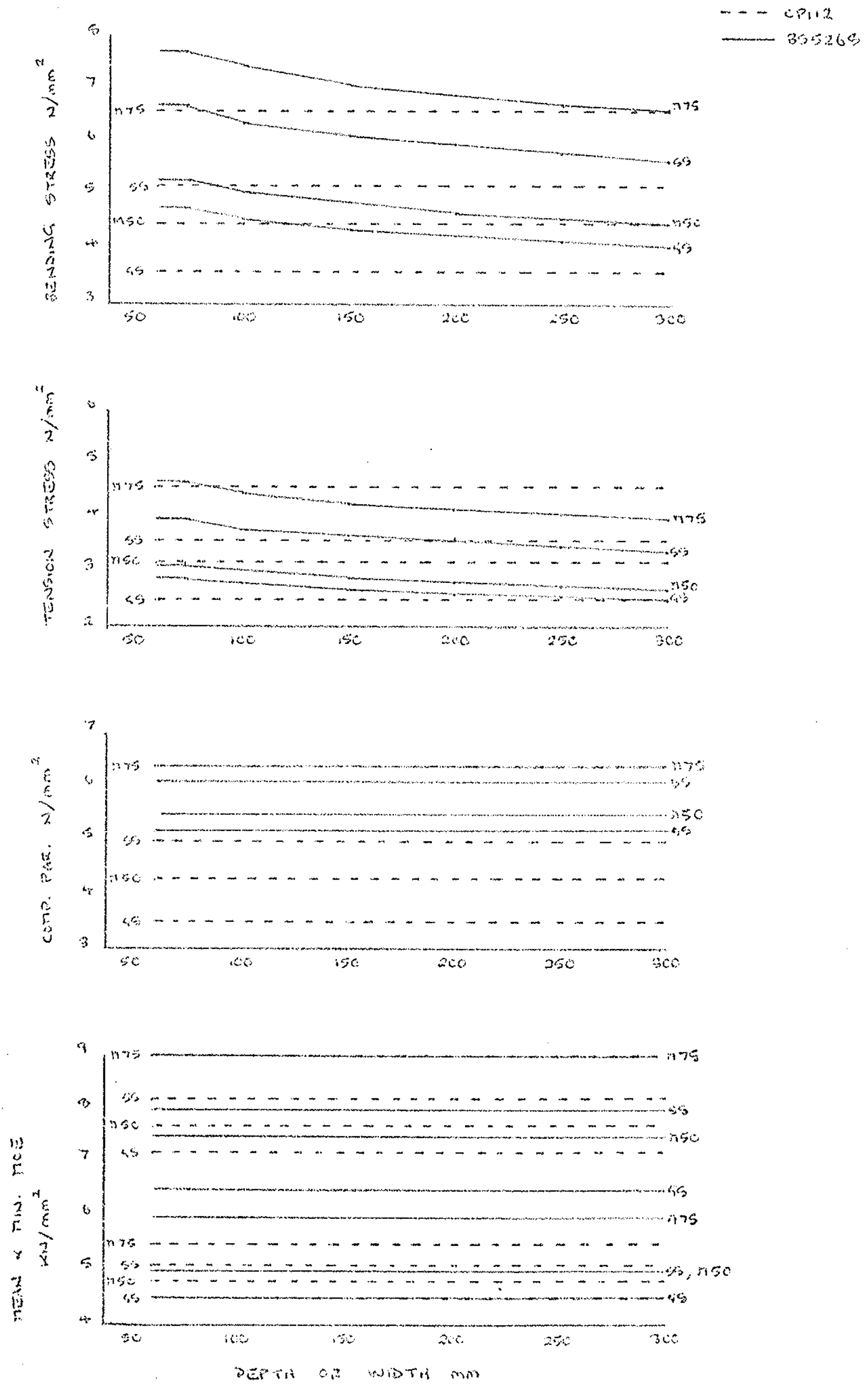
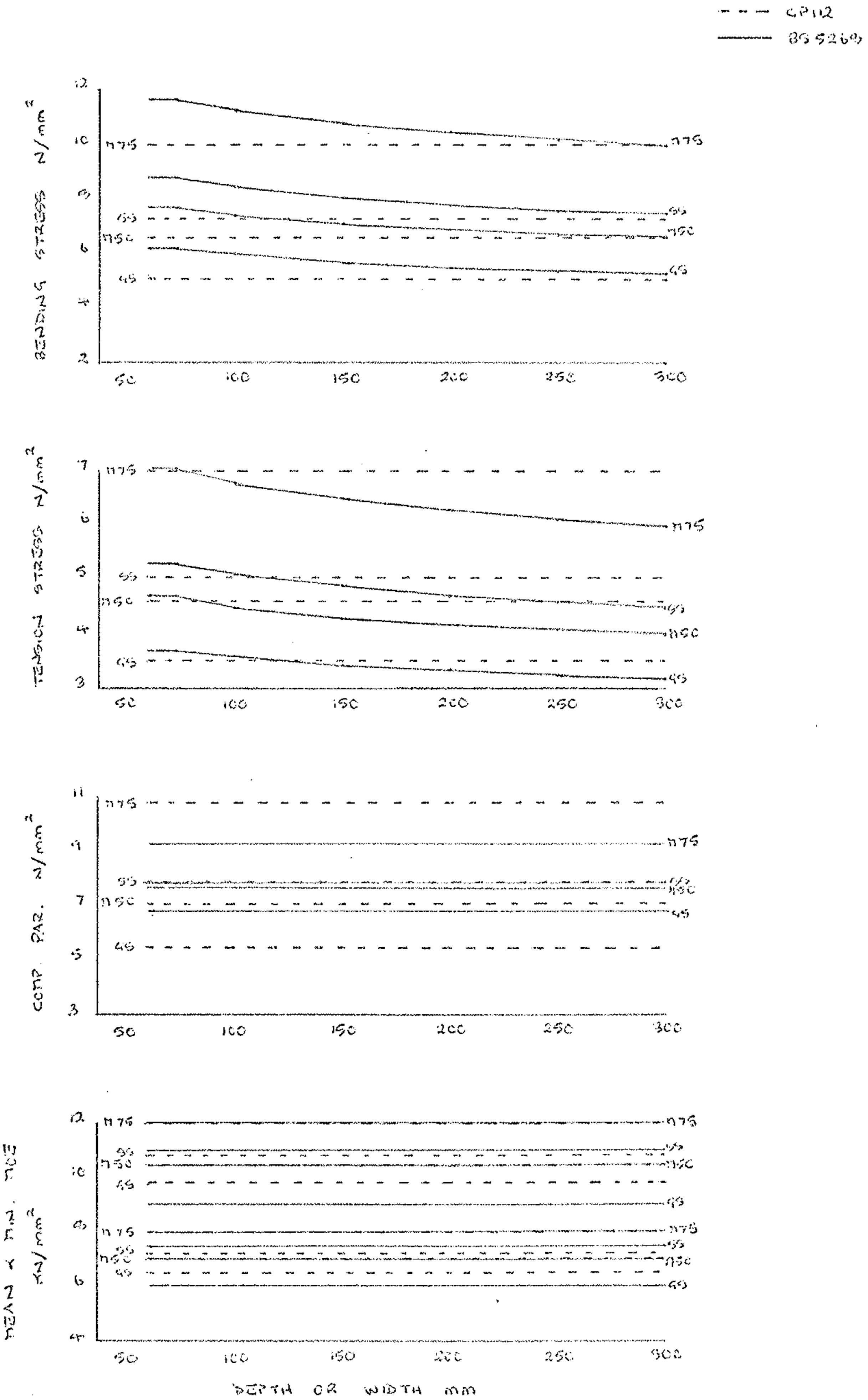


FIG. 2

COMPARISON OF MAJOR STRENGTH PROPERTIES FROM CP112 & BS5268 FOR SITKA SPRUCE - BS4978 GRADES



COMPARISON OF MAJOR STRENGTH PROPERTIES FROM
FIG. 3 CP112 & BS5268 FOR WESTERN HEMLOCK – BS4978 GRADES

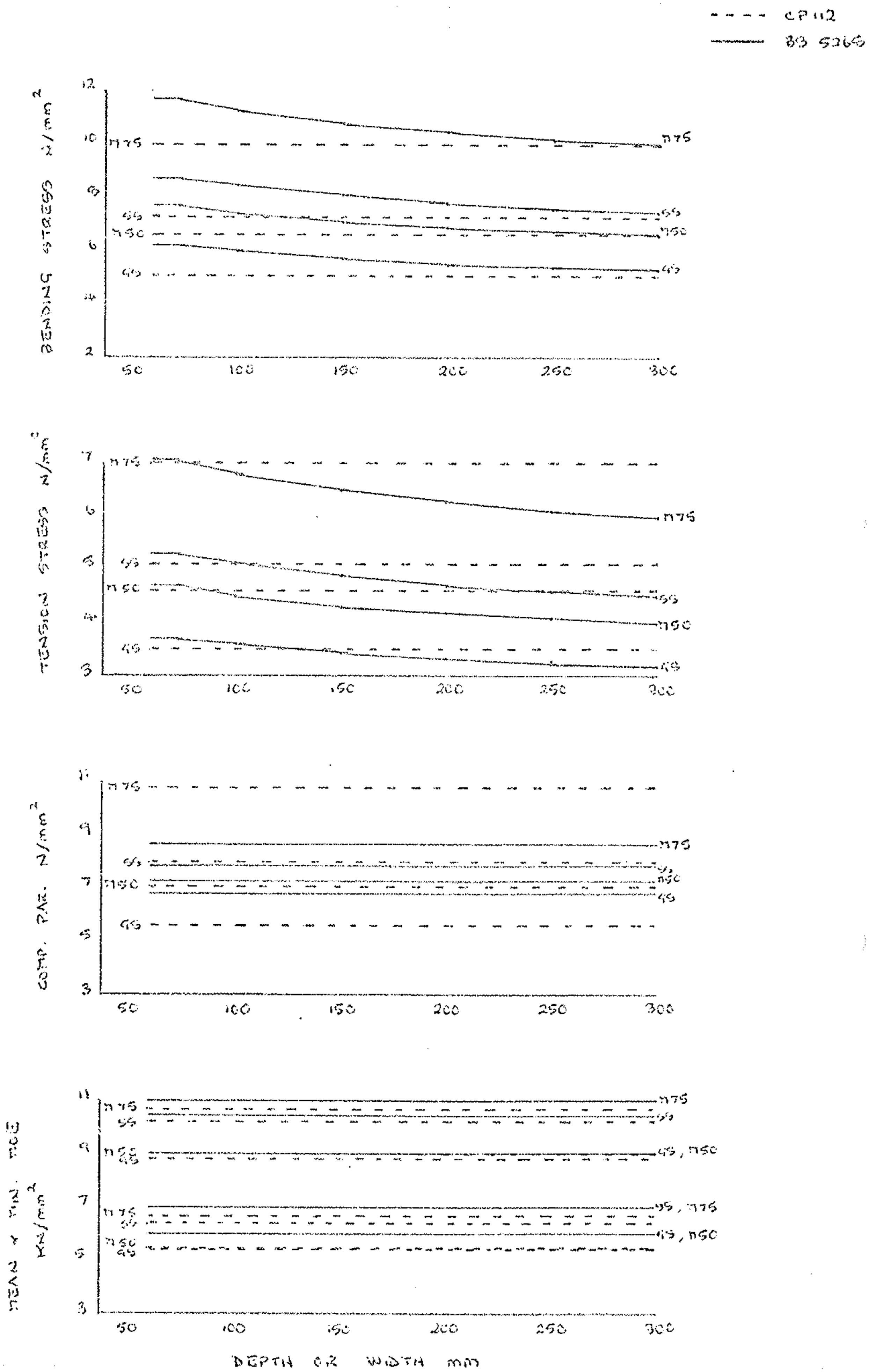


FIG. 4

COMPARISON OF MAJOR STRENGTH PROPERTIES FROM CP112 & BS5268 FOR EUROPEAN REDWOOD / WHITEWOOD – BS4978 GRADES

- - - CP112
——— BS 5268

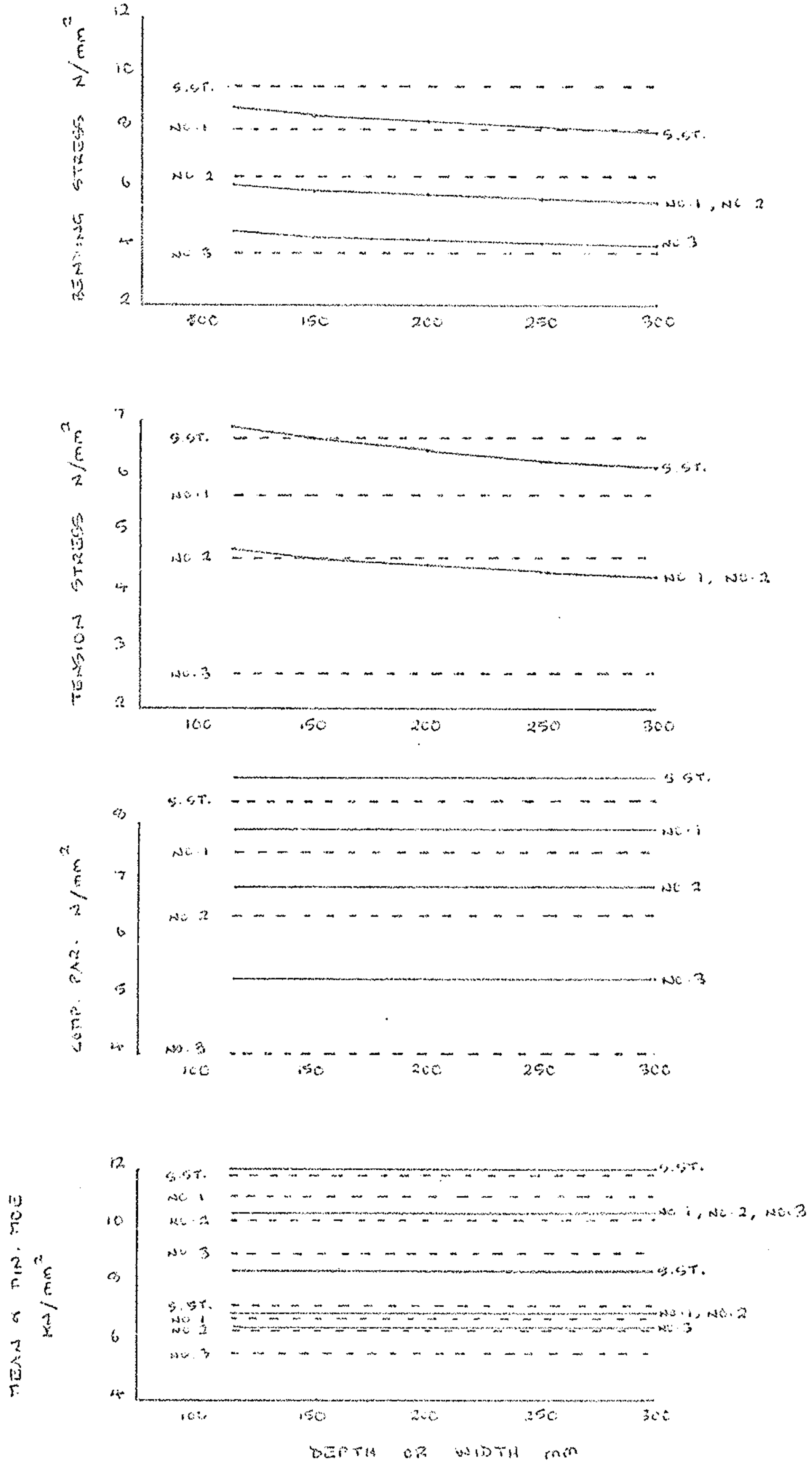


FIG. 5

COMPARISON OF MAJOR STRENGTH PROPERTIES FROM CP112 & BS 5268 FOR HEM-FIR – NLGA JOIST & PLANK GRADES

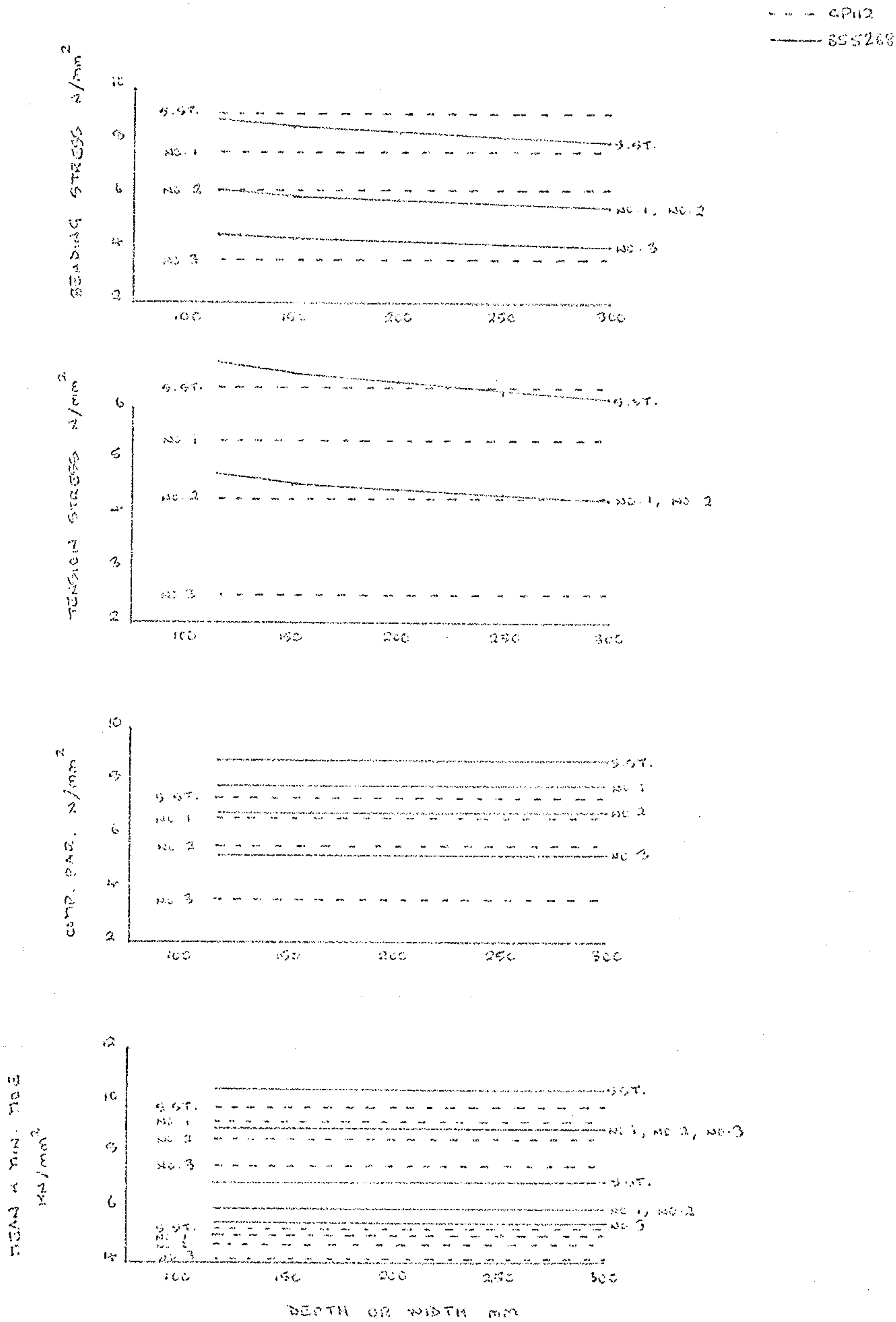


FIG. 6 COMPARISON OF MAJOR STRENGTH PROPERTIES FROM GP112 & BS5268 FOR SPRUCE - PINE - FIR - NLGA JOIST & PLANK GRADES

CIB-W18/17-7-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

MECHANICAL PROPERTIES OF NAILS AND THEIR INFLUENCE ON MECHANICAL PROPERTIES OF NAILED TIMBER JOINTS SUBJECTED TO LATERAL LOADS

by

I Smith, L R J Whale

Timber Research and Development Association
United Kingdom

and

C Anderson, L Held

Polytechnic of the South Bank, London
United Kongdom

RAPPERSWIL

SWITZERLAND

MAY 1984

CONTENTS

1. Introduction

The work reported in this paper is part of TRADA's current programme of work on characteristic properties of mechanical joints in structural timberwork. The programme of work consists of two projects; one partly financed by the U.K. Government Department of the Environment (D.O.E.) and the other partly financed by the Commission of the European Communities (C.E.C.). Within both the D.O.E. and C.E.C. projects the following material properties are being collected as input data to analytical models used to predict characteristic properties of nailed or bolted joints:

(i) load-embedment relationships for nails or bolts bearing on various solid timbers, various plywoods and tempered hardboard,

(ii) diameters, bending yield stresses and bending elastic moduli for bright wire circular nails,

(iii) diameters, tensile yield stresses, tensile ultimate strengths and tensile elastic moduli for black bolts.

Emphasis is being placed within the D.O.E. project upon timber species most specifically relevant to U.K. construction; British-grown sitka spruce and scots pine, greenheart and keruing; and TE grade tempered hardboard. Emphasis is being placed within the C.E.C. project upon timber species and plywoods of common interest to all European countries: European redwood, European whitewood, Canadian spruce-pine-fir and plywoods: Finnish birch, Finnish spruce, French pine and Canadian Douglas-fir.

The remainder of this paper is devoted to presentation and analysis of results from bending tests on bright wire circular nails.

2. Objectives and scope of nail bending tests

The objectives of nail tests are:

(a) Estimation of statistical variations in diameters, yield stresses and elastic moduli between manufacturers (within U.K. and between countries of the Euopean Communities) and within and between batches of nails.[+]

[+] A batch of nails is here taken to mean nails manufactured in a common production run and from the same consignment of steel.

(b) To provide source data for the estimation of the proportion of variability in strength and stiffness properties of laterally loaded nailed joints that can be attributed to variability in nail properties.

(c) Following from (b) above, to enable a rational consideration of questions such as:

"Which nail properties should be regulated through national codes or standards?",

"What tolerances should be specified on the properties that are to be regulated?".

It was decided at the outset to restrict the study to nails manufactured in countries of the European Economic Communities with preliminary emphasis on nails manufactured in the U.K.. The scope of the tests on U.K. nails is as follows:

nominal diameters: 2.65, 3.35, 4.00, 5.00, 6.00 mm
number of manufacturers: 4
number of replicates per batch: 20
number of batches per combination of diameter and manufacturer: 3

Total number of tests on U.K. nails:1/
5 diameters x 4 manufacturers x 20 replicates x 3 batches = 1200

At the time of writing tests have been completed on 20 replicates from one batch from each of four manufacturers for the five diameters, i.e. first 400 tests are completed, and tests are in progress on nails from second batches.

Nails are being collected from mainland-Europe and at least 400 additional tests on these nails are intended.

1/ Nails tested are intended to comply with the requirements of:
BS 1202: Part 1. Specification for nails: Part 1. Steel nails. London, British Standards Institution. 1974.

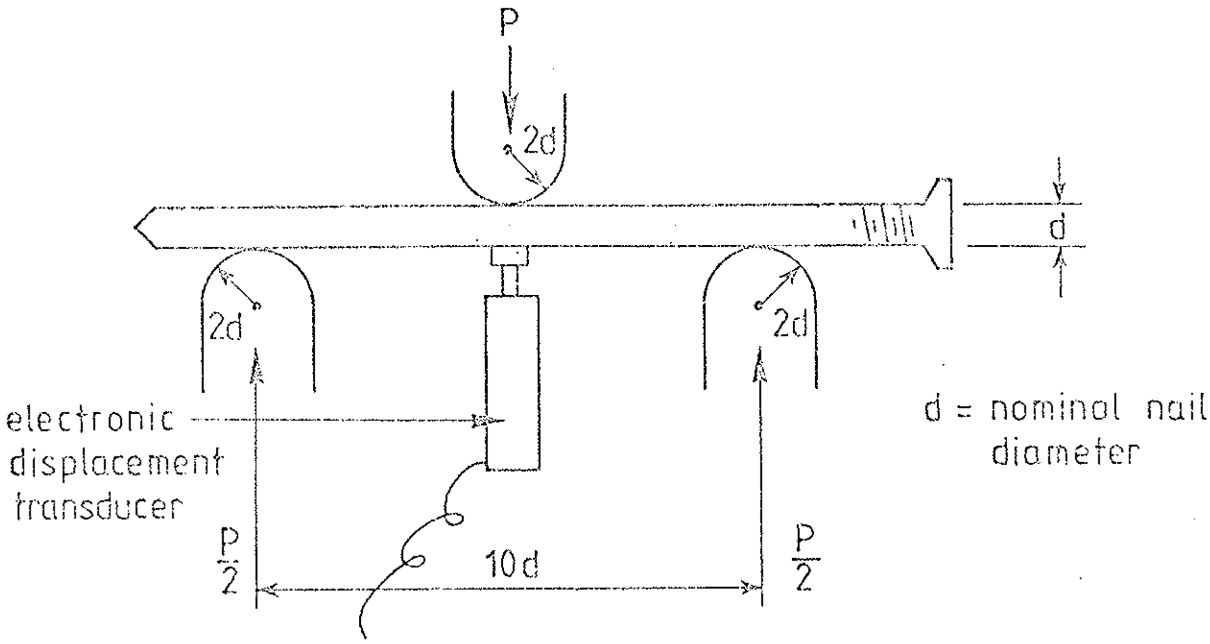
3. Test methods and procedures

The method conforms substantially to the recommendations for nail bending tests by Noren and published as "Nordtest project 77-77"[2/], Fig.1.

Maximum load and the associated mid-span deflection, initial slope of the load-deflection diagram and limit of proportionality for each nail were all extracted from plots of load v. mid-span deflection, Fig.2.

Test arrangement

Fig. 1



2/ NOREN, B. Method of testing nails in wood (second draft, August 1980). CIB-W18 paper 14-7-2, Warsaw, May 1981.

PLATE 1 Test apparatus in use (Nail at maximum bending)

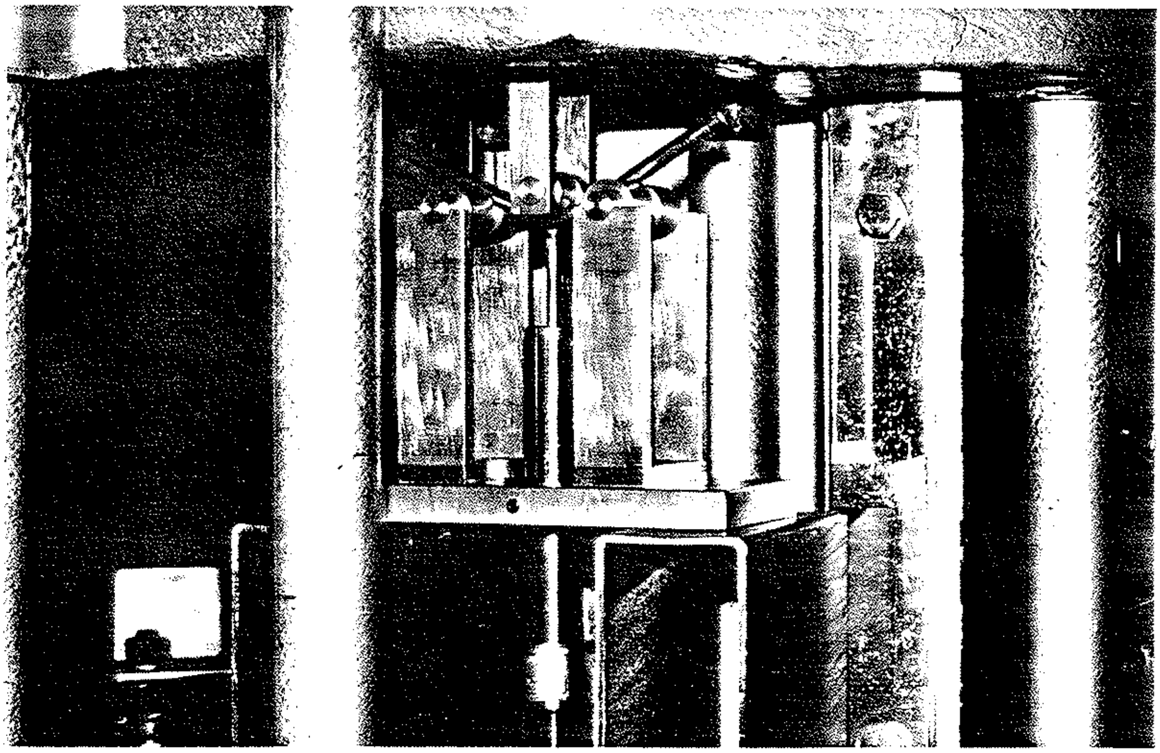
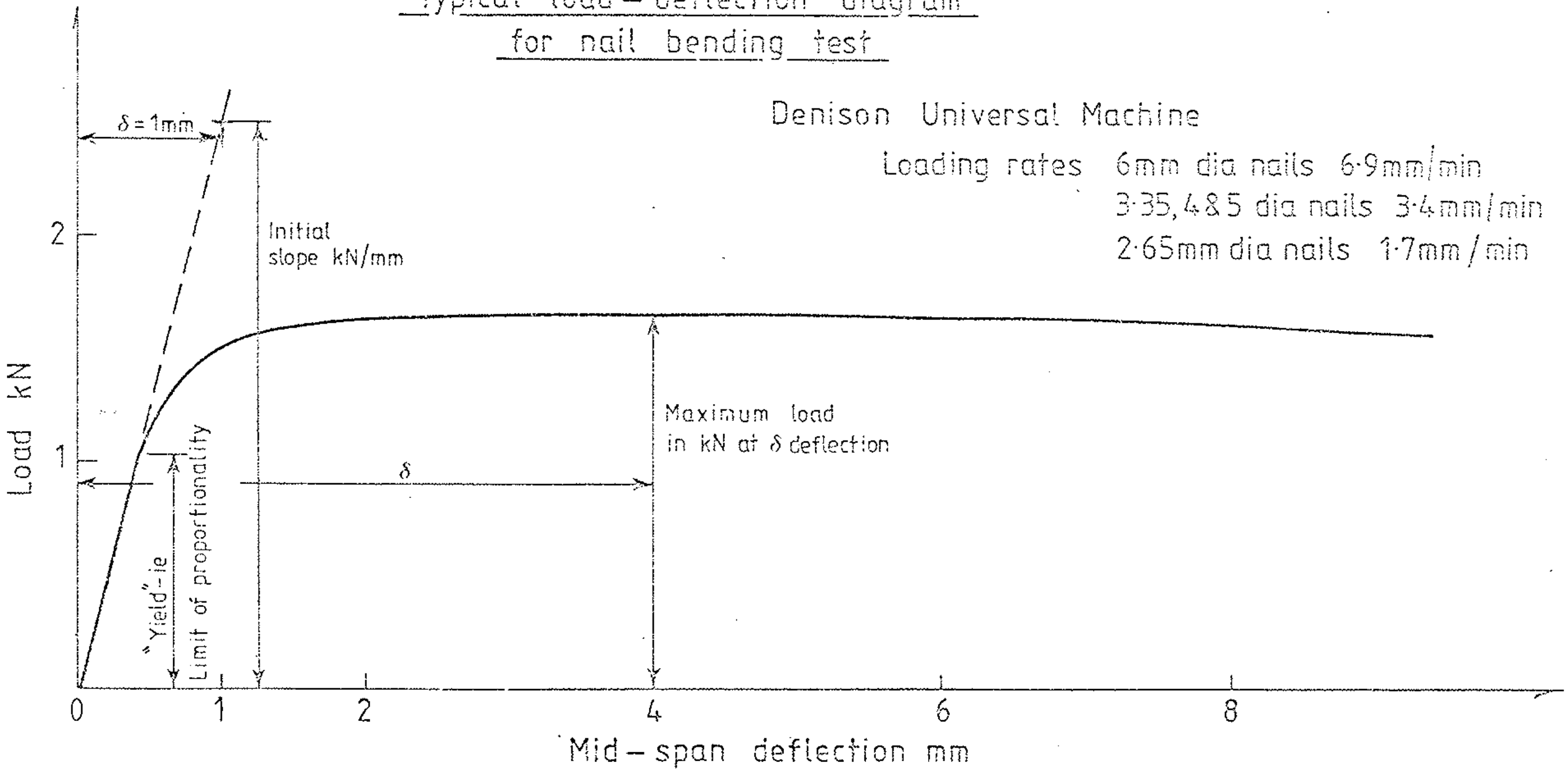


Fig. 2

Typical load – deflection diagram for nail bending test



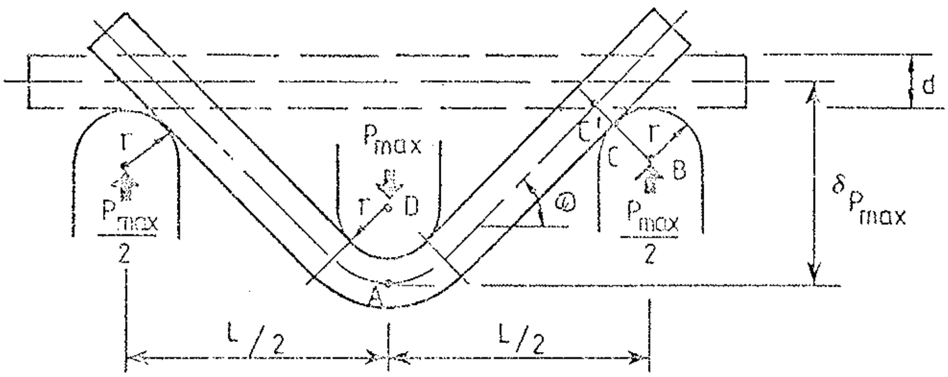
4. Method of analysis:

4.1 Yield stress at formation of full plastic hinge

Observation of nails under test and the large amount of plastic deformation apparent in tested nails lead the writers to suspect that a small deflexion theory estimate for yield stress at the formation of a full plastic hinge could be substantially in error, see Plate 1. Comparisons of estimated elastic mid-span deflections with observed mid-span deflections at maximum loads showed that estimated elastic deflections were in the order of $\leqslant 10\%$ of corresponding observed deflections. On the basis of test observations and the comparison of estimated elastic and observed deflections at maximum loads, the following deformed shape is assumed for estimating yield stress at formation of a full plastic hinge:

Fig. 3

Assumed deformed shape used as basis for estimating yield stress

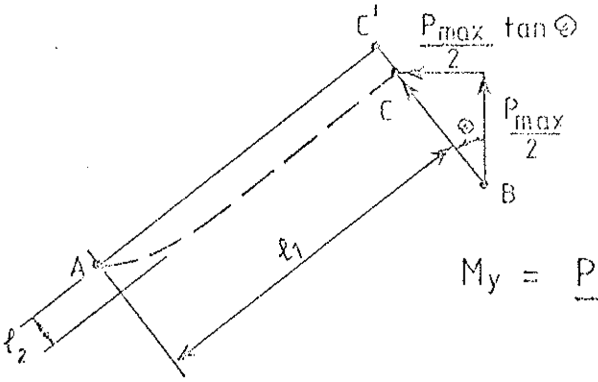


P_{max} = maximum load = load at which full plastic hinge forms

δP_{max} = mid-span deflection at which P_{max} is attained

Starting from Fig.3 an approximate large deflexion solution can be developed for yield stress, f_y.

Plastic yield moment M_y is developed by the resultant reaction applied through lever-arm l_1:



$$M_y = \frac{P_{max}}{2} \sec\theta \; \ell_1 \qquad (1)$$

From geometry:

$$\delta_{P_{max}} = \frac{L}{2}\tan\theta - 2\left(r + \frac{d}{2}\right)(\sec\theta - 1) \qquad (2)$$

Again from geometry:

$$M_y = \frac{P_{max}}{2}\left[\frac{L}{2}\sec^2\theta - \left(r + \frac{d}{2}\right)(2\sin\theta - \tan\theta)\right] \qquad (3)$$

Taking $\theta = 0$ equation (3) reduces to small deflexion solution:

$$M_y = \frac{P_{max} L}{4} \qquad (4)$$

Assuming that the nail cross-section is circular and remains undistorted with diameter equal to d:

$$M_y = \frac{f_y d^3}{6} \qquad (5)$$

(Use of equation (5) implies an assumption that the nail steel does not exhibit strain hardening).

Equating equations (3) and (5):

$$f_y = \frac{3P_{max}}{d^3}\left[\frac{L}{2}\sec^2\theta - \left(r + \frac{d}{2}\right)\left(2\sin\theta - \tan\theta\right)\right] \qquad (6)$$

For the small deflexion approximation ($\theta = 0$):

$$f_y = \frac{3P_{max}L}{2d^3} \qquad (7)$$

For the approximate large deflexion solution θ is found iteratively using equation (2) and then used in equation (6) to find f_y. For the small deflexion solution f_y is found directly using equation (7).

4.2 Yield stress at proportional limit

$$f_y = \frac{8P_{PL}L}{\pi d^3} \qquad (8)$$

where: P_{PL} = proportional limit load.

4.3 Modulus of elasticity

$$E = \frac{4L^3}{3\pi d^4}\left(\frac{P}{\delta}\right)_{init} \qquad (9)$$

where: $\left(\frac{P}{\delta}\right)_{init}$ = initial slope of load-mid-span deflection curve.

5. Results (First batches of U.K. nails only)

The following notation is used in the presentation of test results for nail diameters, yield stresses and elastic moduli:

d = nail diameter

$f_{yL.D.}$ = yield stress at formation of full plastic hinge as calculated by the approximate large deflexion solution, equation (6),

$f_{yS.D.}$ = yield stress at formation of full plastic hinge as calculated by the small deflexion solution, equation (7),

$f_{yP.L.}$ = yield stress at proportional limit, equation (8),

E = modulus of elasticity, equation (9),

S.D. = standard deviation,

C.V. = coefficient of variation

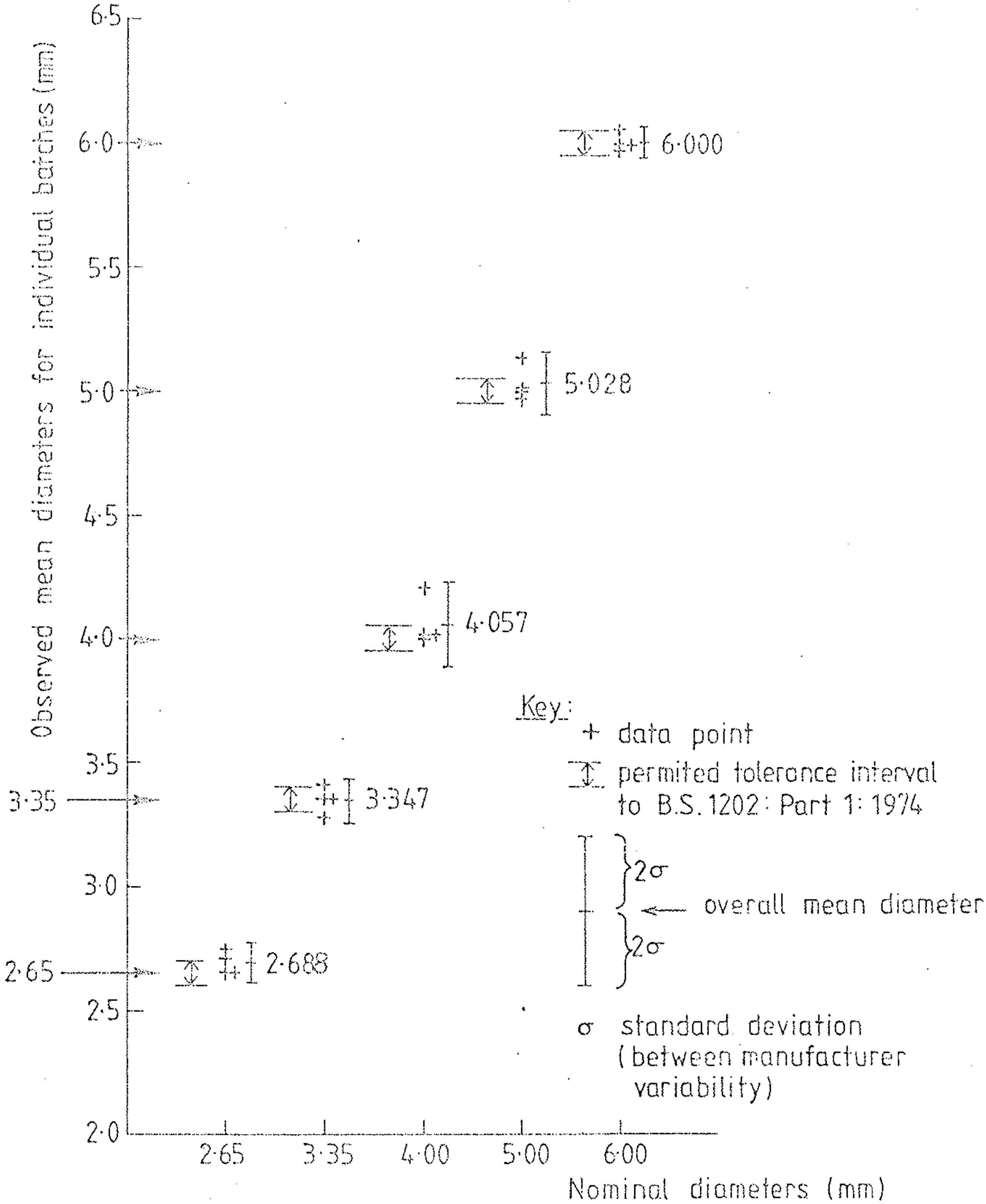
With the exception of $f_{yP.L.}$ it was found that within batch variation of individual nail properties is relatively small especially for nail diameter. The observed relatively high variability in estimated values for $f_{yP.L.}$ is thought to be associated with the well known difficulty in estimating proportional limit from a test record, (i.e. not only variability in material property). In most structural nailed joints more than one nail is employed. If nails in a given set of joints are from a common batch the coefficient of variation associated with between joint variation in average nail property is $1/\sqrt{n}$ x coefficient of variation associated with variability between individual nails, (where n = number of nails per joint). On the basis of theory which will be presented in Section 6 of this paper, it can be proven that within batch variation in nail properties d, f_y and E has negligible influence on variability of characteristic joint properties.

Fig.4 shows a comparison of observed mean batch diameters with the tolerance intervals for d permitted by BS 1202: Part 1, (nominal diameter $\pm$ 0.05 mm for nail diameters $\geqslant$ 2.65 mm). In only one instance did an observed mean batch diameter fall below the permitted tolerance interval, at a nominal diameter of 3.35 mm. In five instances (25% of batches) observed mean batch diameter was above the permitted tolerance interval. The question of influence of variability in mean batch diameter on the strengths of joints which fail by simultaneous formation of a plastic hinge(s) in the nail(s) and a bearing failure of the timber beneath the nail(s) will be discussed in Section 6. Another consideration not dealt with later, is that exceedance of the upper tolerance limit for d can result in substandard edge distances, end distances and spacings, which will usually be sized on the basis of nominal diameters, leading to the possibility of reduced joint strength caused by premature splitting of the wood. The preliminary conclusion to be drawn from the above is that manufacturers experience difficulty in meeting the BS 1202: Part 1 specification for d.

Comparison of mean values for $f_{yS.D.}$ and $f_{yL.D.}$ showed that the small deflexion estimates for yield stress at formation of a full plastic hinge are 4 to 6% higher than corresponding large deflexion estimates. Comparison of mean values for $f_{yP.L.}$ and $f_{yL.D.}$ showed that proportional limit yeild stresses are in the order of 75 to 95% of corresponding full plastic hinge large deflexion estimates. This shows that the nail steel undergoes strain hardening. As a consequence of this there is no absolute value

Fig. 4

Comparison of observed mean batch diameters with the tolerance intervals for d permitted by BS 1202: Part 1: 1974



for f_y. Any estimates for f_y based on equation (5) are nominal stresses. The choice of coordinate set P_{max} and δ_{Pmax} in development of equation (2) is purely arbitrary. Any other coordinated set beyond the mid-span deflection at which a plastic hinge formed could have been chosen. Fig.5 illustrates the interrelationship between mid-span deflection δ and estimated f_y.

Table 1 summarises between batch variation in $f_{yL.D.}$ which is presumed to be the best estimate for f_y. There is no distinct trend relating nominal diameter and $f_{yL.D.}$. The question of between manufacturer variation in $f_{yL.D.}$ cannot be properly judged on the basis of these results and will be considered when all 1200 tests on U.K. nails are completed. Provisionally it is proposed that between batch mean and variability in f_y be estimated by combining results for all twenty batches, i.e. no distinction on basis of manufacturer or nominal diameter.

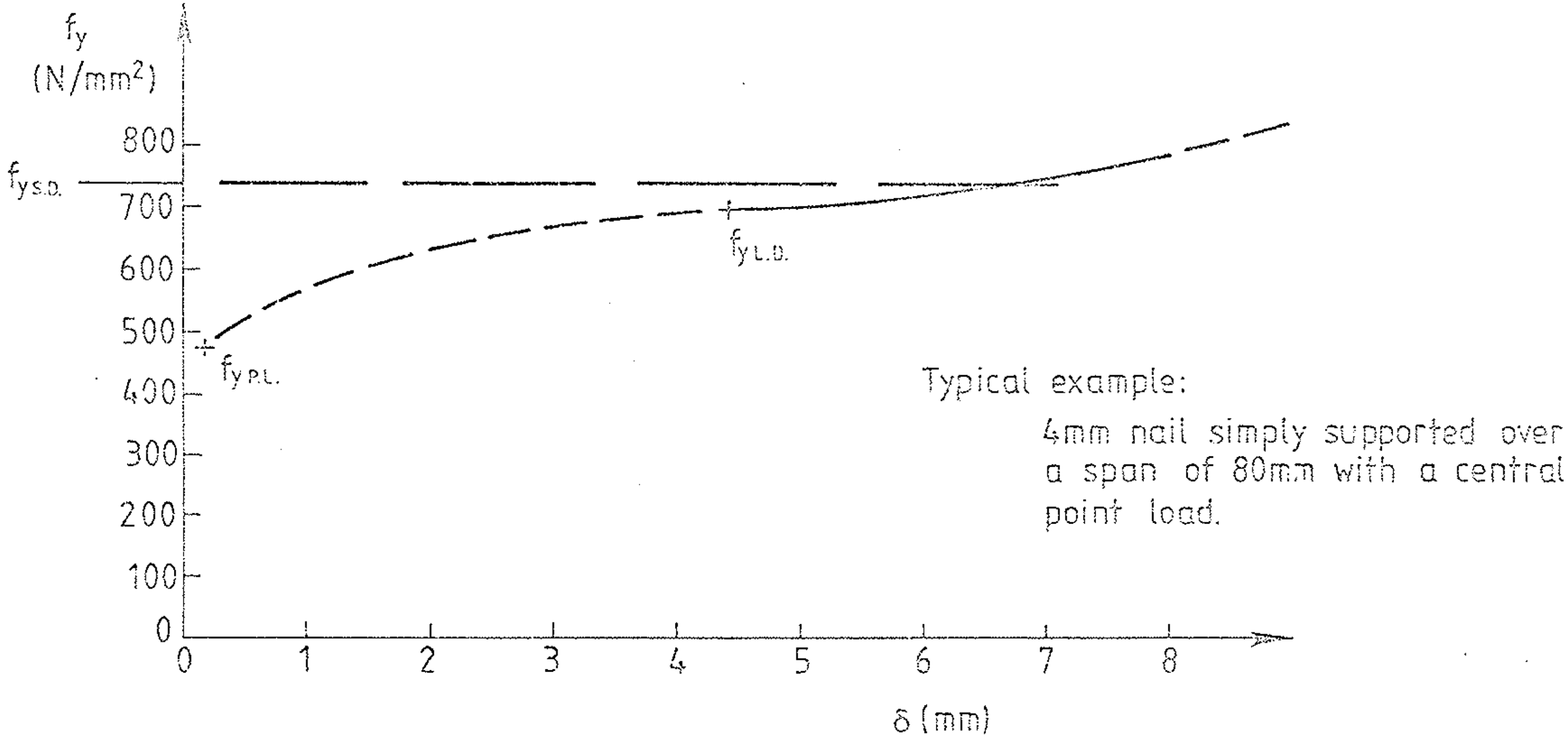
Table 2 summarises between batch variation in E. There is no distinct variation in E between manufacturers. Also there is no distinct variation in E between nominal diameters 3.35, 4.00, 5.00 and 6.00 mm. The E values for 2.65 nominal diameter nails are on average about 6% lower than those for other nominal diameters. Apart from these observations it is apparent that E values are in the order of 10% less than those usually associated with steels.

6. Application of results

As mentioned in Section 1 an important objective of the work is to enable rational consideration of questions associated with regulation of nail properties and predictability of joint properties as a function of regulation of nail properties. Below are developed explicit expressions relating expected values and variances of joint properties to means and variances of nail and wood properties. Through application of these expressions the questions about nail properties can be answered.

Fig. 5

Interrelationship between mid-span deflexion δ and yield stress f_y as predicted using P_δ and δ in equations (2) and (6) in lieu of $P_{\delta max}$ and δ_{max}



Between manufacturer and between diameter distribution statistics for yield stress, $f_{y\ L.D.}$

TABLE 1

		Within batch mean yield stresses							
Nominal diameter (mm)		2.65	3.35	4.00	5.00	6.00	Mean	S.D.	C.V. (%)
Manufacturer	1	771.0	822.8	623.3	588.9	683.6	697.9	87.9	12.59
	2	888.2	764.7	831.9	756.6	808.0	809.9	48.0	5.92
	3	677.2	786.2	702.0	790.9	764.3	744.1	46.1	6.19
	4	683.6	767.5	721.6	679.8	785.4	727.6	42.9	5.89
	Mean	755.0	785.3	719.7	704.1	760.3			
	S.D.	85.4	23.2	74.5	77.7	46.9			
	C.V. (%)	11.31	2.95	10.35	11.04	6.17			

Units for $f_{y\ L.D.}$ are N/mm^2

Global (all combinations of manufacturer and diameter combined):

Mean	744.9	N/mm^2	between combination variability
S.D.	72.0	N/mm^2	
C.V.	9.66	%	

Between manufacturer and between diameter distribution statistics for modulus of elasticity, E

TABLE 2

		Within batch mean modulus of elasticity							
Nominal diameter (mm)		2.65	3.35	4.00	5.00	6.00	Mean	S.D.	C.V. (%)
Manufacturer	1	183,094	191,238	183,655	179,478	188,137	185,120	4,114	2.22
	2	177,091	190,239	189,077	188,898	190,356	187,132	5,055	2.70
	3	168,342	187,453	182,067	194,164	187,382	183,882	8,666	4.71
	4	177,367	188,192	182,510	182,959	188,814	183,968	4,196	2.28
	Mean	176,474	189,281	184,327	186,375	188,672			
	S.D.	5,271	1,523	2,803	5,619	1,096			
	C.V. (%)	2.99	0.80	1.52	3.01	0.58			

Units for E are N/mm^2

Global (all combinations of manufacturer and diameter combined):

Mean	185,026	N/mm^2	
S.D.	5,959	N/mm^2	between combination variability
C.V.	3.22	%	

6.1 Theory

6.1.1 Yield load for a nailed joint

Consideration will be restricted to joints which fail by:

a) bearing failure of the timber beneath the connector, or

b) simultaneous development of a bearing failure of the timber beneath the connector and one or more "plastic hinges" in the connector.

The load at which either state is first attained is referred to as the yield load, P_y. Various explicit solutions are available for predicting P_y for different combinations of joint geometry, number of hinges and wood properties. Each solution is based on assuming that both the nail steel and the wood are ideal rigid-plastic materials. For convenience Larsen's expressions and notation will be used.[3/] It can be shown that the influences of nail properties d and f_y on P_y are maximised by a so-called mode 3 failure, Fig.6, for which;

$$P_y = 0.40825\, S_H^{1/2}\, d^2\, \phi^{1/2}\, f_y^{1/2} \qquad (10)$$

where: S_H = compressive strength of timber beneath the connector (N/mm^2)

$\phi = 4\beta/(1+\beta)$, β defined in Fig.6 .

for both two piece and three piece joints.

Using the method for generation of system moments and assuming uncorrelated variables the expected value for P_y is;[4/]

$$E(P_y) = 0.40825\left(\bar{S}_H\,\bar{\phi}\,\bar{f}_y\right)^{1/2}(\bar{d})^2\left(1 + V_d^2 - \frac{1}{8}\left(V_{S_H}^2 + V_\phi^2 + V_{f_y}^2\right)\right) \qquad (11)$$

and variance for P_y is:

$$Var(P_y) \doteq (\bar{d})^4(\bar{S}_H\,\bar{\phi}\,\bar{f}_y)\left(0.041667\left(V_{S_H}^2 + V_\phi^2 + V_{f_y}^2\right) + 0.66667\,V_d^2\right) \qquad (12)$$

where: $\bar{S}_H$, $\bar{\phi}$, $\bar{f}_y$ and $\bar{d}$ are mean values of S_H, ϕ, f_y and d respectively.

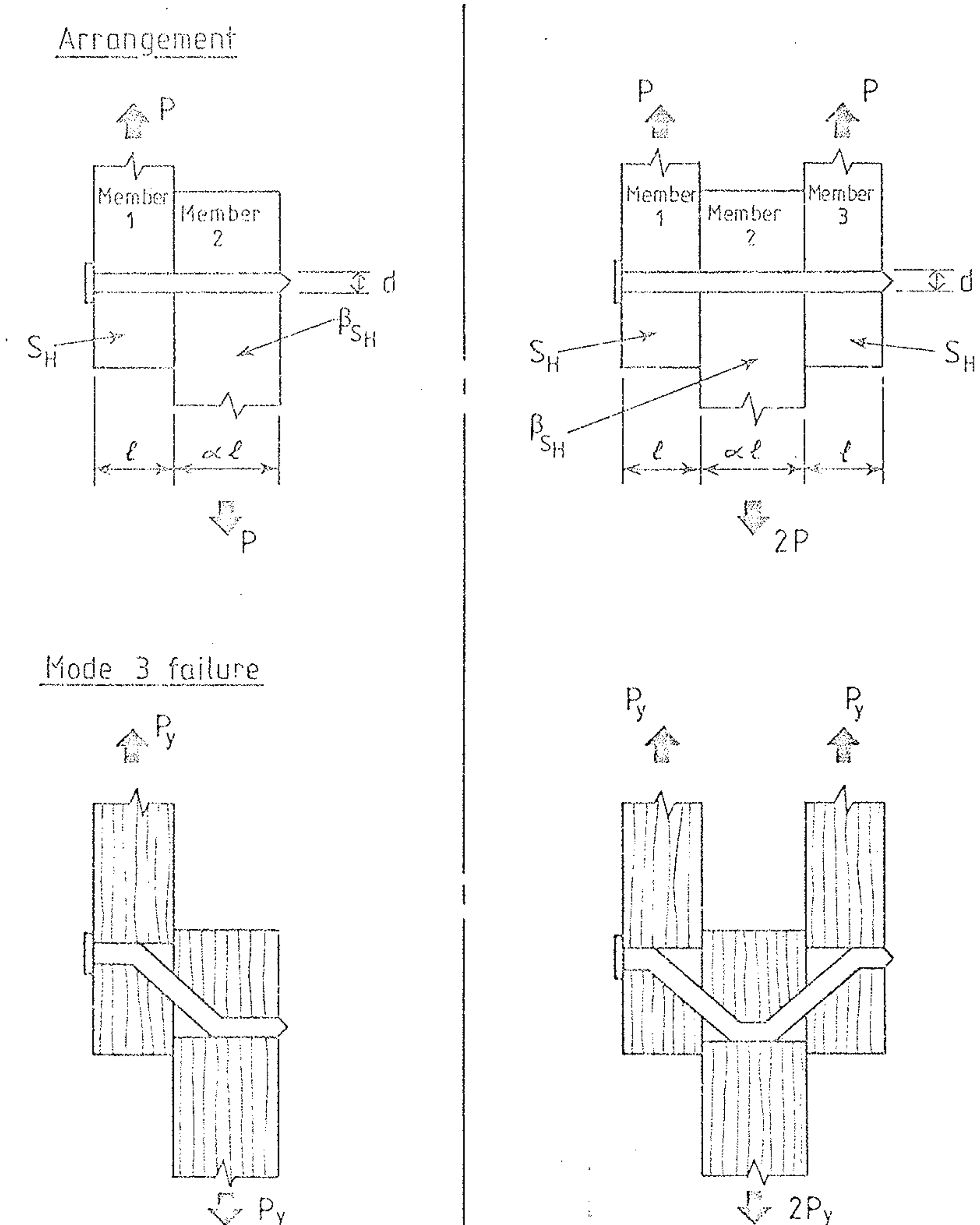
V_{SH}, V_ϕ, V_{f_y} and V_d are coefficients of variation of S_H, ϕ, f_y and d respectively.

3/ LARSEN, H.J. K.W. Johansen's nail tests. CIB-W18, Bordeaux, October 1979.

4/ HAHN, G.J. and SHAPIRO, S.S. Statistical models in engineering. London, John Wiley and Sons. 1967.

Fig. 6

Arrangement diagrams and assumed deformed shapes for calculation of P_y values



Two piece non-symmetrical joint

Three piece symmetrical joint

To a good approximation equation (11) may be simplified to:

$$E(P_y) = 0{\cdot}40825(\bar{S}_H\,\bar{\phi}\,\bar{f}_y)^{1/2}(\bar{d})^2 \qquad (13)$$

6.1.2 Initial stiffness for a nailed joint

For nailed joints in which the nail has a high slenderness ratio, i.e. joints which would tend to produce a mode 3 failure, the initial joint stiffness is approximated by Wilkinson's linear elastic solution 5/:

$$\frac{P}{\delta} = 0{\cdot}665668\ E^{1/4}\,k_{01}^{3/4}\,d^{7/4}\,\beta_1 \qquad (14)$$

where: P = joint load (N) at joint displacement δ (mm)

E = modulus of elasticity of nail (N/mm^2)

k_{01} = elastic bearing constant of member 1, Fig.6.

= stress per unit embedment (N/mm^3)

k_{02} = elastic bearing constant of member 2

d = nail diameter

$$\beta_1 = \frac{r(r+r^{3/4})}{2(r+r^{1/4})(r+r^{3/4})-(r-r^{3/2})^2}$$

$$r = k_{02}/k_{01}$$

Equation (14) can be used to quantify the influence of nail properties E and d on joint stiffness.

Using the method for generation of system moments and assuming uncorrelated variables the expected value for (P/δ) is:

$$E(P/\delta) = 0{\cdot}665668(\bar{E})^{1/4}(\bar{k}_{01})^{3/4}(\bar{d})^{7/4}(\bar{\beta}_1)\left(1+\frac{3}{32}\left(7V_d^2 - V_E^2 - V_{k_{01}}^2\right)\right) \qquad (15)$$

and variance for (P/δ) is:

$$Var(P/\delta) \doteqdot 0{\cdot}443114\,(\bar{k}_{01})^{3/2}(\bar{d})^{7/2}(\bar{\beta}_1)^2(\bar{E})^{1/2}\left(V_E^2/16 + 9V_{k_{01}}^2/16 + 49V_d^2/16 + V_{\beta_1}^2\right) \qquad (16)$$

To a good approximation equation (15) may be simplified to:

$$E(P/\delta) = 0{\cdot}665668\,(\bar{E})^{1/4}(\bar{k}_{01})^{3/4}(\bar{d})^{7/4}(\bar{\beta}_1) \qquad (17)$$

5/ WILKINSON, T.L. Analysis of nailed joints with dissimilar members. Journal of Structural Division, ASCE, September 1972, 98 (ST9) 2005-2013

6.2 Sensitivity of joint performance to:

6.2.1 Variation in mean yield stress $\bar{\bar{f}}_y$

Let global average yield stress f_{yg} be used as the index for mean yield stress, $f_{yg} = 744.9$ N/mm^2 (Table 1). From equations (12) and (13):

$$\frac{E(P_y)_{\bar{\bar{f}}_y}}{E(P_y)_{f_{yg}}} = \left(\frac{\bar{\bar{f}}_y}{f_{yg}}\right)^{1/2} \quad (18), \qquad \frac{Var(P_y)_{\bar{\bar{f}}_y}}{Var(P_y)_{f_{yg}}} = \frac{\bar{\bar{f}}_y}{f_{yg}} \quad (19)$$

If it is assumed that P_y is normally distributed, then from equations (18) and (19) P_y for $\bar{\bar{f}}_y \neq f_{yg}$ is $(\bar{\bar{f}}_y/f_{yg})^{\frac{1}{2}}$ times P_y for $\bar{\bar{f}}_y = f_{yg}$ at all levels of exclusion. Table 3 gives data from which can be quantified the sensitivity of P_y to variations in $\bar{\bar{f}}_y$.

Table 3

Sensitivity of P_y to variations in $\bar{\bar{f}}_y$

$\bar{\bar{f}}_y / f_{yg}$	0.70	0.80	0.90	1.00	1.10	1.20	1.30
$\frac{P_y(\bar{\bar{f}}_y \neq f_{yg})}{P_y(\bar{\bar{f}}_y = f_{yg})}$	0.8367	0.8944	0.9487	1.0000	1.0488	1.0954	1.1402
		(at all levels of exclusion)					

On the basis of Table 3 it is suggested that a target value for f_y should be specified for nails if reliable predictions of P_y are to be attained. Provisionally it is proposed that a suitable target value for f_y is 750 N/mm^2. (The question of a suitable tolerance interval on f_y is considered in Section 6.2.2). The above also suggests that caution should be exercised when using test data for joints with nails of unkown yield strength.

6.2.2 Variation in coefficient of variation for yield stress V_{fy}

From equation (12):

$$\frac{Var(P_y)_{V_{f_y}}}{Var(P_y)_{V_{f_y}=0}} = \frac{V_{S_H}^2 + V_\phi^2 + 16V_d^2 + V_{f_y}^2}{V_{S_H}^2 + V_\phi^2 + 16V_d^2} \quad (20)$$

The following are taken as representative values;

$V_{SH} = 0.20$, $V_\phi = 0.10$, $V_d = 0.015$.

Substituting in equation (20):

$$\frac{V_{P_y}(V_{f_y})}{V_{P_y}(V_{f_y}=0)} = \left(\frac{Var(P_y)_{V_{f_y}}}{Var(P_y)_{V_{f_y}=0}}\right)^{1/2} = \left(\frac{0\cdot0536 + V_{f_y}^2}{0\cdot0536}\right)^{1/2} \qquad (21)$$

Table 4 gives data from which can be quantified the contribution of V_{fy} to the coefficient of variation V_{Py}.

Table 4

Contribution of V_{fy} to V_{P_y}

V_{f_y}	0.0	0.05	0.10	0.15	0.20
$\frac{V_{P_y}(V_{f_y})}{V_{P_y}(V_{f_y}=0)}$	1.0000	1.0231	1.0893	1.1915	1.3215

From Table 1 the observed value of V_{fy} associated with between batch variation in mean yield stresses for individual batches was 0.0966.

Let it be assumed that the true value of V_{fy} is 0.10. As an illustration consider a case where V_{fy} is incorrectly estimated to be 0.20, which yields an over estimate of V_{Py} by a factor of 1.3215/1.0893 = 1.2132 times. Thus if P_y were normally distributed a 100% overestimate in V_{fy} would result in the following ratios of predicted value to true values for P_y at 5% and 95% levels of exclusion.

Table 5

Ratios of predicted to true values for P_y if 100% overestimate in V_{fy} occurred

	$V_{Py}\,(V_{fy}=a)$	$\frac{P_{y\ 5\%}\,(V_{fy} = 2a)}{P_{y\ 5\%}\,(V_{fy} = a)}$	$\frac{P_{y\ 95\%}\,(V_{fy} = 2a)}{P_{y\ 95\%}\,(V_{fy} = a)}$
Typical range	0.10	0.9580	1.0301
	0.15	0.9302	1.0422
	0.20	0.8955	1.0528
	0.25	0.8511	1.0621
	0.30	0.7923	1.0704
	0.35	0.7107	1.0779
	0.40	0.5898	1.0846

where $\frac{V_{sH}}{V_{fy}} = \frac{0.20}{a}$ $\quad \frac{V_\phi}{V_{fy}} = \frac{0.10}{a}$ $\quad \frac{V_d}{V_{fy}} = \frac{0.015}{a}$

It is apparent from Table 5 that the lower tail of the P_y distribution is more sensitive to variations in V_{fy} than is the upper tail of that distribution. Examination of Tables 4 and 5 reveals that between batch variations in f_y are unlikely to greatly influence ratios of P_y between nominally identical joints. Also from the same tables it can be deduced that tolerances on f_y that it may be decided should be included in codes of practice need not be specified to a high precision. The main objective of a tolerance interval would be to preclude use of unusually low quality steel in wire nails which would result in highly inconsistent joint strengths. On the basis of the tests on first batches of U.K. nails a suggested tolerance interval on f_y is 750 ± 150 N/mm^2, see Section 6.2.1 for basis of selection of a target value of 750 N/mm^2. For quality control purposes it would be adequate to estimate f_y using the small deflexion solution of equation (7).

6.3 Typical order of between joint variations in joint properties P_y and (P/δ)

The purpose of this section is to give a general indication of relative sensitivity of joint properties to variations in nail properties and to variations in wood properties.

The following are taken to be representative properties;
$V_d = 0.015$, $V_{fy} = 0.10$, $V_E = 0.03$, $V_{SH} = 0.20$, $V_{k01} = 0.35$, $V_\phi = 0.10$, $V_{\beta_1} = 0.15$.

From equations (12) and (13) the coefficient of variation for P_y is:

$$V_{P_y} = 0.5\left(V_{S_H}^2 + V_\phi^2 + V_{f_y}^2 + 16\,V_d^2\right)^{1/2} \qquad (22)$$

From equations (16) and (17) the coefficient of variation for (P/δ) is:

$$V_{(P/\delta)} = 0.25\left(V_E^2 + 9\,V_{k_{01}}^2 + 49\,V_d^2 + 16\,V_{\beta_1}^2\right)^{1/2} \qquad (23)$$

Substituting typical values for V_{SH}, V_ϕ, V_{fy} and V_d in equation (22); $V_{Py} = 0.1261$. Table 6 shows the influence on V_{Py} of taking various components of equation (22) to be null.

Table 6

Sensitivity of equation (22)

	V_{Py}
$V_{S_H} = 0$	0.0768
$V_{\phi} = 0$	0.1158
$V_{fy} = 0$	0.1158
$V_d = 0$	0.1225

Thus fairly low variability in strengths between nominally identical nailed joints is predicted, a conclusion consistent with experience.[6/] Variability in strengths between joints is not dominated by variability in any single component property, but is most strongly influenced by variability in S_H and least influenced by variability in d.

Substituting typical values for V_E, V_{k01}, V_d and $V_{\beta 1}$ in equation (23); $V_{(p/\delta)} = 0.3036$. Table 7 shows the influence on $V_{(p/\delta)}$ of taking various components of equation (23) to be null.

Table 7

Sensitivity of equation (23)

	$V_{(p/\delta)}$
$V_E = 0$	0.3035
$V_{k01} = 0$	0.1525
$V_d = 0$	0.3024
$V_{\beta 1} = 0$	0.2639

Fairly high variability in initial stiffnesses between nominally identical nailed joints is therefore predicted, a conclusion consistent with experience[6/]. Variability in initial stiffnesses between joints is dominated by variability in k_{01}. Variability in E and d have negligible influence on variability of initial stiffnesses.

6/ SMITH, I. Interpretation and adjustment of results from short term lateral load tests on whitewood joint specimens with nails or bolts. TRADA Research Report 5/82. Hughenden Valley, TRADA. 1982.

7. Conclusions

The following provisional conclusions are based on the analysis of results from bending tests on first batches of U.K. manufactured nails and on the results of the sensitivity studies in Sections 6.2 and 6.3.

1/ Within a batch of nails variation in diameter, yield stress at formation of a full plastic hinge and modulus of elasticity is small, especially in the case of diameter. The primary source of variation in these nail properties is between batch variations.

2/ On the basis of the nails tested manufacturers appear to experience difficulty in meeting the BS 1202: Part 1 tolerance interval on diameter, (nominal diameter $\pm$ 0.05 mm for diameters greater than or equal to 2.65 mm).

3/ Provided that quality control ensures nail diameters fall within the BS 1202: Part 1 tolerance interval it is reasonable to neglect deviations from the nominal diameter.

4/ There is no distinct trend relating nominal diameter and yield stress at formation of a full plastic hinge.

5/ Consistent with experimental experience, it is shown theoretically that fairly low variability in strengths of nominally identical nailed joints is to be expected. Varaibility in strengths of joints is not dominated by variability in any single component property.

6/ Consistent with experimental experience, it is shown theoretically that fairly high variability in initial stiffnesses of nominally identical nailed joints is to be expected. Variability in initial stiffnesses of joints is dominated by variability in the bearing stiffness of the timber beneath the nails.

7/ A target value for the yield stress for nails should be specified in codes of practice for structural timberwork. This would result in reliably predictable joint strengths. It is also necessary to specify a tolerance interval on yield stress to preclude use of unusually low quality steel in wire nails which would result in highly inconsistent joint strengths. A proposed tolerance interval on yield stress is 750 ± 150 N/mm^2, on the basis of yield stress corresponding to formation of a full plastic hinge. Because nails strain harden it is necessary to match any specified tolerance interval to the test method which will be used to determine yield stress in quality control processes.

Questions such as the influence of within manufacturer versus between manufacturer variations in nail properties, will be investigated when the full programme of nail tests is completed.

IS/AL
26th March 1984

Timber Research and Development Association
Hughenden Valley, High Wycombe, Bucks.

CIB-W18/17-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

NOTES ON THE EFFECTIVE NUMBER OF DOWELS AND NAILS IN TIMBER JOINTS

by

G Steck

University of Karlsruhe
Federal Republic of Germany

RAPPERSWIL

SWITZERLAND

MAY 1984

Notes on the Effective Number of Dowels and Nails in Timber Joints

G. Steck

1. Introduction

In timber joints using mechanical fasteners it often occurs that several fasteners are in a row parallel to the direction of the load. In this case the load isn't distributed uniformly over the fasteners (see Fig. 1). The design of the fasteners in Fig. 1 is as follows

$$\max F = F_1 \text{ (resp. } F_{11}) \leqq F_{allowable}$$

$$\text{respectively } \frac{N}{r \cdot n_{ef}} \leqq F_{allowable}.$$

The term $n_{ef} = \eta \cdot n$ denotes an effective number of fasteners and η the efficiency of the joint. Lantos /1/ proposed a theoretical solution for the distribution of axial force N within the joint. In this paper the results of Lantos' solution will be compared to the formulas (6.1.1.1b) and (6.1.2.1g) of the CIB-Code /2/.

In case of interest in the total distribution of load and not only in the maximum loaded fastener exists another treatment of Lantos' difference equation. Especially in the case of calculation the probability of failure it is more practicable to interpret the difference equation as a equation system.

2. The Efficiency η of a Joint

2.1 Notation and Abbreviations

The following notation and abbreviations are used to show Lantos' solution and to define the efficiency η.

Notation (see Fig 1 and 2)

N	axial force
N_{1i}	axial force of member 1 in section i

N_{2i}	axial force of member 2 in section (n-i)
F_i	load of fastener i
$F_m = \frac{N}{r \cdot n}$	average load of fastener
e	distance between the fasteners in a row
x	section of member 1 between the fasteners x and (x+1)
n	number of fasteners in a row
r	number of rows
$E_1A_1(E_2A_2)$	stiffness of member 1 (2)
K	slip modulus

Abbreviations

$$\psi = \frac{r\,K\,e}{E_2A_2}$$

$$\omega = 2 + rKe\left(\frac{1}{E_1A_1} + \frac{1}{E_2A_2}\right)$$

$$\mu = \frac{\psi}{2-\omega}$$

$$m_1 = \frac{\omega + \sqrt{\omega^2-4}}{2} \qquad m_2 = \frac{\omega - \sqrt{\omega^2-4}}{2}$$

2.2 General Formulas

Lantos' derivation yields the difference equation

$$-N_{1(k-1)} + \omega N_{1k} - N_{1(k+1)} = \psi N; \quad k = 1 \dots (n-1).$$

The solution is

$$N_{1x} = \alpha_x N = \left(-\mu + m_1^x (1+\mu) - (m_1^x - m_2^x) \frac{m_1^x(1+\mu)-\mu}{m_1^n - m_2^n}\right) N$$

with $x = 0,1 \ldots n$.

The maximum loaded fastener is fastener 1 or n and the maximum values appertaining to this are

$$F_1 = \frac{1}{r}(N-N_{11}) = \frac{N}{r}(1-\alpha_1)$$

$$F_n = \frac{1}{r} N_{1(n-1)} = \frac{N}{r} \alpha_{n-1}.$$

The definition of η derives from the terms

$$F_1 \leqq F_{allowable} \text{ (resp. } F_n \leqq F_{allowable}\text{) and } \frac{N}{r \cdot n_{ef}} \leqq F_{allowable} .$$

From this results

$$\eta = \frac{n_{ef}}{n} = \frac{1}{n(1-\alpha_1)} \quad \text{and} \quad \eta = \frac{n_{ef}}{n} = \frac{1}{n\alpha_{n-1}}$$

The efficiency of a joint therefore depends on the number of fasteners in a row, distance e, slip modulus K and finally from stiffness of the jointed members. The efficiency is reduced in the case of decreasing stiffness EA. Therefore as stiffness is appointed the product of cross section $A = \dfrac{N}{f_{c(t),0,d(esign)}} = \dfrac{r \cdot \eta \cdot n\ F_{d(esign)}}{f_{c(t),0,d}}$

and modulus of elasticity $E = 1{,}4\ k_{mod}\ E_0$, which belongs to the strength class of $f_{c(t),0}$.

F_d for dowels and nails depends among others on $\sqrt{\rho}$, and so there results the standard stiffness from min $\left(\frac{\sqrt{\rho}E_o}{f_{c(t),0}}\right)$.

For the values in table 42.1 and 42.2 of CIB-Code, annex 42, is the standard strength class SC 15 determining with

$$E = 1{,}4 \cdot k_{mod} \cdot E_o = 1{,}4 \cdot 0{,}7 \cdot 4.600 \cong 4.600 \text{ N/mm}^2$$

$$A = \frac{r\,\eta\,n\,F_d}{\frac{f_{c,0} \cdot k_{mod}}{\gamma_m}} = r\,\eta\,n\,F_d \cdot \frac{1}{14{,}5 \cdot \frac{1}{3}}$$

2.3 Joints with Dowels

To calculate the effective number of dowels in a joint the following determinations are used:

$$F_d = \frac{1}{3} F_k$$

$$K = \frac{F_d}{0{,}1d} = \frac{F_k}{0{,}3d}$$

$$\alpha = 0° \rightarrow k_{\alpha,1} = k_{\alpha,2} = 1{,}0$$

$$t_1 = t_2 = t$$

$$f_y = 240 \text{ N/mm}^2$$

$$\lambda = \frac{t}{d} \geqq \lambda_{lim} = \frac{2{,}083}{\sqrt{\rho}} \; .$$

From these we attain

$$F_k = 93{,}75 \sqrt{\rho}\, d^2 \quad \text{and}$$

$$K = 312{,}5 \sqrt{\rho}\, d \; .$$

The distance between dowels may be $e = 7\cdot d$ and with $\rho = 0{,}4$ the stiffness $EA = r\eta n\frac{F_k}{3}\cdot\frac{3}{14{,}5}E = 1{,}881\cdot 10^4\ d^2 r\eta n$. The factor $\psi = \frac{rKe}{EA} = 7{,}355\cdot 10^{-2}\ \frac{1}{\eta\cdot n}$ is independent from the diameter d of dowel. In Fig. 3 different formulas for joint efficiency are shown:

$$\eta_{CIB} = \frac{n_{ef}}{n} = \frac{1}{n}\left(4+\frac{2}{3}(n-4)\right)$$

η_{LANTOS} and a straigth approaching line for η in accordance with Lantos' result.

2.4 Joints with Nails

In accordance with paragraph 2.3 the following determinations are used:

$$F_d = \frac{1}{3}F_k$$

$$K = 3{,}464\ \frac{F_k}{d} = 438{,}2\ d^{0{,}7} \quad \text{with } F_k = 200\ \sqrt{0{,}4}\ d^{1{,}7}$$

(see also Ehlbeck/Larsen /3/).

The distance between nails may be $e = 10d$ and with $\rho = 0{,}4$ the stiffness $EA = 4{,}013\cdot 10^4 d^{1{,}7}\ r\,\eta\,n$.
The factor $\psi = \frac{rKe}{EA} = 10{,}92\cdot 10^{-2}\ \frac{1}{\eta n}$ is independent from the diameter of nail.

In Fig. 4 different formulas for joint efficiency are shown:

$$\eta_{CIB} = \frac{n_{ef}}{n} = \frac{1}{n}\left(10+\frac{2}{3}(n-10)\right)$$

η_{LANTOS} for $\sigma_o = f_d$ resp. $\sigma_o = 0{,}8\ f_d$

and a straight approaching line η.

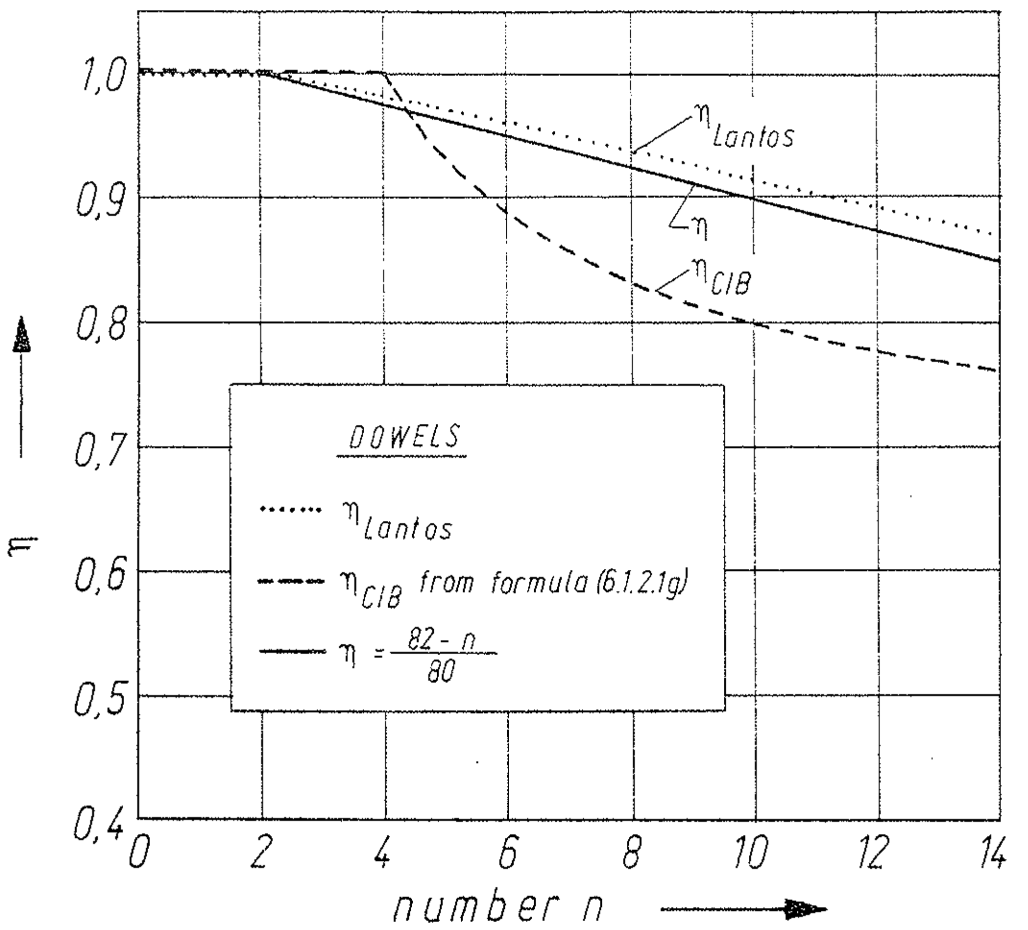


Fig. 3 Efficiency η of joints with dowels

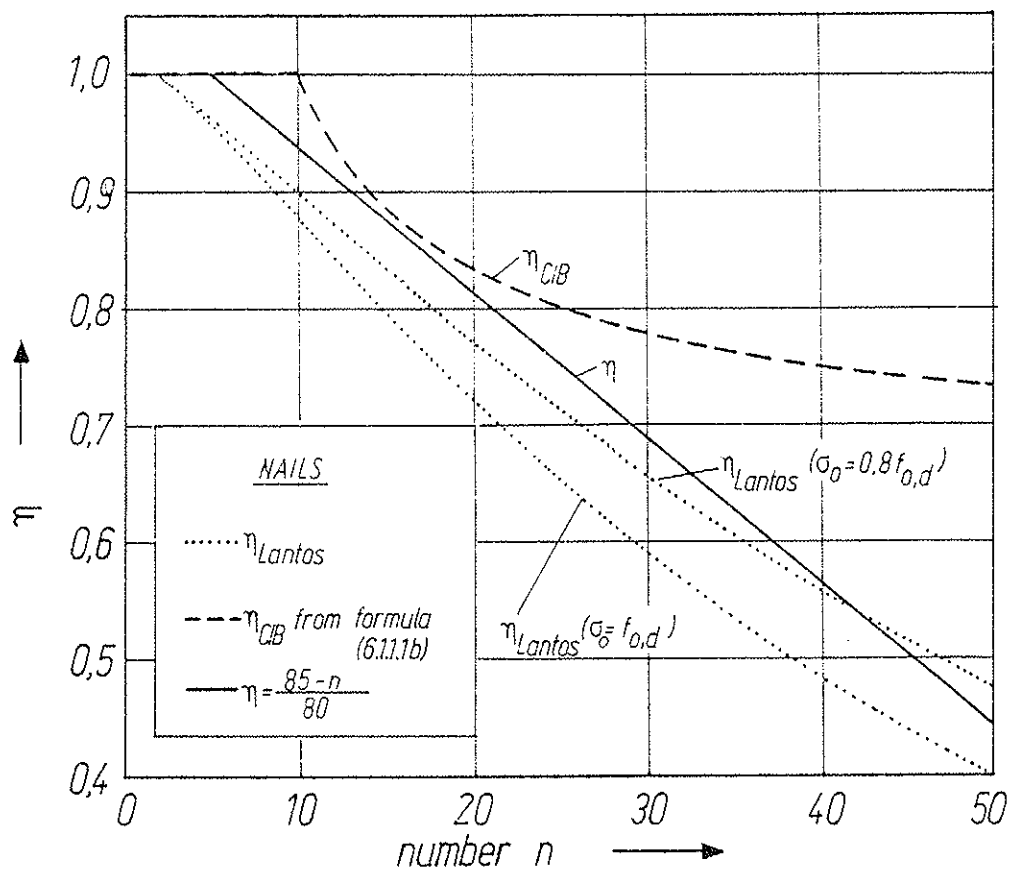


Fig. 4 Efficiency η of nailed joints

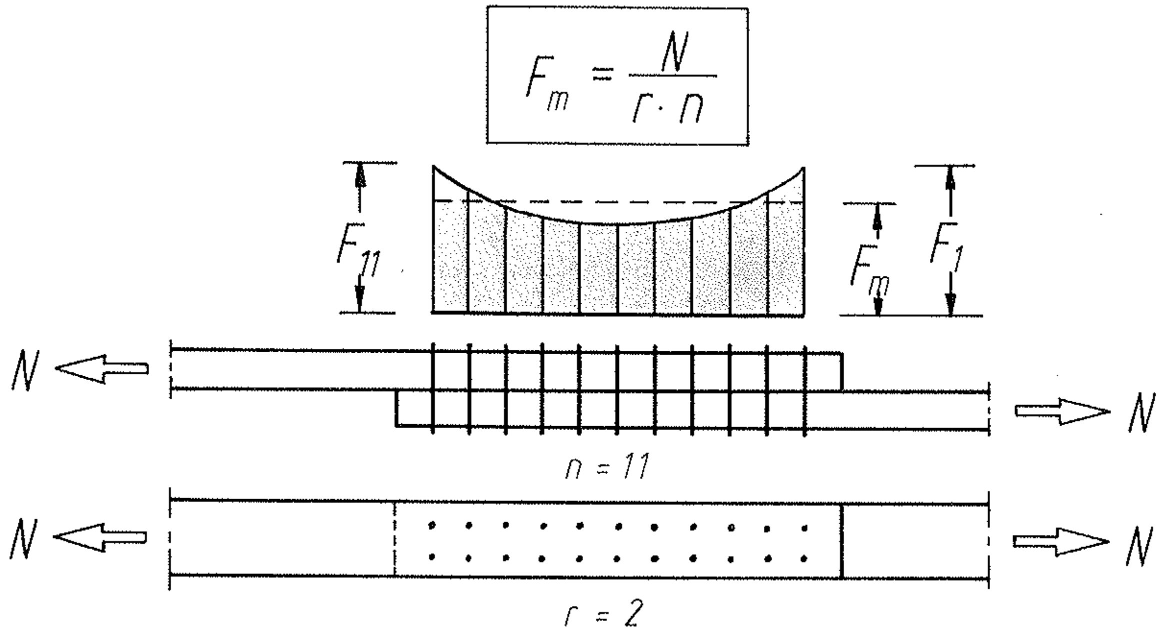


Fig. 1 Load distribution in joints with mechanical fasteners

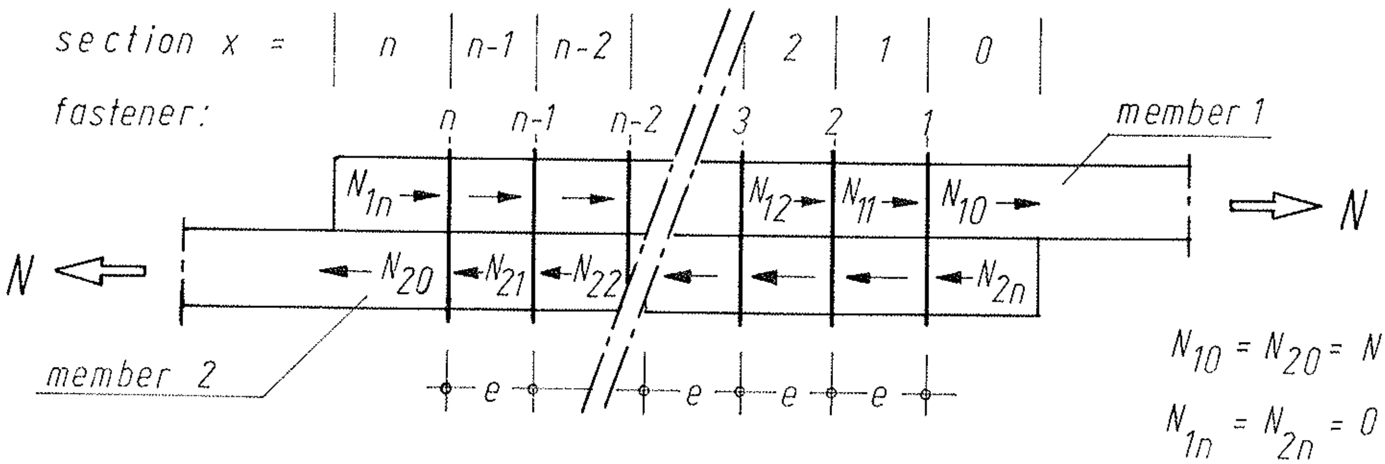


Fig. 2 Notation

3. Conclusion

Based on Lantos' investigation /1/ the formulas (6.1.1.1b) and (6.1.2.1g) of CIB-Code /2/ were checked and improved approaching functions were proposed. In case of joints with dowels the efficiency η is for n = 3 and 4 a bit lower, and for n > 4 higher in comparison to the CIB-formula. For nailed joints the values of η are in every case below the values of η_{CIB}.

4 REFERENCES

/1/ Lantos, G.: Load Distribution in Multiple Mechanical Fasteners in Joints Subjected to Direct Load. Timber Research and Development Association (TRADA), Research Report No. E/RR/28. High Wycombe, Bucks. 1967

/2/ CIB Structural Timber Design Code. CIB-Report Publication 66, 1983

/3/ Ehlbeck, J. and H.J. Larsen: Load-Slip-Relationship of Nailed Joints. Proceedings CIB-W18 meeting, Warsaw, Poland, May 1981, Paper 14-7-3.

CIB-W18/17-9-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ON THE LONG-TERM CARRYING CAPACITY OF WOOD STRUCTURES

by

Y M IVANOV and Y Y SLAVIK
Russia

RAPPERSWIL

SWITZERLAND

MAY 1984

ON THE LONG-TERM CARRYING CAPACITY OF WOOD STRUCTURES

by Yu.M.IVANOV and Yu.Yu.SLAVIK

The design carrying capacity P_d (in using limit state design method) of structure made of material with pronounced influence of load duration on its strength ought to be equalled to the long-term carrying capacity P_{lt} at the close of the service life, i.e.

$$P_d \simeq P_{lt} . \qquad (1)$$

This principal statement presented to the CIB 7th Congress/1/ is detailed below.

The ensuring problem of the wood structure safety consists thus in prediction of the P_{lt}-value. The long-term wood structure carrying capacity P_{lt} being tightly connected with material properties mentioned is obviously depends on the long-term strength of wood. The results attained recently in the uncovering the physical nature of the solid strength/2,3/ establish an general relationship for different materials between time-to-failure t, sec, and stress σ by the equation

$$\log t = \log A - \alpha\sigma . \qquad (2)$$

The half-logarithmic dependence $\log t(\sigma)$ was proposed for wood by Wood/4/, Leont'ev/5/, Pearson/6/. There is now the possibility to reveal the physical sense of the values entered into the equation (1). Here

$$\log A = \frac{U_o}{2,3\ RT} + \log \tau_o \ ; \alpha = \frac{\gamma}{2,3RT} \ ; \qquad (3)$$

U_o is the activation energy of failure process; τ_o is the period of atom oscillations; γ is activation volume; R is gas constant; T is temperature, K.

The process of gradual accumulation of submicroscopic damage in solid occupys the most portion of the time to moment of solid failure under load. Accelerated approching to this moment begins when submicroscopic cracks spreaded in solid and stabilized reache some limit concentration and start to coalesce. This smaller portion of the time t corresponds to the process of crack development where might be applied the fracture mechanic methods. However, the last stage of the failure process according to/2/ obeys also to the relationship mentioned above. The failure has here a cumulative nature having been inherent to a deterministic mechanism of chemical bond cleavage. Therefore, the relative wood strength decrease under load is also deterministic, in contrast with absolute wood strength values wich are random variables. This circomstance enables the prediction of the long-term wood strength for given service life of structures.

As a basis of long-term wood strength prediction serves the constancy (at T=Const) of the abscissa portion log A being cut by the straight line according to (2). For its defining we need to know constants U_o and τ_o. Corresponding to valency bond cleavage $\log \tau_o = -13$ as for many polymers. The energy value U_o we will find by the compression along the grain test results for oven-dry oak wood having been carried out with constant rate and at different temperature +25+100°C. Here the relationship $\sigma_{ul}(T)$ (at log t=Const) is linear one with the confidence interval ±1,61 p.c. and confidence level 0,95/7/.

According to (2) we can write

$$\gamma_i = \frac{U_o - 2,3RT(\log t + 13)}{\sigma_i} .$$

Excluding from this equation coefficient γ_i for each two pairs of values T_i ; σ_i , we will obtain the equation for U_o:

$$U_o = 2,3R(\log t + 13) \frac{T_1\sigma_2 - T_2\sigma_1}{\sigma_2 - \sigma_1} . \qquad (4)$$

For 10 pairs of the values T_i and σ_i it was obtained the magnitude $U_o = 170 \pm 2,9$ kJ/mole.

Disposing the energy value U_o we can precalculate the magnitude of log A for comparison with its values found on graghs log t(σ) for experimentation at different temperatures. Under common atmosphere conditions (~20°C) has been adopted rounding-off log A=17 which satisfys /with small error within ~ 2-3 p.c./ both its theoretical magnitude found above (by means of the constants given) and the experimental data presented in Fig.1a with the following confidence intervals: for

Fig.1

tension along the grain of larch wood at uninterrupted loading (Fig.1a, 1; m.c.^{w}=14,7 p.c. /8/) ±3,6 p.c.; compression along the grain of pine at uninterrupted loading (Fig.1a,2;w=15 p.c. ±2,10 p.c. and Fig.1a,3;w= =30 p.c. /9/ ±2,04 p.c.; shear in torsion tests with tubular specimens of Douglas fir at ramp loading (Fig.1a,4;w=10-12 p.c. /10/ ±5,59 p.c. and for bending under permanent long-term load during up to 5 years of Douglas fir (Fig.1a,5;w=12 p.c. /4/)x/ ±3,4 p.c. The straight line by the equation (2) represents the long-term strength of a common lumber with ±6 p.c. confidence interval and confidence level 0,90. Here for uninterrupted loading t defined according to the test duration $t'_{/}$ from (7), see below.

From the similarity of triangles (Fig.1b) we will obtain

$$\frac{\sigma_{lt}}{\sigma_t} = \frac{\log A - \log \tilde{t}}{\log A - \log t}.$$

This equation is used for prediction of wood long-term strength σ_{lt} by short-term values of the ultimate stress σ_t and time characteristic t. From here

$$\sigma_{lt} = \frac{\sigma_t}{K_1(t)}, \qquad (5)$$

where $K_1(t)=\frac{\log A-\log t}{\log A-\log \tilde{t}} > 1$ is the coefficient of long-term strength.

The $K_1(t)$ values might be found by their dependence on load actions applied to structure. Inasmuch as the failure mentioned has a cumulative nature (i.e.wood long-term strength depends on the summary load action during structure service life) the strength control only by maximum load at its one-fold action during service period does not give to structure any guarantee of reliable work. In securing the latter it needs to allow not only the maximum load variability but also their time characteristics. Under structure service conditions and in testing might be encountered different cases of load action (see schemes in Fig.2). At σ=Const (Fig.2a) time action of σ equalls to t; at periodical application of σ=Const (Fig.2b) obviously $t=\sum \Delta t_i$;for each action of stress σ_i during Δt_i the addition to the time-to-failure

Fig.2

---

x/ The tests in work /4/ were carried out at 26,6°C; log A=16,7 < 17 i.e. ~ 2 p.c. less; that in work /10/ at 21,1°C.

is

$$\frac{\Delta t_i}{t(\sigma_i)},$$

where $t(\sigma_i)=Ae^{-\alpha\sigma_i}$ is the time-to-failure t_f at σ=Const and the failure will occure when the known condition is fulfilled(11)

$$\sum_{i=1}^{n}\frac{\Delta t_i}{t(\sigma_i)}=1,$$

or at uninterrupted change of stress σ in function of time t as the integral

$$\int_0^{t_f}\frac{dt}{t(\sigma)}=1 . \qquad (6)$$

By means of (6) we can find time-to-failure in case of uninterrupted loading (Fig.2c) with constant rate $\overline{w}$ as

$$t=\frac{1}{\alpha\overline{w}} ,$$

where $\overline{w}=\frac{\sigma}{t'_{\prime}}$ and $t'_{\prime}$ is test duration up to the failure moment at $\sigma=\sigma_{ul}$. Thus $t=\frac{t'_{\prime}}{\alpha\sigma}$; $\alpha\sigma=\frac{t}{2,3(\log A-\log t)}$. (7)

The ramp loading with sufficient number of load fractions might be roughly equalled to constant rate loading. The rest cases (Fig.2,e-h) are combinations of that mentioned.

Having found the wood long-term strength for the given load duration under structure service conditions we can predict long-term structure carrying capacity. Indeed, if the invariability of calculation scheme and the conservation of initial wood quality are ensured during service life, the progressive decrease under load of structure carrying capacity will be determined only by the long-term strength of wood. Knowing the latter, we can find the design carrying capacity P_d (in the limit state method design stresses are equalled to the long-term strength).

The control of the value P_d obtained realises by direct determination of controllable short-time str-ucture carrying capacity P_t (i.e. ultimate load) from short-term test with time characteristic t.

Replacing according to the mentioned above in (5) σ_{1t} by (P_{1t})

and σ_t by P_t, we will find/12/

$$(P_{1t}) = \frac{P_t}{K_1(t)} , \qquad (8)$$

where $K_1(t)=B_1-C_1\log t$ is the time-component of structure safety factor (s.f.).

The value (P_{1t}) obtained is obviously found with some error inasmuch as a confined number of structure specimens are tested. Possible changes of (P_{1t}) in understate direction ought to be compensate augmenting it ba $K_2 > 1$ as much, where K_2 is the probability-component of s.f., which allows a variability of a structure workmanship quality. Let us give an example. Glue laminated beams (by 9m span and 60cm cross section height, with 2-3 grade lumber and phenolformaldehyde cold setting glue) in number 292 were tested up to failure. By statistical analysis of these data on computer the applicability was substantiated of log-normal distribution(13), according to which $K_2= e^{\varkappa\nu}$, where $\nu=0,143\pm$ $\pm0,01$; $\varkappa=1,64$ at the confidence level 0,95; from here $K_2=e^{1,64.0,153}=$ $=1,3$.

According to stated above the further normalization improvement in the calculations of design wood structure carrying capacity is developed differentiating both values of $K_1(t)$ and K_2 by field of application and kind of structures.

REFERENCES

1. Ivanov Yu.M. Report to the CIB 7th Congress, Edinburgh,1977.
2. Zurkov S.N.,Korsukov V.E.Fisika tverdogo tela,1973,v.15,N 7.
3. Zurkov S.N.,Kuksenko V.S.,Petrov V.A. In book:"Budusee nauki",1983, M.N 16.
4. Wood L.W. Forest Products Laboratory, NR 1916.Dec.1951.
5. Leont'ev N.L.Dlitel'noe soprotivlenie drevesiny.M.1957.
6. Pearson R.G. Holzforschung,1972,v.26,H.4.
7. Ivanov Yu.M. Lesnoi jurnal,1982,n 5.
8. Perelygin L.M. Zavodskaiia laboratoriia.1938,N 1.
9. Liubosiz M.I.Vliianie skorosti ispytaniia na predel plastičeskogo tečeniia drevesiny sosny.Candidat.diss.,Minsk.1950.
10. Madsen B. Forest Products Journal,1975.v.25,N 10.
11. Bailey J. Journal Glas Industry.1939, N 3.
12. Ivanov Yu.M. Report to the First Conference on wood fracture.Banff. Alberta, Canada.1978.
13. Krakovskii M.B.,Znamenskii E.M.,Zvetkov A.K.,Slavik Yu.Yu.Stroitel'naiia mehanika i rasčet soorujenii.1983,N 6.

SUMMARY

An attempt has been made, allowing a cumulative nature of the deterministic relative decrease of wood strength under load of various duration, to substantiate the prediction of long-term strength of wood and long-term carrying capacity of structures on the base of short-term test results.

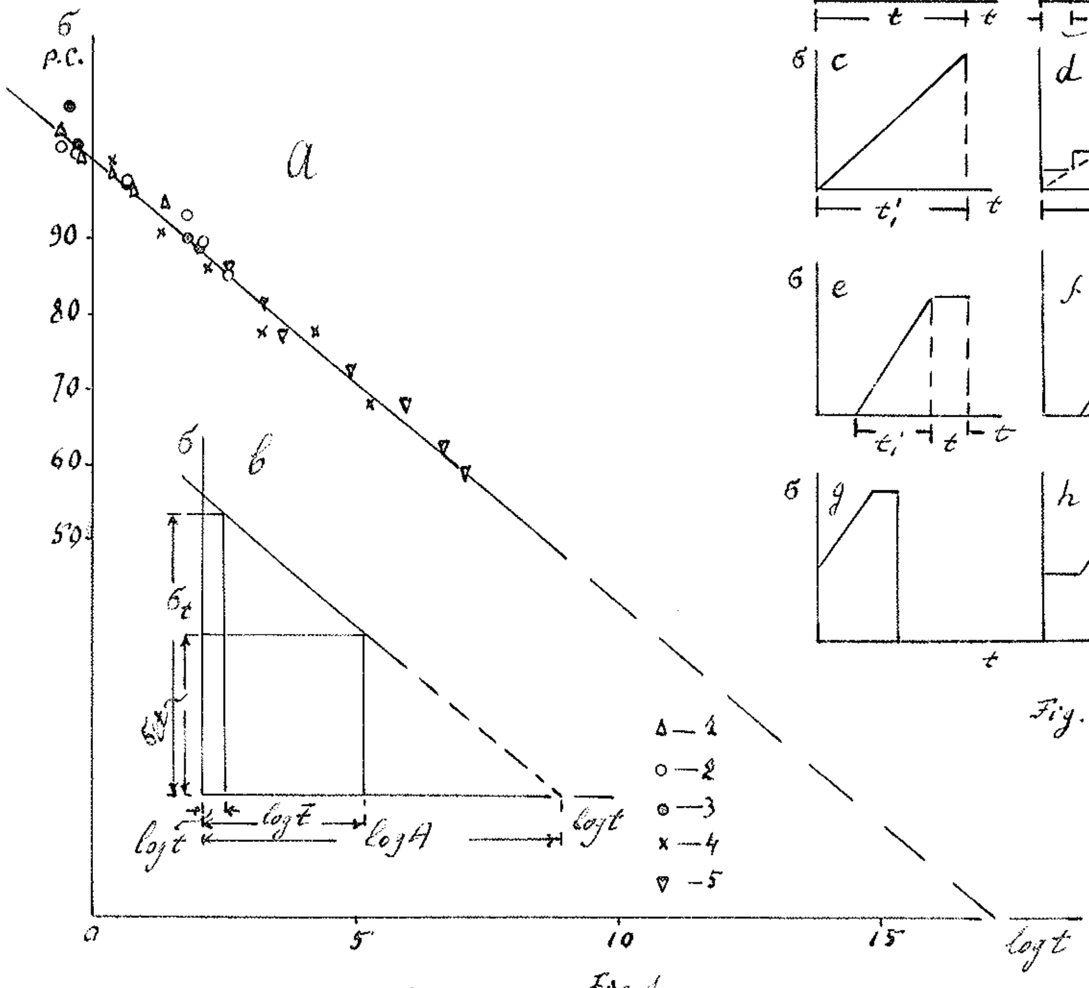


Fig. 1

Yu. M. Ivanov, Yu. Yu. Slavik

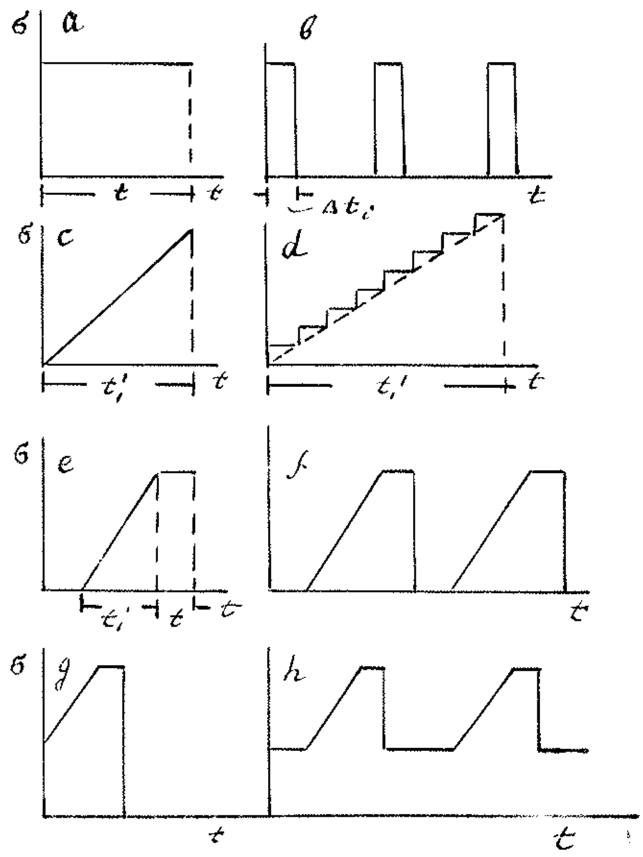


Fig. 2

CIB-W18/17-14-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DATA FROM FULL SCALE TESTS ON PREFABRICATED TRUSSED RAFTERS

by

V Picardo

Institute for Industrial Research & Standards
Ireland

RAPPERSWIL

SWITZERLAND

MAY 1984

The data shown in the following tables supplement CIB-W18 Paper 16-14-1 presented in Norway (June 1983) and is given here at the request of various members of the Group. Because of the volume of data it was necessary to present the data in 5 separate tables as follows

Table A.1	Physical Measurements of Trusses
Table A.2	Plate Sizes
Table A.3	Deflections from 24 hour deflection tests
Table A.4	Deflections from Strength Tests
Table A.5	Failure Loads and Description of Failure
Table A.6	Deflection Recovery & Span/Deflection Ratios at Long Term Design Load.

Tables 1 & 2	Phase I included Truss numbers 1-48 Phase II included Truss numbers 49-61.
Tables 3 & 4	For description of tests see IS 193 P : 1978 Appendix A clause A2 or BS CP 112 : Part 3. Although deflections were taken at 5 points on the ceiling tie only the deflections at centre span and C.T. node points are given.
Table 5	Load factors are given separately for ceiling-tie, rafter and tank loads. For design loads see CIB 16-14-1.

Table A.1 PHYSICAL MEASUREMENTS OF TRUSSES

Truss No.	Span mm	Pitch deg	Timber Dimensions: Rafter mm	Timber Dimensions: Ceiling Tie mm	Truss Self Wt kg	Moisture Content Range at Test (%)
3	8300	30	98 x 36 100 x 37	98 x 36 98 x 35	37.5	14-15
4	8300	30	97.5 x 35	112 x 34.5	42	14-16.5
5	8300	30	98 x 35 96.5 x 35	112 x 36.5 112 x 36	40	14-15
6	8300	30	97 x 35 98 x 35	111 x 35 111 x 36	37	13-15
7	8300	30	112 x 35 112 x 35	97 x 35 96 x 35	41	13-15
8	8300	30	112 x 35 109 x 35	96.5 x 34.5 96 x 34.5	38	13.5-15
9	8300	30	110 x 34.5 110 x 35	96.5 x 35 96 x 35	41	13.5-16.5
10	9100	30	109.5 x 34.5 110 x 34	113 x 35.5 110 x 34	53	14.5-15.5
11	9100	30	110.5 x 35 110 x 35	111 x 34 111 x 34.5	48.5	13.5-15
12	9100	30	110 x 34.5 110.5 x 35	112 x 35 112.5 x 35	45.5	13-14.5
13	6000	30	71 x 34.5 71 x 35	73 x 35 72 x 35	21.5	13.5-14.5
14	5990	30	71 x 35.5 71.5 x 34	72.5 x 35 73 x 35	21.5	13-14.5
15	6000	30	71.5 x 34.5 71.5 x 34.5	72.5 x 35.5 72 x 35	22	13-15
16	7580	17.5	96.5 x 34	96 x 35	32	13.5-15
17	7586	17.5	96 x 35 98 x 35.5	96 x 34.5 98 x 35	32	13.5-15
18	7586	17.5	97 x 34.5 95.5 x 34.5	97 x 35 96.5 x 35	29	13.5-14
19	5369	17.5	72.5 x 34 72.5 x 35	72 x 34.5 72.5 x 34	17.5	13-14
20	5410	17.5	72 x 35.5 71.5 x 34.5	72 x 35.5 72 x 35	16.5	13-14
21	5394	17.5	72 x 35 72 x 34	72 x 35 72 x 34.5	18	12.5-13.5

Table A.1 continued

Truss No.	Span mm	Pitch deg	Timber Dimensions: Rafter mm	Timber Dimensions: Ceiling Tie mm	Truss Self Wt kg	Moisture Content Range at Test (%)
22	5600	22.5	72 x 35 72 x 35	72 x 35 72 x 35	18	12-13.5
23	5600	22.5	72 x 35 72 x 35	72 x 35 72 x 35	19	12-13.5
24	5600	22.5	72 x 35 72 x 35	72 x 35 72 x 35	18	12-13
25	8985	17.5	119 x 35 117 x 35	121 x 35 118 x 35	45	16-17.5
26	9015	17.5	118 x 34 120 x 35	121 x 34 120 x 34	45	13-15%
27	8990	17.5	119 x 34 118 x 34	120 x 35 119 x 34	45	13-14.5
28	8365	17.5	110 x 34	112 x 34	37	12-12.5
29	8388	17.5	110 x 34	111 x 34	37	11-12.5
30	8384	17.5	109 x 34	110 x 34	37	11.5-12.5
31	8077	22.5	97 x 34	96 x 34	34	12-13
32	8080	22.5	93 x 34 94 x 34	94 x 34 96 x 34	34	11-12
33	8092	22.5	95 x 34 96 x 34	95 x 34 95 x 34	34	11.5-12
34	9290	22.5	119 x 34 115 x 34	119 x 34 119 x 33	48	11-13.5
35	9300	22.5	116 x 34 116 x 34	118 x 35 119 x 33	48	11-13.5
36	9290	22.5	117 x 34 118 x 35	120 x 35 116 x 35	44.5	12-14
37	8692	22.5	96 x 35 97 x 35	110 x 35 110 x 35	37.5	11-14
38	8695	22.5	97 x 35 97 x 34	111 x 35 110 x 35	37	13-14
39	8700	22.5	97 x 35 96 x 34	111 x 35 110 x 35	38	12.5-13.5
40	8690	22.5	112 x 35 110 x 35	96 x 35 97 x 35	37.5	12.5-15
41	8709	22.5	110 x 35 110 x 35	97 x 35 98 x 35	39.5	13.5-14
42	8696	22.5	110 x 35 110 x 35	96 x 35 97 x 35	38	13-14

Table A.1 continued

Truss No.	Span mm	Pitch deg	Timber Dimensions: Rafter mm	Timber Dimensions: Ceiling Tie mm	Truss Self Wt kg	Moisture Content Range at Test (%)
43	8696	22.5	110 x 35 107 x 34	110 x 35 110 x 34	41.5	12-14.5
44	8680	22.5	110 x 35 109 x 34	110 x 35 109 x 34	37.5	13-14.5
45	8696	22.5	110 x 36 110 x 34	109 x 34 110 x 35	41	13-14.5
46	9790	30	120 x 34 120 x 35	120 x 34 117 x 34	51.5	14-17
47	9780	30	120 x 35 120 x 35	117 x 34 120 x 35	54	15-17
48	9786	30	118 x 33 118 x 34	117 x 34 117 x 35	53	13.5-16.5
50	8408	30	111 x 34 111 x 34	111 x 34 111 x 34	41	13.5-18.5
51	8404	30	111 x 34 111 x 34	111 x 34 111 x 34	40	13.5-15.5
52	8300	30	97 x 41 97 x 41	97 x 41 97 x 41	41	15.5-18.5
53	8307	30	97 x 41 97 x 41	97 x 41 97 x 41	41	17.5
54	8297	22.5	117 x 34 117 x 34	117 x 34 117 x 34	41.5	15-18.5
55	8306	22.5	117 x 34 117 x 34	117 x 34 116 x 34	37.5	16.5-20
56	8110	22.5	97 x 41 97 x 41	97 x 41 96 x 41	37	14.5-19
57	8090	22.5	96 x 41 96 x 41	96 x 41 96 x 41	38.5	13-14.5
58	7590	17.5	99 x 41 98 x 41	98 x 41 95 x 41	35.5	14-16
59	7608	17.5	97 x 41 97 x 41	96 x 41 98 x 41	35.5	16.5-19.5
60	5885	17.5	96 x 34 96 x 34	96 x 34 96 x 34	24.5	14-15
61	5884	17.5	96 x 34 96 x 34	97 x 34 97 x 34	23	15.5-17

Table A.1 : PHYSICAL MEASUREMENTS OF TRUSSES

Table A.2 PLATE SIZES

Truss No.	Heel Plate	Rafter Node	Ceiling Tie Node	Ceiling Tie Splice	Apex
3	160 x 76	80 x 50	121 x 100	180 x 100	121 x 100
4	180 x 100	80 x 50	120 x 100	259 x 124	180 x 100
5	180 x 100	80 x 50	120 x 100	260 x 104	180 x 100
6	180 x 100	80 x 50	120 x 100	259 x 125	180 x 100
7	180 x 100	80 x 50	120 x 100	180 x 100	180 x 99
8	180 x 100	80 x 50	120 x 100	180 x 100	180 x 100
9	180 x 100	80 x 50	120 x 100	180 x 100	180 x 100
10	180 x 100	80 x 50	120 x 100	180 x 100	180 x 100
11	180 x 100	80 x 50	120 x 100	180 x 100	180 x 100
12	180 x 100	80 x 50	120 x 100	180 x 100	180 x 100
13	120 x 75	60 x 50	120 x 76	166 x 73	120 x 100
14	120 x 76	60 x 50	120 x 75	160 x 71	120 x 100
15	120 x 75	60 x 50	120 x 75	160 x 76	120 x 100
16	220 x 100	80 x 50	120 x 100	304 x 92	180 x 100
17	220 x 100	80 x 50	120 x 100	305 x 102	180 x 100
18	220 x 100	80 x 50	120 x 100	302 x 100	180 x 100
19	160 x 76	80 x 50	120 x 100	160 x 65	120 x 100
20	160 x 76	80 x 50	120 x 100	160 x 60	120 x 100
21	160 x 76	80 x 50	120 x 100	160 x 64	120 x 100
22	180 x 64	50 x 60	75 x 120	180 x 64	100 x 120
23	180 x 64	50 x 60	75 x 120	183 x 64	100 x 120
24	180 x 64	50 x 60	75 x 120	181 x 64	100 x 120
25	260 x 125	50 x 80	120 x 100	260 x 125	260 x 125
26	260 x 125	50 x 80	120 x 100	310 x 102	260 x 125
27	260 x 125	50 x 80	120 x 100	260 x 125	260 x 125
28	220 x 100	50 x 80	120 x 100	220 x 100	260 x 125
29	220 x 100	50 x 80	120 x 100	220 x 100	260 x 125
30	220 x 100	50 x 80	120 x 100	220 x 100	260 x 125
31	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
32	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
33	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
34	180 x 100	50 x 80	120 x 100	301 x 102	261 x 125

Table A.2 continued

Truss No.	Heel Plate	Rafter Node	Ceiling Tie Node	Ceiling Tie Splice	Apex
35	180 x 100	50 x 80	120 x 100	260 x 125	260 x 125
36	180 x 100	50 x 80	120 x 100	260 x 125	260 x 125
37	180 x 100	50 x 80	120 x 100	260 x 125	260 x 125
38	180 x 100	50 x 80	120 x 100	260 x 125	260 x 125
39	180 x 100	50 x 80	120 x 100	260 x 125	260 x 125
40	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
41	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
42	180 x 100	50 x 80	120 x 100	220 x 100	260 x 125
43+	180 x 100	50 x 80	120 x 100	307 x 100	260 x 125
44+	180 x 100	50 x 80	120 x 100	307 x 100	260 x 125
45+	180 x 100	50 x 80	120 x 100	300 x 100	260 x 125
46+	180 x 100	50 x 80	100 x 120	260 x 125	260 x 125
47+	180 x 100	50 x 80	100 x 120	260 x 125	260 x 125
48+	180 x 100	50 x 80	100 x 120	260 x 125	260 x 125
50	157 x 101	50 x 100	101 x 117 101 x 118	184 x 101	129 x 151
51	157 x 101	50 x 100	117 x 101 119 x 101	185 x 101	129 x 151
52	140 x 101	50 x 100	101 x 101	120 x 76	129 x 126
53	140 x 101	50 x 100	100 x 101	200 x 76	129 x 126
54	185 x 101	50 x 100	100 x 101	185 x 100	169 x 126
55	185 x 101	50 x 100	100 x 101	185 x 101	167 x 126
56	200 x 101	50 x 100	100 x 101	229 x 76	143 x 126
57	200 x 101	50 x 100	100 x 101	229 x 75	143 x 126
58	256 x 101	50 x 100	128 x 101 127 x 101	256 x 101	156 x 126
59	256 x 101	50 x 100	126 x 101	256 x 101	157 x 126
60	257 x 75	50 x 100	100 x 101	229 x 75	157 x 126
61	257 x 75	50 x 100	100 x 101	229 x 76	157 x 126

Table A.2 : PLATE SIZES

+ Rafter splice plate 220 x 100

Table A.3 DEFLECTIONS FROM 24 HOUR TEST

Truss No.	Deflection 24 Hour (mm)			After Release Load (mm)		
	Node 1	Node 2	Centre	Node 1	Node 2	Centre
3	Strength Only					
4	7.3	7.6	10.1	2.1	2.1	2.4
5	6.2	6.5	7.4	1.3	1.4	1.1
6	7.9	6.9	9.7	1.9	1.6	2.1
7	5.9	6.0	8.2	1.4	1.5	1.9
8	6.3	6.7	9.5	1.4	1.7	2.0
9	6.1	5.7	8.1	1.1	0.9	0.9
10	7.0	6.9	9.8	1.4	1.3	1.6
11	6.1	6.6	8.3	1.0	1.1	1.1
12	Strength Only					
13	5.3	5.6	7.9	0.9	1.1	1.4
14	4.8	4.8	5.9	0.8	0.8	1.0
15	4.7	4.5	5.9	0.9	0.9	1.1
16	9.7	9.3	11.2	1.8	1.5	1.9
17	10.4	10.5	13.2	0.8	1.0	1.0
18	Strength Only					
19	Strength Only					
20	6.9	6.4	8.0	0.6	0.7	0.6
21	Strength Only					
22	Strength Only					
23	Strength Only					
24	6.2	6.2	6.6	0.8	0.8	0.4
25	Strength Only					
26	Strength Only					
27	17.8	15.6	20.0	6.5	5.4	6.7
28	Strength Only					
29	Strength Only					
30	14.76	14.32	16.8	3.8	3.88	4.48

Table A.3 continued

Truss No.	Deflection 24 Hour (mm)			After Release Load (mm)		
	Node 1	Node 2	Centre	Node 1	Node 2	Centre
31		Strength Only				
32		Strength Only				
33	9.6	10.9	12.4	1.6	2.0	2.2
34	11.6	11.68	12.8	2.8	3.2	4.0
35		Strength Only				
36		Strength Only				
37		Strength Only				
38		Strength Only				
39	10.7	10.9	13.2	1.5	1.8	2.1
40		Strength Only				
41		Strength Only				
42	8.8	9.4	11.1	1.5	1.6	1.5
43		Strength Only				
44	8.7	9.3	12.1	1.6	2.2	2.5
45		Strength Only				
46		Strength Only				
47	10.7	10.8	11.6	*	2.4	2.7
48		Strength Only				
50		Strength Only				
51		Strength Only				
52		Strength Only				
53		Strength Only				
54		Strength Only				
55		Strength Only				
56		Strength Only				
57		Strength Only				
58		Strength Only				
59		Strength Only				
60		Strength Only				
61		Strength Only				

* Transducer Slipped.

Table A.3 : DEFLECTIONS FROM 24 HOUR TEST

TABLE A.4 : DEFLECTIONS FROM STRENGTH TEST

No.	Design (Strength) Without Point			Design + Pt. Load (Strength)		
	Node 1	Node 2	Centre	Node 1	Node 2	Centre
3	6.1	6.4	8.2	7.3	7.4	15.7
4	5.4	5.8	7.0	6.5	6.9	13.8
5	5.0	4.8	6.4	5.8	5.7	12.6
6	5.7	5.3	7.6	6.9	6.2	13.9
7	4.7	4.5	5.8	5.2	5.3	12.4
8	5.9	5.9	8.3	6.8	6.8	15.2
9	5.3	5.2	7.6	6.3	6.0	14.6
10	5.6	5.6	7.4	6.4	6.4	13.7
11	6.4	6.6	8.2	7.4	7.4	14.5
12	6.7	6.0	8.5	7.6	7.6	16.7
13	4.4	4.5	6.6	5.2	5.4	12.8
14	4.1	4.1	5.0	5.2	5.1	10.1
15	4.0	3.6	5.1	4.8	4.5	11.9
16	9.2	9.9	10.9	11.0	11.3	16.6
17	9.4	9.4	11.7	11.2	11.3	17.6
18	9.0	9.2	11.2	10.8	11.1	17.4
19	6.3	6.3	7.9		No Pt. Load	
20	5.8	5.1	6.8		No Pt. Load	
21	6.7	6.5	7.7		No Pt. Load	
22	5.1	5.3	6.1		No Pt. Load	
23	5.2	5.6	6.0		No Pt. Load	
24	5.8	5.6	7.0		No Pt. Load	
25	11.4	11.6	13.7	13.0	13.4	19.1
26	12.4	13.4	14.6	13.8	15.0	20.8
27	12.2	11.8	14.0	13.8	13.6	19.8
28	11.24	10.96	14.52	12.96	12.4	20.36
29	9.6	10.4	11.0	11.2	12.0	16.4
30	12.0	11.96	13.6	13.64	13.52	19.76
31	6.8	7.6	9.0	7.6	8.8	15.9

Table A.4 continued

No.	Design (Strength) Without Point			Design + Pt. Load (Strength)		
	Node 1	Node 2	Centre	Node 1	Node 2	Centre
32	6.52	7.28	9.04	7.68	8.8	16.32
33	7.8	7.9	10.6	8.8	9.4	18.8
34	8.7	9.0	10.6	10.2	10.2	17.6
35	9.4	10.0	11.6	10.4	11.2	18.4
36	9.6	9.4	10.8	10.8	10.6	16.4
37	9.1	9.4	11.4	10.4	10.8	16.8
38	8.4	10.4	11.2	9.8	10.5	17.6
39	8.9	8.8	10.3	10.0	10.0	15.9
40	8.0	8.88	11.32	9.2	10.36	21.24
41	7.68	7.88	10.08	8.88	9.2	18.4
42	7.4	7.9	10.1	8.6	9.1	18.3
43	7.3	7.6	8.8	8.2	8.7	15.6
44	7.2	7.6	9.4	8.5	8.6	15.4
45	7.9	8.0	9.6	9.2	9.2	14.8
46	6.2	6.9	8.6	7.2	7.9	15.9
47	8.4	6.4	7.9	9.2	7.3	14.2
48	7.0	6.8	8.0	8.0	7.6	14.2
50	5.2	5.8	6.6	6.0	6.8	14.2
51	5.2	5.8	6.4	6.0	6.8	12.4
52	6.2	6.0	7.4	7.0	7.0	15.0
53	5.8	5.9	8.0	7.0	7.0	18.0
54	7.4	8.4	8.6	8.6	9.8	14.2
55	7.0	7.2	7.8	8.0	8.3	13.0
56	7.2	7.4	8.4	8.4	8.6	16.0
57	7.4	7.0	8.4	8.6	8.2	15.3
58	9.4	9.8	11.0		No Pt. Load	
59	9.2	9.6	11.0		No Pt. Load	
60	-	6.6	8.6		No Pt. Load	
61	6.8	7.0	8.6		No Pt. Load	

Table A.4 : DEFLECTIONS FROM STRENGTH TEST

Table A.5 : FAILURE LOADS & DESCRIPTION OF FAILURE

(Figures in brackets are Load Factors).

Truss No.	Maximum Load at Failure C. Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	Description of Failure
3	7.18 (2.92)	21.67 (2.86)	2.14 (2.85)	32.25	Lateral movement of apex joint causing fractures in both rafters.
4	6.94 (2.82)	20.36 (2.69)	2.14 (2.85)	30.75	Test terminated due to excessive bow in left rafter.
5	7.88 (3.20)	25.84 (3.41)	2.14 (2.85)	37.15	Test terminated due to excessive bow & twist in left rafter.
6	6.00 (2.44)	18.35 (2.42)	1.94 (2.59)	27.55	Test terminated due to excessive bow & twist in left rafter
7	9.06 (3.68)	28.76 (3.79)	2.33 (3.11)	41.45	Test terminated due to excessive bow & twist in right rafter
8	7.18 (2.92)	21.24 (2.80)	2.31 (3.08)	32	Test terminated due to excessive bow in right rafter.
9	9.65 (3.92)	30.54 (4.03)	2.51 (3.35)	44	Test terminated due to extreme bow in left rafter.
10	10.0 (3.70)	30.93 (3.72)	2.25 (3.01)	44.6	Test terminated due to excessive bow & twist in both rafters.
11	9.18 (3.40)	30.04 (3.61)	2.25 (3.01)	42.85	Left rafter : timber failure just over strut joint, followed by collapse of C.Tie
12	8.00 (2.96)	23.70 (2.85)	2.06 (2.75)	35.1	Lateral failure at apex joint due to excessive bowing.
13	4.71 (2.66)	15.70 (2.88)	1.88 (2.51)	23.4	Timber failure : right rafter between strut intersections and apex.

Table A.5 continued

Truss No.	Maximum Load at Failure				Description of Failure
	C. Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	
14	4.71 (2.66)	15.02 (2.75)	2.06 (2.75)	22.9	Timber failure: right rafter 1 m from apex.
15	5.65 (3.19)	17.88 (3.28)	2.25 (3.01)	23.4	Timber failure: left rafter between strut node & heel, followed by C.Tie collapse.
16	5.53 (2.46)	16.57 (2.51)	1.88 (2.51)	25.2	Timber failure: right rafter midway between strut node and heel.
17	5.41 (2.41)	16.79 (2.54)	1.88 (2.51)	25.3	Test terminated - excessive bowing in left rafter.
18	5.53 (2.46)	14.65 (2.22)	1.88 (2.51)	23.25	Test terminated - excessive bow in top half of both rafters, followed by buckling at apex.
19	5.88 (3.70)	17.94 (3.84)	2.25 (3.01)	26.25	Test terminated - excessive bow. No Point Load.
20	5.06 (3.18)	13.87 (2.97)	2.25 (3.01)	21.35	Test terminated - left rafter fractured due to lateral distortion also severe bow on right rafter. No Point Load.
21	4.59 (2.89)	14.13 (3.03)	2.25 (3.01)	21.15	Timber failure in right rafter between heel plate & strut node. No Point Load.
22	3.06 (1.85)	15.32 (3.11)	1.49 (1.99)	20.05	Timber failure: right rafter between heel & node, followed by break near heel joint - both occurred at knots.

Table A.5 continued

Truss No.	Maximum Load at Failure C.Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	Description of Failure
23	5.18 (3.14)	15.59 (3.17)	2.45 (3.27)	23.4	Plate shear at right node in C.T.
24	4.71 (2.85)	14.26 (2.90)	2.35 (3.14)	21.5	Terminated due to excessive lateral distortion.
25	7.18 (2.69)	19.85 (2.53)	2.08 (2.77)	30.45	Timber failure: left rafter adjacent to heel.
26	7.18 (2.69)	20.55 (2.62)	2.08 (2.77)	31.15	Timber failure: right rafter between heel & node followed by timber failure at node.
27	5.76 (2.16)	15.96 (2.04)	1.69 (2.25)	24.75	Timber failure: left rafter between node and heel (lateral failure).
28	4.12 (2.46)	17.96 (2.46)	1.86 (2.48)	27.2	Timber failure mid-way between apex & strut - left rafter, followed by timber failure at apex left rafter & shearing of C.T. splice plate.
29	7.88 (3.17)	24.02 (3.29)	2.43 (3.24)	35.6	Timber failure adjacent to apex on right rafter, followed by failure between apex & strut on right rafter
30	4.82 (1.94)	14.77 (2.02)	1.49 (1.99)	22.35	Timber failure adjacent to heel plate on left rafter followed by timber failure midway between heel plate & strut on left rafter
31	6.47 (2.70)	18.99 (2.65)	2.06 (2.75)	28.75	Timber failure: right rafter at strut.

Table A.5 continued

Truss No.	Maximum Load at Failure				Description of Failure
	C. Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	
32	6.47 (2.70)	19.09 (2.67)	2.06 (2.75)	28.85	Excessive bowing
33	7.06 (2.94)	19.75 (2.76)	2.25 (3.01)	30.3	Timber failure mid-way between strut & apex on left rafter
34	6.71 (2.43)	20.49 (2.49)	1.88 (2.51)	30.45	Timber failure on right rafter adjacent to apex.
35	6.00 (2.17)	20.19 (2.45)	1.69 (2.25)	29.25	Test terminated due to springing of apex.
36	6.71 (2.43)	20.23 (2.46)	1.88 (2.51)	30.15	Excessive bowing in both rafters.
37	6.24 (2.42)	17.61 (2.29)	1.88 (2.51)	27.0	Timber failure left rafter adjacent to strut, followed by timber failure at left heel plate.
38	4.35 (1.69)	13.64 (1.77)	1.29 (1.73)	20.55	Terminated due to excessive distortion in right rafter
39	6.24 (2.42)	18.31 (2.38)	1.88 (2.51)	27.7	Timber failure: right rafter between loading pts
40	7.53 (2.92)	21.44 (2.78)	2.25 (3.01)	32.5	Timber failure mid-way between strut node & heel of right rafter, followed by failure at node pts
41	7.53 (2.92)	20.10 (2.69)	2.24 (2.98)	31.75	Timber failure: right rafter midway between node & heel followed by timber failure adjacent to heel joint on right ceiling tie.
42	7.53 (2.92)	21.54 (2.80)	2.24 (3.01)	32.6	Test terminated due to excessive bowing in left rafter.

Table A.5 continued

Truss No.	Maximum Load at Failure				Description of Failure
	C. Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	
43	6.94 (2.69)	20.94 (2.72)	2.06 (2.75)	31.25	Rafters spliced: Brash timber failure in right ceiling tie at node, followed by failure at node in left rafter.
44	7.53 (2.92)	22.11 (2.87)	2.06 (2.75)	33	Rafters spliced: timber failure in left rafter between 3rd & 4th load pts followed by timber failure at apex & plate shear in C.T.
45	8.82 (3.42)	25.75 (3.34)	2.27 (3.03)	38.15	Rafters spliced: timber failure in left rafter midway between heel & node then over heel plate
46	7.88 (2.71)	23.93 (2.67)	2.08 (2.77)	35.3	Rafters spliced: right rafter severe lateral distortion
47	7.88 (2.71)	24.76 (2.76)	2.08 (2.77)	36.15	Rafters spliced: left rafter compression failure at load point approx 1 m from heel
48	6.59 (2.26)	23.85 (2.66)	2.19 (2.85)	34	Rafters spliced: severe lateral distortion in right rafter just over heel plate at knot cluster.
50	6.59 (2.65)	20.37 (2.66)	2.14 (2.74)	30.4	Timber failure right rafter over node pt followed by failure in ceiling tie between heel & node & plate withdrawal at right heel
51	8.00 (3.21)	24.58 (3.20)	2.53 (3.24)	36.4	Timber failure midway between heel joint & node on left rafter

Truss No.	Maximum Load at Failure C.Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	Description of Failure
52	6.59 (2.68)	19.37 (2.56)	2.14 (2.74)	29.4	Timber failure: right rafter between heel & node followed by timber fracture at node & heel in right rafter.
53	6.59 (2.68)	20.02 (2.64)	2.14 (2.74)	30.05	Timber failure: right rafter at strut node, followed by C.T. failure between heel & node on right side & right Q.T. withdrawal from C.T. node.
54	4.12 (1.67)	11.70 (1.59)	1.37 (1.76)	18.5	Right Q.T. withdrawal at C.T. node Inside plate not correctly positioned.
55	7.88 (3.20)	21.80 (2.97)	2.55 (3.27)	33.5	Right Q.T. plate withdrawal at C.T. node & left Q.T. withdrawal at apex
56	6.47 (2.70)	19.53 (2.72)	2.14 (2.74)	29.4	Timber failure - left rafter between node & apex
57	5.88 (2.45)	16.10 (2.25)	1.94 (2.49)	25.2	Right Q.T. withdrawal at C.T.node followed by timber failure at midspan of C.T.
58	6.00 (2.67)	16.91 (2.56)	2.14 (2.74)	25.4	Timber failure : right rafter between apex & strut node - No Point Load

Table A.5 continued

Truss No.	Maximum Load at Failure C. Tie kn (LF)	Rafter kn (LF)	Tank kn (LF)	Total kn	Description of Failure
59	5.41 (2.41)	15.18 (2.30)	1.96 (2.51)	22.9	Timber failure : right rafter between apex & strut node after severe distortion - No Pt. Load
60	4.59 (2.64)	13.73 (2.69)	2.14 (2.74)	20.7	Terminated due to excessive distortion - No Point Load
61	4.59 (2.64)	13.95 (2.74)	2.14 (2.74)	20.9	Timber failure : right rafter between heel joint & strut node. - No Point Load

LF = Load Factor.

Table A.5 : FAILURE LOADS & DESCEIPTION OF FAILURE.

Table A.6

Truss No.	Recovery %			Average Recovery %	Centre Span/Defl Ratio	Midspan/ Node Defl Ratio
	Node 1	Node 2	Centre			
4	71.2	72.4	76.2	73.3	812	1.36
5	79.0	78.5	85.1	80.9	1108	1.16
6	75.9	76.8	78.3	77.0	845	1.311
7	76.3	75.0	76.8	75.9	1000	1.38
8	77.8	74;6	78.9	77.1	863	1.46
9	82.0	84.2	88.9	85.0	1012	1.37
10	80.0	81.2	83.7	81.6	918	1.41
11	83.6	83.3	86.8	84.6	1084	1.30
13	83.0	80.4	82.3	81.9	747	1.45
14	83.3	83.3	83.0	83.2	999	1.23
15	80.8	80.0	81.4	80.7	1000	1.28
16	81.4	83.9	83.0	82.8	668	1.18
17	92.3	90.5	92.4	91.7	567	1.26
20	91.3	89.1	92.5	90.9	664	1.20
24	87.1	87.1	93.9	89.4	834	1.07
27	63.48	65.4	66.5	65.1	445	1.20
30	74.2	72.9	73.3	73.5	493	1.16
33	83.3	81.6	82.3	82.4	655	1.20
34	75.9	72.6	68.7	72.4	718	1.10
39	86.0	83.5	84.1	84.5	652	1.22
42	83.0	83.0	86.5	84.1	774	1.22
44	81.6	76.3	79.3	79.1	709	1.34
47	59.8	77.8	76.7	71.4	835	1.08

Table A.6 : DEFLECTION RECOVERY & SPAN/DEFLECTION RATIOS

CIB-W18/17-102-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAFETY PRINCIPLES

by

H J Larsen

Danish Building Research Institute

and

H Riberholt

Structural Research Laboratory
Technical University of Denmark
Denmark

RAPPERSWIL

SWITZERLAND

MAY 1984

SAFETY PRINCIPLES

by

H. J. Larsen

Through the work of among other organizations CEB (Comité Euro-International du Beton), CIB, EEC (European Economic Community), ECE (Economic Commission of Europe), ISO (International Standardization Organization), JCSS (Joint Committee on Structural Safety) and NKB (Nordic Committee on Building Regulations) agreement has been reached on a number of the basic principles for the safety design of structures, e.g. that it should be based on limit state design, that the partial coefficient method should be used, and that the partial coefficients in principle should be determined in such a way that a uniform probability of failure is ensured for comparable structures.

The principles of limit state design, the partial coefficient method and the way in which the partial coefficients have been fixed in the Nordic countries, are described in the following.

Further is mentioned some of the problems in harmonizing internationally the partial coefficients and some special problems in relation to wood structures.

The following documents have been used:

- Eurocode No.1. Common Unified Rules for Different Types of Construction and Materials (EEC), 1983.
- Recommendation for Loading and Safety Regulations for Structural Design, NKB-report 36, 1978.
- ECE Compendium of Model Provisions for Building Regulations, Chapter 1 - Structural Performance Requirements, 1983.
- Code of Practice for the Safety of Structures, Danish Standard DS 409 (translation), 1983.

1. LIMIT STATE DESIGN

Limit state design only means that the requirements are related to clearly defined limit states, i.e. states in which any of the performance criteria governing the use of the structure are infringed.

Limit state design can be used in connection with permissible stress method as well as with the partial coefficient method and for both elastic and plastic structures.

Limit states are classified into the following categories:

Ultimate limit states which correspond to the loss of static equilibrium of the structure or parts thereof, considered as a rigid body, or attainment of the maximum resistance capacity of the structural system or of individual members.

Serviceability limit states which correspond to a loss of utility beyond which the service conditions are no longer met. Serviceability limit states may correspond to unacceptable deformations or deflections which affect the appearance or efficient use of a structure or cause damage to finishes or non-structural elements; or vibrations producing discomfort or affecting non-structural elements or equipment.

2. DESIGN PARAMETERS

In analytical verification the limit state is expressed by a calculation model involving various parameters and variables, called basic variables.

The following basic variables are involved:

- actions (F)
- strength (f) and other properties of materials, in particular elastic moduli (E)
- geometrical data (a)

Actions

Permanent actions are generally represented by a unique nominal value G. This applies to dead load e.g. to the self weight of the structure, and the weight of the superstructure and fixed equipment, and also to actions resulting from a practically constant level of water, and deformations imposed by settlements.

For most actions, in particular the self weight of the structure, the unique nominal value G is generally calculated as the mean value, i.e. on the basis of the nominal dimensions and mean unit masses.

In some cases, however, it may be necessary to consider other representative values, i.e. either an upper or a lower value, or both. Wind suction and dead load of a roof is an example, where a lower value of the self weight should be used.

Variable actions, Q, can be represented by different values. The most frequent representative values are the characteristic value Q_K and the combination value ψQ_K where $\psi \leq 1$.

The characteristic values are usually prescribed by the competent public authority in national loading regulations, in many cases (especially for climat loads or wind and snow) corresponding to a specified return period e.g. 50 years.

Properties of materials

Properties of materials are represented by their characteristic values. In general, a characteristic value can be presented as a fractile in the distribution of the material properties and properties with a similar influence on the resistance (e.g. the modulus of elasticity in instability design).

With regard to characteristic values for strength (f), they are specified such that they correspond to a p % (e.g. 5% or 50%) fractile.

In certain cases upper and lower characteristic values have to be considered. e.g. if an increase in the resistance results in a decrease in safety due to reduced deformability.

Geometrical data

In design, account should be taken of the possible variation of the geometrical data. In most cases, the variability of the geometrical data may be considered small, or negligible, in comparison with the variability associated with the actions and the material properties. Hence, in general, the geometrical data may be assumed to be non-random and as specified in the design.

For the dimension of cross sections tolerance limits should be given, e.g. as in CIB Structural Timber Design Code, § 5.1.0.

3. PARTIAL COEFFICIENT METHOD

Ultimate limit states: In the majority of cases the action effects S are compared with the corresponding resistance capacity R whereby reference is made to the respective design values S_d and R_d, see below. This comparison may be written as

$$S_d \leqq R_d \tag{1}$$

The definition of S and R depends on the individual design problem and the relations may be scalar, vectorial (e.g. M, N - interaction) or sometimes more complex.

Serviceability Limit States: The condition for verification is generally expressed by:

$$S_d \leq C_d \tag{2}$$

where S_d is the design value of the action effect and C_d represents a fixed value or a function of certain material properties (and then corresponds to R_d).

An action effect is introduced into calculations by its design value S_d. Depending on the design situation, several actions may have to be considered simultaneously which can be done by applying "combination rules", based on the following format:

$$S_d = S(\gamma_{F,1}F_{rep,1}, \ldots \gamma_{F,i}F_{rep,i}, \ldots) \tag{3}$$

where $F_{rep,i}$ are the representative values of the actions. The partial coefficients (load factors) $\gamma_{F,i}$ depend on the nature of the actions and the limit state under consideration.

In some publications the following format is given:

$$S_d = \gamma_S(\gamma_{f,1}F_{rep,1}, \ldots \gamma_{f,i}F_{rep,i}, \ldots) \tag{4}$$

but with

$$\gamma_{F,i} = \gamma_S\gamma_{f,i} \tag{5}$$

the two formats will give exactly the same results, and (4) therefore represents an unnecessary complication.

Corresponding to (3) the following combinations are used (in symbolic presentation):

$$\gamma_{G,max}\, G_{max} + \gamma_{G,min}\, G_{min} + \gamma_{Q,1}Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{o,i}\, Q_{k,i} \tag{6}$$

where

G_{max}	permanent actions the effects of which increase the effect of the variable actions
G_{min}	permanent actions the effects of which reduce the effect of the variable actions
$Q_{k,1}$	basic (variable) action (characteristic value)
$\psi_{o,i}Q_{k,i}$	accompanying variable actions (combination values)

In the loading regulations of the Nordic countries only the products $\gamma_{Q,i}\ \psi_{o,i}$ are given.

The resistance capacity is introduced into calculation by the design value R_d

$$R_d = (f_{k,1}/\gamma_{M,1}, \dots f_{k,i}/\gamma_{M,i}, \dots E_{k,j}/\gamma_{M,j}, \dots) \tag{7}$$

In some publications the following format is given:

$$R_d = R(f_{k,1}/\gamma_{M,1}, \dots f_{k,i}/\gamma_{M,i}, \dots E_{k,j}/\gamma_{M,j}, \dots)/\gamma_R \tag{8}$$

where

$$\gamma_{M,i} = \gamma_{m,i}\gamma_R \tag{9}$$

but this again represents an unnecessary complication: Except for very rare situations, e.g. in connection with friction, a common factor can freely be moved from the left to the right side of the inequality (1) without any influence on the result, i. e. the following three conditions are equal

$$\left.\begin{aligned} \gamma_S \cdot S(\dots\dots) &\leqq R(\dots\dots)/\gamma_R \\ S(\dots\dots) &\leqq R(\dots\dots)/(\gamma_R\gamma_S) \\ \gamma_R\gamma_S \cdot S(\dots\dots) &\leqq R(\dots\dots) \end{aligned}\right\} \tag{10}$$

Having this in mind many futile discussions could be avoided, e.g. which uncertainties should be covered by γ_S, γ_f, γ_m and γ_R respectively, or is it against nature to have a γ_M-factor on E?

The system of partial factors should be given values facilitating the design. In accordance with this γ_F is in the Nordic codes in most cases put equal to 1 for dead load because dead load in some cases has a favourable effect, in others an unfavourable effect.

Geometrical parameters are generally introduced into the calculation by their nominal values ($a_d = a_{nom}$). In some cases safety elements Δa are introduced ($a_d = a_{nom} \pm \Delta a$).

4. DETERMINATION OF PARTIAL COEFFICIENTS' PRINCIPLES

The partial coefficients should in principle be so determined that a uniform probability of failure is obtained for comparable structures irrespective of the materials used.

The probability of failure can in principle be determined as follows, where the most simple case is studies, viz. one action effect S and one resistance parameter R. Both are assumed to be stochastic values, see figure 1.

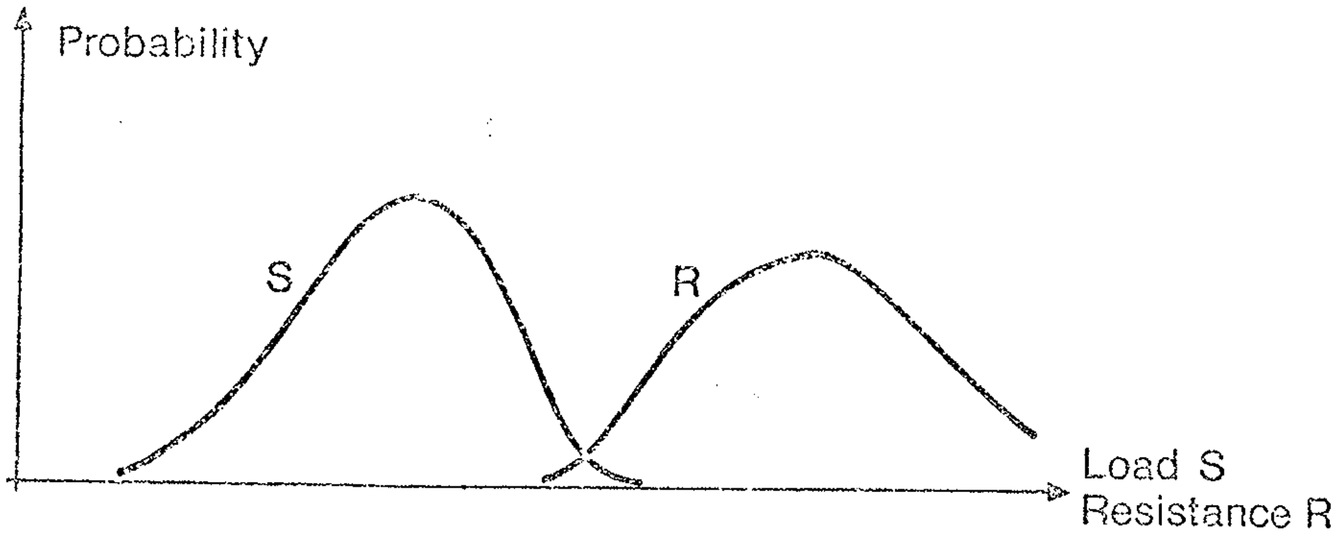


Figure 1. Density function for an action effect S and the corresponding resistance R

The probability of failure p_f is equal to the probability of getting values of S greater than R, or with M = R-S, see figure 2.

$$p_f = P(R<S) = P(M<0)$$

The value of p_f is very sensitive to the form of the tails (the distribution function of high values of S and low values of R) and in practice the necessary information is never available.

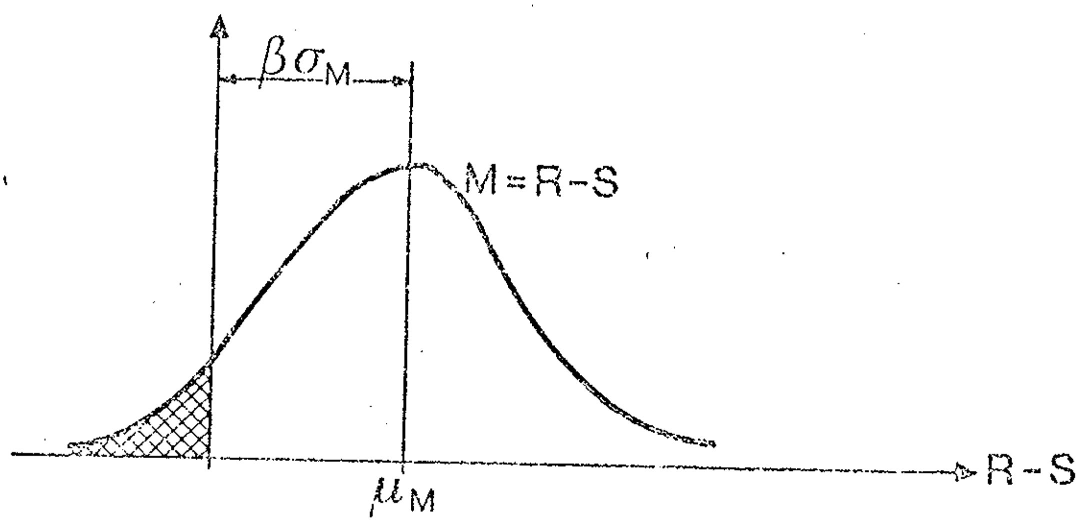


Figure 2. Density function for the failure function M = R-S. The probability of failure is hatched

Usually only the mean value and the variance of R and S are known, together with a rough knowledge of the tails.

Instead of the true, unknown distribution function, standardized distribution functions are used: For S usually a normal distribution, for R a lognormal.

For these conditions it is possible to calculate the probability of failure, i.e. $P(R-S < 0)$, but it is only a formal probability of failure because of the use of the standardized distribution functions instead of the true, unknown.

To underline the formal character, the safety is normally not given by the formal probability of failure, but by a so-called safety-index. The safety index β is defined by

$$\beta = \mu_M/\sigma_M \tag{11}$$

where μ_M is the mean and σ_M the variance for the failure function $M = R-S$, see figure 2.

The relation between formal probability of failure per year and the safety index is illustrated in table 1, which gives the values recommended in (NKB-report 36, 1978), dependent on the type of failure and safety class.

Table 1. Recommended β-values (NKB 1978), and (in brackets) corresponding formal probabilities of failure per year

		Failure type		
		Ductile		Brittle
Safety Class:		with strain-hardening	without strain-hardening	
Low	$\beta =$	3.09 (10^{-3})	3.71 (10^{-4})	4.26 (10^{-5})
Normal	$\beta =$	3.71 (10^{-4})	4.26 (10^{-5})	4.75 (10^{-6})
High	$\beta =$	4.26 (10^{-5})	4.75 (10^{-6})	5.20 (10^{-7})

The Nordic partial coefficient system

The described principles and the β-values in table 1 form the background for the partial coefficients used in the Nordic countries.

The partial coefficients and load combination rules have been calibrated so that the partial coefficient method for a wide range of design situations gives the same dimensions of the structures as the safety index method.

The following is a very brief extract from the Danish Code of practice for the safety of structures, which has been based on NKB-Report 36.

In table 2 are given the load factors for the most common combinations of actions for serviceability and for ultimate limit states. Besides there are further combinations involving accidental loads.

Table 2. Action combinations with corresponding partial coefficients, γ_f

	serviceability limit state	ultimate limit state		
	1	2.1	2.2	2.3
permanent action				
weight of structural members				
G_k fixed action 3)	1.0	1.0	0.85	1.0
$0.15\ G_k$ free action 3)	-	-	-	1.0
weight of earth and ground water	1.0	1.0	1.0	1.0
variable action 1)				
one variable action	2)	1.3	1.3	-
other variable actions	2)	ψ	ψ	-

1) In the table the ψ is to be taken as $1.0 \cdot \psi$, i.e. the usual action, ψQ_k is given the partial coefficient $\gamma_f = 1.0$

2) The variable actions and their values in the serviceability limit state are given in the structural codes.

3) A fixed action takes either the value 0, or its "maximum" value simultaneously on all parts of the structure where it can possibly act. A free action can at any point independently take any value between 0 and maximum.

The principles for the determination of material factors are given in Annex A. For ordinary timber structures with normal degree of control a value of $\gamma_m = 1.5$ is used corresponding to a coefficient of variation of 15% for the structural strength (for the materials the value is usually higher), and ductile failure without extra load-carrying capacity (strainhardening).

For structures made under extended control, i.e. glulam structures, $\gamma_m = 1.35$ is used.

International agreement

No international agreement has been reached on the partial coefficient. There are a number of reasons for that.

- A requirement for the harmonization is that the loads are agreed internationally. This is not the case for the time being, and there is very little progress in harmonizing the loading regulations.
- In most countries the coefficients are not determined on a rational basis, but by comparison with the existing safety factors, and some countries only pretend to use the partial coefficient system. Instead they are trying to maintain the permissible stress system (by using the same load factor on all loads).
- Some of the international organizations, among others CEB, have tried to force through partial coefficients especially suited for their materials.

Most of the proposed systems are, however, in agreement with the following recommendations in the ECE-Compendium, viz.

For safety problems the partial coefficients γ_F for actions may be chosen in the following way.

For permanent actions which are unfavourable	$\gamma_F = 1.0 - 1.3$
For permanent actions which are favourable	$\gamma_F = 0.8 - 1.0$
For variable actions	$\gamma_F = 1.3 - 1.6$
For accidental actions	$\gamma_F = 1.0$

For serviceability problems the values of γ_F may be differentiated to some extent and should be chosen with regard to the nature of the problem. Normally the value of γ_f is about 1.0.

The partial coefficients γ_m for the strength of materials and modulus of elasticity are strongly dependent of the type of material and the variability of the material properties. Following values may be given as examples:

	Strengths of materials	Modulus of elasticity
For concrete	$\gamma_M = 1.25 - 1.5$	$\gamma_M = 1.0 - 1.2$
For steel	$\gamma_M = 1.0 - 1.1$	$\gamma_M = 1.0 - 1.1$

The additive geometrical quantity Δa for cross section dimensions may be chosen so that $a_k \pm \Delta a$ approximately correspond to the tolerance limits.

A partial coefficient γ_n, by which the consequences of failure are taken into account, may be introduced with values varying from $\gamma_n = 0.9$ for consequences which are not serious to $\gamma_n = 1.1$ for consequences which are very serious. With these values γ_n should be multiplied either to γ_F or to γ_M.

5. SPECIAL PROBLEMS FOR TIMBER STRUCTURES

The partial coefficient system described has been developed for concrete and steel and is based on the assumption of time-independent material properties. This assumption is not given explicitely but through the fact that the β-values correspond to a fixed probability of failure per year.

If the γ_m-values proposed for concrete or steel are used together with the usual time-dependent strength and stiffness reductions, timber structures will get a higher safety level than other structures: Only at the end of the assumed lifetime will timber structures have the prescribed β-values, in most of the lifetime - especially for permanent and long-term loading - they will have much higher β-values.

ANNEX A. WEIGHTS AND WEIGHT DENSITIES

In the following, lists are given of the weights, weight densities, masses and mass densities respectively of a number of building materials and goods in bulk.

Where nothing else is stated, the weights of structural parts and weight densities of building materials given may be assumed to include the moisture content which the material concerned at the situation in the building structure will generally have.

In the case of some materials, the weight of which is not very well-defined, a range is given within which the weight may generally be assumed to be found.

When an angle of internal friction is given in the form of a range, the lowest value for the material in question may be assumed to be within the range.

It should be noted, however, that deviations from the values given may occur.

Tables of weights or weight densities and masses or mass densities, respectively, of building materials and goods in bulk:

Table A1. Weight densities and mass densities of building materials

materials	weight density kN/m³	mass density kg/m³
concrete, hardened		
(newly cast concrete about 1 kN/m³ greater)		
reinforced concrete, aggregate natural stone (granite, flint)	24-26	2450-2650
unreinforced concrete, aggregate natural stone (granite, flint)	23-25	2350-2550
structural lightweight concrete	14-20	1430-2040
mortar, hardened		
(freshly mixed mortar about 1 kN/m³ greater)		
cement mortar	21	2140
lime-cement mortar	19	1940
lime mortar	17	1730
masonry cement	19-20	1940-2040
gypsum-lime plaster for Rabitz rendering	15	1530
plaster of Paris	10	1020
natural stone		
basalt	30	3060
granite, gneiss	27	2750
marble, dense limestone	21-27	2140-2750
limestone, porous	13-21	1330-2140
sandstone, dense	22-27	2240-2750
sandstone, loose	14-24	1430-2450
slate	27	2750
manufactured, solid building blocks		
concrete blocks	23	2350
sand-lime bricks	18-20	1840-2040
light-concrete blocks	4-14	400-1430
moler brick	12	1220
brick	14-20	1430-2040
brick, hard burned	17	1730
clinker brick	19-20	1940-2040
wood, air dried (about 15% moisture)		
Scandinavian hardwood	7	710
Scandinavian softwood	5	510
(impregnated wood may have a higher weight density than those given)		

Table A2. Weight densities and mass densities of metals

metal	weight density kN/m³	mass density kg/m³
aluminium	27.0	2750
lead	111.8	11400
bronze	84.3	8600
copper	87.3	8900
brass	83.4	8500
steel, rolled	77.0	7850
cast iron	71.1	7250
cast steel	77.0	7850
tin	72.6	7400
zinc	70.6	7200

Table A3. Weights and masses of roofing materials per m² of roof surface[1]

material	weight kN/m²	mass kg/m²
ordinary roofing slate	0.20-0.25	20-25
asbestos-cement corrugated board	0.15-0.20	15-20
asbestos-cement slate	0.20-0.25	20-25
timber boarding (25 mm thick)	0.15	15
thatch (200 mm thick)	0.50	50
tiled roof, interlocking pantiles[2]	0.55	55
tiled roof, »romer» tiles[2]	0.65	65
tiled roof, »vinge» tiles[2]	0.45	45
laths for tiled roofs	0.05	5
river gravel (for flat roofs) (10 mm thickness)	0.17-0.19	17-19
roofing felt, single layer	0.02-0.05	2- 5

1 Weight per m² of finished roof surface inclusive of normal overlapping of roofing material. When nothing else is mentioned the weight comprises only the actual roofing material. The weight of battens, purlins, rafters, and the like is not included.

2 The weight of tiled roofs are given as the weight of dry roof tiles + about 14% of this weight (corresponding to maximum moisture content) + weight of sheathing (felt or plastic sheet) 0.04 kN/m². If pointing is applied, 0.09 kN/m² shall be added.

Table A4. Weights and masses of non-rendered brickwork per m^2 of wall[3 4]

type of wall	thickness of wall (nom., mm)	weight kN/m^2	mass kg/m^2
half brick	108	1.9	190
broad brick	168	2.9	290
one brick	228	3.9	400
half brick + half brick (cavity wall with wire ties)	290-350	4.0	410
half brick + half brick (cavity wall with headers)	350	4.4	450
brick and a half	350	5.9	600
two brick	470	7.9	800

3 The weights given in the table refer to brickwork of solid bricks with a weight density of 17 kN/m^3 and mortar with a weight density of 17 kN/m^3.

4 The weight of rendering (about 10 mm thick) is assumed to be 0.20 kN/m^2.

Table A5. Weight densities and mass densities of materials for wearing surfaces

material	weight density kN/m^3	mass density kg/m^3
asphaltic concrete, mastic asphalt	22-24	2240-2450
bitumen	10	1020
asphaltic concrete, base course material	21-23	2140-2350
fine cold asphalt	20-22	2040-2240
cement rendering, terrazzo	20-23	2040-2350
tiles of limestone, marble, slate	27	2750
floor clinker, ceramic tiles	20-22	2040-2240
cork tiles	1- 5	100- 510
linoleum, magnesite, rubber, plastics	10-20	1020-2040

Table A6. Weight densities and mass densities of covering and insulating materials

material	weight density kN/m³	mass density kg/m³
asbestos-cement board	18 -22	1840-2240
asbestos-cellulose-cement board	14 -15	1430-1530
asbestos-silicate board	7 - 8	710- 820
gypsum board	9	920
glass	26	2650
plywood, pine	6 - 7	610- 710
expanded sintered clay	2.5- 5	250- 510
mineral wool	0.2- 2	20- 200
foamed plastics	0.1- 0.4	10- 40
chipboard, heavy	8 -10.5	820-1070
chipboard, medium heavy	4 - 8	410- 820
woodfibre board, hard	8 -10	820-1020
woodfibre board, semihard	6 - 8	610- 820
woodwool slabs	3 - 5	300- 510

Table A7. Weight densities and mass densities of soils[5]

soil	weight density kN/m³	mass density kg/m³
natural deposits		
dry sand and gravel	15-18	1530-1840
moist sand and gravel	15-20	1530-2040
water-saturated sand and gravel	19-22	1940-2240
water-saturated moraine clay	19-23	1940-2350
water-saturated plastic clay	16-20	1630-2040
water-saturated silt	10-16	1020-1630
water-saturated peat	10-12	1020-1220

5 Since the weight density of soils may vary within a fairly wide range, it should in principle be determined by measuring and weighing undisturbed samples.

Table A8. Weight densities, mass densities and characteristic angles of internal friction, φ_k, of inorganic materials

material	weight density kN/m³	mass density kg/m³	φ_k°
cement			
in silo	12-16	1220-1630	20
in bags	16	1630	
limestone and lime			
quarried stone	18	1840	45
hydrated lime	6	610	25
hydraulic lime	7	710	25
coke ash, slag	6- 9	610- 900	25
ground limestone			
(additive to livestock feed)	10-14	1020-1430	
salts			
in bulk (common salt)	12	1250	40
rock salt	22	2240	45
dicalcium phosphate	8- 9	820- 920	40
sand, dry	16	1630	30
shingle and pea shingle, dry	16-18	1630-1840	30
broken stone			
crushed brick, dry	12	1220	35
natural stone, dry	16-18	1630-1840	35

Table A9. Weight densities, mass densities and characteristic angles of internal friction, φ_k, of solid fuels[6 7]

fuel	weight density kN/m³	mass density kg/m³	φ_k°
brown coal, loosely deposited	6-8	650-820	35
brown coal briquettes, stacked	13	1330	
fire wood, stacked	5	550	
coke and cinders, loosely deposited	3-5	350-550	45
coal, loosely deposited	7-8	710-820	35
charcoal, loosely deposited	2-4	200-410	
peat, loosely deposited	3-6	300-610	
peat, compressed	6-9	610-920	

6 The weight densities are given as gross values, i.e. inclusive of any packing mentioned per m³ of the article in stored condition.

7 The weight densities given refer to dry fuels.

Table A10. Weight densities and mass densities of liquids

liquid	weight density kN/m³	mass density kg/m³
alcohol	7.8	800
petrol	7.4	750
benzene	8.8	900
liquid gas		
propane	5.0	510
butane	5.7	580
glycerol	12.3	1250
coal tar	10.8-12.7	1100-1300
milk	10.1	1030
oils	7.8-10.8	800-1100
fuel oil	7.8- 9.8	800-1000
diesel oil	8.3	850
linseed oil	9.2	940
creosote oil	10.8	1100
lubricating oil	8.8	900
white spirit	8.5	870
paraffin oil	8.3	850
hydrochloric acid (40 per cent)	11.8	1200
sulphuric acid (87 per cent)	17.7	1800
water	9.8	1000
wine	9.8	1000
ether	7.4	750
beer	10.3	1050

Table A11. Weight densities, mass densities and characteristic angles of internal friction, φ_k, of organic substances[8]

substance	weight density kN/m³	mass density kg/m³	φ_k°
cotton and wool, in bales and stacked	13	1330	
books, stacked	8	820	
books, on shelves and in filing cabinets	6	610	
fruit, in crates	3	350	
fruit, in bulk	4	450	35
rubber	10-12	1020-1220	
hides and skins, stacked	9	920	
flax, in bales and stacked	3	300	
groundnuts	4	410	
coffee	6	610	
meat	8	820	
malt	5	550	25
margarine, packed	7	710	
flour and oats	6- 8	610- 820	25
paper, stacked (see also books)	11	1120	
butter, packed	8	820	
sugar, in bulk	8	820	35
tobacco, in bales	3	350	

8 The weight densities are given as gross values, i.e. inclusive of any packing mentioned per m³ of the article in stored condition.

Table A12. Weight densities, mass densities, and characteristic angles of internal friction, φ_k, of agricultural crops

material	weight density kN/m³	mass density kg/m³	φ_k°
hay crops			
hay, hard compressed	2.0	200	
hay, medium compressed			
stacked to a height of 8 m	1.5	150	
stacked to a height of 2 m	1.0	100	
root crops			
potatoes			
in bulk or in bags	7.0	710	
fodder roots, cut	7.0	710	30
green crops, silage			
direct-cut silage	10.0	1020	
pre-wilted silage crops	7.5	760	
green fodder, loosely stacked	4.0	410	
silage, sugar beet waste	10.0	1020	

Table A13. Weight densities, mass densities, and characteristic angles of internal friction, φ_k, of seed and grain [10]

material	weight density[9] kN/m³ 3 m	12 m	mass density[9] kg/m³ 3 m	12 m	φ_k°
light seeds	3.8	4.5	390	460	20-30
cocks foot			290	340	
rye grass, meadow fescue			350	420	
rough stalked meadow grass			370	410	
mangold			260	300	
sugar beet			390	460	
heavy seeds	8.3	8.4	850	860	20-30
leguminous fruit			850	860	
clover, cleaned and sorted			830	840	
lupine			810	820	
lucerne, cleaned and sorted			820	830	
rape			700	730	
timothy			610	650	
grain	7.9	8.3	810	850	20-30
barley, Danish			730	800	
oats, Danish			640	680	
wheat, fodder			810	850	
maize, unsorted			740	800	
rye, Danish			790	840	

9 Minimum and maximum weight and mass densities are given at depths of 3 m and 12 m, respectively, below the surface of the stored material.

10 The weight densities given are inclusive of the moisture content which the materials concerned generally have in stored condition:

Grain: apx. 14-18 per cent by weight
Grass seeds: apx. 14-16 per cent by weight
Other seeds: apx. 11-17 per cent by weight

Table A14. Weight densities, mass densities, and characteristic angles of internal friction, φ_k, of concentrates [11]

material	weight density [9] kN/m³ 3 m	12 m	mass density [9] kg/m³ 3 m	12 m	φ_k°
cottonseed cake, unhulled	5.3	5.8	540	590	35
cottonseed flakes	6.5	6.7	660	680	
fish meal	6.5	6.8	660	690	
meal	3.9		400		
wheat bran	3.4	5.2	350	530	
linseed flakes	6.6	6.9	670	700	
linseed, rolled	5.3	6.3	540	640	
groundnut flakes	7.3	7.5	740	760	
coconut flakes	6.3	6.8	640	690	
meat bone meal	7.9	8.3	810	850	
lucerne meal	3.8	4.1	390	420	
oil cake	9.8		1000		
oil meal	4.9		500		
rape meal	6.0	6.2	610	630	
soybeans, rolled	5.1	5.7	520	580	
soybean meal, Danish	5.8	6.1	590	620	
sunflower flakes	7.4	7.6	750	770	
sunflower meal	5.8	6.0	590	610	

9 Minimum and maximum weight and mass densities are given at depths of 3 m and 12 m, respectively, below the surface of the stored material.

11 The weight densities given are inclusive of the moisture content which the materials concerned generally have in stored condition:

Flakes:	apx. 5 - 11 per cent by weight
Meat bone meal, fish meal:	apx. 5 - 6 per cent by weight
Others:	apx. 9 - 17 per cent by weight

Table A15. Weight densities, mass densities, and characteristic angles of internal friction, φ_k, of manure and fertilizers

material	weight density kN/m³	mass density kg/m³	φ_k°
manure			
urine, liquid manure, sludge	10.8	1100	
compost	11.8	1200	
farmyard manure, solid			
to a height of 1.5 m	5.9	600	
to a height of 3.0 m	9.8	1000	
fertilizers			25-35
ammonium nitrate sulphate	7.8- 9.0	800- 920	
Chile saltpetre	11.3-12.3	1150-1250	
agricultural lime	12.3	1250	
potassium fertilizers	8.6-11.8	880-1200	
calcium ammonium nitrate	7.8- 9.8	800-1000	
calcium cyanamide	8.4- 8.9	860- 910	
calcium nitrate	11.5-12.4	1170-1260	
Kola apatite concentrate	12.9-18.2	1320-1860	
NKP fertilizers	8.1-11.9	830-1210	
PK fertilizers	10.6-13.5	1080-1380	
phosphate rock	12.4-16.7	1260-1700	
superphosphate, granular	10.0-10.8	1020-1100	
superphosphate, pulverized	9.8	1000	
ammonium sulphate	9.8-10.8	1000-1100	
potassium sulphate	12.3-14.5	1250-1480	
Thomas phosphate	17.5-19.8	1780-2020	
Thomas meal	19.6-21.6	2000-2200	
urea	7.1- 7.4	720- 750	

CIB-W18/17-102-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PARTIAL COEFFICIENTS LIMIT STATES DESIGN CODES FOR STRUCTURAL TIMBERWORK

by

I Smith

Timber Research and Development Association
United Kongdom

RAPPERSWIL

SWITZERLAND

MAY 1984

INTRODUCTION

The United Kingdom in common with countries such as Canada and the USA is moving towards production of limit states design codes with a partial coefficients methodology for primary structural materials such as concrete, steel, masonry and timber. Efforts are being made within these countries to harmonise national loading codes and the individual materials codes. Hopefully a climate will be created wherein designers are encouraged through the exercise of expertise to produce more relevant and if appropriate more uniform levels of reliability between design than are attained with present working stress solutions. When appropriate designers should be permitted to gain economic advantage through use of a more rational probability based design procedure.

Codes for materials such as concrete have been introduced in the United Kingdom by a process of calibration with existing designs. This has resulted in load factors which are not universally applicable across both light and heavy weight materials and are to some extent illogical. It is felt that there is no great pressure to produce a UK partial coefficients timber code immediately and thought should be given to the matter before carrying out the type of procedure applied to other materials.

This note was originally produced for the British Standard Institution Code Committee CSB32, which oversees the production of UK timber codes, and gives a TRADA view of the current situation with respect to methods for calibrating partial coefficients timber codes.

1. What is limit states design?

A limit states design is one wherein certain limit states may not be exceeded. The limit states may relate to the load-carrying capacity of the structure (ultimate limit states) or to its function in normal use (serviceability limit states). Limit states design is neither inherently probabilistic nor inherently associated with a particular code formatting. The BS CP112 : Part 2: 1971 and its intended replacement BS5268 : Part 2 : 1984 are working stress codes with a rudimentary limit states design procedure, i.e. separate checks are made for strength and deformation. Reference is also made to fire, durability and corrosion.

2. Ultimate limit states

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

Ultimate limit states may for example correspond to: (1)

- Loss of static equilibrium of the structure, or part of the structure, considered as a rigid body (overturning),

- rupture of critical sections of the structure due to exceeding the material strength (in some cases dependent on the loading history),

- loss of stability (due to, among other things, buckling),

- unlimited slip of the whole structure or mutually between parts of it.

---

(1) CIB. CIB structural timber design code. International Council for Building Research Studies and Documentation Publication 66. Sixth edition, January 1983.

3. Serviceability limit states

Serviceability limit states are related to criteria governing normal use.

Serviceability limit states may for example correspond to: (1)

- deformations which affect the efficient use of a structure or the appearance of structural or non-structural elements,
- excessive vibrations producing discomfort or affecting non-structural elements or equipment (especially if resonance occurs)
- local damage which reduces the durability of a structure or affects the efficiency or appearance of structural or non-structural elements,
- local buckling of thin plates (for example in webs or flanges) without rupture,
- excessive impressions due to stresses perpendicular to the grain and not affecting the ultimate strength

4. What is probabilistic design?

A probabilistic design is one wherein a component or structure is sized so as to attain in the long run a desire of probability of load exceeding resistance at any time during the design life. This long run probability is often referred to as frequentist probability. The interrelationship exists:

$$R = 1 - P_f \qquad (1)$$

where: P_f = frequentist probability of load exceeding resistance

R = frequentist reliability

Load and resistance are here used in a general sense and could for example represent deformations within a serviceability limit states calculation.

Within any design methodology it is necessary to decide whether the objective is to design for a target level of individual element reliability or a target level of system reliability. The limit states against which probabilistic design should be made, target levels of reliability and the choice between individual elements or system reliability all depend upon factors such as the consequences of failure, cost of increased security against failure, ability of the system to redistribute load in the event of failure of a single element and the interaction of these factors.

Explicitly probabilistic design inherently includes statistical modelling of at least loads and resistances.

Notionally probabilistic design makes qualitative (subjective) allowance for the probabilistic nature of loads and resistances. Qualitative allowance must however be translated into quantitative calculations. This is achieved through agreement within code committees upon relativities indexed to a base proven satisfactory by experience.

5. What is rational design?

Traditionally timber components or structures are designed on an individual element basis ignoring interactions and complex support conditions. There is now a general acceptance for some forms of timber structures that design should follow more rational procedures which take account of redundancies and partial fixities between component and which design to a target level of reliability, or some related measure such as a safety index (see item 10.2).

'Explicitly rational design' therefore involves use of analytical models which include elasto-plasticity, viscosity, relative slip between adjacent semi rigidly connected elements, etc. in conjunction with statistical modelling of stiffnesses, loads and resistances. Ideally an explicitly rational analysis would be undertaken for every structure. More realisitically, approximate rationality can be achieved by assuming the influences of factors such as interactions, complex supports, statistical nature of loads, statistical nature of resistances, time in service on mechanical properties, volume etc. can be treated as independent.

Under an approximately rational procedure the design equation for an element by element design is:

$$\lambda F_k \leqslant \phi f_k K_i K_t K_e K_v K_d M \qquad (2)$$

where: F_k = characteristic design force,

λ = coefficient adjusting for statistical variation in loads causing design force,

f_k = characteristic design resistance for reference;time in service, interaction condition, environment, volume, direction of loading, etc.,

ϕ = coefficient adjusting for statistical variation in element resistance

K_i = modification factor for interaction conditions,

K_t = modification factor for time in service,

K_e = modification factor for environment,

K_v = modification factor for volume,

K_d = modification factor for direction of loading,

M = modification factor for design method (corrects for neglect of factors such as partial fixities and complex support conditions).

In general the equality in equation (2) corresponds notionally to exact attainment of the desired probability of failure within the target design life. The equality in equation (2) corresponds exactly to attainment of the desired probability of failure within the target design life only at the calibration point(s) used during the assignment of λ, ϕ, K_i, K_t, K_e, K_v, K_d, etc.

Within the context of rational design equations λ and ϕ can be calibrated on either an explicitly probabilistic basis or a notionally probabilistic basis, see item 4.

6. What is a partial coefficients limit states design methodology?
A partial coefficients limit states design methodology (also sometimes referred to in North America as load and resistance factor design) is a systematic procedure wherein checks are made against the possibility of attaining any of a selected range of ultimate and/or serviceability limit states. The objective is for the design force not to exceed the design resistance for the critical limit state employing within each limit state check the approximately rational procedure previously described, see item 5, and embodied in equation (2). Equation (2) is the basic form of the partial coefficients limit states design equation when applied within the framework of a systematic check for each limit state.

There are many variations and notations employed within generalised approximative expansions of equation (2). The form and notation used in this note are:

$$\sum_{i=1}^{n} \gamma_{f,i}\, I_{f,i}\, F_{k,i} + \sum_{j=1}^{m} \gamma_{f,j}\, I_{f,j}\, \psi_{o,j}\, F_{k,j} = \frac{K\, I_m\, M\, f_s(0.05)}{\gamma_M} \qquad (3)$$

where: γ_f = load factor (partial coefficient),
I_f, I_m = importance factors,
F_k = characteristic design force (subscripts i and j denote dead, snow, wind or other loads),
ψ_o = load combination factor,
K = composite modification factor,
= f (K_e, K_t, K_i, ...) $K \simeq K_e \times K_t \times K_i \ldots$,

K_e = modification factor for environment,

K_t = modification factor for duration of loading,

K_i = modification factor for interaction of elements

M = design method factor,

f (0.05) = short term characteristic strength as determined in a laboratory test and converted when appropriate to reference; specimen volume, environmental conditions, test method and test procedure,
= 5% exclusion value at ξ level of confidence*,
γ_m = materials factor (partial coefficient)

The two essential ingredients of a properly formulated partial coefficients limit states design methodology are:

(i) Provision for checks against all appropriate limit states relevant to a particular form of construction, e.g. a check should be included on vibrational performance of lightweight floors in addition to traditional checks on static strength and deformation performance.

(ii) Rational and reliable basis for calibration of all coefficients and factors in equation (3).

* Other levels of exclusion could be employed without violating the validity of the design equation, but associated partial coefficients would be numerically different.

7. Reason for choice of a partial coefficients methodology in preference to a working stress methodology for a limit states design code.

That design engineers are able to exercise a degree of discretion over the level of reliability for a given structure or structural element is desirable and by implication rational design incorporating probabilistic concepts is desirable. (Acceptance of this does not implicitly imply acceptance of any particular level of sophistication at which the objective will be persued.)

At the outset it must be acknowledged that exactly the same end product could be achieved under working stress and partial coefficients methodologies within the context of a limit states design procedure. The advantage in using a partial coefficients methodology is that it concisely formalises the design procedure when the engineer is permitted to make systematic decisions with respect to levels of reliability required, as a function of the target design life and the consequences and nature of failure.

Designers should be encouraged through the exercise of expertise to produce more relevant and if appropriate more uniform levels of reliability between designs and when appropriate gain economic advantage through use of a more rational probability based design procedure. Without the returns indicated, especially without a net economic advantage, it is to be expected that design engineers will be extremely sceptical about the potential superiority of limit states codes with a partial coefficient methodology relative to tradition working stress codes such as CP112 · Part 2. The adverse reaction by designers to BS CP110 bears witness to the truth of this observation.

It is desirable that we have a similar probability of failure across a wide range of structures and that we obtain a sensible balance between performance and economy. These aims are only achievable if a partial coefficients code calibration does not result only in a manipulated reformatting of existing working stress codes without discriminative levels of reliability.

In summary: Code drafting bodies should remember at all times that the purpose of the exercise is to produce understandable approximations to rational design. This is generally agreed by those who keep the objective in mind to be best achieved through a limit states design procedure with a partial coefficients methodology.

8. Choice of indices for coefficients and factors in a partial coefficients design equation

8.1 Modification factor for duration of loading, K_t

Within BS CP112 : Part 2 and other similar structural timber design codes modification factors for duration of loading are assigned to various loading classifications. (Loading classifications are notional loading histories adopted as the basis from which are calculated modification factors for the duration of load effect.) In CP112 : Part 2 : 1971 the following classifications are adopted:

1. Long term (e.g. dead + permanent imposed)
2. Medium term (eg. dead + snow, dead + temporary loads)
3. Short term (e.g. dead + imposed + wind, dead + imposed + snow + wind)

Short term loading in CP112 terminology relates to a situation where the design wind load is sustained for a total of 15 hours over a design life of 50 years, i.e. the definition is not synonymous with short term test durations typically in the order of 5 to 20 minutes. Medium term loading in CP112 terminology relates to a situation where the design snow loading is sustained for a total of 30 days over a design life of 50 years. Long term loading in CP112 terminology is dead load sustained in perpetuity. Numerically CP112 duration of loading modification factors are the proportions of long term duration strength which when continuously sustained cause failure in 15 hours, 30 days and at infinity. Basic strength values quoted in CP112 are indexed to long term loading and:

$K_{t\,CP112}$ (short term) = 1.50

$K_{t\,CP112}$ (medium term) = 1.25

$K_{t\,CP112}$ (long term) = 1.00

It can easily be demonstrated by use of damage accumulation models (2) and (3) that in situation where transient loadings do not cause resultant forces much greater than those resulting from dead and permanent imposed loadings and where transient loadings are of relatively short duration time to failure is dominated by the dead and permanent imposed loadings. This immediately raises the question of the general validity of the CP112 modification factors for short term and medium term loading classifications and why these at times apparently excessively liberal factors do not produce a significant incidence of failure. The reason probably lies in various factors including:

(i) Conservative estimates of design loads, especially imposed floor loads, snow loads and wind loads.

(ii) Codified working stress resistances are often estimated on the basis of considerations other than catastrophic collapse, e.g. resistances for mechanical joints and bearing strengths perpendicular to grain.

(iii) Lack of account of beneficial effects from partial fixities, 'non-structural' sheathings, composite actions, etc.

(iv) Conservative test methods and procedures, e.g. selective positioning of test specimens to propagate failure at the weakest cross-section.

---

(2) GERHARDS, C.C. Time-related effects of load on strength of wood. Proceedings of Conference on Environmental Degradation of Engineering Materials. Virginia Polytechnic Institute and State University. October 1977.

(3) FOSCHI, R.O. Load duration testing and damage accumulation modelling in timber joints. Technische Hogeschool Delft, Stevin Laboratory, Report 4-79-7. Delft, THD. 1979.

(v) Conservative interpretation and adjustment of test results during estimation of working stress resistances, e.g. conservative adjustment of results from short term test resistance to resistance for load sustained in perpetuity and choice of a larger reference volume than occurs in the design without permitting the designer to increase design resistance for elements with a volume less than the reference volume.

If we introduce new design procedures taking more rational account of factors such as interactions and complex support conditions it will be necessary to also introduce a more rational system for assigning K_t factors. A more rational basis for K_t is believed to be target design life in lieu of loading class, with say 60 years as the norm. Under the proposed basis:

$$K_t \text{ (60 years)} = 1.0$$

It can be shown by using damage accumulation models that expected K_t factors for other practical target design lives (in the range 10 years to 200 years) would be in the region of unity$\pm$10% for most types of solid wood elements and mechanical joints, (higher deviations from unity could well be observed for reconstituted wood products such as particleboards).

Preliminary consideration of equation (3) would seem to suggest that K_t should be indexed to 'short term test duration'. This, however, is rejected for the practical reason that for all design lives likely to be of interest a design engineer would have to apply a reduction factor to the resistance side of the design equation. Provided that a suitable norm for target design life could be found the design engineer would not need to calculate K_t unless he deviated from that norm, i.e. K_t would be unity in most instances. Under these circumstances it would presumably be acceptable for either K_t to be conservatively estimated for other design lives or to follow more complex selection processes than would be accepted across the board.

8.2 Short term characteristic strength, $f_s(0.05)$

As indicated within the definition of $f_s(0.05)$ given under item 6 it is necessary to index characteristic properties so that they are associated with standardised reference; specimen volumes (or dimensions), environmental conditions (e.g. climate classes 1; temperature of 20 $\pm$ 2degC and a relative humidity of 65 $\pm$ 5%), test methods and procedures, etc. Also it is highly desirable that a standardised statistical technique(s) is used to estimate $f_s(0.05)$ from test data. Use of an inappropriate statistical technique(s) could significantly effect the design equation relative to its basis of calibration. With the exception of K_t all modification factors in equation (3) will be numerically equal to unity at the reference points used in the definition of $f_s(0.05)$.

In the case of all but the commonest; timber-species-grade combinations, sheet materials and joints it is standard practice to measure in the laboratory only short term characteristic properties, i.e. $f_s(0.05)$ and associated stiffness properties measured in reference conditions. Using information of a more extensive nature gathered from tests on the more common materials extrapolations are made to predict the behaviour of less common materials in non-reference conditions. It would be a relatively simple matter for new materials to be rapidly introduced into the market place if reputable relevant organisations had merely to perform tests and estimate a suitable value of $f_s(0.05)$ and the associated stiffness properties.

8.3 Materials factor, γ_m

Under a strict definition γ_m is a partial coefficient which is a function of; the statistical variation in resistance, the target probability of failure, the target design life and coefficients and factors on the force side of the design equation. Within this context statistical variation in resistance is the net variation after due allowance has been made for random variability in cross-sectional area, cross-section distortion, modulus of rupture, etc, i.e. variability in section/element resistance, not variability in critical stress.

Unfortunately use of a γ_m conforming with the above definition is not consistent with adoption of a design life other than 'short term test duration' as the index for K_t. It is suggested that, within a partial coefficients limit states structural timber design code, a modified definition for γ_m be adopted so that γ_m includes in its derivation an adjustment factor balancing the transformation of the index for K_t from 'short-term test duration' to a 'normal design life'. Thus;

$$\gamma_m(\text{modified}) = \gamma_m(\text{pure}) \times K_t \ (\text{short term test duration}) \qquad (4)$$

with the value of K_t used indexed to 'normal design life'. The modified form of γ_m given in equation (4) is the inverse of the resistance or performance factor $\varnothing$ which is proposed within the draft revision to Canadian Structural Timber Design Code CSA 086.

Any given numerical value of γ_m (in modified or unmodified forms) is when used in conjunction with pre-specified values of γ_f, I_f, ψ_0 K, I_m and M only strictly appropriate to a particular type of structure, a particular location within that structure, a particular geographic location and the particular type of element being considered. However, some rationalisation will be necessary before practical design calculations can be sensibly performed on a day to day basis. Balancing the considerations influencing rationalisation of γ_m values will prove to be the major problem once a code formatting stage is reached.

9. Type of analysis used in conjunction with partial coefficients limit states design codes for structural timberwork.

The following types of analysis are thought appropriate:

Type of analysis	Static	Equivalent static	Dynamic	Linear elastic	Elasto-plastic -viscous
Ultimate l.s. calculations					
D	X			X	X
D + S	X			X	X
D + W		X	X	X	
D + S + W		X	X	X	
D + E		X(1)	X(2)	X(1)	X(2)
Serviceability l.s. calculations					
D	X			X	X
D + S	X			X	X
D + W		X		X	
D + S + W		X		X	
I (Domestic floors)			X	X	

Where:
D = dead plus permanent imposed loading,
S = snow loading,
W = wind loading,
E = earthquake loading,
I (Domestic floors) = imposed loads causing transient vibrations in domestic and similar lightweight floors
(1) or (2) indicates that the type of analysis is assciated with another type of analysis marked with the same character.

10. Methods for calibrating partial coefficients limit states design equations.

Below are briefly described three methods of calibration. These should not however be considered as discrete as code drafting agencies will of necessity require to utilise facets of each.

10.1 Soft conversion

Under a so-called 'soft conversion' partial coefficients designs are balanced against accepted working stress designs to find combinations of γ_m, K factors, I_f, γ_f, γ_o and I_m which produce comparable designs by both methods. This implies an acceptance that the working stress design used in the calibration will produce the desired probability of failure within the design life of a structure or elements.

For a working stress design the design equation is:

$$\sum_{k=1}^{\ell} F_{k,k} = \frac{K\, f_s(0.05)}{\Omega} \qquad (5)$$

where: Ω = composite factor adjusting from test conditions and including a factory of safety.

From equations (3) and (5) for a soft conversion:

$$\gamma_m = \frac{\Omega\, I_m\, M \sum_{k=1}^{\ell} F_{k,k}}{\sum_{i=1}^{n} \gamma_{f,i}\, I_{f,i}\, F_{k,i} + \sum_{j=1}^{m} \gamma_{f,j}\, I_{f,j}\, \gamma_{o,j}\, F_{k,j}} \qquad (6)$$

Via various manipulations it is possible to produce combinations of coefficients and factor in equation (3) which will be acceptable to code users, e.g. CSA 086 committee has proposed manipulation of K_t factors to produce a common value of γ_m for all loading classifications. Such manipulations are merely a question of code formatting and are not discussed further.

In the derivation of various design resistances in Approved Draft BS5268 : Part 2 the Ω value used to transform f_s (0.05) to working sress resistance varied between timber species and between strength properties as a function of the type of raw data. This may appear at first to be a significant obstacle to attainment of a coherent set of γ_m and K_t values by the procedure outlined above. The need during drafting of BS5268 : Part 2 for variable Ω lies in:

i) Use of inconsistent definitions for f_s (0.05) between timbers or between strength properties. Numerical values assigned to f_s (0.05) are not always true ultimate strengths and may relate to deformation limits.

ii) Differences in physical processes governing strength as a function of time, environment, nominal stress level, etc.

The solution lies in:

a) Adoption of a constant Ω common to all timber species and all strength properties. (This implies neglect of the influences of differences in physical processes governing strength. Although questionable this simplification seems consistent with the state of current knowledge.)

b) Use of only true values of f_s (0.05) in ultimate limit states calculations. (This is the requirement which prompted the revised assessment of strength data on joints undertaken during recent redrafting of CSA 086).

10.2 Second moment method (4) (Semi-probabilistic, may also be semi-rational).

Under this method γ_f , ψ_o , I_f , γ_m and I_m in equation (3) can be determined explicity for non-aging materials.

Through an approximative uncoupling of force and resistance the design equation coefficients γ_f, ψ_o and I_f are not functions of the material properties of the structure being considered but are functions of the target probability of failure within the design life and variability of loads. Similarly γ_m and I_m are not functions of the loading but are functions of the target probability of failure within the design life and variability of the resistance property being used. (Modification factors such as K_t are evaluated independently for aging materials such as timber as per current practice in writing working stress codes.)

Briefly the theory is:

$$\beta = \frac{\overline{f(R)} - \overline{f(u)}}{(\sigma^2_{f(R)} + \sigma^2_{f(u)})^{1/2}} \qquad \text{(7)}$$

where: β = safety index
$f(R)$ = material resistance
$f(U)$ = applied force
$\overline{f(R)}$ = mean value $f(R)$
$\overline{f(U)}$ = mean value $f(u)$
$\sigma_{f(R)}$ = standard deviation of $f(R)$
$\sigma_{f(u)}$ = standard deviation of $f(u)$

Taking both R and U to be log-normally distributed and making the approximations;

$$\overline{\ln(R)} - \overline{\ln(u)} \simeq \ln(\bar{R}/\bar{u}) \text{ , and}$$

$$(\sigma^2_{f(R)} + \sigma^2_{f(u)})^{1/2} \simeq \alpha\,(V_R + V_u) \qquad \text{(8)}$$

and recasting within the safety checking equation;

$$\overline{f(u)}\,\lambda = \phi\,\overline{f(R)} \qquad \text{(9)}$$

yields:

$$\phi = \exp(-\alpha\beta V_R) \text{ , and}$$

$$\lambda = \exp(\alpha\beta V_u) \qquad \text{(10)}$$

where: V_R and V_u are coefficient of variation for R and U respectively.

Equations (10) can be used in conjunction with a series of co-ordinated numerical exercises to calibrate γ_f , ψ_o , I_f , γ_m and I_m , i.e. λ decomposes into γ_f , ψ_o and I_f values and ϕ decomposes into γ_m and I_m values).

(4) MacGREGOR. J.G. Safety and limit states design for reinforced concrete. 1975-1976 National Lecture Tour sponsored by Structural Division Canadian Society for Civil Engineering in co-operation with Portland Cement Association Canadian Division.

10.3 Numerical reliability studies (Rational and probabilistic)

Numerical reliability studies simulate the reponse of structures or components to in service loadings over their entire design life and include an estimation of the random variation in the response between structures. Within such studies full account is taken of interactions between statistical processes and realisitc idealisations of structural assemblies are used.

For aging materials such as timber the assumption that derivation of γ_m and I_m can be uncoupled from derivation of γ_f, ψ_o, I_f and K factors is not proven. It is therefore necessary to carry out selective numerical reliability studies for key structural application such as floor joists, joists in flat roofs and trussed rafter members and joints to establish the validity/applicability of semi probabilistic methods such as the second moment method, section 10.2 Such studies are currently being performed on: joists in flat roofs by Dr. R. O. Foschi at FORINTEK in Vancouver, and joints in pitched roof trusses at TRADA.

When calibrating the design equation using results from numerical reliability studies it is usual to employ preselected coefficients and factors on the force side of the design equation with calibration restricted to the coefficients and factors on the resistance side of the design equation. Also for calibration purposes it is necessary to select a reference condition for each of the K factors, e.g. $K_t = 1.0$ at a design life of 60 years.

11. Short to medium term options

The best short to medium term option appears to be a hybrid calibration process for a partial coefficients limit states code with calibration being a mixture of soft conversion and probabilistic analysis. The hybrid envisaged contains the following:

a) Assignment of ψ_o, γ_f and I_f factors through application of the second moment method described by MacGregor (4). In general it would be desirable to specify ψ_o and γ_f values appropriate to all structural materials in national loading codes.+ I_f factors would be best specified within codes for individual materials but derived from criteria common to all materials. F_k values would best be specified, or the basis for their estimation would best be specified, in national loading codes applicable to design with any structural material.

(The force side of the design equation would be calibrated entirely on the basis of explicit semi-probablistic analysis. Owing to the general tendency for forces resulting from a combination of loads to be less variable than forces resulting from single loads the uncertainty concerning variability of various loadings would not be as troublesome as might be supposed.)

+ This is the approach already adopted in the USA through ANSI A58.1-1982 (5). The second moment method was the basis of the A58,1 calibration process and it is therefore semi-probabilistic. The background to the calibration process as is guidance on derivation of compatible $\phi = 1/\gamma_m$ values for the various structural materials which will be used in structures designed with A58.1 loading combinations are given in reference 6.

b) For components where the time to failure is principally governed by permanent loadings, e.g. roof trusses and most structural joints there is no need to resort to soft conversion techniques when calibrating γ_m, I_m and K_t factors. For such situations γ_m and I_m can be estimated using the second moment methods and K_t factors can be estimated using Wood's curve (7) or similar data.

c) For components such as joists in flat roofs and floor joists where transient imposed loadings are a significant influence upon time to failure K_t and γ_m can't be regarded as uncoupled. Soft conversion of current working stress codes is the best interim option in such cases, though some assistance is available from the work by Foschi on roofs, see Section 10.3.

(The resistance side of the design equation would be calibrated in some cases on the basis of explicit semi-probabilistic analysis and in others on the basis of a soft conversion interpreted in the light of results from limited numerical reliability studies. The choice of technique in idividual structural applications would depend upon the loading mix).

12. <u>Important points to be covered by committees dealing with overall concepts for design in any structural material</u>

(a) Classification of structures and assignment of suitable probabilities of failure and target design lives.

(b) Determination of F_k, ψ_o and γ_f values.

13. <u>Important points to be considered by committees dealing with design of structural timberwork</u>

(a) Interpretation of data currently being collected on duration of loading influeces on strength.

(b) Grouping of components on the basis of failure mode, ability of the assembly in which that component is located to redistribute load in the event of failure of the component, loading mix, etc.

(c) Appropriateness of evaluation of individual component or system performance for various structural problems.

(d) Assignment of γ_m, I_m and M factors on the resistance side of the design equation.

(e) Assignment of f_s (0.05) values.

(f) Assignment of I_f factors on the force side of the design equation.

---

(5) AMERICAN NATIONAL STANDARDS INSTITUTE. American national standard minimum design loads for buildings and other structures. ANSI A58.1-1982. ANSI.1982.

(6) ELLINGWOOD, B., GALAMBOS, T.V., MacGREGOR, J.G. and CORNELL, C.A. Development of a probability based load criterion for American national standard A58: Building code requirements for minimum design loads in buildings and other structures. National Bureau of Standards Special Publication 577. Washington. US Government Printing Office. 1980.

(7) WOOD, L.W. Relation of strength of wood to duration of load. US Forest Products Laboratory Report No.1916. UDSA. 1951.

NOTES AND PAPERS

OF THE RILEM-MEETING

RAPPERSWIL

SWITZERLAND

MAY 1984

June 1984

Minutes of RILEM committee 57-TSB at Rapperswil, Switzerland,
21st May 1984

Prepared by: R.F. Marsh

Present : E. Aasheim
L.G. Booth
D.H. Brown
H.J. Burgess
D. Cresswell
J. Ehlbeck
A. Epple
U.A. Girhammar
P. Glos
U. Korin
J. Kuipers (chairman)
R. Lackner
R.F. Marsh
U. Meierhofer
B. Norén
U. Saarelainen
I. Smith
C.K.A. Stieda
J.G. Sunley (part time)

1.0 U. Meierhofer welcomed the RILEM 57-TSB committee to Switzerland on behalf of EMPA and J. Kuipers thanked EMPA.

2.0 J. Kuipers reported changes to the funding of RILEM which necessitated a request to organisations and individuals to become members or pay administrative fee. He expected that a request for fees will be sent to committee members if they were not either representing a RILEM member organisation or were already individual RILEM members.

3.0 J. Ehlbeck asked for clarification as to the terms of reference of this committee and of the status of the committee 3-TT. J. Kuipers stated that 3-TT had been set up to process a number of subjects including test methods for nails and staples but that its work being virtually complete, committee 3-TT was effectively disbanded.
This committee reference is 57-TSB and is to handle:

testing of timber based sheet materials other than plywood
testing of structures.

It also acts as a forum for finalizing the work of 3-TT.

4.0 J. Kuipers reported that the document 3TT-1; C, Testing methods for joints with mechanical fasteners in load-bearing timber structures, annex C: "Staples", had been published in the CIB/W18 Proceedings of the Lillehammer meeting, May 1983. No written comments had been received. No verbal comments were received at this meeting. The document was agreed but was to be put to the CIB/W18 meeting for formal approval. On approval it will be sent to RILEM (Paris) for publication.

Act. Kuipers N.B. Also CIB/W18 agreed that the document should be published.

5.0 Testing methods for timber structures, 2nd draft, prepared by J. Torey.

J. Kuipers stated that the 1st draft had been discussed at Lillehammer and a 2nd draft had been circulated to permanent members of the committee. Written comments had been received from B. Norén and W. Nożyński.

5.1 J. Kuipers introduced the paper and suggested it should be considered on a clause by clause basis.

5.2 For the purpose of these minutes a copy of draft 2 is included and reference will be made to the clause numbers in that draft.

5.3 There was long and detailed discussion on all points of the 2nd draft. These minutes record either those points on which agreement was reached or highlights those clauses which require to be reconsidered together with comments as to the objects to be achieved in revising the clauses.

5.4 Section 0. Introduction.

5.4.1 Paragraphs 0.1 to 0.3 were agreed in that they form a background explanation to the application of the testing method but would not have to be included in the final ISO Test Method Standard. U. Korin and others pointed out that paragraph 0.2 (d) implied quality control and that the test methods proposed were too cumbersome for this. It was agreed that it was the decision of the supervising engineer to decide which elements of the test method were to be used for quality control.

5.4.2 There was general dissatisfaction with clause 0.4. The following two alternatives are proposed for consideration:

0.4 (a) In order for a structure to satisfy the proposed ISO code on the design of timber structures, the relevant national code, or the draft code on the Design of Timber Structures proceeded by CIB/W18, it shall be tested in accordance with the methods defined in this code to meet the requirements specified in the particular structural code.

0.4 (b) If testing of components or structures has been chosen as an accepted method to demonstrate that this component or structure was designed in conformation with a particular timber design code, this code shall specify which part(s) of the methods of the underlying standard must be used and which requirements fulfilled to be accepted.

5.5 1.0 Scope and Field of Application.

1.1 Agreed.

1.2 This was agreed but following the review of section 7.0 may require revision.

5.6 2.0 Symbols.

The symbols were agreed except that γ_t should read γ_{test}. It is necessary to confirm that these symbols are consistent with the ISO symbols. It was agreed that further information should be given to the factors which govern the selection of the value γ_{test}.
Thus in clause 2.0 should indicate

γ_{test} = factor for increading design load.

5.7 3.0 Definitions.

3.1 Dead load.

Add: ... acting on the structure being tested and excluding the selfweight of that structure.

3.2 Design load.

Accepted.

3.3 Imposed load.

Accepted.

3.4 New heading: Self load.

Force due to the static weight of the supporting structure being tested.

3.5 Wind load.

Accepted.

N.B. The chairman doubted if in a definition can be said:
(dead) load = force (due to ...), etc.

3.6 γ_{test} factor.

The γ_{test} factor is to take into account such factors as:

the variation in workmanship

the nature of the structure being tested

the desired or required confidence level in the structure being tested

the relationship of the in use to testing environment

the national safety codes

etc.

N.B. It was agreed that CIB/W18 be asked to produce a paper indicating

Act. Kuipers the factors which need to be considered in assigning a value to γ_{test}.

3.7 Supervising Engineer.

Agreed as original 3.4.

5.8 4.0 Sampling.

4.1 Number of tests.

After ... variability in manufacture, add: desired or required level of confidence;

after ... structures to be produced, add: the cost of testing each element of structure.

4.2 Manufacture and quality of test structure. To read:

The manufacture and assembly of the test structure shall comply with the design specification. The manufacturing methods used shall be, or should simulate as closely as possible, those which are used in production. The quality of the materials used shall be checked against the design specification and recorded in the test report.

5.9 5.0 Conditioning and Testing Climates.

Accepted, but add the following sentence:

Where it is not possible to define the in-use environment or reproduced that environment in the testing laboratory then allowance should be made in assigning a value to the γ_{test} factor.

5.10 6.0 Apparatus and General Testing Requirements.

6.1 Rewrite as follows:

The accuracy of the positioning of the applied loads ($F_{dead} + F_{imp}$) and the measurement of deflection and the applied loads, shall be within the tolerance of $\pm$ 3%.

6.2 Accepted.

6.3 Accepted

6.4 Omit the first sentence.
Otherwise agreed.

5.11 7.0 Loading and Recording Procedures.

7.1 Agreed except that at end of the first sentence change to ... deformations at supports.

Clauses 7.2 to 7.5 produced a long and intense discussion. B. Norén and others indicated that the test procedures as defined were both long and cumbersome and as a consequence expensive. The procedures did not recognise modern testing methods with automatic load application and recording techniques. He also doubted the basis of the definition of the periods of time for which the loads were to be applied and held in position.

J.G. Sunley and others stated that the methods described had been in use for many years and by their nature gave the Supervising Engineer an indication of the long term behaviour of the structure. It was particularly pointed out that the British Code required that in clause 7.3 the rate of increase of deflection must decrease during the 2nd period of sustained design load.

R.F. Marsh and J. Ehlbeck commented that the Supervising Engineer would be aware whether the structure under test was sensitive to 'bedding down' or slip and thus it might be reasonable to define minimum periods of time required to either apply or sustain the loads in order to meet the requirements of the test procedure.

It was agreed that this section should be rewritten in a manner to give the Supervising Engineer options in chosing the test method to be adopted.

In defining this option P. Glos indicated that consideration should be given to cyclic loading tests but pointed out that the loading tests must be defined in such a manner that whichever testing option is chosen by the Supervising Engineer the resulting structure must be safe.

It was agreed that B. Norén be invited to define alternative testing procedures to satisfy the criteria defined above. Particular attention should be paid to rate of loading and the periods of maintaining the loads for defined periods.

The intention is:

Act. Norén — to include a 3rd draft (by Norén) in the proceedings of this meeting;

to invite written comments prior to the next meeting of RILEM - CIB/W18;

to adopt a revised 3rd draft at the next meeting to be circulated as a RILEM paper.

5.12 8.0 Test Report.

This clause was agreed with the addition of the following points:

- details of pre-conditioning of the structure
- specification of the materials of construction
- load/deflection curves for each loading case.

5.13 Diagrammes.

As a consequence of the revision of clause 7.0 the figure 1 will have to be revised.

P. Glos suggested that it would be valuable to include a typical load-deflection curve.

6.0 Test Methods for Particle Boards.

J. Kuipers stated that he had just received copies of a suggest draft for the testing of particle board by I.D.G. Lee and an American paper on panel shear tests.

In addition he had also received the hand written draft of a paper

- Test Methods for Basic Properties of Wood-based Panels: Past experience; to-day's needs, by J. Dobbin McNatt.

The method of testing proposed by I.D.G. Lee involved the use of test samples to the same dimensions required for plywood samples.

Both J. Ehlbeck and B. Norén strongly criticized the basis on which this choice was based but were unable to provide justification for the use of ISO recommended test sample sizes.

D.H. Brown indicated that from U.S.A. experience large (i.e. as plywood test method) test samples were required because for some types of particle board the flake size was of a similar order as the ISO test sample size.

J. Kuipers also indicated that tests on particle boards in Holland had involved the adoption of test samples to the size defined in the plywood testing code.

J. Ehlbeck felt that the size of test samples should be related to the characteristics of the type of particle board being tested.

J. Kuipers reminded the meeting that the test method was intended to cover all types of timber-based sheet materials and that such tests for producing strength data as a basis for structural design should be carried out only once for a certain type of material.

L.G. Booth proposed that for the sake of progress that a test method for sheet materials be circulated based on a test sample size as that for plywood. This would solicit a response from interested parties to justify the use of smaller test samples.

With reservations B. Norén and J. Ehlbeck agreed to this.

Act. Kuipers J. Kuipers agreed to prepare a draft test method for Timber Based Sheet Materials based on the plywood test method, possibly together with suggestions or additions of clauses on creep tests. The draft is to be based on the ISO plywood testing method.
This draft is to be included in the proceedings at Rapperswil and written comments invited.

7.0 Determination of Panel Shear Strength and Panel Shear Modulus of Beech Plywood in Structural Sizes by J. Ehlbeck and F. Colling. Paper CIB/W18 - 17-4-1.

In this paper the authors have shown that the testing methods of the ISO draft code for testing plywood were difficult to achieve for thicker sections of structural plywood.

I. Smith and D.H. Brown gave verbal comments on their experience and would submit these in writing to J. Ehlbeck. They did express some doubts on the correlation of the ISO test method to the new method proposed by Ehlbeck and Colling.

L.G. Booth stated that the authors had undertaken valuable work on reviewing methods proposed in a RILEM/ISO draft code. RILEM should now submit to ISO an alternative method, based on this work, to be included in the ISO draft code.

It was agreed that this should be further discussed at the CIB/W18 meeting prior to submission to ISO.

8.0 Any Other Business.

J. Kuipers introduced some work that his laboratory had undertaken on the testing of glulam samples with the suggestion of producing a standard method.

He stated that these tests involved the basic parameters other than that of tension.

P. Glos stated that as the basic mode of failure in glulam members was a brittle tension fracture resulting from bending that he did not appreciate the value of J. Kuipers proposal.

J. Kuipers pointed out that as little was recorded on the basic properties of glulam, that if the body of knowledge was increased by a series of simple tests then this must be valuable.

R.F. Marsh pointed out that cube tests (compression) we used in concrete to indicate other properties of the material including tensile properties with proper calibration similar tests of glulam sections would give an indication of the other properties of glulam.

J. Ehlbeck asked if delamination tests for glulam should be considered by this committee.

Act. Kuipers J. Kuipers explained that glulam was not strictly within the terms of reference of this committee but that he would urge RILEM to set up a group to study this topic.

There being no further business J. Kuipers thanked everybody for attending and contributing to the meeting.

Note - Must numbered paragraph

TESTING METHODS FOR TIMBER STRUCTURES (Second Draft) J. Torey

0 INTRODUCTION

0.1 This draft testing standard gives recommendations for the load testing of timber structures where such testing is to be used in conjunction with, or as an alternative to, calculations to verify the structural adequacy of a design. The draft standard provides a method of testing to satisfy the design requirements of the CIB Structural Timber Design Code.

0.2 The testing of components or of complete structures may be necessary:

a where a component or structure is not amenable to calculations, or where calculation is deemed impracticable;

b where materials or design methods are used which are not adequately defined by the relevant structural code;

c where there is uncertainty as to whether the structure will perform adequately because of doubt or disagreement over compliance with the design rules or the quality of the materials used;

d where a routine check of a mass-produced structure or part of a structure is required by prior agreement between a client and a manufacturer.

0.3 Whenever possible more than one structure of the same design should be tested to permit the assessment of the likely variability in performance.

0.4 It is recognised that national requirements may differ but for the purposes of design to conform with the CIB Code all components and structures should be tested in accordance with clauses 7.2 and 7.4 or 7.5 and at least one structure of a particular design should be tested in accordance with clause 7.2.

CIB decision ie should only occur in CIB code

1.0 SCOPE AND FIELD OF APPLICATION

The methods of testing given in this draft standard are for timber components and structures. They are not appropriate to the testing of individual pieces of timber, joints or structural models.

1.1 Four loading procedures are given in this draft standard:

i pre-load

ii deflection test

iii proof test

iv strength test

1.2 It may not be necessary to carry out deflection, proof and strength tests, depending on the purpose of the test. But all deflection, proof and strength tests should be preceded by the pre-load loading procedure.

2.0 SYMBOLS

F_{dead}	dead load (N)
F_{design}	design load (N)
F_{imp}	imposed load (N)
F_{self}	self-weight of the structure (N)
F_{wind}	wind load (N)
γ_{test}	the factor by which the design load should be increased for the proof load test (national safety codes etc). (Should be given dependence - material, workmanship ? workmanship

3.0 DEFINITIONS

3.1 Dead load

Force due to the static weight of all permanent floors, roofs, partitions, cladding, finishes and other permanent construction. acting on the structure to be tested. (excludes self weight)

3.2 Design load

Force due to the most adverse combination of dead, imposed and wind loads (including self-weight).

3.3 Imposed load

Force due to internal occupancy or use, including distributed, concentrated, impact, inertia and snow loads, but excluding wind loads.

3.4 Supervising engineer

A suitably qualified engineer responsible for the conduct of the test.

3.5 Wind load

Force due to the effect of wind pressure or suction.

4 SAMPLING

4.1 Numbers of tests

The number of components or structures to be tested and the method of selection, will normally depend on the objective, taking account of the probable variability in manufacture, the required level of confidence, the number of similar components

desired level of confidence

or structures to be produced and the difficulty of defining the most severe loading condition.

4.2 Manufacture and quality of test structure

The manufacture and assembly of the test structure should comply with the design specification, and the manufacture methods used should be, or should simulate as closely as possible, those which are to be used in production. The quality of the materials used should be checked against the design specification and recorded in the test report.

5.0 CONDITIONING AND TESTING CLIMATES

All components and structures to be tested should be conditioned to and tested in environmental conditions as near as practicable to those that will be experienced by the component or structure in use.

6.0 APPARATUS AND GENERAL TESTING REQUIREMENTS

6.1 The accuracy of loading and of deflection and load measurement should be within ± 3 per cent. The position of load and accuracy of defl & load measurement should be within ± 3%

6.2 The method by which the loading is to be applied, and the positions at which the deflections are to be measured, can only be decided by reference to the particular component or structure being tested and to the particular loading conditions being investigated.

6.3 The test loading should be both applied and resisted in a manner which reasonably approximates actual service conditions or which induces in the test structure the maximum stresses and deflections that can be anticipated in service. Eccentricities, other than those necessary to simulate service conditions, should be avoided at points of loading and reaction, and care should be taken to ensure that no inadvertent restraints are present. Lateral restraint should also be consistent with the service conditions.

? other wording

6.4 ~~To establish the loading condition all reasonable combinations of dead, imposed and wind loads should be considered.~~ The loading condition is, for test purposes, referred to as the design load and it is this load that the test should simulate. Where a single worst loading condition cannot be readily identified it may be necessary to test more than one structure under different design loads.

(needs assessment of [illegible])

7 LOADING AND RECORDING PROCEDURES

The loading spectra for the pre-load, deflection, proof and strength tests are illustrated in Figure 1.

7.1 Load/deflection recording

When plotting load/deflection curves the self-weight of the structure shall be taken into account and allowance shall be made for ancillary loading equipment and deflections at the supports.
Deformations

Load/deflection readings shall be taken and preferably plotted during a test. Such plots serve as a check against mistakes and will show up irregularities in the behaviour of the structure to enable a particular weakness to be investigated as the test progresses.

7.2 Pre-load

A load equal to the dead load shall be applied, maintained for a period of minimum of 5 mins ~~30 minutes~~ and then released. Deflections of the structure shall be measured immediately before and immediately after the load has been released and again after a further minimum 5 mins (15 minutes). The last measurement taken is to serve as the datum for all subsequent deflection measurements in the deflection and strength tests. — only when subject to slip

The self-weight of the structure and the weight of any ancillary equipment shall be recorded.

Where camber is provided it shall be measured relative to the support points after the release of all loads other than self-weight and loading equipment and immediately before the start of the deflection test.

7.3 Deflection test Design Load

Immediately following the pre-load test and the establishment of the deflection datum points the dead load shall be applied again. This is to be maintained for 15 minutes and then additional load shall be applied, either at a continuous rate or in at least four equal increments with equal time between increments, until the design load is reached. The rate of loading should be such that the time taken to reach the design load from self weight is at least 30 minutes and preferably not more than 45 minutes. The design load shall be maintained for 24 hours; the load shall then be reduced to dead load only, held for 15 minutes, and then released.

Deflection readings shall be taken during the deflection test as follows:

a immediately* before the dead load is applied (this reading is coincident with that at the end of the pre-load test);

b immediately* on achieving dead load;

c 15 minutes after the application of the dead load;

d either continuously as the load is increased or at each increment of load;

e immediately* on achieving the design load;

f at sufficient intervals throughout the 24 hour period under the design load to enable a deflection/time curve to be plotted for each point at which deflection is recorded;

g. at the end of the 24 hour period under the design load;

h immediately* after reducing the load to dead load

i 15 minutes after the load has been reduced to the dead load;

j immediately* after release of the dead load;

k 15 minutes after the release of the dead load.

7.4 Proof load test

must record defl^n.

Within 1 hour of completing the preceding deflection test (or pre-load test if no deflection test is carried out) the design load should be applied again in the same manner and at the same rate as for the deflection test. The load shall then be increased to a value of γ_t times the design load and maintained for 15 minutes. The rate of loading for this additional load should be the same as that used between the dead and design loads (ie using the same increments of load and time).

7.5 Strength test

Immediately following the proof load test (including the 15 minutes at γ_{test} times the design load) the load shall be further increased at the same rate until failure occurs.

8 TEST REPORT

preconditioning of structure

The test report shall include:

a the conditions of testing, including the methods of loading and of measuring loads and deflections;

b the type(s) and position(s) of fracture(s)

c the moisture content of the timber

*For the purposes of this standard 'immediately' is equivalent to 'within 1 minute'

a) Specification of material

d the nature and size of defects in the materials which may have contributed to failure;

e the quality of material relative to that specified.

Photographs shall be used to illustrate important points of the report.

Load/deflection data for each loading process.

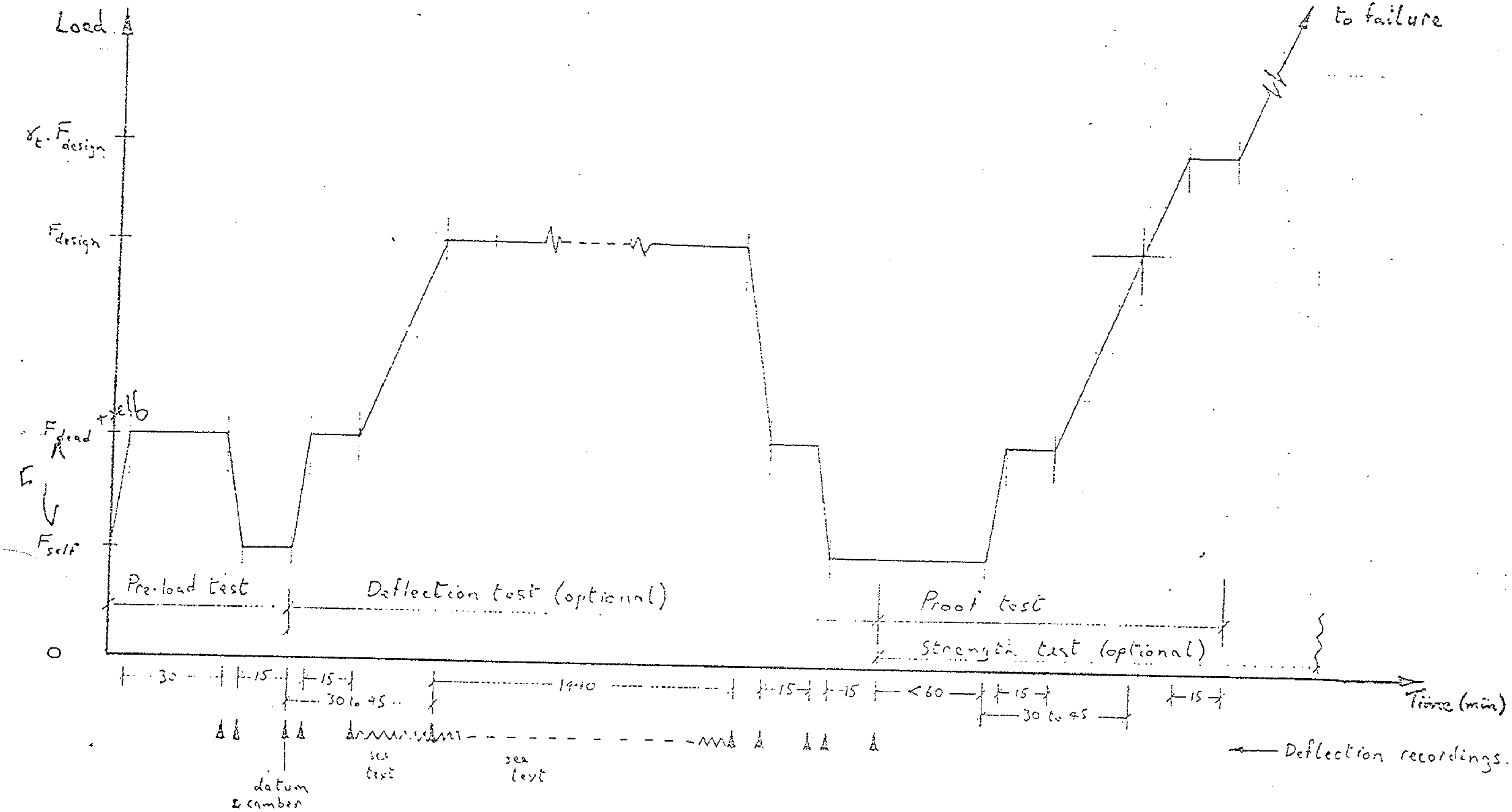


Fig. 1 : Loading Spectra

REPORT

AFRICAN, CARIBBEAN AND LATIN-AMERICAN SUB-GROUP

by

R S Beckett

University of Zimbabwe
Zimbabwe

RAPPERSWIL

SWITZERLAND

MAY 1984

TräteknikCentrum
INSTITUTET FÖR TRÄTEKNISK FORSKNING
Swedish Institute for Wood Technology Research

1984-08-16 Bengt Norén/YL

Written comments and proposals are very welcome to be send to the author and to Jan Kuipers-Delft, before next meeting

TESTING METHODS FOR TIMBER STRUCTURES (THIRD DRAFT)

INTRODUCTION

This standard concerns load testing of timber structures or structural components to verify adequacy of design. Thus the standard provides a method to satisfy design requirements of the CIB Structural Timber Design Code.

Verification by testing of components or structures may be applied for example where materials or design methods are not adequately defined by the structural code or where there is uncertainty as to wether the structure will perform in compliance with the quality of the materials or the design rules used.

Whenever possible more than one structure of the same design should be tested to permit the assessment of the likely variability in performance.

1. Recommended field of application

This standard is intended for testing of timber components and structures and generally not appropriate to the testing of individual pieces of timber, joints or structural models. Relevant parts of the standard may be applied for proof loading or for testing of structures in service.

2. Symbols

G_0 gravity load of the structure ("self-weight")

P applied load (G_0 not included)

P_D value of P corresponding to design load

γ the factor for P_D

3. Sampling

The quality of the materials and the manufacture and assembly of the tested structures should comply with the design specification.

The number of components or structures to be tested and the method of selection will depend on the probable variability in manufacture, the required level of confidence and the number of loading conditions to be applied.

4. General testing requirements

The accuracy of loading and of deflection and load measurement should be within ± 3 per cent.

The test loading should be distributed and resisted approximating actual service conditions. Irrelevant eccentricities at points of loading and reaction and inadvertent restraints should be avoided.

5. Loading procedure

5.1 Basic program (Alt. 1)

The standard defines a basic program for the application of the test load in figure 1 (full curve 1 to 6). It is based on a "design" value of the applied load, P_D, see 5.3.

The periods are

- 0	Only G_0 acting, i.e. $P = 0$	$\geq$ 0 min
0 - 1	Loading to $P = 0.5\ P_D$	$\geq$ 2 min
1 - 2	Load removed, $P = 0$	$\geq$ 2 min
2 - 3	Loading to $P = P_D$	$\geq$ 4 min
3 - 4	Constant load $P = P_D$	$\geq$20 min
3 - 6	Load increased to $P = P_B$ (failure)	($\geq$10 min)

Maximum rate of loading $dP/dt = \pm P_D/4$ (N/min).

Important: If considerable yield is developed in the structure before failure, the rate of loading must be reduced thus that the maximum rate of the deformation, as stipulated in the relevant testing standards for members and joints, is not exceeded.

5.2 Alternative programs

The standard offers alternative programs after the load has reached the value γP_D (point 5 in figure 1, dashed curves):

Alt. 2 Load is removed at $P = \gamma P_D$ and the testing interrupted.

Alt. 3 The load $P = \gamma P_D$ is kept constant during T minutes, then removed and recovery is measured during a period (8 - 9) of T_r minutes.

Alt. 4 As Alt. 3 but the structure is reloaded to failure ($P = P_B$).

Alt. 2 is intended for "proof loading" and the case when the capacity at more than one load combination is tested. The value of γ depends on the confidence required in estimating capacity. Alt. 3 and 4 are intended for the study of deformation and capacity at long-term loading.

5.3 Reference value of applied load

The reference value of load, P_D in figure 1, should approximately correspond to the "design" value of the load at the servicability limit state, normally the total of permanent loads (characteristic values) and variable loads (reduced characteristic values). It should then be observed that code value usually includes the gravity load from the structure (G_0), while P_D does not.

It should also be observed that the load caused by the "self-weight" may be favourable or unfavourable to the load-carrying capacity of the structure. Generally, when simulating combination of loads with different distribution over the structure or over its service time, the influence on stresses and deformations of the partial loads should be considered in chosing the P_D-values for the different members.

6. Recordings of load and deformation

The deformation (deflection) shall be measured at the number of points prescribed or regarded necessary for the judgement of the performance of the structure. A minimum requirement is that the deflection is measured at the point of maximum displacement.

Measurements of load and deformation shall be recorded, preferably continously. A minimum requirement is that load and deformation is recorded when load application respectively removal of load is started or finished (i.e. at the points marked by circles in figure 1) and, additionally, at each loading increment of $\Delta P = 0.5\ P_D$.

During constant load, time and deformation should preferably be recorded continously or at least five times during the period (three points between the starting and final points).

7. Test report

The test report shall include:

a) Specification of material (for timber species, grade, density and moisture content). Deviations from specifications.

b) Specifications of design. Deviations.

c) Conditions of testing, including methods of loading and of measuring loads and deflections.

d) Test results. Maximum load and deformation. Load/deformation curves.

e) Type and position of fractures.

f) Nature and size of defects in the materials which contributed to failure.

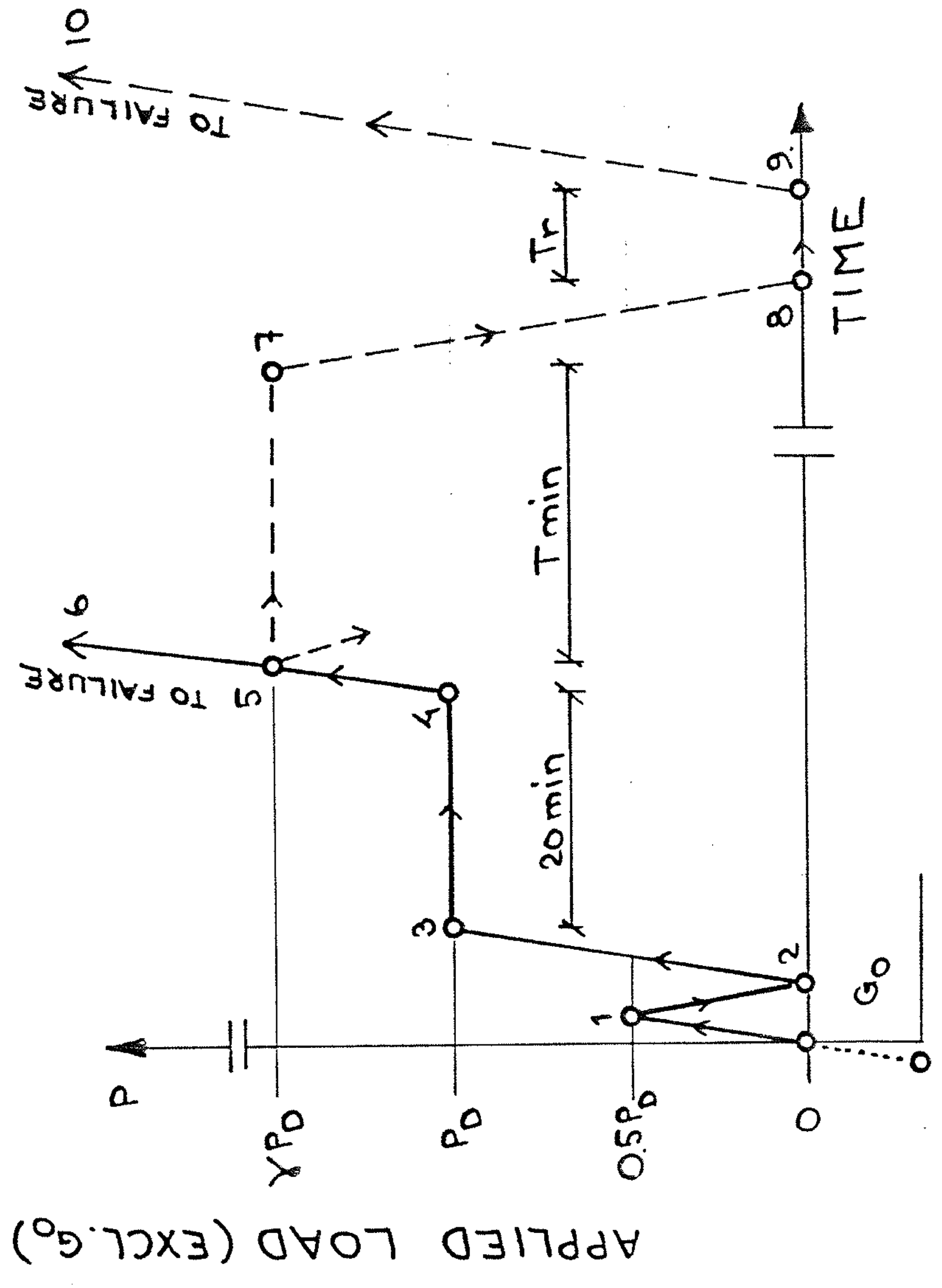


FIGURE 1 LOADING PROGRAM

Written proposals for changes in the text hereunder in such a form that they directly could replace the draft-text are welcomed by J. Kuipers, Delft University of Technology, Department of Civil Engineering, P.O. Box 5048, 2600 GA Delft, The Netherlands. Also other written comments will be accepted gratefully. Please send such proposals/comments before 1st January 1985.

D R A F T - July 1984

TSB-1. Testing methods for wood-based-board materials in structural grades for use in load-bearing structures

FOREWORD

The CIB working committee W18 "Timber Structures" is drafting a timber design code for international use in the design of timber structures. To support such a document it is necessary to have acceptable test methods to enable the development of comparative design information for different wood-based-board materials. Consequently, the CIB W18 committee asked the joint committee 3TT of RILEM/CIB to develop acceptable test methods for structural plywoods, which were published in "Materials and Structures", Vol. 14 - No 79, January - February 1981.

It was felt that an equal urgent need exists for testing methods for the determination of properties of other board materials used in load bearing timber structures. It was considered necessary, therefore, to start drafting an international standard on this subject, where special emphasis should be given to the purpose of the application of the test results, i.e. the determination of characteristic strength values for structural design.

This task was started within a new joint committee 57-TSB of RILEM and CIB: "Testing methods for structures and board materials". The underlying recommendations were worked out on the basis of the recommendation TT-2: "Testing methods for plywood in structural grades for use in load-bearing structures". In that document, TT-2, medium-size test specimens were adopted for the determination of characteristic strength and stiffness properties of plywood. Although various board materials may not demonstrate defects like some types of plywood, the different dimensions of the wood elements may have comparable effects on the properties. It is proposed, therefore and for reasons of standardisation, to maintain the same test pieces as for plywood. It should be kept on mind that these recommendations are not intended for quality control tests.

Testing methods for wood based board materials in structural grades for use in load-bearing structures

INTRODUCTION

This Recommendation, which is based on the recommendations of W18 - Timber Structures - Commission of CIB and 57-TSB - Testing methods for structures and board materials - Committee of RILEM/CIB, specifies standard methods for the determination of some physical and mechanical properties of commercial wood-based-board material.

It is known from current unfinished research programmes that the size and shape of the test piece, and the size of the wood elements influence the strength of the test piece, but these relationships are not yet established for all commercial sheet materials.

The strength values from the test pieces described in this Recommendation may need to be modified before being used in the design of structural components: for example, the design strength and stiffness of a full panel may differ from the values found from the test specimens defined in this Recommendation. It is expected that the relationship between the strength of the test piece and that of boards in structural components will be given in Codes of Practice for the Design of Structural Components.

Sampling techniques, the selection of test pieces and the analysis of data will be dealt with in further Recommendations which are in preparation.

Test of the glue properties in particle panel products are not included in this Recommendation.

It is not intended that this Recommendation should be used for routine quality control testing, where smaller test pieces may prove adequate.

1. SCOPE AND FIELD OF APPLICATION

This Recommendation specifies standard methods for determining some physical and mechanical properties of commercial wood based particle panel products intended for use in load-bearing timber structures.

2. SYMBOLS

A, cross-sectional area (mm^2);
a_1, distance from the centre of the test piece to the point where the deflection is measured (mm);
b, width of test piece or sample (mm);
EA, direct stiffness (N);
EI, bending stiffness (N/mm^2);
F, load (N);
f, strength (N/mm^2);
G, shear modulus (N/mm^2);
l, length of test piece or sample (mm);
l_1, gauge length (mm);
M, moment (N/mm);
m_ω, mass of strip immediately after testing (g);
m_0, constant mass of strip or sample after drying (g);
t, thickness of test piece (mm);
W, section modulus (mm^3);
w, deflection, deformation or slip (mm);
$\rho_{0\omega}$, nominal density (kg/m^3);
ω, moisture content.

Subscripts applied to capacities, stengths, stiffnesses, and moduli of elasticity :
b, bending;
c, compression;
max, maximum;
p, panel shear;
r, in plane of plies shear;
t, tension.

Prefix applied to loads, moments, deflections, deformations and slips:
Δ, increment.

3. SAMPLING

3.1. Sampling of panels

The panels from which the test pieces are cut shall be sampled in accordance with ISO 0000 (1).

3.2. Sampling of test pieces from panels

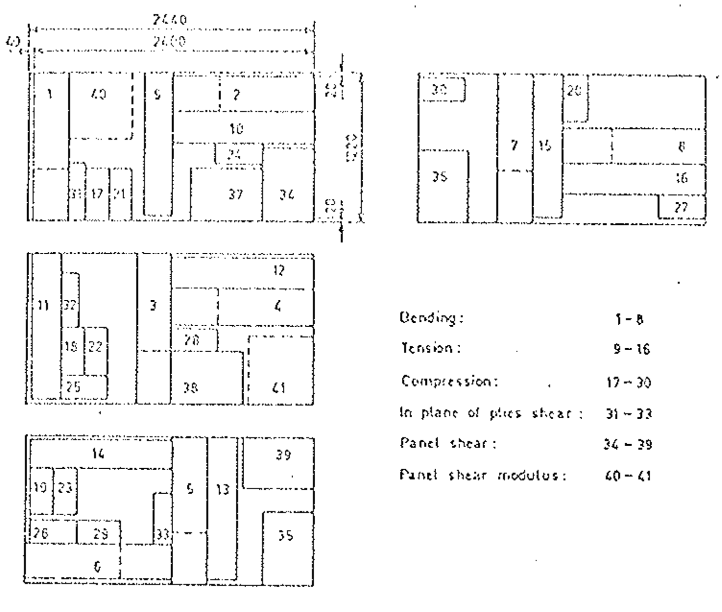
Test pieces shall normally be cut from the panels in accordance with the cutting given in figure 1. This cutting schedule is based on a sampling unit of four panels. Alternative schedules may be developed when the test pieces are cut other than parallel and perpendicular to the two main directions of the panel, or when only some of the strength properties are to be developed or when dictated by the special needs of the test.

SECTION ONE: PHYSICAL PROPERTIES

4. DIMENSIONS OF TEST PIECES

4.1. Method of measurement

The method of taking measurements and the type of equipment to be used shall be in accordance with ISO 3804.



Bending:	1 - 8
Tension:	9 - 16
Compression:	17 - 30
In plane of plies shear:	31 - 33
Panel shear:	34 - 39
Panel shear modulus:	40 - 41

Fig. 1. — Cutting schedule.

(1) In preparation

4.2. Measurements to be taken

The thickness of the test piece shall be measured to the nearest 0.01 mm at four points and the average recorded. The width and length of the test piece shall be measured to the nearest 1 mm at two points and the average recorded.

The dimensions of the test piece shall be measured at the points specified in 4.3.

4.3. Points of measurements

The measurements of the dimensions of the test piece shall be taken at the following points:

(*a*) bending; the thickness at four points, two on each edge 100 mm from the mid-length; the width at two points 100 mm from the mid-length;

(*b*) compression; the thickness at four points, two on each edge 100 mm from the mid-length; the width at two points 100 mm from the mid-length;

(*c*) tension; the thickness at four points, two on each edge 400 mm from the mid-length; the width at two points 400 mm from the mid-length;

(*d*) panel shear strength; the thickness at four points, two on each edge 100 mm from the mid-length, the length at two points 50 mm from each side;

(*e*) panel shear modulus; the thickness at the mid-points of the four sides; the length and width at the edges;

(*f*) in plane of plies shear; the thickness at four points, two on each edge 100 mm from the mid-length; the length at two points 50 mm from each side.

5. MOISTURE CONTENT

The moisture content shall be determined from a strip taken not nearer than 150 mm from the end of the test piece or from a separate matched test piece. The strip shall have the same thickness and width as the test piece and shall have a length of 25 ± 5 mm.

The strip shall be weighed immediately after testing and then dried to a constant mass (2) in a vented oven at a temperature of 103 ± 2°C. The balance used shall be capable of measuring the mass to an accuracy of 0.1%.

The moisture content shall be calculated from the following formula:

$$\omega = (m_\omega - m_0)/m_0.$$

where ω is the moisture content; m_ω is the mass of the strip immediately after testing, in grams; m_0 is the constant mass of the strip after drying, in grams.

The moisture content shall be calculated to three decimal places.

(2) Constant mass is considered to be reached when two successive weighing operations, carried out at an interval of 6 hours, do not differ by more than 0.1% of the mass of the strip.

6. DENSITY

The density of each test piece shall be determined from the test piece or from a sample taken from the same portion of the panel as the test piece. Where suitable, the strip which is prepared for the measurement of moisture content (*see* clause 5) may also be used to determine density.

The balance used shall be capable of measuring the mass to an accuracy of 0.1%.

The density, based on the mass when oven-dry and the volume at test, shall be calculated from the following formula:

$$\rho_{0,\omega} = 10^6 \times m_0 / lbt$$

where $\rho_{0,\omega}$, is the denstity (kg/m³); m_0, is the mass of the sample after drying (g); l, is the length of the sample (mm); b, is the width of the sample (mm); t, is the thickness of the sample (mm).

The density shall be calculated to three significant figures.

If the density is obtained on a different basis, then the basis of the density value with respect to volume and moisture content shall be stated.

SECTION TWO: MECHANICAL PROPERTIES

7. CONDITIONING AND TESTING CLIMATES

All test pieces shall normally be conditioned, prior to final machining and testing, to constant mass (2) and moisture content in an atmosphere of relative humidity 65 ± 5% and temperature 20 ± 2°C (3).

Where possible, conditions of testing should be the same as those in the conditioning chamber, but where this is not possible tests should be undertaken immediately after the test pieces have been removed from the conditioning chamber.

8. BENDING STRENGTH AND STIFFNESS

8.1. Test piece

The test piece shall be rectangular in cross-section.

The depth of the test piece shall be equal to the thickness of the panel.

The width of the test piece shall be 300 mm.

The length of the test piece will depend on the method used for applying the load (*see* 8.3) but shall be sufficient to ensure that the length of the zone subjected to the uniform moment shall not be less than 300 mm.

? Unless otherwise specified, an estimate shall be made of the worse face of the test piece and this face shall be stressed in tension during the bending test.

8.2. Sampling of test pieces from panels

Eight test pieces shall be cut from the sampling unit of four panels in accordance with the schedule given in figure 1. A further eight test pieces shall be cut from the four panels in accordance with ISO .... (4)

8.3. Loading method and equipment

The method used for applying the load shall be such that a zone of length not less than 300 mm at the middle of the length of the test piece shall be subjected to a uniform moment. The method of applying the load shall be such that direct tension or compression forces are not applied to the test piece at large deflections.

Note. — Large deflections may occur when specimens with small bending stiffness are tested to failure and alternative test arrangements may be required. In general the test method described in this clause is not suitable for a specimen with a thickness lower than 6 mm.

The loading equipment shall be capable of measuring the load to an accuracy of 1%.

8.4. Test procedure

8.4.1. *Rate of application of load*

The load shall be applied with a continuous motion throughout the test. The rate of loading shall be adjusted so that the maximum load be reached within 300 ± 120 seconds.

The time taken from the beginning of the loading to the maximum load shall be measured and recorded to the nearest 30 seconds.

(3) The test methods specified in this Recommendation may also be used at other testing climates.

(4) ISO draft international standard "Determination of apparent modulus of elasticity in bending and of bending strength" N 17.

8.4.2. *Measurement of deformation*

The deflection of the test piece shall be measured midway between two points on the longitudinal axis of the test piece located in the zone of uniform moment. The distance between the two points (gauge length) shall be not less than 250 mm and the points shall be spaced as far apart as possible consistent with maintaining adequate clearance between the gauges and the loading equipment.

The deflection shall be measured to the nearest 0.01 mm.

Note. — The curvature of the test piece may be obtained by measuring the angular rotation at the ends of the zone of uniform moment.

Note — The eight small specimens according to 8.2 shall be tested according to ISO .... ([4]).

8.5. Expression of results

8.5.1. *Bending stiffness and modulus of elasticity*

The bending stiffness of the test piece shall be calculated from the following formula:

$EI = \Delta M l_1^2 / 8 \Delta w,$

where EI is the bending stiffness of the test piece (N/mm²); ΔM is the increment of moment at mid-length on the straight line portion of the load-deflection curve (N/mm), Δw is the increment of deflection corresponding to ΔM (mm); l_1 is the gauge length (mm).

The bending stiffness of the test piece shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the second moment of area shall be stated.

8.5.2. *Ultimate moment capacity and bending strength*

The ultimate moment capacity of the test piece, which is the maximum moment resisted by the test piece, shall be recorded to three significant figures.

If a value of the bending strength is subsequently calculated from the ultimate moment capacity it shall be calculated from the following formula:

$f_b = M_{max} / W,$

where f_b, is the bending strength (N/mm²); M_{max} is the maximum moment (N/mm); W is the section modulus (mm³).

The bending strength shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the section modulus shall be stated.

8.5.3. *Creep in bending*

Test to be developed; suggestions are welcome!

9. COMPRESSION STRENGTH AND STIFFNESS

9.1. Test piece

The test piece shall be rectangular in cross-section.

Several pieces of the panels to be tested shall be glued face to back until the thickness of the test piece is not less than 40 mm. No test piece shall be made entirely from material from one panel, except where the panel thickness is 40 mm or more.

The width of the test piece shall be 200 mm and its length shall be 400 mm.

Care shall be taken in preparing the test piece to make the end surfaces smooth and parallel to each other and at right angles to the length.

9.2. Sampling of test pieces from panels

The test piece shall be made from the sampling unit of four panels in accordance with the schedule given in figure 1.

9.3. Loading method and equipment

The load shall be applied through a hinged connection on the upper head of the testing machine to allow for any deviation from parallel of the ends of the test piece and permit adjustment to the end of the test piece in one direction. The test piece shall be loosely held by smooth side restraining rails. Suitable loading apparatus is given in annex A[1].

The loading equipment shall be capable of measuring the load to an accuracy of 1%.

9.4. Test procedure

9.4.1. *Rate of application of the load*

The load shall be applied with a continuous motion throughout the test. The rate of loading shall be adjusted so that the maximum load be reached within 300 ± 120 seconds.

The time taken from the beginning of the loading to the maximum load shall be measured and recorded to the nearest 30 seconds.

9.4.2. *Measurement of deformation*

Data for load-deformation curves shall be taken to determine the compression stiffness and the modulus of elasticity.

The deformation shall be taken over the central portion on both sides of the test piece using a gauge length of not less than 125 mm but not greater than 200 mm. The average of the two readings shall be used in the calculation of the stiffness and modulus of elasticity of the test piece.

The deformation shall be measured to the nearest 0.01 mm.

9.5. Expression of results

9.5.1. *Compression stiffness and modulus of elasticity*

The compression stiffness of the test piece shall be calculated from the following formula:

$EA_c = \Delta F l_1 / \Delta w.$

where EA_c is the compression stiffness of the test piece (N); ΔF is the increment of load on the straight line portion of the load-deformation curve (N); l_1 is the gauge length (mm); Δw is the increment of deformation corresponding to ΔF over the gauge length l_1 (mm).

The compression stiffness of the test piece shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the cross-sectional area shall be stated.

9.5.2. *Ultimate compression capacity and compression strength*

The ultimate compression capacity of the test piece, which is the maximum compression load resisted by the test piece, shall be recorded to three significant figures.

If a value of the compression strength is subsequently calculated from the ultimate compression capacity it shall be calculated from the following formula:

$f_c = F_{max}/A,$

where f_c is the compression strength (N/mm²); F_{max} is the maximum compression load (N); A is the cross-sectional area (mm²).

The compression strength shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the cross-sectional area shall be stated.

10. TENSION STRENGTH AND STIFFNESS

10.1. Test piece

The test piece shall be rectangular in cross-section.

The thickness of the test piece shall be equal to the thickness of the board.

The test piece may have a constant width of 250 mm throughout its length or may be necked down to a constant width of 250 mm for a length of 600 mm.

Note. — If the test piece is necked down to 250 mm then the cutting schedule given in figure 1 will need to be modified.

The length of the test piece shall be 1,200 mm.

Note. — For certain species of board the test specimen defined in this clause may cause a large percentage of failures at, or within, the grips. For these boards a more appropriate test specimen must be designed. The width of the specimen shall be not less than 150 mm and details of its geometry shall be given in the test report.

10.2. Sampling of test pieces from panels

Eight test pieces shall be cut from the sampling unit of four panels in accordance with the schedule given in figure 1.

10.3. Loading method and equipment

The test piece shall be held in grips which apply the required loads to the test piece with the minimum influence on load at, or location of, failure. Such devices shall not apply a bending moment to the test piece, allow slippage under load, or inflict damage or stress concentrations to the test piece.

For ideal test conditions, the grips should be self aligning. The type of grips used shall be recorded.

Suitable loading apparatus is given in annex (5).

The loading equipment shall be capable of measuring the load to an accuracy of 1%.

10.4. Test procedure

10.4.1. *Rate of application of load*

The load shall be applied with a continuous motion throughout the test. The rate of loading shall be adjusted so that the maximum load is reached within 300 ± 120 seconds.

The time taken from the beginning of the loading to the maximum load shall be measured and recorded to the nearest 30 seconds.

(5) In preparation

10.4.2. *Measurement of deformation*

Data for load-deformation curves shall be taken to determine the tension stiffness and the modulus of elasticity.

The deformation shall be taken over the central portion on both sides of the test piece using a gauge length of not less than 125 mm but not greater than 400 mm. The average of the readings shall be used in the calculation of the test piece stiffness and the modulus of elasticity.

The deformation shall be measured to the nearest 0.01 mm.

10.4.3. *Unacceptable test results*

Any test piece that fails at, or within, the grips shall be rejected.

10.5. Expression of results

10.5.1. *Tension stiffness and modulus of elasticity*

The tension stiffness of the test piece shall be calculated from the following formula:

$$EA_t = \Delta F l_1 / \Delta w,$$

where EA_t is the tension stiffness of the test piece (N); ΔF is the increment of load on the straight line portion of the load-deformation curve (N); l_1 is the gauge length (mm); Δw is the increment of deformation corresponding to ΔF over the gauge length l_1 (mm).

The tension stiffness of the test piece shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the cross-sectional area shall be stated.

10.5.2. *Ultimate tension capacity and tension strength*

The ultimate tension capacity of the test piece, which is the maximum tension load resisted by the test piece, shall be recorded to three significant figures.

If a value of the tension strength is subsequently calculated from the ultimate tension capacity it shall be calculated from the following formula:

$$f_t = F_{max} / A,$$

where f_t is the tension strength (N/mm²); F_{max} is the maximum tension load (N); A is the cross-sectional area (mm²).

The tension strength shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the cross-sectional area shall be stated.

11. PANEL SHEAR STRENGTH

11.1. Test piece

The test piece shall be rectangular in cross-section.

The thickness of the test piece shall be equal to the thickness of the panel.

The width of the test piece shall not be less than 430 mm and the distance between the rails shall be 200 mm (see figure 2).

The length of the test piece shall be 600 mm.

Note. — It is recommended that the stronger direction of the specimens be oriented perpendicular to the rails. For thin board it may be necessary to use this orientation in order to preclude failure by buckling. Buckling of thin specimens may be avoided by laminating two or more layers together to develop adequate stiffness. Because laminating can affect panel shear properties, the report of test results must so indicate when specimens are laminated.

Some reconstituted panel materials have high panel shear strength but low internal bond strength and interlaminar shear strength required to transfer these stresses from the rails into the panel. The rail along with a surface covering of fiber or particles may shear from the panel. This may be prevented by applying lateral pressure to the rails. A simple method of applying this pressure is by means of bolts as illustrated in figure 3.

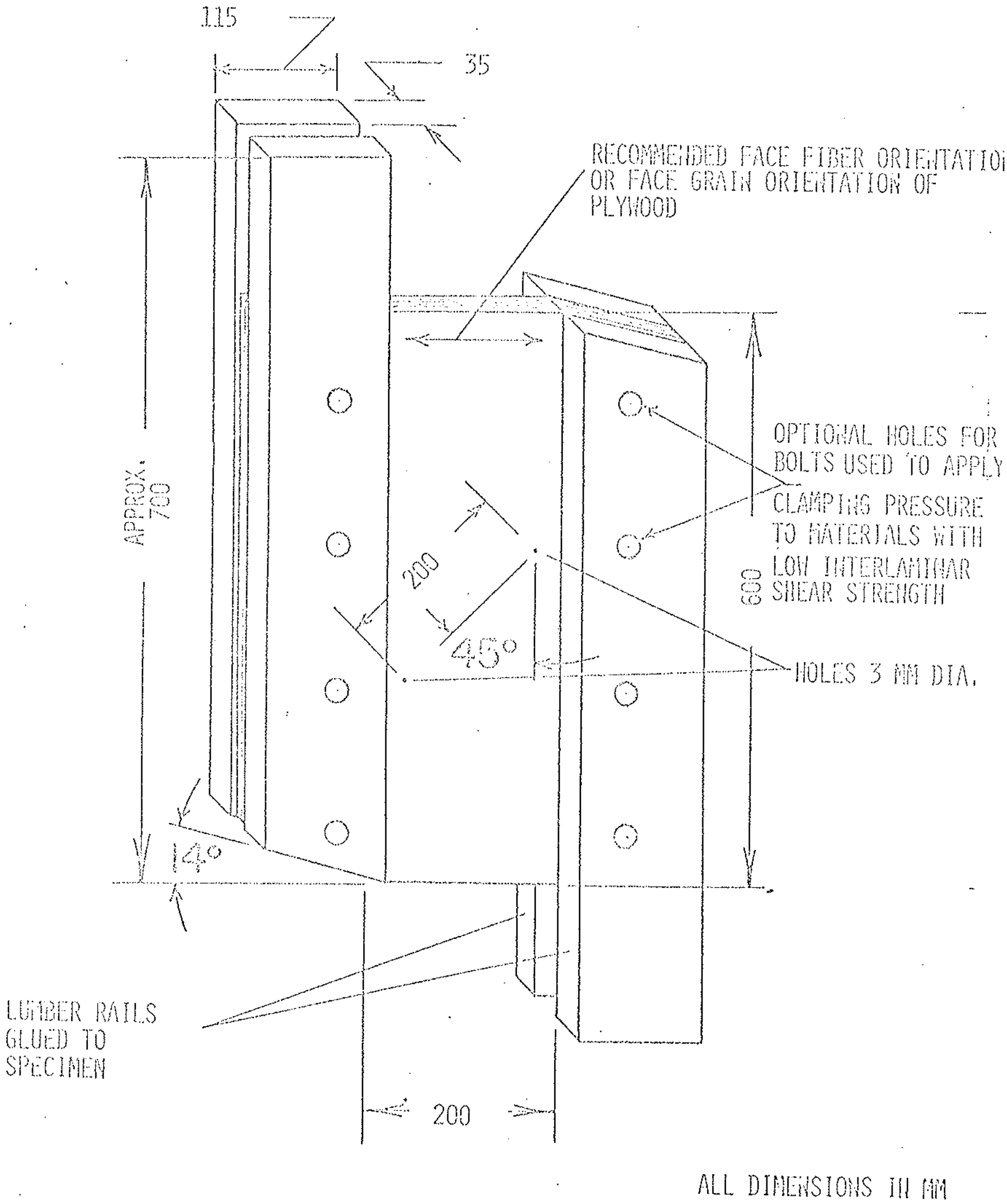


Figure 2. Details of panel-shear test piece.

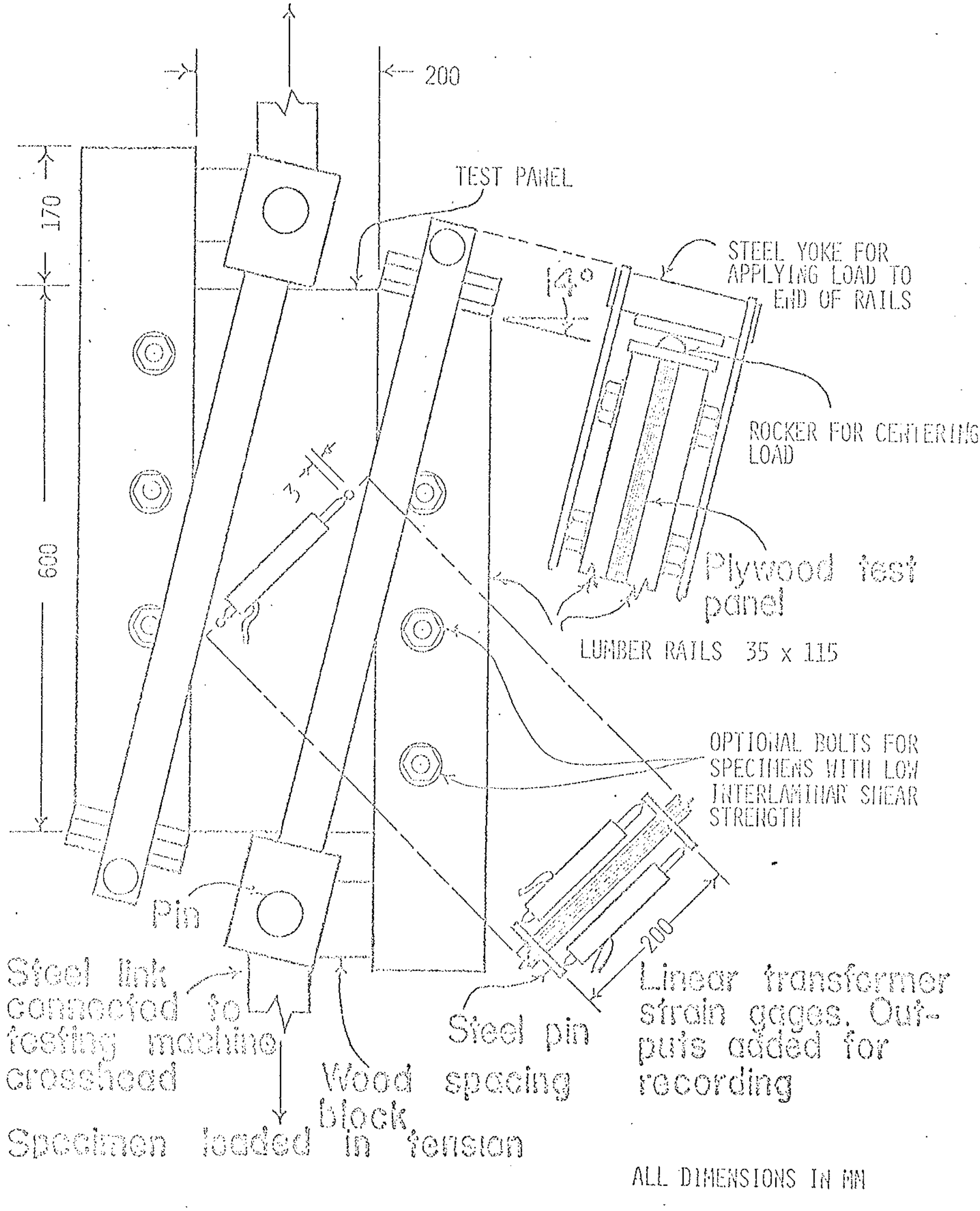


Figure 3. Loading and strain-measuring apparatus for panel-shear test piece.

Timber rails having minimum dimensions of 35 by 115 mm by approximately 700 mm long shall be glued to both sides of the test piece at each edge. The width of the rails may be increased to eliminate a shear failure between the rails and the test piece. The rails shall be spaced 200 mm apart with their ends even with the plywood test piece at two diagonally opposite corners as shown in figure 2. Prior to gluing, the rails and the test piece shall be conditioned to the approximate moisture content at which the test piece is to be tested. After gluing, a bevel of 14 deg. shall be cut on the end of both pairs of timber rails where the major compression load is to be applied. (6).

Steel rails may be substituted for lumber rails and clamping may be substituted for gluing provided that no crushing of the specimen or slippage between rail and specimen occurs. Special rail facings may be needed to develop adequate friction between rail and specimen. The clamping method is particularly well suited to reconstituted panel materials that would otherwise require bolting to prevent interlaminar shear in the specimen under the rails.

When modulus of rigidity is to be determined, 3 mm holes shall be drilled through the panel at each end of a 200 mm gage length as shown in figures 2 and 3. The gage length along which deformation is measured shall be the compression diagonal at 45 degrees to the rails passing through the center point of the shear area. The gage length shall be centered between the rails along this line.

(6) If the width of the rails is other than 115 mm, the bevel shall be cut at such an angle to ensure that the applied loads are collinear.

11.2. Sampling of test pieces for panels

Three test pieces shall be cut from the sampling unit of four panels in accordance with the schedule given in figure 1.

Note. — Provision is also made in the cutting schedule for three test pieces with their long dimension parallel to panel length.

11.3. Loading method and equipment

The loading shall be applied so that the resultant of the forces applied to a pair of rails shall be a single force acting along the longitudinal axis of the test piece both in the plane of the test piece and in the thickness direction. The load on the rails shall be applied by separating the machine cross-heads.

The loading equipment shall be capable of measuring the load to an accuracy of 1%.

A suitable apparatus for applying equal loads to the rails is shown in figure 3. The opposing collinear forces applied to pins located on the longitudinal axis of the test piece and perpendicular to its plane are divided into two components, a major compression force applied to the end of the rail by a loading yoke free to pivot about the pin; and a minor lateral force applied to the projecting end of the rail by a block that keeps the pin spaced the correct distance from the rail it loads. The major compressive load is applied through a two-way rocker and bearing-plate arrangement to distribute the load uniformly to the rail end. The rigid block applying the lateral force to the projecting rail ends ensures that the pin remains perpendicular to the plane of the test piece.

Other loading methods can be used and may be more appropriate with steel rails. Any other methods must result in the same shears and moments applied to the portion of the test specimen between rails. A possible alternative has been given in figure 3 a.

11.4. Test procedure

11.4.1. Rate of application of load

The load shall be applied with a continuous motion throughout the test. The rate of loading shall be adjusted so that the maximum load is reached within 300 $\pm$ 120 seconds.

The time taken from the beginning of the loading to the maximum load shall be measured to the nearest 30 seconds.

11.4.2 Measurement of deformation

Insert 3 mm pins through the holes described in 11.1. Use linear transformers or other suitable means of measuring the compression strain between pins as shown in figure 3. Use one gage on each side of the panel and average the results either electronically by appropriate circuitry at time of test or when performing the calculations. Accuracy shall be to 2% of total elongation.

11.4.3 Unacceptable test results

Any test piece that fails in other than shear or tension between the rails shall be rejected.

Note. — Because shear stresses applied by the rails also produce equal tensile stresses at 45 degrees to the rails, materials having tensile strength less than or approaching their shear strength will usually display one or more tension breaks at approximately 45 degrees to the rails and often extended beneath them. Many reconstituted panel materials display this characteristic. Such results are acceptable.

11.5. Expression of results

11.5.1 Panel shear strength

The panel shear strength shall be calculated from the following formula:

$$f_p = F\ max/lt$$

where f_p is the panel shear strength (N/mm^2); F max is the maximum load (N); l is the length of test piece (mm); t is the thickness of test piece (mm).

The panel shear strength shall be calculated to three significant figures.

The thickness (normal* or measured) used to calculate the panel shear strength shall be stated.

11.5.2 Panel shear modulus of rigidity

Panel shear modulus of rigidity shall be calculated from load and deformation data according to the formula.

$$G = 0.5\ (P_g/d)\ (L/lt)$$

where G is modulus of rigidity, N/mm^2; (P_g/d) slope of the force-deformation diagram, N/mm; and L is the gage length, mm. Because the force-deformation curves are often curvilinear at low stress levels, it is recommended that the portion of the curve

* nominal ??

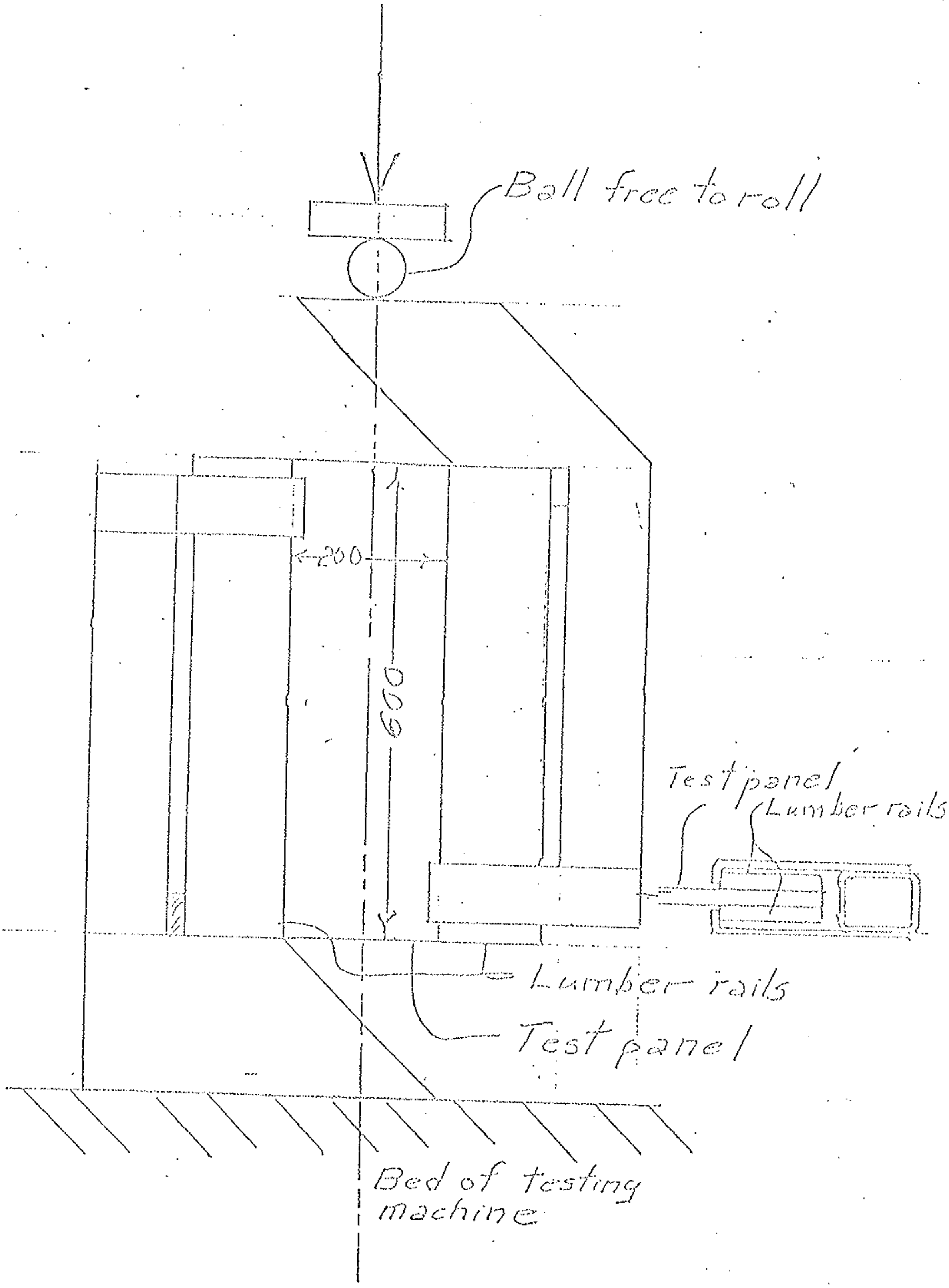


figure 3a - Simplified method of loading two-rail shear specimens in compression selected for computation of G represent the approximate range of panel shear design stress.

It is recommended that the shear modulus calculated by the above formula be multiplied by a factor of 1.19 to account for higher than average shear stress distribution at the location of strain measurement. The report shall clearly state whether this factor has been applied.

12. INTERLAMINAR SHEAR STRENGTH AND EFFECTIVE SHEAR MODULUS

12.1 Test piece

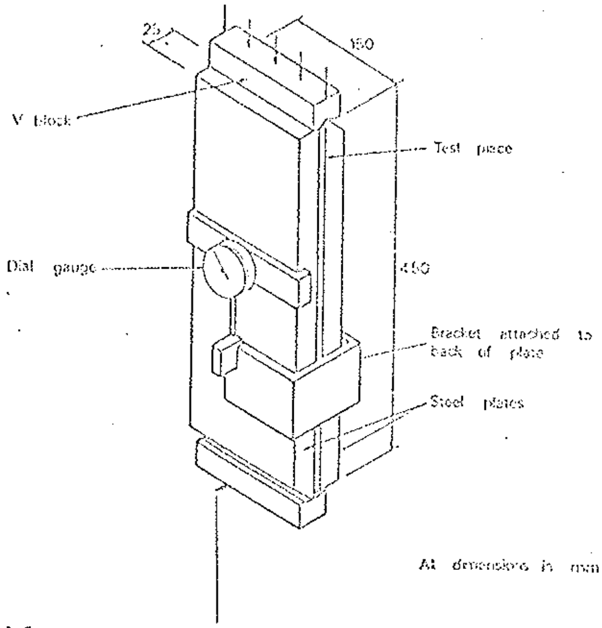
The test piece shall be rectangular in cross-section.

The thickness of the test piece shall be equal to the thickness of the panel.

The width of the test piece shall be 150 mm and its length shall be 450 mm.

The test piece shall be glued between steel plates 25 mm thick, 450 mm long and 150 mm wide. The plates shall be bonded to the test piece with an adhesive sufficiently rigid to preclude a significant contribution of adhesive creep to the measured deformation. One end of each plate shall be provided with a knife edge projecting 6 mm beyond the end of the test piece as shown in figure 6.

The long dimension of the test piece may be either parallel or perpendicular to the length of the original panel. The orientation shall be recorded.



V block supported on rocker with axis perpendicular to plane of specimen.

figure 4 - Interlaminar shear test using a dial gage for measuring plate slip.

12.2. Sampling of test pieces from panels

Three test pieces shall be cut from the sampling unit of four panels in accordance with the schedule given in figure 4.

12.3. Loading method and equipment

The load shall be applied through V blocks so that it is uniformly distributed along the knife edges. The V blocks shall be vertically positioned in the machine, one above the other, causing the forces applied to the test piece to act parallel to the axis of the machine. The test piece itself will be slightly inclined when placed in the machine.

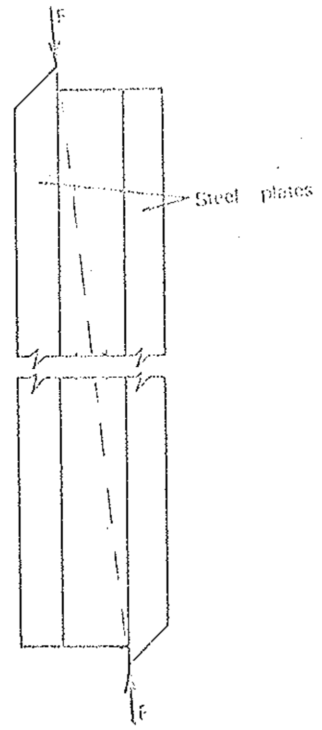


Fig. 5 - Interlaminar shear test

Note. - Pivots permitting rotation about an axis parallel to the knife edge or spherical seats free to pivot in this manner should not be used as they create unstable loading which may cause violent ejection of the test piece from the machine and a hazard to operating personnel.

The loading equipment shall be capable of measuring the load to an accuracy of 1%.

12.4. Test procedure

12.4.1 Measurement of deformation

The load shall be applied with a continuous motion throughout the test. The rate of loading shall be adjusted so that the maximum load is reached within 300 $\pm$ 120 seconds.

The time taken from the beginning of the loading to the maximum load shall be measured and recorded to the nearest 30 seconds.

12.4.2 Measurement of deformation

Data for load-slip curves shall be taken to determine the effective interlaminar shear modulus.

A suitable method of measuring the slip between the steel plates is shown in figure 4.

The slip between the steel plates shall be read to the nearest 0.002 mm.

12.4.3 Unacceptable test results and types of failure

Any test assembly that fails wholly or partially in the bond between the metal plates and the test piece shall be rejected. Acceptable failures in reconstituted materials may occur anywhere within the thickness of the test piece, but not at the glued interface with the steel plate.

12.5. Expression of results

12.5.1 Effective interlaminar shear modulus

The effective interlaminar shear modulus shall be calculated from the following formula:

$$G_r = \Delta Ft/\Delta wlb$$

where G is the effective interlaminar shear modulus (N/mm^2); Δ F is the increment of load on the straight line portion of the load-slip curve (N); Δ w is the increment of slip corresponding to Δ F (mm); t is the thickness of the test piece (mm); l is the length of the test piece (mm); b is the width of the test piece (mm).

The effective interlaminar shear modulus shall be calculated to three significant figures.

The thickness (nominal or measured) used to calculate the effective in interlaminar shear modulus shall be stated.

12.5.2 Interlaminar shear strength

The interlaminar shear strength shall be calculated from the following formula:

$$f_r = F\ max/lb$$

where f_r is the interlaminar shear strength (N/mm^2); F max is the maximum load (N); l is the length of test piece (mm); b is the width of test piece (mm).

The interlaminar shear strength shall be calculated to three significant figures.

13. TEST REPORT

The test report shall include details of the test material, the method of test, and the test results. The amount of detail given under each of these headings will depend on the purpose of the tests.

The following data on material shall normally be given: particle type, orientation, adhesive, surface treatment, manufacture, manufacturer's grade or product standard grade, and any other information pertinent to the purpose of the tests.

The following data concerning the test conditions shall normally be given: the type of test, the accuracy and method of loading, the accuracy of measurements of deformation, the method of determining the slope of the load-deformation curve, the temperature and relative humidity of the time of test.

For individual test pieces the following data shall be given: test piece dimensions, moisture content, time of failure, maximum loads, description of failure, and the calculated values of stiffness and capacity.

When moduli and strengths are calculated the basis on which they have been determined shall be stated. The thickness (nominal or measured) used in the calculation shall be stated.

Additional data may be required in some cases. This may include the following: full details of method of manufacture, full details of any natural defects or manufacturing features which influence the test results, density and load-deformation diagrams. If the test pieces are not cut in accordance with the schedule shown in figure 1, then details of the cutting schedule shall be given.

The number of test pieces and panels tested for each property shall be stated in the test report, and if a statistical treatment of the data is possible then the value of the standard deviation or coefficient of variation for each property shall also be given, as well as the mean.

CIB W.18 SUB-GROUP AFRICAN, CARIBBEAN AND LATIN AMERICAN REGIONS

REPORT FOR MEETING 17 AT RAPPERSWIL SWITZERLAND MAY 1984

With the development of new approaches to stress grading and proof loading techniques, the possibility of more convenient strength groupings of structural lumber and the continued thrust towards more enlightened Codes of Practice based on limit state concepts there is even more justification for the continued interest in this CIB Sub-Group.

The African Continent is recognising the importance of renewable building resources such as structural timber. It is expected that Zimbabwe will take initiatives to secure its position as a local centre of technical excellence within the Southern African Development Coordination Conference (SADCC) and reach out to the African Network of Scientific and Technical Institutions (ANSTI) in Nairobi under the auspices of UNESCO to promote post-graduate courses in Timber Engineering, based in Harare.

It is, perhaps, not entirely fortuitous that Zimbabwe has again been chosen by the FAO to establish and fund a Technical Training Centre for timber utilisation, sawmilling and secondary processing which complements the Timber Engineering Research Centre now being funded by the Norwegian Organisation NORAD.

The UNIDO (Vienna) organisation continues to welcome the initiative of the Sub-Group and the opportunity was taken at the Pacific Timber Engineering Conference in Auckland (May 21 - 25) to encourage possible cooperation with the Pacific Area Standards Conference (PASC) convenor Dr. R.H. Leicester (Australia). This was supported by Mr. R. Hallett (UNIDO) who also attended the conference in addition to CIB W18 members Dr. H.J. Larsen (Denmark) and Prof. B. Madsen (Canada) as keynote speakers.

It is regretted that the dates of the CIB W.18 meetings in Switzerland and PTEC in New Zealand had to clash. Fortunately Zimbabwe was able to send Eng. D. Cresswell, a distinguished local Consulting Engineer, to Switzerland at this time.

News from the Caribbean and Latin American Regions has been scant but it is believed that my colleague Dr. Amantino De Freitas (Brazil) has recently had additional responsibilities thrust upon him.

There are signs, however, that interest is growing in Chile, Mexico and Argentina and it is hoped that Dr. De Freitas will report in due course. It is thought that Architect Carlos Alberto De Abreu Maffei, Head of the Brazilian Housing Programme and CIB Board member, has been having talks with Dr. R. Wright, President of CIB, about possibility of establishing a new CIB Working Commission to deal with the utilisation of tropical hardwoods in house construction.

I trust our CIB W.18 Chairman will be able to coordinate all our

/2....

activities as there are considerable interests in tropical hardwoods in Africa and the Far East (PASC Region).

The response from RILEM & IUFRO Groups may also be of some importance in this area.

In conclusion, I would confirm not only wide support for the establishment of regional technical groups but that I have noted in all personal contacts made in the southern hemisphere that many Third World nations have reached a common threshold of development in timber engineering and aspire avidly not only to rationalise the use of their forest product and preservation resources but to apply modern technology more readily within their respective building industries.

RSBeckett

R.S. Beckett
JOINT COORDINATOR

Dean of Engineering
University of Zimbabwe

14th June, 1984