Hemmer

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB-W18

MEETING NINE

PERTH, SCOTLAND

JUNE 1978

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1 LIST OF DELEGATES

AUSTRIA

E Armbruster	European Federation of Building Joinery Manufacturers' Association, Wien

BELGIUM

A Egerup	Automated Building Components, Brussels

CANADA

J D Barrett	Western Forest Products Laboratory, Vancouver
N Donkervoort	Council of Forest Industries of British Columbia, London, UK
T A Eldridge	Lamco Structural Products Ltd, Quebec
F J Keenan	University of Toronto, Toronto

DENMARK

M Johansen	Danish Building Research Institute, Horsholm
H J Larsen	Aalborg University Centre, Aalborg

EIRE

V Picardo	Institute for Industrial Research and Standards, Dublin

FEDERAL REPUBLIC OF GERMANY

K Hemmer	Karlsruhe University, Karlsruhe
H Kolb	Otto-Graf Institute, Stuttgart

FRANCE

P E H Crubilé	Centre Technique du Bois, Paris

NETHERLANDS

J Kuipers	Stevin Laboratory, Delft

NORWAY

N I Bovim	Norsk Treteknisk Institutt, Oslo
O B Kristiansen	Norges Byggstandardiseringsrad, Oslo

POLAND

W Marosz	Union of Building Joinery Industry, Warsaw
W Nozynski	Centralny Osrodek Badawczo, ~~Laskowa~~ Rozwojowy Stolarki Budowlanej „STOLBUD" Wołomin, ul. Laskowa 4 /b. Warschau/

SWEDEN

B Edlund	Chalmers University of Technology, Goteborg
B Noren	Swedish Forest Products Research Laboratory, Stockholm
B Thunell	Swedish Forest Products Research Laboratory, Stockholm

UNITED KINGDOM

L G Booth	Imperial College, London
H J Burgess	Timber Research and Development Association, High Wycombe
W T Curry	Building Research Establishment, Princes Risborough
P Grimsdale	Swedish-Finnish Timber Council, Retford
P G Jackson	Automated Building Components Ltd, Farnham
R Marsh	Arup Associates, London
R A Swann	Hydro-Air International, Ltd, High Wycombe
1 J G Sunley	Timber Research and Development Association, High Wycombe
2 J R Tory	Building Research Establishment, Princes Risborough

UNITED STATES OF AMERICA

F T Kurpiel	American Plywood Association, Chelmsford, UK

1 Co-ordinator and Chairman

2 Technical Secretary

2 CHAIRMAN'S INTRODUCTION

MR SUNLEY, co-ordinator of CIB-W18 and chairman of the meeting welcomed delegates to Scotland for the ninth meeting of the Commission. He particularly welcomed those from North America and several other new participants.

MR SUNLEY drew the attention of the meeting to the absence of Professor Karl Möhler because of a recent illness. He expressed appreciation for the positive contributions that Professor Möhler had made to the work of the Commission and looked forward to his participation in future meetings. A letter conveying these sentiments and best wishes for a speedy recovery was sent from the meeting.

The time and venue for the next full meeting of the Commission had not been discussed but MR SUNLEY explained that a brief meeting would take place on the morning of Friday 25 August 1978 in Vancouver, Canada. This meeting will be during the IUFRO S5.02 meeting and will serve the purpose of introducing some of the work of the Commission to Americans and Canadians.

DR BARRETT told the meeting that there were three important events planned for August 1978 other than the W18 meeting. These were: A Conference on Fracture Mechanics at Banff; the IUFRO Wood Engineering meeting at Vancouver, BC; and a Symposium on Nondestructive Testing of Wood at Vancouver, Washington. Details of these events are available from Dr Barrett, Western Forest Products Laboratory; Professor Madsen, University of British Columbia; and Dr Pellerin, Washington State University respectively.

Finally MR SUNLEY outlined an agenda for the meeting and this was agreed.

3 FIBRE BUILDING BOARD

MR BURGESS introduced 'The Structural Use of Tempered Hardboard' (CIB-W18/9-13-2) in the absence of Dr Chan, the author. He explained that the paper had formed the basis for the submission that tempered hardboard should be included in the English code CP 112. Dr Chan had set out to explain in some detail the derivation of the stress values for tempered hardboard and to evolve draft clauses for inclusion in CP 112.

PROFESSOR LARSEN considered the value of the paper to the CIB Timber Code to be limited because it was so detailed and it was, he thought, a very complicated approach for the introduction of a new material. He also pointed out that for the CIB Timber Code characteristic stress values were required and the elimination of safety factors might cause Dr Chan to have second thoughts about some of the stress values. PROFESSOR LARSEN concluded his observations by emphasising the need for agreed test methods for fibreboard before stress values could be considered.

DR BOOTH told the meeting that tempered fibreboard had not yet been accepted into CP 112. Some of the members of the sub-committees responsible for that code had reservations about the long term performance of the material under load and they would welcome comments from other countries that had more experience with the structural applications of tempered hardboard.

MR SUNLEY summed up the views of the delegates by saying that tempered hardboard, as an increasingly important structural material, should be included in the CIB Timber Code. Although this particular paper did not provide suitable clauses for the Code it had presented possible methods for the derivation of stresses for the material and had shown that it could not be treated in the same way as plywood. There was also a need for standardised test procedures so that comparable characteristic stresses could be produced by different countries.

4 CIB TIMBER CODE

Paper CIB-W18/9-100-1 'CIB Timber Code (Second Draft)' was introduced by the chairman. He told the meeting that the paper had evolved from a preliminary draft by Professor Larsen which had then been discussed and amended by a code drafting sub-committee meeting at the Princes Risborough Laboratory in March 1978. It was important, continued MR SUNLEY, that the meeting reach some agreement on producing from this paper a base for the CIB Timber Code, which was intended to be an international code for national code writers. At this stage the code did not adequately cover all timber based structural material, said MR SUNLEY, giving as examples the omission of fibre and particleboards; but it could probably form a sound base that could be amended as more information became available

PROFESSOR LARSEN told the delegates that at the code drafting sub-committee meeting Dr Kuipers had offered an alternative version of the draft code. It was not possible to say which of the two versions offered the better layout since neither had been used by practising engineers. However, the List of Contents of the draft now before the meeting had been widely circulated and the comments received from six countries had been taken into account.

MR CURRY agreed with Mr Sunley that the present document formed a good basis for a code but pointed out that it could not be used in its present form since there were no safety factors to link loads and stresses.

PROFESSOR LARSEN said that the present draft, with Chapter 3 omitted, was equally applicable to deterministic or limit states design. The choice of design methodology and the values of safety factors or partial factors were national governmental problems and could not be embodied into the CIB Code. In answer to a question from Dr Booth, Professor Larsen said that Chapter 3 should mention that design by calculation could be carried out by a theory of elasticity or by a theory of plasticity or both.

MR SUNLEY asked if reference to fire and corrosion should apprear in section 1.2.

PROFESSOR LARSEN suggested that fire was a limit state problem and should therefore be mentioned in Chapter 3. Corrosion was covered by the existing reference to correct maintenance.

The meeting then considered the paper in detail and the following comments and amendments were made:

Section 1.1	It was agreed that in paragraph 4 the words after ' ... or both' should be deleted. A further paragraph should be inserted making reference to the permitted methods of design.
Section 1.2	The fourth requirement is to read ' ... the structure, by design or the use of suitable materials or by impregnation, is

protected against attack by fungi, insects, shipworm, gribble, etc; and its integrity is ensured by correct maintenance.'

Table 1.3 — '180/π' to to be replaced by 'π/180 rad".

Section 2.1 — PROFESSOR LARSEN explained that the environmental conditions at which characteristic values were defined were not necessarily those at which testing would be conducted. It would be necessary to introduce corrections to some test data to derive characteristic stresses.

It was agreed that the characteristic bending strength values for glued-laminated timber should also be related to a section depth of 200 mm.

Section 2.2 — PROFESSOR KEENAN introduced paper 9-11-1 'Climate Classes for Timber Design' in support of two rather than four climate classes. He proposed that the climate class definitions should be governed by the response of the most important material, which was solid timber, and the work of Madsen and Aplin indicated that two climate classes were all that was needed.

PROFESSOR LARSEN agreed that for the strength of solid timber two climate classes were probably sufficient but deflection should also be considered and the response of composite materials which were included in the code could be more influenced by moisture content variations. PROFESSOR LARSEN favoured retaining the four climate classes.

DR NOREN suggested that since no distinction was made between classes 0 and 1 in Tables 6.0.2 and 5.1.0.b it should be possible to dispense with one of these two classes.

It was agreed that climate class 0 should be deleted.

Section 2.3 — MR ELDRIDGE asked if the Madison curve was to be adopted for load duration effects and whether loading was to be considered to have a cumulative effect on strength.

PROFESSOR LARSEN pointed out that the Madison curve was not relevant to load classification. He also said that it was quite impossible to accumulate loads throughout the history of a structure since their effects, if cumulative, probably also depended on the ratio of loading to relaxation times.

DR NOREN reminded the meeting that the problem of the transformation of loading history into a single loading class had been discussed in some detail at an earlier meeting.

Section 4.1.1 — 'Classification of Structural Timber' (CIB-W18/9-6-1) was presented by PROFESSOR LARSEN who explained that the system of strength classes used in Australia worked well for a large number of species and he had adopted a simplified version of that system for the present draft of the Code. He said that although only four strength classes were proposed at this time it might eventually be necessary to extend the number of classes to perhaps twelve before submitting the Code to ISO.

MR CURRY did not support the introduction of the strength class system, prefering the more usual approach of specifying stresses for individual species and grades which he said permitted better utilisation of timber.

He also considered that Table 4.1.1 was orientated too much towards European redwood and whitewood.

Nor was MR CURRY convinced that the strength class system was the most suitable. He thought that different species and grades could have different responses to moisture content and duration of load. He also questioned whether the relativities between the different material properties would be valid for machine graded timber.

It was agreed that strength classes should be retained at this time although alternative recommendations would be sought and studied. References to the UN/ECE grades in Table 4.1.1 and its footnotes should be deleted.

Section 4.2.0 DR ARMBRUSTER asked who was to exercise external control over finger jointing and glued laminated timber and what form would the controls take.

DR NOREN agreed that these were very important questions but said that the answers were outside the scope of an international code.

Section 4.3.1 DR ARMBRUSTER asked if there was evidence to support the footnote beneath Table 4.3.1 and what other lay-ups would produce equivalence with the proposed standard glulam classes.

DR ELDRIDGE asked how characteristic stress values were to be established for the varied species, grades and lay-ups of glulam, taking into account the prohibitive cost of a large test programme.

Testing was only one method of deriving characteristic strength values said PROFESSOR LARSEN. Established methods of analysis and calculation were also available.

DR ARMBRUSTER agreed to consider in depth this section on glued laminated members and how the proposed stresses might be achieved.

Section 4.4 MR SUNLEY asked Professor Larsen why strength classes had not been introduced for board materials.

PROFESSOR LARSEN replied that there was no background of strength classes for board materials and in the case of plywood the promotional agencies provided a considerable amount of design information adequately supported by test results.

Section 4.5 "Strength" is to be changed to read "strength and durability".

Section 4.7 It was agreed that "for heavy steel parts" should be deleted from paragraph 1. The format of Table 4.7 is to be revised. ",eg fire retardant," and "on unprotected steel" are to be deleted from the footnote.

Section 5	Comments are invited on the tentative values given in Table 5.1.0a. Reference to climate class 0 is to be deleted from Table 5.1.0b.
Section 5.1.1	The sub sections are to be arranged in the same order as the title.
Section 5.1.1.0	DR KUIPERS did not agree with paragraph 4. The Dutch code takes into account nail and screw holes and he pointed out that if the recommendations of Chapter 6 were adhered to the reduction in cross-sectional area due to nail or screw holes could amount to more than 15 per cent. After some discussion it was agreed that since round nails or screws separated the fibres without necessarily severing them the reductions in area need not be taken into account. However, the size of nail or screw for this relaxation should be reduced from 6 mm to 5 mm.
Section 5.1.1.1	PROFESSOR LARSEN introduced paper CIB-W18/9-6-2 "Code Rules for Tension Perpendicular to the Grain". DR BARRETT informed the delegates that the figures in Table 18 of CSA 086-1976 (Figure 2 in paper 9-6-2) were based on Douglas fir. It was possible that the constants would be different for other species. Professor Larsen's proposal for the Code was accepted. Paper CIB-W18/9-6-3 "Tension at an Angle to the Grain" was introduced by MR HEMMER. He said that the tests on which this paper was based were carried out on a relatively obscure species of Swiss fir and the constants 10 and 50 in the equation could be peculiar to that species. PROFESSOR KEENAN suggested that in producing this paper it might have been the intention of Professor Möhler to show that the Hankinson formula was inadequate and possibly unsafe. PROFESSOR EDLUND told delegates that he had tested glulam beams at an angle to the grain and had shown the Hankinson formula to be optimistic. It was agreed that for the present no recommendation should be made on stresses for tension at an angle to the grain and that Professor Möhler should be asked if there was any evidence for the validity of the formula in his paper for other species.
Section 5.1.1.3	DR BARRETT commented that some species would exhibit more significant size effects than those indicated by 5.1.1.3b but he was unable to quantify these at the meeting. Dr Barrett is to produce a paper for the next meeting on depth effect, detailing the problems and making recommendations for the Code. He asked for assistance from Mr Curry as regards European species.
Section 5.1.1.4	After consideration of two papers: CIB-W18/9-10-2 "Beams Notched at the Ends" and CIB-W18/9-10-1 "Distribution of Shear Stresses in Timber Beams" it was agreed that the sentence "Loads placed more than .... " should be deleted.

Section 5.1.1.6	Professor Larsen is to amend this section taking into account the content of paper CIB-W18/9-6-4 "Consideration of Combined Stresses for Lumber and Glued-laminated Timber".
Section 5.1.1.7	DR EGERUP asked why such a complicated expression was necessary for columns. MR BURGESS said that the expressions covered the whole range of slenderness ratios and complicated formulae were less of a problem with modern calculators. The background to this column analysis has been fully explained in earlier papers to the Commission.
Section 5.2.0	A footnote is to be inserted below Table 5.2.0 to indicate that the values given are tentative proposals.
Section 5.2.1	K_{depth} is to be related to a 200 mm depth.
Section 5.2.2	PROFESSOR LARSEN asked for written comments on this section and in particular he wished to know the views of the Canadians.
Chapter 6	Because of shortage of time Chapter 6 was not discussed.
Section 7.1.1	PROFESSOR LARSEN is to make some amendments to this section in consultation with DR KUIPERS
Section 7.3	DR EGERUP is to give further consideration to this section and will take into account joint rigidity.
Section 8.1	Paragraph 2 - delete 'locally'.
Chapter 8	PROFESSOR LARSEN suggested that within this chapter a section should be devoted to preservative and fire retardant treatments.

It was accepted that Professor Larsen should prepare a further draft of the CIB Timber Code and that this should be ready for circulation at the IUFRO meeting in Vancouver BC in August 1978. The code drafting sub-committee are to see the next draft at an early stage. After that date comments would be sought from further afield but the Code was not yet to be submitted to ISO/TC 165.

5 TRUSSED RAFTERS

Paper CIB-W18/9-14-1 "Timber Trusses - Code Related Problems" was presented by DR SWANN. The main point of the paper, he said, was that Mr Williams had shown that although there were various different national loading and design procedures the end product was always substantially the same.

PROFESSOR LARSEN did not think the problem of harmonized design could be solved as easily as might first be supposed. He saw the major obstacle to harmonization not in design procedures but in national loading codes. In answer to MR ELDRIDGE, PROFESSOR LARSEN said that to specify trussed rafters in terms of performance criteria could not help solve the problem either since the performance criteria could themselves be defined in terms of loads and stresses. A further difficulty could also be that the predominant mode of failure for say South African trusses could be different from those for European trusses since the South Africans tended to have flimsier, less stable designs.

DR BOOTH told the meeting that there must be design guidance in the code for trussed rafters in the same way that there was for beams and other members. And it was also important that members should try to influence their national loading committees and press for harmonization in loading codes.

MR CURRY agreed with Dr Booth and suggested the need for a more comprehensive paper to the Commission.

DR EGERUP agreed to form a sub-group to identify the problems with trussed rafter design and to make recommendations for the timber code. DR NOREN, DR SWANN, MR CRUBILE agreed to assist Dr Egerup who also asked for a German representative.

Paper CIB-W18/9-7-1 "The Design of Truss-Plate Joints" was referred to Dr Egerup's sub-group for trussed rafters and also to Dr Kuipers for consideration by the CIB-W18/RILEM 3-TT group.

6 PLYWOOD

DR NOREN introduced his paper "The Sampling of Plywood and the Derivation of Strength Values (Second Draft)" (CIB-W18/9-4-3). He said that the paper presented the basic principles for sampling and analysis but permitted some choice of methodology provided it was recorded. Several important parts of the paper, continued DR NOREN, were unchanged from the first draft which had been presented in Brussels.

DR BOOTH thought that in defining the sampling of a population prominence should have been given to the case for the total population. The material from a single source should then be treated as a special case.

DR BARRETT said that on many Canadian construction sites material was provided from only one source which might be from the lower end of the total strength spectrum. How could one deal with this sort of problem?

Exactly the same problem arose with solid timber, or with any other material, answered DR NOREN. It was impractical to consider every individual source of supply independently.

DR BOOTH was critical of section 6.2. He said that any method of analysis could be used to derive lower percentiles and would comply with these recommendations. Standards should be very precise and closely defined procedures. If they were allowed alternatives manufacturers would naturally select the one that produced the most favourable results.

Agreeing with Dr Booth DR BARRETT said that it was better to be precise and wrong, rather than imprecise.

The meeting agreed that the sub-committee responsible for the plywood testing standard, comprising DR NOREN, DR BOOTH, DR KUIPERS, DR WILSON, should be reconvened, together with Finnish and American representatives, to redraft the paper in a more precise format.

DR BOOTH suggested that the sub-committee should also consider the content of his paper "The Evaluation of Test Data on the Strength Properties of Plywood" (CIB-W18/9-4-2) and combine the two subjects into one standard.

It was agreed that this should be done and that the standard should be submitted to the next meeting. If it was then approved it would be published and also submitted to ISO/TC 165.

PROFESSOR LARSEN said that in addition to establishing stresses for plywood on the basis of test results there was also a need for a theoretically predictive method, particularly for American plywood since they had so many possible lay-ups, grades and sizes.

DR BOOTH replied that Dr Wilson of COFI was considering a method based on limited test work to derive veneer stresses which could then be applied to other lay-ups. He also said that it might be necessary to have larger safety factors for theoretically methods of establishing stresses.

DR BOOTH and PROFESSOR LARSEN told the meeting that they had resolved the problems concerning the shear and torsional rigidity of plywood. They were agreed that those two material properties could differ and that the test specified in the CIB/RILEM standard measured torsional stiffness. They proposed that more test work should be carried out to quantify the differences between the two properties and if those tests indicated significant differences then both properties should be included in future W18 publications.

Papers CIB-W18/9-4-1 "Shear and Torsional Rigidity of Plywood" and CIB-W18/9-4-4 "On the Use of the CIB/RILEM Plywood Plate Twisiting Test: A Progress Report" were presented as background papers and were not discussed in detail.

7 RILEM 3-TT

DR KUIPERS reported on the progress of work within the CIB-W18/RILEM 3-TT sub-group.

The joint testing standard has been published and circulated with a request for comments.

The plywood testing standard, which was formally accepted by the meeting, has also been published. Comments on this document will be considered by the sub-group and a final draft will be presented at the next full meeting of CIB-W18.

The structural sized testing standard will be published in the RILEM Journal within the next few weeks.

The annex to the joints testing standard on punched metal plate fasteners will soon be ready for publication. A final draft, taking into account comments received, will be presented at the next meeting of CIB-W18.

Dr Noren and Mr Kolb are to produce a joint proposal on the testing of nails and staples for the next meeting of the sub-group.

DR KUIPERS asked for members' opinions on whether the sub-group should continue its work. He said that he had received a letter from the RILEM secretariat sug- that RILEM 3-TT should disband by 1980 when the original time-scale planned for the group would expire. He said that if the sub-group was to disband by that date then the work planned on particleboard and fibreboard would not be started.

After some discussion MR SUNLEY summarised the views expressed by other delegates and asked Dr Kuipers to continue to lead the group concerned with the standardisation of test procedures. Not only was this work of value to the

Commission, said MR SUNLEY, but the RILEM Journal offered the opportunity for the publication of testing standards and it was also desirable to have links with those concerned with other materials.

8 OTHER BUSINESS

MR GRIMSDALE circulated for information, paper CIB-W18/9-102-1 "Svensk Byggnorm 1975 (2nd Edition); Chapter 27: Timber Structures".

DR BOOTH circulated a paper of general interest "Two Laminated Timber Arch Railway Bridges Built in Perth in 1849" (CIB-W18/9-12-2).

Consideration of three papers was deferred until the next meeting. They were:

CIB-W18/9-7-2	Staples - K Möhler
CIB-W18/9-12-1	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
CIB-W18/9-13-1	Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto, J Ehlbeck

9 NEXT MEETING

The chairman informed the meeting that DR ARMBRUSTER, on behalf of Sub Commission GLULAM, had kindly offered to act as host to the next meeting which would be held in Vienna. It was agreed that the next meeting should take place 27-30 March 1979 and would be preceded on 26 March by a meeting of the RILEM-3TT/CIB-W18 sub-group.

Topics for discussion will include:

1 CIB Timber Code (Third Draft)
2 Glulam
3 Sampling of plywood and evaluation of test results
4 Trussed Rafters
5 Fibre and Particleboards

10 PAPERS PRESENTED AT THE MEETING

CIB-W18/9-4-1	Shear and Torsional Rigidity of Plywood - H J Larsen
CIB-W18/9-4-2	The Evaluation of Test Data on the Strength Properties of Plywood - L G Booth
CIB-W18/9-4-3	The Sampling of Plywood and the Derivation of Strength Values (Second Draft) - B Noren
CIB-W18/9-4-4	On the Use of the CIB/RILEM Plywood Plate Twisting Test: a progress report - L G Booth
CIB-W18/9-6-1	Classification of Structural Timber - H J Larsen
CIB-W18/9-6-2	Code Rules for Tension Perpendicular to Grain - H J Larsen
CIB-W18/9-6-3	Tension at an Angle to the Grain - K Möhler
CIB-W18/9-6-4	Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler
CIB-W18/9-7-1	The Design of Truss-Plate Joints - F J Keenan
CIB-W18/9-7-2	Staples - K Möhler
CIB-W18/9-10-1	The Distribution of Shear Stresses in Timber Beams - F J Keenan
CIB-W18/9-10-2	Beams Notched at the Ends - K Möhler
CIB-W18/9-11-1	Climate Classes for Timber Design - F J Keenan
CIB-W18/9-12-1	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
CIB-W18/9-12-2	Two Laminated Timber Arch Railway Bridges built in Perth in 1849 - L G Booth
CIB-W18/9-13-1	Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto and J Ehlbeck
CIB-W18/9-13-2	The Structural Use of Tempered Hardboard - W W L Chan
CIB-W18/9-14-1	Timber Trusses - Code Related Problems - R F Williams
CIB-W18/9-100-1	The CIB Timber Code (Second Draft)
CIB-W18/9-102-1	Svensk Byggnorm 1975 (2nd Edition); Chapter 27: Timber Construction.

11 CURRENT LIST OF CIB-W18 TECHNICAL PAPERS

Technical papers presented to Working Commission W18 - Timber Structures are classified by a code identifying the meeting at which the paper was presented, the subject heading and the number of the paper. The full classification number of a document will start with CIB-W18, although where the context is clear this prefix may be omitted.

Example: CIB-W18/4-102-5

refers to paper 5 (Extract from Norwegian Standard NS 340 - "Timber Structures") on subject 102 (Structural Design Codes) presented at the fourth meeting of W18 (Paris, February 1975).

Published documents emanating from the Commission will simply be numbered in the order in which they appear.

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, Feb/March 1977
8 Bruxelles, Belgium, October 1977
9 Perth, Scotland; June 1978

Subjects are denoted by the following numerical classification:

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
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

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 Paper 5 Limit State Design - H J Larsen

1-1-2 Paper 6 The use of partial safety factors in the new Norwegian design code for timber structures - O Brynildsen

1-1-3 Paper 7 Swedish code revision concerning timber structures - B Norén

1-1-4 Paper 8 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

TIMBER COLUMNS

2-2-1 Paper 3 The Design of Solid Timber Columns - H J Larsen

3-2-1 Paper 6 Design of Built-up Timber Columns - H J Larsen

4-2-1 Paper 3 Tests with Centrally Loaded Timber Columns - H J Larsen and Svend Sondergaard Pedersen

4-2-2 Paper 4 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 Kälsner and B Norén

5-100-1 Design of Solid Timber Columns - H J Larsen

6-100-1 Comments on Document 5-100-1, Design of Timber Columns - H J Larsen

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 and Perry Formulas for Solid Timber Columns - H J Larsen

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

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

SYMBOLS

3-3-1 Paper 5 Symbols for Structural Timber Design - J Kuipers and B Norén

4-3-1 Paper 2 Symbols for Timber Structure Design - J Kuipers and B Norén

1 Symbols for Use in Structural Timber Design

PLYWOOD

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

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

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

4-4-4 Paper 5 Standard Methods of Testing for the Determination of Mechanical Properties of Plywood - Council of Forest Industries, BC

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 Specification - 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 In-grade versus Small Clear Testing of Plywood - C R Wilson

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 the Mechanical Properties of Plywood - L G Booth, J Kuipers, B Noren, C R Wilson

7-4-2 Comments 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 Noren

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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 Noren

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

STRESS GRADING

1-5-1 Paper 10 Quality specifications for sawn timber and precision timber - Norwegian Standard NS 3080

1-5-2 Paper 11 Specification for timber grades for structural use - British Standard BS 4978

4-5-1 Paper 10 Draft Proposal for an International Standard for Stress Grading Coniferous Sawn Softwood - ECE Timber Committee

STRESSES FOR SOLID TIMBER

4-6-1 Paper 11 Derivation of Grade Stresses for Timber in 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 the 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

TIMBER JOINTS AND FASTENERS

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4-7-1 Paper 8 Proposal for a Basic Test Method for the Evaluation of Structural Timber Joints with Mechanical Fasteners and Connectors - RILEM, 3TT Committee

4-7-2 Paper 9 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 Proposals for Testing Joints with Integral Nail Plates - 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 The Design of Truss-Plate Joints - F J Keenan

9-7-2 Staples - K Möhler

LOAD SHARING

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4-8-1 Paper 12 A Review of Load Sharing in Theory and Practice - E Levin

4-8-2 Paper 13 Load Sharing - B Norén

DURATION OF LOAD

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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 Theilgaan

TIMBER BEAMS

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4-10-2 Paper 7 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

ENVIRONMENTAL CONDITIONS

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6-11-1 Climate Grading for the Code of Practice - B Noren

9-11-1 Climate Classes for Timber Design - F J Keenan

LAMINATED MEMBERS

6-12-1 Manufacture of Glued Timber Structures - J Kuipers

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8-12-2 Instructions 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

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

9-6-4 Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler

PARTICLE AND FIBRE BUILDING BOARDS

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TRUSSED RAFTERS

4-9-1 Paper 14 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

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9-14-1 Timber Trusses - Code Related Problems - T F Williams

9-7-1 The Design of Truss-Plate Joints - F J Keenan

CIB TIMBER CODE

2-100-1 Paper 2 A Framework for the Production of an International Code of Practice for the Structural Use of Timber - W T Curry

5-100-1 Design of Solid Timber Columns - H J Larsen

5-100-2 A Draft Outline of a Code of Practice for Timber Structures - L G Booth

6-100-1 Comments on Document 5-100-1; Design of Timber Columns - H J Larsen

6-100-2 A CIB Timber Code - H J Larsen

7-100-1 CIB Timber Code Chapter 5.3 Mechanical Fasteners; CIB Timber Standard 06 and 07 - H J Larsen

8-100-1 CIB Timber Code: List of Contents (second draft) - H J Larsen

9-100-1 The CIB Timber Code (Second Draft)

LOADING CODES

4-101-1 Paper 19 Loading Regulations - Nordic Committee for Building Regulations

4-101-2 Paper 20 Comments on the Loading Regulations - Nordic Committee for Building Regulations

STRUCTURAL DESIGN CODES

1-102-1 Paper 2 Survey of status of building codes, specifications etc, in USA - E G Stern

1-102-2 Paper 3 Australian codes for use of timber in structures - R H Leicester

1-102-3 Paper 4 Contemporary Concepts for Structural Timber Codes - R H Leicester

1-102-4 Paper 9 Revision of CP 112 - First draft, July 1972 - British Standards Institution

4-102-1 Paper 15 Comparison of Codes and Safety Requirements for Timber Structures in EEC Countries - Timber Research and Development Association

4-102-2 Paper 16 Nordic Proposals for Safety Code for Structures and Loading Code for Design of Structures - O A Brynildsen

4-102-3 Paper 17 Proposals for Safety Codes for Load-Carrying Structures - Nordic Committee for Building Regulations

4-102-4 Paper 18 Comments to Proposal for Safety Codes for Load-Carrying Structures- Nordic Committee for Building Regulations

4-102-5 Paper 21 Extract from Norwegian Standard NS 3470 "Timber Structures"

4-102-6 Paper 22 Draft for Revision of CP 112 "The Structural Use of Timber" - W T Curry

8-102-1 Polish Standard PN-73/B-3150: 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

INTERNATIONAL STANDARDS ORGANISATION

3-103-1 Paper 2 Method for Preparation of Standards Concerning the Safety of Structures - published by International Standards Organisation (ISO/DIS 3250).

4-103-1 Paper 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 Consultation 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

JOINT COMMITTEE ON STRUCTURAL SAFETY

3-104-1 Paper 1 International System of Unified Standard Codes of Practice for Structures - Published by Comité Européen du Béton (CEB)

7-104-1 Volume One: Common Unified Rules for Different Types of Construction Material - CEB

CIB PROGRAMME, POLICY AND MEETINGS

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5-105-1 The Work and Objectives of CIB-W18 - Timber Structures - J G Sunley

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

7-106-1 Time and Moisture Effects - CIB W18/IUFRO S5.02-03 Working Party

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WORKING COMMISSION W18 - TIMBER STRUCTURES

TESTING METHODS FOR PLYWOOD IN STRUCTURAL GRADES FOR USE IN LOAD-BEARING STRUCTURES

by

RILEM/CIB-3TT

JUNE 1978

JOINT COMMITTEE RILEM/CIB-3TT: TESTING METHODS OF TIMBER

Testing methods for plywood in structural grades for use in load-bearing structures

The texts presented hereunder are drafts which are published in order to be submitted to comments The final text will be drawn from the draft with regard to the possible comments.

Comments be sent to:
J. Kuipers, Stevin Laboratory, Stevinweg 4, Delft, Netherlands; before September 30, 1978.

FOREWORD

The CIB working committee W. 18 "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 plywoods. Consequently, the CIB W. 18 committee asked the joint committee 3TT of RILEM/CIB W. 18 to develop acceptable test methods for structural plywoods.

The underlying recommendations were worked out by the above committee in close co-operation with the CIB Working Committee W. 18 "Timber Structures". Both committees feel that there exists an urgent need for testing methods for the determination of properties of commercial plywood 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.

The agreed recommendations will be sent to the ISO TC. 139 "Plywood", seeking co-operation on how these should be processed. It is expected that comments on these tentative recommendations will be dealt with in RILEM/CIB-3TT and that after consideration thereof the "final recommendations" will be published in Materials and Structures and passed as a final draft to ISO TC. 139.

It is hoped that in future anyone wishing to carry out tests on this type of material will follow these recommendations to permit an easier exchange of test results between research and commercial testing organisations throughout the world.

INTRODUCTION

This Recommendation, which is based on the recommendations of W. 18—Timber Structures—Commission of CIB, and 3TT—Testing methods of timber—Committee of RILEM/CIB, specifies standard methods for the determination of some physical and mechanical properties of commercial plywood containing defects permitted by the manufacturing specification.

It is known from current unfinished research programmes that the size and shape of the test piece, and the size of the defects influence the strength of the test piece, but these relationships are not yet established for all commercial plywoods. When this information is available it may be found that for some plywoods, for example softwood plywoods with large defects, larger test pieces may give higher strength values which are a better indication of the strength and stiffness of plywood when used as a full panel. In the absence of this information the sizes chosen for the test pieces in this Recommendation are the minimum desirable for the majority of plywoods used for structural purposes.

The strength values found from the test pieces described in this Recommendation may need to be modified before being used in the design of structural components. It is expected that the relationship between the strength of the test piece and that of plywood 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.

Tests of the glue lines in plywood 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 will be adequate.

1. SCOPE AND FIELD OF APPLICATION

This Recommendation specifies standard methods for determining some physical and mechanical properties of commercial plywood, which is 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, strengths, 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 schedule given in figure 1. Alternative schedules may be developed when the test pieces are cut at an angle to the grain, 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.

4.2. Measurements to be taken

The thickness of the test piece shall be measured to the nearest 0.02 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;

(1) In preparation.

b) compression; the thickness at four points, two on each edge 100 mm from each end; the width at two points 100 mm from each end;

c) tension; the thickness at four points, two on each edge 400 mm from each end; the width at two points 400 mm from each end;

d) panel shear strength; the thickness at four points, two on each edge 100 mm from each end; 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 each end; the length at two points 50 mm from each side.

4.4. Thickness of each ply

When needed for the interpretation of test results, the thickness of each ply in the test piece shall be measured to the nearest 0.02 mm at the same points at which the thickness of the test piece is measured, and the average recorded.

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 be free from visible knots, knot holes, core gaps and other voids in any ply. The strip shall have the same thickness and width as the test piece and shall have a length of 25 $\pm$ 5 mm.

The strip shall be weighed immediately after testing and then dried to a constant mass (1) in a vented oven at a temperature of 103 $\pm$ 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 significant figures.

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 6) may also be used to determine density.

(1) 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.

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 density (kg/m^3); 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 (1) and moisture content in an atmosphere of relative humidity 60 $\pm$ 2 per cent and temperature 23 $\pm$ 3°C (2).

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 plywood.

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 panel

Four test pieces shall be cut from each panel in accordance with the schedule given in figure 1.

(1) Constant mass is considered to be reached when the results of 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 test piece.

(2) The test methods specified in this Recommendation may also be used at other testing climates.

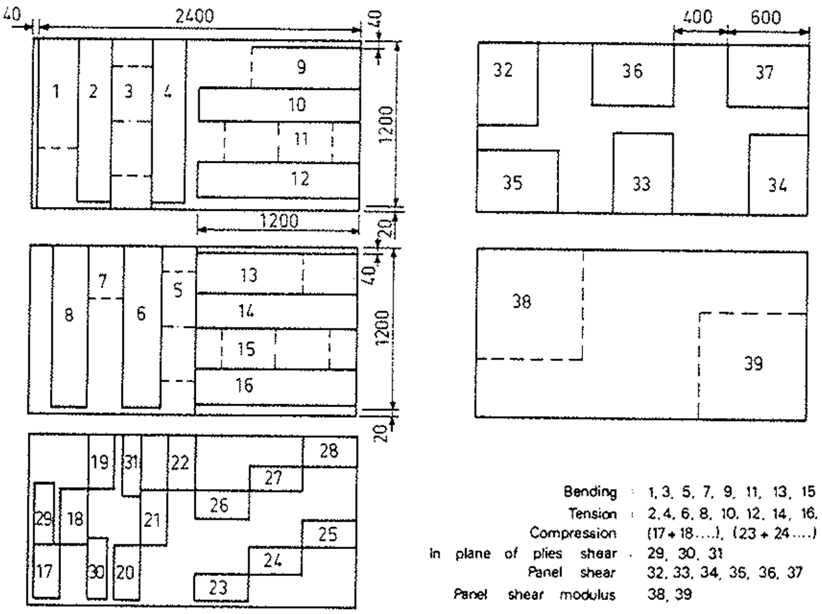

Fig. 1. — Cutting schedule.

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.

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 0,02 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.

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.

If a value for the bending modulus of elasticity is subsequently calculated from the bending stiffness, the method for specifying the second moment of area (for example full cross-sectional area, parallel plies only) shall be stated.

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 method for specifying the section modulus (for example full cross-sectional area, parallel plies only) shall be stated.

The thickness (nominal or actual) used to calculate the section modulus shall be stated.

9. COMPRESSION STRENGTH AND STIFFNESS

9.1. Test piece

The test piece shall be rectangular in cross-section.

Several pieces of the plywood to be tested shall be glued face to back until the thickness of the test piece is not less than 40 mm.

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 a panel

The test piece shall be made from each panel 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. 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.

If a value for the compression modulus of elasticity is subsequently calculated from the compression stiffness, the method of specifying the cross-sectional area (for example full cross-sectional area, parallel plies only) shall be stated.

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 method of specifying the cross-sectional area (for example full cross-sectional area, parallel plies only) shall be stated.

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 plywood.

(1) In preparation.

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.

The length of the test piece shall be 1 200 mm.

10.2. Sampling of test pieces from a panel

Four test pieces shall be cut from each panel 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 A [1].

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.

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. 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. *Acceptable 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:

$$EA_t = \Delta F\, l_1 / \Delta w,$$

(1) In preparation.

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.

If a value for the tension modulus of elasticity is subsequently calculated from the tension stiffness, the method of specifying the cross-sectional area (for example full cross-sectional area, parallel plies only) shall be stated.

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 method of specifying the cross-sectional area (for example full cross-sectional, parallel plies only) shall be stated.

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 plywood.

The width of the test piece shall not be less than 430 mm and the distance between the rails shall be 200 mm (see *fig.* 2).

The length of the test piece shall be 600 mm.

Note: For thin plywood it may be necessary to orientate the face grain perpendicular to the rails in order to preclude failure by buckling.

Timber rails having minimum dimensions of 35 mm by 115 mm by approximately 700 mm long shall be glued to both sides of the plywood test piece at each edge. The width of the rails may be increased to eliminate a shear failure between the rails and the plywood. 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. Steel rails may be used as an alternative to timber rails. Prior to gluing, the rails and the test piece shall be conditioned to the approximate

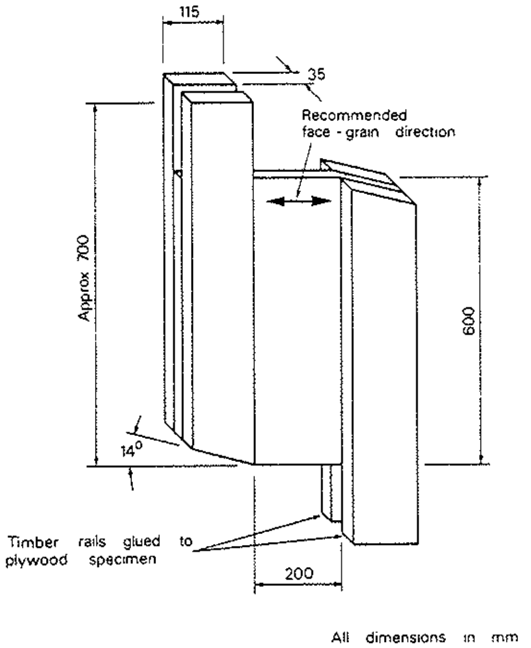

Fig. 2. — Details of panel shear test piece.

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 (¹).

11.2. Sampling of test pieces from a panel

Six test pieces shall be cut from each panel in accordance with the schedule given in figure 1.

11.3. Loading method and equipment

The load 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 crossheads.

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.

(¹) 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.

11.4. 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 ± 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.

11.4.2. *Acceptable test results*

Any test piece that fails in other than shear between the rails shall be rejected.

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/mm²); 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 (nominal or actual) used to calculate the panel shear strength shall be stated.

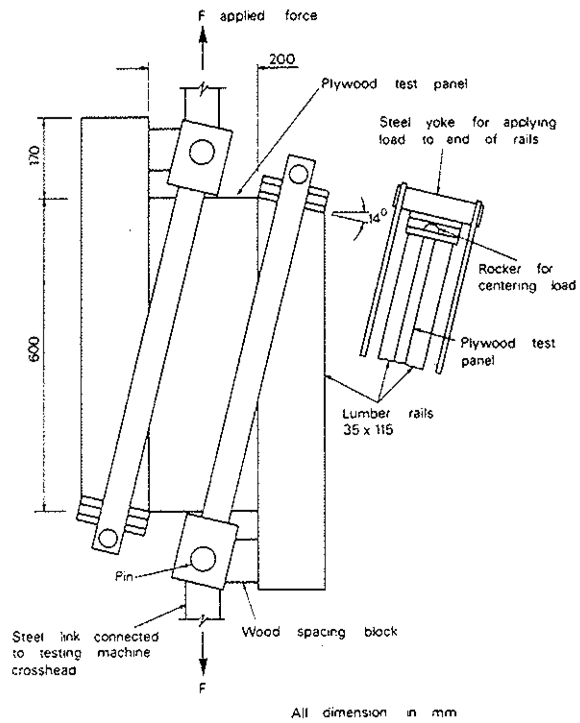

Fig. 3. — Loading apparatus for panel shear test piece.

12. PANEL 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 plywood.

The test piece shall be square and the length and width not less than 25, nor more than 40, times the thickness.

The grain direction of the individual plies shall be parallel or perpendicular to the edges of the test pieces.

Test pieces shall be reasonably flat. Any showing excessive initial curvature shall be rejected.

12.2. Sampling of test pieces from panel

Two test pieces shall be cut from each panel in accordance with the schedule given in figure 1.

12.3. Loading method and equipment

The test piece shall be supported on rounded supports, having a radius of curvature not greater than 6 mm at the opposite ends of a test piece diagonal and loaded in a similar manner on the opposite ends of the other diagonal. In order that the loads may be applied at the corners, metal plates shall first be attached as shown in figure 4. The loading and supporting frame shall be rigid. The loading equipment shall be capable of measuring the load to an accuracy of 1%.

12.4. Test procedure

The load shall be applied with a continuous and uniform motion of the movable head at a rate of 0.000 05 times the length of the specimen (mm./sec.) within a permissible variation of $\pm$ 25%.

The time taken from the beginning of the loading to the maximum load shall be measured and recorded to the nearest 30 seconds.

12.4.1. *Measurement of deflection*

The deflection shall be measured at two points on each diagonal equidistant from the centre of the test piece. These measurements shall be made at the quarter points of the diagonals. A satisfactory arrangement for measuring relative deflections is indicated in figures 4 and 5; the dial readings in this case give twice the increment of deflection relative to the centre (i. e. 2 Δw, *see* 12.5.1).

The test piece shall not be stressed beyond its proportional limit. To eliminate the effects of any slight initial curvature, two sets of data shall be obtained, the second set with the test piece rotated 90° about an axis through the centre of the test piece and perpendicular to the plane of the plies. The two results shall be averaged to obtain the shear modulus for the test piece.

The deflection shall be measured to the nearest 0.02 mm.

12.5. Expression of results

12.5.1. *Panel shear modulus*

The panel shear modulus shall be calculated from the following formula:

$$G_p = 3\,a_1^2\,\Delta F / 2\,t^3\,\Delta w,$$

where G_p, is the panel shear modulus (N/mm²); ΔF, is the increment of load applied at each corner on the straight line portion of the load-deflection curve (N); t, is the thickness of the test piece (mm); a_1, is the distance from the centre of the test piece to the point where the deflection is measured (see *fig.* 5) (mm); Δw, is the increment of deflection relative to the centre of the test piece corresponding to ΔF (mm).

The shear modulus shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the shear modulus shall be stated.

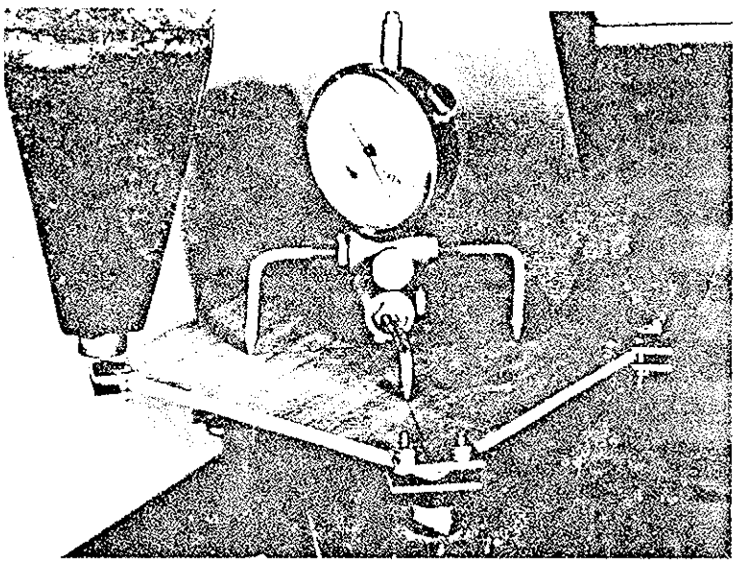

Fig. 4. — Shear modulus test of plywood showing method of loading and measuring differential deflection along the two diagonals.

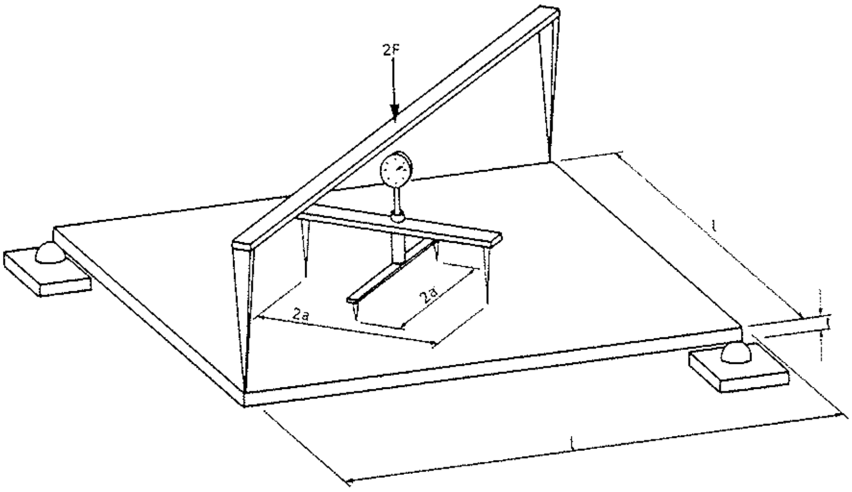

Fig. 5. — Panel shear modulus of plywood showing method of measuring deflection.

13. IN PLANE OF PLIES SHEAR STRENGTH AND MODULUS

13.1. 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 plywood.

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 plywood 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 face grain of the plywood shall be parallel to the length of the test piece.

The orientation of the lathe checks may be such that they open (see *fig.* 7 *a*) or close (see *fig.* 7 *b*) during loading. The orientatiom of the lathe checks in each veneer shall be recorded.

13.2. Sampling of test pieces from a panel

Three test pieces shall be cut from the panel in accordance with the schedule given in figure 1.

13.3. 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.

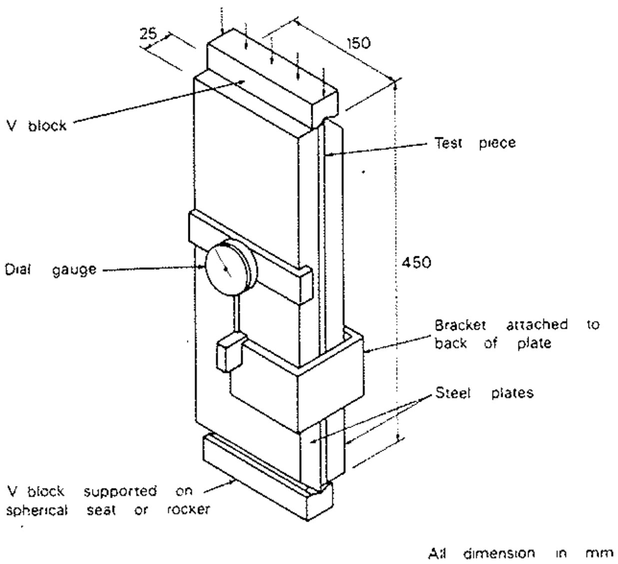

Fig. 6. — In plane of plies shear test using a dial gauge for measuring plate slip.

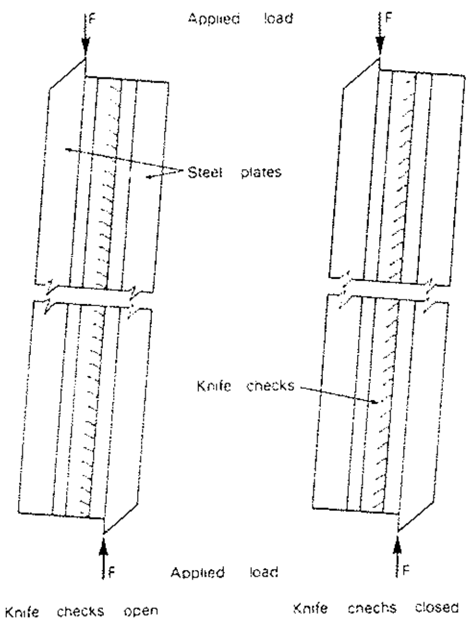

Fig. 7. — Orientation of knife checks.

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%.

13.4. Test procedure

13.4.1. *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.

13.4.2. *Measurement of deformation*

Data for load-slip curves shall be taken to determine the effective in plane of plies shear modulus.

A suitable method of measuring the slip between the steel plates is shown in figure 6.

The slip between the steel plates shall be read to the nearest 0.002 mm.

13.4.3. *Acceptable test results*

Any test piece that fails in the bond between the metal plates and the plywood shall be rejected. Failure may occur in rolling shear, or in shear parallel to the grain, or as a combination of these failure modes with the failure plane crossing a glue line. The type of failure shall be recorded.

13.5. Expression of results

13.5.1. *Effective in plane of plies shear modulus*

The effective in plane of plies shear modulus shall be calculated from the following formula:

$$G_r = \Delta F\, t/\Delta w l b,$$

where G_r, is the effective in plane of plies shear modulus (N/mm²); ΔF, is the increment of load on the straight line portion of the load-slip curve (N); Δw, is the increment of slip corresponding to ΔF (mm); t, is the thickness of the test piece (mm); l, is the length of the test piece (mm); b, is the width of test piece (mm).

The effective in plane of plies shear modulus shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the effective in plane of plies shear modulus shall be stated.

13.5.2. *In plane of plies shear strength*

The in plane of plies shear strength shall be calculated from the following formula:

$$f_r = F_{max}/lb,$$

where f_r, is the in plane of plies shear strength (N/mm²); F_{max}, is the maximum load (N); l, is the length of test piece (mm); b, is the width of test piece (mm).

The in plane of plies shear strength shall be calculated to three significant figures.

The thickness (nominal or actual) used to calculate the in plane of plies shear strength shall be stated.

14. 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 material data shall normally be given: the species and nominal thickness of each veneer, the grain direction, the adhesive, the overall thickness, the number of veneers, the surface treatment, the manufacturing standard and the grade of the panels from which the test pieces were cut.

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 temperature and relative humidity at the time of test.

For individual test pieces the following data shall be given: test piece dimensions, moisture content, time to 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 (parallel plies only, full cross-section, etc.) shall be stated. The thickness (nominal or actual) used in the calculations shall be stated.

Additional data may be required in some cases. This may include the following: full details of method of manufacture, actual thickness of each veneer, 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.

REFERENCES

ISO 3804, Plywood. *Determination of dimensions of test pieces* (draft).

ISO 4841, Plywood. *Determination of modulus of elasticity in tension and of tensile strength* (in preparation).

ISO 4842, Plywood. *Determination of modulus of elasticity in compression and of compressive strength* (draft).

ISO 4843, Plywood. *Determination of apparent modulus of elasticity in bending and of bending strength* (draft).

ISO 0000, Timber structures. Plywood. Structural grades. *Sampling of panels for the determination of some physical and mechanical properties* (in preparation).

ISO 0000, Timber structures. Plywood. Structural grades *Analysis of test data* (in preparation).

CIB-W18/9-4-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SHEAR AND TORSIONAL RIGIDITY OF PLYWOOD

by

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PERTH, SCOTLAND
JUNE 1978

A board as shown in fig. 1 with an uneven number of plies symmetrically about the middle ply is considered.

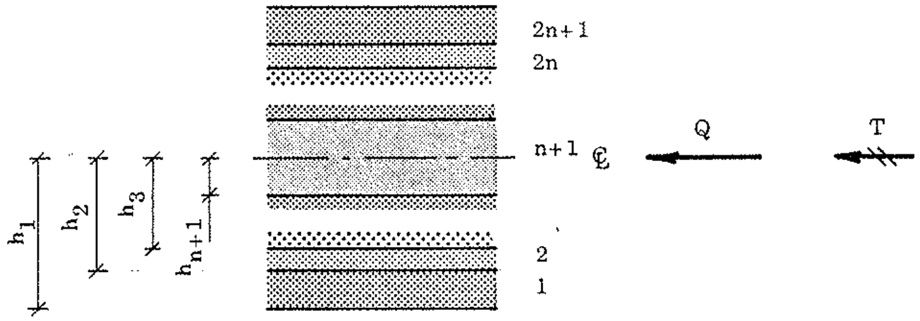

Figure 1

The i^{th} ply has the shear modulus G_i and the distance from the middle plane to the far side of the ply is h_i.

For shear in the plane the equivalent shear modulus is

$$G = (h_{n+1} G_{n+1} + \sum_{i=1}^{n} (h_i - h_{i+1}) G_i)/h_1 \tag{01}$$

The equivalent torsional modulus is*

$$G_t = (h_{n+1}^3 G_{n+1} + \sum_{i=1}^{n} (h_i^3 - h_{i+1}^3) G_i)/h_1^3 \tag{02}$$

It is evident that theoretically $G_t \neq G$.

To estimate the difference in practice a simple cross-section as shown in fig. 2 is considered. It should be noted that it is not necessarily a 3-layer plywood we are dealing with, but only a plywood composed of two different species.

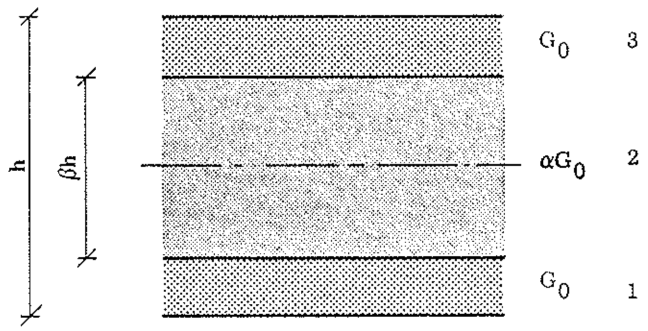

Figure 2.

$$G = (1 - \beta(1 - \alpha))G_0 \tag{03}$$

$$G_t = (1 - \beta^3 (1 - \alpha))G_0 \tag{04}$$

* Cf. S. G. Lekhnitskii: Anisotropic Plates, 1968, p. 296.

G/G_t is given in fig. 3. For realistic values of α and β the G/G_t-ratio varies between 1 and about 0.75. It does not seem reasonable to disregard such a basic error.

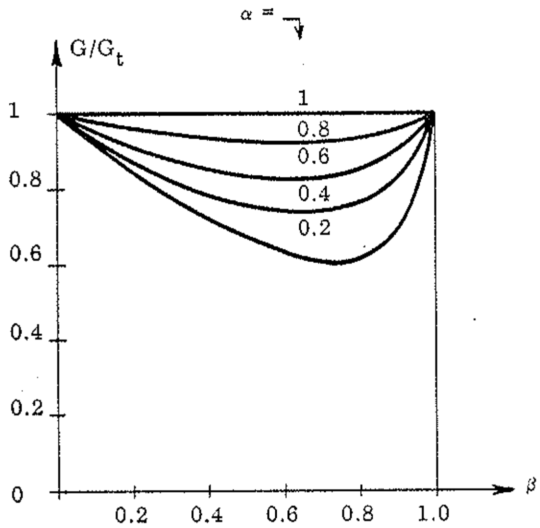

Figure 3

CIB-W18/9-4-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE EVALUATION OF TEST DATA ON THE STRENGTH PROPERTIES OF PLYWOOD

by

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PERTH, SCOTLAND
JUNE 1978

continue to exist in individual countries, but at some time we may have to make a decision on whether the strength properties should be specified in the CIB Code as parallel plies or full cross-section stresses, or alternatively as capacities. The relative merits of the different methods were considered at a previous CIB meeting.

It is convenient to consider the three methods separately. In each method the calculation of f from equation 1 depends on the method of calculating W and on the value of k: the alternatives are listed for each method.

3.1 Parallel plies

$$W = \frac{1}{6} bd^2$$

where b = nominal specimen width
or actual specimen width

d = actual thickness of parallel plies
or nominal thickness of parallel plies as specified in the production standard
or minimum thickness of parallel plies as specified in the production standard
or characteristic thickness of parallel plies as determined from the test sample

k = 0.85 for parallel to the face grain, 1.50 for 3 ply perpendicular to the face grain, 1.00 for 5 or more plies perpendicular to the face grain (eg Canada)
or 1.00 for all cases (eg Sweden)

3.2 Full cross-section

b = nominal width of specimen
or actual width of specimen

d = actual thickness of specimen
or nominal thickness of specimen as specified in the production standard
or minimum thickness of specimen as specified in the production standard
or characteristic thickness of specimen as determined from test sample

k = 1

3.3 Capacity

No calculation of f is required. If a characteristic value of M is determined from the sample then the sampling procedure should cover the variation of thickness likely for a given nominal thickness of plywood.

4. PRESENTATION OF STRESSES AND SECTION PROPERTIES IN A CODE

If stresses (either parallel plies or full cross-section) are specified in a Code subsequent calculations for design purposes of the capacity will be made from

$$M = kWf \qquad (2)$$

The components of the right hand side of equation (2) have been tabulated in different countries in four different ways:

k, W and f
or W and (kf)
or kW and f
or when k = 1 as W and f

This is a matter of choice and as with parallel plies and full cross-section stresses it is likely that individual countries will make their own choice, but at some stage a CIB choice may be required. Whichever choice is made, it is important that the information should be clearly specified.

When section moduli are published, the magnitude of W depends on the thickness used in the calculation. The chosen thickness may be

nominal thickness specified in the production standard
or minimum thickness specified in the production standard
or average thickness to be maintained to ensure that the individual panel thicknesses do not fall below the minimum thickness specified in the production standard
or characteristic thickness as determined by sampling

THE EVALUATION OF TEST DATA ON THE STRENGTH PROPERTIES OF PLYWOOD

L G BOOTH
Imperial College, London

1. INTRODUCTION

At the last meeting of CIB W18 (Brussels, October 1977) Noren's paper on "Testing of structural plywood for assigning characteristic strength values" was discussed. Noren's paper considers the problems of sampling of the test specimens and the subsequent determination of characteristic strengths. During the discussion it was noted that some aspects of the problem had not been considered and I agreed to prepare some notes for discussion at the next meeting (Perth, June 1978).

At the 5th meeting of CIB W18 (Karlsruhe, October 1975) I reported on "The determination of design stresses for plywood in the revision of CP 112". That paper briefly described a major study undertaken on behalf of the Princes Risborough Laboratory during which strength test data from Canada, Finland and Sweden were examined. It was found that there were large differences in the test specimens used in the different countries and that different procedures were used to evaluate the test data. The differences in test specimens had been discussed previously at W18 and subsequently we prepared the CIB/RILEM test standard. We must now examine the procedures for transforming the measured test parameter (eg ultimate bending moment on the specimen) into a stress (eg ultimate bending stress).

The purpose of these notes is to set out the options that are available for dealing with this second problem. If firm decisions can be made, it should be possible to prepare a draft standard on Evaluation of Test Results: this could be a separate standard or could form part of a standard on Sampling and Evaluation of Test Results.

It is not the purpose of these notes to discuss the statistical aspects of the determination of characteristic values: this problem will continue to be dealt with by Noren. We must, however, decide whether we wish to find a characteristic stress for a plywood or a characteristic capacity. This choice should influence the method of sampling since the capacity is the product of strength and sectional property, both of which may vary independently.

2. PRODUCTION STANDARDS

Prior to discussing the evaluation of the test results, mention must be made of the parts of the production standards for plywood that deal with the tolerances on the thickness of the veneers and on the thickness of the finished panels. The appropriate extracts from CSA O121 M-1978: Douglas fir plywood, and SFS 4093: Dimensions and tolerances for Finnish structural plywoods, are given in Appendix A. Similar restrictions apply to Canadian softwood plywood, Finnish birch and Finnish softwood (conifer) plywood.

It can be seen from the extracts in Appendix A that for a given nominal thickness of Douglas fir plywood, the manufacturer has considerable latitude in choosing the veneer thicknesses and that the final panel thicknesses are allowed a plus and minus tolerance.

For Finnish birch faced plywood the thicknesses of the veneers are specified in SFS 4091: the birch veneers are 1.4 mm whilst the softwood veneers may be either 1.4 or 2.2 mm. For some nominal thicknesses (eg 15 mm) two lay ups are permitted, one with 9 plies using the 2.2 mm veneers, the other with 11 plies using 1.4 mm veneers. The final panel thicknesses are allowed a plus and minus tolerance.

All these aspects should be taken into account during the sampling, the evaluation of the test results and the specification of the geometrical properties.

3. EVALUATION OF TEST RESULTS

In the following discussion the behaviour in bending is used to illustrate the problem: similar problems apply to the other strength properties and stiffnesses.

$$f = \frac{M}{kW} \qquad (1)$$

where f = bending strength
M = ultimate bending moment
k = flexure factor
W = section modulus

The calculation may be undertaken using parallel plies only (eg Canada) or using the full cross-section (eg Finland). These two methods are likely to

5. DECISIONS REQUIRED

The decisions facing CIB W18 can be divided into two groups: those which are desirable and those which are essential.

It is desirable that we decide whether to adopt the parallel plies or the full cross-section method of specifying stresses, or alternatively we may specify capacities and stiffnesses. This is desirable but not essential: the stresses and capacities are equivalents and no matter what the method of specification the design calculations will result in the same thickness of plywood being used.

It is essential that we specify the thickness of plywood (ie actual, nominal, minimum or characteristic) to be used in the analysis of the test data. We must also decide if any experimentally determined factors should be applied to the test data (eg the Canadian flexure factor). If these decisions are not made then each country may have to undertake some re-analysis of the test data instead of accepting the characteristic values calculated by the test laboratory.

It is essential that we specify the thickness of plywood (ie nominal, minimum, average or charcteristic) to be used in the calculation of section properties. If this decision is not made then each country may have to recalculate the section properties calculated by the plywood manufacturer.

APPENDIX A

The following is an extract from CSA O121 M-1978: Douglas fir plywood.

4.3 Ply Thickness

4.3.1 General. Thicknesses shall apply to the plies in a finished panel and to the moisture content (approximately 6 per cent), existing at the time of manufacture.

4.3.2 Nominal Ply Thicknesses.

4.3.2.1 The greatest nominal ply thickness for inner plies for all grades shall be 5.0 mm.

4.3.2.2 The least nominal ply thickness of all plies for unsanded production shall be 2.4 mm.

4.3.2.3 The greatest nominal ply thickness for face and back plies for unsanded grades shall be 3.2 mm except as listed below:

(a) Three ply 12 mm panels having all plies of equal nominal thickness

(b) Four ply 15 mm panels having all plies of equal nominal thickness

(c) Five ply 18 mm panels having all plies of equal nominal thickness

4.3.2.4 The minimum nominal ply thickness for face and back plies for unsanded four ply 12 mm panels and six ply 18 mm panels shall be 3.2 mm.

4.3.3 Average Ply Thickness.

4.3.3.1 The average thickness of all nominal 5.0 mm inner plies for all grades shall be not greater than 5.0 mm

4.3.3.2 The average thickness of all nominal 2.4 mm plies for unsanded grades shall be not less than 2.4 mm

4.3.3.3 The average thickness of all nominal 3.2 mm face and back plies for unsanded grades shall be not greater than 3.2 mm

4.4.3 Permissible Size Tolerances.

4.4.3.1 Thickness. The maximum permissible panel thickness tolerances, as measured in accordance with Clause 4.4.2.1, shall be as follows:

(a) Regular Sanded Grades: ± 0.5 mm

(b) Cleaned and Sized Select Grades: - 1.0, + 1.0 mm

(c) Regular Unsanded and Overlaid Grades up to and including 20.5 mm: - 0.5, + 1.0 mm

(d) Regular Unsanded and Overlaid Grades over 20.5 mm: - 0.5 mm, + 8 per cent of panel thicknesses

The following is an extract from SFS 4093: Dimensions and tolerances for Finnish structural plywoods.

Table 2: Standard thicknesses of birch faced plywood

Nominal Thickness mm	6,5	9	12	15	18	21	22	23	25	26

Table 8: Tolerances for birch faced plywood

Thickness tolerance		
Thickness mm	Unsanded mm	Sanded mm
4	± 0,3	± 0,3
6,5	± 0,4	± 0,4
9 - 23	± 0,5	± 0,5
> 23	± 3%	± 3%

CIB-W18/9-4-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE SAMPLING OF PLYWOOD AND THE DERIVATION OF STRENGTH VALUES (Second Draft)

by

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PERTH, SCOTLAND

JUNE 1978

2.3 The period during which the plywood referred to as a population is produced should be as long as possible without involving such changes in the production which can be expected to have a significant influence on the properties to be established by the testing.

2.4 A population consisting of plywood from several factories may be substituted by plywood from a limited number of factories, if it is proved beyond doubt that this will not increase the estimated characteristic strength. On similar conditions a population consisting of plywood of several constructions (thicknesses) may be substituted by a population consisting of a limited number of selected constructions.

If one characteristic strength value shall be evaluated for a plywood of standard type, construction and thickness, made at for example 150 factories, one may find out from a limited number of testing (compared with what is stipulated in p. 3.1), either that the population must be divided into a number of populations, c.f. 2.1, or - if the difference between factories is comparatively small - that the strength values can be evaluated from a number of those factories that are at the lower end with respect to the strength of their produced plywood.

3 Sampling of panels

3.1 The number of panels in a sample, drawn for testing from a population or substitute population, defined in p. 2, must allow each strength property to be tested on specimens from at least N panels of each construction (thickness) and thereby from at least n panels from each factory. The values of N and n should be at least:

	N	n
Establishing characteristic strength	60	10
Assigning to strength grade	30	5

Note 1 It is recommended that sampling for approval is made at at least two occations, each with $N \geqq 30$ ($n \geqq 5$). A provisional approval can then be based on $N = 30$ ($n = 5$).

3.2 Samples shall be drawn at random over the production time defined for the population (p. 2.3). When the number of panels from one factory (production line) for testing one specific strength property is n, these panels must be drawn from n different batches produced on different days.

4 Sampling of specimens from panels

4.1 A specific schedule shall be used for the cutting of test specimens from panels. This schedule shall define the distance between the specimens and, as a rule, the position of the specimens relatively to the edges of the panel. If a characteristic feature (such as a joint) occurs on a regular distance from the edges of the panels, the position of the cutting schedule relatively to the edges shall be changed at random from panel to panel.

4.2 When the size of the cutting schedule is larger than the panel, the schedule may be applied on two or more adjacent panels in the batch.

The number of specimens and cutting schedules are generally given in testing standards. (For structural plywood see ISO/TC 165 N, document 14E.)

5 Definition of characteristic value

5.1 For characteristic values of strength or moduli of elasticity (rigidity etc.) the 1-, 5-, 10- or 50-percentile is used. As a rule the 5-percentile is used for the strength and moduli of elasticity for calculating strength (verification of limit state of failure), while the 50-percentile is used for calculation of deformation at the serviceability limit state.

6 Derivation of characteristic values of strength and stiffness

6.1 When characteristic strength values are estimated for a population of plywood panels (p. 2) from test results from samples of a limited number of specimens (p. 3), the credibility of the results should be duly considered. This is achieved by applying an increased confidence.

THE SAMPLING OF PLYWOOD AND THE DERIVATION OF STRENGTH VALUES

B Norén - Swedish Forest Products Research Laboratory, Stockholm

0. Introduction
1. Purpose
2. Definition of the population
3. Sampling of panels from the population
4. Sampling of test specimens
5. Definition of characteristic values
6. Derivation of characteristic values
7. Compliance with minimum characteristic value of grade

0 Introduction

At the CIB - W18 meeting in Brussels October 1977 the author presented a paper (8-4-1) "Sampling Plywood and the Evaluation of Test Results", including an Appendix "Testing of Structural Plywood for Quality Control". This is a second draft in which the comments according to the record of the Brussels meeting has been considered. The sections 2, 3 and 4 are essentially the same as in the first draft, sections 5, 6 and 7 are simplified. The Appendix is omitted.

1 Purpose

The purpose of the sampling and testing may be either to establish characteristic strength and stiffness values for an approval as structural plywood (6) or to check for compliance with a strength grade for an approval or for quality control.

2 Definition of the population

2.1 In defining the population of plywood and in choosing sampling method that conditions in production, marking and end-use shall be considered. The population shall be limited thus that the strength deviation at end-use in principally due to random variations. Hence, the population shall be unambigously specified with respect to type (species) and grade (reference to a product standard), thickness and construction (lay-up) and - possibly - source (factory) and production time.

2.2 Integrating panels of different thickness and construction or from different sources into a mixed population is permitted either if it is proved that there is no significant deviation of characteristic strength values between sub-groups or if these are mixed in a random way when the plywood is used.

Plywood strength is sometimes expressed by a single veneer stress value to be applied on parallel plies. If the approximation of this model is accepted at different constructions (thicknesses), a population of mixed constructions could be considered in assigning characteristic strength values for design and possibly in sampling for continous quality control testing. For the kind of testing, here dealt with, plywood of different constructions (thicknesses) should, however, as a rule be considered as belonging to different populations.

Sometimes the product standard (In particular the specifications for grading the veneers) do not guarantee that the plywood produced at different factories will have the same strength, for example due to different timber sources. In such cases the population may have to be specified with respect to source.

With "increased confidence" is here meant that the method of estimation should imply a probability higher than 0.5 that the estimated characteristic value is lower than the real value.

6.2 The characteristic value may be derived from the test results using any recognized statistic method, either a non-parametric or one based on a reliable distribution function. As a rule the characteristic value may be estimated as

$$m_k = \bar{m} \exp(-k \frac{s}{\bar{m}}) \qquad (6:1)$$

and if $k \frac{s}{\bar{m}} \leqq 0.25$ as

$$m_k = \bar{m} - ks \qquad (6:2)$$

In (6:1) and (6:2) m_k denotes the characteristic value estimated for the population, $\bar{m}$ is the mean value and s the standard deviation for the sample. The value of the coefficient k depends on the required probability that m_k is not overestimating the characteristic value of the population, on the percentile used to define this characteristic value and on the number of individual values (N) on which $\bar{m}$ and s are based.

If the real 5-percentile of a population is M_k and a probability 0.75 (75% confidence) is required, that m_k calculated from a sample by (6:2) is lower than M_k, the k-values given in table 6.1 may be used.

Table 6.1 Value of k in (6:2) for estimating characteristic strength (5-percentile) from N strength values in a sample[1)]

N =	15	20	30	60	100	$\geq$ 200
k =	1.99	1.93	1.87	1.80	1.75	1.70

1) For other percentiles and confidence claims, see for example Bowker, Lieberman: Engineering statistics. Prentice Hall 1961.

7 Compliance with minimum characteristic value of grade

A population (p. 2) can be considered in grade if a value, calculated from test results from a sample of limited number of panels (p. 3) is at least equal to the characteristic value m_k, required for the grade:

$$\bar{m}\,\exp(-k\,\frac{s}{\bar{m}}) \geqq m_k \qquad (7:1)$$

or if $k\,\frac{s}{m} \leqq 0.25$

$$\bar{m} - ks \geqq m_k \qquad (7:2)$$

(Denotations see p. 6.2)

The value of k depends on the required confidence of the compliance checking and on the number of panels tested in the sample. If not otherwise stipulated, the value of k may be determined from the non-central t-distribution, table 7.1.

Table 7.1 Value of k in (7:2) for assigning to grade[2)]

N =	15	20	30	60	100
k =	1.88	1.80	1.71	1.60	1.54

2) If the coefficient of variance in the population is known, lower k-values may be applied.

CIB-W18/9-4-4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ON THE USE OF THE CIB/RILEM PLYWOOD PLATE TWISTING TEST: A PROGRESS REPORT

by

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PERTH, SCOTLAND
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RILEM/CIB 3TT

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On the use of the CIB/RILEM plywood plate twisting test: a progress report.

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1. Introduction

During recent meetings of CIB-W18 and RILEM-3TT the use of the plate twisting test as a method of determining the panel shear modulus of plywood has been questioned. The purpose of this progress report is to summarize the relevant theory for generally and specially orthotropic plates and to apply it to plywood laid up in the most general way with veneers of different species and thickness. For stage two of the project it is hoped that confirmatory experimental work will be undertaken before the next meeting. The final report will also contain the full references and discuss alternative methods of test.

The application of the plate twisting test as a method of determining the panel (in-plane) shear modulus of plywood was first proposed by March, Kuenzi and Kommers of the US Forest Products Laboratory. The original report places no restrictions on its use apart from specifying that the face grain should be aligned with the edges of the plate. The paper and the subsequent ASTM did not restrict its use to plywood laid up with only one species, possibly on the grounds that mixed species were rarely used at that time for structural plywood in North America.

In recent years mixed species plywood has become the norm in North America and the plate twisting test has continued to be used. The four-rail shear test is considered to give a uniform shear distribution and the most accurate panel shear modulus. Recently this test has generally been superseded by the two-rail test, but it is acknowledged that this method does not give a uniform shear distribution: a shear modulus measured from it requires a modification factor which depends on the lay up of the plywood.

It was for these reasons that the plate twisting test was included in the CIB/RILEM standard as the only recommended method for determining the panel shear modulus.

The method has been criticised at recent meetings by Hans Larsen on the grounds that it measures the torsional shear stiffness and not the in-plane shear stiffness. This progress report determines the circumstances under which the test does measure the in-plane shear stiffness, and determines the theoretical inaccuracies in those cases when the method is invalid.

2. Theory

The elastic behaviour of an anisotropic material is given by the following strain-stress relations

$$\begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \epsilon_3 \\ \gamma_{23} \\ \gamma_{31} \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & S_{14} & S_{15} & S_{16} \\ S_{12} & S_{22} & S_{23} & S_{24} & S_{25} & S_{26} \\ S_{13} & S_{23} & S_{33} & S_{34} & S_{35} & S_{36} \\ S_{14} & S_{24} & S_{34} & S_{44} & S_{45} & S_{46} \\ S_{15} & S_{25} & S_{35} & S_{45} & S_{55} & S_{56} \\ S_{16} & S_{26} & S_{36} & S_{46} & S_{56} & S_{66} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \\ \tau_{23} \\ \tau_{31} \\ \tau_{12} \end{Bmatrix} \quad (1)$$

In the case of timber there are three orthogonal planes of material property symmetry. The strain-stress relations in co-ordinates aligned with the principal material directions (ie the L, T, R directions) now become

$$\begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \epsilon_3 \\ \gamma_{23} \\ \gamma_{31} \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & 0 & 0 & 0 \\ S_{12} & S_{22} & S_{23} & 0 & 0 & 0 \\ S_{13} & S_{23} & S_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & S_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & S_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & S_{66} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \\ \tau_{23} \\ \tau_{31} \\ \tau_{12} \end{Bmatrix} \quad (2)$$

If we now consider a rotary cut veneer in the 1-2 (ie LT) plane, equation (2) reduces to

$$\begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & 0 \\ S_{12} & S_{22} & 0 \\ 0 & 0 & S_{66} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \quad (3)$$

The compliances S_{11}, S_{12} and S_{66} can be written in terms of the engineering constants (Young's moduli and Poisson's ratios) as follows:

$$S_{11} = 1/E_1 \; ; \; S_{12} = -\nu_{21}/E_2 = -\nu_{12}/E_1 \; ; \; S_{66} = 1/G_{12} \tag{4}$$

In equation (3) the principal material directions (ie L, T) of the veneer were aligned with the co-ordinate axes (x, y). A more general case is when the principal material directions are not aligned with the co-ordinate axes: in this case the strain-stress relation, previously defined by equation (3), takes the form

$$\begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \bar{S}_{11} & \bar{S}_{12} & \bar{S}_{16} \\ \bar{S}_{12} & \bar{S}_{22} & \bar{S}_{26} \\ \bar{S}_{16} & \bar{S}_{26} & \bar{S}_{66} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \tag{5}$$

where

$$\begin{aligned}
\bar{S}_{11} &= S_{11}\cos^4\theta + (2S_{12} + S_{66})\sin^2\theta\cos^2\theta + S_{22}\sin^4\theta \\
\bar{S}_{12} &= S_{12}(\sin^4\theta + \cos^4\theta) + (S_{11} + S_{22} - S_{66})\sin^2\theta\cos^2\theta \\
\bar{S}_{22} &= S_{11}\sin^4\theta + (2S_{12} + S_{66})\sin^2\theta\cos^2\theta + S_{22}\cos^4\theta \\
\bar{S}_{16} &= (2S_{11} - 2S_{12} - S_{66})\sin\theta\cos^3\theta - (2S_{22} - 2S_{12} - S_{66})\sin^3\theta\cos\theta \\
\bar{S}_{26} &= (2S_{11} - 2S_{12} - S_{66})\sin^3\theta\cos\theta - (2S_{22} - 2S_{12} - S_{66})\sin\theta\cos^3\theta \\
\bar{S}_{66} &= 2(2S_{11} + 2S_{22} - 4S_{12} - S_{66})\sin^2\theta\cos^2\theta + S_{66}(\sin^4\theta + \cos^4\theta)
\end{aligned} \tag{6}$$

where θ is the angle between the x axis and the 1 - axis measured in a counter-clockwise direction.

If a veneer is considered as an orthotropic plate and is subjected to a uniform twisting moment (for example as prescribed in clause 13 of the RILEM/CIB plywood test standard), the deflection w measured along the z axis is given by

$$t^3 w = 6M_{xy}(\bar{S}_{61}x^2 + \bar{S}_{62}y^2 + \bar{S}_{66}xy) \tag{7}$$

where t = veneer thickness; M_{xy} = twisting moment.

Using the applied loads, the co-ordinate axes and the measured deflections defined in the RILEM/CIB test we have

$$t^3w = 3Fa_1^2(\overline{S}_{61} + \overline{S}_{62} + \overline{S}_{66})/2 \qquad (8)$$

Hence for a generally orthotropic plate (ie when the principal material directions are not aligned with the co-ordinate axes) the measured deflection gives the sum of the three elastic constants, $\overline{S}_{61}$, $\overline{S}_{62}$ and $\overline{S}_{66}$.

For a specially orthotropic plate (ie when the principal material directions are aligned with the co-ordinate axes) we have $\theta = 0^o$. Hence from equations (6) and (4), $\overline{S}_{16} = \overline{S}_{26} = 0$ and $\overline{S}_{66} = S_{66} = 1/G_{12}$. Hence equation (8) becomes

$$t^3w = 3Fa_1^2S_{66}/2 = 3Fa_1^2/2G_{12} \qquad (9)$$

Hence the CIB/RILEM test, when performed on an individual veneer, measures the in-plane shear modulus (G_{12}) of the veneer.

We must now consider the case of a plywood plate composed of veneers which in the most general case are of different species and different thickness. Although plywood is occasionally made with the grain of adjacent veneers at angles other than 90^o to each other, the majority of commercial plywood has adjacent veneers at right angles to each other: only the latter case will be considered in the following analysis.

The CIB/RILEM test specifies that the face grain direction of the plywood shall be parallel with the sides of the test specimen and hence we need only consider the analysis of a specially orthotropic plate.

For each veneer within the plate, equation (3) is applicable. If the veneers are of different species, then the magnitudes of the elastic constants (S) will be different for each veneer.

Equation (3) may be rewritten in the inverse form

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} = \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ Q_{12} & Q_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix} \begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \gamma_{12} \end{Bmatrix} . \qquad (10)$$

where Q is the stiffness matrix and $Q_{66} = 1/S_{66} = G_{12}$.

Using the Kirchhoff hypothesis for plates, the strain and stress variations within the plate are given by

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix}_k = \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ Q_{12} & \bar{Q}_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix}_k \left\{ \begin{Bmatrix} \epsilon_x^0 \\ \epsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} + z \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \right\} \tag{11}$$

where σ = stress in k^{th} veneer at height z; ε^o = middle surface strain; κ = middle surface curvature.

Given the stress variation we can now determine the resultant forces and moments on the plate using equations of the form

$$N_x = \int_{-t/2}^{t/2} \sigma_x \, dz \qquad\qquad M_x = \int_{-t/2}^{t/2} \sigma_x z dz \tag{12}$$

Hence

$$\begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} = \sum_{k=1}^{N} \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ \bar{Q}_{12} & Q_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix}_k \left\{ \int_{z_{k-1}}^{z_k} \begin{Bmatrix} \epsilon_x^0 \\ \epsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} dz + \int_{z_{k-1}}^{z_k} \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} zdz \right\}$$

$$= \begin{bmatrix} A_{11} & A_{12} & 0 \\ A_{12} & A_{22} & 0 \\ 0 & 0 & A_{66} \end{bmatrix} \begin{Bmatrix} \epsilon_x^0 \\ \epsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} + \begin{bmatrix} B_{11} & B_{12} & 0 \\ B_{12} & B_{22} & 0 \\ 0 & 0 & B_{66} \end{bmatrix} \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \tag{13}$$

and

$$\begin{Bmatrix} M_x \\ M_y \\ M_{xy} \end{Bmatrix} = \begin{bmatrix} B_{11} & B_{12} & 0 \\ B_{12} & B_{22} & 0 \\ 0 & 0 & B_{66} \end{bmatrix} \begin{Bmatrix} \epsilon_x^0 \\ \epsilon_y^0 \\ \epsilon_{xy}^0 \end{Bmatrix} + \begin{bmatrix} D_{11} & D_{12} & 0 \\ D_{12} & D_{22} & 0 \\ 0 & 0 & D_{66} \end{bmatrix} \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \tag{14}$$

where $$A_{ij} = \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z_k - z_{k-1})$$

$$B_{ij} = \frac{1}{2} \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z_k^2 - z_{k-1}^2) \tag{15}$$

$$D_{ij} = \frac{1}{3} \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z_k^3 - z_{k-1}^3)$$

and N is the number of veneers.

In equation (15), A is the extensional stiffness, B the coupling stiffness and D the bending stiffness.

If the plate consists of veneers that are symmetrically arranged about the mid depth, then $B = 0$.

The in-plane shear stiffness (A_{66}) and the torsional stiffness (D_{66}) can be found by putting $(Q_{66})_k = 1/(S_{66})_k = (G_{12})_k$ in equation (15). Hence

$$
\begin{aligned}
A_{66} &= \Sigma(G_{12})_k(z_k - z_{k-1}) \\
D_{66} &= \tfrac{1}{3}\Sigma(G_{12})_k(z_k^3 - z_{k-1}^3)
\end{aligned}
\tag{16}
$$

where $(G_{12})_k$ is the in-plane shear modulus of the k^{th} veneer and the summation is taken over all the veneers.

From equation (9) we saw that w measures S_{66} and hence for the plywood plate D_{66}.

Hence we may conclude that, in general, the plate twisting test does not measure the in-plane shear stiffness and the corresponding (effective) in-plane shear modulus.

3. A numerical comparison of the in-plane shear and torsional stiffnesses of some commercial plywoods

We saw in section 2 that the plate twisting test measures the torsional stiffness of the plywood and that, in general, we cannot use this value to calculate an in-plane shear modulus unless we know the lay up and shear moduli of the individual veneers.

Equations (16) can be used to compare D_{66} and A_{66} for different lay ups of plywood. Three cases will be considered:

birch plywood with all veneers of the same species, but not necessarily of the same thickness;

Douglas fir plywood with face veneers of Douglas fir and centres and cross bands of other softwood species;

Finnish Combi plywood with face veneers of birch and a core of alternating birch and spruce veneers.

3.1 Birch plywood

In equation (16), $(G_{12})_k$ = constant = G_{12}. Hence

$$A_{66} = tG_{12} \qquad D_{66} = t^3G_{12}/12$$

$$A_{66}/D_{66} = 12/t^2$$

Hence in this case the plate twisting test measures both A_{66} and D_{66}.

3.2 Douglas fir plywood

Let the face veneers have a modulus of rigidity G_F and the centres and cross bands G_c. Let the thickness of each face veneer be t_F and the total thickness of the remainder be t_c. Hence

$$A_{66} = G_F(2t_F) + G_c(t_c) = G_F(a_F) + G_c(a_c)$$

$$D_{66} = \frac{1}{12} G_F(t^3 - t_c^3) + \frac{1}{12} G_c(t_c^3) = G_F(d_F) + G_c(d_c)$$

$$A_{66}/D_{66} = (a_F + ga_c)/(d_F + gd_c)$$

where $g = G_c/G_F$ and a, d depend on the geometry of the lay up.

There is little information available on the shear moduli of Douglas fir and the species permitted in the core. If it is assumed that the shear moduli are proportional to the moduli of elasticity in bending, then the value of g will usually be in the range 0.8 to 0.95 (g could be greater than 1 if the centres and cross bands are composed entirely of Western larch).

The ratios $\left(\frac{t^2}{12}\right)\left(\frac{A_{66}}{D_{66}}\right)$ have been calculated in Table 1 for some imperial lay ups currently approved by Council of Forest Industries. These calculations were undertaken immediately after the last CIB/RILEM meeting (October 1977): since that date a new set of metric lay ups has been proposed but final details are not yet known. It is, however, unlikely that the ratios in Table 1 will alter significantly.

Table 1 shows that the ratios of the in-plane to torsional values lie in the range 0.920 to 0.987 for values of g between 0.8 and 0.95.

3.3 Finnish Combi plywood

In the absence of values of the shear moduli of birch and spruce it has been assumed that they are proportional to the bending moduli and g has been taken as 0.75.

The ratios $\left(\frac{t}{12}\right)\left(\frac{A_{66}}{D_{66}}\right)$ have been calculated in Table 2 for some lay ups formerly specified by Finnish Plywood Development Association. As with the Canadian plywood it is understood that new lay ups are proposed.

Table 2 shows that the ratios of the in-plane to torsional values lie in the range 0.940 to 0.966 for g = 0.75

4. Decisions required

From the theory outlined in section 2, and from the calculations described in section 3, it is clear that the CIB/RILEM plywood test standard requires an amendment to clause 13, which deals with the panel shear modulus.

We must first answer question 4.1 .

4.1 (a) Do we require both in-plane shear and torsional stiffnesses and/or effective moduli for design purposes?

(b) If yes; should the test standard give methods (or a method) for both?

(c) If no; which property should be determined?

Having answered question 4.1, there are several possible amendments to the test standard. Questions 4.2, 4.3 and 4.4 give some alternatives.

4.2 (a) Should we restrict clause 13 to torsional properties, and

(b) determine in-plane properties from

(i) an existing test (eg two-rail), or
(ii) a previous test (eg four-rail), or
(iii) a new test (eg tube, sandwich cross-beam)?

or,

4.3 (a) Should we use clause 13 for torsional properties, and

(b) modify the text to include theoretical modification factors so that in-plane properties can also be determined from the torsional properties?

or,

4.4 (a) Should we determine the in-plane properties from

(i) an existing test, or
(ii) a previous test, or
(iii) a new test, and

(b) include theoretical modification factors so that the torsional properties can also be determined from the in-plane properties?

Table 1. Douglas fir plywood.
Ratio of in-plane shear stiffness (A_{66}) to torsional stiffness (D_{66}) for $g = G_F/G_c$, where G_F = shear modulus of face veneers and G_c = shear modulus of centres and cross bands.

Thickness t in	Number of veneers	$\frac{t^2}{12}\frac{A_{66}}{D_{66}}$ g = 0.95	g = 0.90	g = 0.80
5/16	3	0.985	0.970	0.940
3/8	3	0.982	0.964	0.927
3/8	3	0.987	0.974	0.947
1/2	3	0.983	0.966	0.931
1/2	3	0.987	0.974	0.949
1/2	4	0.981	0.961	0.920
1/2	4	0.981	0.963	0.925
1/2	5	0.981	0.961	0.920
5/8	5	0.981	0.962	0.922
5/8	5	0.981	0.961	0.920
11/16	5	0.982	0.964	0.924
11/16	5	0.981	0.961	0.920
11/16	7	0.982	0.964	0.925
3/4	5	0.983	0.965	0.927
3/4	5	0.981	0.962	0.921
3/4	6	0.983	0.965	0.927
3/4	6	0.981	0.962	0.921
3/4	7	0.983	0.965	0.927
3/4	7	0.981	0.962	0.921

Table 2. Finnish Combi plywood.
Ratio of in-plane shear stiffness (A_{66}) to torsional stiffness (D_{66}) for $g = G_S/G_B = 0.75$, where G_S = shear modulus of spruce and G_B = shear modulus of birch.

Thickness t mm	Number of veneers	$\frac{t^2}{12} \frac{A_{66}}{D_{66}}$ $g = 0.75$
6.5	5	0.949
9	7	0.947
12	9	0.949
15	9	0.940
15	11	0.953
18	11	0.946
18	13	0.957
21	13	0.952
21	15	0.960
24	15	0.956
24	17	0.963
27	17	0.960
27	19	0.966

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CLASSIFICATION OF STRUCTURAL TIMBER

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INTRODUCTION

Where structural timber is concerned, it is really quite adequate to define the term »characteristic values» in the drawing-up of a timber code, and to state which testing methods, including sampling methods, are to be used to determine them. In order to make the code operational it is of course necessary to draw up grading rules and tables of the characteristic strengths which the tests have shown them to secure, but this task can be left to other organs.

The question as to whether the CIB-Timber Code should go further than absolutely necessary arises partly from the fact that the code would certainly be more concrete, if the properties of the materials were known, and partly from consideration of the contribution which would thus be made to the international harmonization which is the basic reason for the work being done.

To avoid misunderstandings it must immediately be pointed out that it is out of the question that CIB-W18 should attempt to draw up grading rules - it completely lacks the authority required for these to be accepted in practice. It is simply suggested that it should lay down certain limits.

There is no doubt that the most effective utilization of timber is achieved when individual strength and stiffness parameters for bending, tension, shear etx. are determined for each species and grade, but in practice this is only practicable for the most dominant timber species and grades, except in countries or regions where only a few are used. In the other cases it is necessary to combine timber species and grades in groups to which properties corresponding to the poorest in the group are then ascribed. This is made necessary partly because many timber species are grown and sold together - unaided, it is often impossible to tell the difference, when they have been sawn - and partly because a stable and reasonably clear delivery programme is necessary for design and construction work.

Although, as previously stated, individual treatment gives the most effective utilization of the individual timber species, grouping will probably often result in a better utilization of a region's resources, because the customer will otherwise concentrate his consumption on just a few in order to be sure of obtaining the required quality.

THE AUSTRALIAN SYSTEM

The Australian system, which is mentioned in [1], [2] and [3]*, which include further references, can be used as an example of a classification method used in practice.

The timber species are first separated into strength groups according to density, bending strengths, modulus of elasticity and shear strength for small clear samples. The grouping depends upon whether the timber is graded and built-in in green or dry condition.**

The grouping is carried out in such a way that the ratio between the bending strengths in two adjacent groups is app. 1.25 ($=\sqrt[10]{10}$), it being assumed that the grading rules are drawn up so that there are three

* [1] Leicester, R. H. & Kloot, N. H.: Evaluation of the characteristics of timber structures and structural elements. Draft. January 1977.

[2] Kloot, H.: The strength group and stress grade systems. CSIRO Forest Products Newsletter, No. 394, Melbourne 1973.

[3] Pearson, R. G.: The establishment of working stresses for groups of species. CSIRO, Division of Forest Products. Technological Paper No. 35, Melbourne 1965.

** To the uninitiated it is surprising to learn that the conditions present during a short construction period can have a decisive effect upon the allowable stresses during the entire lifetime of the structure.

CONSEIL INTERNATIONAL DU BÂTIMENT ★ INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

grades* with the same ratio (i.e. 1.25) between bending strengths for each species. This gives in all 12 stress grades denoted F4 to F34 as shown in table 1. The number gives the allowable bending stress, f_b in MPa, rounded off, see table 2. They are derived from the characteristic short-term values by multiplying by 9/16 (time factor), and dividing by 1.25 (safety factor). Also given in table 2 are the tensile strength f_t, compression strength f_c, shear strength f_v and modulus of elasticity E in relation to f_b. The compression strength perpendicular to the grain and the strength of the connections are dependent only upon the strength group, not upon the stress grades.

Table 1

	strength group						
	S1	S2	S3	S4	S5	S6	S7
0.75 grade	F27	F22	F17	F14	F11	F8	F7
0.60 grade	F22	F17	E14	F11	F8	F7	F5
0.48 grade	F17	F14	F11	F8	F7	F5	F4

Table 2. Values in MPa

stress grade F	charact. f_b	allow. f_b	f_t/f_b	f_c/f_b	f_v/f_b	E/f_b 1)
34	77	34.5			0.071	280
27	61	27.5			0.075	305
22	49	22.0			0.077	325
17	38	17.0			0.085	370
14	31	14.0			0.089	400
11	24.5	11.0			0.095	430
8	19	8.6	0.80	0.75	0.100	475
7	15.5	6.9			0.104	515
5	12	5.5			0.112	565
4	9.5	4.3			0.120	640
3	7.5	3.4			0.126	690
2	6	2.8			0.129	725

1) For f_b the characteristic value is used.

In the above it is assumed that the same ratios between the essential parameters apply for all qualities of a particular grade. This is not the case in practice, which gives rise to a number of additional rules, e.g. in certain cases a grade is grouped in a higher class than the bending strength entitles if the modulus of elasticity is two classes higher.

UTILIZATION IN CIB TIMBER CODE

In order to explore the possibility of using a classification system in the CIB Timber Code, the members were asked to complete a questionnaire. A resumé of the replies is given in table 3.

The values for the European countries concern closely related timber species, somewhat corresponding to the Australian strength groups 4 and 5.

The credibility of the figures is modest. Even the best-founded parameter, from a testing point of view, namely the bending strength, must be regarded with reservation. As an example, f_b is the same for the German class I and the Finnish T40, although stricter grading rules apply to the latter (and it is doubtful whether even they can secure the assumed strengths).

* In certain cases there can be 5 grades, namely 0.375 and 0.30 grade in addition to those given in table 1.

Table 3. Characteristic values for typical grades corresponding to short-term testing (5-10 min.) and climate class interior (ω < 15-18%)

country	bending f_b	tension par. f_{t0}/f_b	compr. par. f_{c0}/f_b	bending E/f_b	bending E/f_c	shear f_v	tension perp. f_{t90}	compr. perp. f_{c90}
Austria								
gutes B	34 1)	0.85	0.85	300	330	3.0		6
Canada								
Dougl. F, Sel.	32	0.60	0.75	390	540	1.3		7
Cott. W, 3	7	0.60	0.75	1000	1320	0.8		1.5
Denmark								
T30	31	0.85	0.85	290	340	3.0		6
U/K	21	0.40	0.70	333	470	2.4		6
Fed.Rep.Germ.								
I	39	0.80	0.85	260	300	2.7		>4
III	21	2)	0.85	480	560 3)	2.7		>4
Finland								
T40	39	0.85	0.80	270	340	3.6		9
T20	21	0.70	1.00	330	330	3.0		6
Netherlands								
constr.	26	0.90	0.95	420	440	3		6
stand.	20	0.70	1.0	500	500	3		6
Nordic draft								
T30	30	0.65	0.95	330	320	3.0		6
T18	18	0.45	0.95	390	410	3.0		6
Norway								
T30M	37	0.75	0.75	320	430			
20	20	0.70	1.0	440	440			
Poland								
III	30 1)	0.75	1.0	310	310	3.0 1)		6
South Africa								
MG	13	0.50	1.25	580	470	1.3		5
10	22	0.80	1.40	570	410	2.2		9
4	9	0.55	1.35	680	500	0.9		3.5
United Kingdom								
SS	22	0.70	0.70	330	470	2.7		2
GS	16	0.70	0.70	400	580	2.7		2

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CODE RULES FOR TENSION PERPENDICULAR TO GRAIN

by

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JUNE 1978

CODE RULES FOR TENSION PERPENDICULAR TO GRAIN

H. J. LARSEN

At the CIB-meeting in Aalborg in 1976 it was agreed that the code rules for tension perpendicular to grain should take into account the size effect and besides, that the work of the Canadian code committee with converting the theoretical considerations [1], [2], into practical code rules should be awaited.

In the following comments on the rules now available in [3] will be given:

For glulam of Douglas Fir a permissible tensile strength perpendicular to grain, $f_{t,90}$, is given generally as 65 psi (0.45 MPa). On the background of the investigations in [1] and [2] this value is surprisingly high. According to these investigations the characteristic strength (5-percentile) should be 267 psi, and if the usual Canadian factor of $1.6 \cdot 1.3 = 2.1$ (1.6 for long-term effect and 1.3 as safety factor) is taken into account the permissible stress at uniform stress distribution should be

$$f_{t,90} \sim \frac{130}{V^{0.2}} = 65\,\frac{2}{V^{0.2}} = 65 \cdot k_{size}$$

where V is the loaded volume in inch³.

Thus, the 65 psi correspond to a volume of about 32 inch³. Considering that it has been found in [4] that the time effect is probably in this case much greater than assumed the value corresponds to a quite inferior volume of perhaps 5-10 inch³.

Tension perpendicular to grain is normally only significant in curved beams or in tapered, curved or straight beams. For these beams the Canadian code states that the maximum tensile stresses, σ_t, perpendicular to grain should not exceed $Kf_{t,90}$, where K is given in fig. 2. The notations used are shown in fig. 1. The stresses, σ_t, are calculated from the expressions, among others given in [5].

The factors in fig. 1 are in principle derived in [2]. Considering the approximations made by the derivations and considering the coarse grading in »uniformly distributed load» and »all other loading» it is perhaps a little excessive to have 3 significant figures in the factor $f_{t,90}$ which is roughly determined. And furthermore, to obtain a smooth transition from double tapered with $d_a/R > 0$ to $d_a/R = 0$ it should have been stated that the smaller of the relevant factors applies.

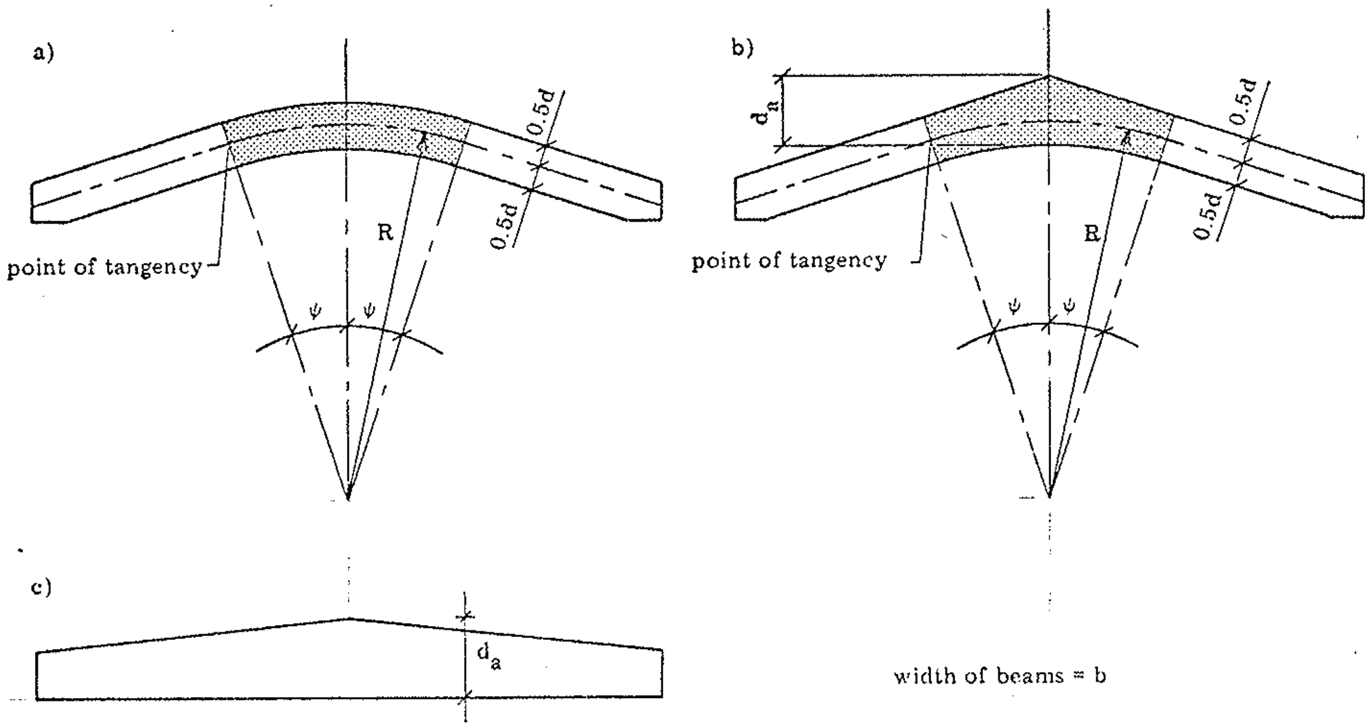

Fig. 1. a) Curved beam with constant section. b) Double tapered curved beam. c) Double tapered beam with flat soffit.

CODE FOR THE ENGINEERING DESIGN OF WOOD

TABLE 18

SIZE FACTORS K_s

Type of Member	Uniformly Distributed Load	All Other Loading
a) Curved d constant $d/R>0$	$\frac{6.74}{(dbR\psi)^{0.2}}$	$\frac{5.52}{(dbR\psi)^{0.2}}$
b) Double Tapered, Curved $d_a/R>0$	$\frac{9.74}{(bd_aR\psi)^{0.2}}$	$\frac{6.13}{(bd_aR\psi)^{0.2}}$
c) Double Tapered $d_a/R=0$ (flat soffit)	$\frac{5.24}{(bd_a^2)^{0.2}}$	$\frac{3.30}{(bd_a^2)^{0.2}}$

NOTE: *where* b = *width of cross-section, inches*
R = *radius of curvature of centreline of member, inches*
ψ = *enclosed angle between midspan and tangent point, degrees*
d_a = *depth of cross-section apex, inches*

Fig. 2

In the table ψ is assumed in degrees. With the conversion into radians, i.e. the factor $(\frac{180}{\pi})^{0.2}$, taken out of the bracket in the denominator, $(dbR\psi)$ is equal to half of the volume corresponding to the hatched area in fig. 1 a). In the case of b) the ratio between the volume corresponding to half of the hatched area and $(dbR\psi)$ for $0.05 \leqslant d/R \leqslant 0.40$ and $\psi < 0.4$ ($\sim 25°$) is between 1 and 0.55. If a ratio of 0.75 is assumed the error made due to the power of 0.2 will be no more than 6%.

If, instead of $(dbR\psi)$, the volumes are introduced in m^3 corresponding to the hatched areas the size factor, k_{size}, given in table 1 is found in the cases of a) and b).

Table 1. Size factor k_{size}

type of member	uniformly distributed load	all other loading
a) curved d constant $d/R > 0$	$\frac{0.38}{V^{0.2}}$	$\frac{0.31}{V^{0.2}}$
b) double tapered★ curved $d_a/R > 0$	$\frac{0.52}{V^{0.2}}$	$\frac{0.33}{V^{0.2}}$
c) double tapered flat soffit $d_a/R = 0$	$\frac{0.52}{(0.6 \cdot bd_a^2)^{0.2}}$	$\frac{0.33}{(0.6 \cdot bd_a^2)^{0.2}}$

★V is the volume in m^3 corresponding to the hatched areas of figs. 1 a) and 1 b), i.e. the areas between the points of tangency. In case b), V should not be assumed less than $0.6 \cdot bd_a^2$.

In case c) the following expression is found for uniformly distributed load by conversion into metres

$$k_{size} = \frac{5.24}{(39.4^3)^{0.2}(bd_a^2)^{0.2}} = \frac{0.578}{(bd_a^2)^{0.2}} = \frac{0.52}{(0.6 \cdot bd_a^2)^{0.2}}$$

The latter form has been chosen because it gives directly the transition between the cases b) and c). The volumes corresponding to $k_{size} = 1$ are between $2.86 \cdot 10^{-3}$ m^3 (~ 175 $inch^3$) and $38 \cdot 10^{-3}$ m^3 (~ 2300 $inch^3$), i.e. greater volumes than corresponding to uniform stress distribution.

Since the CIB-code gives a characteristic value, $f_{t,90}$, of 0.3 MPa corresponding to a Canadian permissible value of 0.3/1.6 MPa (the Canadian rules have a reduced safety factor since the cracking of the beams is considered a serviceability limit state) for wood species which for tension perpendicular to grain are at least not inferior to Douglas Fir, where 0.45 MPa is assumed, the values for k_{size} can be multiplied by $(0.45 \cdot 1.6/0.3)^5 = 80$. Thus, for uniform tensile stress distribution $k_{size} = 1$ corresponds to $80 \cdot 32$ $inch^3 \sim \frac{1}{25}$ m^3.

REFERENCES

[1] Barrett, J. D.: Effect of size on tension perpendicular-to-grain strength of Douglas-Fir. Wood and fibre, Vol. 6, No. 2, 1974, pp. 126-143.

[2] Barrett, J. D., Foschi, R. O. & Fox, S. P.: Perpendicular-to-grain strength of Douglas Fir. International Union of Forestry Research Organizations, Wood Engineering Group, Delft 1975.

[3] Canadian Standards Association, CSA Standard 086-1976: Code for the Engineering Design of Wood.

[4] Madsen, Borg: Duration of load tests for wood in tension perpendicular to grain. The University of British Columbia, Structural Research Series, Report No. 7, 1972.

[5] Draft 1977 of CIB-Timber Code.

CIB-W18/9-6-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TENSION AT AN ANGLE TO THE GRAIN
(concerning section 5.1.1.1 of CIB Timber Code)

by

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PERTH, SCOTLAND
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Tension at an Angle to the Grain
(concerning section 5.1.1.1)

Basing on tests performed by Baumann at test specimens of the "Gotthardttanne", Stüssi specified in 1945 in the "Schweizerische Bauzeitung" the following formula for tension at an angle to the grain:

$$\sigma_\varphi = \sigma_\parallel \cdot \frac{\cos^2\varphi}{\sqrt{1+50\cdot\sin^2\varphi}} + \sigma_\perp \cdot \frac{\sin^2\varphi}{\sqrt{1+10\cdot\cos^2\varphi}}$$

Regarding to the diagram given the strength decreases rapidly with an increasing angle. Setting in the values of SGL 38 (f_{t0} = 15 MPa, f_{t90} = 0,3 MPa) into the formula given, you receive a deminuation to 61,3% of f_{t0} at an angle of 10°. At angles of 20° or 30° there are already deminuations to 33,8° or 20,6°.

As the received values of "50" and "10" in this formula are only true for a special kind of wood out of a limited growing area, further tests with European whitewood should be performed.

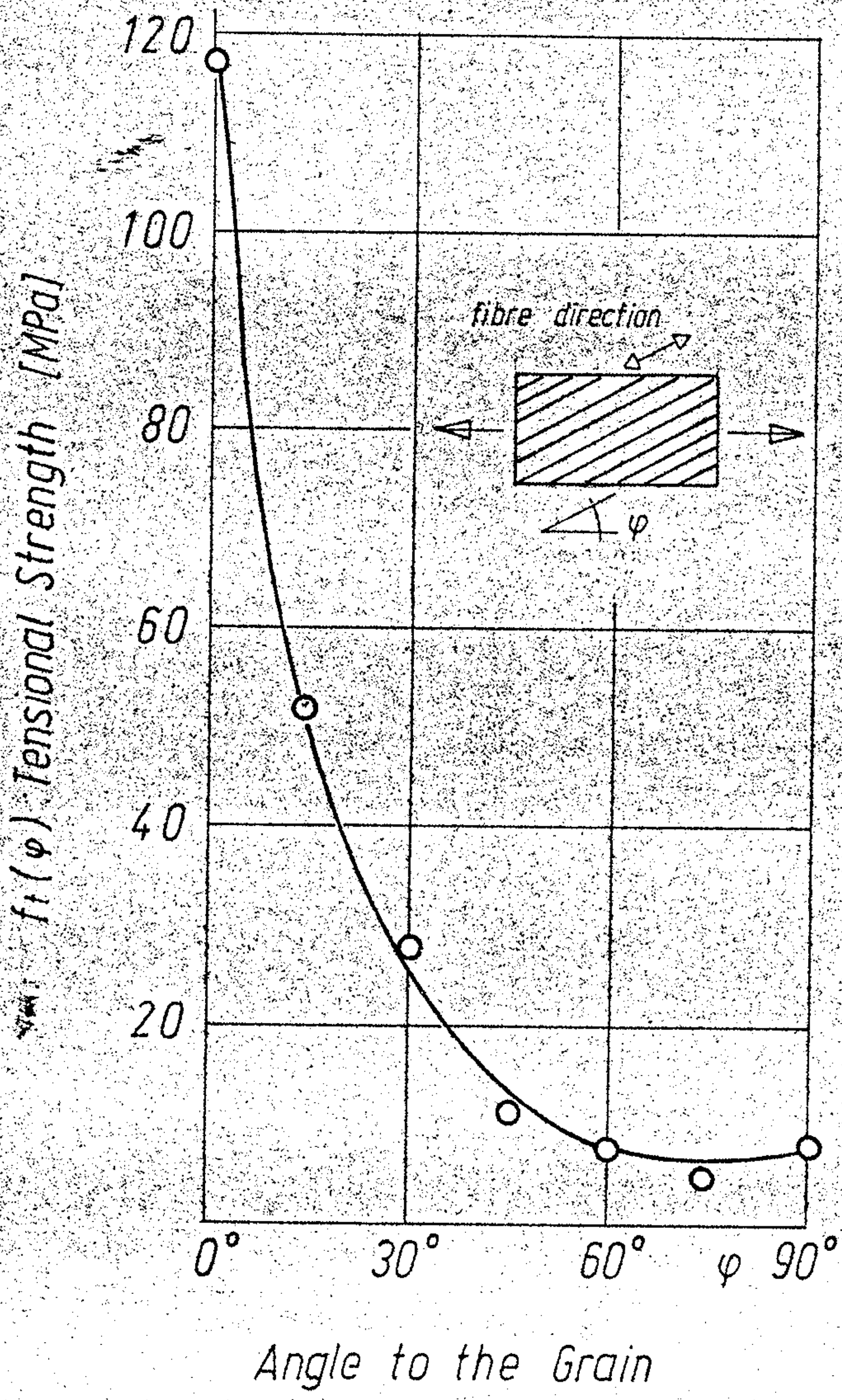

diagram concerning section 5.1.1.1
[wood material: Gotthardtanne]

CIB-W18/9-6-4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CONSIDERATION OF COMBINED STRESSES FOR LUMBER AND GLUED LAMINATED TIMBER

by

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JUNE 1978

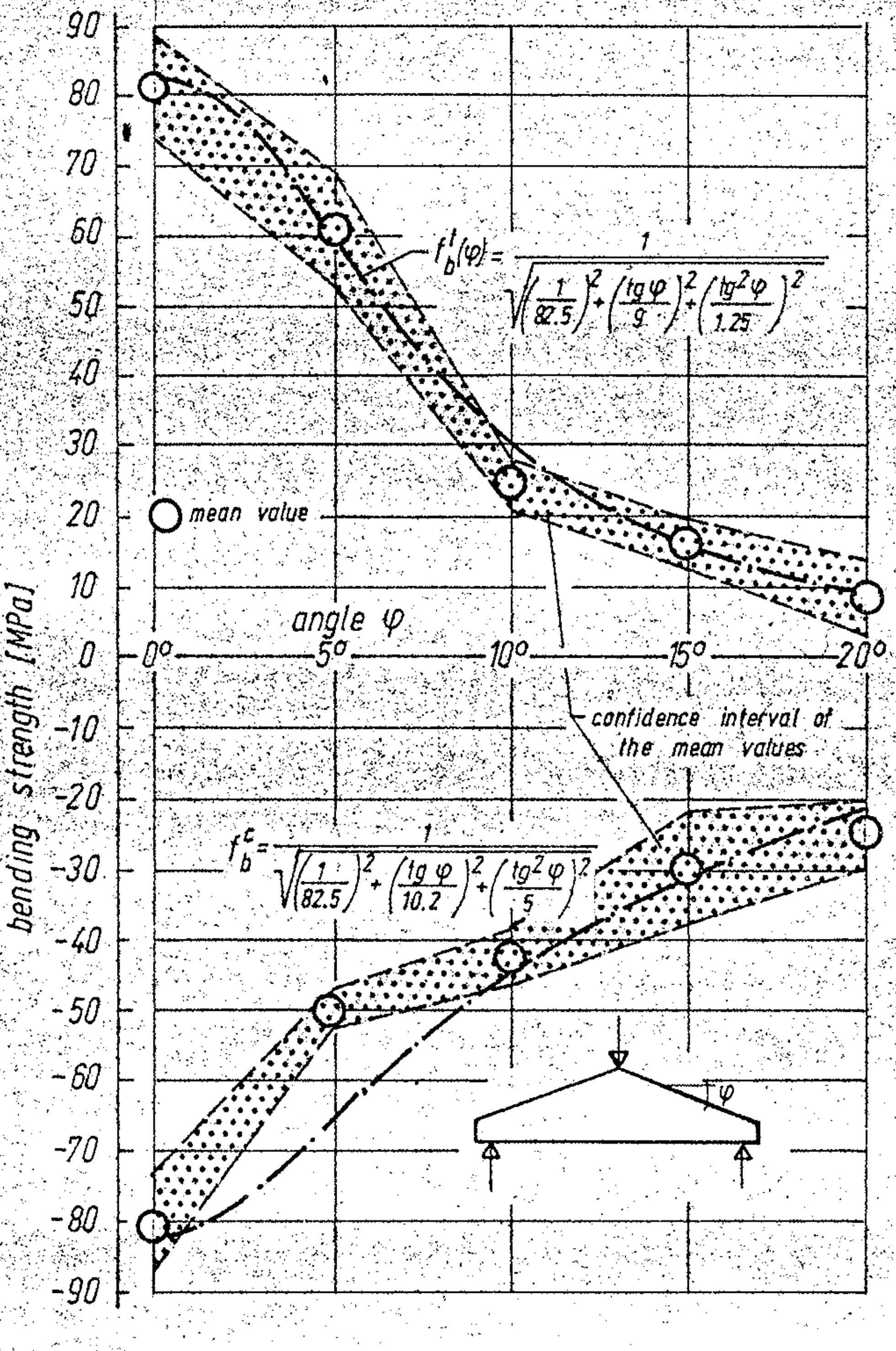

Figure 1: Relation between the bending strength and the angle φ [faultless lumber]

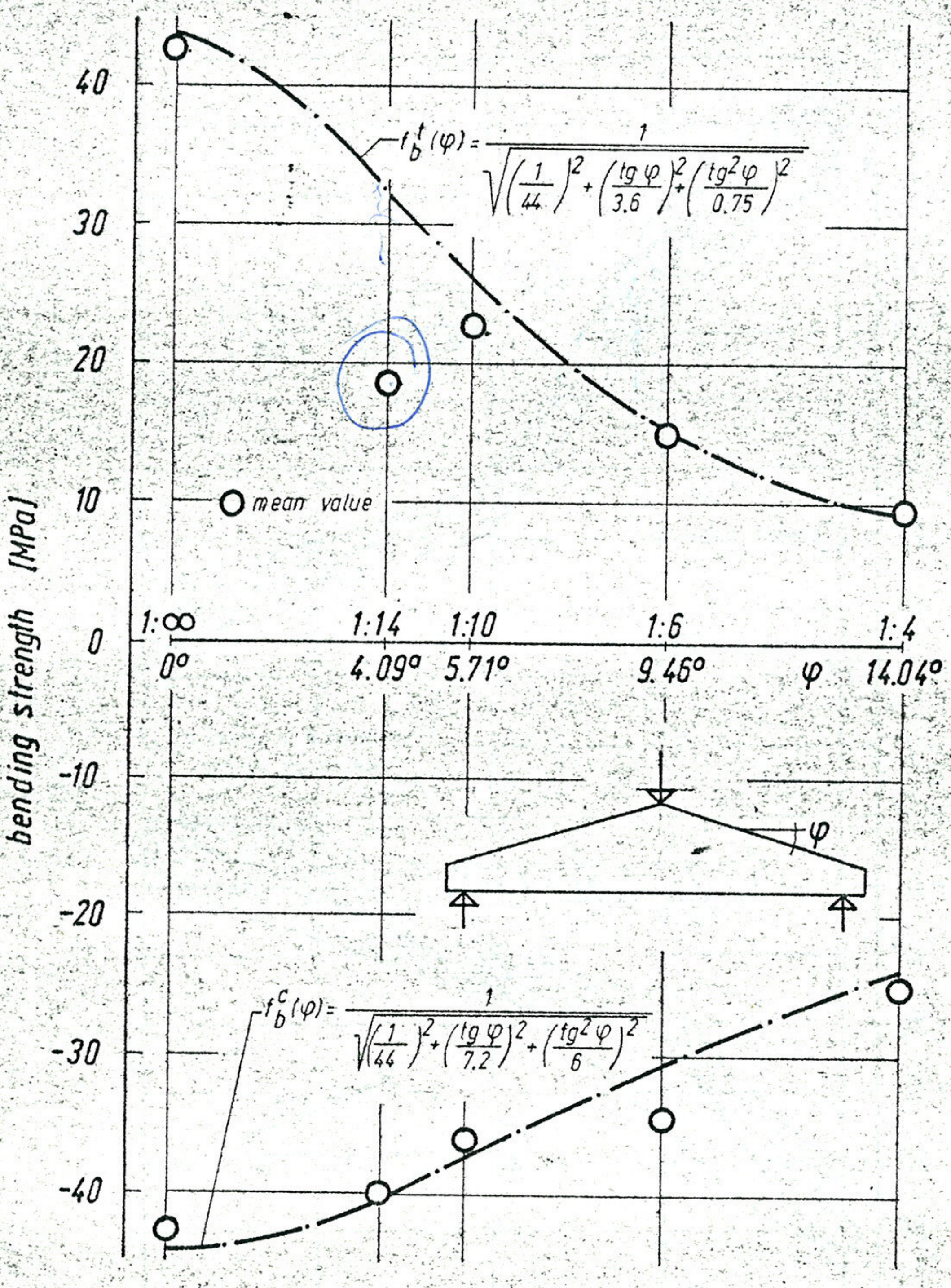

Figure 2: Relation between the bending strength and the angle φ [glued laminated timber]

1. Preface

If several stresses are acting simultaneously at the same place of a wooden beam rupture will occur at stresses, which are rather inferior than the strength of wood in the fibre direction, perpendicular to the fibredirection or than the shear strength. In consequence of some cases of loss and because the German design standard DIN 1052 doesn't give any rules for those cases tests with spruce lumber and glued laminated timber were performed, to determine values for a calculation at which the bearing resistance of a constructionalpart under combined stresses could be guaranteed.

2. Mathematical Treatment of the Problem of Combined Stresses for the Anistropic Building Material Wood

Studying the present literature and considering the shapes of the ruptures which occured at some pre-tests in consequence of stress-combinations it was reasonable to analyze the test results with the method of Norris (1). In this publication on which the formula given in the US-standard is basing the acting shear stress and the stresses in and perpendicular to the fibre direction are used for the combined stress calculation. Opposite to this proposal for example in Switzerland a method is in discussion to use the tension or compression stresses at the angle of the slope of the borders against the fibre direction at tapered beams for the calculations. As the ruptures didn't occur in planes perpendicular to this stress, it seems better to disist from an analysis in this direction.

3. Results of the Tests

At the performed investigations three types of beams were tested:

1) beams of faultless European whitewood lumber
2) beams of whitewood lumber with knots, cracks caused by seasoning, and with deviations of the fibre-direction
3) glued laminated timber of European whitewood of grade II of the DIN 1052/1.

The test series with the faultless tapered timber beams encolsed 74 beams tested in bending with an angle of the edge of the beams of 0^o, 5^o, 10^o, 15^o and 20^o. The mean-values of each series obtained are plotted in figure 1. Evaluating the test results with the formulas

$$\sigma_b \| = M/W$$
$$\tau = \sigma_b \| \cdot tg\varphi$$
$$\sigma \perp = \sigma_b \| \cdot tg^2\varphi$$

for areas with tension in fibre direction and tension perpendicular to the grain the following formula can be assumed:

$$f_b^t(\varphi) = \frac{1}{\sqrt{(\frac{1}{82,5})^2 + (\frac{tg\,\varphi}{9,0})^2 + (\frac{tg^2\varphi}{1,25})^2}}$$

For areas with compression in fibre direction and compression perpendicular to the grain the formula

$$f_b^c(\varphi) = \frac{1}{\sqrt{(\frac{1}{82,5})^2 + (\frac{tg\,\varphi}{10,2})^2 + (\frac{tg^2\varphi}{5,0})^2}}$$

can be used.

At the test series with European whitewood lumber with faults consisting of 25 test specimens, a minuation of the bending strength according to an increasing angle of the beams edges could be evaluated. As the deviations of the test results from the mean values were rather large a significant analysis could not be realized.

The test series with 26 beams of glued laminated whitewood included test specimens with an inclination of the edges of 1:∞ (≙ 0°), 1:14 (≙ 4,09°), 1:10 (≙ 5,71°), 1:6 (≙ 9,46°), and 1:4 (≙ 14,04°). The mean values of the single series are plotted in figure 2. Applying the formula of Norris to the test results the following relations can be obtained; tension acting perpendicular and parallel to grain:

$$f_b^t\,(\varphi) = \frac{1}{\sqrt{\left(\frac{1}{44}\right)^2 + \left(\frac{tg\,\varphi}{3,6}\right)^2 + \left(\frac{tg^2\varphi}{0,75}\right)^2}}$$

compression acting perpendicular and parallel to grain:

$$f_b^c\,(\varphi) = \frac{1}{\sqrt{\left(\frac{1}{44}\right)^2 + \left(\frac{tg\,\varphi}{7,2}\right)^2 + \left(\frac{tg^2\,\varphi}{6}\right)^2}}$$

The wood used had the following properties tested with compression specimens taken from the test beams: while the faultless whitewood lumber obtained a mean compression strength of 44,9 MPa at a moisture content of 12,2 % and at a specific gravity of 0,48 g/cm^3, glued laminated timber beams received values of 44,7 MPa at a moisture content of 9,3 % and at a density of 0,45 g/cm^3.

4. Conclusions

As these investigations showed, the interaction of shear stresses with stresses parallel and perpendicular to the grain at wooden beams must be taken into account, as in such cases rupture will occur at stresses remarkably below the strenth values. The formula of Norris can be recommended.

Literature

(1) Norris, C.B.: Strength of orthotropic materials subjected to combined stresses.
Forest Products Laboratory, Report No. 1816

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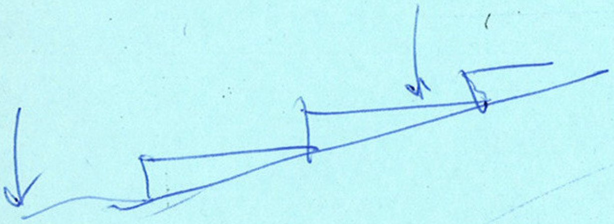

DESIGN OF TRUSS PLATE JOINTS

by

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INTRODUCTION

The purpose of this note is to inform the members of CIB-W18 of the provisions of the Canadian Standards Association Standard 086-1976, 'Code for the Engineering Design of Wood', pertaining to the testing and design of truss plate joints.

Because of the large number of proprietary truss plate designs, CSA 086 does not list allowable loads, but rather, provides a method for determining the allowable loads which is based on tests conducted according to an approved procedure.

The next two pages of this note are CSA 086's Section 9.7, 'Design of Truss Plate Joints'; this section tells how to derive allowable loads from the results of tests conducted according to CSA Standard S347-1976, 'Method of Test for Evaluation of Truss Plates used in Lumber Joints'.

CSA S347 was originally developed as a voluntary industry standard by all the member companies of the Truss Plate Institute of Canada. After several drafts and years of experience in using it, it was put forward to CSA and, after lengthy consideration and some modification, was approved as CSA S347. All truss plate designs used in Canada have been tested according to this standard, and allowable loads derived therefrom are accepted nationally by Canadian building officials. To date, it has performed well in providing consistent levels of safety and economy in truss plate design.

Section 9.7 'Design of Truss Plate Joints" of CSA 086-1976, 'Code for the Engineering Design of Wood'.

9.7 Design of Truss Plate Joints

9.7.1 General

9.7.1.1 Joint design shall be based on joints with truss plates placed on opposing faces in such a way that, at each joint, the plates on opposing faces are identical and are placed directly opposite each other.

9.7.1.2 In cases where nail-on plates are used, the word "nail" should be read in place of "tooth".

9.7.1.3 Design criteria for truss plates are based on the following conditions:

(a) The plate is prevented from deforming during installation;

(b) The teeth are normal to the surface of the lumber; and

(c) Tooth penetration in joints is not less than that used in the tests referred to in Clause 9.7.3.1.1.

9.7.1.4 Thickness of members used in joints shall be not less than twice the tooth penetration.

9.7.2 Load Capacity of Plate

9.7.2.1 Each metal plate connector shall be of sufficient size to transfer the required load without exceeding the allowable load per tooth, per rosette, or per unit area, consideration being given to the species, the orientation of teeth relative to the load, and the direction of load relative to grain.

9.7.2.2 Each metal plate connector shall have sufficient load capacity in tension, consideration being given to the orientation of the plate relative to the direction of load

9.7.2.3 Truss plates shall not be considered to be effective in transferring compression load at a joint.

9.7.2.4 For joints subjected to shear, each metal plate connector shall have sufficient shear capacity, consideration being given to the orientation of the plate relative to all possible lines of shear.

9.7.3 Determination of Allowable Loads

9.7.3.1 Lateral Resistance

9.7.3.1.1 Unit values of lateral resistance for truss plates loaded parallel and perpendicular to grain and parallel and perpendicular to the teeth or primary axis of the plate shall be determined by tests carried out in accordance with CSA Standard S347, Method of Test for Evaluation of Truss Plates Used in Lumber Joints.

9.7.3.1.2 For design in accordance with this Standard, the allowable value of lateral resistance of teeth for normal duration of load shall be the lesser of:

(a) Average unit value at wood-to-wood slip of 0.030 inch divided by 1.6;

(b) The average ultimate unit value divided by 3.0.

9.7.3.1.3 The unit values of lateral resistance of teeth shall be expressed as per tooth, per rosette, or per net area, whichever is appropriate or preferred. The design shall be based on net area method using the test values or on gross area method using 80 per cent of the test values, as defined in Paragraphs (a) and (b):

(a) The gross area is defined as the total area of a member covered by a connector plate.

(b) The net area is defined as the total area of a member covered by a connector plate less the area within a given distance of the edge or end of the member, as shown in Figure 12. For net area calculation, the minimum end distance "e", measured parallel to grain, shall be the greater of ½ inch or one-half the length of the tooth; the minimum edge distance "a", measured perpendicular to grain, shall be the greater of ¼ inch or one-quarter the length of the tooth.

9.7.3.1.4 When the load is applied at an angle to grain other than parallel or perpendicular the allowable values of lateral resistance shall be determined by the following formulae:

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta} \qquad N^1 = \frac{P^1Q^1}{P^1\sin^2\theta + Q^1\cos^2\theta}$$

where N = allowable value for plates loaded at angle θ to the grain with the teeth parallel to the load

N^1 = allowable value for plates loaded at angle θ to the grain with the teeth perpendicular to the load

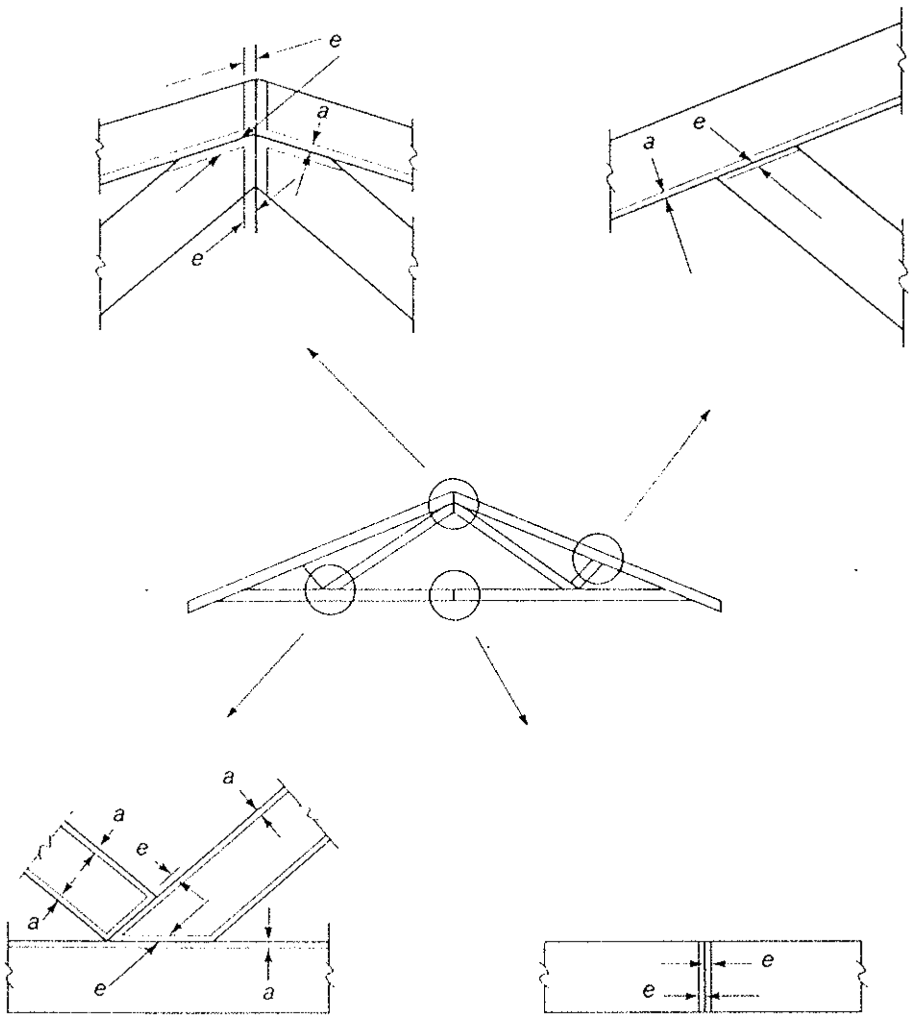
P = allowable value for plates loaded parallel to the grain with the teeth parallel to the load

Q = allowable value for plates loaded perpendicular to the grain with the teeth parallel to the load

P^1 = allowable value for plates loaded parallel to the grain with the teeth perpendicular to the load

Q^1 = allowable value for plates loaded perpendicular to the grain with the teeth perpendicular to the load

When the teeth are oriented at an angle other than parallel or perpendicular to the direction of the load, the allowable value shall be determined by linear interpolation between the values N and N^1.



NOTE e = *end distance*
a = *edge distance*

FIGURE 12

END AND EDGE DISTANCES

9.7.3.1.5 The allowable value shall be adjusted for duration of load other than normal in accordance with the provisions of Clause 3.3.2.2. For material treated with fire-retardant chemicals, the provisions of Clause 3.3.2.3 shall apply. For various seasoning and service conditions of timber, the reduction factors for moisture content given in Table 33 shall apply.

9.7.3.1.6 To allow for moment effects at the heel joint of pitched trusses, the heel plate shall be designed to have sufficient capacity to withstand the direct axial forces in the top and bottom chords using the following reductions in allowable tooth, rosette, or area value:

Under 3/12 slope	85% of allowable
3/12 to less than 4/12 slope	80% of allowable
4/12 to less than 5/12 slope	75% of allowable
5/12 to 5.5/12 inclusive	70% of allowable
Over 5.5/12 slope	65% of allowable

9.7.3.2 Tensile and Shear Strength. The average unit tensile and unit shear strengths of the truss plates shall be determined by tests carried out in accordance with CSA Standard S347, Method of Test for Evaluation of Truss Plates Used in Lumber Joints. The allowable design strengths shall be determined by dividing the average tensile and shear strengths by a factor of 2.5.

CIB-W18/9-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STAPLES

(concerning section 6.1.1.3 of CIB Timber Code)

by

K Möhler

University of Karlsruhe
Federal Republic of Germany

PERTH, SCOTLAND
JUNE 1978

Staples

(concerning section 6.1.1.3)

As the long-range performance of coated staples is mostly unknown until now, only non-coated staples should be mentioned in the design specifications. The performance of coated staples must be tested in each case and allowable loads for coated staples must be derived from the individual tests.

The tests with non-coated staples performed to-date indicate that, under given conditions, staples can be used in the same manner as wire nails to transmit shear as well as withdrawal loads in wood-to-wood as well as wood-base panel-to-wood joints.

The allowable shear loads in the direction perpendicular to the direction of the staple axis can be computed by using the following nail formula, provided the rules for the distances and the depth of penetration applicable to nails are satisfied and the staple crown is driven at an angle of at least 35° to the wood grain:

$$N_1 = 2 \cdot \frac{500\, d^2}{10 + d} \quad , \; N$$

where d = nominal wire diameter, mm.

If the angle of the staple crown to the wood grain is less than 35°, the allowable load shall be reduced by 30%. If the staples are driven into partially dry wood, the allowable load shall be reduced by one-sixth.

CIB-W18/9-10-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE DISTRIBUTION OF SHEAR STRESSES IN TIMBER BEAMS

by

F J Keenan

University of Toronto
Canada

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PERTH, SCOTLAND
JUNE 1978

INTRODUCTION

At the March 13-14, 1978 meeting of the Code Drafting Sub-committee of CIB W18, Professor Möhler proposed that, in calculating the design shear force for timber beams that

a all loads acting within a distance d from the support be ignored, where d is the depth of the member, and

b half of the load acting between distances d and 2d from the support also be ignored.

Provision (a) has appeared in several codes over the years but (b) appears to be a new proposal. I understand that Professor Möhler's reason for putting it forward is (from the meeting minutes) "it was unreasonable to consider that the load jumped suddenly in value".

The purpose of this note is to agree with provision (a) and to disagree with provision (b). Historically, (a) comes from some work in the 1930s at the US Forest Products Laboratory on severely checked timber beams. It came originally from considerations of a "two-beam" theory which, over the years, has been misapplied in many cases to all sawn and laminated beams. As a result of some work by myself, and later by Foschi and Barrett, the two-beam theory and its resulting equations have been dropped from CSA 086 and from some other codes. (See attached 'Shear Strength of Timber Beams' by F J Keenan.)
Many of the effects previously ascribed to two-beam action now appear to be aspects of a size effect in shear strength.

However, it was found during this investigation that provision (a) above is still a reasonable reflection of the distribution of shear stresses in a beam but for reasons more akin to St Venant's Theorem* than to two-beam theory. The calculations to support this were made using finite element methods and the details of these calculations are also appended.

The particular point to observe in the calculations is that when a/d = 1, the maximum shear stress = 1.5 V/A. This means that all loads greater than distance d from the support produce the full shear stress values.

RECOMMENDATION

Based on the above, it is recommended that only provision (a) be adopted, ie "In the calculation of longitudinal shear stress in beams, all loads acting within a distance from the support equal to the depth of the beam may be neglected".

---

*Roughly, that a distance d is required for concentrated actions (the support face) to have distributed responses (shear stresses).

Extract from:

"THE SHEAR STRENGTH OF GLUED-LAMINATED

TIMBER BEAMS"

by

F J Keenan

(PhD Thesis, University of Toronto, 1973)

5.2 The Finite Element Method of Stress Analysis

The Finite Element method seems to have achieved the rank of "production tool" in structural engineering and engineering mechanics research and thus hardly needs any introduction beyond listing a few significant references: Turner et al (1956), Clough (1960, 1965), Zienkiewicz and Cheung (1967), Connor and Will (1969), Rowan and Hackett (1969). The method has been used for the analysis of wood structural members in a few recent research projects: Hibbert (1965), Hooley and Hibbert (1967), Maki (1968), Stieda (1969), Al-Dabbagh (1970), Etherington et al (1972), Al-Dabbagh, Goodman and Bodig (1972).

The particular manifestation of the method which was used in the thesis was the ISOQUAD program for small-displacement plane stress analysis of isotropic linearly elastic continua written by Frind and Berndl (1970), based on theory developed by Ergatoudis, Irons and Zienkiewicz (1968). The program was modified slightly by the author to permit orthotropic behaviour and is listed in the Appendix.

5.3 Beams and Elements

We wished to find the distribution of shear stress and transverse compression stress in a glulam beam as the $\frac{a}{d}$ ratio varied, to see if we could find anything which might predict the relationship plotted in Figure 2.5.1. Consequently, beams were analyzed for six loading conditions corresponding to $\frac{a}{d}$ values of 1, 2, 3, 4, 5 and 6. Since there were no discontinuities or cusps in the findings, only the results of the $\frac{a}{d} = 1$, 3 and 5 analyses are reported here. Symmetry was used in the analyses and thus, the left half only of a beam for which $\frac{\ell}{d} = 13.5$ was studied. (This span to depth ratio is representative of the beams in Figure 2.5.1.)

For each $\frac{a}{d}$ ratio, three compatible discretizations were used in order to examine convergence of the results; one set of discretizations and the corresponding boundary node constraints are shown in Figures 5.3.1 and 5.3.2(a).

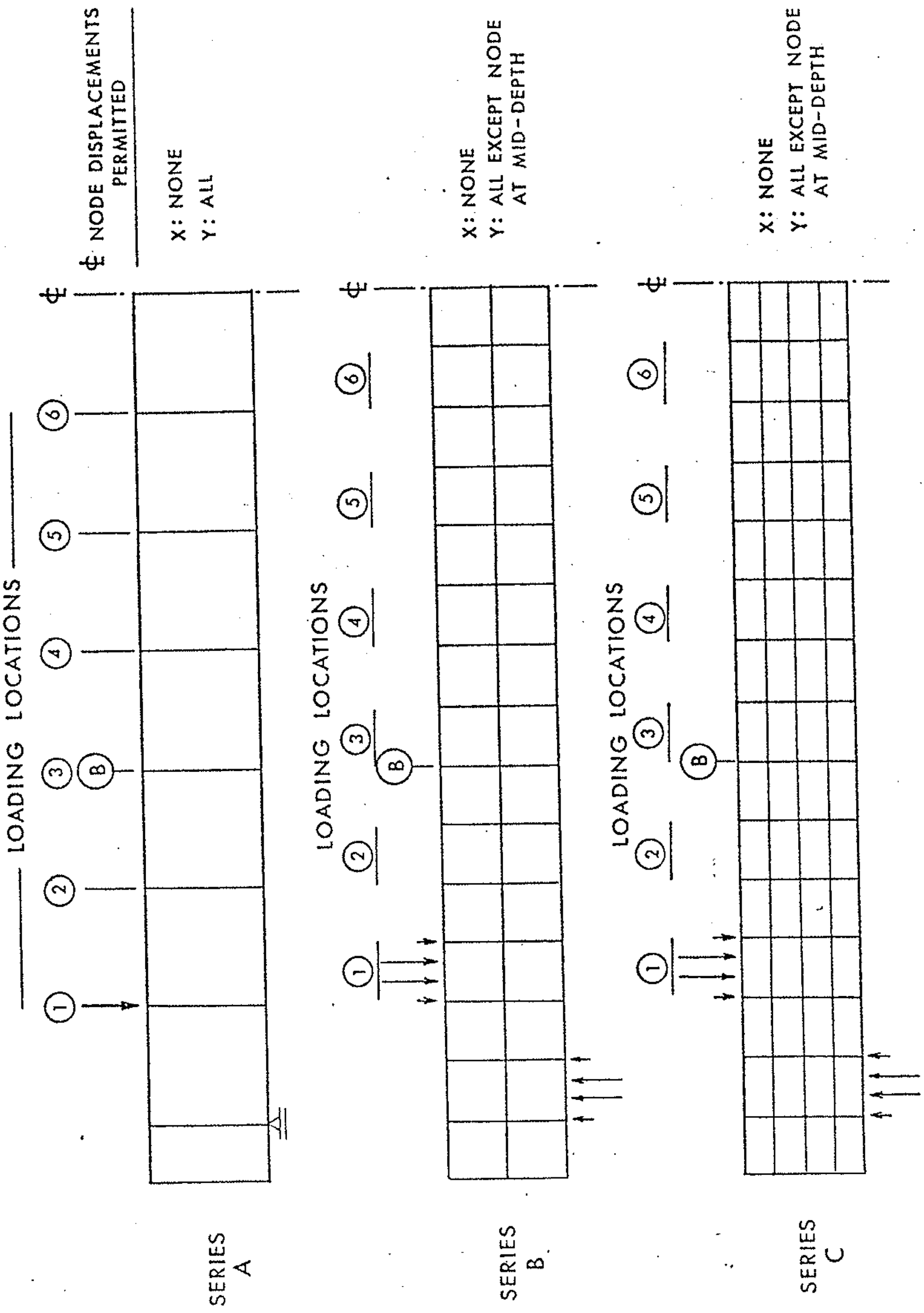
The element used was a cubic isoparametric quadrilateral element as referenced above, with four corner nodes and two interior nodes on each side as shown in Figure 5.3.2(b). The node forces used to represent a distributed boundary force must be consistent with the energy formulation of the element; for a cubic side, Connor and Will show that the load distribution depicted in Figure 5.3.2(b) should be used.

Figure 5.3.1 - Discretizations and Node Restraints

For illustration, the three arrangements analyzed for $\frac{a}{d} = 1$ are shown as well as the loading locations for the other five $\frac{a}{d}$ ratios. Elements which appear square in this Figure are square; the other elements have their long side equal to twice their short side. The Series C restraints are shown in more detail in Figure 5.3.2(a).

Section B is referred to later in Figure 5.4.1.

The X-direction is horizontal; the Y-direction is vertical.

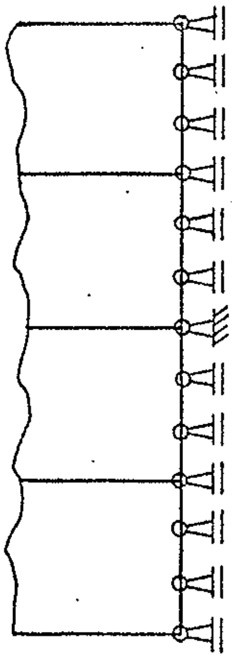


LOADING LOCATIONS
1
2
3
B
4
5
6
℄
℄ NODE DISPLACEMENTS PERMITTED
SERIES A
X: NONE
Y: ALL
SERIES B
X: NONE
Y: ALL EXCEPT NODE AT MID-DEPTH
SERIES C
X: NONE
Y: ALL EXCEPT NODE AT MID-DEPTH

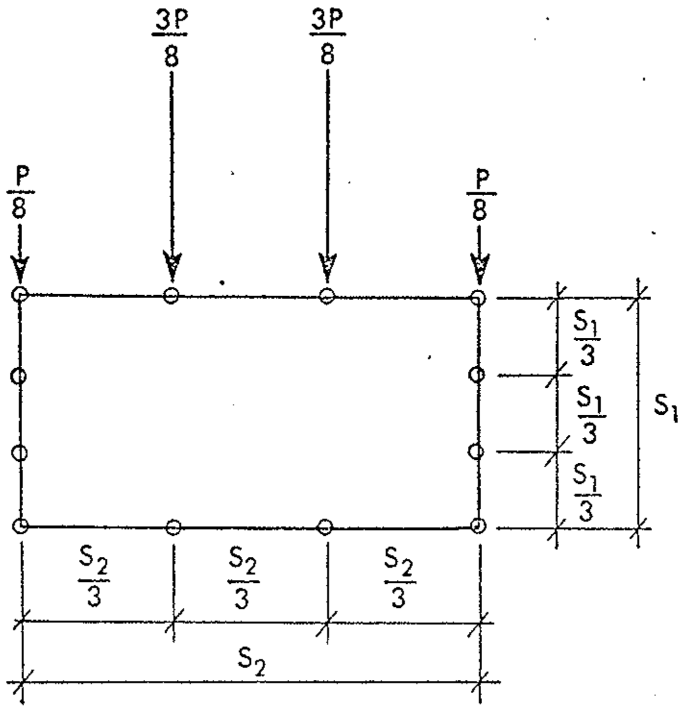
Figure 5.3.2 - Element Details

(a) Centreline node restraints used for Series C beams.

(b) Element nodes and boundary loads.



℄ RESTRAINTS FOR SERIES C



As mentioned, the program was modified to accommodate orthotropic materials. The elastic constants chosen for these analyses were derived from Projects D and C of the thesis and are:

$$E_x = 1.92 \times 10^3 \text{ ksi}$$

$$E_y = 1.92 \times 10^2 \text{ ksi}$$

$$n = 10$$

$$G_{xy} = 1.21 \times 10^2 \text{ ksi}$$

$$\mu_{xy} = 0.410$$

$$\mu_{yx} = 0.041$$

5.4 Results

Convergence of the results in going from Series A to Series B to Series C was reasonable. For illustration, Figure 5.4.1 shows the results for the three Series at Section B for the loading case where $\frac{a}{d} = 3$. This was the section, for this loading, which displayed the greatest differences in shear stress values between the three Series, and thus represents the worst agreement in values obtained from the three Series.

The results of the analyses were plotted for all of the Series C results for all $\frac{a}{d}$ ratios. Figures 5.4.2(a), (b) and (c) show the shear stress and transverse compression stress contours for $\frac{a}{d}$ = 1, 3 and 5, respectively.

Figure 5.4.1 - Shear Stress Convergence

Shown are the differences in shear stress between the nodal values obtained for the same node from two adjoining elements, for the three discretizations. These particular data are for Section B shown in Figure 5.3.1 for $\frac{a}{d} = 3$.

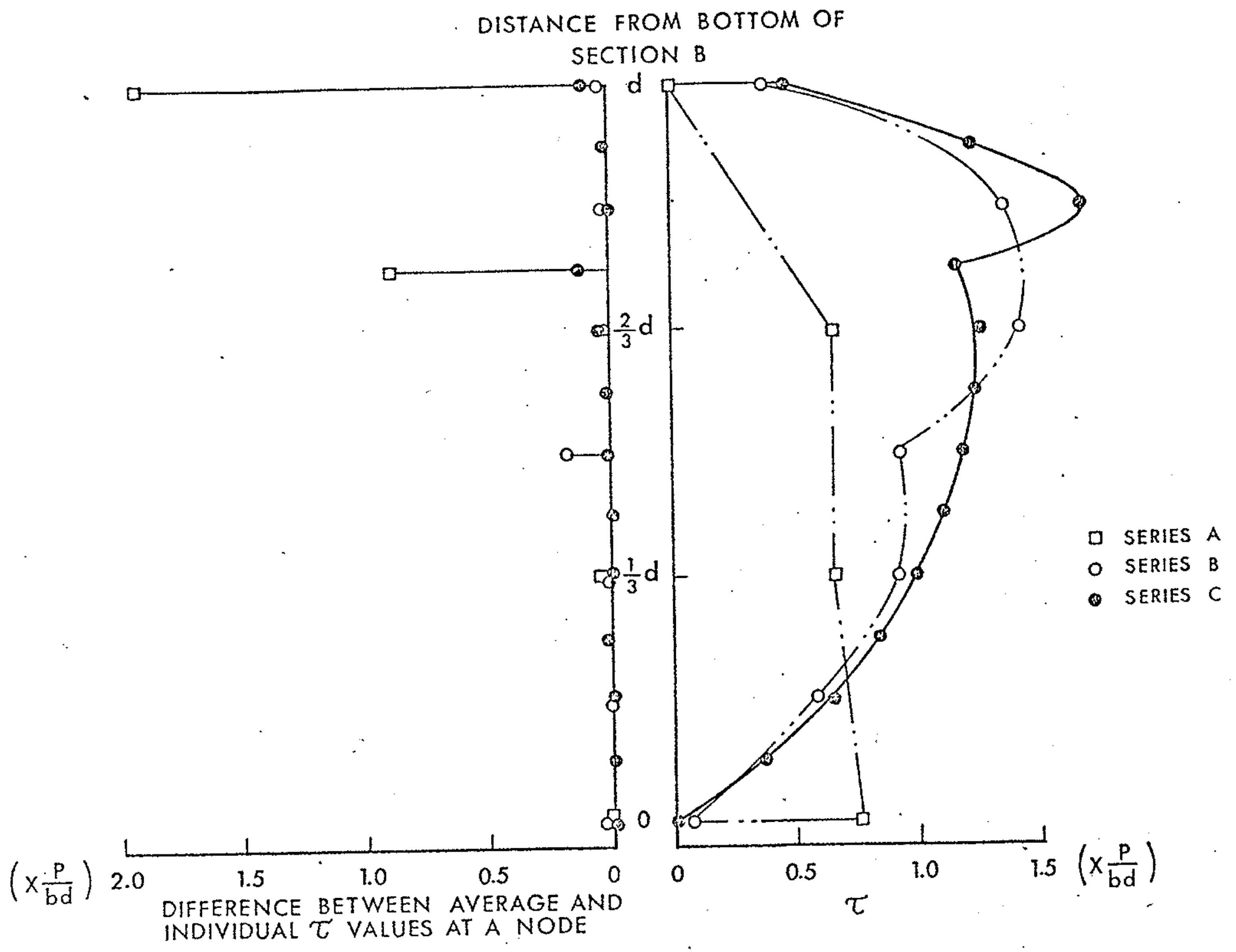

DISTANCE FROM BOTTOM OF SECTION B
d
$\frac{2}{3}$d
$\frac{1}{3}$d
0
($\times \frac{P}{bd}$) 2.0
1.5
1.0
0.5
0
DIFFERENCE BETWEEN AVERAGE AND INDIVIDUAL τ VALUES AT A NODE
0
0.5
1.0
1.5 ($\times \frac{P}{bd}$)
τ
SERIES A
SERIES B
SERIES C

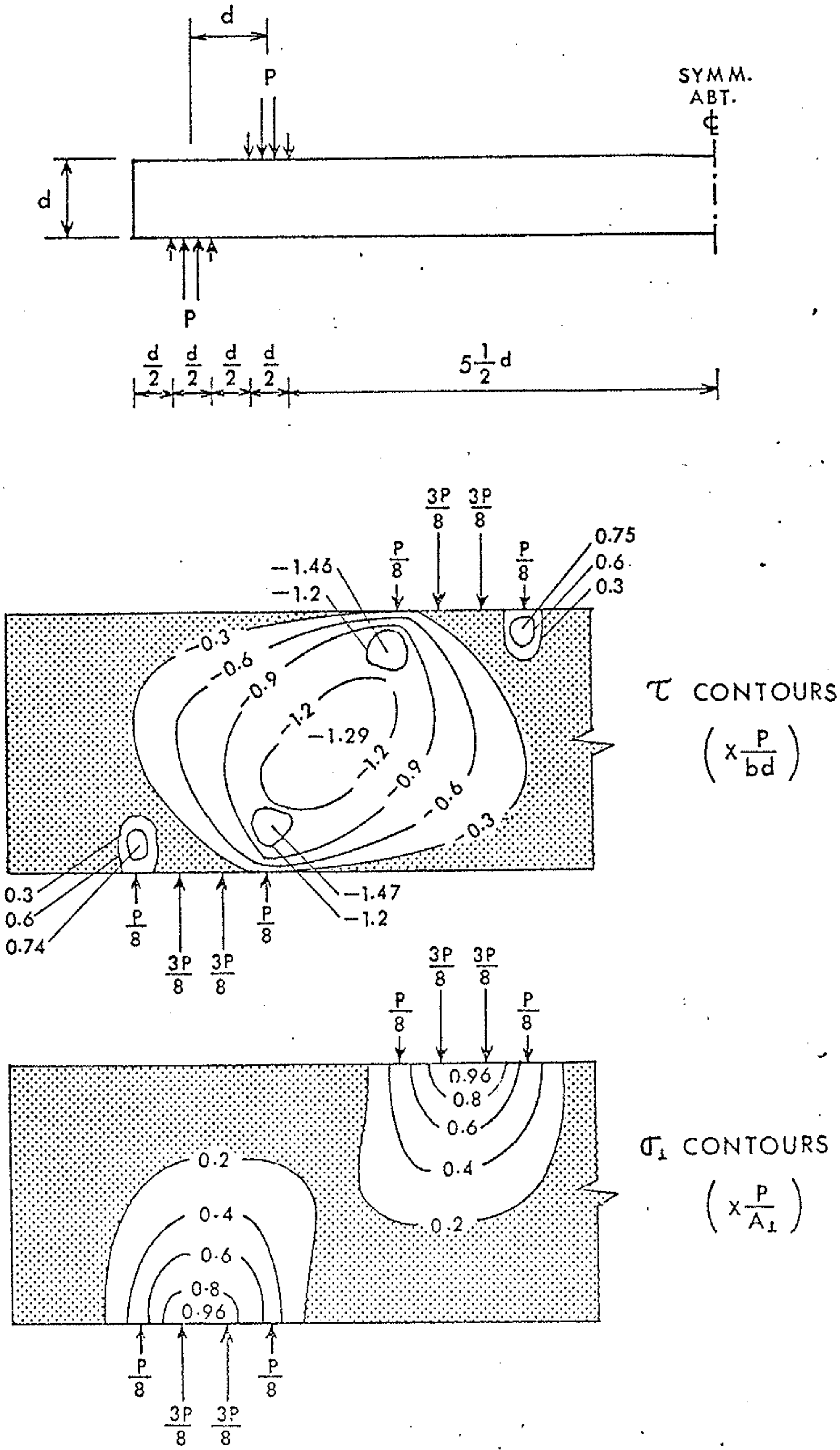
Figure 5.4.2 - Stress Contours

(a) $\frac{a}{d} = 1$ (page 240)

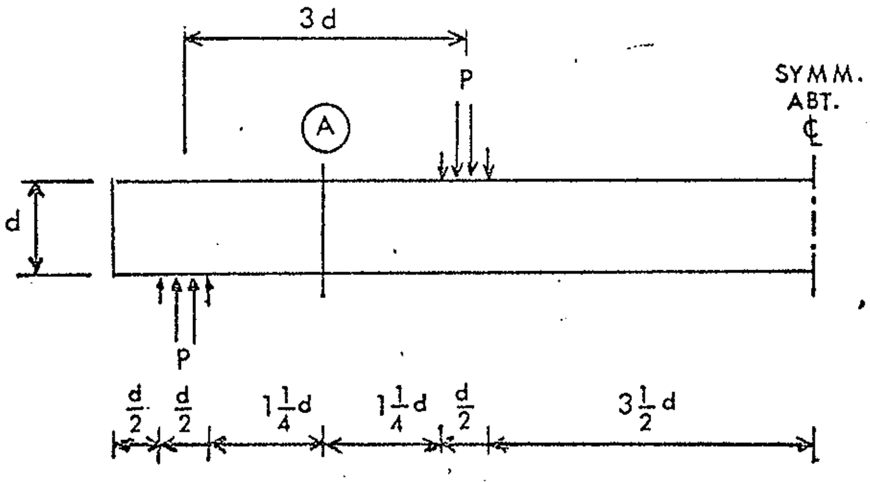
(b) $\frac{a}{d} = 3$ (page 241)

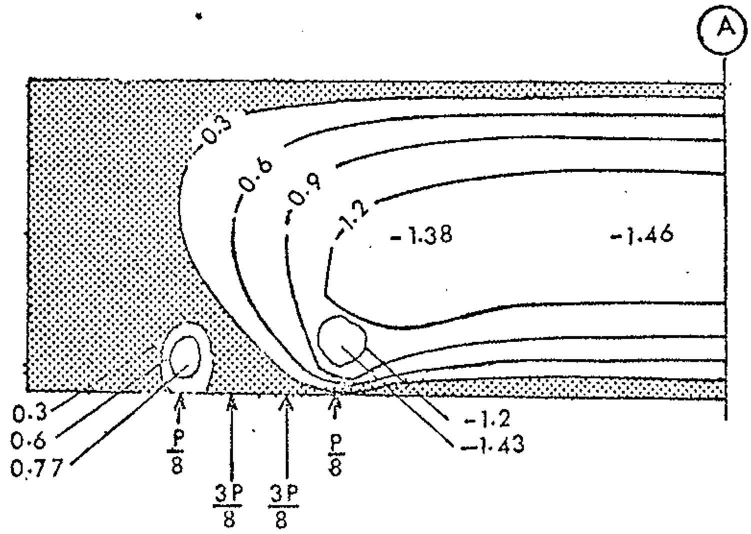
(c) $\frac{a}{d} = 5$ (page 242)

$A_\perp$ is the contact area between the distributed load and the beam. The shaded areas represent zones of stress coefficients lower in absolute value than 0.3 for shear and 0.2 for transverse compression.



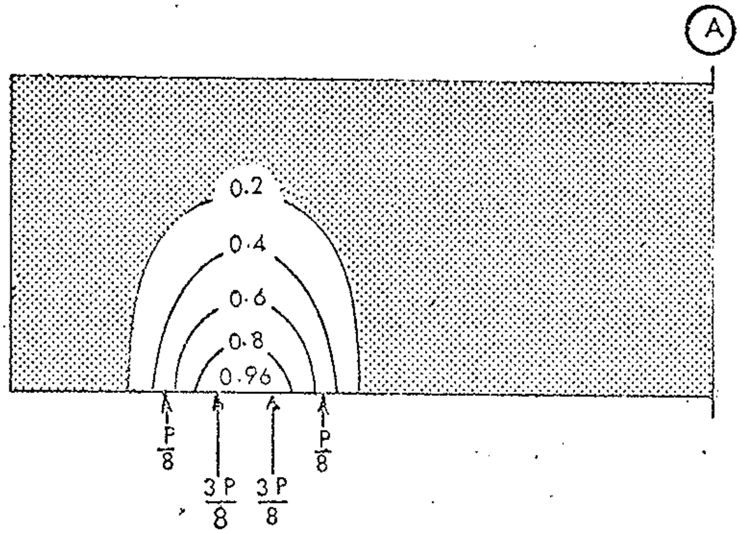
d
P
SYMM.
ABT.
℄
d
P
d/2 d/2 d/2 d/2
5 1/2 d
3P/8 3P/8
P/8
P/8
−1.46
−1.2
0.75
0.6
0.3
−0.3
−0.6
−0.9
−1.2
−1.29
−1.2
−0.9
−0.6
−0.3
−1.47
−1.2
0.3
0.6
0.74
τ CONTOURS
(x P/bd)
0.96
0.8
0.6
0.4
0.2
σ⊥ CONTOURS
(x P/A⊥)





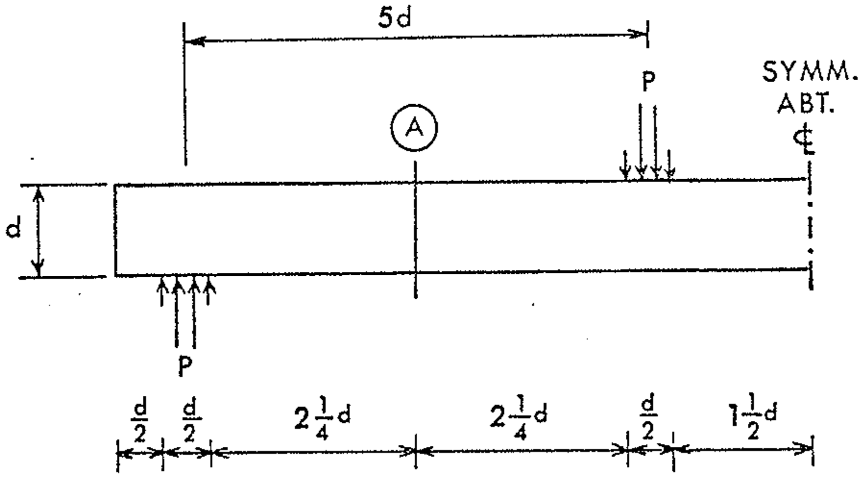
τ CONTOURS

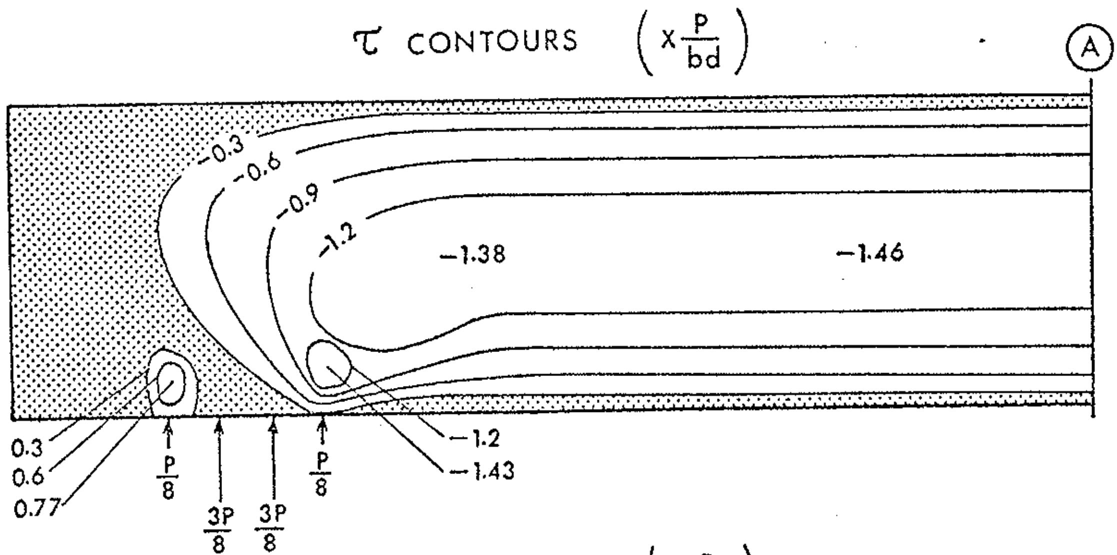
$\left(\times \frac{P}{bd}\right)$

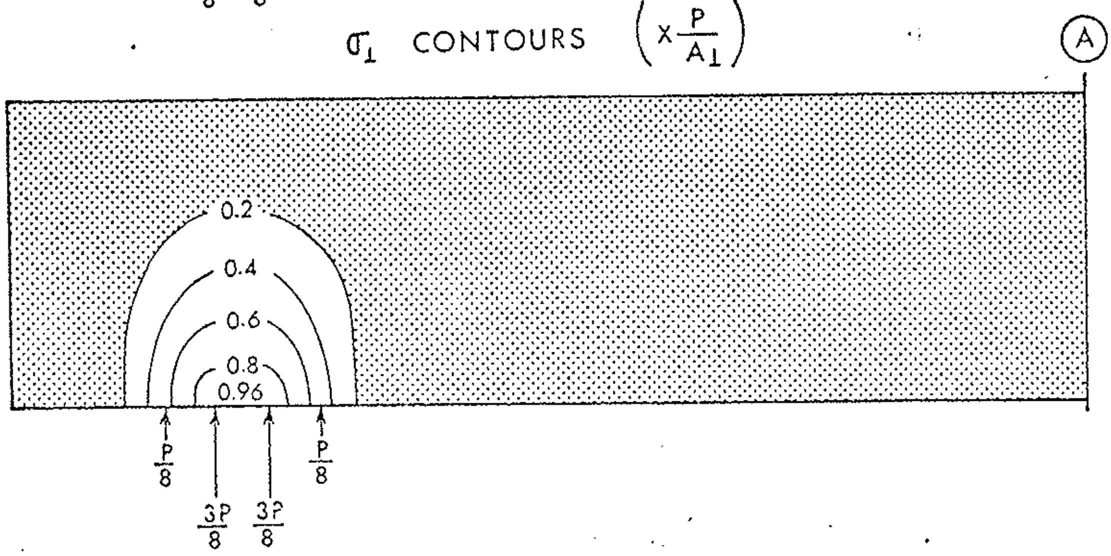


$\sigma_\perp$ CONTOURS

$\left(\times \frac{P}{A_\perp}\right)$


5d
P
SYMM.
ABT.
℄
A
d
P
d/2
d/2
2 1/4 d
2 1/4 d
d/2
1 1/2 d


τ CONTOURS (x P/bd)
A
-0.3
-0.6
-0.9
-1.2
-1.38
-1.46
0.3
0.6
0.77
P/8
3P/8
3P/8
P/8
-1.2
-1.43


σ⊥ CONTOURS (x P/A⊥)
A
0.2
0.4
0.6
0.8
0.96
P/8
3P/8
3P/8
P/8

5.5 Observations and Conclusions

Three major observations can be drawn from these graphs:

(a) In no cases do significant transverse compressive stresses coincide with the entire regions of high shear stress; thus, even if transverse compressive stresses did have a substantial beneficial effect on shear strength, the two types of stress are not distributed in a manner which would allow them to interact.

(b) There is some reduction in maximum midheight shear stress as the $\frac{a}{d}$ ratio decreases, but this diminution is not nearly large enough to help to explain the behaviour displayed in Figure 2.5.1.

(c) Except for the very localized high shear stresses which are present in the immediate vicinity of the applied loads, in general, the midheight shear stresses are greater than the shear stresses above and below the midheight.

The localized shear stress concentrations which have been observed by other researchers (Hooley and Hibbert 1967, Cowan 1962, Bohannan 1966b, Wilson and Cottingham 1947) are not considered to be of major significance to the shear strength of the beam and the reasons for stating this are described throughout Chapter 6. In connection with this point, when analyzing shear stresses in beams, it is considered important by the author to delineate over how large a region these stress concentrations occur. For this reason it was decided to plot contours of stress rather than stress profiles at certain sections of a beam.

FPRS FOREST PRODUCTS JOURNAL VOL. 24, NO. 9 SEPTEMBER 1974

Shear Strength of Wood Beams

F. J. Keenan

Abstract

The shear strength of wood beams is not a constant but decreases as the shear span-to-depth ratio increases. In the past, two reasons have been postulated for this – one is that the shear strength of wood was thought to increase significantly due to the application of perpendicular compression stress, e.g., at beam supports; the other is that, when the applied load is close to the support, the maximum shear stress was thought to be considerably less than that calculated in the usual way, through "two-beam" action. In experimental and analytical studies on dry Douglas-fir, these two postulates were found to be not significant in explaining the behavior of glued-laminated beams. Finally, a shear strength model is proposed which reconciles all wood shear strengths regardless of how they are determined; also shown is where current shear design practice may be unsafe and where it is conservative.

THE SHEAR STRENGTH OF WOOD displays many anomalies, most of which are well known and many of which have tantalized researchers and hampered timber designers for years. Two of the more striking of these anomalies are the following:

a) The shear strength of wood does not appear to be a material property of constant value but seems to be related to the method of shear strength determination, even when all of the factors which normally affect the mechanical properties of wood (such as a moisture content (MC), specific gravity, rate of loading, presence of defects, etc.) are controlled. One of the best known contrasts is between ASTM shear block (2) test results and shear strengths obtained in beam tests. To illustrate, air-dry Douglas-fir shear blocks have a mean strength of 1,382 psi (18) whereas glued-laminated beams made of air-dry Douglas-fir display shear strengths in the range 400 to 900 psi (14); generally, attempts to reconcile these two strengths through studies of matched specimens have not been successful (29, 33). Other ways in which the shear strength of Douglas-fir has been measured include the use of torsion tubes (17, 22, 36), panel-shear specimens (27), glued shear blocks (28), modified solid blocks (11), bending tests of different sizes of solid beams (3, 16, 17, 21, 32), tension tests on connector joints (15), and oblique grain tension and compression specimens (17, 36); in general, each of these techniques has yielded estimates of shear strength different from the shear block results, different from the laminated beam results, and different from each other.

b) The shear strength of laminated Douglas-fir beams is not a constant but has been shown to be related to the shear span-to-depth ratio (a/d) of the beams, where the shear span is the distance between a reaction force and the nearest applied concentrated load. Huggins, Palmer, and Aplin (14) published a graph, on which Figure 1 is based, showing that beam shear strengths increase from about 400 psi to 900 psi as the a/d ratio decreases from 6 to 1. However, the use of the a/d ratio as a general predictor of beam shear strength is questionable, since solid Douglas-fir beams of cross-sectional dimensions smaller than those of the glued-laminated beams referred to in Figure 1, when tested at the same a/d ratios, resist much greater shear stresses than the corresponding glued-laminated beams (3, 17). This difference does not seem to be caused by the presence or absence of "defects" since Wilson and Cottingham (35) have shown that there was no significant difference between the shear strengths of knotty and clearer glued-laminated beams.

In the past, two reasons have been given for the a/d effect. The first is that the shear strength of wood parallel to the grain was thought to increase when compressive stresses perpendicular to the grain were applied to the shear plane (25), in a manner analogous to the effect of a normal force on the frictional resistance of a sliding surface. Thus, as a load applied to a beam moved closer to the beam reaction, i.e., as a/d decreased, the transverse compressive stresses caused by the bearing of the loads on the beam acted in the same general zone

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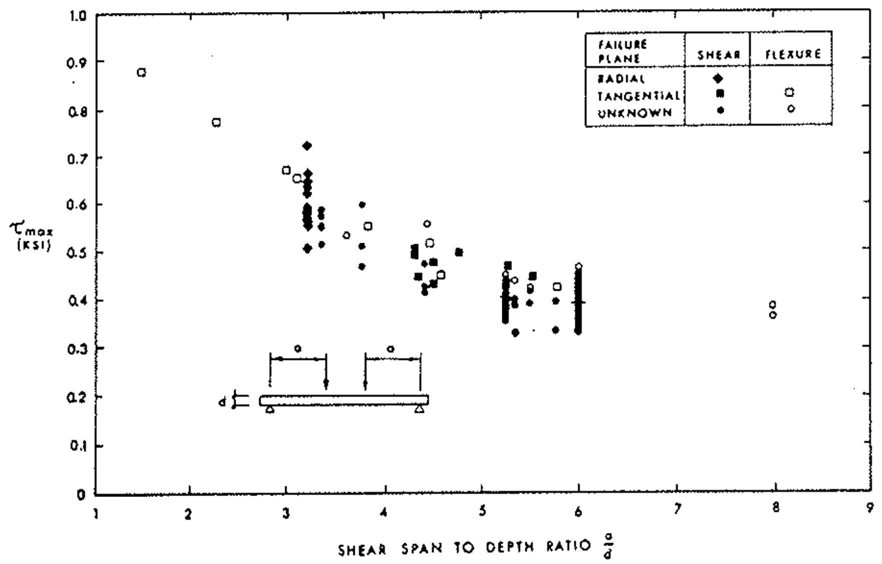


Figure 1. — Effect of shear span-to-depth ratio on the shear strength of glued-laminated Douglas-fir beams. (After Huggins, et al. (14)). The solid symbols represent shear failures; each unshaded symbol represents the highest shear stress sustained by beams at that particular a/d ratio which did not fail in shear.

as the maximum shear stresses in the beam, and it was expected that beam shear strength would thus increase.

The second reason is that "two-beam" action, which was identified in severely checked wood beams by Newlin, Heck, and March (26), is assumed to also exist in glue-laminated beams; this is manifested by the inclusion in most North American design codes (e.g., 6) of a formula developed by Newlin, Heck, and March (26) which permits a designer to use a shear force less than that calculated using statics. The "two-beam" action theory predicts a lesser value for shear stress at mid-depth and higher values of shear stress above and below mid-depth of a beam than the values predicted by conventional analysis. The overall effect is to reduce the maximum shear stress acting at a section of a beam, and the magnitude of this reduction increases as the applied load gets closer to the beam reaction, i.e., again, as the *a/d* ratio decreases.

The objectives of the present study are, through experimental and analytical projects, to see if the above two rationales for the *a/d* effect actually have any significant applicability to glued-laminated Douglas-fir beams, and if they do not, to develop a new model for the shear strength of laminated Douglas-fir beams that can be extrapolated to predict the shear strength of air-dry Douglas-fir regardless of how this strength is determined.

Experimental Work – Effect of Transverse Compression Stress on Shear Strength

Materials and Methods

Three different types of specimens – ASTM shear blocks, oblique grain compression specimens, and torsion tubes – were tested to investigate the effect of compression stress perpendicular to grain ("transverse compression stress") on shear strength parallel to grain. All specimens were air-dry Douglas-fir; sample sizes, MC's, and specific gravities are listed in Table 1. Specimen dimensions and methods of load application are shown schematically in Figure 2.

In Project A, the standard ASTM shear block was subjected to transverse compression stress by means of tightening the nuts on drawbars of a clamping arrangement mounted around the specimen. The clamping force passed through a roller bearing before impinging on the specimen so that the tendency of the clamping arrangement to inhibit shear distortion was minimized; the clamping force was measured by a load cell on the drawbars. In this project, the transverse compression stress was applied first, to a level of 260 psi, 500 psi, 750 psi, or 1,000 psi, and then the specimen was tested in shear. The specimens were manufactured so that half were tested in the tangential plane of the wood and the other half in the radial plane.

In Project B, the specimens were cut from planks so that 16 specimens had tangential planes of obliquity when tested in compression parallel to the specimen's axis and 68 had radial planes of obliquity. From elementary mechanics, if the grain direction makes an angle Θ with the direction of the applied load, the applied axial compression stress, σ_{max}, induces both a shear stress parallel to grain, τ, and a compression stress normal to the grain, $\sigma_{\perp}$, as follows:

$$\tau = \frac{\sigma_{max}}{2} \sin 2\Theta \qquad [1]$$

Table 1. — TEST SPECIMENS.

Project	No. of speci-mens	Specimen type	MC (%)	Basic SG
A	33	ASTM shear blocks	6.1±0.4	0.50±0.02
B	90	Oblique grain compression specimens	10.4±0.5	0.50±0.04
C	47	Torsion tubes	8.0±1.0	0.38±0.06

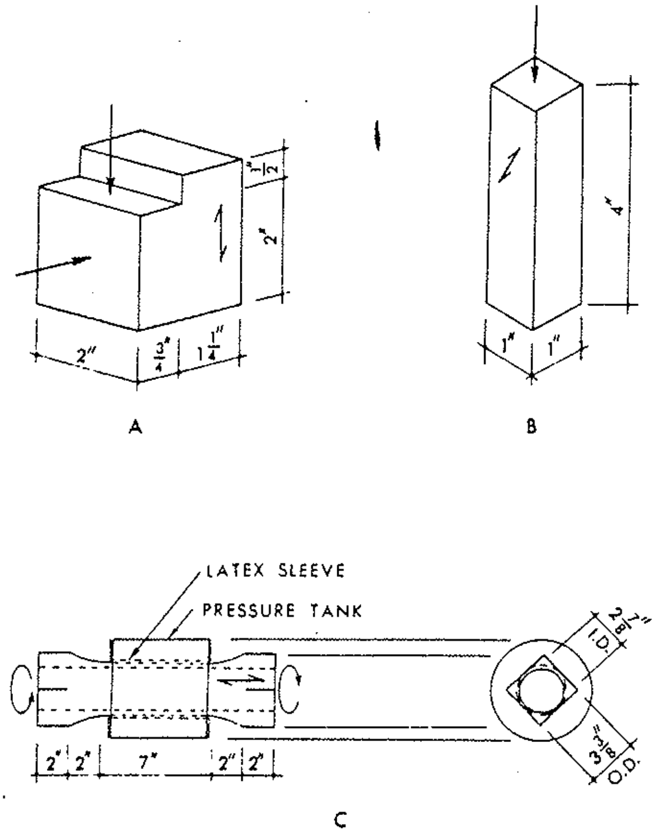


Figure 2. — Test specimens and schematic representation of loading method (the one-sided arrows indicate the grain direction): A—ASTM shear block; B—oblique grain compression specimen; C—torsion tube.

$$\sigma_\perp = \sigma_{max} \sin^2\Theta \qquad [2]$$

The relative proportions of τ and $\sigma_\perp$ are thus controlled by the selection of the angle of obliquity Θ. The tangential plane specimens had obliquity angles of 20°, 25°, and 30°; the radial plane specimens had obliquity angles of 0°, 15°, 20°, 25°, 30°, 45°, 60°, 75°, and 90°. The specimens were loaded in compression at a rate of 0.012 inches per minute until failure, and each specimen was examined to determine the type of failure.

In Project C, thin-walled torsion tubes were turned down on a lathe to the approximate dimensions shown in Figure 2. The specimens were tested in shear at an average rate of 134 psi of shear stress per minute until failure, by rotating one end of the specimen with respect to the other end. Twenty-two specimens were tested in torsional shear alone and another 22 had a hydraulic pressure tank and thin latex sleeve applied over the region of minimum wall thickness. The hydraulic pressure induced a circumferential compression stress in the specimen, which of course, was perpendicular to the grain at all points in the wall of the specimen. Because of the geometry of the specimen, the induced perpendicular compression stress was equal to about seven times the applied hydraulic pressure. The rate of application of the hydraulic pressure was proportional to the rate of application of torsional shear, with different ratios of stress being used to produce a variation of transverse compression stress with shear strength.

Results and Discussion

The test results for the three projects are shown in Figure 3. Each point plotted for Projects A and B and the control point in Project C is the average strength for specimens in a group; the group size is shown in parentheses under the point.

In Project A, two of the radial plane specimens and four of the tangential plane specimens failed at transverse compression stresses between 750 psi and 1,000 psi before shear could be applied. For the specimens which failed in shear, a distinct difference is seen between the responses of the tangential plane and the radial plane shear strengths to transverse compression: the regression shear strength for the radial plane increases at a rate of 73 psi per 100 psi of transverse compression, but the tangential plane shear strength increases by only 7 psi per 100 psi of transverse compression; i.e., the tangential plane shear strength is much less sensitive to transverse compression than is the radial plane. More specifically, the linear regression analyses of the individual test results yielded the following equations for shear strength τ_{max}:

$$\hat{\tau}_{max}\ \text{(psi)} = 1{,}287 + 0.727\ \sigma_\perp\ \text{(radial plane)} \qquad [3]$$

$$\hat{\tau}_{max}\ \text{(psi)} = 1{,}125 + 0.066\ \sigma_\perp\ \text{(tangential plane)} \qquad [4]$$

and the correlation coefficients were 0.81 and 0.10; the radial plane correlation was significant at the 1 percent

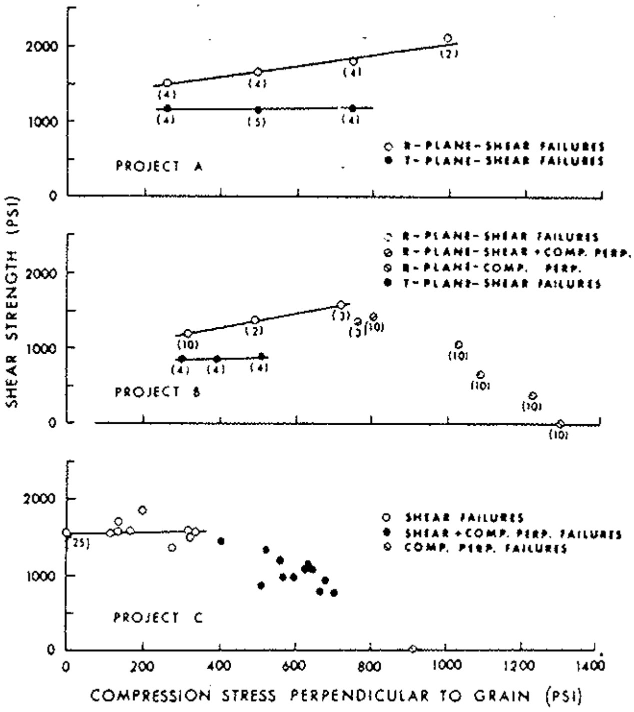


Figure 3. — Tests results—effect of transverse compression on shear strength.

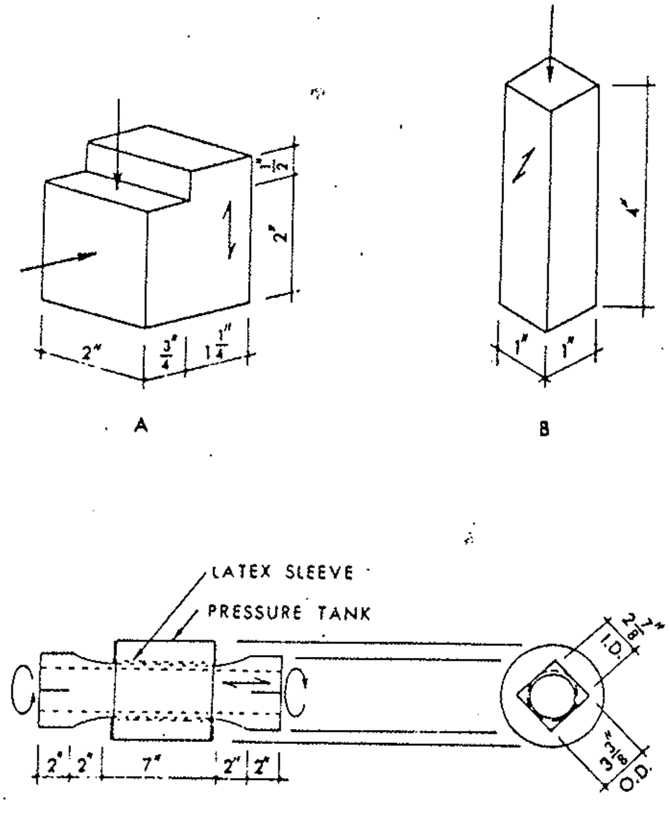


Figure 2. — Test specimens and schematic representation of loading method (the one-sided arrows indicate the grain direction): A—ASTM shear block; B—oblique grain compression specimen; C—torsion tube.

$$\sigma_{\perp}=\sigma_{max}\sin^2\Theta \quad [2]$$

The relative proportions of τ and $\sigma_{\perp}$ are thus controlled by the selection of the angle of obliquity Θ. The tangential plane specimens had obliquity angles of 20°, 25°, and 30°; the radial plane specimens had obliquity angles of 0°, 15°, 20°, 25°, 30°, 45°, 60°, 75°, and 90°. The specimens were loaded in compression at a rate of 0.012 inches per minute until failure, and each specimen was examined to determine the type of failure.

In Project C, thin-walled torsion tubes were turned down on a lathe to the approximate dimensions shown in Figure 2. The specimens were tested in shear at an average rate of 134 psi of shear stress per minute until failure, by rotating one end of the specimen with respect to the other end. Twenty-two specimens were tested in torsional shear alone and another 22 had a hydraulic pressure tank and thin latex sleeve applied over the region of minimum wall thickness. The hydraulic pressure induced a circumferential compression stress in the specimen, which of course, was perpendicular to the grain at all points in the wall of the specimen. Because of the geometry of the specimen, the induced perpendicular compression stress was equal to about seven times the applied hydraulic pressure. The rate of application of the hydraulic pressure was proportional to the rate of application of torsional shear, with different ratios of stress being used to produce a variation of transverse compression stress with shear strength.

Results and Discussion

The test results for the three projects are shown in Figure 3. Each point plotted for Projects A and B and the control point in Project C is the average strength for specimens in a group; the group size is shown in parentheses under the point.

In Project A, two of the radial plane specimens and four of the tangential plane specimens failed at transverse compression stresses between 750 psi and 1,000 psi before shear could be applied. For the specimens which failed in shear, a distinct difference is seen between the responses of the tangential plane and the radial plane shear strengths to transverse compression: the regression shear strength for the radial plane increases at a rate of 73 psi per 100 psi of transverse compression, but the tangential plane shear strength increases by only 7 psi per 100 psi of transverse compression; i.e., the tangential plane shear strength is much less sensitive to transverse compression than is the radial plane. More specifically, the linear regression analyses of the individual test results yielded the following equations for shear strength τ_{max}:

$$\hat{\tau}_{max}\text{ (psi)} = 1{,}287+0.727\ \sigma_{\perp}\text{ (radial plane)} \quad [3]$$

$$\hat{\tau}_{max}\text{ (psi)} = 1{,}125+0.066\ \sigma_{\perp}\text{ (tangential plane)} \quad [4]$$

and the correlation coefficients were 0.81 and 0.10; the radial plane correlation was significant at the 1 percent

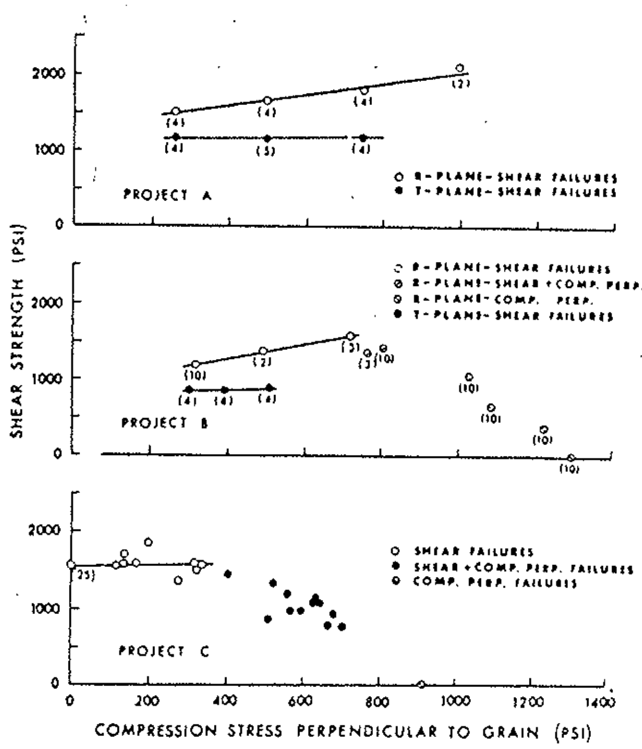


Figure 3. — Tests results—effect of transverse compression on shear strength.

level but a significant correlation for the tangential plane was not demonstrated.

In Project B, as expected (36), failure type was associated with angle of obliquity Θ: at Θ=0°, failures were solely in compression parallel to the grain; at Θ between 15° and 25°, failure was by shear; at Θ=30°, the specimens showed transverse compression failure wrinkles before finally failing in shear; and for Θ>30°, failures were by transverse compression with some shear distortion. As in Project A, the radial plane shear strength was more responsive to transverse compression than was the tangential plane shear strength. Regression equations on the shear failure data are:

$$\hat{\tau}_{max}\ (\text{psi}) = 910+0.882\ \sigma_{\perp}\ (\text{radial plane}) \quad [5]$$

$$\hat{\tau}_{max}\ (\text{psi}) = 761+0.227\ \sigma_{\perp}\ (\text{tangential plane}) \quad [6]$$

with point scatter and significance being similar to that in Project A.

Thus, in terms of response of shear strength to transverse compression, of onset of transverse compression failures, and of statistical significance, the trends found in Project B are consistent with those in Project A.

In Project C, it was possible to differentiate between specimens which failed in shear and those which failed in a combination of shear and transverse compression, as described below. Since all points on the circumference of each specimen were uniformly stressed in both shear and transverse compression, it was expected that the failure surface would seek the plane of greatest weakness. From Projects A and B, the plane of greater weakness responded very sluggishly to the influence of transverse compression, and the Project C response was consistent with this expectation as shown in Figure 3. The regression line for the shear failures has the equation:

$$\hat{\tau}_{max}\ (\text{psi}) = 1{,}518+0.100\ \sigma_{\perp} \quad [7]$$

The correlation coefficient was 0.06 and the correlation was not significant.

The observation drawn from these three projects is that when a shear failure in dry Douglas-fir is free to seek its plane of greatest weakness (as is usually the case in glued-laminated beams), its shear strength parallel to grain is not significantly increased by the application of compressive stress perpendicular to grain. This helps to explain why the present results differ from those of Mandery (25); an examination of his paper suggests that his specimens were not able to seek their plane of greatest weakness.

Anatomical Studies

After the testing, the failure surfaces of the specimens in Projects A and C were examined using scanning electron microscopy. The results of these studies will be described in another paper, but a few major observations can be reported here since they support the trends of the strength properties test results. In Project A, the radial plane failure surfaces were dominated by intrawall failures of the latewood tracheids, in which the failure takes place in the primary wall and the compound middle lamella. Failure in this surface is by longitudinal sliding of one tracheid past another, and the analogy of increasing "friction" in the wood caused by increases in transverse compression seems apt. In contrast, the tangential plane failures displayed almost exclusively earlywood transwall failures in which the tracheid walls seemed to be ripped along their length rather than sliding past each other; such a failure appearance seems to be associated with a frictional analogy to a much lower degree of expectation.

In Project C, the failure crack surfaces of specimens which were subjected to transverse compression stresses less than about 370 psi sought a tangential orientation and occurred largely in earlywood tracheids, but showed no strong preference for any particular failure location around the circumference of the specimen; the failure surfaces appeared essentially the same as the tangential plane failure surfaces of Project A. Above 370 psi, however, a distinct change was noted in the failure location and appearance: the failures were transwall in the earlywood tracheids and showed a "lateral shearing" distortion of the tracheid walls as well as longitudinal ripping. Moreover, the failure surface had a strong tendency to seek the point on the circumference of the specimen where the annual rings made an angle of about 60° to 75° with the tangent drawn to the specimen wall, i.e., with the direction of circumferential compressive stress perpendicular to grain. This was puzzling at first until it was recalled that it is approximately this annual ring orientation which produces the smallest strengths of wood in compression perpendicular to the grain (19, 20). The lateral movement on the failure surface was then thought to be the same as the diagonal shearing movement seen in a piece of wood tested in compression perpendicular to the grain when the annual ring orientation is neither perpendicular nor parallel to the direction of load. This agrees with Figure 3 in which the onset of transverse compression failure occurs earlier in Project C than in either Project A or Project B.

Analytical Studies–Stress Distributions in Glued-Laminated Beams

Method

In order to see if "two-beam" action, which was identified for severely checked beams, had any applicability to glued-laminated beams, which normally have only minor checking (13), the finite-element method (37) was used to carry out stress analyses of typical glued-laminated beams. This analytical method, which has been used by other wood researchers (1, 9, 12, 24), involves replacing the actual continuum by an assemblage of discrete, or finite, elements connected together at their corners or "nodes." The mechanical properties of the finite elements can be specified to duplicate the material properties of the actual members; distributed forces can be allocated on a rational basis to the various nodes, and the loaded structure analyzed, for example, by common structural stiffness analysis methods. Normally, the accuracy of the derived stresses can be monitored by discretizing the member into compatible coarser or finer subdivisions and checking the convergence of the results as finer meshes are used.

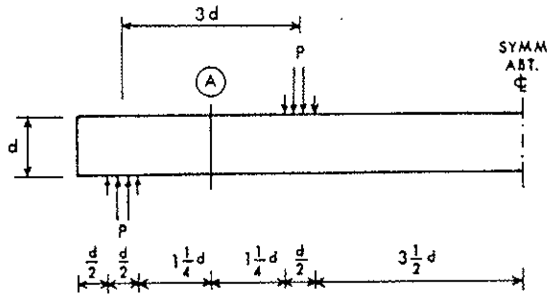
In the present study, the "cubic isoparametric quadrilateral" element (8, 10) was used to investigate the shear and transverse compression stress distributions in a glued-laminated beam typical of those whose results are shown in Figure 1 as the a/d ratio decreased from 6 to 1. The mechanical properties specified for the element are typical of dry Douglas-fir:

$$\text{Elastic moduli: } E_x=1.92\times10^3 \text{ ksi}$$
$$E_y=1.92\times10^2 \text{ ksi}$$
$$G=1.21\times10^2 \text{ ksi}$$
$$\text{Poisson's ratios: } \mu_{xy}=0.410$$
$$\mu_{yx}=0.041$$

Three discretizations were used for each a/d ratio, the finest consisting of elements whose lengths were equal to 0.5d and whose widths were 0.25d. At first glance, this element seems a bit large, but each element has four nodes per side, making the effective discretization much finer. No problems were encountered with convergence.

Results and Discussion

Figure 4 shows typical results; these are the shear stress contours and the compression stress contours for a beam with a/d=3. The conclusions drawn from a study of all of the results are as follows:



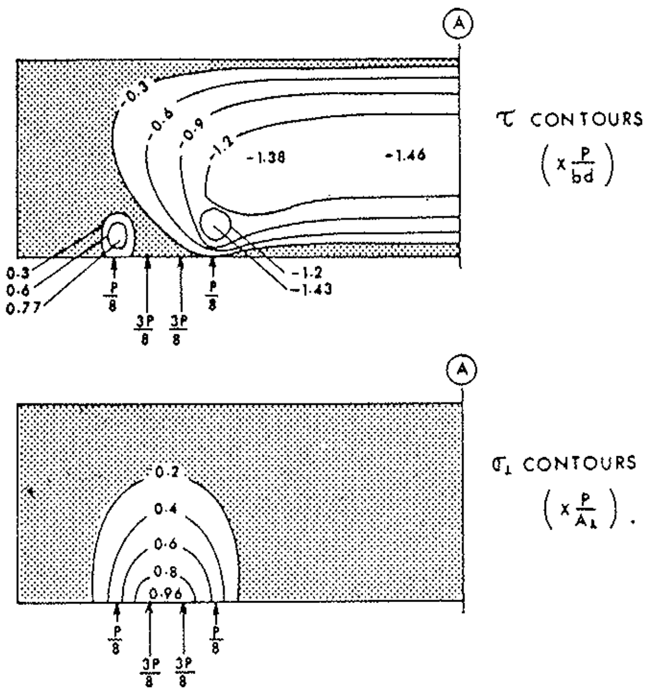


Figure 4. — Contours of shear stress and transverse compression stress for a beam with a/d=3. $A_\perp$ is the contact area between the loads and the beam.

a) In no case do significant transverse compressive stresses coincide with the entire regions of high shear stress; thus even if transverse compressive stresses did have a substantial beneficial effect on shear strength, the two types of stress are not distributed in a manner which would allow them to have an overall effect on the beam.

b) Except for extremely localized shear stress concentrations which occurred in the immediate vicinity of the applied loads, the midheight shear stresses are greater than the shear stresses elsewhere. "Two-beam" action and its corresponding reduction in maximum shear stress was not detected in these analyses of glued-laminated beams, nor would one expect this action to be present: Newlin, et al. (26) studied severely checked beams and clearly intended their findings to apply only to beams for which severe checking is present or is expected to occur. Properly made glued-laminated timber does not fit this description (this was verified by Huggins and Aplin (13) insofar as creosoted beams are concerned). Thus it appears that the findings of Newlin, et al., have been misapplied in design codes, which permit glued-laminated beams to be designed using Newlin's method and thus incorrectly imply that "two-beam" action could occur in unchecked glued-laminated beams.

c) There is some reduction in maximum midheight shear stress as the a/d ratio decreases (as low as 1.3 V/A when a/d=1, where V is the transverse shear force and A is the cross-sectional area of the beam), but this diminution is not nearly large enough to explain the behavior seen in Figure 1.

Size Effects

Size effects in timber mechanics have been documented for bending strength (5, 7) and for tension strength perpendicular to grain (23). The general effect is that strength increases as the size of the specimen decreases. It was decided to attempt to quantify a single-size effect for the shear strength of dry Douglas-fir which would be applicable regardless of how the shear strength was determined. The single geometric parameter which appeared to be appropriate to all types of test specimens was the sheared area, A_s, which is defined here as that portion of the shear failure surface of the specimen on which the maximum shear stress was acting prior to failure. In the case of beams this is taken to be the product of the beam width and the shear span a. The literature was searched, some tests were carried out, and several investigators were contacted for unpublished data; the shear strength test results uncovered were: 124 glued-laminated beams (4, 30, 31, 34, 35), 25 large sawn beams (16, 21), 28 small clear beams (3, 17), 19 notched joists (32), 441 ASTM shear blocks (18), 154 glued shear blocks (28), 30 panel-shear specimens (27), nine connector joint tension specimens (15), and 22 torsion tubes (17).

For the glued-laminated beams, arithmetic and semilog regression analyses were carried out and the better fit was given by:

$$\hat{\tau}_{max}\text{ (psi)} = 675-439 (\log A_s-2) \qquad [8]$$

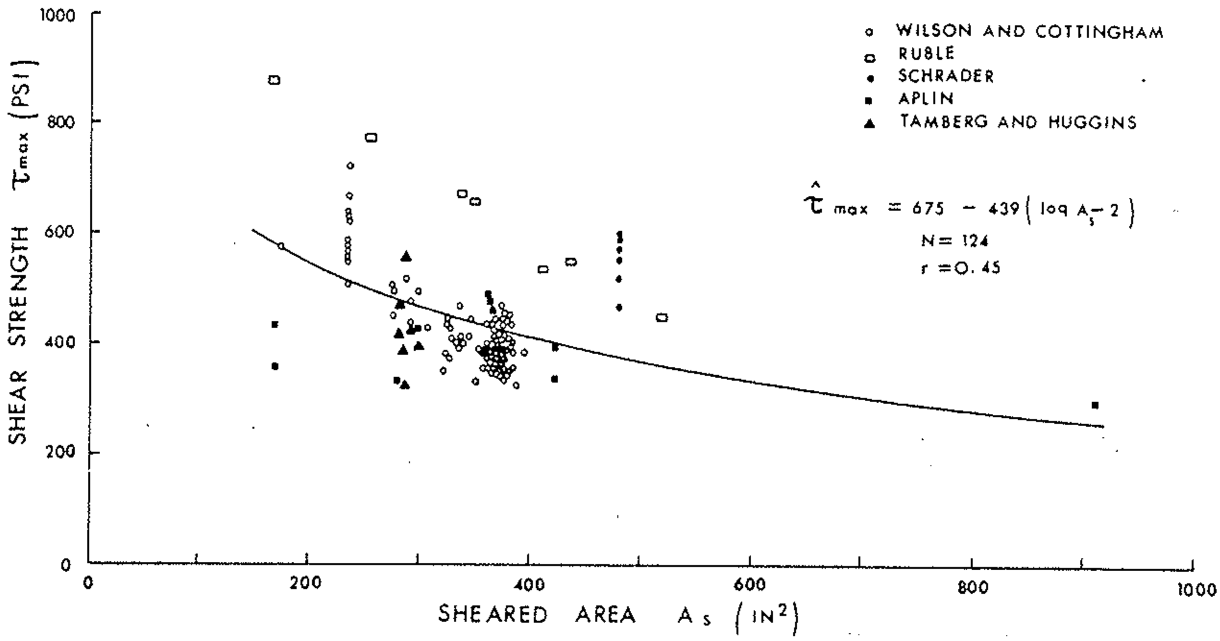


Figure 5. — Relationship between the shear strength of Douglas-fir glued-laminated beams and sheared area A_s. The solid line is a plot of the semilog regression equation.

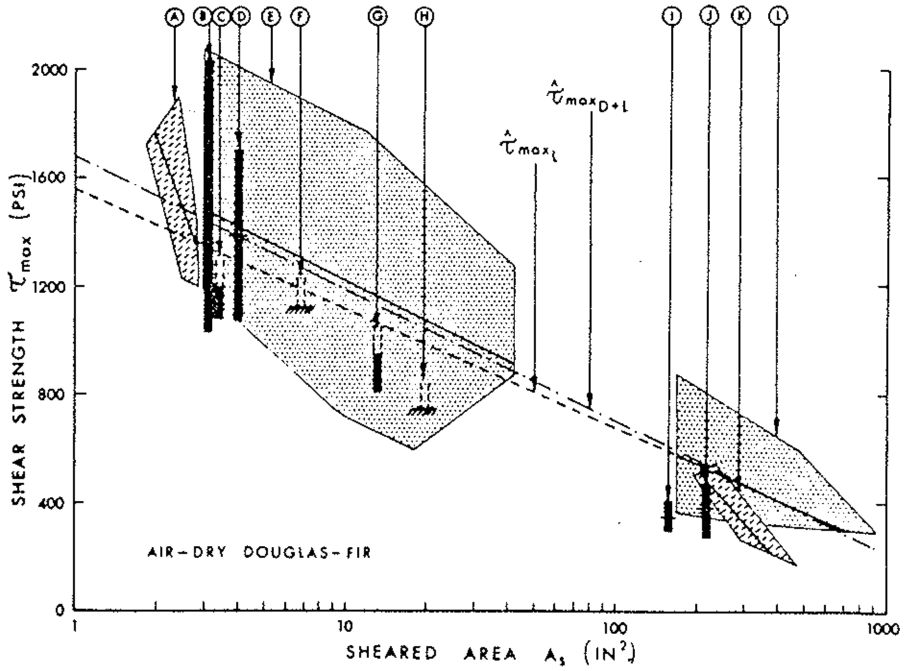


Figure 6. — Relationship between shear strength of dry Douglas-fir determined by various methods and sheared area A_s. The shaded polygons represent the scatter of test results for which the solid lines are plots of the regression equations. The short horizontal lines on the vertical bands are the means of the bands which represent the spread of test results at a particular A_s value.

A - 22 torsion tubes (17)
B - 30 panel shear specimens (27), adjusted to Kennedy (18)
C - 10 small beams (17) - band of test results are lower bound data
D - 441 ASTM shear blocks (18) - the 90 percent confidence interval is shown
E - 154 glued shear blocks (28)
F - 14 small beams (3) - lower bound data
G - 4 small beams (17) - band of test results are lower bound data
H - 19 notched joists (32) - lower bound data
I - 4 sawn beams (16, 21)
J - 21 sawn beams (16, 21)
K - 9 connector joints (15)
L - 124 glued-laminated beams (4, 30, 31, 34, 35) (Fig. 5)
$\hat{\tau}_{max_L}$ is regression Equation [8] or [9] for glued-laminated beams (group L specimens).
$\hat{\tau}_{max_{D+L}}$ is Equation [10] or [11] which joins ASTM shear block results (group D) and the glued-laminated beam results (group L).

as shown in Figure 5. The correlation coefficient was 0.45 and the regression was significant at the 1 percent level of probability.

If we then plot against A_s, as in Figure 6, all shear strength data obtained for dry Douglas-fir, there appears to be a definite continuity of behavior; i.e., shear strength seems to be generally related to A_s. Furthermore, the regression equation found for glued-laminated beams, when extrapolated back over two orders of magnitude of A_s, appears to run right through the collection of shear strength data plotted in the figure.

The regression line, Equation [8], was written in the form

$$\hat{\tau}_{max}\ (\text{psi}) = 675-439\ (\log A_s-2)$$

to reflect its applicability to glued-laminated beams. Since it now appears to be more generally applicable, it can be rewritten as:

$$\hat{\tau}_{max}\ (\text{psi}) = 1{,}553-439 \log A_s \qquad [9]$$

It was encouraging to find this relationship between shear strength and A_s over such a wide domain of A_s values and to see that Equation [9] might not be a bad first approximation of the mean size effect. However, recalling that the ASTM shear block results (group D specimens in Fig. 6) form the benchmark for determining allowable shear stresses for beams, it might be useful to alter the regression line slightly to reflect the relationship between ASTM shear blocks and glued-laminated beams. To do so, a line was drawn between the following two points:

(i) $\tau_{max} = 1{,}382$ psi @ $A_s = 4$ in.^{2} (ASTM shear block mean strength)
(ii) $\tau_{max} = 431$ psi @ $A_s = 360$ in.^{2} (strength corresponding to the median glued-laminated beam A_s value)

The equation of this line is:

$$\hat{\tau}_{max}\ (\text{psi}) = 1{,}673-486 \log A_s \qquad [10]$$

and it can be seen that it is not too different from Equation [9]; all the same, it appears to fit the plotted data at the left end of the graph a little better and yet deviates from the glued-laminated beam regression line at the right end of the graph at most by about the thickness of a pencil line. To emphasize the relationship of this estimating equation to the ASTM shear block results (for which A_s=4 in.^{2}), Equation [10] can be rewritten as:

$$\hat{\tau}_{max}\ (\text{psi}) = 1{,}382-486 \log \left(\frac{A_s}{4}\right) \qquad [11]$$

This equation is offered as a first approximation of the mean size effect for the shear strength of dry Douglas-fir in the range 1 in.$^2 < A_s < 1{,}000$ in.^{2}.

Conclusions

1) The apparent dependence of the shear strength of Douglas-fir glued-laminated timber beams on the shear span-to-depth ratio (a/d) does not appear to be satisfactorily explained by either the presence of compressive stresses perpendicular to grain or by "two-beam" action, since:
   a) when a shear failure in dry Douglas-fir is free to seek its plane of greatest weakness, as is usually the case in glued-laminated beams, its shear strength has been found to be not significantly increased by the application of compressive stress perpendicular to grain, and,
   b) finite-element stress analyses of unchecked Douglas-fir glued-laminated beams revealed shear stress distributions, for most of the a/d domain in question, approximately the same as those predicted by the usual equations of mechanics, and no indication of "two-beam" action was found.
2) There appears to be a general relationship between the shear strength of dry Douglas-fir, regardless of how it is determined, and "sheared area," A_s. For beams, A_s is the product of the beam width and the shear span. The general trend is approximately given by:

$$\hat{\tau}_{max}\ (psi) = 1{,}382-486 \log \left(\frac{A_s}{4}\right)$$

and this equation is offered as a first approximation of the mean size effect for the shear strength of all dry Douglas-fir shear test specimens in the range 1 in.$^2 < A_s < 1{,}000$ in.^{2} Thus it is suggested that the a/d effect is actually just one manifestation of this overall size effect.
3) From the above, it is suggested that "two-beam" shear behavior is not applicable to Douglas-fir glued-laminated timber beams and that consideration should be given to modifying design codes to reflect this; in fact, the assumption of "two-beam" action in conjunction with large A_s values may lead to unsafe situations. On the other hand, members which have low A_s values may be designed too conservatively.

Literature Cited

1. Al-Dabbagh, A., J. R. Goodman, and J. Bodig. 1972. Finite element method for wood mechanics. J. Str. Div. ASCE 98 (ST3): 569-586.
2. American Society for Testing and Materials. 1968. Standard methods of testing small clear specimens of timber. ASTM standard D143. *In* Part 16, 1968 book of ASTM standards. Philadelphia, Pa.
3. Aplin, E. N. 1965. Shear tests of small Douglas-fir beams. Unpublished data.
4. ———. 1973. Douglas-fir glulam beams shear failure stresses. Personal communication.
5. Bohannan, B. 1966. Effect of size on bending strength of wood members. USDA Forest Serv. Res. Paper FPL 56. Forest Prod. Lab., Madison, Wis.
6. Canadian Standards Association. 1970. Code of recommended practice for engineering design in timber. CSA Standard 086-1970. Rexdale, Ontario.
7. Comben, A. J. 1957. The effect of depth on the strength properties of timber beams. Forest Prod. Research Lab. Special Report No. 12. HMSO, London, England.
8. Ergatoudis, I., B. M. Irons, and O. C. Zienkiewicz. 1968. Curved isoparametric "quadrilateral" elements for finite element analysis. Int. J. Solids and Structures, Vol. 4.
9. Etherington, M. D., R. J. Pick, and D. J. Burns. 1972. Finite element analysis of laminated wood structures. Univ. of Waterloo, Waterloo, Ontario, Canada.
10. Frind, E. O., and A. Berndl. 1970. ISOQUAD: a program for plane stress analysis with mixed isoparametric quadrilateral elements. Unpublished documentation. Dept. of Civil Engineering, Univ. of Toronto, Toronto, Ontario.
11. Hilbrand, H. C. 1964. Comparison of block shear methods for determining shearing strength of solid wood. FPL Report 030, Madison, Wis.

12. HOOLEY, R. F., and P. D. HIBBERT. 1967. Stress concentrations in timber beams. J. Str. Div. ASCE: April 1967.
13. HUGGINS, M. W., and E. N. APLIN. 1965. Study of checking and delamination in glulam bridge members. Engineering J. (June) pp. 44-48.
14. ________, J. H. L. PALMER, and E. N. APLIN. 1966. Evaluation of the effect of delamination. Engineering J. (Feb) pp. 32-41.
15. ISYUMOV, N. 1965. Tension tests of multi-connector joints. Unpublished data.
16. JESSOME, A. P. 1971. The bending strength of lumber in structural sizes. Publ. No. 1305. Canadian Forestry Serv., Dept. of Environment, Ottawa, Ontario.
17. KEENAN, F. J. 1973. Shear strength of glued-laminated timber beams. Ph.D. thesis, Dept. of Civil Engineering, Univ. of Toronto, Toronto, Ontario.
18. KENNEDY, E. I. 1965. Strength and related properties of woods grown in Canada. Dept. of Forestry Publication No. 1104, Forest Prod. Research Branch, Dept. of Forestry, Ottawa, Ontario.
19. KENNEDY, R. W. 1968. Wood in transverse compression. Forest Prod. J. 18(3):36-40.
20. KOLLMANN, F. F. P. 1959. The problem of transverse compressive strength of timber. Translated by DSIR (Great Britain) from Holzforschung und Holzverwertung 11(5):109-121.
21. LITTLEFORD, T. W. 1968. Relationship between horizontal shear strength and modulus of elasticity. Internal report VP-24. Dept. of Forestry and Rural Dev.
22. MADSEN, B. 1972. Duration of load tests for dry lumber subjected to shear. Structural Research Series Report No. 6. Dept. of Civil Engineering, Univ. of British Columbia, Vancouver, B.C.
23. ________. Duration of load tests for wood in tension perpendicular to grain. Structural Research Series Report No. 7. Dept. of Civil Engineering, Univ. of British Columbia, Vancouver, B.C.
24. MAKI, A. C. 1968. Finite element techniques for orthotropic plane stress and orthotropic plate analysis. Paper FPL 87. USDA Forest Service. Forest Prod. Lab. Madison, Wis.
25. MANDERY, W. L. 1969. Relationship between perpendicular compressive stress and shear strength of wood. Wood Sci. 1(3):177-182.
26. NEWLIN, J. A., G. E. HECK, and H. W. MARCH. 1934. New method of calculating longitudinal shear in checked wooden beams. Trans. ASME:739-744.
27. NORRIS, C. B. 1957. Comparison of standard block-shear test with the panel-shear test. Forest Prod. J. 7(9):299-301.
28. OROSZ, I. 1962. Unpublished data.
29. RHUDE, M. J. 1950. A survey of methods for making shear tests of wood. M.S. thesis, Civil Engineering, Univ. of Wisconsin, Madison, Wis.
30. RUBLE, E. J. et al. 1962. Laboratory investigation to determine static and repeated load strength of full size Douglas-fir glued laminated stringers. Report No. ER-26. Assoc. of Amer. Railroads, Chicago, Ill.
31. SCHRADER, O. 1945. Tests on creosoted laminated stringers. Engineering News-Record (Nov. 15) pp. 80-83.
32. SENFT, J. F., and S. K. SUDDARTH. 1967. Critical shear stress in light framing lumber. Forest Prod. J. 17(12):48-49. Supplemented by personal communication.
33. STRONG, J. M. 1964. The effect of delamination on the shear strength of glulam beams. M.Sc. thesis, Dept. of Civil Engineering, Univ. of Alberta, Edmonton, Alberta.
34. TAMBERG, K. G., and M. W. HUGGINS. 1960. Glued laminated timber for bridge beams. Ontario Joint Highway Research Programme Report No. 11. Dept. of Civil Engineering, Univ. of Toronto, Toronto, Ontario.
35. WILSON, T. R. C., and W. S. COTTINGHAM. 1947. Tests on glued laminated wood beams and columns and development of principles of design. Report No. R1687. USDA Forest Service. Forest Prod. Lab., Madison, Wis.
36. YLINEN, A. 1963. A comparative study of different types of shear tests for wood. The State Institute for Technical Research. Helsinki, Finland.
37. ZIENKIEWICZ, O. C., and Y. K. CHEUNG. 1967. The Finite Element Method in Structural and Continuum Mechanics. McGraw-Hill. London.

CIB-W18/9-10-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

BEAMS NOTCHED AT THE ENDS

(concerning section 5.1.1.4 of CIB Timber Code)

by

K Möhler

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PERTH, SCOTLAND
JUNE 1978

Beams Notched at the Ends

(concerning section 5.1.1.4)

For glued laminated beams rectangularly notched at the bottom side the allowable shear force acting in the remaining cross section can be calculated as follows, if there are no reinforcements :

$$V = \frac{2}{3} \cdot b \cdot h_e \cdot k \cdot f_v .$$

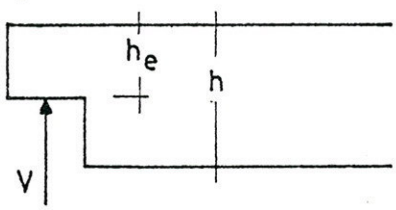
In this formula the following expressions stand for

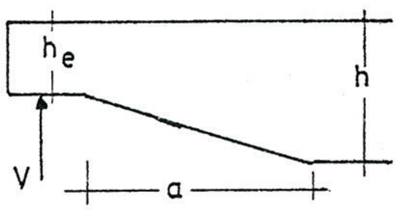
f_v : allowable shear stress

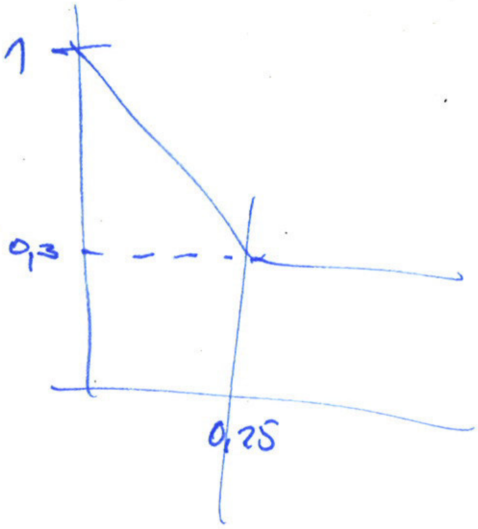
$k = (1 - 2{,}8 \frac{h - h_e}{h}) \geqq 0{,}3$ deminuation value due to the simultaneously acting of shear stresses and of tensional stresses perpendicular to the grain.

0,75h

For not rectangular notches with $a \geqq 2{,}5 \cdot h$ or $a \geqq 14 \cdot (h - h_e)$ the value "k" can be set equal "1" .







CIB-W18/9-11-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CLIMATE CLASSES FOR TIMBER DESIGN

by

F J Keenan

University of Toronto
Canada

PERTH, SCOTLAND

JUNE 1978

INTRODUCTION

At the March 13-14, 1978 meeting of the Code Drafting Sub-committee of CIB W18, the matter of "Climate Classes" was discussed. For Section 2.2 of the draft standard, the minutes of the meeting record the following:

"Dr Kuipers told the committee that in Holland they proposed to introduce six climate classes.

"Mr Curry said that four climate classes should be sufficient for world-wide use and not all of those would be necessary for Europe. Professor Keenan suggested that since the exposure of structures to changes in moisture content was very slow two or even one climate class would be sufficient.

"Dr Booth asked Professor Keenan and Dr Kuipers to produce short papers in support of their proposals. He pointed out that fibre and particle boards would respond to climatic variations more readily than solid timber."

Accordingly, I would like to submit the following comments for the consideration of the committee. Because the majority of structually designed members consist of solid wood lumber, timber or glulam, rather than composite panels, let us concentrate on the behaviour of this material and then treat the composite products as special cases. This approach will lead to a more convenient code format and ease of use than a comprehensive system which is not needed in the majority of cases.

In solid wood, there are possibly two arguments for having not more than two climate classes.

(a) The first one is that Madsen (for one) has found, in a very large testing program of structural lumber across Canada, that strength is independent of moisture content at the fifth percentile level values of the populations. Some typical data from one of Madsen's early reports is appended. The behaviour of lumber at this level of strength is most important because this is the level which provides the basis for the calculation either of allowable stresses or of characteristic strength values.

(b) In the case of glued-laminated timber in particular, structural members change their moisture content rather slowly in response to changes to environmental conditions, ie the combinations of temperature and relative humidity which produce a value of equilibrium moisture content. In other words, the moisture content variation in the member over time is considerably less than the variation in the calculated EMC values which correspond to temperature and relative humidity. Data in support of this contention is contained in the appended report by E N Aplin of the Eastern Forest Products Laboratory, Canada, "Monitoring Glulam Beam Performance for Nine Years". Aplin recorded simultaneous values of temperature, relative humidity and beam moisture content, and was thus able to compare EMC with actual MC. His data shows that as EMC varied from 4 to 16 per cent, actual MC varied only from 7 to 10 per cent although both averaged about 8 per cent.

RECOMMENDATION

Based on the above it is recommended that, for solid wood members (lumber, timber and glulam) that there be two climate classes:

(i) "Dry service condition": a condition in which the average equilibrium moisture content over a year is 15 per cent or less, and does not exceed 19 per cent*;

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*Note: a table of relative humidities and temperatures that produce equilibrium moisture contents in the range specified is also appended.

(ii) "Wet service condition": all service conditions other than dry.

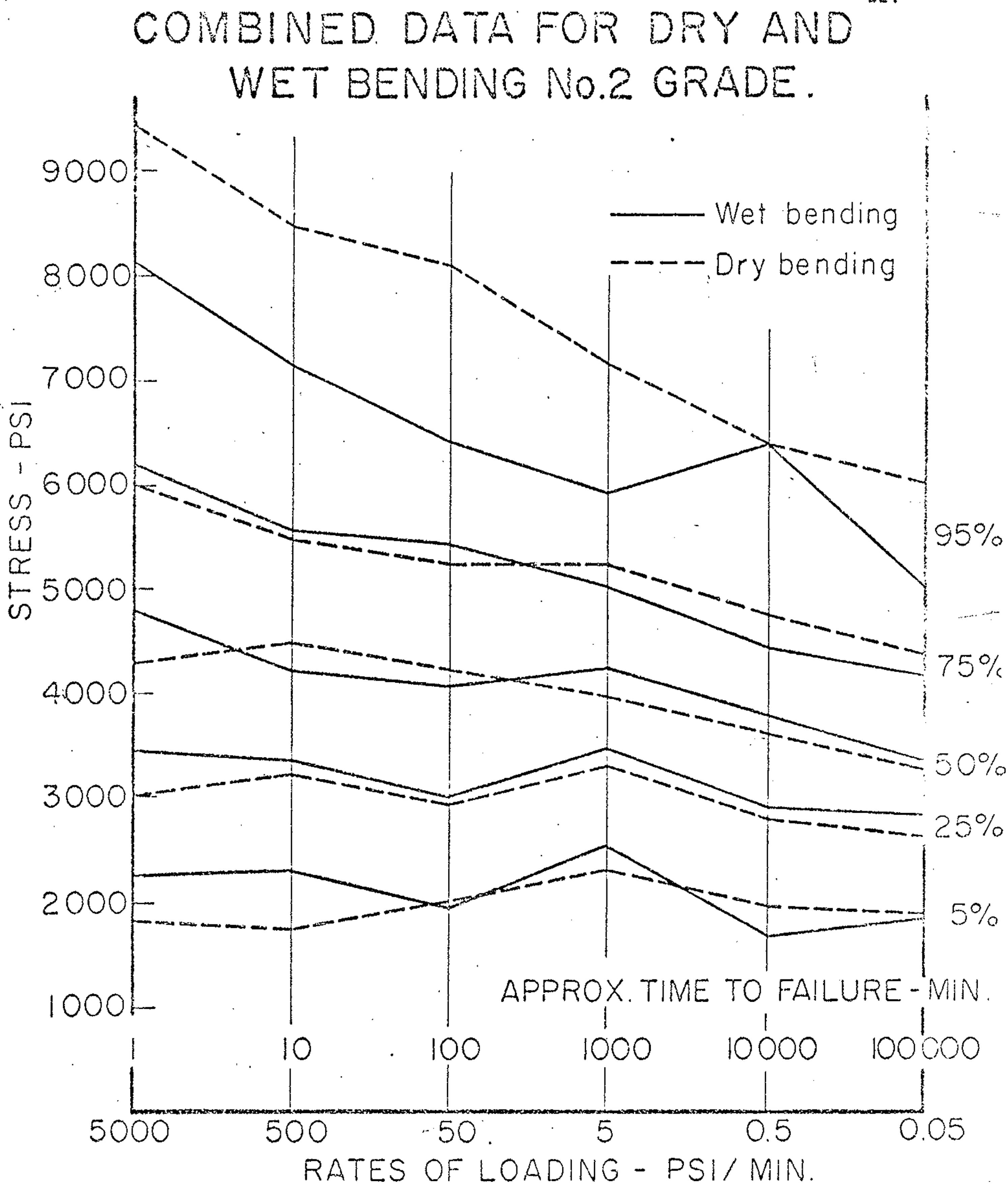


FIG. 13 STRESS vs. RATE OF LOADING.

Monitoring Glulam Beam Performance for Nine Years

by

E.N. Aplin

Eastern Forest Products Laboratory

October 1975

A programme was begun in 1964 to monitor the performance of 9 pitched-cambered glulam beams. Deflections, moisture content changes, interior building "climate", and checking in the beams have been recorded. The following is a summary of the data and was prepared for the Research Programme Committee on Wood Engineering.

When a new research sawmill was constructed in 1964 at the Petawawa Forest Experiment Station, the opportunity arose to observe the condition of the glulam beams used, starting shortly after erection and continuing, periodically, until October 1973. The structural frame consists of pitched and tapered glulam beams spaced at 15' centres, on a clear span of 58'-8", supported by glulam columns. The roof pitch is 3 in 12 and the glulam beams are 9" x 32-1/2" at the tangent points with a 32 foot radius of curvature at mid-span and a depth of 26" at the face of the columns. See photograph.

Before the first winter brass spools were fixed in position on each beam so that measurements of mid-span deflections could be made. Electrodes were placed in the laminations of one beam near mid-span so that moisture contents could be monitored using an electrical resistance moisture meter. A recording hygrothermograph was placed on a shelf close to the electrodes so that relative humidity and temperature records could be obtained and permit the calculation of equilibrium moisture content for the local "climate" the beams would be exposed to.

The beams were found to deflect only very slightly. Snow accumulations on the roof were observed to be rare due to the exposed location and the slippery roofing material used. Deflections therefore are the result of dead load and have not been found to increase with time. Except for the end wall beams one of which deflected 1/4 inch and the other which deflected 1/2 inch, the deflections of the beams have been negative, i.e., upward and the mean upward movement was 1/10 inch.

Moisture contents at depths of 1/2 inch, one inch and three inches, as determined by meter, have fluctuated with seasonal changes of relative humidity. Figure 1 shows the EMC which wood will attain (if a steady state condition of temperature and relative humidity is maintained) for the conditions prevailing near the sawmill roof plotted against time. Figure 2 shows the mean MC at 1/2 inch depth also plotted against time. Moisture content changes of about 1.5% MC were found to occur with the seasonal changes of EMC. The "core" of the beam for which MC was monitored continues to have a higher MC than the outer "shell", with the average difference being 1.8% MC.

In the course of monitoring deflections and moisture contents, our staff noticed checking developing and by 1967 it was severe enough that a programme of inspections to map crack severity was begun. A total of 175 checks have been observed in the 16 visible beam faces of the nine beams. They have been observed to open and close up again with seasonal MC changes. Figure 3 shows the change in crack severity for one check as monitored for 6 years. Several checks are rather severe; for example, the length of one check exceeds 13 feet and it has a maximum depth of 2-3/8 inch probed with a 0.004 inch feeler gauge, and an estimated average depth of 1-1/2 inch. The measure of crack severity used to present the data is the product of crack length times estimated average depth, i.e., crack area.

Figure 4 shows a histogram of the crack areas for March 10, 1967, i.e., toward the end of the heating season for that winter. Figure 5 shows comparable data for Sept. 15, 1967, i.e., before the start of the following heating season. The change reflects the tendency for cracks to close up or even to disappear with higher MC levels in the beams.

It is important to note that these nine glulam beams are in good condition except for the checking and virtually no delamination has been observed. Only the periodic inspections by ladder required to obtain data on beam deflections led to the observation that much checking had occurred and thus the need for a monitoring programme was recognized. A floor level inspection gave no indication of the extent of checking. Examination from floor level using binoculars would never reveal the existence of most checks, even some of the more severe ones. It was only because close inspection was carried out using a ladder, that this information was obtained.

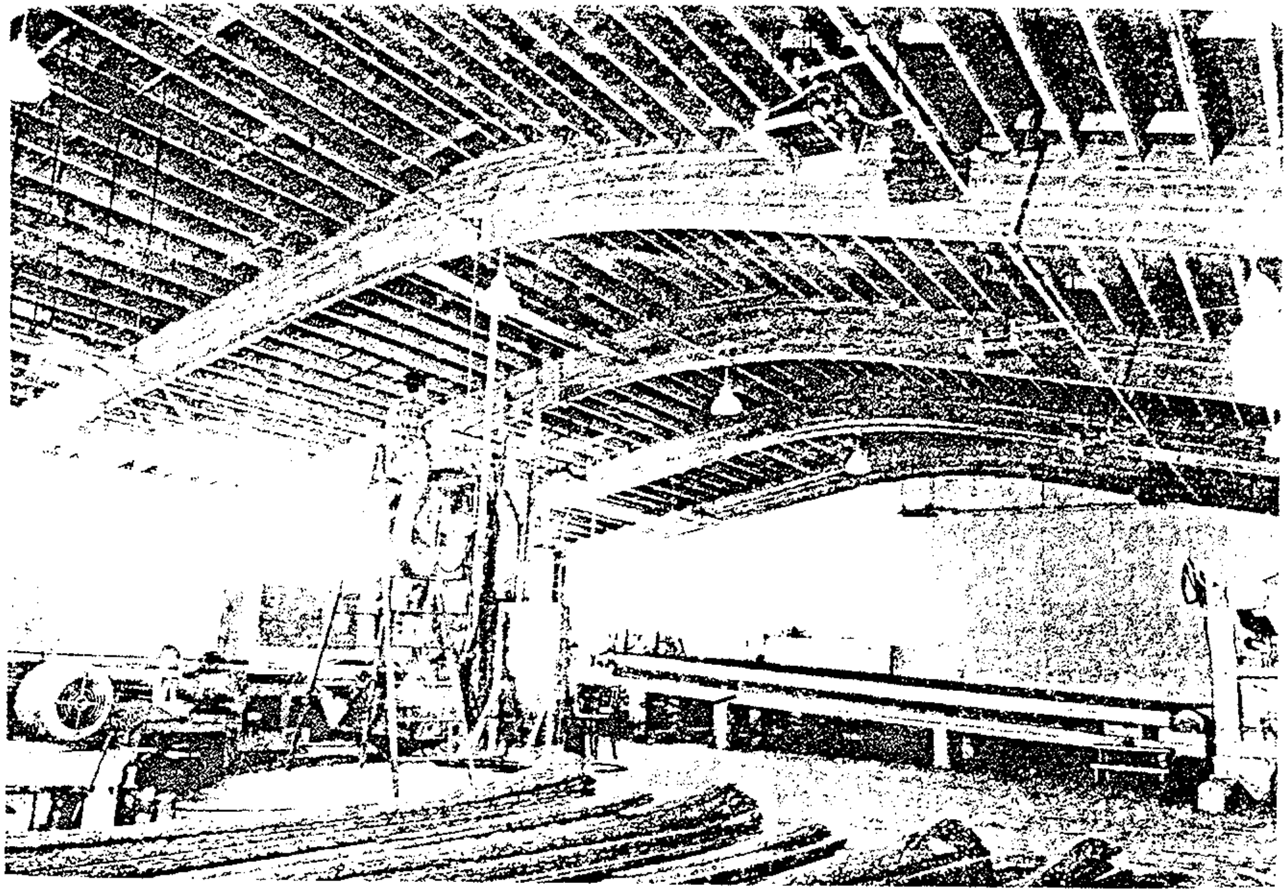
Conclusions

When inspection of glued-laminated timber structural members is required, it is essential to perform a close-up examination using a ladder or scaffolding to permit probing of checks or delamination with a feeler gauge.

Moisture content changes of 1.5% MC were found to occur with seasonal fluctuations of EMC in this sawmill. The "core" of one beam continues to have a higher MC than the outer "shell" of the beam and the difference is approximately 1.8% MC.

Acknowledgement

The efforts of Mr. S.U. Grierson and Mr. L.R. Bouchard in collecting the information reported, often under difficult working conditions, is most gratefully acknowledged.



Photograph - General view of roof structure

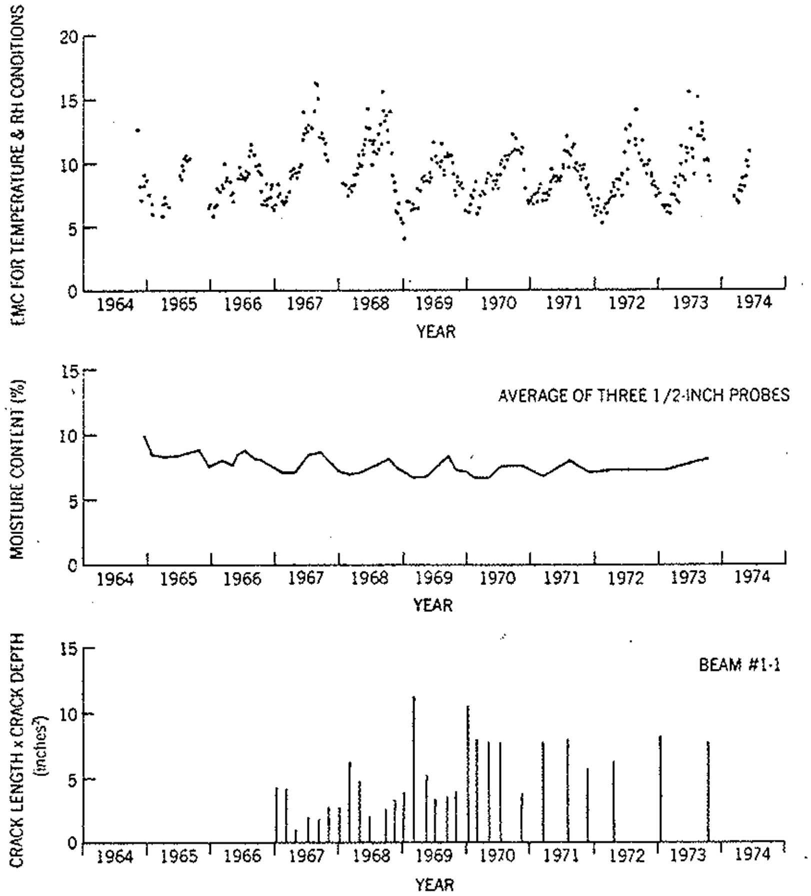


Figure 1 Top

Figure 2 Centre

Figure 3 Bottom

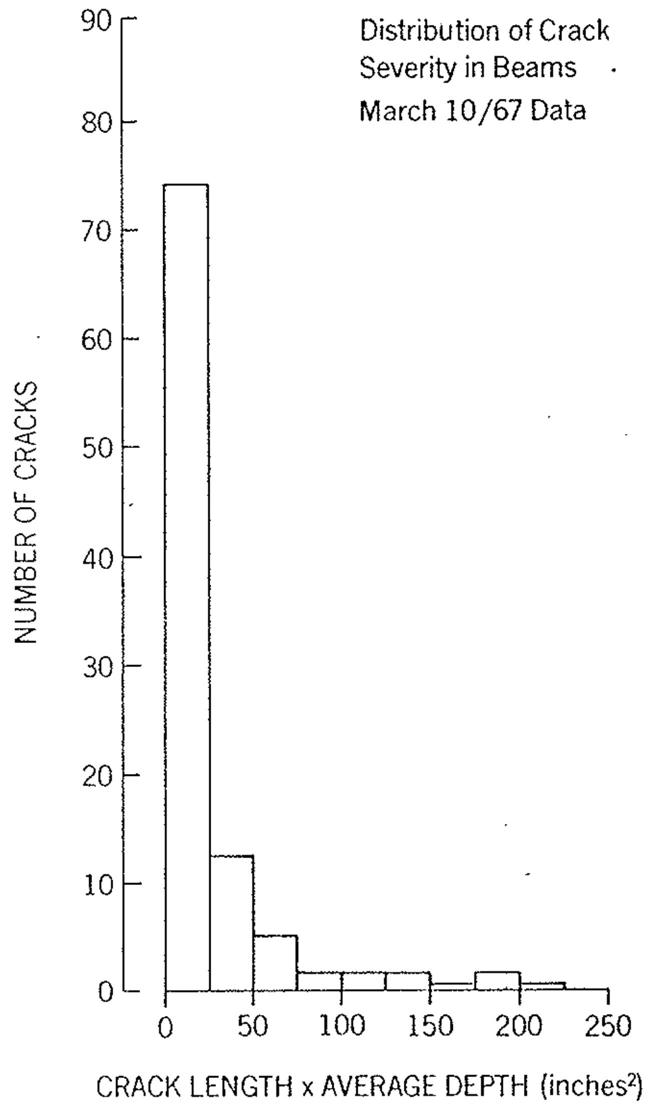


Figure 4

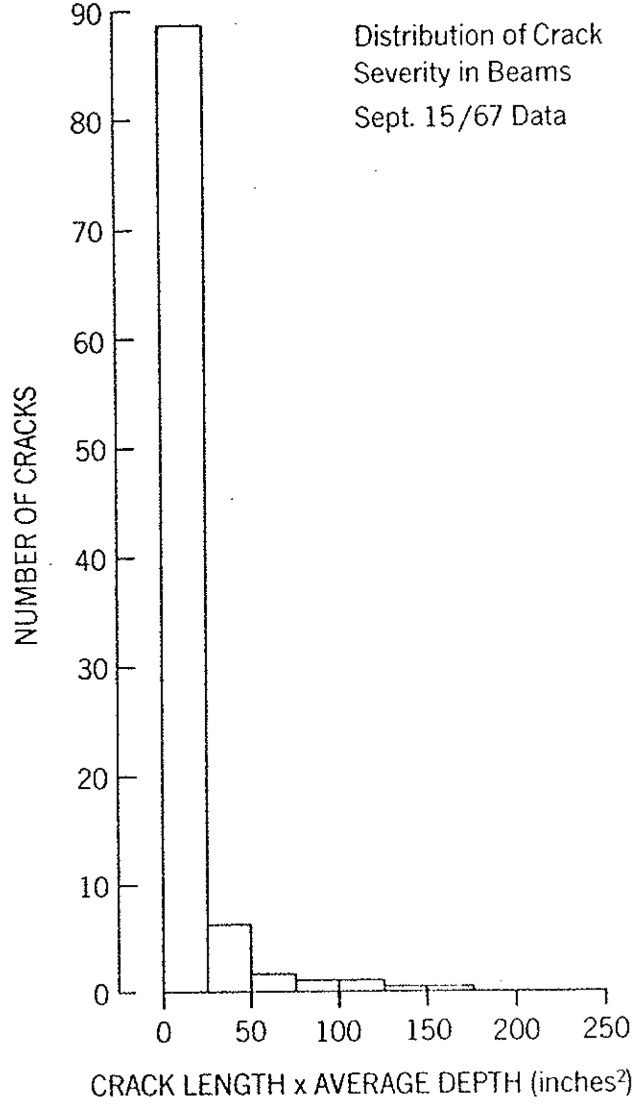


Figure 5

APPENDIX C

MOISTURE CONTENT OF WOOD IN EQUILIBRIUM WITH STATED DRY-BULB TEMPERATURE AND RELATIVE HUMIDITY

NOTE: *This Appendix is not a mandatory part of this Standard.*

TABLE C1
MOISTURE CONTENT OF WOOD* IN EQUILIBRIUM WITH STATED DRY-BULB TEMPERATURE AND RELATIVE HUMIDITY

Dry-Bulb Temperature °C	Relative Humidity, Per Cent							
	30	40	50	60	70	80	90	98
0	6.3	7.9	9.5	11.3	13.5	16.5	21.0	26.9
5	6.3	7.9	9.5	11.3	13.5	16.5	21.0	26.9
10	6.3	7.9	9.5	11.2	13.4	16.4	20.9	26.9
15	6.2	7.8	9.4	11.1	13.3	16.2	20.7	26.8
20	6.2	7.7	9.3	11.0	13.1	16.0	20.5	26.6
25	6.1	7.6	9.1	10.9	13.0	15.8	20.3	26.4
30	6.0	7.5	9.0	10.6	12.7	15.5	20.0	26.1
35	5.8	7.3	8.8	10.4	12.4	15.2	19.6	25.8
40	5.7	7.1	8.6	10.2	12.2	14.9	19.3	25.4
45	5.5	6.9	8.3	9.9	11.9	14.6	19.0	25.1
50	5.3	6.8	8.1	9.6	11.6	14.3	18.5	24.6
55	5.2	6.6	7.9	9.4	11.3	14.0	18.2	24.2
60	5.0	6.3	7.7	9.1	11.0	13.6	17.7	23.7
65	4.8	6.1	7.4	8.8	10.6	13.1	17.2	23.1

**Values for plywood are approximately 2 per cent lower than those given in the Table.*

FA Auflagerverstärkungen

CIB-W18/9-12-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

EXPERIMENTS TO PROVIDE FOR ELEVATED FORCES AT THE SUPPORTS OF WOODEN BEAMS, WITH PARTICULAR REGARD TO SHEARING STRESSES AND LONG-TERM LOADINGS

by

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Vienna

PERTH, SCOTLAND
JUNE 1978

C O M P E N D I U M

of the final report on the research project:

" Experiments to provide for elevated forces of the supports by suitable design of the supports of wooden beams, with particular regard to shearing stresses and long-time loadings "

prepared at the " Wood Research Institute " at the University for " BODENKULTUR ", Vienna.

By Prof.Dipl.Ing.F.Wassipaul and D.Ing.Dr.R.Lackner

Vienna, April 1978

1. INTRODUCTION

After the preliminary examinations had been finished, which had the purpose to find a method to reinforce the supports of wooden beams in order to improve their resistance to transversal compression loads, some questions were still unanswered.
These examinations didn't show undoubtely, which arrangements and numbers of the reinforcing elements were the best to reduce indentation. For this reason, two test series were carried through in order to complete the preliminary examinations.
A further question was particularly the behaviour of the reinforcing elements during the static long-time test. According to DIN 1052 the strength of wooden structural elements decreases essentially with increasing duration of loading. Various explanations are given for this fact and it was attempted to determine the influence of long-time loading on the strength of reinforced wooden structural elements in comparison with unmodified ones.
Further on, it is known that the strength values obtained by testing depend substancially on the size of the samples. Since with model tests conditions may be caused by optimum choice of the wood and by optimum machining, as they as a rule cannot be achieved in practice, and further on, the so-called sample-volume (e.g.ratio of support distance to sample height or cross-section of sample) is influenced, it is necessary in most cases to carry through experiments on roughly shaped samples of commercial size in order to prove the results obtained by the model tests. For this purpose, 5 straight laminated beams made of wood, 16 cm wide, 80 cm high and 350 cm long were for their behaviour in a static long-time test with variations of their support areas.
Finally it was to be elucidated whether the reinforcing elements were able to contribute to the absorption of the often high shearing stresses, which may occur within a beam in the zone of the supports.

This effect would have to be anticipated and had been mentioned in literature. In order to prove the effect, a method decribed in literature was used: The cross-section, which is deciding for the ability to absorb shearing stresses, was reduced within the zone of the highest shearing stresses; thus an improvement of the ability to absorb shearing stresses, caused by the reinforcing elements, would have had the possibility to show itself.

2. PROGRAM:

2.1 Completing experiments to determine the optimum number of reinforcing elements.

2.2 Long-time experiments on model beams at standard climate

2.3 Short-time experiments on beams of commercial size, 16 x 80 cm, 350 cm long

2.4 Short-time experiments to determine a possible absorption of shearing stresses by the reinforcing elements.

3. PERFORMANCE of the PROJECT:

3.1 Completing experiments to determine the optimum and arrangements of the reinforcing elements.

The previous tests determining advantageous distances between the dowels didn't reveal undoubtedly, which distances and which amount of dowels resulted in the highest improvements with respect to decrease of indentation.
Therefore, another test series was carried through in sections of industrially manufactured beams (provided by " Österreichischer Leimbauverband "). With constant distance of the supports the number of dowels in longitudinal and transverse direction was varied. The tests were carried out in two parts at the " Technische Forschungs-und Materialprüfungsanstalt " of the Technical University of Vienna.

The arrangements of the dowels in three support areas were recorded respectively.
The distances of the dowels in longitudinal and transversal directions were arranged in the support area sized 12 x 10 cm in the following way :

The choosen longitudinal distances from centre to centre of the dowels (diameter 14 mm, diameter of the drilled hole 14.5 mm) were:

$l_1 = 3$ cm
$l_2 = 4$ cm
$l_3 = 5$ cm
$l_4 = 6$ cm

The respective transversal distances were:

$b_1 = 3$ cm
$b_2 = 4$ cm
$b_3 = 5$ cm
$b_4 = 6$ cm

Combinations of these distances resulted in the following arrangements of the dowels: (Fig. 1a).

Fig. 1a : Variations of the arrangements of the dowels

1. test series

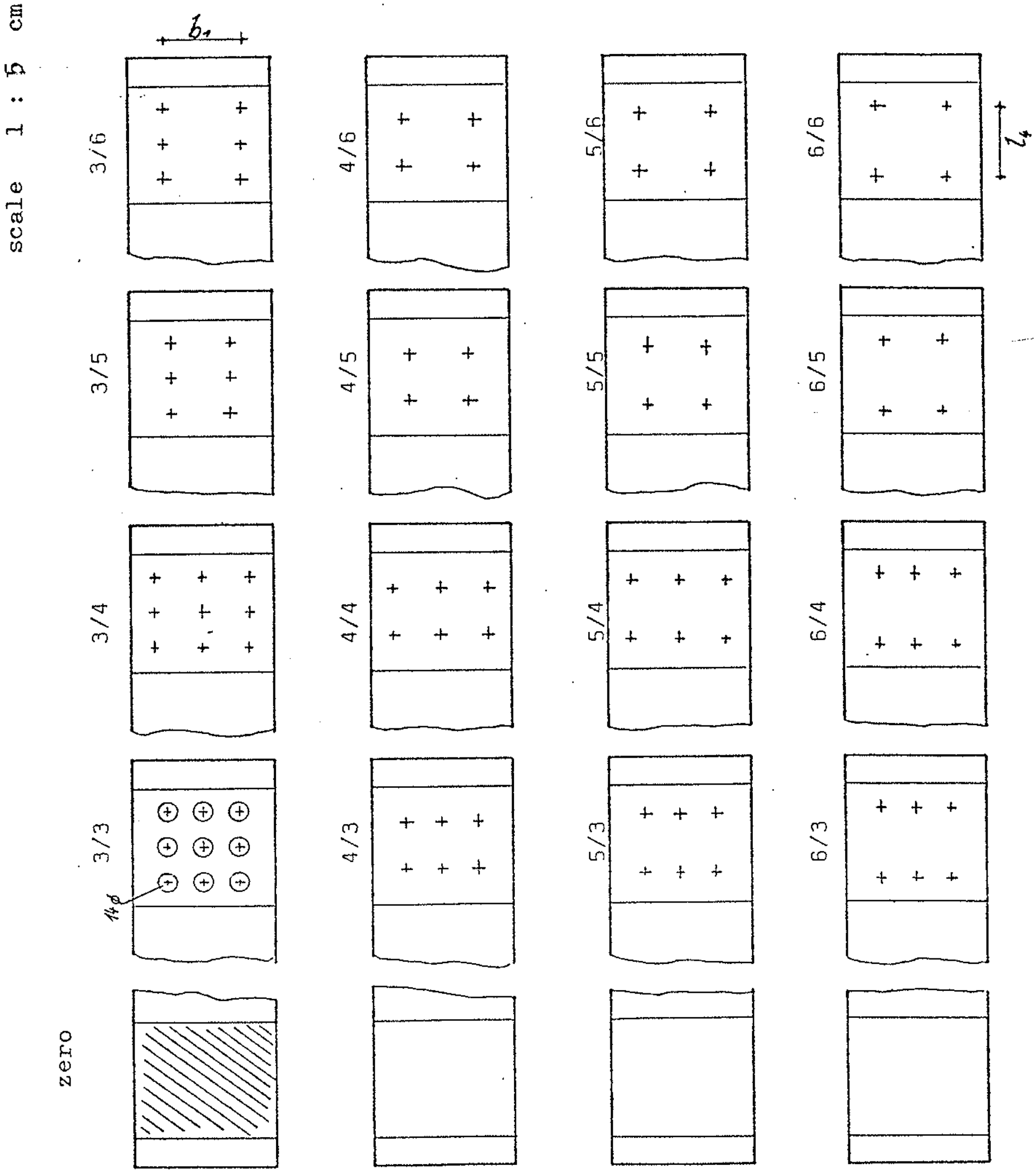
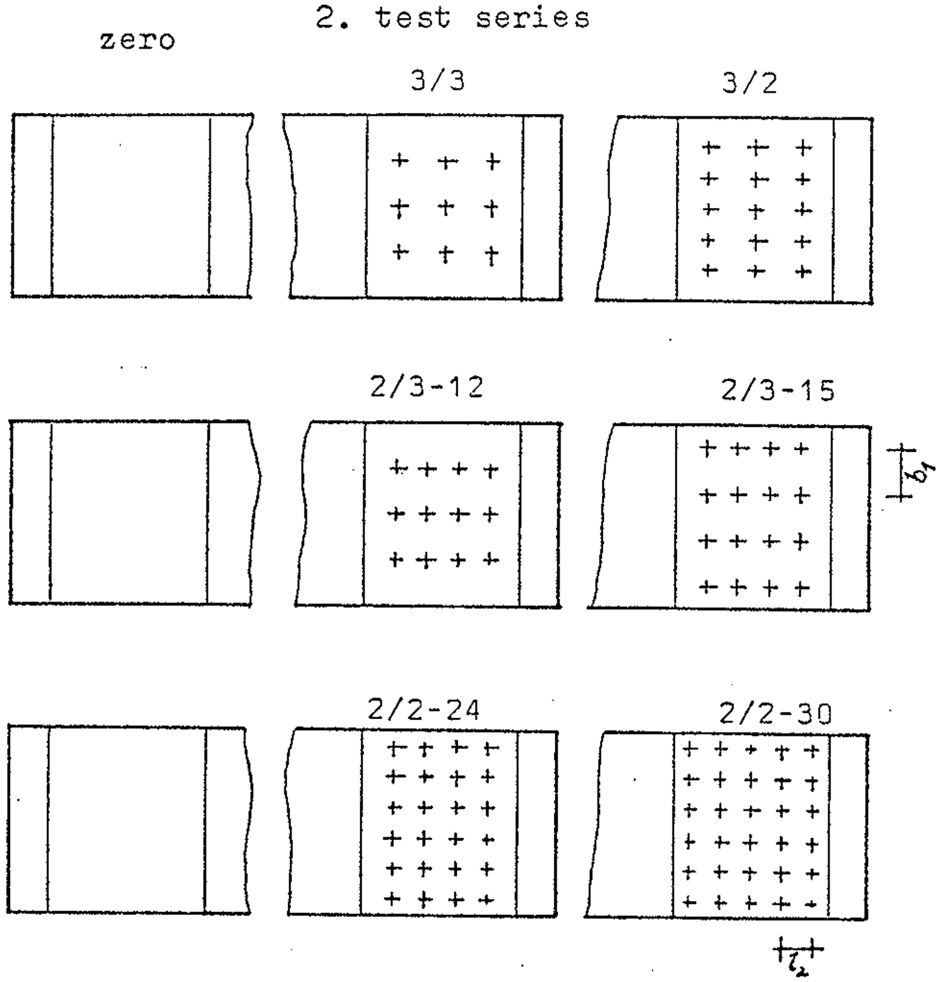


Fig. 1 b: Variations of the arrangements of the dowels

2. test series



l/b -	number of the dowels		
3/3 - 9	4/3 - 6	5/3 - 6	6/3 - 6
3/4 - 9	4/4 - 6	5/4 - 6	6/4 - 6
3/5 - 6	4/5 - 4	5/5 - 4	6/5 - 4
3/6 - 6	4/6 - 4	5/6 - 4	6/6 - 4

whereby the first number gives the distance between the dowels in longitudinal direction,($l_1 \dots l_4$), the second number gives the transversal distance ($b_1 \dots b_4$) and the third number gives the numbers of dowels (14 mm diameter) within the area sized 120 cm^2 .

In addition to this test arrangement another two areas without reinforcing elements were examined as controls (" zero-areas") thus showing the achievable unprovement by reduction of the indentation.

The improvements were shown by comparison of the indentations on the non- reinforced supports at a permissible transversal compression stress of $\sigma_{perm\, d\perp} = 25\ kp/cm^2$ and the loads resulting in the same indentations on the reinforced supports. (fig. 2).

The improvements varied, depending on the numbers of dowels, within the range from 40 to 80 %, that means, in place of the permissible transversal compression stress of 25 kp/cm^2 now 35 - 45 % caused the same indentation, whereby considerable deviations may occur within one series.

The influence of the distances between the dowels could not be evidenced indoubtely, although fig. 3 shows a certain dependance of the amounts of the improvements on the arrangement of the dowels.

Fig. 2 : Stress- deformation-diagram (schematical) to dtermine the percentage of improvement (VERBP)

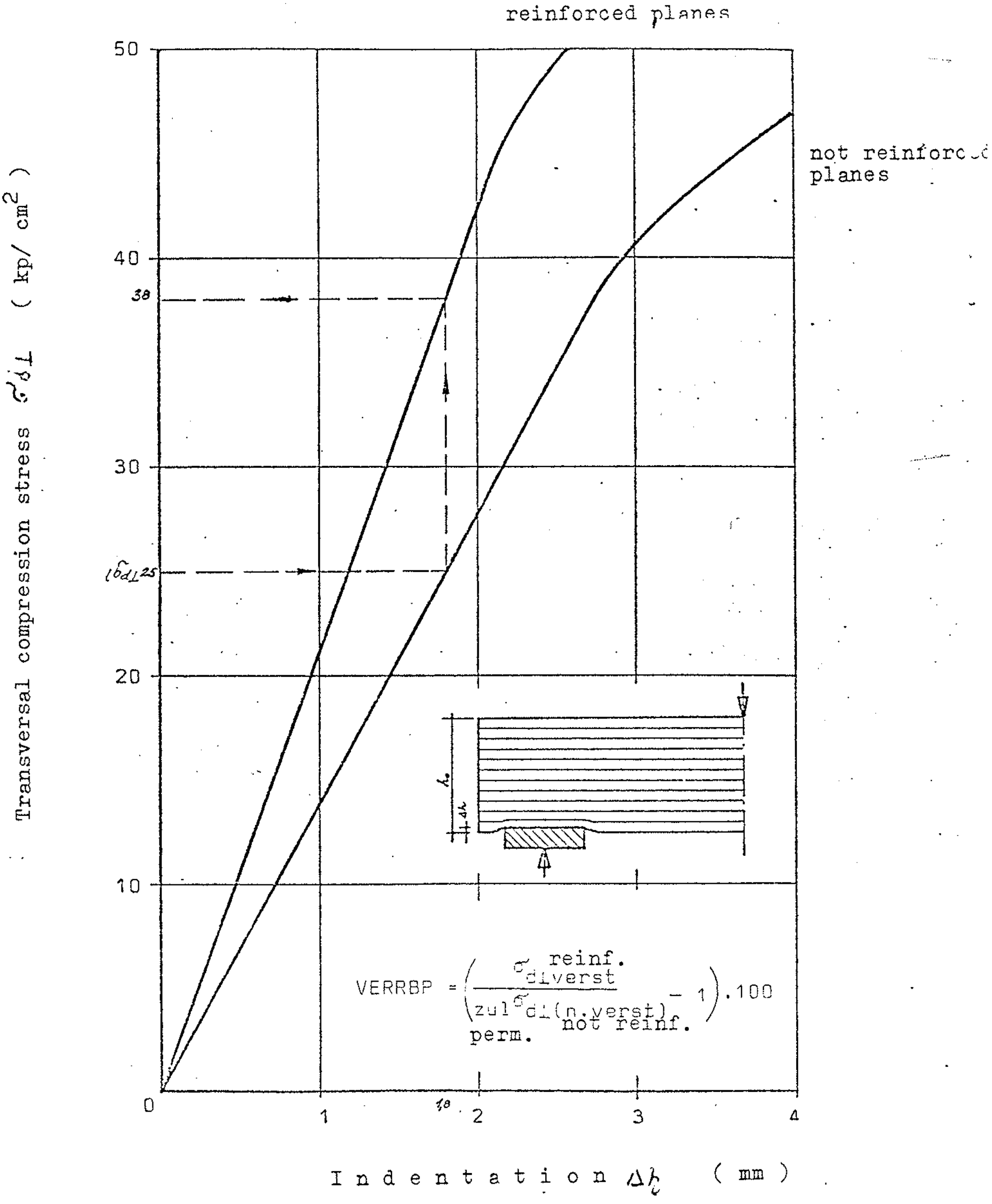
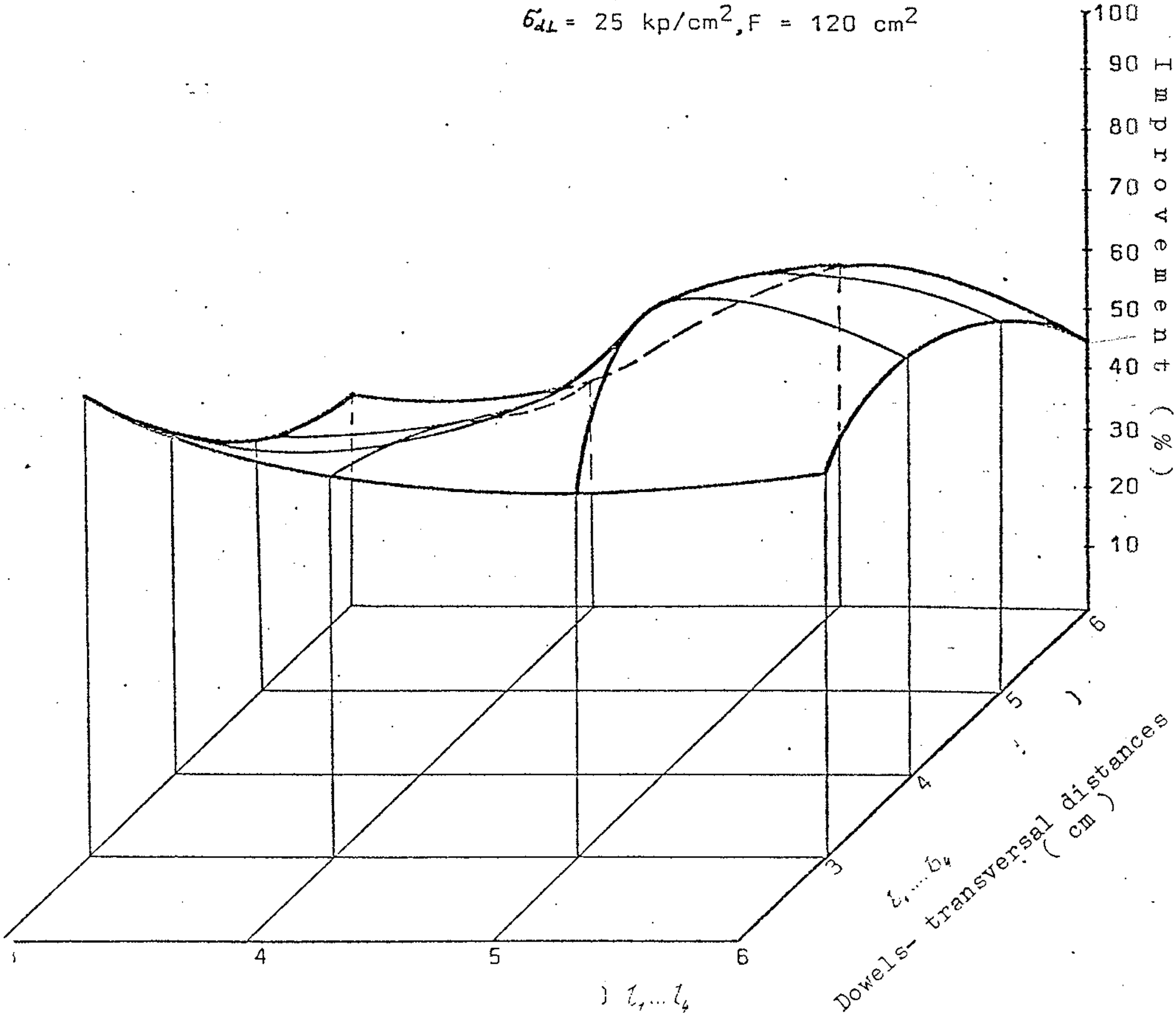


Fig. 3 : Percentage of improvement (VERBP) at the support, depending on the arrangement of the dowels (centre of dowel-centre of dowel_ with 14 mm - dowels



The number of dowels within the load-bearing area, resp.their added areas as percentage of the total support area, has turned out to be an essentially more significant factor than the distanced between the dowels.

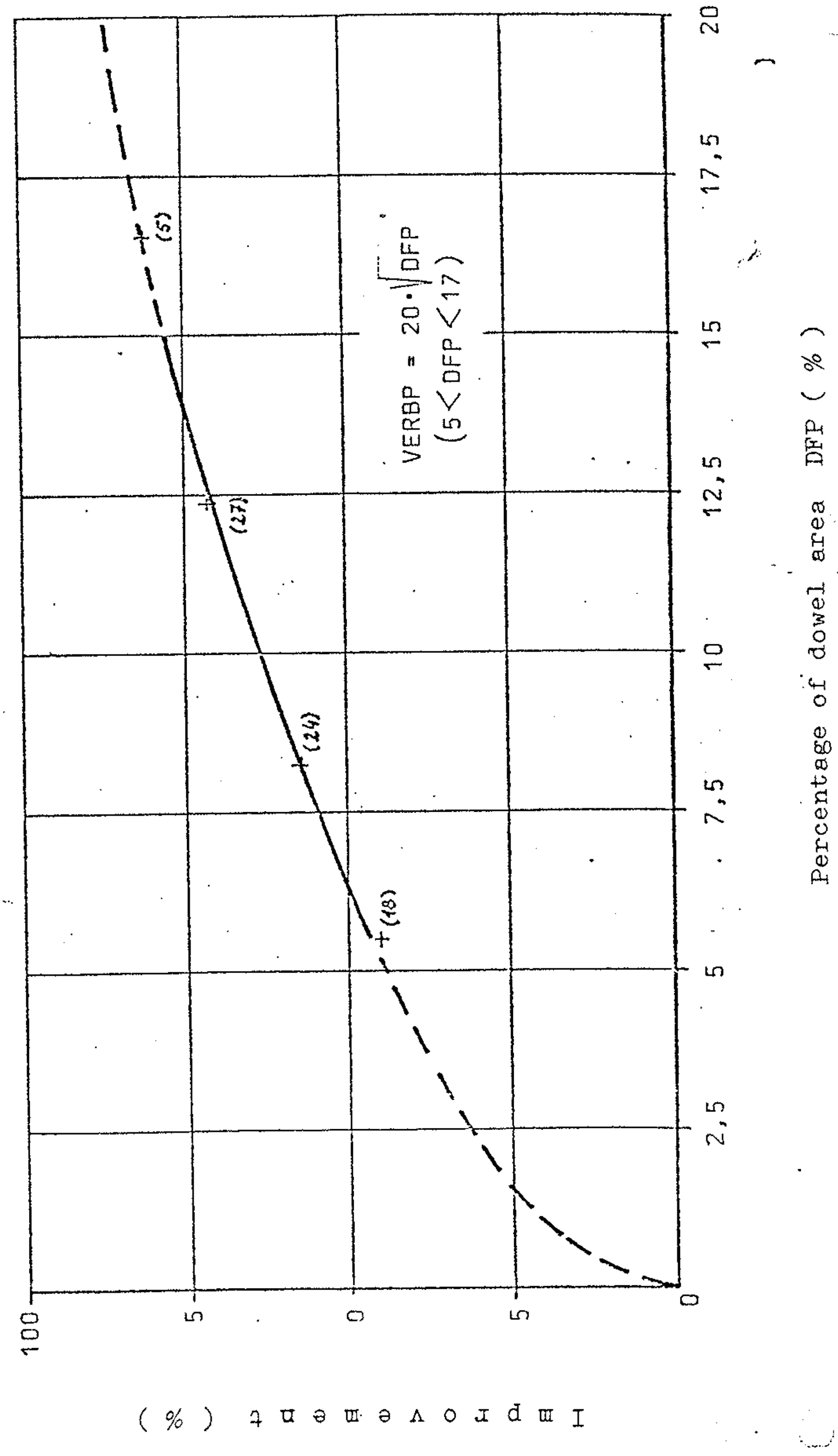
In fig. 4 these interrelationships are summarized.The percentage of the area of the dowels, related to the support area, is plotted oarallel to the absvissa (DFP), the respective, achieved improvements are plotted parallel to the oordinate. (VEB).
The numbers in parentheses beneath the test points represent the numbers of observations.

The improvements, achieved with area percentages between 5.5 % (4 dowels measuring 14 mm in diameter, within 120 cm^2 support area) and 16.5 % (12 dowels within the support area) may very well be determined approximately by means of the following parabel equation, which has been found empirically:

$$VERBP = 20 \sqrt{DFP}$$

This formula cannot be derived mathematically considering the values obtained for the individual elements in the prliminary tests, since, besides the non- uniform alterations of the prooerties of the material caused by the gluing in of the wooden dowels, the size of the sample etc., cause influences, which have not yet been examined here in detail. Further on, the test values above about DFP = 15% (more than 12 dowels within the support area) show a discontinuity causing a theoretical improvement of up to 150% and more which cannot be extrapolated from the diagram. Hence it has no significance in this range.
With regard to the danger of too great a reduction of the cross-sectional area and hence too low a capability of shearing stress absorption on one hand and to the good results of about 12% DFP, obtained by tests on commercial scale, no further investigation of the abovementioned problem (discontinuity of the curve in the diagram) above DFP = 15% was made. Consequently, for practice, a dowel area percentage of about 10 to 15% is recommended.

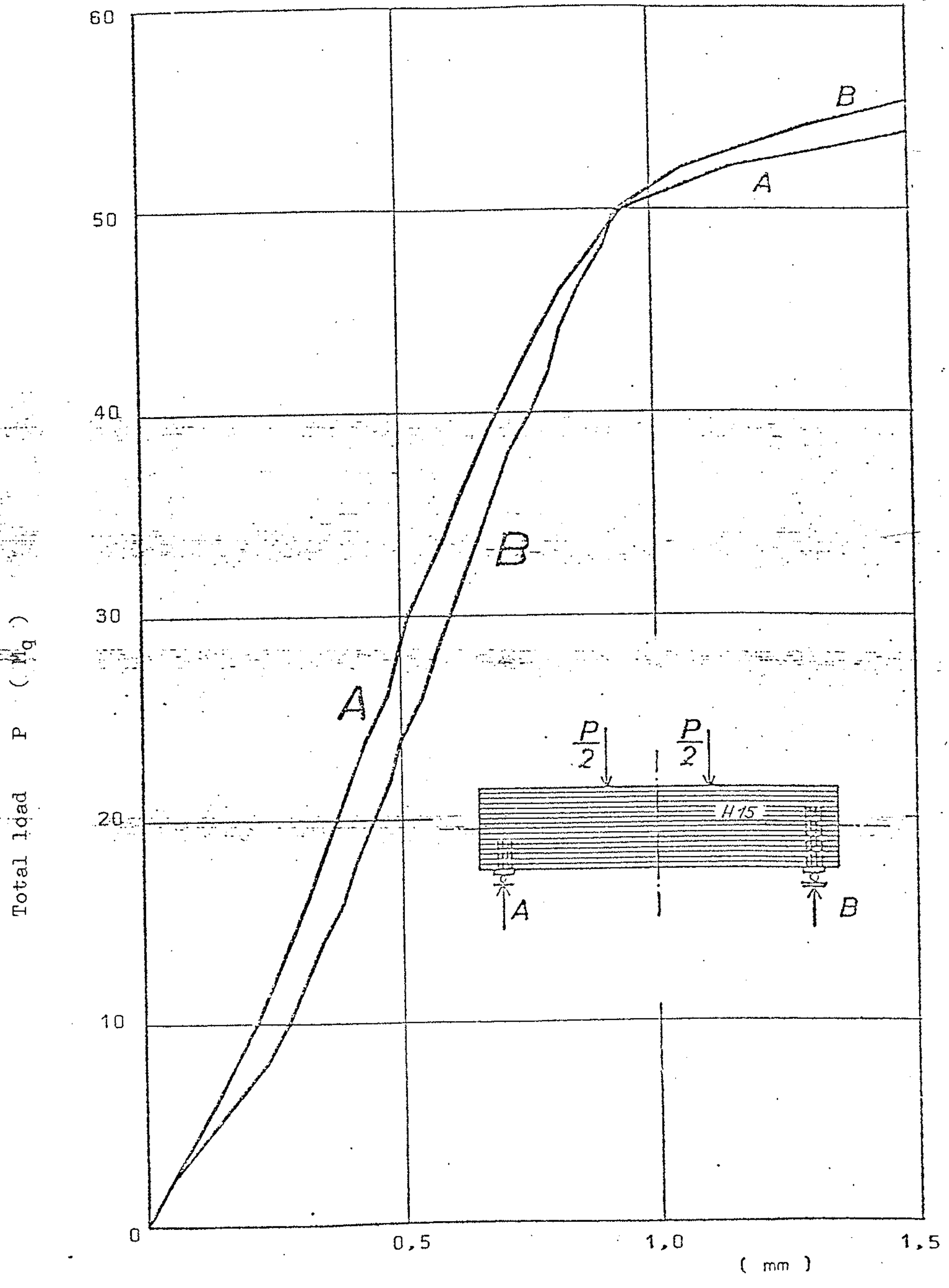
Fig. 4 : Dependance of the percentage of improvement on the percentage of dowel area



RESULTS:

1. The improvements found by model tests, were verified with tests on commercial scale, whereby partially even higher values of improvement were achieved (up to 200%).
2. There is no essential difference between the improvement with supports reinforced with dowels 30 cm long and those reinforced with dowels 60 cm long (on the basis of the same number).
   In fig.5 the direct comparison of both the supports at beam H 15 is shown. The explanation, found by Moehler (1972), for the better bearing capability of the 100 cm long steel screw as compared to the 80 cm long one could not be verified with this test.
3. Breaking occures at the reinforced support because the longitudinal compression strength of the beech wood was exceeded in a depth of about L_B = 6 cm, thus permitting the conclusion that there is a maximum of the compression stress within this range (probably a high compression of these lamellae). Current examinations on the distribution of the stresses along the dowels created by the adhesion of the dowels in the wood should aid in this context to elucidate these problems.
4. Machining in the zones of the supports was good enough to achieve improvements, despite the inaccuracies occuring in practice. Hence, the method is practically applicable.
5. The dowels should be choosen shorter than half the height of the beam with respect to the shearing stresses, because in this way a disadvantageous influence caused by reductions of the cross-sections in the zones of maxima of the shearing stresses is kept as low as possible.
6. The beams should be reinforced with steel screws if higher loads have to be absorbed because the strength of steel permits the transmission of higher forces than does the wood dowel.

Fig. 5 : Comparison of the compressions on the support A (dowels: 30 cm long)and support B (dowels 60 cm long)

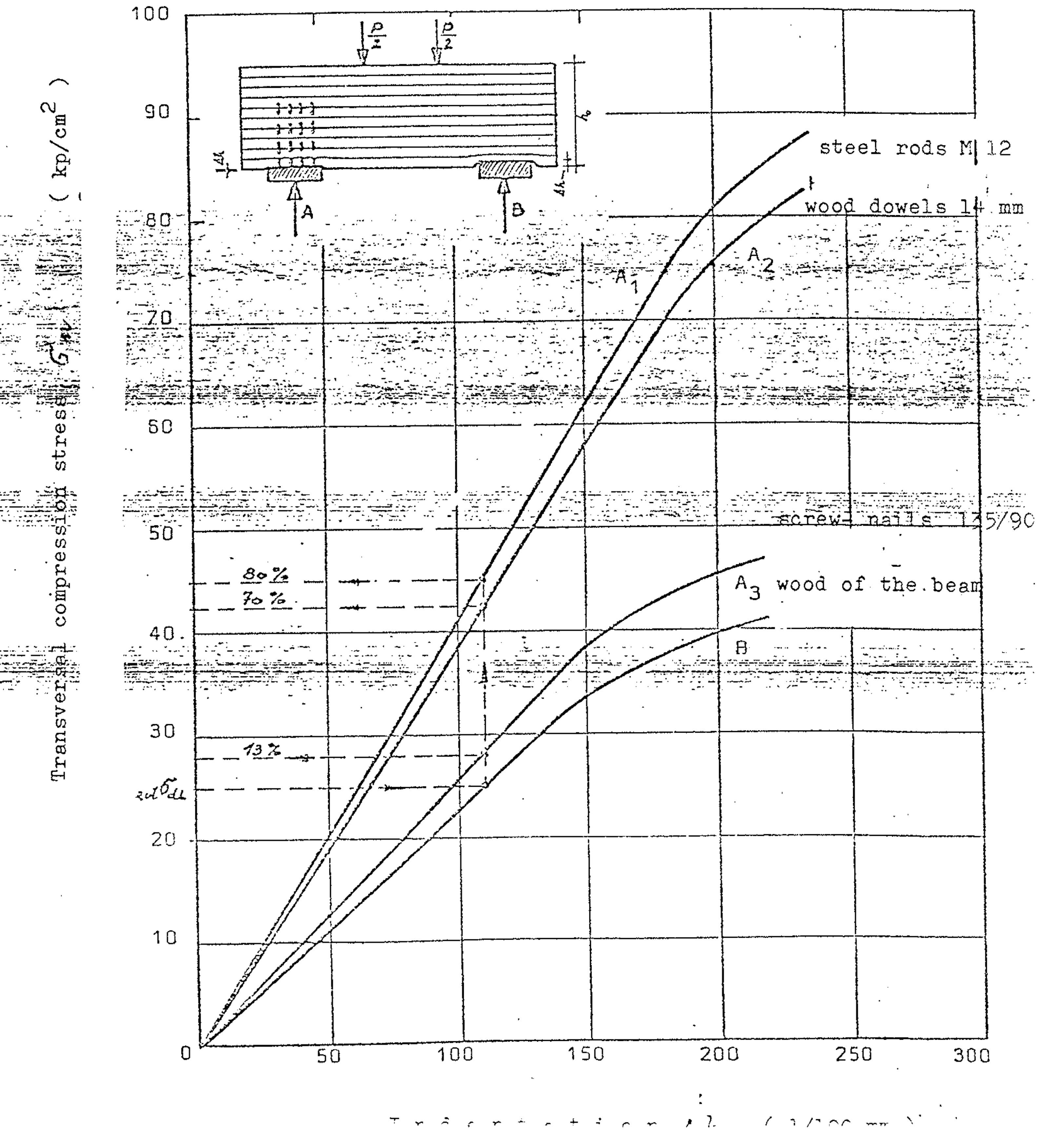


S U M M A R Y:

The low resistancy of spruce wood, which is frequently used for wood constuction, against transversal compression loads (perm. $\sigma_{d\perp}$) often causes difficulties with rating of structural elements particularly if high main loads or supplementary loads (f.e. snow and wind loads) are to be considered.

In this report which completes previous reports on the research project mentioned at the beginning, a method to reinforce the supports of laminated beams, developed at the " Wood Research Institute ", Vienna, is discribed.
For reinforcing, elements being able to absorb the forces initially, are used, chiefly nails, dowels made of steel and wood, which are arranged in the wood of the beam in the direction of the supporting forces and which lead the forces impacting at the support into the core of the beam.
The reinforcing elements were tested single and in the compound for their effectiveness, partcularly the transmission of forces, the size and arrangement of the elements in the static short-time test and with long time load were investigated.
Inserting nails with special shaped sgafts (special nails for wooden constructions) as well as inserting dowels made of those made of steel with coil profile results in a reduction of the indentations perpendicular to the grain at the supports of laminated beams made of wood.
A statement on beams made of solid wood cannot be made due to too low a number of samples.
According to ÖNORM B 4100, 2.Part (Austria) with " good timber for constructional use " compression stresses perpendicular to the grain up to $\sigma_{perm} = 25\ kp/cm^2$ are permissible, " provided shallow indentations are unobjectionable". If the indentations, caused by these stresses at the not reinforced support, are considered as a criterion, inserting the abovementioned reinforcing elements results in an improvement, i.e. an increase or equal indentation, within the ranges shown in the following.(Fig.6.Lackner, 1977)

Fig. 6 : Transversal compression stresses- deformations-diagram curves (schematic_ at the support with not reinforced (B) wood of the beam and with wood reinforced with various elements (A)



Static short-time test on model beams:

"Reinforcing elements"	Improvement	Increase of stress up to kp/cm^2
nails 34/90	10	27.5
screw-nails 35/90	to 30	32
beech wood dowels 14 ø	40 - 70 - 100	42
steel screws M 12	to 80	45
Static short-time test on beams of commercial size 16/80:		
beech wood dowel 20 ø	100	50
Static long-time test on model beams:		
beech wood dowel 14 ø	100	50
screw-nails 35/90	7 - 13 - 19	28.5

The arrangement of nails in the plane of the support is made according to ÖNORM B 4100, 2.Part.
With the dowels the improvement turned out to be dependant on the sum of the cross-sectional areas of the built-in dowels, with other words, on the percentage proportion of the entire support area they covered (DFP), whereby about 12 to 15% DFP appear to be advantaheous.
From initial tests it may be concluded that the dowels may be arranged in squares; thus the distances between the sentres of the dowels are about 2.5 times as long as the chossen diameted of the dowels.

If an uniform distribution of stresses in the glueline between the reinforcing element and the wood of the beam is assumed, an effective inserting length of L_W = 3 D (diameter of the dowel) results for beech wood dowels and of L_W = 20 D for steel dowels of 4-D-quality.

It became evident that the values obtained in the static-short-time test are also obtainable in the long-time test, resp. with beams of commercial size.
No improvement of the capability to absorb shearing stresses by gluing-in of dowels made of hardwood or steel screws, as

compared to not reinforced wood could be found, therefore the reinforcing elements should not touch the plane of the highest shearing stresses, because they may cause a reduction of the cross-sectional area absorbing the shearing stresses. However, the choosen inserting length has at least to be as much as 3 D with beech wood dowels and 20 D with steel dowels.

It will be the task of further investigations- a part of them being already under way- to dtermine accurately the distribution of stresses along the reinforcing elements, as well as to examine the behaviour of reinforced supports under extremly high variations of the moisture content.

This kind of reinforcement of the supports which has been described here, is basically different from othe methods:

It is cheap and simplre to manufacture, which is important for practical use.

The reinforcement is not visible on the beam from the outside, the outline of the construction is not altered.

The amount of improvement is high, as determined by means of static short-time tests on model an commercial beams; it may be used, after considering the safety criteria, as a basis for elevation of the standard values of the permissible transversal compression loads.

CIB-W18/9-12-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TWO LAMINATED TIMBER ARCH RAILWAY BRIDGES BUILT IN PERTH IN 1849

by

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Imperial College
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PERTH, SCOTLAND

JUNE 1978

Two laminated timber arch railway bridges built in Perth in 1849

L G Booth
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Laminated arches, in which the laminations were cut to profile and then notched or bolted to the adjacent lamination without being bent, were frequently used for bridges in Europe in the eighteenth century. The use of timber arched bridges in which the laminations were bent into position was pioneered in Europe by Wiebeking, in Bavaria during the years 1807-09. The same method was used in France for roof structures by Emy in the 1820s. In England no example of the method has been found until its development for railway bridges. (Booth, 1971a).

Laminated timber railway bridges were built by a number of different engineers for some twenty railway companies during the period 1835-50. Details of the construction and location of 34 of these bridges have been given by Booth (1971b).

Two bridges were built in Perth in 1849 and the following note may be of interest to some participants at this CIB-W18 meeting (see Figure 1).

The general method of construction, which was developed by John Green for the Newcastle and North Shields Railway in 1839, used 3 inch thick softwood laminations fastened together with oak pegs (trenails) and/or bolts. The laminations were butt jointed and an attempt, usually unsuccessful, was made to preserve them against decay.

The thrust northwards of the railways from Glasgow to Aberdeen was in the hands of Locke and Errington as Engineers to the Scottish Central Railway (Greenhill to Perth), the Scottish Midland Junction (Perth to Forfar), and the Aberdeen Railway (Forfar to Aberdeen). They were also responsible for the Dundee, Perth and Aberdeen Junction. There were nine laminated timber arch bridges on these lines, but for the purpose of this note I shall mention only two bridges.

The Dundee and Perth Railway had been opened as far as Barnhill on the north bank of the Tay in May 1847, and when the site of the General Station of all the companies was finally agreed, the Company was faced with building an extension across the River Tay into Perth. On 17 February 1849, Captain Simmons (the Government Inspector) examined the extension across the river. 444 yards of the extension were carried on timber segmental laminated arches resting on timber piles, the spans being 50 ft and rise 6½ ft. Simmons required a speed limit "in as much as there is a great deal of movement in the timber laminated arches, extending in one that I tried to a rise of ½ inch and fall of 1½ inch or a movement altogether of two inches in the passing of an engine, I should strongly recommend that some method be tried by which the structure may be stiffened, as this constant movement must materially tend to the destruction of the bridge".

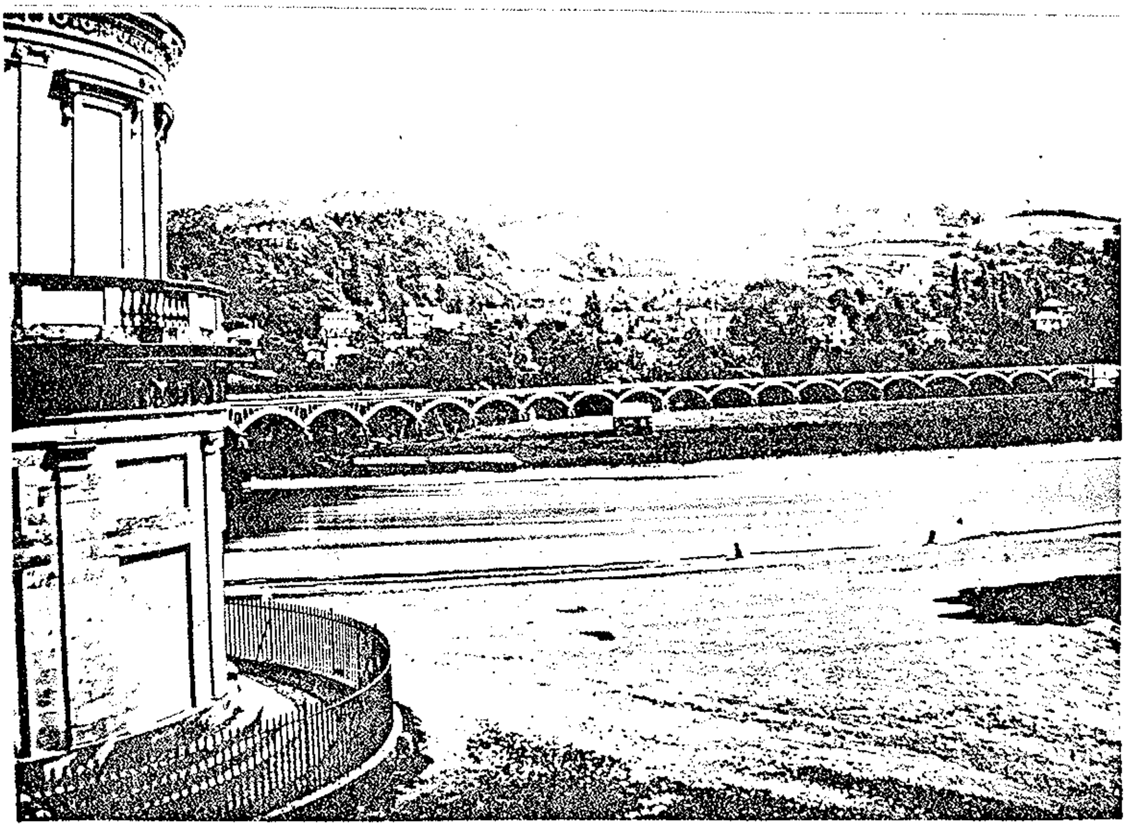
In July 1849 the Board of Trade Inspector (this time Captain Laffan) returned to Perth to approve another laminated bridge on the D and PR. The line crossed a deep cut, which had been made to form the future approach to the Scottish Central Station, by a single-span bridge. This bridge behaved far better than the Company's viaduct over the Tay, for as Laffan pointed out, "there being but one arch and the abutments on either side being firm and unyielding there cannot be so much play and there is none of the reaction caused by the deflection of one arch affecting its neighbour through lofty and slender and therefore yielding piers".

Whilst he was in Perth, Laffan reinspected the D and PR's Tay viaduct. Laffan's object was to see if the constant vibration had loosened the joints and increased the flexibility of the viaduct. The main cause of the trouble was that the laminated arches were supported on wooden piles. As Laffan pointed out "These piles are of course wholly insufficient to bear the horizontal thrust of the arches, they simply supported the weight and horizontal thrust is disposed of by making one arch butt against another. The consequence of this arrangement is that one arch cannot be depressed without immediately affecting its neighbour, and thus the effect of any movement at one end of the bridge caused a series of vibrations to run through the whole structure to the extremity". Laffan measured the deflection during the passage of an engine and found that the movement had increased 50 per cent since Simmons' inspection. He therefore imposed a speed limit of 8 miles an hour. The viaduct remained in service until it was replaced in 1862.

References

Booth, L.G. (1971a). The development of laminated timber arch structures in Bavaria, France and England in the early nineteenth century. Journal of Institute of Wood Science. (29). July 1971.

Booth, L.G. (1971b). Laminated timber arch railway bridges in England and Scotland. Transactions Newcomen Society. XLIV.



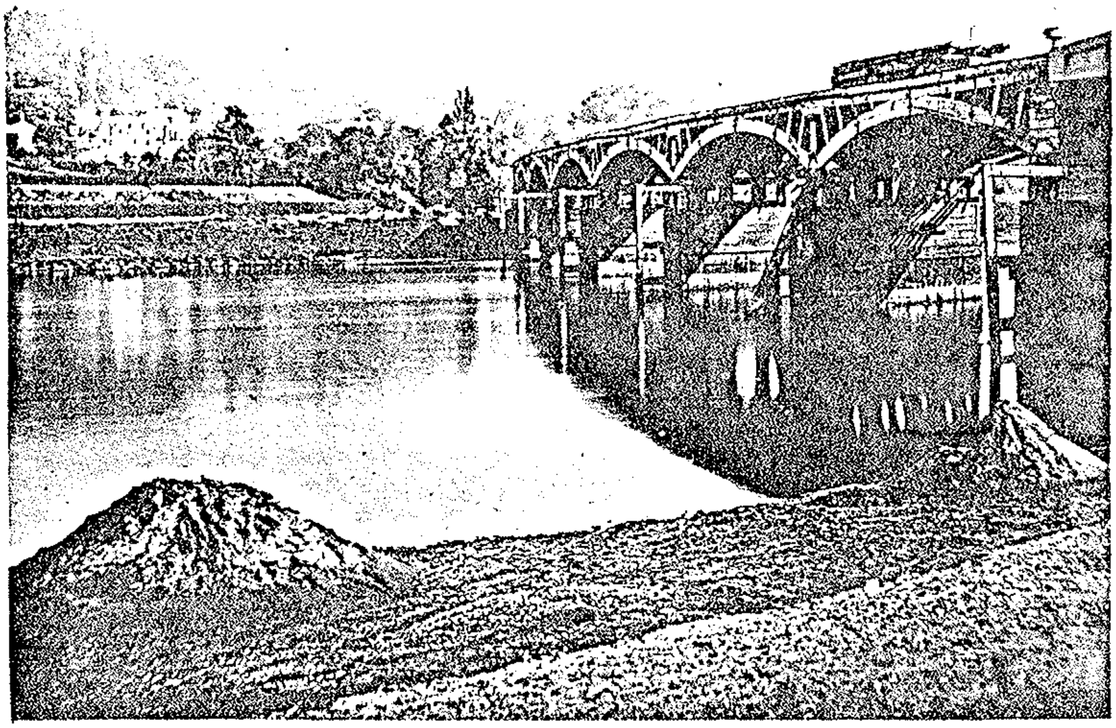


Figure 1. Laminated timber arch bridge on the Dundee and Perth Railway. Built 1849. Replaced 1862. (Courtesy City of Perth: Art Gallery and Museum)

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DETERMINATION OF THE BEARING STRENGTH AND THE LOAD-DEFORMATION CHARACTERISTICS OF PARTICLEBOARD

by

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Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard

Ordered by: Bundesministerium für Raumordnung, Bauwesen und Städtebau, Bonn - Bad Godesberg (West Germany)

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1. Problem

Load transmitting joints of particleboard to solid or laminated timber are predominantly made by using mechanical fasteners (i.e. nails, screws, staples) of cylindrical shape. The appropriate codes of timber structures are missing a safe basis for the design of such joints. The load-carrying capability and stiffness of particleboard-to-wood joints can be better comprehended when the bearing strength and the load-deformation characteristics under bearing pressure are known. Therefore, the aim of this investigation was to determine the bearing strength of particleboards as well as the elastic bearing constants describing the deformation behaviour of the joints mentioned before. These values

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depend on the particleboard properties and the slenderness (= ratio of board thickness to fastener diameter) of the fastener. In cooperation with the European Federation of Associations of Particleboard Manufacturers (FESYP) these properties have been determined with nine different, 8 to 38 mm thick particleboards from six European countries. Fastener diameters from 2 to 10 mm were used. Under these conditions it was possible to find out reliable relationships for the practical field of application of particleboards for timber structures.

2. Results

Since no test methods previously have been established for such testing, it was necessary to perform preliminary investigations concerning the shape and the size of the test specimen as well as the test procedure. Special test equipments were developed in order to plot automatically the load-deformation curves with a two-component recorder as well as to determine the ultimate load-carrying capacities and the appertaining ultimate deformations of the particleboard. The samples were cut from the original particleboards according to a predetermined cutting schedule. The boards were taken in the plants at random in order to assure that the test results could be considered representative for the product.

The bearing strength β_ℓ is the ultimate resistance of the hole surface against the pressure of a rigid cylindrical tack. Different from the exact distribution of the stress, the bearing strength is defined as the ultimate load maxF referred to the projection of the loaded hole surface area:

$$\beta_\ell = \frac{\max F}{a \cdot d} \qquad (\mathrm{N/mm^2})$$

where a is the board thickness in mm and d is the fastener diameter in mm.

Independent of the direction in the plane of the board the bearing strength increases when the density of the board increases or the fastener diameter decreases. A straight increase with joint slenderness was established for a slenderness below 6. As the three-layer boards have a face-layer of higher density, the bearing strength was considerably influenced by the amount of surface-layer thickness according to the total board thickness.

Since the compressive strength in plane of the board of all types tested was known (being about 70 percent of the bending strength β_B), a direct relation between the bending strength and the bearing strength could be found by analysing statistically all test values available. This relationship can be described by the formula:

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$$\beta_\ell = 0.7 \cdot \beta_B + \frac{4.3a+69}{d} \qquad (N/mm^2)$$

It is of interest that the bearing strength of particleboards is always higher than their compressive strength. For three-layer boards of 19 mm thickness the bearing strength of the face-layer is about 2.5 times as high as the bearing strength of the core-layer. Consequently, the bearing strength of the face-layer amounts to about 1.6 to 1.7 times the bearing strength of the total particleboard.

From the load-deformation records a distinct straight-line relationship was established for the lower part of the curves. At the proportional limit impressions of 0.15 to 0.30 mm were stated, depending on the percentage of the surface-layer thickness to the total board thickness. In the proportional area the elastic bearing constants C_o were generally determined by the formula:

$$C_o = \frac{\sigma_\ell}{\delta(\sigma_\ell)} = \frac{F}{a \cdot d \cdot \delta(F)} \qquad (N/mm^3)$$

where $\delta(\sigma_\ell)$ is the impression of the fastener into the particleboard under a bearing stress $\sigma_\ell = F/a \cdot d$. Statistical analysis of all load-deformation curves resulted in approximate mathematical formulae for the relationship between the elastic bearing constant C_o and the density of

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particleboard ϱ_N (g/cm^3) in normal climate

$$C_o = 622 \cdot \varrho_N - 322 \qquad (N/mm^3)$$

as well as the board thickness a (mm)

$$C_o = 311 - 72 \cdot \ln a \qquad (N/mm^3)$$

Similar to the bearing strength it was found that the elastic bearing constants of the surface-layers are higher than those of the core-layers. The values of the surface-layers of a 19 mm thick board were 70 to 80 percent higher than the values of the total board.

Beyond the proportional limit the load-deformation characteristic is highly dependent on the board properties. The elastic bearing constants C_δ in the plastic range can be defined as a secant modulus for a definite impression δ (e.g. 1 mm), and can be described by the formula:

$$C_\delta = D_1 \cdot \frac{a}{d} + D_2 \qquad (N/mm^3)$$

where

$$D_1 = \frac{k_1 \cdot \ln\delta + k_2}{\delta} \quad \text{and} \quad D_2 = \frac{k_3 \cdot \ln\delta + k_4}{\delta}$$

The constants k_1 to k_4 have been determined statistically from all tests performed. With these constants the secant muduli of all particleboards can be calculated approximatively, provided that the thickness and the density of the particleboards are known.

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3. Possibilities of Application of the Results for Practical Use

The investigations have established basis values of the bearing strength and the elastic bearing constants of particleboards under compressive stresses by cylindrical metal fasteners. With the relationships of these characteristic values to the bending strength and the density of the particleboards as well as to the slenderness of the fasteners the ultimate load-carrying capacity and the limit values of the deformation of particleboard-to-wood joints can be evaluated. Hence, the allowable loads of these joints can be fixed on the basis of the safety required.

Full-length report: 26 pages, 33 figures, 8 tables and appendix (9 tables of detailed test data).

FIDOR

THE STRUCTURAL USE OF TEMPERED HARDBOARD

Proposal for submission to BLCP17 for inclusion in CP112 "The Structural Use of Timber"

Prepared for the Fibre Building Board Development Organisation Ltd
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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

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Fig 3: TE GRADE TEMPERED HARDBOARD TO BS 1142: TEST RESULTS OF MEAN AND MINIMUM (5% EXCLUSION) DRY MODULI

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Table 49A (Section 6.0)

Wet grade stresses and instantaneous moduli for tempered hardboard.

Table 49B (Section 6.0)

Dry grade stresses and instantaneous moduli for tempered hardboard.

Table 49C (Section 6.0)

Dimensions and properties of tempered hardboard complying with BS 1142.

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Modification factor K_{27} on stresses for duration of loading for tempered hardboard.

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Modification factor K_{28} on modulus of elasticity and modulus of rigidity for tempered hardboard.

Table 52A (Section 6.0)

Modification factor K_{29} for number of tempered hardboard components tested.

1.0 INTRODUCTION

In 1973 the BSI code drafting committee BLCP17 agreed to accept for consideration proposals to be submitted by FIDOR for the inclusion of tempered hardboard in the revised CP112.

This report presents proposals and includes the experimental evidence, methods for deriving permissible stresses and draft clauses for inclusion in CP112.

Tempered hardboard is identified by BS1142 Part 2 1971. It is the most dense, strong, and durable of the family of fibre building boards. It should not be referred to as 'hardboard' as this term is oftern associated with standard hardboard which has significantly different properties.

There are two types, TE and TN, of tempered hardboard defined in BS1142 Part 2. TE is the superior of the two in having higher strength, lower water absorption and moisture movement. This report and the supporting test data cover only the TE type for inclusion in the revised CP112.

Tempered hardboard is being used in the UK for structural purposes and tests on their strength properties, racking resistance and performance in built-up timber structural elements have been carried out by PRL, TRADA and the Surrey University in recent years.

Tempered hardboard is listed under Building Regulation B3 as an acceptable material without limitation of thickness for the weather-resisting part of external walls.

The more obvious uses for tempered hardboard include flange and web elements of built up beams, roof and floor boarding, sheathing in timber frame walls, door coverings and concrete formwork.

TE tempered hardboard is readily available in the UK, mainly from Scandinavian and South African mills.

1.0 (continued)

The strength properties for structural design are written in this report in terms of permissible stresses based on a 5% exclusion of test results and appropriate factors of safety. Their conversion into the limit state design convention to suit a later revision of the Code is being withheld pending further clarification by subcommittee BLCP17/2, but will be a relatively simple matter at the appropriate time.

To illustrate how the recommendations for the structural use of tempered hardboard would be incorporated in the revised CP112, the draft clauses in section 6.0 are written as if they would be published as an Amendment slip to the current CP112: Part 2: 1971.

The main testing programme on the structural properties of tempered hardboard was carried out recently for FIDOR at Princes Risborough Laboratory and Yarsley Testing Laboratory Limited, of Ashtead, Surrey. In addition, reference is made to previous data published by PRL, Swedish experience and other research information.

Grateful acknowledgement is made to the staff of PRL for their advice and assistance in testing, sampling materials, supervising the Yarsley work, and statistical analysis of data.

Introduction to revised January 1978 edition

The April 1977 first edition was considered by BSI subcommittee BLCP17/12 'Wood based sheet materials' from June to October 1977, during which a number of corrections and improvements to the contents were agreed.

This revised edition incorporates the corrections and improvements, which are identified by a vertical line in the left-hand margin, except for Figs 1, 2 and 3 which have been re-drawn with corrections but have no identifying lines.

The revised Section 6 'Draft clauses for the revision of CP112' up to clause 5.4.7 was accepted by BSI committee BLCP17 in December 1977, subject to further consideration by subcommittee BLCP17/2 'Working stresses' as regards compatibility with CP112 Part 2 1971 and by subcommittee BLCP17/9 'Prototype testing' as regards clauses 6.2.6 to 7.2.8. This revised edition has been prepared principally for consideration by these two subcommittees.

2.0 SPECIFICATION FOR TE TEMPERED HARDBOARD

2.1 Definitions

TE tempered hardboard is defined in BS1142 Part 2 1971 'Specification for fibre building boards. Part 2: Mediumboard and hardboard'.

The relevant clauses of BS1142 are as follows:

1.3 DEFINITIONS

For the purposes of this British Standard, the following coding and definitions apply:

	Type
TE	Tempered hardboard
TN	Tempered hardboard
S	Standard hardboard
LME	Low density medium board
LMN	Low density medium board
HME	High density medium board
HMN	High density medium board

1.3.1 Fibre building board. Fibre building board is a sheet material, usually exceeding 1.5 mm in thickness, manufactured from fibres of ligno-cellulosic material with the primary bond deriving from the felting of the fibres and their inherent adhesive properties. Bonding, impregnating or other agents may be added during or after manufacture to modify particular properties of the board.

1.3.4 Tempered hardboard. Type T fibre building boards are generally derived from wood fibre, treated during manufacture to improve strength and resistance to water absorption with a density normally exceeding 960 kg/m^3. The physical properties shall comply with Table 2 for Types TE and TN.

BS1142: Part 2 Table 2 'Test levels for standard and tempered hardboard' gives the following requirements for TE grade:

Type	Specified manufacturing thickness	Minimum mean bending strength	Changes after water immersion		Effect of relative humidity change from 33 % to 90 %	
			Maximum mean absorption by weight	Maximum mean thickness swelling	Maximum mean increase in length and width	Maximum mean increase in thickness
	mm	N/mm^2	%	%	%	%
TE	2	59	30	15	0.30	10
	2.5		20	12		
	greater than 2.5 but equal to or less than 3.2		15	10		
	greater than 3.2 but less than 8.0	45	11	8		
	equal to or greater than 8.0 but equal to or less than 10.0		8	5		
	greater than 10.0		6	3		

Note: BS1142: Part 1: 1971 defines the 'Methods of Test'

The actual density of TE tempered hardboard, as determined from tests, averages 1000 kg/m^3 and this value and the nominal thickness are used for calculating the board weights given in Table 49C of Section 6.0.

2.2 Sizes

Thicknesses of board normally available include 3.2, 4.8, 6.4, 8.0, 9.5 and 12.7mm. However on grounds of availablility of supply and strength data, it is proposed that only thicknesses up to 8.0mm should be included in the next revised Code.

Widths of board can be 1220mm or 1700mm but only 1220mm wide boards are generally available.

Lengths vary from 610 to 5486mm. The most common lengths are 2440mm, 2745 and 3050mm.

2.3 Tolerances

Extracts from BS1142: Part 2 are given below:

2. REQUIREMENTS FOR TYPE HM AND LM MEDIUM BOARDS AND TYPE T AND S HARDBOARDS

2.1 PERMISSIBLE DEVIATION, DIMENSIONAL TOLERANCES, SQUARENESS AND STRAIGHTNESS OF BOARDS

2.1.1 Unless otherwise agreed between the manufacturer or his representative and the purchaser, the boards shall be rectangular and shall have square edges. The boards shall not deviate more than 2 mm from the right angle over a span of 1000 mm. Measurement shall be made by the methods described in **4** of Part 1.

2.1.2 Length, width and thickness shall be as agreed between the manufacturer or his representative and the purchaser. Measurements shall be made by the methods described in **4** of Part 1. Lengths and widths of boards manufactured for use in dimensionally co-ordinated buildings designed to the controlling reference system given in BS 4330 shall, as a first preference, be selected from the co-ordinating sizes given in Clause 3 of BS 4606 : 1970. As far as length and width is concerned the deviation shall be within the permissible deviation given in Table 3. For thickness each of the 12 measurements reported as the mean thickness of each bending strength test specimen shall be within the permissible deviation given in Table 3. Materials available to closer permissible deviation may be the subject of negotiation between the manufacturer or his representative and the purchaser.

2.1.3 Straightness of the edges shall be measured by the method described in **4** of Part 1, and the deviations shall not exceed 0.125 % of the length of the appropriate edge.

2.3 Tolerances (continued)

TABLE 3. PERMISSIBLE DEVIATION IN LENGTH, WIDTH AND THICKNESS OF MEDIUM BOARD AND HARDBOARD

Type	Mean manufacturing thickness	Permissible deviation	
		Thickness	Length and width
	mm	mm	mm
T and S	equal to or greater than 2 but less than 3.2	± 0·4	
	equal to or greater than 3.2 but equa to or less than 6.4	± 0.5	
	greater than 6.4	± 0.7	± 3.0

NOTE. Closer permissible deviations than those given above can be obtained by agreement.

The minus tolerances given above have been used to determine the nett thicknesses and other section properties of the boards, given in Table 49C of this report.

2.4 Identification

Extracts from BS1142: Part 2 are given below

6. MARKING

6.1 COLOUR CODING

The edges of boards not less than 1200 mm wide and not less than 1500 mm long shall be colour marked, when deemed to comply with the requirements of this specification by one of the methods outlined in 4. Boards shall be colour coded in the stack by the manufacturer before despatch, according to the following code:

	Type	Code
TE	Tempered hardboard	Two red stripes

The stripe or stripes shall occur once on each long edge and on diagonally opposite corners, and shall be at least 25 mm wide. If boards are despatched in paper wrappings, the stripes shall be applied to the boards as well as the paper.

2.5 Verification of compliance with BS 1142

Extracts from BS 1142: Part 2 are given below:

4. VERIFICATION OF COMPLIANCE

4.1 GENERAL

The supplier shall satisfy himself that the boards comply with the requirements of this standard.

One of the following methods may be used to verify compliance with the appropriate requirements as stipulated by the purchaser.

4.1.1 By reference to the manufacturer's certificate of compliance with the standard which shall be provided at the request of the purchaser.

4.1.2 By reference to the records of quality control tests carried out by the manufacturer, where such tests are made at regular intervals and the records are deemed acceptable by the purchaser.

4.1.3 By carrying out tests, using the methods described in Part 1 of this standard, on samples drawn from the consignment under consideration. Where such tests are required, they shall be carried out by a mutually acceptable testing authority. A method for selection of samples for consignment testing is given in **3** of Part 1.

4.2 COMPLIANCE REQUIREMENTS

To be accepted as complying with this standard, each type of board shall be constituted as defined in **1.3** and comply with the requirements of **2**. In the case of consignment testing, this requirement applies to the results from the various test pieces, coupled with a 10 % acceptable quality level.

3.0 STRENGTH DATA

3.1 Method of test

BS1142: Part 2 specifies five tests for tempered hardboard, of which four are concerned with water absorption and dimensional stability and only one with strength (in bending). As BS1142 is not primarily a standard for structural uses of the material, clearly additional methods of test are required to derive other strength characteristics required for structural purposes.

With advice from PRL, it was decided to adopt, where no relevant test method existed for fibre building board, test methods for plywood.

Modulus of rupture: Tested to BS1142: Part 1: 1971, with specimen size of 25 times thickness plus 50mm, by 75mm and a span of 25 times thickness; testing speed 30 $\pm$ 3mm/minute.

Modulus of elasticity in bending: No standard method available, but measured during modulus of rupture tests.

Since the BS test is by centre-span point loading, the apparent value E_a measured contains a shear deflexion component, so that the true modulus E would be slightly higher. Thus taking (see ref 20)

$$\frac{Pl^3}{48E_aI} = \frac{Pl^3}{48EI} + \frac{0.3Pl}{GA}$$

and assuming G = 0.5E (see ref 9) and taking the BS specimen size of span: thickness = 25, it can be shown that $E = 1.0038E_a$, demonstrating that the error incurred in taking E_a as E is negligible.

Tensile strength: Tested to ISO/TC89 (WG9/1) 227 (Working Group draft), specimen size 400mm x 50mm, waisted to 20mm, testing speed 1mm/minute.

Tensile strength normal to board plane: This was not included in the PRL/Yarsley programme, previous work having been carried out by PRL (6, 7) for FIDOR on alternative methods of test, and by Lundgren (9). Since the PRL/Yarsley tests were completed, a draft BS method, based on ISO/DIS3931 proposal, has been circulated for

3.1 Methods of test (continued)

public comment. For the purposes of the revised code it is proposed that nominal values of permissible stress are assigned, based on the Swedish Type Approval values (16).

Modulus of elasticity in tension: Tested according to the ISO tensile strength method given above, with a gauge length of 50mm.

Compressive strength (in board plane): Tested to BS4512 1969 for plywood, specimen size 100mm x 25mm, with the test rig restraining the specimen from buckling; test speed 0.3mm/minute.

Modulus of elasticity in compression: Tested according to the compressive strength method given above, without using deflectometer, the strain being obtained from movement of loading head, with a gauge length of 100mm. Since the loading head is subject to local bedding-down at both ends of the specimen , the moduli thus obtained are unreliable; in fact the test results (Fig 3) show, as expected, that the values were excessively low compared to those for bending and tension.

Modulus of rigidity: Tested to BS4512 1969, with minor modifications to test rig which did not affect the testing procedure. Specimen size 25 times thickness, square; test speed 0.003 x 25 x thickness mm/minute.

Panel shear: Tested to BS4512 1969, effective specimen size 229 x 229mm, testing speed 1.8mm/minute.

Double lap tensile shear (or rolling shear): Tested to BS4512 1969, specimen shape slightly modified, shear area 2 x (25 x 25)mm^2, testing speed (modified to suit gripping) 1.9mm/minute.

Bearing strength: No recognised test method exists for hardboard or plywood. The ball indentation test previously adopted for solid timber is considered inappropriate. Tempered hardboard possesses high bearing strength and the mode and consequences of failure under

3.1 Methods of test (continued)

load are generally of little significance in structural application. A reasonable bearing strength can be assigned based on the press pressures applied during board manufacture as recommended by Lundgren (9). A permissible bearing stress of $5N/mm^2$ is thus proposed for dry exposure and reduced (arbitrarily) to $4N/mm^2$ for wet exposure.

Long term strength and deformation tests: These were not included in the PRL/Yarsley tests. Data on the effect of duration of load on strength were obtained from previous tests carried out by PRL (10) for FIDOR, with multiple specimens coupled and loaded according to BS1142: Part 1, and from the work of Lundgren (9) and McNatt (13). Data on the effect of creep on the elastic moduli of tempered hardboard in dry and wet conditions were obtained from the work of Lundgren (9).

Effect of wet exposure: The PRL/Yarsley tests were carried out at $20^{\circ}C/65\%RH$ and $25^{\circ}C/85\%RH$, corresponding to dry and damp categories of internal environments. In addition, Lundgren's work (9) on occasional and prolonged exposure to 90% RH and above has also been used. It was realised after completion of the PRL/Yarsley tests that the equilibrium moisture content for specimens at $25^{\circ}C/85\%RH$ was insufficiently high for the subsequently adopted definition of wet exposure (see 4.7) and was in fact only slightly different from that of specimens under $20^{\circ}C/65\%RH$ (dry exposure). It was therefore decided to use the PRL/Yarsley dry test values only and to use Lundgren's data to obtain conversion factors from dry to wet strengths and moduli.

Gluing: Tests have been carried out by PRL (22), by manufacturers of proprietary glued structural assemblies (23, 24) and through the double-lap tensile shear and panel shear tests mentioned above. They show that tempered hardboard can be successfully glued but that it is advisable when gluing the smooth face to sand it lightly; this is reflected in 6.0, clause 5.4.7.

3.1 Methods of test (continued)

Mechanical fastenings: Tests have been carried out by PRL, on a range of plywoods and tempered hardboard and a further series of tests have been recently sponsored by FIDOR. The results are being analysed and will be submitted separately to subcommittee BLCP17/7.

3.2 Materials

Two commonly available brands of TE grade tempered hardboard to BS1142: Part 2, designated Brand A and Brand B were selected for the PRL/Yarsley 1975 tests. The boards were supplied from commercial stock in 3.2, 4.8 and 8.0mm nominal thicknesses. The 3.2mm thick boards from Brand B were subsequently shown to be standard hardboard and although tested, the results are not given.

It will be noted that relatively few tests were included for 3.2mm boards and that no 6.4mm boards were included. In the case of 3.2mm boards, PRL had already carried out some of the tests for FIDOR in an earlier programme (11). In the case of the 6.4mm boards, it was considered that the strength properties could be interpolated between those of 4.8mm and 8.0mm boards; subsequent analysis of test results and the decision not to differentiate thicknesses in design stresses have justified this simplification and saving in testing.

3.3. Sampling

This was carried out by PRL in accordance with BS1142: Part 1 clause 3.2. The following numbers of sheets of 2438mm x 1219mm were provided for the PRL/Yarsley tests (excluding the 3.2mm Brand B mentioned in 3.2).

Brand	Thickness (mm)	No. of sheets
A	3.2	12
A	4.8	12
A	8.0	12
B	4.8	12
B	7.9	12

3.3 Sampling (continued)

of each consignment of board thickness, six sheets were allocated to Yarsley tests and six sheets to PRL tests. Material was selected according to BS1142: Part 1 by the use of random number tables. Test material was sent to Yarsley from PRL in the prepared test sizes.

3.4 Test results

Summaries of the PRL/Yarsley 1975 tests (Brands A and B) are given in Figs 1 to 3 and Table 5. Other test results are also given in Figs 1 to 3 and the references 9, 10, 11, 13, 17 and 18 but preference is given to the PRL/Yarsley 1975 tests because of their emphasis on the thickest board (8mm) tested which has the lowest strength properties.

4.0 DERIVATION OF DESIGN STRESSES

4.1 General

The PRL/Yarsley 1975 programme of tests were designed to supplement the test data and experience from Sweden and the previous work carried out for FIDOR by PRL. A summary of the test results is given in Figs 1 to 3 and Table 5. The programme confirmed that TE grade tempered hardboard tested from samples drawn from UK stock:

Complied with BS1142: Part 2, except for modulus of rupture (bending strength), which was up to 7% less in the 8mm boards.

Has strength properties close to those of tempered hardboard tested by Lundgren and others (9, 13, 14, 15, 17), and previous PRL tests (10, 11, 12).

To derive permissible stresses from test results, consideration must be given to the effects on strength properties of duration of load, smooth and mesh face, board thickness, machine direction, creep, moisture content, and the choice of appropriate safety factors. These are discussed in the following subsections.

4.2 Effect of smooth or mesh face

This affects strength properties in bending perpendicular to the plane of the board, the mesh face being slightly weaker in tension. However, the difference does not exceed about 10% and to prescribe different bending design stresses for different face orientations would impose an unnecessary complication. It is therefore proposed to base the design bending stress on the test values with the mesh face in tension (11) thus erring on the safe side.

4.3 Effect of board thickness

In the range of thickness tested, from 3.2mm to 8.0 mm, most of the strength properties reduce slightly with increased thickness, although not in a strictly consistent pattern (Figs 1 to 3). Because of this and the complication of prescribing different design stresses for various thicknesses, it is proposed to prescribe a single set of design stresses to cover any thickness up to 8mm, based on the lowest strength values; this errs on the safe side.

Fig. 1

TE GRADE: TEMPERED HARDBOARD TO BS 1142

TEST RESULTS OF MEAN AND MINIMUM (5% EXCLUSION) DRY STRENGTHS

P1 = PRL (1970)

P2 = PRL (1969)

P3 = PRL/Yarsley (1975)

L = Lundgren (1969)

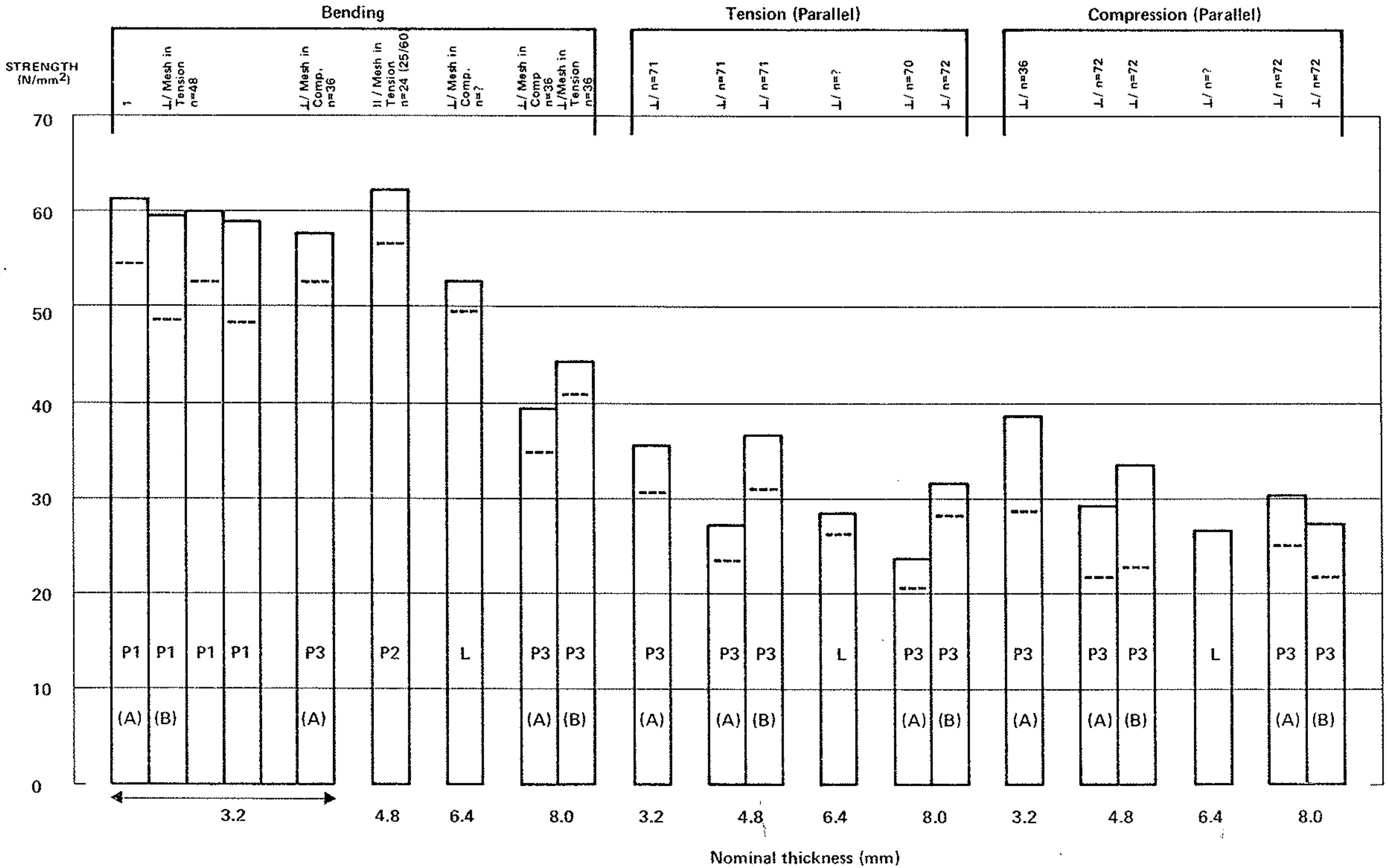


Fig. 2

TE GRADE TEMPERED HARDBOARD TO BS 1142

TEST RESULTS OF MEAN AND MINIMUM (5% EXCLUSION) DRY STRENGTHS

P3 = PRL/Yarsley (1975)
P4 = PRL (1973)   to BS1142: Pt2: 1971 (TE grades)
L = Lundgren (1969)

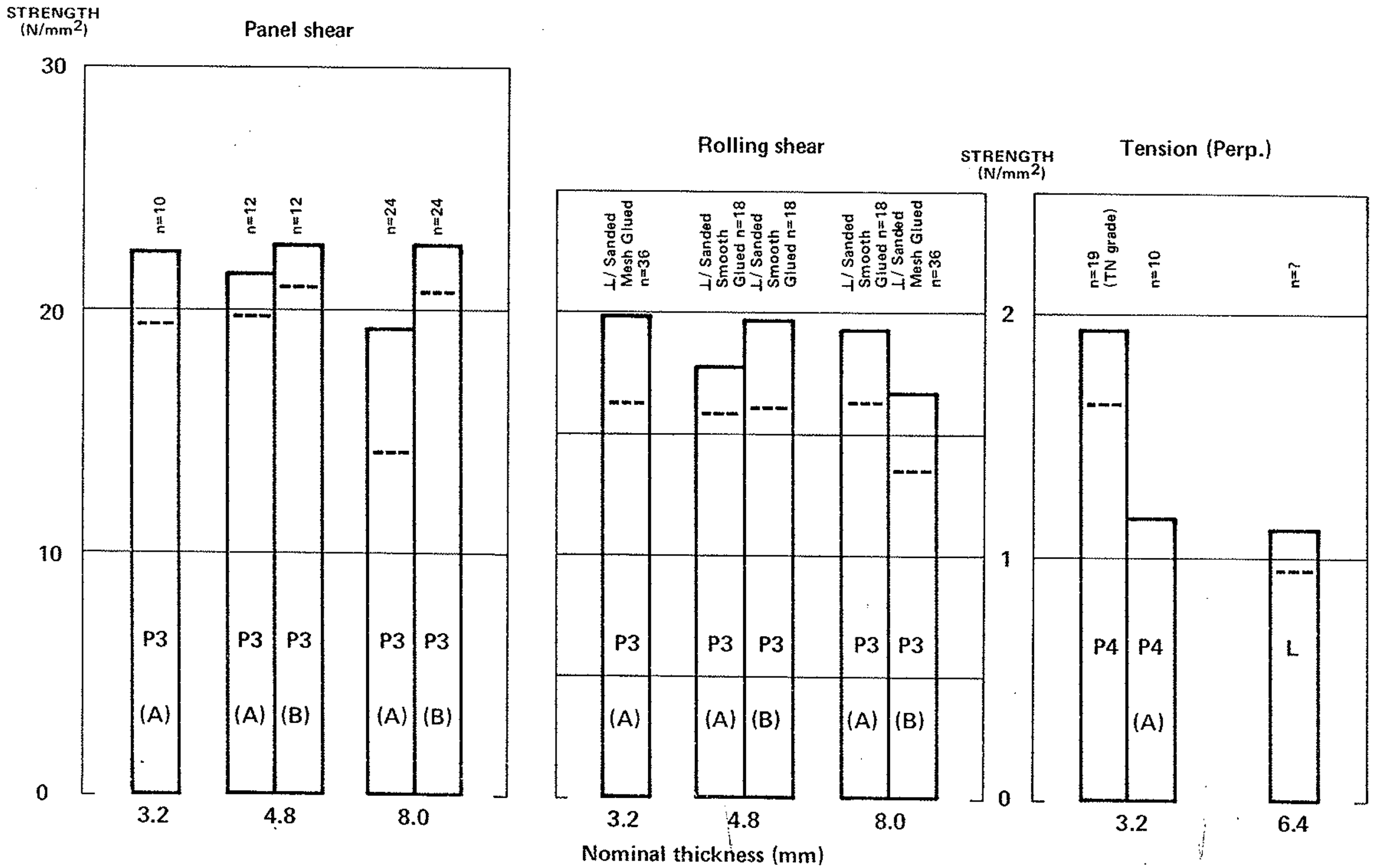


Fig. 3

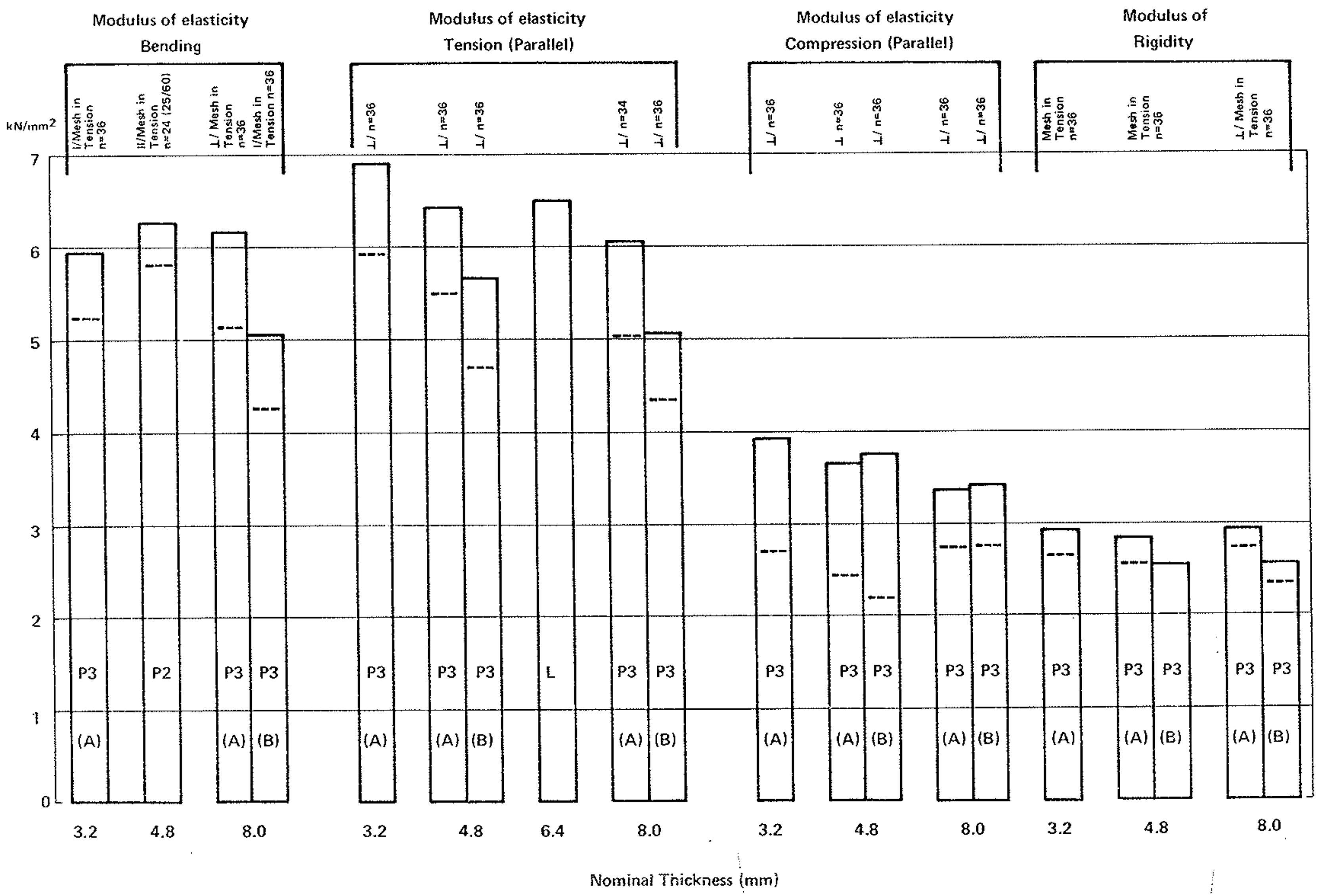
TE GRADE TEMPERED HARDBOARD TO BS 1142

TEST RESULTS OF MEAN AND MINIMUM (5% EXCLUSION) DRY MODULI

P2 = PRL (1969) to BS1142–1961

P3 = PRL/Yarsley (1975) to BS1142 Pt2 : 1971 (TE grade)

L = Lundgren (1969)



4.4 Effect of machine direction

Owing to the orientation of the felt fibres in the manufacturing process, it is known that the strength properties of the board in the direction of travel during manufacture are slightly higher than those transversely. Again the difference is small. To prescribe different design stresses for the machine direction and transversely would not only be an unnecessary complication, but would also be impractical to implement in fabrication and construction, since it would be extremely difficult to identify visually the machine direction on the board once it has been cut. It is therefore proposed to base the design stresses on the strength properties transverse to the machine direction, again erring on the safe side (see ref 11).

4.5 Duration of load

The strength of hardboard is modified according to duration of load, similar to but to a more pronounced extent than for solid timber.

Several independent sources of duration-of-load test results are available, from PRL (10) with specimens loaded in bending, Lundgren (9) and McNatt (13) with specimens loaded in tension. Very close agreement was obtained between these tests, the specimens of which were at or very close to the 'dry' 20°C/65%RH condition and loaded for not less than one year. Other tests by Lundgren (9) indicate that the same slope of the logarithmic plot can be used for 90%RH. It is proposed to adopt the strength and load duration relationship derived by PRL (10). The test specimens included a mixture of brands of tempered hardboard as well as some standard hardboard; the inclusion of the latter, a weaker material, means that the results would err on the safe side. The relevant equation is:

$$S = 78.2 - 6.8 \log T$$

where S is the strength as a percentage of the instantaneous strength;
T is the time to failure in hours.

Using the above equation, modification factors for duration of load are derived as follows:

Table 1. The influence of duration of load on strength

Duration category	Notional duration	S%	As factor of long-term load
Long term	50 years ($10^{5.656}$hrs)	39.74	1.00
Medium term	1 month ($10^{2.86}$hrs)	58.75	1.48
Short term	1 minute ($10^{-1.78}$h)	90.30	2.27
Very short term	5 seconds ($10^{-2.86}$h)	97.65	2.46
Laboratory test	20 seconds ($10^{-2.25}$h)	93.50	2.35
Prototype strength test (see 5.3)	Effective 40 minutes ($10^{-0.176}$h)	79.40	2.00
Prototype deflection test (see 5.2)	24 hours ($10^{1.38}$ hrs)	68.82	1.73

It is proposed not to make allowances in design rules for the effect of repeated loading on the duration of load factor. For medium term loads there is evidence to show that the likely number of applications and the intervals between loading do not lead to progressive weakening, particularly as a structure with medium term load always carries some long term dead load (9). For short or very short term loads there is complete recovery on unloading (see ref 9).

4.6 Moisture content

The strength and elastic moduli of tempered hardboard reduce with increased moisture content and, when very wet, reduce to a greater extent than those for solid timber.

While solid timber can be used structurally at reduced stresses in fully saturated or permanently submerged conditions, the use of tempered hardboard in similar conditions would be inadvisable and the draft in 6.0 for the revised code reflects this accordingly.

4.6 Moisture content (continued)

However, for the more usual dry or humid conditions of use indoors or outdoors, tempered hardboard attains a much lower equilibrium moisture content for a given humidity and temperature than timber, plywood or chipboard.

Lundgren (9) has carried out extensive measurements on humidity, temperature and equilibrium moisture content in the laboratory and outdoors. He also demonstrated that the humidity curves for Nykoping, Sweden, where much of the outdoor and indoor measurements were made, are very similar to UK ones. He showed that tempered hardboard unpainted and exposed outdoors never rose above 9% moisture content.

From these considerations, it is proposed to adopt the Swedish Type Approval definition of dry and wet exposure (Environmental Groups I and II) as follows (see reference 16):

<u>Dry Exposure</u> Structural elements indoors;
Attic flooring and roof structures in cold but ventilated, under-roof areas over permanently heated premises;
External walls of permanently heated buildings, protected by ventilated, vapour proof cladding;

and similar structures if normally exposed to air with a relative humidity not higher than 65%; the relative humidity of the air may, however rise to 75% for brief periods.

* <u>Wet Exposure</u> Roof sarking;
External walls of heated buildings where there is a risk of moisture concentration;
Building scaffolding, concrete formwork and similar temporary structures;

and similar structures which are not permanently heated but which are ventilated, such as garages, unheated stores and underfloor access spaces, if normally exposed to air with a relative humidity not higher than 85%, or not higher than 95% for brief periods.

* The term 'wet exposure' has been revised to 'damp exposure' (see 6.0).

4.6 Moisture content (continued)

The relationship between relative humidity, moisture content and strength has been studied by Lundgren (9), McNatt (14) Bristow and Back (15) and at Princes Risborough Laboratory (11).

Tests at Princes Risborough Laboratory (11) have shown that the equilibrium moisture content of tempered hardboard is 8.6% and 15.5% for 65% and 90% relative humidity respectively for three different brands drawn from three different sources.

Bristow and Back (15) showed that tempered hardboard taking up moisture to reach equilibrium will attain a lower equilibrium moisture content than if it were losing moisture to reach equilibrium at the same humidity level.

The relationship between strength, stiffness and relative humidity from the above sources of work, and from values given in the Swedish Type Approval (16) are summarised in Table 2.

Table 2 Relationship between strength and relative humidity

Property	Strength ratio 90%RH:65%RH	
	*Tests	**Swedish Type Approval stresses
Bending	0.88	0.60
Compression in board plane	0.70	0.45
Compression perpendicular to board plane	-	1.00
Tension in board plane	0.90	0.64
Tension perpendicular to board plane	0.73	0.40
Panel shear	0.90	0.67
Rolling shear	0.76	0.86

* References 9, 14 ** Reference 16

Table 2 shows that, comparing the Swedish Type Approval with tests, a margin of safety (ie dividing the figures in the first column by those in the second) of around 1.4 to 1.5 applied for bending, compression in board plane and tension in board plane. A much higher margin is applied to tension normal, mainly because of the large variability of results inherent in current methods of test (5, 6, 7, 9). For rolling shear, a lower margin is applied, based on Lundgren's findings (9) that it is relatively little affected by high humidities.

4.6 Moisture content (continued)

The margins of safety applied in the last column are based on judgement, to allow for hysterisis effects and variability of material. It is therefore proposed to base the 'wet' permissible stresses on the factors given in the last column of the above table, multiplied by the 'dry' permissible stresses; the derivation of the 'wet' elastic moduli is given in 4.8 and 4.9.

4.7 Conversion of strength values into permissible stresses

In order to convert strength values from laboratory short term tests, allowance has to be made for the effect of duration of load

In timber the basic grade stress (long term load duration) in the current CP112 is derived from strength test results by:

$$\text{Basic stress} = (m - t_{99}.\ s)\ \frac{1}{F}$$

where m is the mean of strength tests;
s is the standard deviation of strength from tests;
t_{99} is the Student's t value for 99% exclusion;
F is a global factor of safety with a maximum value of 2.25 when applied to bending,in which value is incorporated a load duration factor of 1.6 and a size effect factor of 1.3.

If the same basic stress were to be derived on the basis of 95% exclusion, the value of F would be increased to a value not exceeding 2.5 (based on current information from Princes Risborough Laboratory).

For tempered hardboard, a commensurate equivalent global factor of safety for 95% exclusion can be deduced from the one used for timber described above by allowing for a different value of the factor for duration of load of 2.35 (see Table 1) so that:

$$F = 2.5 \times \frac{2.35}{1.6} = 3.67$$

and it is proposed that this value of F is to be used to derive the dry permissible stresses for bending, compression and tension in board plane, panel shear and rolling shear (see Tables 4 and 5).

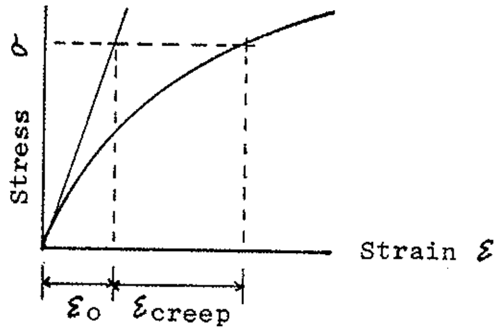
4.8 Modulus of elasticity

The value of the 'effective' modulus of elasticity E under sustained load is less than the instantaneous value E_o because of creep deformation. Thus for a given stress level,

$$E = \frac{\text{stress } \sigma}{\varepsilon_o + \varepsilon_{creep}}$$

$$= \frac{E_o}{1 + \dfrac{\varepsilon_{creep}}{\varepsilon_o}}$$

$$= \frac{E_o}{1 + \varphi} \qquad \text{.....(a)}$$



The value of E_o may be taken as that measured in laboratory tests.

According to Lundgren's work (9)

$$\varphi = \alpha \cdot \frac{k^z}{k^3} \qquad \text{..........(b)}$$

$$z = \log \frac{T}{T_o}$$

where T_o is assumed to be 10^{-3} hours.

The values of α and k measured by Lundgren in creep tests on tempered hardboard at a tensile stress of 15% of the ultimate strength were:

Relative humidity %	α	k
65	0.071	2.10
90	0.108	2.38

Measurement in compression gave similar results.

Lundgren also established that creep is approximately proportional to the level of stress. Thus equation (a) may be modified to allow for the effect of stress level as follows:

$$E = \frac{E_o}{1 + \beta\varphi}$$

where β is the ratio of $\dfrac{\text{Applied stress}}{\text{mean ultimate stress}}$ to the 15% mean ultimate stress level for which Lundgren has obtained α and k values.

As Lundgren's tests were based on 15% of the test stress to mean ultimate stress and as the mean E values will be published in the proposed inclusion in CP112, the value of β should relate to

$$\beta = \frac{\text{Applied stress}}{\text{Mean ultimate stress}} \div 0.15$$

The applied stress is the 'grade stress' modified according to the various durations.

Thus for long term duration of load, the notional applied stress for the purpose of calculating β may be obtained by averaging the grade stresses for bending, compression and tension: (see Table 4 and 5)

Stress	Grade stress	Mean ult.stress	$\frac{\text{Grade stress}}{\text{Mean ult.stress}}$
Bending	10.92	47.5	0.2299
Compression in plane	6.23	28.6	0.2178
Tension	5.55	27.7	0.2004
		Average = 1/3 x	0.6481
		=	0.216

4.8. Modulus of elasticity (continued)

Thus from 4.7 and 4.5 we have

Duration of load	Dry grade stress / Mean Lab.strength	β_{DRY} = Previous column / 0.15	β_{WET} =Previous column x 0.56*
Long	0.216	1.44	0.81
Medium	0.216 x 1.48 = 0.320	2.13	1.19
Short	0.216 x 2.27 = 0.490	3.27	1.83
Very short	0.216 x 2.46 = 0.531	3.54	1.98
Prototype testing (24 hours)	0.216 x 1.73 = 0.374	2.49	1.395

* The value of 0.56 is the mean of the values in the last column of Table 2 for bending, compression in board plane and tension in board plane.

Since the effective value of modulus of elasticity is used in calculating deformations, it is necessary to consider the cumulative effect of deformations arising from the number of times which a load of given duration is applied:-

(i) Long term: No allowance for cumulative deformation is required since the load is assumed to be sustained continuously over a notional period of 50 years or $10^{5.656}$ hours (4.5). A practical structure may wholly or partly carry such a load in terms of dead load or types of imposed load which are regarded as permanently applied (e.g. floor load).

(ii) Medium term: The notional duration per application may be assumed to be one month ($10^{2.86}$ hours) and refers mainly to snow load in the UK of 0.75 kN/m^2. The frequency of occurence, duration and the actual intensity is not known (although the intensity of 0.75 kN/m^2 is known to be conservative and a lower value would proportionately reduce creep) but it would be reasonable to assume that the total duration of load for the purpose of estimating creep is 1/4 of 50 (once every year in life of structure) x $10^{2.86}$ hours = $10^{3.96}$ hours (the factor of 1/4 makes allowance for the fact that the recovery strain on unloading over an 11 month period is at least 3/4 of the creep strain developed for a one month application of load - see Lundgren Fig 107, ref 9).

(iii) Short and Very Short terms: These durations of one minute and 5 seconds respectively are so short that on unloading, full elastic recovery occurs (9). It is therefore unnecessary to consider cumulative deformations. A practical structure subject to wind loads would carry partly short term loads and partly long (and perhaps medium term loads), as in a roof, or may carry short or very short term loads only, as in a wind girder or timber frame sheathing.

Combining the above considerations of stress levels and load durations, the derivation of values for effective modulus of elasticity can be summarised as follows:

Table 3. Effect of load-duration and stress level on effective moduli

Duration of load Category	T hrs	α	k	φ	β	$\beta\varphi$
65% RH						
Long term	$10^{5.656}$	0.071	2.10	4.718	1.44	6.79
Nett medium term	$10^{3.96}$	0.071	2.10	1.341	2.13	2.86
Nett short term	$10^{-1.78}$	0.071	2.10	0.019	3.27	0.06
Nett very short term	$10^{-2.86}$	0.071	2.10	0.009	3.54	0.03
Prototype deflection test	$10^{1.38}$	0.071	2.10	0.198	2.49	0.49
90% RH						
Long term	$10^{5.656}$	0.108	2.38	14.570	0.81	11.80
Nett medium term	$10^{3.96}$	0.108	2.38	3.347	1.19	3.98
Nett short term	$10^{-1.78}$	0.108	2.38	0.023	1.83	0.04
Nett very short term	$10^{-2.86}$	0.108	2.38	0.009	1.98	0.02
Prototype deflection test	$10^{1.38}$	0.108	2.38	0.360	1.395	0.50

(For meaning of 'nett' see remainder of 4.8)

In practice a structure may be stressed to below the maximum permissible stress or carrying a mixture of long, medium, short and very short durations of load. All these conditions modify the effective elastic moduli because of the modified stress level and the load-durations causing different rates of creep.

In general the effective modulus can be expressed as follows:

$$E = \frac{E_o}{1 + \frac{\sigma}{f}\beta\varphi}$$

where σ is the actual applied stress
f is the permissible stress.

Applying this to long term duration of load we have:

For dry exposure, $E = \dfrac{E_o \text{ (dry)}}{1 + 6.79\ \sigma_L/f_L}$

For wet exposure, $E = \dfrac{E_o \text{ (wet)}}{1 + 11.80\ \sigma_L/f_L}$

In the case of medium term duration of load, which by current CP112 definition also includes any long term load which may be present (which invariably is), the calculation for the maximum deformations under medium term load needs to be carried out in two parts: the first relates to the long term loads present, for which the effective E is as given above; the second part is for the medium term total load <u>less</u> the long term portion already accounted for in the first part, (or nett medium term load), i.e.:

$$E = \frac{E_o}{1 + \frac{(\sigma_M - \sigma_L)}{f_M}\beta\varphi}$$

Substituting the appropriate values of $\beta\varphi$ from Table 3 into the above equation we have, for medium term load <u>less</u> long term loads (or nett medium term load):

For dry exposure, $E = \dfrac{E_o \text{ (dry)}}{1 + \frac{2.86}{f_M}(\sigma_M - \sigma_L)}$

For wet exposure, $E = \dfrac{E_o \text{ (wet)}}{1 + \frac{3.98}{f_M}(\sigma_M - \sigma_L)}$

4.8 Modulus of elasticity (continued)

In the case of short term duration load, less any other loads of longer duration which may be present (or nett short term load), we have by the same reasoning:

For dry exposure, $$E = \frac{E_o \text{ (dry)}}{1 + \frac{0.06}{f_S} (\sigma_S - \sigma_{ML})}$$

For wet exposure, $$E = \frac{E_o \text{ (wet)}}{1 + \frac{0.04}{f_S} (\sigma_S - \sigma_{ML})}$$

Similarly in the case of very short load less any other loads of longer duration which may be present (or nett very short term load), we have

For dry exposure, $$E = \frac{E_o \text{ (dry)}}{1 + \frac{0.03}{f_{VS}} (\sigma_{VS} - \sigma_{SML})}$$

For wet exposure, $$E = \frac{E_o \text{ (wet)}}{1 + \frac{0.02}{f_{VS}} (\sigma_{VS} - \sigma_{SML})}$$

In the above expressions,

σ and f are actual and permissible stresses respectively;

Suffixes L, M, S and VS refer to long, medium short and very short term durations of load respectively;

Suffixes ML refer to medium or long term durations of load which may be present;

Suffix SML refers to short, medium or long term durations of load which may be present.

4.9 Modulus of rigidity

The modulus of rigidity G can be expressed in terms of the modulus of elasticity E and Poisson's ratio m in the form

$$G = \frac{m\,E}{2(1+m)}$$

For tempered hardboard, Lundgren (9) has found that G varies between 0.44E for 65%RH to about 0.4E for 90%RH. The Swedish Type Approval for K-Board adopts a value of G = 0.5E. Direct measurements of G values were made in the PRL/Yarsley 1975 tests and are given in Fig 3 and Table 4; these values are used.

Adopting the general principle that G is proportional to E, the equations given in 4.8 also apply by replacing E with G and replacing E_o with G_o, the instantaneous modulus of rigidity.

4.10 Basis for Draft Amendment Slip to CP112: Part 2: 1971

The draft given in Section 6.0 is based on the considerations in Sections 2.0, 3.0, 4.0 and 5.0, and written to be as compatible as possible with CP112: Part 2: 1971. The following points should be made:

a. All grade stresses are based on 5% exclusion of test results.

b. All grade stresses are expressed as long term stresses.

c. All instantaneous moduli are given as mean and minima (5% exclusion).

d. Modifications factors are expressed as multipliers of grade stresses and moduli; for moduli, the modification factors also allow for effect of level of stress.

e. Strength values for mechanical fastenings are being prepared and will be presented in a separate submission report.

f. Advice is given on gluing tempered hardboard.

g. Advice is given on conditioning and storage.

TABLE 4. Summary of PRL/Yarsley test results for 8.0mm nominal thickness board (brands A and B combined) from which proposed grade stresses are derived

Property	Number of tests	Mean	S D	C V (%)	$t_{0.95}$	Minimum (5% exclusion)
Bending (mesh in tension)	144	47.5	4.46	9.4	1.645	40.16
Compression in-plane	144	28.6	3.47	12.1	1.645	22.89
Tension in-plane	142	27.7	4.44	16.0	1.645	20.40
Panel shear	48	20.9	2.79	13.4	1.68	16.21
Rolling shear SG/US	72	3.38	0.421	12.4	1.67	2.68
Modulus of elasticity:						
Bending (mesh in tension)	144	5810	824	14.2	1.645	4455
Tension	141	5740	643	11.2	1.645	4682
Modulus of rigidity						
(mesh in tension)	144	2840	223	7.8	1.645	2473

Notes:

1 S D = Standard deviation

C V = Coefficient of variation

$t_{0.95}$ = Student's t values for 5% exclusion (from statistical tables)

Minimum value = Mean - $t_{0.95}$ (SD)

2 SG/US = Smooth face glued, unsanded specimens tested in double lap tension.

3 Only the lowest test results are summarised (see 4.2 to 4.4).

The modulus of elasticity in compression are not given or used because of the unreliability of measurement (see 3.1).

Bearing strength and tensile strength normal to board plane were not tested but Swedish values have been adopted (see 3.1).

4 Test condition 20°C, 65%RH (Dry exposure).

5 All strength values in N/mm^2.

Table 5. Derivation of grade stresses (long term load duration) (N/mm^2)

Property	Dry test results+	Dry grade stress		Wet grade stress	
	Five percentile	Multiplier on test result	Value	Multiplier on dry grade stress	Value
Bending	40.16	0.272	10.92	0.6	6.55
Compression parallel to board plane	22.89	0.272	6.23	0.45	2.80
normal to board plane	-		5.00 **	-	4.00 **
Tension parallel to board plane	20.40	0.272	5.55	0.64	3.55
normal to board plane	-	-	0.11 *	-	0.04 *
Panel shear	16.21	0.272	4.41	0.67	2.95
Rolling shear	2.68	0.272	0.73	0.86	0.63
Instantaneous modulus of elasticity: mean			(5775) x	0.87 xx	(5024)
minimum 5 percentile			(4568)	0.87	(3974)
Instantaneous modulus of rigidity: mean			(2840)		(2471)
minimum 5 percentile			(2473)		(2152)

+ From tests on 8mm thick boards, Brands A + B combined (see Table 4)

* Values adopted from those given in Swedish Type Approval (16); the Swedish values have been adjusted in the ratio of the characteristic strength in the samples tested to the minimum characteristic strength requirement in the Swedish Type Approval.

x Average of moduli in bending and in tension.

xx The factor of 0.87 is the ratio of wet to dry moduli of elasticity from Lundgren's creep tests (9).

** See 3.1.

5.0 PROTOTYPE TESTING OF STRUCTURAL COMPONENTS

5.1 Comparison with timber components

The recommendations given in clause 6.2 of CP 112 Part 2 1971 apply to timber and plywood structural components.

They do not apply to tempered hardboard components because the permissible stresses and elastic moduli are based on safety factors and creep allowances which are significantly different from those for timber and plywood. Thus different recommendations are required in the revised code to deal with tempered hardboard components.

However, for composite structure of timber and tempered hardboard, the situation is more complex and no specific and simple recommendations can be given, particularly as the location of critical stress or mode of failure, or the influence on deformation, may predominate in one or both materials. The designer must be left to apply his judgement to prescribe the appropriate magnitudes of loads for the deflection and strength tests and to interpret the results in such cases.

5.2 Deflection test

For timber or plywood structures clause 6.2.6 of CP 112 requires for acceptance that the deflection under full design load after 24 hours does not exceed 0.8 times the specified design deflection. No distinction is made between the durations of load which may be assumed to occur in service, since (rightly or wrongly) the grade elastic moduli are assumed to be independent of load duration; in practice some creep must occur, and the factor of 0.8 is understood to be a combined allowance for creep, joint slip and bedding down of the structure during the test. The following treatment for the deflection test takes full account of creep effects and accordingly it would be justified to apply a factor of 0.9 to tempered hardboard.

For tempered hardboard, it was demonstrated earlier that creep related to the level of stress and duration of load both modify the value of the effective elastic moduli. These relationships will now be used to determine how the deflection in a prototype deflection test over 24 hours ($10^{1.38}$ hours) should be related to service conditions having various durations of load.

5.2 Deflection test (continued)

It was shown in Table 3 and 4.8 that the modulus of elasticity E_{24}, of a tempered hardboard component loaded for 24 hours at a stress level σ is:

For dry exposure, $$E_{24} = \frac{E_o}{1 + 0.49 \frac{\sigma}{f}}$$

For wet exposure, $$E_{24} = \frac{E_o}{1 + 0.50 \frac{\sigma}{f}}$$

The values of 0.49 and 0.50 in the above equations are so close that a common value of 0.50 may be adopted in both cases for simplicity.

Similar equations obtain for the modulus of rigidity G (see 4.9) and as in general, the modulus of elasticity and the modulus of rigidity may be involved in calculating deflection, they may both be represented by a 'stiffness constant' S, and hence

$$S_{24} = \frac{S_o}{1 + 0.5\ \sigma/f}$$

If the prototype structure under test is assumed to be of 'balanced design', in the sense that it satisfies both the deflection and strength design requirements to their maximum permissible limits, σ/f may for simplicity be assumed to be unity, and hence

$$S_{24} = \frac{S_o}{1.50}$$

The design load W_D comprises one or more of the following constituent loads thus:

$$W_D = W_L + W_{NM} + W_{NS} + W_{NVS}$$

where the suffixes L, NM, NS and NVS denote long term, nett medium term, nett short term and nett very short term loads respectively (see 4.8).

However, because the effective stiffness of tempered hardboard is modified according to level and duration of load (or stress), and because the relative magnitude of the constituents of the design load may vary from one design to another, the appropriate load W_{24} to be applied in the 24 hour deflection test should be modified accordingly:

5.2 (continued)

From 4.5 Table 1,

$$W_{24} = 1.73\left(W_L + \frac{W_{NM}}{1.48} + \frac{W_{NS}}{2.27} + \frac{W_{NVS}}{2.46} \right)$$

$$= 1.73W_L + 1.17W_{NM} + 0.76W_{NS} + 0.70W_{NVS}$$

Now for a structure of given geometric properties and load distribution, the deflection y may be written as:

$$y = K\frac{W}{S}$$

where K is a constant for the geometry and load distribution;

and the deflection for the various duration of load cases under discussion may be written:

$$y_{24} = K\frac{W_{24}}{S_{24}}$$

$$y_L = K\frac{W_L}{S_L}$$

$$y_{NM} = K\frac{W_{NM}}{S_{NM}}$$

and similarly for y_{NS} and y_{NVS}.

Also, $$S_L = \frac{S_o}{1 + 6.79\frac{\sigma_L}{f_L}}$$ (dry exposure)

$$= \frac{S_o}{1 + 6.79\dfrac{W_L}{W_L + \dfrac{W_{NM}}{1.48} + \dfrac{W_{NS}}{2.27} + \dfrac{W_{NVS}}{2.46}}}$$

$$= \frac{S_o}{1 + 6.79\dfrac{1.73W_L}{W_{24}}} = \frac{S_o}{1 + 11.75\dfrac{W_L}{W_{24}}}$$

and $$S_{NM} = \frac{S_o}{1 + 2.86\dfrac{W_{NM}}{\left(W_L + \dfrac{W_{NM}}{1.48} + \dfrac{W_{NS}}{2.27} + \dfrac{W_{NVS}}{2.46}\right)1.48}}$$ (dry exposure)

$$= \frac{S_o}{1 + 2.86\dfrac{1.73 \times W_{NM}}{W_{24}}} = \frac{S_o}{1 + 3.34\dfrac{W_{NM}}{W_{24}}}$$

and similarly for S_{NS} and S_{NVS}, and S values for the damp exposure condition.

By substitution between the foregoing equations we derive simplified equations for predicting the deflections under service conditions from the 24 hour test, from:

$y = K\frac{W}{S} = \frac{y_{24}}{W_{24}} . S_{24} \cdot \frac{W}{S}$ as tabulated on the following page.

5.2 (continued)

"

Equations for calculating the predicted service design deflections, from the 24 hour deflection test under load W_{24}

Exposure condition*	Duration of load	Equation
Dry	Long term	$y_L = y_{24} \frac{W_L}{W_{24}} (0.67 + 7.83 \frac{W_L}{W_{24}})$
	Nett medium term	$y_{NM} = y_{24} \frac{W_{NM}}{W_{24}} (0.67 + 2.23 \frac{W_{NM}}{W_{24}})$
	Nett short term	$y_{NS} = y_{24} \frac{W_{NS}}{W_{24}} (0.67 + 0.03 \frac{W_{NS}}{W_{24}})$
	Nett very short term	$y_{NVS} = y_{24} \frac{W_{NVS}}{W_{24}} (0.67 + 0.01 \frac{W_{NVS}}{W_{24}})$
Damp	Long term	$y_L = y_{24} \frac{W_L}{W_{24}} (0.67 + 13.61 \frac{W_L}{W_{24}})$
	Nett medium term	$y_{NM} = y_{24} \frac{W_{NM}}{W_{24}} (0.67 + 3.10 \frac{W_{NM}}{W_{24}})$
	Nett short term	$y_{NS} = y_{24} \frac{W_{NS}}{W_{24}} (0.67 + 0.02 \frac{W_{NS}}{W_{24}})$
	Nett very short term	$y_{NVS} = y_{24} \frac{W_{NVS}}{W_{24}} (0.67 + 0.01 \frac{W_{NVS}}{W_{24}})$

* The prototype must be tested in the same condition of exposure as the structure in service for these equations to apply.

(End of revision to 5.2)

5.3 Strength test

For timber or plywood components clause 6.2.6 of CP112 requires for acceptance that the component should sustain at least 2½ times the design load without failure. Assuming for the worst case that the design load is of long term duration, this global factor of safety can be considered as being the product of a duration of load factor, taken as 1.6 (see 4.7), and a safety margin of 2.5 ÷ 1.6 = 1.5625.

For structures wholly of tempered hardboard the load duration factor for a prototype test of 40 minutes is 2.0 (see 4.5) and therefore the corresponding global factor of safety may be deduced as 1.5625 x 2.0 = 3.125 for long term design load.

For loads of various durations the global factor of safety are:

Duration of load	Prototype test: multiplier on design load	
Long		3.125
Medium	3.125 x 1/1.48 =	2.111
Short	3.125 x 1/2.27 =	1.377
Very Short	3.125 x 1/2.46 =	1.270

It should be explained that the prototype testing duration of 40 minutes is arbitrary. CP112 does not specify the rate of loading for the strength test, but from experience the time taken to complete a typical test is about one hour; however during this time the load is being progressively increased so that the effective duration of load affecting strength is less than one hour, say 40 minutes.

5.4 Tests on similar components

Clause 6.2.8 of CP 112 gives the modification factor K_{26} which permits factors of safety to be lower than 2½ for two and more similar components tested. Since the scatter of strength and elastic moduli values for tempered hardboard are no greater than those for timber, the basis of deriving K_{26} could be applied to components of tempered hardboard.

6.0 DRAFT CLAUSES FOR THE REVISION OF CP 112

The clauses in this Section are drafted in the form of an Amendment Slip to the existing CP112: Part 2: 1971.

The system of clause numbering is related to CP112 and not to the forgoing sections of this report.

Amendment Slip No. ..., published ...(date)...

to CP112: Part 2: 1971

The structural use of timber

Part 2. Metric units

NOTE. This amendment extends the scope of CP112 to include the use of TE grade tempered hardboard complying with BS1142: Parts 1 and 2 'Specification for fibre building boards'.

Revised text

Page 2 — In the list of reference standards, insert the following: 'BS1142: Parts 1 and 2 'Specification for fibre building boards'.

Clause 1.1 — Scope. Insert in the first line after the word 'timber': 'and certain wood based sheet materials'.

Clause 2.1 — Materials. Under the heading 'Timber, insert the following: 'Specification for fibre building boards' BS1142.

Clause 3.24.2 — In the title and the text, add, after 'plywood', the words 'or tempered hardboard'.

Section 4A — Between Section 4.PLYWOOD and 5.WORKMANSHIP insert the following new Section 4A:

4A. TEMPERED HARDBOARD

4A.1 MATERIALS

The information on tempered hardboard applies only to TE grade tempered hardboard complying with BS1142: Part 2, for nominal thicknesses up to 8mm.

Where it is proposed to use other hardboards defined in BS1142 for stressed members, advice should be sought from the Princes Risborough Laboratory. Particular attention is drawn to the need to condition tempered hardboard prior to fixing (see Clause 5.2.2.).

4A.2 DURABILITY

The risk of decay of tempered hardboard in the uses specified under clause 4A 5.3 is low since the fungi will only attack boards with a moisture content well above their equilibrium moisture content in a saturated atmosphere.

The design of structural components in tempered hardboard should ensure that the material is not permanently exposed to saturated wet conditions; providing this is done, no further precautions by preservative treatment need be considered.

Wood-boring insects which occur in the UK will not normally attack fibre building boards, since much of the food (starches and sugars) they eat has been removed during manufacture.

4A.3 GRADE STRESSES FOR TEMPERED HARDBOARD

The grade stresses for TE grade tempered hardboard are given in Tables 49A and 49B. The geometric properties given in Table 49C are based on the minimum thicknesses given in BS1142. Where minimum thickness exceeds these values, the geometric properties may be calculated from the actual thickness of the board used.

4A.4 PERMISSIBLE STRESSES AND EFFECTIVE MODULI

The permissible stresses for tempered hardboard are governed by the particular conditions of exposure and load duration. They should be taken as the product of the grade stress and the appropriate modification factor given in 4A.5.

The effective modulus of elasticity and modulus of rigidity are governed by the particular conditions of exposure, load duration and magnitude of applied stress. The effective modulus should be obtained from Table 49C or 49D.

TABLE 49A. DAMP GRADE STRESSES
TEMPERED HARDBOARD*

Type of stress	Value of stress
	N/mm^2
Extreme fibre in bending	6.55
Tension in board plane	3.55
Tension perpendicular to board plane	0.04
Compression in board plane	2.80
Bearing	4.00
Rolling shear	0.63
Panel shear	2.95

*The conditions for damp exposure are defined under Clause 4A 5.3.

TABLE 49B. DRY GRADE STRESSES
TEMPERED HARDBOARD*

Type of stress	Value of stress
	N/mm^2
Extreme fibre in bending	10.90
Tension in board plane	5.55
Tension perpendicular to board plane	0.11
Compression in board plane	6.23
Bearing	5.00
Rolling shear	0.73
Panel shear	4.41

*The conditions for dry exposure are defined under Clause 4A 5.3.

TABLE 49C DAMP EXPOSURE VALUES OF MODULUS OF ELASTICITY AND MODULUS OF RIGIDITY FOR TEMPERED HARDBOARD

Ratio of applied stress to permissible stress	Modulus of elasticity (N/mm^2) for durations of load of				Modulus of Rigidity (N/mm^2) for durations of load of			
	Long term	Nett Medium term	Nett Short term	Nett Very Short term	Long term	Nett Medium term	Nett Short term	Nett Very Short term
0.00	5020	5020	5020	5020	2470	2470	2470	2470
0.05	3157	4187	5010	5015	1553	2060	2465	2468
0.10	2303	3591	5000	5010	1133	1767	2460	2465
0.20	1494	2795	4980	5000	735	1375	2450	2460
0.30	1106	2288	4960	4990	544	1126	2441	2455
0.40	877	1937	4941	4980	432	953	2431	2450
0.50	727	1679	4922	4970	358	826	2422	2446
0.60	621	1482	4903	4960	306	729	2412	2441
0.70	542	1326	4883	4951	267	652	2403	2436
0.80	481	1200	4864	4941	237	590	2393	2431
0.90	432	1096	4846	4931	213	539	2384	2426
1.00	392	1008	4827	4922	193	496	2375	2422

TABLE 49D DRY EXPOSURE VALUES OF MODULUS OF ELASTICITY AND MODULUS OF RIGIDITY FOR TEMPERED HARDBOARD

Ratio of applied stress to permissible stress	Modulus of elasticity (N/mm^2) for durations of load of				Modulus of Rigidity (N/mm^2) for durations of load of			
	Long term	Nett Medium term	Nett Short term	Nett Very Short term	Long term	Nett Medium term	Nett Short term	Nett Very Short term
0.00	5780	5780	5780	5780	2840	2840	2840	2840
0.05	4315	5057	5763	5771	2120	2485	2831	2836
0.10	3443	4494	5745	5763	1692	2208	2823	2831
0.20	2451	3677	5711	5745	1204	1807	2806	2823
0.30	1903	3111	5678	5729	935	1528	2790	2815
0.40	1555	2696	5645	5711	764	1325	2774	2806
0.50	1315	2378	5612	5694	646	1169	2757	2798
0.60	1139	2128	5579	5678	560	1046	2741	2790
0.70	1005	1925	5547	5661	494	946	2726	2782
0.80	899	1758	5515	5645	442	864	2710	2774
0.90	813	1617	5483	5628	399	795	2694	2765
1.00	742	1498	5453	5612	365	736	2679	2757

(Footnote to Tables 49C and 49D)

In the above Tables:

Nett Medium term load means the medium term load defined in Table 49F of 4A.5.1 less any long term load;

Nett Short term load means the short term load defined in Table 49F of 4A.5.1 less any long term and Nett Medium term loads; and

Nett Very Short term load means the very short term load defined in Table 49F of 4A.5.1 less any long term, Nett Medium term, and Nett Short term loads.

TABLE 49E DIMENSIONS AND PROPERTIES OF TEMPERED HARDBOARD COMPLYING WITH BS 1142

Nominal thickness	Section properties for a 1m width*				Average weight
	Thickness (nett)	Area	Section modulus	Second moment of area	
mm	mm	$10^3 mm^2$	$10^3 mm^3$	$10^3 mm^4$	$kg f/m^2$
3.2	2.7	2.7	1.22	1.64	3.2
4.8	4.3	4.3	3.08	6.63	4.8
6.4	5.9	5.9	5.80	17.11	6.4
8.0	7.3	7.3	8.88	32.42	8.0

*These section properties are based on the minimum thickness given in BS1142.

4A.5 MODIFICATION FACTORS

4A.5.1 Duration of load. The grade stresses given in Tables 49A and 49B are applicable to long term loading; where the effects of duration of load are being considered, these stresses should be multiplied by the modification factor K_{27} given in Table 49F.

TABLE 49F. MODIFICATION FACTOR K_{27} ON GRADE STRESSES FOR DURATION OF LOADING FOR TEMPERED HARDBOARD

Duration of load	Value of K_{27} for stress
Long term (eg dead + permanent imposed)	1.00
Medium term (eg dead + snow, dead + temporary loads)	1.50
Short term (eg dead + imposed + wind, dead + imposed + snow + wind)	2.25
Very short term (eg dead + imposed + 5 second wind gust)	2.50

When advantage is taken of this clause to use a modification factor K_{27} greater than unity, the design should be checked to ensure that the permissible stresses are not exceeded for any other condition of loading which may be relevant.

4A.5.2 Exposure condition. Tempered hardboard attains an equilibrium moisture content substantially lower than that for solid timber or plywood for a given exposure condition. However, tempered hardboard should not be used in continuously saturated or immersed conditions; it should be restricted to protected conditions of structural use. The following definitions of exposure apply to tempered hardboard:

Damp condition. Structures exposed to a relative humidity normally not exceeding 85% or for brief periods not exceeding 95%, eg structural members such as protected external walls of unheated buildings when there is a risk of moisture concentration (garages, unheated buildings, ground floor access spaces) or temporary structures (scaffolding or concrete formwork).

4A 5. 2 Exposure condition (continued)

Dry condition. Structures exposed to a relative humidity normally not exceeding 65%, or for brief periods not exceeding 75%, eg structural members located indoors such as floors, external walls of permamently heated buildings protected by ventilated cladding.

4A.6 DEFLECTION

The deflection of box beams and I-beams should be calculated from the full section properties of the tempered hardboard and timber, together with the appropriate modulus of elasticity and modulus of rigidity for tempered hardboard from Tables 49A, 49B and 49E.

The shear deflection of a beam may be significant and should be taken into account.

The total deflection should be obtained by adding together the deflections calculated separately for long term, nett medium term, nett short term and nett very short term loads appropriate to the structural member.

4A.7 LATERAL STABILITY OF BOX AND I-SECTIONS

Clause 4.10 applies to tempered hardboard.

4A.8 MECHANICAL FASTENINGS IN TEMPERED HARDBOARD

Clause 4.10 applies to tempered hardboard.

ıuse 5.2.2 Add a new second paragraph as follows:

Tempered hardboard expands linearly by up to 0.3% with relative humidity change from 33% to 90%. If a dry board is fixed securely to a rigid framework and later takes up more moisture, there would be a tendency for the board to buckle, depending on the thickness and the unrestrained length and width. To avoid this, boards should be conditioned with water before fixing. Water should be applied to the mesh face at the rate of approximately 0.5 litre for every 3.2mm thickness for a board size of 1220 x 2440, two or three days before fixing. After water is applied, the boards should be stacked mesh to mesh on a flat surface away from sunlight or heat sources.

Clause 5.4.7 Glued joints. In the second paragraph add new sentence after the first sentence, as follows:

'Tempered hardboard can be glued on either face but the smooth face if glued should first be lightly sanded.'

Clause 6.2.6 Insert between second and third paragraphs the following:

Where the structure or element is of tempered hardboard acting compositely with timber, the magnitude of the loads to be applied and the interpretation of the deflection and strength tests can only be decided with reference to the relative strengths and stiffnesses of the composite materials in the particular structure or element to be tested.

Clause 6.2.6 (2) Insert between first and second paragraphs the following:

For a structure or element wholly of tempered hardboard, a modified maximum design load W_{24} to be applied for the deflection test, to take into account the effects of duration of load and level of stress, should be determined by:

$$W_{24} = 1.73W_L + 1.17W_{NM} + 0.76W_{NS} + 0.70W_{NVS}$$

where W_L, W_{NM}, W_{NS} and W_{NVS} are the loads under conditions of service of long term, Nett Medium term, Nett Short term and Nett Very Short term durations respectively (see Tables 49C and 49D), as appropriate to the investigation.

Clause 6.2.6 Method of testing

(3) Strength test. Add new sentence to end of first paragraph as follows:

For a structure or element composed wholly of tempered hardboard, the load to be applied should be determined as follows:

Duration of load	Number of times design load to be applied
Long term	3.0
Medium term	2.0
Short term	1.4
Very Short term	1.3

Where the design load is of medium, short, or very short duration, the load to be applied should be checked to ensure that it is not less than the load to be applied for the worst condition of loading which may be relevant.

Clause 6.2.7 Insert between the first and second paragraphs the following:

For a structure or element wholly of tempered hardboard, the deflection at the end of the 24 hour period loaded to the modified maximum design W_{24} should be used to derive the predicted deflections under service conditions, in accordance with the equations given in Table 52A. The predicted deflections under service conditions so derived should not exceed 0.9 times the specified amount in the design.

TABLE 52A.

Equations for calculating the predicted service design deflections, from the 24 hour deflection test under load W_{24}

Exposure condition*	Duration of load	Equation
Dry	Long term	$y_L = y_{24} \frac{W_L}{W_{24}} (0.67 + 7.83 \frac{W_L}{W_{24}})$
	Nett medium term	$y_{NM} = y_{24} \frac{W_{NM}}{W_{24}} (0.67 + 2.23 \frac{W_{NM}}{W_{24}})$
	Nett short term	$y_{NS} = y_{24} \frac{W_{NS}}{W_{24}} (0.67 + 0.03 \frac{W_{NS}}{W_{24}})$
	Nett very short term	$y_{NVS} = y_{24} \frac{W_{NVS}}{W_{24}} (0.67 + 0.01 \frac{W_{NVS}}{W_{24}})$
Damp	Long term	$y_L = y_{24} \frac{W_L}{W_{24}} (0.67 + 13.61 \frac{W_L}{W_{24}})$
	Nett medium term	$y_{NM} = y_{24} \frac{W_{NM}}{W_{24}} (0.67 + 3.10 \frac{W_{NM}}{W_{24}})$
	Nett short term	$y_{NS} = y_{24} \frac{W_{NS}}{W_{24}} (0.67 + 0.02 \frac{W_{NS}}{W_{24}})$
	Nett very short term	$y_{NVS} = y_{24} \frac{W_{NVS}}{W_{24}} (0.67 + 0.01 \frac{W_{NVS}}{W_{24}})$

* The prototype must be tested in the same condition of exposure as the structure in service for these equations to apply.

Clause 6.2.7 At end of last sentence replace full-stop by semi-colon and add:

For a structure or part of a structure wholly of tempered hardboard, the corresponding load to be sustained without failure is given in 6.2.6 (3).

7.0 REFERENCES

1 BS1142 1971: Specification for fibre building boards.

2 Part 1. Methods of test.
Part 2. Medium board and hardboard.

2 BS4512 1969: Methods of test for clear plywood.

3 Tests on the strength properties for the structural use of tempered hardboard. Princes Risborough Laboratory (in course of preparation).

4 Draft standard method of test for fibre building boards - determination of the elastic modulus and the tensile strength parallel to the faces. ISO/TC89(WG9/1) 227, 1972.

5 The Z strength of fibre building boards. Swedish Forest Products Research Laboratory, 1968.

6 G A Bennett, Comparisons between two methods of determining Z strength of hardboard. Princes Risborough Laboratory, Nov 1973.

7 Determination of Z strength test procedures for hardboard. Princes Risborough Laboratory report PR/117/079 RW862 F251, 1973.

8 Draft standard method of test for fibre building boards - determination of transversal internal bond (ISO/DIS3931). BSI document 75/12607DC, Nov 1975.

9 S Ake Lundgren. Wood-based sheet as a structural material. Part 1. The Swedish Wallboard Manufacturers Association, 1969.

10 The structural properties of hardboard. FPRL report FPR195/01, 1969-71.
Part I. Plan of experiments and methods of test.
Part II. Shear strength
Part III. Bending strength
Part IV. The effect of duration of load
Part V. Derivation of basic stresses.

11 An investigation of some properties of tempered hardboard. BRE Princes Risborough Laboratory Timberlab report no.25, 1970.

12 G A Bennett, Finding more about fibre building boards. BRE Princes Risborough Laboratory Timberlab report no.42, 1971.

13 J D McNatt, Design stresses for hardboard - effect of rate, duration and repeated loading. Forest Products Journal Vol 20, no.1, 1970.

14 J D McNatt, Effect of equilibrium moisture content changes on hardboard properties. UK Forest Products Journal, Vol 24, no.2, 1974.

7.0 REFERENCES (Continued)

15 J A Bristow and E L Back, The moisture content of fibre building boards. Swedish Forest Products Journal Vol 24, no.2, 1974.

16 Structural fibre building board, Type Approval certificate T75/71. Swedish Department of Town Planning, March 1972.

17 H Kolb, Properties of fibre hard boards and their application in construction. Holz als Rob- und Werkstoff, Bd.29 (1971) Seite 10-23, Springer-Verlag (German, with English translation for FIDOR.

18 B S Aswathanarayana, V J Victor and J George, A study of the properties of hardboards. Part 1: Physical and mechanical properties of oil tempered hardboards. Indian Plywood Industries Research Institute Journal Vol 3, no.4, 1973.

19 CP112: Part 2: 1971. The structural use of timber. BSI.

20 Revision of CP112, Draft Section Five. Paper prepared by Princes Risborough Laboratory for subcommittee BLCP17/2, 1974.

21 O Brynildsen, H J Larsen and B Noren; Structural use of particle board and fibre board. Paper presented to the International Union of Forestry Research Organisation (IUFRO) 1975.

22 R A Laidlaw and R W Hudson, The gluing of tempered hardboard. Note N/75, Princes Lisborough Laboratory, September 1975.

23 Tecton beams with hardboard webs. Princes Risborough Laboratory report 136/032 S476, October 1972.

24 Strength and deflection tests on a hardboard lintel. BRE Princes Risborough Laboratory Report ST/GEPD0109, SI107 S505, February 1973.

25 Predicting performance of hardboard in I-beams. USDA Forest Service Research Paper FPR 185, 1972.

26 M J Superfesky and T J Ramaker. Hardboard-webbed I-beams subjected to short-term loading. USDA Forest Service Research Paper FPL 264, 1976.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TIMBER TRUSSES - CODE RELATED PROBLEMS

by

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TIMBER TRUSSES - CODE RELATED PROBLEMS

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INTRODUCTION

A review of the design procedures in Europe and the Nordic countries shows that despite a vastly differing selection of design procedures, the section sizes required to support a common load are within 10% of each other. This is assuming either high grade material or medium grade material. The point here is that each country with its own regulations is arriving at roughly the same conclusions.

A timber Engineer in France when attempting to use the design rules for say Denmark or vice versa, will have difficulty in understanding the regulations and applying them in the spirit in which they were conceived. It presumably matters not that were he to calculate his structure using his own country's regulations, his answer would be roughly the same. He must submit the calculation in a form required by the country concerned.

With each new revision of National Norms, the design methods, especially for trussed rafters, are being made more complex, and although the results obtained are all within an acceptable structural limit, the design methods are becoming more and more diverse between countries.

The purpose of Codes generally is to inform the uninformed and to contain the over-informed. It is the latter category, unfortunately, that is perhaps the reason for each country becoming more and more specific in regard to design procedures. This attitude is leading to the confusion of the uninformed and the bankruptcy of the over-informed.

Any International company involved in the design of trusses faces an ominous and increasingly expensive task in ensuring that each country's specific regulations are satisfied.

The author will not presume, in this paper, to draft a standard procedure, but will highlight some of the confusing and non-standard procedures currently adopted by individual countries.

COMBINATIONS OF LOAD

The first draft of the CIB Timber Code - Paper 6-100-2 Chapter 3.0 Page 1, paragraphs 3 and 4 state:

"All relevant limit states should be considered in the design in order to ensure an adequate degree of safety and serviceability.

Loads are assumed to be determined in accordance with current national or international rules taking into account the stresses that might occur due to the deformations caused by moisture."

There is nothing wrong with these clauses. There is also nothing wrong with the statement, "The Structure must Stand up". When we expand on this statement and explain how to ensure that the structure "stands up" we are committed to an advisory course and not a regulatory course. The code has expanded in depth on certain esoteric aspects of the design of timber structures but this fundamental element of design has at present been ignored.

In ignoring this aspect we are allowing the current international proliferation of load combinations to continue. To date, the following has been noted:

South Africa: Draft Timber Code 1 May 1978	THREE load cases and four load duration factors. Duration classes differ from European standards.
United Kingdom: Draft BS5268: Part 3 1977 Trussed rafters for roofs	NINE individual load combinations for axial force or bending in rafter or ceiling tie must be combined to locate the critical case. These are worked up using 4 load duration factors.
France: Recommendations pour le calcule de fermes et industrialises en bois	FIVE load combinations with some associated load duration factors.
Germany: DIN1052 and DIN1055	SEVEN load cases.
Nordic Countries: Common draft documents	FOUR - SEVEN load cases with associated load duration factors. Documents are planned to replace existing National Codes.

The determination of critical combinations of load is a necessity in any structural design whether based on limit state or working stress concepts. When it is a definable structural frame with definable service conditions it must surely be possible to simplify the combinations and duration factors, and this is what is required in the CIB code to make the life of the Engineer less complex and the International Trade Barriers less severe.

If the Engineer wishes to further complicate his design method in order to effect greater economy, he may do so by taking advantage of Clause 3.0: General. However, in terms of trussed rafter design, the omissions of load combinations send National Code Drafters scurrying to their text books with the resultant confusion and expense.

ANALYSIS AND DESIGN

In 1970 the Truss Plate Institute of America published "Design Specifications for Light Metal Plate Connected Wood Trusses". Section 300 contained Design Procedures. The basis of these procedures was the assumption that the truss was a pin-jointed continuous frame work in which the joints are assumed to transmit axial forces only. An exception was made of the heel joint and following extensive work at Purdue by Suddarth, "Heel Coefficients" (303.1.a) were derived in order that moment effects and the effect of angle of load to plate at this joint could be taken into account. This basic concept has been in use for some 23 years in the U.S.A. and the incidence of failure due to design method has not merited any significant change to date.

The first European country to publish a National document which set out design procedures for trusses was U.K. (FPRL publication Timberlab 29 November 1970). This document closely followed the concept of the TPI 70 document. The French and German design procedures also began with and are still following this concept. The Scandinavian countries, however, did not accept the principle of pinned joints and consequently developed a semi-rigid design method.

The current position is that each of the pin-jointed countries have differing moment coefficients and column formulae, and the semi-rigid countries have different lever arm calculations about the centroid of a plated area, which affects the amount of force and moment being transferred into a member.

Apart from the Belgians who will accept the French method and the Dutch and Swiss who will accept the German method, every other country using trusses requires a different design method to be submitted to their checking Engineers as proof of the integrity of the structure.

Does the incidence of failure in each country suggest that diversity or complexity in design is the clue to safer structures? It cannot be so. We are now at the stage of development where a rationalised design method for trusses is imperative and a CIB Code is the obvious vehicle for such a method. In order to encompass the more complex semi-rigid, elastic stiffness and plastic design methods, it will be necessary to standardise on such things as the magnitude of the spring constant in a joint. This is a function of the connector plate and will require a test procedure that will measure rotational as well as axial slip at the plate. To convert axial slip for use as a rotational slip in a joint is not comparing apples with apples and if we are to have a sophisticated design method we must be sophisticated about it. Similarly if we have a simple design method which we must, let us be simple about it; selection of moment coefficients is the first step.

QUALITY CONTROL IN MANUFACTURE

Most countries using trusses have instituted a manufacturing Quality Control Scheme. The schemes either form part of an existing national scheme operated by Standards Bureaux or Institutes, or are a separate agreement entered into by members of a trade association. The parameters of the Quality Control are very similar, internationally, and with some 'juggling' of tolerances, W18 could produce a document that would form a valuable Appendix to the Code.

BRACING OF TRUSSES

A report on the contributory factors affecting the failure of timber structures in South Africa (The State of the Art of Timber Engineering S. Kaplan) has shown that the main contributory cause of roof failure is the lack of bracing. There are few codes that pay attention to this critical area of design. Australia and Germany have included chapters on bracing but both acknowledge that the information given is insufficient. Clauses such as "the structure must be adequately braced" are no longer appropriate in our Codes. Should a similar study to the South African one be conducted across Europe, the author has no doubt that the findings would be concurrent.

One of the major problems in setting out a design procedure for bracing is that the elements cannot be isolated and tested in a laboratory in a realistic manner in order to derive acceptable formulae. It is therefore necessary for the Engineer to use his judgement and experience in determining the magnitude and location of the bracing.

Draft 1 of the South African Code of Practice for the Structural Use of Timber, 1 May 1978 contains formulae which will aid the Engineer while he is sucking his thumb. Relevant extracts are repeated below:

7.3.9.1 Single Struts with constant axial forces

For a single strut prevented from buckling by lateral restraints, the force on each lateral restraint is calculated from the formula:

$$F_L = \frac{0,10F_A}{N}$$

where F_L = force on each lateral restraint

N = number of lateral restraints

F_A = average axial force in strut due to dead load only

7.3.9.2. Cumulative effect of a number of struts supported by the same restraint system

Where a number of members (m) are supported by the same restraint system, then the cumulative force in each lateral restraint is calculated from the lesser of $(\frac{m+1}{2})$ and 5 multiplied by the force (F_L) from each member.

7.3.9.5 The interval between braced bays should be calculated from the formula:

interval between bays (metres) = 16,5 - 0,3 x (span in metres)

The formulae given in 7.3.9.1 and .2 are modified formulae taken from the Australian Timber Code. The merit of these formulae is that they enable the Engineer to design a bracing system that will work on paper as well as in the roof. The theoretical background is shaky and indeed the formulae for the interval between bracing has no theoretical base at all. But it works!

Given time and money three important elements of the design of bracing could be isolated and researched. These are:

a) The degree of stiffness afforded by the batten or purlin connection to the rafter.

b) The magnitude of the lateral restraining force required in a strut, and a strut with bending.

c) How many components contribute to the total lateral forces applied to a bracing system.

CONCLUSION

No other timber structure has crossed so many International boundaries or has been the subject of such sophisticated computerised design methods. This puts the timber truss in a special category, especially in my view, of the large and increasing volume of components designed and manufactured every year. It is this special category feature that merits a closer look at the contents of the W18 Code in terms of trusses.

It is the author's conclusion that the type and volume of work necessary to deal properly with this 'special category' is beyond the capabilities of a corresponding subcommittee and should form the brief of a working party.

REFERENCES

1. Anon. CIB Timber Code First Draft 76.04.20 published by Institute for Bygningsteknik, Aalborg, Denmark.

2. Anon. South African Draft Code of Practice for the Structural use of Timber, 1 May 1978.

3. Anon. Draft BS 5268: Part 3, 1977. Trussed rafters for roofs. Published by the British Standards Institute.

4. Recommendations pour le calcule de fermes et industrialises en bois. Published by the C.T.B. Anon.

5 Anon. German Din 1052.

6. Lars Östlun. Draft documents for the Nordic Countries.

7. Anon. TPI 70 Specifications for Light Metal Plate Connected Wood Trusses published by the Truss Plate Institute of America.

8. Anon. Recommendations for trussed rafters for domestic roofs. Timberlab Paper No. 29, 1970. Forests Products Research Laboratory, Princes Risborough, England.

9. S. Kaplan. The State of the Art of Timber Engineering. Published by Wood Southern Africa, July 1977.

10. Anon. Australian Standard 1720 - 1975. SAA Timber Engineering Code 1975.

CIB-W18/9-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE CIB TIMBER CODE
(Second Draft)

PERTH, SCOTLAND
JUNE 1978

CONSEIL INTERNATIONAL DU BÂTIMENT ★ INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING GROUP W18
TIMBER STRUCTURES

CIB TIMBER CODE

Second draft
May 1978

FOREWORD

A first draft of the CIB Timber Code was discussed at CIB-W18 meetings in June 1976 (document CIB-W18/6-100-2), February 1977 (document CIB-W18/6-100-2; Joints) and October 1977 (document CIB-W18/8-100-1; List of Contents).

Based on comments received on these documents a final draft was prepared of the List of Contents and presented to ISO/TC 165 for comment. A new preliminary draft for the Timber Code was prepared and discussed by a Code Drafting Sub-Committee consisting of CIB members G. Booth, W. T. Curry, J. Kuipers, H. J. Larsen, K. Möhler, J. G. Sunley, J. R. Tory and also T. A. Eldridge, F. Keenan and W. R. Meyer representing the Canadian Standards Association and Dutch TNO.

This final version of the code has been prepared by G. Booth, H. J. Larsen and J. R. Tory.

The draft contains only rules for the design of timber structures and recommendations which define their validity. It does not contain rules common to the construction of other structures or safety criteria, and reference is made to Comité Euro-International du Béton, Volume 1, Common Rules for Different Type of Construction and Material. When a final version of this document is produced Chapter 3, Basic Design Rules, will be included.

This draft standard is equally applicable to either deterministic or partial factor methods of design provided material properties are given as characteristic values and suitable safety factors for strength and stiffness parameters are introduced to the design calculations.

Related Documents

The draft code makes reference to other documents at a preliminary stage which have been submitted to ISO/TC 165 for comment. These are:

Timber Structures - Joints - Determination of Strength and Deformation Characteristics of Mechanical Fasteners - prepared by CIB-W18 & RILEM 3TT.

Timber Structures - Plywood - Determination of some Physical and Mechanical Properties.

Timber Structures - Timber in Structural Sizes - Determination of some Physical and Mechanical Properties.

Other documents relating to the sampling of test specimens and the analysis of test data to produce characteristic values will be prepared by CIB-W18.

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1. INTRODUCTION

1.1 Scope

Thr primary purpose of this code is to provide an agreed background for the international bodies and national committees responsible for formulating timber codes, to ensure a reasonable and consistent quality of timber structures.

The code relates primarily to the structural use of timber and is intended for use in the design, execution and appraisal of structural elements made from timber or wood products and of structures substantially composed of such elements.

It is based on the principles of structural mechanics, engineering design, and experimental data, interpreted statistically as far as possible.

Deviations from the requirements of this code and the use of materials and methods of design or construction of wood structures not covered by this code are permitted when the validity is substantiated by analytical and engineering principles or reliable test data, or both, which demonstrate that the safety of the resulting structure for the purpose intended is equivalent to the safety demanded in this code.

1.2 Conditions for the validity of this document

Safety and serviceability are not simply functions of design, but depend also on the care and skill of all personnel involved in the construction process, and on the proper use and maintenance of the structure. Essential requirements are therefore that

- projects are carried out by qualified engineers,
- the construction is carried out by personnel having both the required skill and experience,
- the required supervision is always available,
- the structure, by design or the use of suitable materials or by impregnation, is protected against attack by fungi, insects, shipworm, gribble, etc.,
- the intended life of the structure is ensured by correct maintenance,
- the actual conditions of use of the structure during its life do not depart significantly from those specified during the design stage.

1.3 Units

The units used are generally in accordance with the »International System of Units, SI» and »Rules for the Use of the International System of Units» established by ISO and prepared by ISO/TC98/SC2.

Exceptions are the units for time, temperature and plane angle. In accordance with common and well established practice the Celsius scale is used rather than the Kelvin scale for thermodynamic temperature; degrees are used rather than radians as the non-dimensional units of plane angle; and hours, days, weeks, months and years are accepted as derived units of time.

The following basic units and derived units are used for structural timber design calculations:

Table 1.3 Units for structural timber design

Physical quantity	Unit	Abbreviation (and derivation)
Length	Metre	m
Mass	Kilogram	kg
Temperature	Degree Celsius	°C
Time	Second	s
Plane angle	Degree	° $(1° = \frac{180}{\pi})$
Force	Newton	N (1 N = 1 kgm/s^2)
Stress, pressure Elastic moduli	Pascal	Pa (1 Pa = 1 N/m^2, 1 MPa = 1 N/mm^2)

Only multiples of $10^{\pm 3}$; e.g. MN, kN, N are used.

1.4 Notations

The notations used are in accordance with International Standard ISO 3898.

In addition the notations given in document CIB-W18-1 »Symbols for Use in Structural Timber Design» are used.

The following general terms and symbols are used. Symbols which are not explained here are defined when used.

(will be prepared at completion of the work)

1.5 Definitions

(will be prepared at completion of the work)

2. BASIC ASSUMPTIONS

2.1 Characteristic values

The characteristic strength and stiffness values given in this code for timber and wood-based materials are defined as lower 5-percentile values (i.e. 95% of all possible test results exceed the characteristic value) directly applicable to a very short-term load condition (3 to 5 mins.) at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02.

The characteristic bending strength values for solid timber are related also to a section depth of 200 mm.

For some elastic properties the mean values are also given in this code and are defined at the same temperature and humidity conditions as the characteristic values.

The specific gravity for a species or species group is defined as the lower 5-percentile value with mass measured at moisture content $\omega = 0$ and volume measured at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02.

2.2 Climate classes

Structures dependent on moisture content shall be assigned to one of the climate classes given below:

: The examples given below each climate class definition are particularly appropriate to European conditions.

Climate class 0
The climate class is characterized by a temperature of 23 ± 3°C and an average annual relative humidity not exceeding 0.40.

: This climate class corresponds to conditions in permanently heated buildings without artificial air-moistening.

Climate class 1
The climate class is characterized by a temperature of 23 ± 3°C and a relative humidity of the surrounding air never exceeding 0.80 and only exceptionally, and then only for short periods (less than a week), exceeding 0.60.

: The following structures can be included in this class:
: - structures in outer walls in permanently heated buildings where the structures are protected by a well-ventilated tight cladding.

Climate class 2
The climate class is characterized by a temperature of 23 ± 3°C and a relative humidity of the surrounding air only exceptionally, and then only for short periods (less than a week), exceeding 0.80.

: The following structures can be included in this class:
: - structures in not permanently heated, but ventilated buildings in which no activities particularly likely to give rise to moisture take place, for example, holiday houses, unheated garages and warehouses, together with service space,
: - ventilated roof structures and other structures protected against the weather.

Climate class 3
All other climatic conditions.

: The following structures are included in this class:
: - concrete forms and unprotected scaffolding,
: - marine works.

2.3 Load-duration classes

For strength and stiffness calculations actions are to be assigned to one of the load-duration classes given in table 2.3a.

The load-duration classes are characterized by the effect of a constant load acting for a certain period of time. For variable action the appropriate class is determined on the basis of an estimate of the interaction between the typical variation of the load with time and the rheological properties of the materials or structures.

: In table 2.3b are given examples of loads in the different classes for permanent buildings (i.e. a life time of 50-100
: years).

Table 2.3a Load-duration classes

load - duration class	duration
permanent	$> 10^5$ h ($>$ 10 years)
normal	10^3 - 10^5 h (6 weeks - 10 years)
short-term	10 - 10^3 h (10 h - 6 weeks)
very short-term	$<$ 10 h
instantaneous	$<$ 3 seconds

Table 2.3b Examples of action classifications

permanent	dead load earth and water pressure, loads in some warehouses and storage tanks
normal	floor loads loads in warehouses loads on grandstands and some scaffolds frequent value of snow load in some countries
short-term	load on most scaffolds characteristic value of snow load in some countries frequent value of wind load temperature actions
very short-term	imposed load from persons on roofs not intended for traffic characteristic wind load mooring forces (ships)
instantaneous	wind gusts impact earthquake

REQUIREMENTS FOR MATERIALS

4.0 General

Strength and stiffness properties shall be determined by tests for all actions to which the material may be subjected in the structure.

It must be shown that the form stability, environmental behaviour etc. are satisfactory for the purposes of construction and eventual end-use.

4.1 Solid structural timber

4.1.0 General

Structural timber, i.e. timber where the strength and stiffness are of importance, shall be graded in accordance with rules ensuring that the strength, stiffness and other properties of the timber are satisfactory.

The strength grading rules may be based on a visual assessment of the timber, on the non-destructive measurement of one or more properties or on a combination of the two methods.

Strength and stiffness parameters shall be determined by standardized short-term tests in accordance with ISO/TC 165: Timber structures - Timber in structural sizes - Determination of some physical and mechanical properties.

4.1.1 Standard strength classes

In this code the following standard strength classes are used for solid timber: SC15, SC19, SC24 and SC30.

A given visual grade can be referred to one of the standard strength classes if the characteristic bending strength, f_m (5-percentile), and the mean modulus of elasticity in bending, $E_{0,mean}$, are not less than the values given in table 4.1.1. For machine stress-rated timber it should further be shown that the characteristic tensile strength, $f_{t,0}$, is not less than given in the table.

Table 4.1.1 Standard strength classes. Minimum characteristic values in MPa and UN/ECE grades which comply

standard strength class	SC15	SC19	SC24	SC30
bending f_m	15	19	24	30
bending $E_{0,mean}$	6000	7200	8500	10000
tension $f_{t,0}$	6	9	16	20
UN/ECE-grade		S6	S8	S10

: It is emphasized that the introduction of standard strength classes does not prevent the introduction of other grades
: with for example higher values for $E_{0,mean}/f_m$ and $f_{t,0}/f_m$.
: European redwood/whitewood graded according to UN/ECE Recommended standard for stress grading of coniferous
: sawn timber (UN/ECE TIM/WP.3/AC3/B-Annex I) can be assumed to meet the demands of standard grades as given
: in table 4.1.1.
: Annex 4 contains a survey of which national grades can be assumed to satisfy the requirements of the different
: standard grades. (Under preparation).

The specification of structural timber by strength classes (sections 4.1.1, 4.2.1 and tables 5.1.0a and 5.1.0b) has been included as a background for discussion and as an illustration of a simpler method for the engineer than the one based on species and grades which is generally used.

4.2 Finger jointed structural timber

4.2.0 General

The manufacture of finger jointed structural timber should be subject to external control which does not require less of the production than stated in UN/ECE Recommended standard for finger jointing in structural coniferous sawn timber (UN/ECE TIM/WP.3/AC3/8-Annex II).

Strength and stiffness parameters shall be determined according to section 4.1.0 coupled with the rules in the above-mentioned UN/ECE Recommended standard.

4.2.1 Standard structural classes

Finger jointed structural timber can be referred to one of the standard strength classes stated in 4.1.1 if the characteristic values are not less than given in table 4.1.1.

: European redwood/whitewood finger jointed according to the UN/ECE Recommended standard category A can be
: assumed to satisfy the requirements of SC24, and category B the requirements of SC19.

4.3 Glued laminated timber

4.3.0 General

The manufacture of glued laminated timber (glulam) should be subject to external control which does not require less of the production than stated in (CIB-glulam standard under preparation).

In principle, strength and stiffness parameters shall be determined as given in section 4.1.0, combined with recognized methods for determining the strength and stiffness of the glulam from the properties of the laminae.

4.3.1 Standard glulam strength classes

In this code the following standard glulam strength classes are used: SCL30, SCL38, SCL47.

Glulam made from the same wood species in the entire cross-section may be referred to a standard glulam strength class if the characteristic bending strength, f_m, and its mean modulus of elasticity in bending, $E_{0,mean}$, are not less than the values given in table 4.3.1. In other cases it is furthermore required that the characteristic tensile strength is not less than given in the table.

Table 4.3.1 Standard glulam strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

		standard glulam strength class		
		SCL30	SCL38	SCL47
bending	f_m	30	37.5	47
bending	$E_{0,mean}$	10000	12000	12000
tension	$f_{t,0}$	20	25	30

: Glulam made from finger jointed timber corresponding to SC30 in the extreme eighths of the cross-section on
: either side, however at least two lamellas on either side, and to SC24 in the rest of the cross-section can be con-
: sidered to correspond to SCL38. A corresponding combination of SC24 and SC19 can be assumed to correspond
: to SCL 30.
: CIB-W18 will produce an annex to this code indicating how the requirements of these standard glulam strength
: classes may be met by existing national practices.

4.4 Wood-based sheet materials

The manufacture of plywood, particle board and fibre board for load-bearing structures shall be subject to an approved control arrangement.

The specification of glued laminated members by strength classes (sections 4.3.1 and table 5.2.0) has been included as a background for discussion and as an illustration of a simpler method for the engineer than the one based on species and grades which is generally used.

Testing must be carried out in accordance with the following standards:

For plywood: ISO/TC 165: Timber structures. Plywood. Determination of some physical and mechanical properties.

For particle board and fibre board:

4.5 Glue

Only glue giving joints of such strength that the integrity of the glue-line is maintained throughout the life of the structure, is allowed for timber structures.

4.6 Mechanical fasteners

Refer to chapter 6.

4.7 Steel parts

Nails, screws and other steel parts should as a minimum be protected against corrosion according to Table 4.7. The requirements for protection against corrosion may be relaxed for heavy steel parts where surface corrosion will not significantly reduce the load-carrying capacity.

Table 4.7 Minimum protection against corrosion

climate class	steel parts except nails and screws	nails, screws and bolts
0	none	none
1 2	galvanizing with a min. thickness of 20 μm	none
3	hot galvanizing with a minimum thickness of 70 μm	

: The consideration for the finish of the structures may call for stricter rules for corrosion protection, especially
: in climate class 2. Attention is drawn to the fact that certain woods, e.g. oak, and some treatments, e.g. fire re-
: tardant, may have a corroding effect on unprotected steel.

5. DESIGN OF BASIC MEMBERS

5.1 Solid timber members

5.1.0 Characteristic values

Characteristic values for the standard strength classes defined in section 4.1.1 are given in table 5.1.0 a. For the load-duration classes and climate classes defined in sections 2.2 and 2.3 the factors in table 5.1.0 b should be applied.

Table 5.1.0 a Characteristic values and mean elastic moduli, in MPa

		SC15	SC19	SC24	SC30
characteristic values (for strength calculations)					
bending	f_m	15	19	24	30
tension parallel to grain	$f_{t,0}$	6	9	16	20
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	14	18	23	28
compression perpendicular to grain	$f_{c,90}$	6	7	7	7
shear★	f_v	2.5	3	3	3
modulus of elasticity	E_0	4200	5400	6900	8000
mean values (for deformation calculations)					
modulus of elasticity, parallel	$E_{0,mean}$	6000	7200	8500	10000
modulus of elasticity, perpendicular	$E_{90,mean}$	250	300	350	400
shear modulus	G_{mean}	500	600	700	800

★ In rolling shear the shear strength may be put equal to $f_v/2$

Table 5.1.0 b Factors to the basic values

values for	strength calculations		deformation calculations		
climate classes	0, 1 and 2	3	0 and 1	2	3
permanent	0.6 (0.4)	0.5 (0.35)	0.7	0.6	0.4
normal	0.6 (0.4)	0.5 (0.35)	1	0.8	0.7
short-term	0.7 (0.6)	0.6 (0.5)	1	0.8	0.7
very short-term	0.85 (0.8)	0.7 (0.65)	1	0.8	0.7
instantaneous	1.0 (1.0)	0.85 (0.85)	-	-	-

Where a load case is composed of loads belonging to different load-duration classes the values corresponding to the shortest load may be used.

Values in parantheses apply to tension perpendicular to grain.

5.1.1 Straight beams, columns and tension members

5.1.1.0 General

This section applies to prismatic or cylindrical as well as slightly conical members, e.g. timber logs and poles.

The effective span of flexural members shall be taken as the distance between the centres of areas of bearing. With members extending further than is necessary over bearings the span may be measured between centres of bearings of a length which would be adequate according to this code; attention should be paid to the eccentricity of the load where advantage is taken of this provision.

See footnote on page 4.1.

The effective cross-section and geometrical properties of a structural member shall be calculated from the minimum cross-section acceptable for the given nominal size or from the actual cross-section. Nominal dimensions may be used in calculations when the actual dimensions at a moisture content of 0.20 are not less than the nominal dimensions reduced by 1 mm for dimensions of 100 mm or less; 2 mm for dimensions between 100 mm and 200 mm and 1 per cent for larger dimensions.

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary by nails and screws with a diameter of 6 mm or less.

5.1.1.1 Tension

The stresses shall satisfy the following conditions:

$$\sigma_t \leqslant k_{size,0} f_{t,0} \qquad (5.1.1.1\ a)$$

for tension parallel to the grain direction, and

$$\sigma_t \leqslant k_{size,90} f_{t,90} \qquad (5.1.1.1\ b)$$

for tension perpendicular to the grain, and

$$k_{size,90} = \begin{cases} 1 & \text{for} \quad V \leqslant 0.02\ m^3 \\ \left(\frac{0.02}{V}\right)^{0.2} & \text{for} \quad V \geqslant 0.02\ m^3 \end{cases} \qquad (5.1.1.1\ c)$$

for a volume of V uniformly load in tension perpendicular to the grain. Other examples of $k_{size,90}$ are given in section 5.2.2.

: Recommendations on the size factor $k_{size,0}$ will be produced.
: A recommendation on the method of calculation for tension strength at an angle to the grain is also in the course
: of preparation.

5.1.1.2 Compression without column effect

For compression at the angle θ to the grain the stresses should satisfy the following condition:

$$\sigma_c \leqslant f_{c,0} - (f_{c,0} - f_{c,90})\sin\alpha \qquad (5.1.1.2\ a)$$

cf. fig. 5.1.1.2 a.

: This condition only ensures that the compressive stress directly under the load is acceptable, but not that an ele-
: ment in compression can carry the load in question. Refer to section 5.1.1.9.

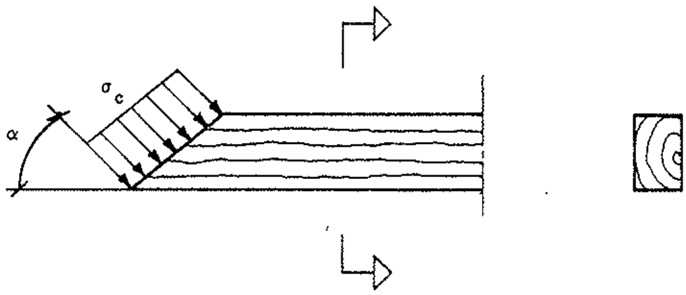


Fig. 5.1.1.2 a

For bearings on the side grain ($\alpha = 90°$) formula (5.1.1.2 a) may be replaced by

$$\sigma_c \leqslant k_{bearing} f_{c,90} \qquad (5.1.1.2\ b)$$

For bearings located at least 75 mm and 1.5 h from the end and 150 mm from other loads $k_{bearing}$ may be taken from fig. 5.1.1.2 b. In other cases $k_{bearing} = 1$.

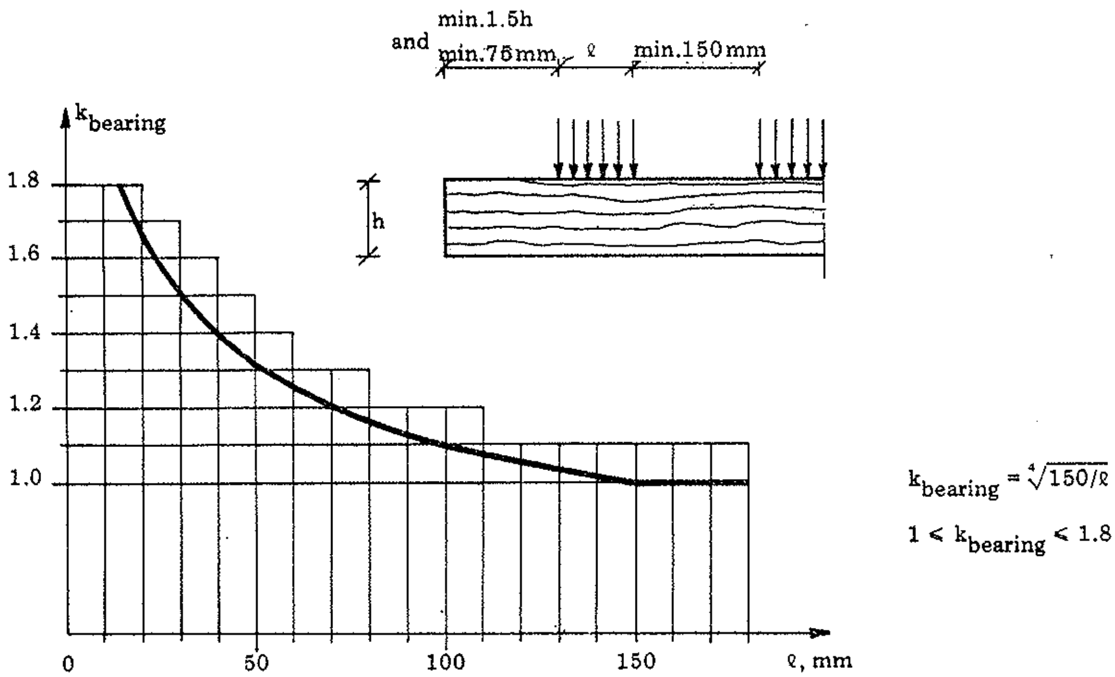


Fig. 5.1.1.2 b

Where the deformations resulting from compression perpendicular to the grain are significant to the function of a structure, an estimate of the deformations shall be made.

: The strain perpendicular to the grain can be estimated as $\sigma_c/(k_{bearing}E_{90,mean})$.

5.1.1.3 Bending

For pure bending the following condition shall be satisfied:

$$\sigma_m \leq k_{depth} k_{inst} f_m \qquad (5.1.1.3\ a)$$

where k_{depth} is a factor (≤ 1) taking into account the reduced strength of deep sections:

$$k_{depth} = \begin{cases} 1 & \text{for } h \leq 200 \text{ mm} \\ \left(\frac{200}{h}\right)^{1/9} & \text{for } h \geq 200 \text{ mm} \end{cases} \qquad (5.1.1.3\ b)$$

where k_{inst} is a factor (≤ 1) taking into account the reduced strength due to failure by lateral instability (lateral buckling). k_{inst} is determined so that the total bending stresses, taking into account the influence from initial curvature, eccentricities and the deformations developed, do not exceed f_m.

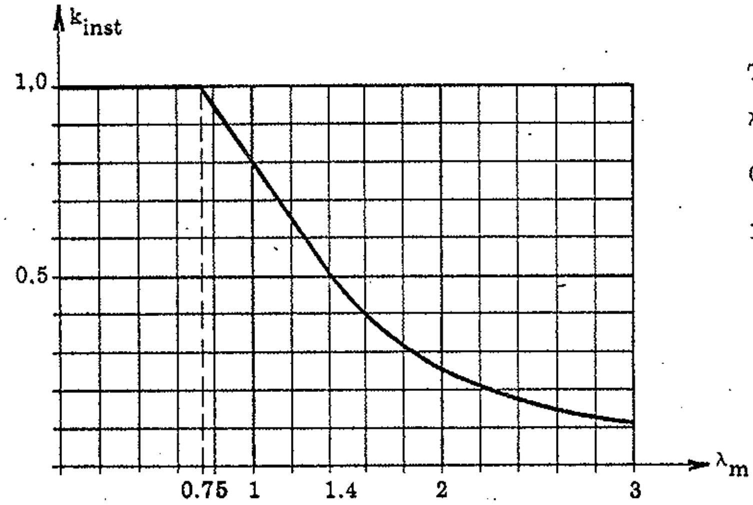
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

$$\lambda_m = \sqrt{f_m/\sigma_{m,crit}} \leq 0.75 \qquad (5.1.1.3\ c)$$

In (5.1.1.3 c) λ_m is the slenderness ratio for bending, and $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability.

k_{inst} may also be put equal to 1 for beams where lateral displacement of the compression side is prevented throughout its length and where torsion is prevented at the supports.

: k_{inst} may be determined from fig. 5.1.1.3 if the lateral deviation from straightness measured at midspan is less than $\ell/200$.



The curve corresponds to

$\lambda_m < 0.75$; $k_{inst} = 1$

$0.75 < \lambda_m < 1.4$; $k_{inst} = 1.56 - 0.75\,\lambda_m$

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m$

Fig. 5.1.1.3

For a beam with rectangular cross-section k_{inst} may be determined from fig. 5.1.1.3 dependent on the slenderness ratio λ_m determined from

$$\lambda_m = \sqrt{\frac{\ell_c h}{b^2}\frac{f_m}{E_0}\sqrt{\frac{E_{0,mean}}{G_{mean}}}} \qquad (5.1.1.3\ d)$$

where ℓ_e is the effective length of the beam. For a number of structures and load combinations ℓ_e is given in table 5.1.1.3 in relation to the free beam length ℓ.

The free length is determined as follows:

a) When lateral support to prevent rotation is provided and no other support to prevent rotation or lateral displacement is provided throughout the length of a beam, the unsupported length shall be the distance between such points of bearing, or the length of a cantilever.

b) When beams are provided with lateral support to prevent both rotation and lateral displacement at intermediate points as well as at the ends, the unsupported length may be the distance between such points of intermediate lateral support. If lateral displacement is not prevented at points of intermediate support, the unsupported length shall be as defined in a).

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

Type of beam and load	ℓ_e/ℓ
Simply supported, uniform load or equal end moment	0.35
Simply supported, concentrated load at centre	0.30
Cantilever, uniform load	0.20
Cantilever, concentrated end load	0.30
Cantilever, end moment	0.35

The values apply to loads acting in the gravity axis. For downwards acting loads ℓ_c is increased by 0.75 h for loads on the top side and reduced by 0.25 h for loads on the bottom side.

5.1.1.4 Shear

The shear stresses shall satisfy the following condition

$$\tau \leqslant f_v \qquad (5.1.1.4\ a)$$

For beams with bearing in the bottom side and load on the top, loads placed nearer than the beam depth from the support may be disregarded in calculation of the shear force. Loads placed more than the beam depth but less than twice the beam depth from the support may be reduced to one-half their true value in calculation of the shear force.

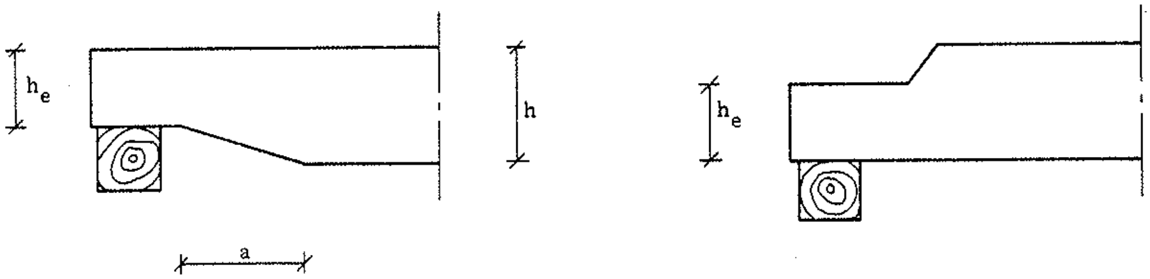


Fig. 5.1.1.4

For beams notched at the ends, see fig. 5.1.1.4, the shear stresses should be calculated on the effective depth h_e, and for notches in the bottom the condition (5.1.1.4 a) should, for $a < 3(h - h_e)$, be replaced by

$$\tau \leqslant \frac{h}{6h - 5h_e} f_v \qquad (5.1.1.4\ b)$$

Notches with $h_e < 0.5\ h$ are not allowed.

: Notches or abrupt changes of section that will produce tension perpendicular to grain stresses at the notch should
: be avoided. Stress concentrations produced are likely to cause splitting at the notch at low tension values and no
: satisfactory means are available for determining this tension stress. A gradual change of section, perhaps by well
: rounded corners, will reduce these stress concentrations.
: Further recommendations on notching are being considered.

5.1.1.5 Torsion

The torsional stresses, τ_{tor}, calculated according to the theory of elasticity shall satisfy the following condition

$$\tau_{tor} \leqslant f_v \qquad (5.1.1.5)$$

5.1.1.6 Combined stresses

General

: Recommendations on combined stresses are being prepared.

Tension and bending

Only the case with tension in the grain direction is considered.

The stresses should satisfy the following condition

$$\frac{\sigma_t}{f_{t,0}} + \frac{\sigma_m}{f_m} \leqslant 1 \qquad (5.1.1.6\ a)$$

and in the parts of the cross-section, if any, where $\sigma_t + \sigma_m \leqslant 0$, furthermore

$$|\sigma_m| - \sigma_t \leqslant f_m \qquad (5.1.1.6\ b)$$

Compression and bending without column effect
Only the case with compression in the grain direction is considered.
The stresses in the parts of the cross-section, where $\sigma_m + \sigma_c \leqslant 0$ should satisfy the following condition

$$\frac{|\sigma_c|}{f_{c,0}} + \frac{|\sigma_m|}{f_m} \leqslant 1 \qquad (5.1.1.6\ c)$$

and in the parts of the cross-section, if any, where $\sigma_c + \sigma_m \geqslant 0$

$$\sigma_m + \sigma_c \leqslant f_m \qquad (5.1.1.6\ d)$$

: The condition only ensures that the stresses directly under the load are acceptable, but not that e.g. a laterally loaded
: column can carry the load in question. Reference is made to section 5.1.1.7.

Torsion and shear
The stress τ from shear and τ_{tor} from torsion calculated as stated in section 5.1.1.4 and section 5.1.1.5 shall satisfy the following condition

$$\frac{\tau^2}{f_v} + \tau_{tor} \leqslant f_v \qquad (5.1.1.6\ e)$$

5.1.1.7 Compression and bending with column effect (columns)
For columns it must be verified that the conditions in section 5.1.1.6 for compression and bending are satisfied, when apart from bending stresses from lateral load, if any, the bending stresses from initial curvature and stresses caused by the deflections are taken into consideration.

These conditions can be assumed satisfied if the stresses meet the following demand:

$$\frac{|\sigma_c|}{k_{col} f_{c,0}} + \frac{|\sigma_m|}{f_m} \frac{1}{1 - \frac{k_{col}}{k_E} \frac{|\sigma_c|}{f_{c,0}}} \leqslant 1 \qquad (5.1.1.7\ a)$$

σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_{col} and k_E are factors depending on the slenderness ratio λ, the material parameters and the initial curvature. The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

$$e = \eta r_{core} \lambda \qquad (5.1.1.7\ b)$$

where r_{core} is the core radius.

$$k_E = \frac{\sigma_E}{f_{c,0}} = \frac{\pi^2 E_0}{f_{c,0} \lambda^2} \qquad (5.1.1.7\ c)$$

where E_0 is the characteristic value of modulus of elasticity

$$k_{col} = 0.5[(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E) - \sqrt{(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E)^2 - 4k_E}\,] \qquad (5.1.1.7\ d)$$

σ_E is the Euler stress.

: Fig. 5.1.1.7 gives k_{col} and k_{col}/k_E for columns with $e < \ell_c/300$, i.e. $\eta \sim 0.005$, $f_{c,0}/f_m = 0.96$ is assumed.

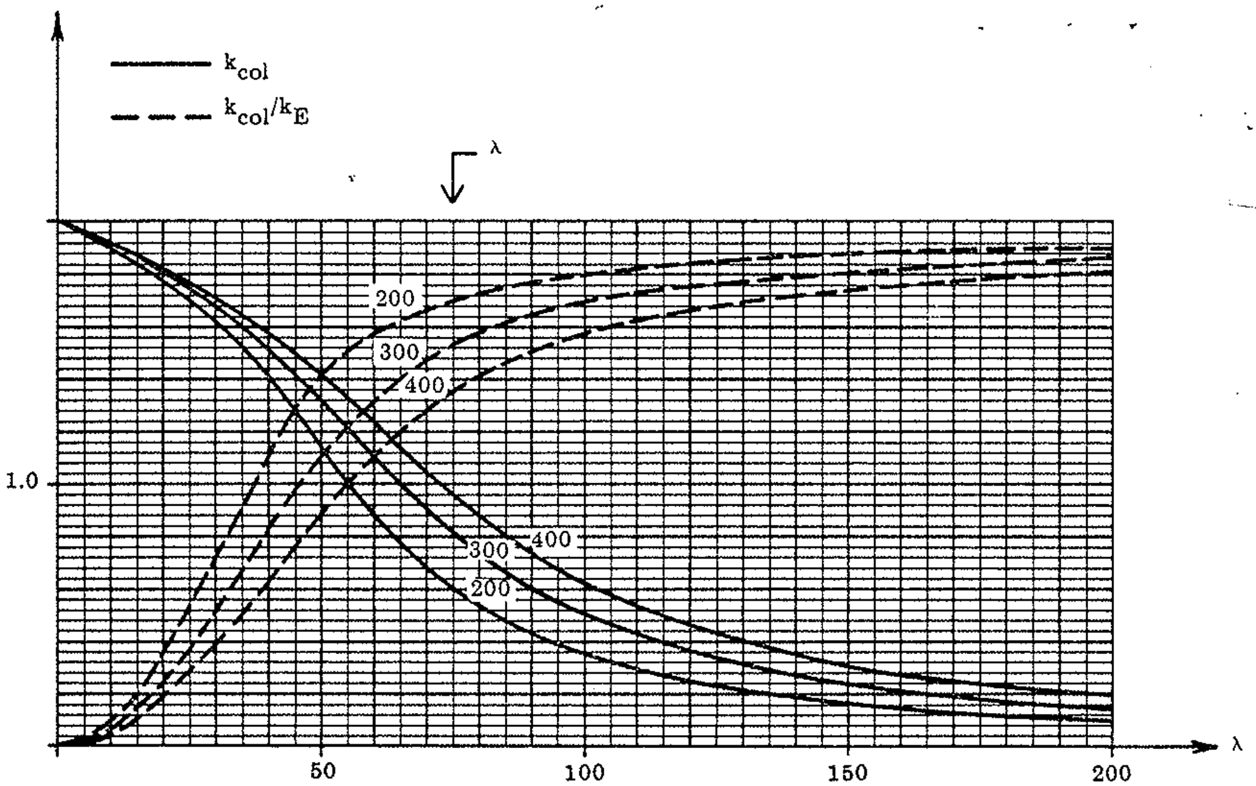


Fig. 5.1.1.7

The condition (5.1.1.7 a) is on the safe side in cases where the tension side is decisive, cf. (5.1.1.6 d).

For the purpose of calculating the slenderness ratio of compression members, the values of the length ℓ_c should be calculated for the worst conditions of loading to which a compression member is subjected, paying regard to the induced moments at the ends or along the length of the compression member and to slip in the connections. The length should be judged to be the distance between two adjacent points of zero bending moment, these being two points between which the deflected member would be in single curvature.

For straight members with mechanical fasteners the values of ℓ_c can be taken from table 5.1.1.7. The actual length of the member is denoted ℓ.

Table 5.1.1.7 Relative effective length of compression members

Condition of end restraint	ℓ_c/ℓ
Restrained at both ends in position and direction	0.7
Restrained at both ends in position and one end in direction	0.85
Restrained at both ends in position but not in direction	1.00
Restrained at one end in position and direction and at the other end partially restrained in direction but not in position	1.50
Restrained at one end in position and direction, but not restrained in either position or direction at the other end	2.00

The slenderness ratio should not exceed 170, or for secondary members, 200.

5.1.2 Cambered beams

Relevant parts of section 5.2.2 may be applied.

5.2 Glued laminated members

5.2.0 Characteristic strength and stiffness values

Characteristic values for the standard glulam strength classes defined in section 4.2.1 are given in table 5.2.0. For the load duration classes and climate classes defined in sections 2.2 and 2.3 the factors in table 5.1.0 b should be applied.

Table 5.2.0 Characteristic values and mean elastic moduli, in MPa

		SCL30	SCL38	SCL47
Characteristic values (for strength calculations)				
bending	f_m	30	38	47
tension parallel to grain	$f_{t,0}$	20	25	30
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	28	36	45
compression perpendicular to grain	$f_{c,90}$	7	7	7
shear★	f_v	3	3	3
modulus of elasticity	E_0	8000	9600	9600
Mean values (for deformation calculation)				
modulus of elasticity parallel to grain	$E_{0,mean}$	10000	12000	12000
modulus of elasticity perpendicular to grain	$E_{90,mean}$	400	500	500
shear modulus	G_{mean}	800	1000	1000

★ In rolling shear the shear strength may be put equal to $f_v/2$

5.2.1 Straight beams and columns

Section 5.1.1 for solid timber applies except that

$$k_{depth} = \begin{cases} 1 & \text{for} \quad h \leqslant 300 \text{ mm} \\ \left(\frac{300}{h}\right)^{1/9} & \text{for} \quad h > 300 \text{ mm} \end{cases} \tag{5.2.1.3}$$

cf. formulas (5.1.1.3 a) and (5.1.1.3 b).

5.2.2 Cambered beams

This section applies to double tapered curved beams with rectangular cross-section (fig. 5.2.2 a) and double tapered beams with flat soffit and rectangular cross-section (fig. 5.2.2 b). In the latter case $h/r_m = 0$, cf. below.

The influence of the cross-sectional variation shall be taken into account. Especially it shall be ensured that the tensile stresses satisfy the condition 5.1.1.1 b, i.e.

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.2.2 a}$$

with

$$k_{size,90} = \begin{cases} \frac{0.5}{V^{0.2}} & \text{for} \quad \text{uniformly distributed load} \\ \frac{0.35}{V^{0.2}} & \text{for} \quad \text{other loading} \end{cases} \tag{5.2.2 b}$$

See footnote on page 4.2.

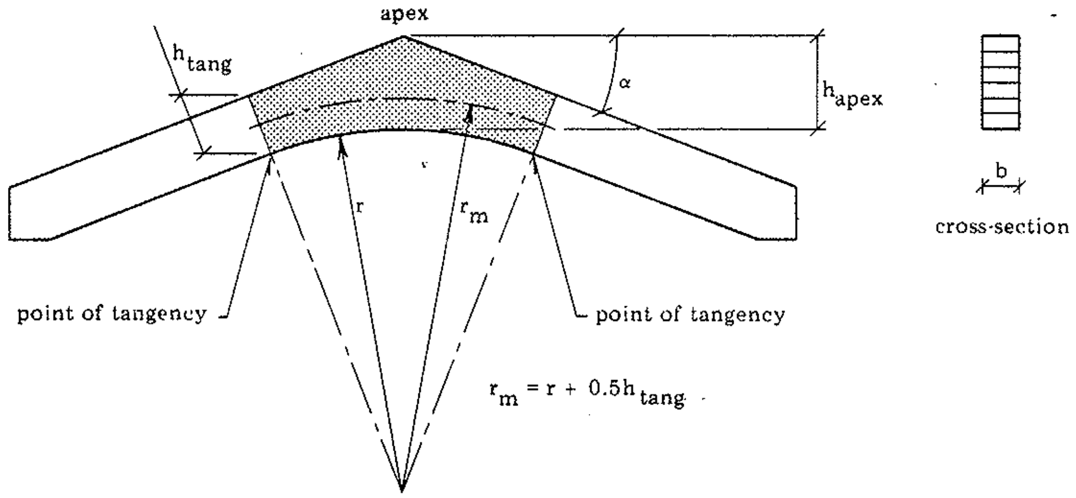


Fig. 5.2.2 a Double tapered curved beam

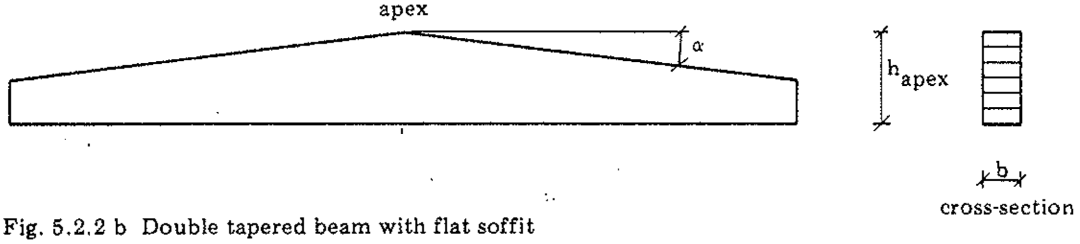


Fig. 5.2.2 b Double tapered beam with flat soffit

For double tapered curved beams V is the beam volume between the points of tangency (corresponding to the shaded area in fig. 5.2.2 a). V shall, however, not be taken as less than $V = 0.6\ bh^2_{apex}$.

For double tapered beams with flat soffit $V = 0.6\ bh^2_{apex}$.

: The following method may be used for calculating the maximum stresses in beams with rectangular cross-section.

: The radial tensile stresses perpendicular to the grain are at a maximum near the mid-depth of the apex, and the maximum value can be calculated as

$$\sigma_t = k_t \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2\ c)$$

: where M_{apex} is the bending moment at the apex-section and k_t is given in fig. 5.2.2 c for $E_{0,mean}/E_{90,mean} = 15$ and $E_{0,mean}/E_{90,mean} = 30$.

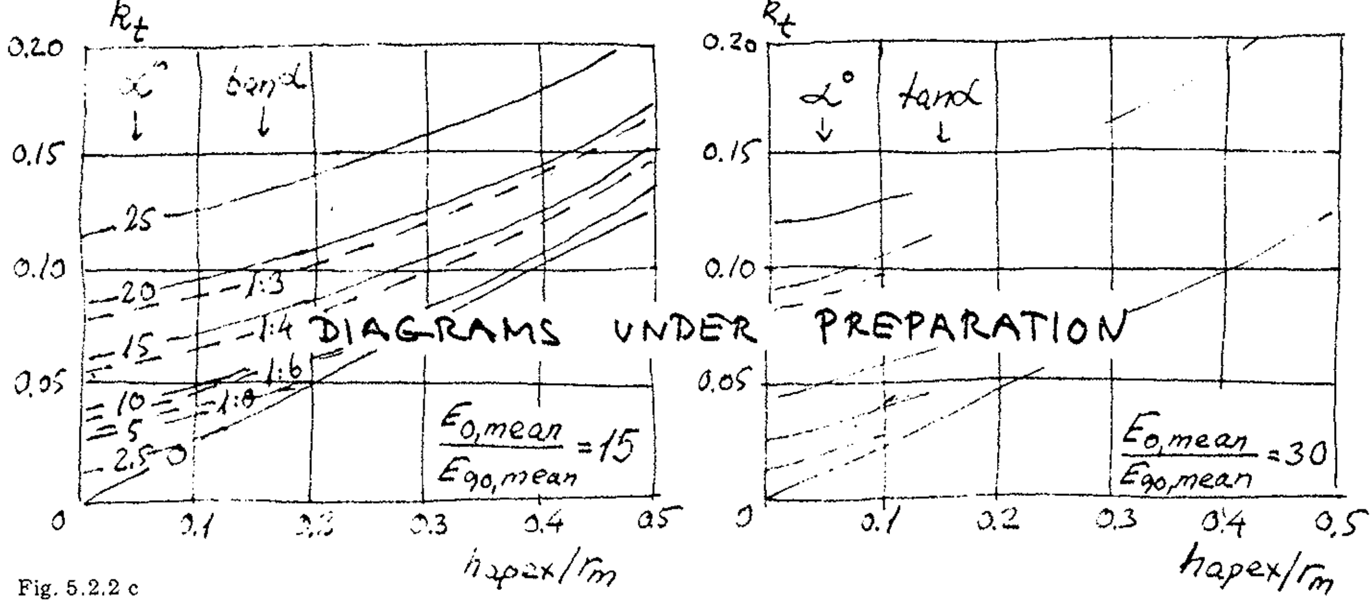


Fig. 5.2.2 c

: The maximum bending stress in the apex cross-section occurs at the lower face and can be calculated as

$$\sigma_m = k_m \frac{6M_{apex}}{bh^2_{apex}} \tag{5.2.2 d}$$

: where k_m is given in fig. 5.2.2 d for $E_{0,mean}/E_{90,mean} = 15$ and $E_{0,mean}/E_{90,mean} = 30$.

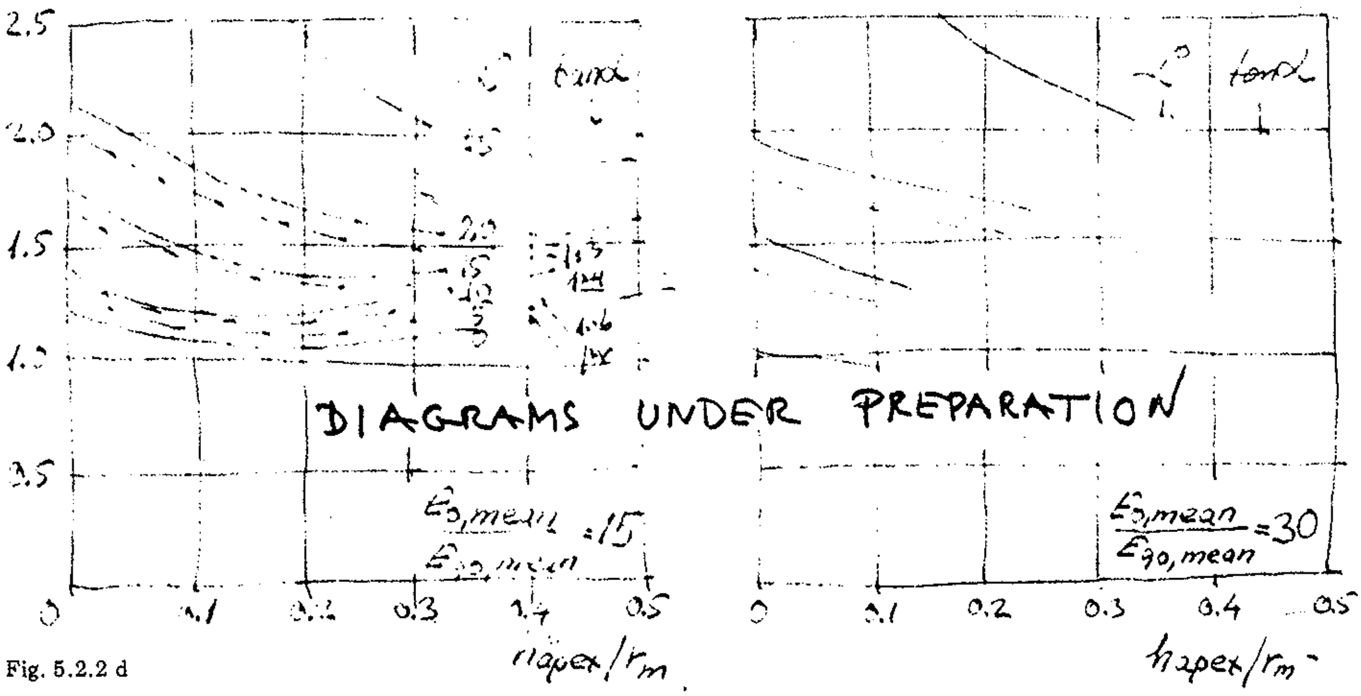


Fig. 5.2.2 d

: The bending stresses between the supports and the points of tangency are calculated as usual.

In deflection calculations contributions from shear force deformations shall be taken into account.

5.2.3 Curved beams

This section applies to curves beams with constant, rectangular cross-section, see fig. 5.2.3 a.

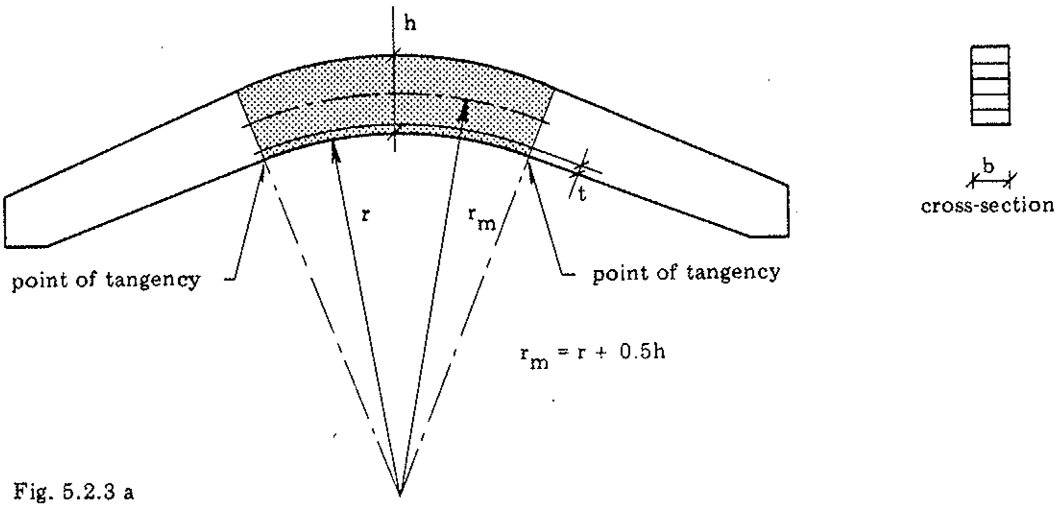


Fig. 5.2.3 a

Reduction of strength

The ratio between the radius of curvature, r, and the laminae thickness, t, should be greater than 125. For r/t < 240 the reduction of the strength in bending, tension and compression parallel to the grain due to the bending of the laminae should be taken into account.

: This can be done by multiplying f_m, $f_{c,0}$ and $f_{t,0}$ by the factor k_{curv}, where

$$k_{curv} = 0.76 + 0.001\frac{r}{t} \tag{5.2.3 a}$$

Distribution of bending stresses

In heavily curved beams (i.e. the ratio between minimum mean-radius of curvature, r_m, and depth, h, less than 15) the influence of the curvature on the distribution of axial stresses from bending moments shall be taken into consideration.

: The bending stresses in the innermost fibre can be calculated as

$$\sigma_{mi} = k_i \frac{6M}{bh^2} \qquad (5.2.3\ b)$$

: while the stresses in the outermost fibre can be calculated by the usual expression

$$\sigma_{mo} = \frac{6M}{bh^2} \qquad (5.2.3\ c)$$

: The modification factor k_i is given in fig. 5.2.3 b.

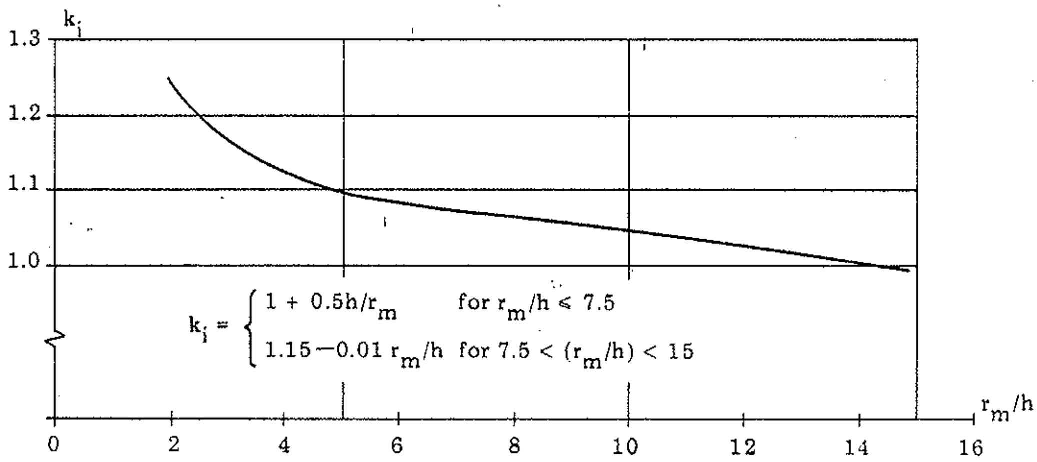


Fig. 5.2.3 b

When the bending moments tend to reduce curvature (increase the radius) the tensile stresses perpendicular to the grain shall satisfy the condition

$$\sigma_t \leqslant k_{size,90} f_{t,90} \qquad (5.2.3\ d)$$

where

$$k_{size,90} = \begin{cases} \dfrac{0.4}{V^{0.2}} & \text{for uniformly distributed load} \\ \dfrac{0.3}{V^{0.2}} & \text{for other loading} \end{cases} \qquad (5.2.3\ e)$$

V is the volume of the curved part of the beam (corresponding to the shaded area in fig. 5.2.3 a).

: The tensile stresses perpendicular to the grain in the curved part may be calculated as

$$\sigma_t = \frac{1.5\,M}{r_m bh} \qquad (5.2.3\ f)$$

6. MECHANICAL FASTENERS

6.0 General

6.0.1 General requirements

When the joint is non-symmetric or where the load is eccentric consideration should be given to these factors by the determination of the load-carrying capacities of the fasteners as well as by the design of the members.

In a joint where several identical fasteners are used symmetrically it can be assumed that each fastener is loaded equally, except where a large number of fasteners are used. However, the load-carrying capacity of a multiple-fastener joint will be less than the sum of the individual fastener capacities.

The entire load on a joint should normally be carried by one type of fastener. In some cases, however, two types of fastener may be used provided they have similar stiffness characteristics.

: Glud and mechanical fasteners have very different stiffness properties and thus they can never be assumed to act
: in unison.

The arrangement of timber joints and the size of the fasteners, mutual distances and distance to end or edge of the timber should be chosen so that the expected strengths can be obtained without splitting or damaging the timber.

6.0.2 Determination of characteristic load-carrying capacities

The characteristic load-carrying capacities are determined from tests carried out in conformity with ISO/TC 165: Timber structures - Joints - Determination of strength and deformation characteristics of mechanical fasteners.

For a number of fasteners characteristic load-carrying capacities are given in section 6.1 - 6.4.

Where nothing else is stated the load-carrying capacities for the load duration classes defined in sections 2.2 and 2.3 are found by multiplication by the factors given in table 6.0.2.

Table 6.0.2 Factors for load-duration and climate

materials to be jointed	load-duration class	climate class 0 and 1	2	3
wood,	permanent and normal	0.6	0.6	0.5
plywood or	short-term	0.7	0.7	0.6
steel to wood	very short-term	0.85	0.85	0.7
	instantaneous	1.0	1.0	0.85
particle board	permanent and normal	0.4	0.3	-
or fibre board	short-term	0.6	0.45	-
to wood	very short-term	0.8	0.6	-
	instantaneous	1.0	0.75	-

The stated load-carrying capacities apply to static loads and may be less for some cases of fluctuating or dynamic loads especially when the stresses alternate between compression and tension.

: Attension is drawn to the fact that certain fasteners, e.g. nails, bolts without connectors and bolts with split ring
: or shear-plate connectors, have only inferior strength and will reveal great slip when exposed to heavy stresses with
: frequently alternating directions or vibrating load.

6.1 Joints with mechanical fasteners

6.1.1 Nails and staples

6.1.1.1 Laterally loaded nails

Timber-to-timber joints

In joints where the timber dimensions, the mutual distances between nails, and the distances between nails and end or edge are sufficient to prevent splitting, the characteristic load-carrying capacity per shear plane can be determined, in N, by:

$$F = k_{nail} d^{1.7} \qquad (6.1.1.1\ a)$$

where d (in mm) is the diameter for round nails and the side length for square nails. The factor, k_{nail}, which is dependent on among other things nail type and yield moment of the nails, wood species and grade (especially the density) must be determined by tests.

A joint should contain at least 2 nails. If there are only one or two nails the values according to formula (6.1.1.1 a) are multiplied by 0.5.

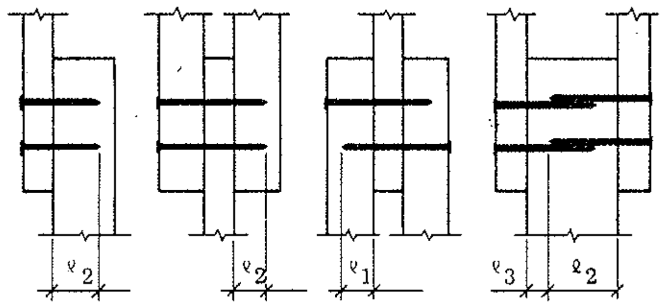
Nails in end grain should normally not be considered capable of transmitting force.

: For round nails with a characteristic tensile strength of at least 40(20 − d)MPa the following equation can be used
: for Nordic softwood and other woods with corresponding properties

$$k_{nail} = 200\sqrt{\rho} \qquad (6.1.1.1\ b)$$

: where ρ is the specific gravity defined in section 2.1.

: For structural timber at least corresponding to SC19 $\rho = 0.36$ and thus $k_{nail} = 120$ can be assumed.



: Fig. 6.1.1.1 a

: The values assume that the nails are driven in perpendicular to the grain, that the thinnest member has a thickness
: of not less than 7d, and that the penetration depths (including the point) satisfy the following conditions (cf. fig.
: 6.1.1.1 a):

Nails in double shear	
(driven in alternating from either side)	$\ell_1 \geq 8d$
Other cases	
smooth nails	$\ell_2 \geq 12d$
annular and spirally grooved nails	$\ell_2 \geq 8d$

: For smaller thicknesses and lengths the load-carrying capacity is reduced in proportion to the length. For smooth nails
: it is required that the nail length in any timber member is at least 5d and that the penetration length ℓ_2, is at least
: 8d. For annular grooved nails the penetration length should at least be 4d.

: If ℓ_3 is greater than 3d (cf. fig. 6.1.1.1 a) nails from the two sides are allowed to overlap in the middle member.

: Minimum distances are given in fig. 6.1.1.1 b. The nails should be staggered in the best possible way, for example
: as shown in fig. 6.1.1.1 b, one nail thickness in relation to the system lines.

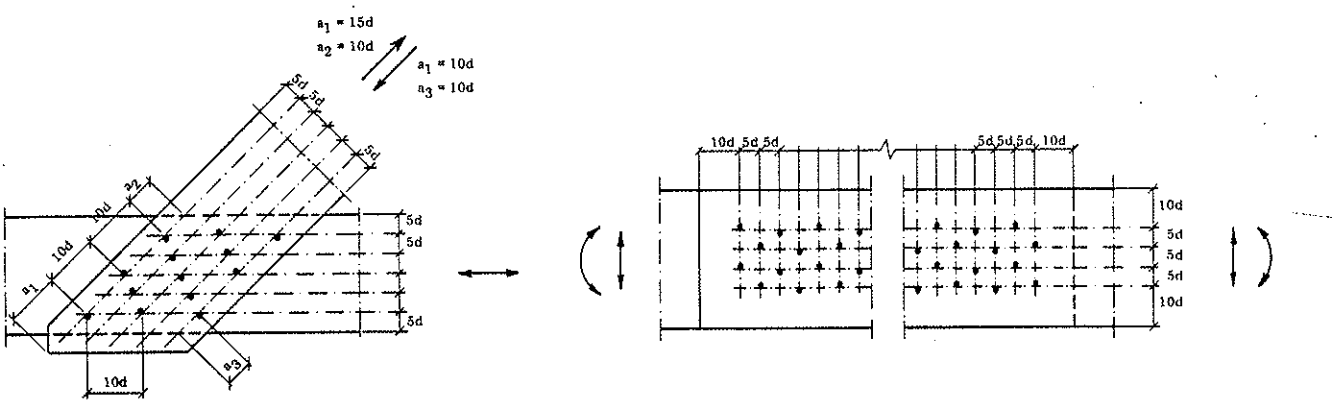


Figure 6.1.1.1 b

Steel-to-timber joints
The recommendations for timber apply, but the load-carrying capacities may be multiplied by 1.25.

Board materials-to-timber joints
What is stated for timber applies, but a board with thickness t can be assumed to correspond to a softwood timber member of quality SC19 with the thickness -

- 2.5 t for plywood of birch, beech, and similar hardwood
- 1.5 t for plywood of fir, pine, and similar softwood
- 2.0 t for plywood with plies of alternating hardwood and fir or pine (combi-plywood)
- 1.0 t for structural particle board and semihard structural fibre board
- 3.0 t for hard or oil-tempered structural fibre board.

This assumes the use of ordinary nails with heads which have a diameter of about 2.5 d.

For smaller heads the load-carrying capacity is reduced. For pins and oval headed nails, for example, the load-carrying capacity in particle boards and fibre boards is reduced by half.

6.1.1.2 Axially loaded nails
The characteristic withdrawal resistance of nails in N *for all climate classes* for nailing perpendicular to the grain and for slant nailing as shown in fig. 6.1.1.2 a - b is calculated as the smallest of the values according to formula (6.1.1.2 a) corresponding to withdrawal of the nail in the member receiving the point, and formula (6.1.1.2 b - c) corresponding to the head being pulled through.

The length of the point is denoted ℓ_p.

$$F = \min \begin{cases} f_{axial} d(\ell - \ell_p) & (6.1.1.2\ a) \\ f_{axial} dh + f_{head} d^2 \quad \text{for smooth nails} & (6.1.1.2\ b) \\ f_{head} d^2 \quad \text{for annular and spirally grooved nails} & (6.1.1.2\ c) \end{cases}$$

: The parameters f_{axial} and f_{head}, depend, among other things, on type of nail, timber species and grade (especial-
: ly) density, must be determined by tests.

For spirally or annular grooved nails only the grooved part is considered capable of transmitting force.

By slant nailing ℓ and h are measured as shown in fig. 6.1.1.2 b and the load-carrying capacity is calculated as if the force were parallel to the nail. Unless otherwise ensured, e.g. by pre-boring, $\alpha = 45°$ is assumed.

Nails in end grain should not be considered capable of transmitting force.

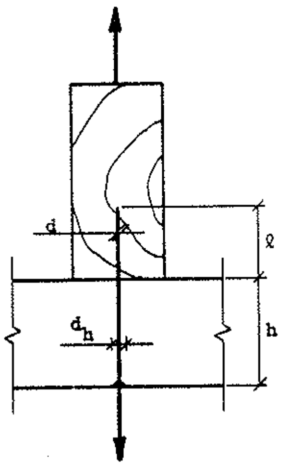


Fig. 6.1.1.2 a

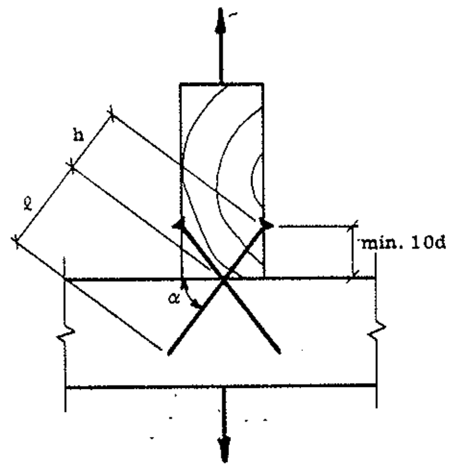


Fig. 6.1.1.2 b

: The distances for laterally loaded nails should be complied with and the distance to loaded edge by slant nailing should be at least 10d, cf. fig. 6.1.1.2 b.

: Normally the values of f given in table 6.1.1.2 can be assumed. For structural timber at least corresponding to SC19 a characteristic density of $\rho \sim 0.36$ is assumed.

: Table 6.1.1.2

	f_{axial} in MPa		f_{head} in MPa	
	general	SC19	general	SC19
ordinary nails, round	$12.5\ \rho^2$	1.6	$45.0\ \rho^2$	58
ordinary nails, square	$15\ \rho^2$	1.9		
spirally grooved nails 1)	...	to be determined by tests		...
annular grooved nails 1)	...	to be determined by tests		...

6.1.1.3 Staples

: Recommendations for staples will be provided.

6.1.2 Bolts and dowels

The characteristic load-carrying capacity in N per shear plane for bolts and dowels with a yield strength f_y of at least 240 MPa (corresponding to ISO grade 4.6) is the smallest value found by the formulas (6.1.2 a) - (6.1.2 e).

$$F = \min \begin{cases} 18\rho(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\ a) \\ 35\rho\, k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ b) \\ 70\rho\, k_1 t_1 d & (6.1.2\ c) \\ 42\sqrt{\rho}\ d^2 + 12\rho\, k_1 t_1 d & (6.1.2\ d) \\ 75d^2\ \sqrt{\rho}\ \sqrt{(k_1 + k_2)/2}\ \sqrt{f_y/240} & (6.1.2\ e) \end{cases}$$

where

- t is timber thickness in mm
- d is the diameter in mm
- k is a factor, obtained from table 6.1.2, taking into consideration the influence of the angle between force and grain direction.

In three-member joints subscript 1 denotes side member and subscript 2 denotes middle member. In two-member joints the subscripts are chosen so that $k_1 h_1 \leqslant k_2 h_2$.

Table 6.1.2 Factor $k(k_1, k_2)$ in calculation of the load-carrying capacity of bolts, dowels and screws

Angle between force and grain direction	Diameter d(mm)		
	6	12	24
0°	1	1	1
30°	1	0.88	0.82
45°	1	0.76	0.70
60°	1	0.70	0.58
90°	1	0.64	0.52

: For structural timber at least corresponding to SC19 (i.e. ρ = 0.36) the following is found by inserting into (6.1.2 a) - (6.1.2 e):

$$F = \min \begin{cases} 6.5(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\ f) \\ 12.5 k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ g) \\ 25 k_1 t_1 d & (6.1.2\ h) \\ 25d^2 + 4.5 k_1 t_1 d & (6.1.2\ i) \\ 45d^2 \sqrt{(k_1 + k_2)/2}\ \sqrt{f_y/240} & (6.1.2\ j) \end{cases}$$

Minimum distances are given in fig. 6.1.2. (The distance from bolt or dowel to loaded end can be reduced to a minimum of 4d provided the load is reduced correspondingly). If the load-carrying capacity is assumed to be higher than corresponding to formula (6.1.2 e) with f_y = 240 MPa the distance in the grain direction should be increased correspondingly.

The stated distances to loaded edge are not always adequate to prevent splitting when the bolts are fully loaded. When the force acts at an angle to the grain it should therefore be shown that the force can be sustained without splitting.

: Where a detailed analysis is not carried out, this can be verified by showing that $V < 2f_v b_e t/3$, where V is the shear force produced by the bolt or dowel, t is the thickness in mm of the member, and b_e is the distance in mm from loaded edge to the farthest point of the bolt.

In multiple shear the load-carrying capacity can be found by considering the structure as a number of three-member joints.

Where the side members are steel plates the loads calculated from the above formulas may be used with t_1 equal to t_2 equal to the thickness of the wood member, and multiplied by 1.25.

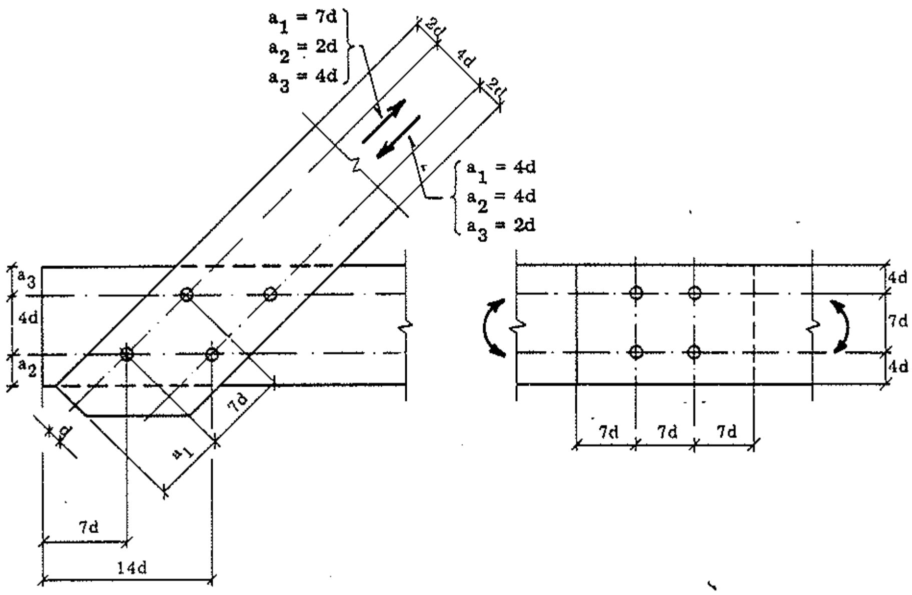


Fig. 6.1.2

Where the middle member is a steel plate formula (6.1.2 b) is omitted and the values of the formulas (6.1.2 d) and (6.1.2 e) can be multiplied by 1.4.

6.1.3 Wood and lag screws

6.1.3.1 Laterally loaded screws

Timber to timber

The characteristic load-carrying capacity in N of screws with a yield strength f_y of at least 240 MPa screwed at right angles to the grain is the smallest of the values from the formulas (6.1.3.1 a) - (6.1.3.1 c)

$$F = \min \begin{cases} 70\rho k_1 td & (6.1.3.1\ a) \\ 42\sqrt{\rho}\, d^2 + 12\rho k_1 hd & (6.1.3.1\ b) \\ 75d^2 \sqrt{\rho}\sqrt{(k_1 + k_2)/2}\sqrt{f_y/240} & (6.1.3.1\ c) \end{cases}$$

where

- t is the thickness in mm of the timber,
- d is the diameter in mm of the screw, measured on the smooth shank,
- k_1, k_2 are factors, obtained from table 6.1.2, taking into consideration the influence of the angle between force and grain direction in the member under the screw head (k_1) and the member receiving the point (k_2).

: For structural timber at least corresponding to SC19

$$F = \min \begin{cases} 25k_1 td & (6.1.3.1\ d) \\ 25d^2 + 4.5k_1 td & (6.1.3.1\ e) \\ 45d^2 \sqrt{(k_1 + k_2)/2}\sqrt{f_y/240} & (6.1.3.1\ f) \end{cases}$$

: is found by inserting $\rho = 0.36$ into (6.1.3.1 a) - (6.1.3.1 c).

In these expressions it is assumed that:

- screws are screwed into pre-bored holes, see section 8.3.
- the minimum distances between screws and between screws and end or edge do not exceed those given for bolts (refer to section 6.1.2),

- the length of the smooth shank is greater than or equal to the thickness of the member under the screw head,
- the penetration depth of the screw, i.e. the length in the member receiving the point, is at least 8d,
- the recommendations of section 8.3 are complied with for lag screw holes.

If the penetration depth is less than 8d the load-carrying capacity is reduced proportionally. However, the penetration depth should be at least 5d.

Screws in end grain should not normally be considered capable of transmitting force.

Steel to timber
The characteristic load-carrying capacity in N is (cf. formula (6.1.3.1 c))

$$1.4 \cdot 75d^2 \sqrt{\rho}\,\sqrt{(1+k_2)/2}\,\sqrt{f_y/240} \qquad (6.1.3.1\ g)$$

and furthermore, what is stated for timber-to-timber joints applies.

6.1.3.2 Withdrawal loads of screws
The characteristic withdrawal strength in N of screws screwed at right angles to the grain is

$$F = (f_0 + fd)(\ell_t - d) \qquad (6.1.3.2\ a)$$

where

d	is the diameter in mm measured on the smooth shank,
ℓ_t	is the threaded length in mm in the member receiving the screw,
f_0 and f	are parameters dependent on among other things the shape of the screw and timber species and grade.

: For screws according to ISO 0000 the following can be assumed for structural timber at least corresponding to SC19
:
: $F = (30 + 12.5)(\ell_t - d)$ (6.1.3.2 b)

It is assumed that the minimum distances and penetration lengths given for laterally loaded screws are complied with and that the strength of the screw is adequate.

6.1.4 Connectors

The characteristic load-carrying capacity $F_{bolt+conn}$ of a fastener comprising bolt (or screw) and connector may be determined as stated in section 6.0.2. The contribution from the bolt (or screw) may be calculated as stated in 6.1. The characteristic load for the connector F_{conn} is then determined from:

$$F_{conn} = F_{bolt+conn} - F_{bolt} \qquad (6.1.4)$$

If a connector is to be used together with several bolt diameters the investigation should comprise at least maximum and minimum bolt diameter.

: For characteristic load-carrying capacities of different types of connector, type approvals, etc. are referred to.

The rules given in section 6.1.2 for bolts should be complied with and the minimum distances between connectors should be sufficient to prevent splitting or shearing of the timber under the maximum permissible load.

When a load is applied at an angle to the grain direction it should be shown that the load can be sustained without causing splitting or shearing of the timber.

: Where a detailed analysis is not carried out this can be proved by showing that $V \leqslant 2f_v t b_e/3$, where t is the thick-
: ness of the member and b_e is the distance from loaded edge to farthest edge of the connectors, cf. fig. 6.1.4.

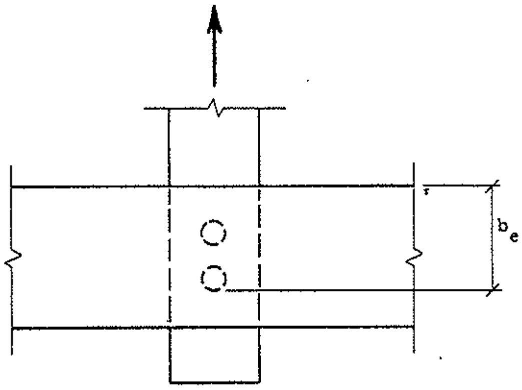


Fig. 6.1.4

6.1.5 Nail plates

: Recommendations for nail plates will be provided.

6.2 Glued joints

For continuous glued joints connecting unjointed laminae (e.g. between laminae, and between flanges and webs in beams or columns) the glued joint may be assumed to have the same strength as the weakest of the jointed materials for the action in question.

For other glued joints consideration should be given to the reduction in strength caused by an uneven distribution of the stresses over the glued area, including concentration of stresses at edges etc.

: For lap joints or gusset joints a characteristic shear strength of $(1.5 - 0.75 \sin\alpha)$MPa, where α is the angle between
: the force and grain direction, may be assumed for structural timber at least corresponding to SC19. The force per
: section, however, should not be assumed greater than $(75 - 37.5 \sin\alpha)$kN corresponding to an area of 0.05 m^2.

7. DESIGN OF COMPONENTS AND SPECIAL STRUCTURES

7.1 Glued components

7.1.1 Thin-webbed beams

The stresses in thin-webbed beams may be calculated under the assumption of a linear variation of strain over the depth. In principle the stresses must satisfy the conditions given in section 5.

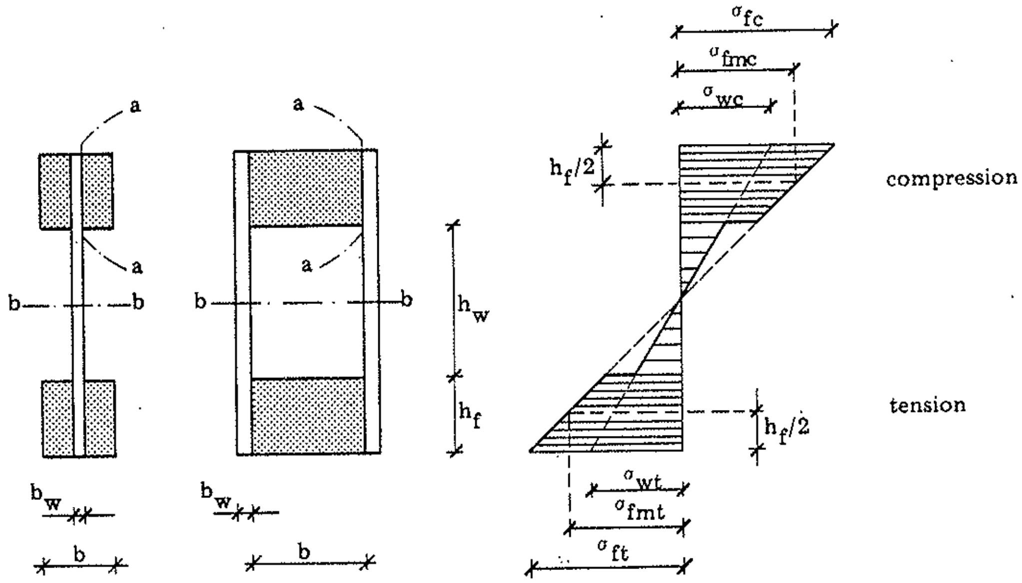


Fig. 7.1.1 a

For a beam with flanges of timber or glulam the stresses in the flanges should satisfy the following conditions, cf. fig. 7.1.1 a:

$$|\sigma_{fc}| \leqslant f_m \tag{7.1.1 a}$$

$$|\sigma_{fmc}| \leqslant k_{col} f_{c,0} \tag{7.1.1 b}$$

$$\sigma_{fmt} \leqslant f_{t,0} \tag{7.1.1 c}$$

$$\sigma_{ft} \leqslant f_m \tag{7.1.1 d}$$

k_{col} is determined according to section (5.1.1.9) with $\lambda = \sqrt{12}\,\ell_c/b$, where ℓ_c is the distance between the sections where lateral deflection of the compression flange is prevented, and b is given in fig. 7.1.1 a. If a special investigation into lateral instability of the beam is made as a whole $k_{col} = 1$ may be assumed.

For box beams an investigation of the lateral instability may be omitted if (7.1.1 a) is replaced by

$$|\sigma_{fc}| \leqslant k_{inst} f_m \tag{7.1.1 e}$$

where k_{inst} is determined according to section 5.1.1.3. This is on the safe side.

The web stresses σ_{wc} and σ_{wt} should be limited according to the materials used.

: If the web is made of plywood or other sheet materials the following conditions should be satisfied:

$$|\sigma_{wc}| \leq f_{wc} \qquad (7.1.1\ f)$$

$$\sigma_{wt} \leq f_{wt} \qquad (7.1.1\ g)$$

: where f_{wc} is the compression strength and f_{wt} the tensile strength.

The shear stresses may be assumed uniformly distributed over the width of the sections a-a and b-b shown in fig. 7.1.1 a.

It must be shown that the webs do not buckle.

If the webs are made from structural plywood, structural particle board or fibre board and the free depth, h_w, of the webs is less than $2h_{max}$, where h_{max} is given in table 7.1.1 and the shear force V satisfies the following conditions:

$$V \leq \begin{cases} f_v b_w (h_w + h_f) & \text{for } h_w \leq h_{max} \\ f_v b_w h_{max}(1 + \dfrac{h_f}{h_w}) & \text{for } h_{max} \leq h_w \leq 2h_{max} \end{cases} \qquad (7.1.1\ h)$$

a buckling investigation is not necessary.

It is assumed that the web is stiffened at the supports and under concentrated loads. The stiffeners should be fastened to the web and tightly fit between the top and bottom flanges. The cross-section of the stiffeners are chosen so that the whole force can be transferred between flange and stiffener.

Table 7.1.1.2

Web	h_{max}
Plywood with $\varphi < 0.5$	$(20 + 50\,\varphi)b_w$
Plywood with $\varphi \geq 0.5$	$45\ b_w$
Particle board or fibre board with $\varphi \approx 0.5$	$35\ b_w$

φ is the ratio between the bending stiffness of a strip with the width 1 cut perpendicularly to the beam axis and the bending stiffness of a corresponding strip cut parallelly to the longitudinal direction of the beam

In cases where a special investigation must be carried out it can be done according to the linear elastic theory for perfect plates simply supported along flanges and web stiffeners.

: For the case shown in fig. 7.1.1.b these assumptions lead to the following condition:

$$\frac{\sigma}{\sigma_{crit}} + \left(\frac{\tau}{\tau_{crit}}\right)^2 \leq 1 \qquad (7.1.1\ i)$$

: where σ_{crit} is the critical stress if only the axial stresses were acting and τ_{crit} the critical stress if only the shear stresses were acting.

: σ_{crit} can be determined as

$$\sigma_{crit} = k_{buck} \frac{\pi^2 \sqrt{(EI)_x (EI)_y}}{ta^2} \qquad (7.1.1\ j)$$

: where k_{buck} for a number of cases is given in fig. 7.1.1 c, and τ_{crit} can be determined as

$$\tau_{crit} = k_{buck} \frac{\pi^2 \sqrt[4]{(EI)_x^3 (EI)_y}}{ta^2} \qquad (7.1.1\ k)$$

: where k_{buck} for pure shear is given in fig. 7.1.1d.

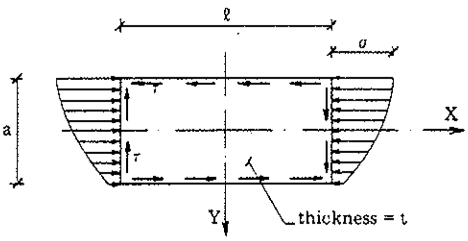


Fig. 7.1.1 b

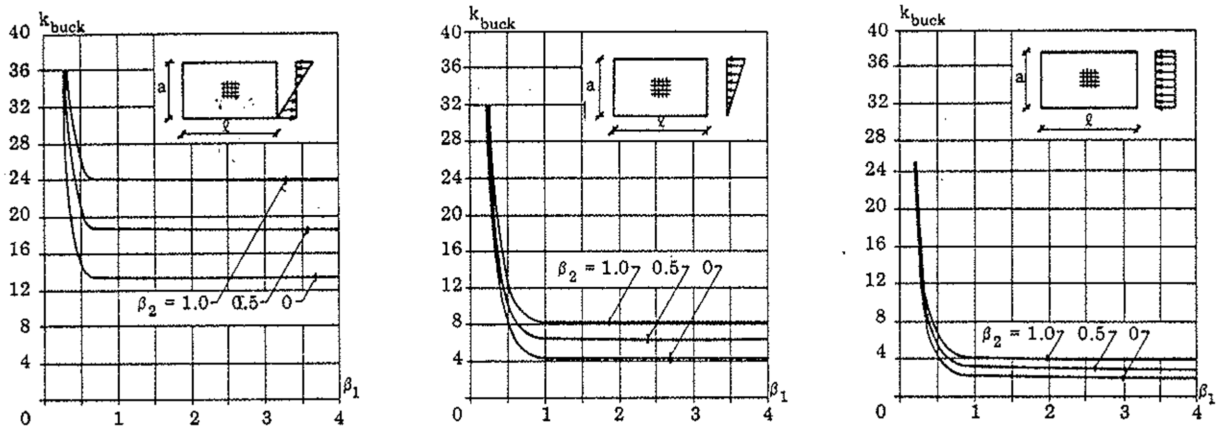


Fig. 7.1.1 c

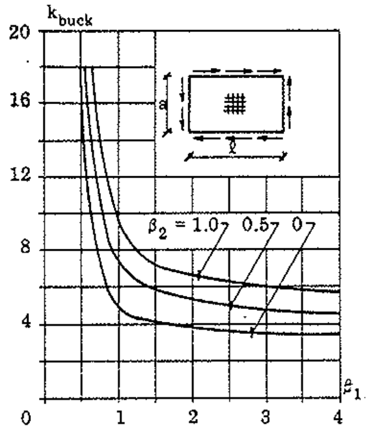


Fig. 7.1.1 d

The following notations are used (cf. fig. 7.1.1 b):

$(EI)_x$ is the bending stiffness of the panel per unit length in bending about the X-axis. For a homogeneous orthotropic panel with the main directions X and Y, $(EI)_x = \frac{1}{12} Et^3/(1 - \nu_{xy}\nu_{yx})$, where ν_{xy} and ν_{yx} are Poisson's ratios. For wood-based panels $\nu_{xy}\nu_{yx} \approx 0$ can be assumed.

$(EI)_y$ as $(EI)_x$, but in bending about the Y-axis.

$(GI)_v$ is the torsional stiffness per unit length of the panel. For a homogeneous orthotropic panel, $(GI)_v = Gt^3/3 + [\nu_{xy}(EI)_x + \nu_{yx}(EI)_y] \approx Gt^3/3$.

β_1 $= \frac{\ell}{a} \sqrt[4]{(EI)_x/(EI)_y}$. For an isotropic panel, $\beta_1 = \ell/a$.

β_2 $= 0.5\,(GI)_v/\sqrt{(EI)_x(EI)_y}$. For an isotropic panel, $\beta_2 = 2G/E$.

In calculations of deflection the contributions from the shearing stresses in the webs should be taken into account.

7.1.2 Thin-flanged beams (stiffened plates)

The stresses may be calculated under the assumption of a linear variation of strain over the depth and the stresses must in principle satisfy the conditions given in section 5.

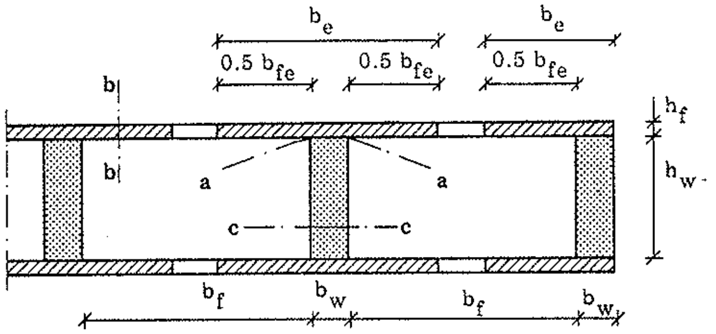


Fig. 7.1.2

The influence of the stresses not being uniformly distributed over the flange width should be taken into consideration. Unless otherwise proved the calculations can be based on an effective flange width, b_e, cf. fig. 7.1.2 , where

$$b_e = b_{fe} + b_w \tag{7.1.2 a}$$

or

$$b_e = 0.5\ b_{fe} + b_w \tag{7.1.2 b}$$

respectively.

The free effective width, b_{fe} ($\leqslant b_f$), is given in table 7.1.2.

Unless an investigation into the buckling instability of the compression flange is made, b_{fe} should not be put higher than b_{max}, also given in table 7.1.2.

Table 7.1.2

Flange	b_{fe}/ℓ	b_{max}
Plywood with fibre direction in extreme plies		
parallel to the web	0.1	25 h_f
perpendicular to the web	0.1	20 h_f
Particle board or fibre board w. random fibre orientation	0.2	30 h_f

ℓ is the span, however, for continuous beams ℓ is the distance between the points with zero moment

The buckling investigation of the compression flange can be made according to section 7.1.1.

The buckling instability of webs made of plywood or other sheet materials should be investigated in accordance with section 7.1.1, unless $h_w \leqslant 0.5\ h_{max}$ where h_{max} is given in table 7.1.1.

The shear stresses may be assumed uniformly distributed over the width of the sections a-a, b-b and c-c shown in fig. 7.1.2.

7.1.3 I- and box columns, spaced columns, lattice columns

To I- and box columns the relevant parts of section 5.1.1.9, 7.1.1 and 7.1.2 apply.

What is stated for solid columns (section 5.1.1.7) applies to spaced columns and lattice columns, but furthermore, the deformation due to shear and bending in packs, battens, shafts and flanges and to the extension of the lattice should be taken into consideration.

: Design methods for spaced columns are given in appendix 7B and for lattice columns in appendix 7C.

7.2 Mechanically jointed components

If the cross-section of a structural member is composed of several parts connected by mechanical fasteners consideration must be given to the influence of the slip occurring in the fasteners.

In addition the recommendations of sections 5 and 7.1 apply.

Calculations may be carried out according to the theory of elasticity. For slip modulus the values given in table 7.2 may be used.

Table 7.2

Fastener	Slip modulus (N/mm)
Round nails with $d < 6$ mm☆	$0.017\ E_0 d$
Round nails with $d > 6$ mm☆	$0.1\ E_0$
Bolts with pressed-in connectors	$1.3\ E_0$

E_0 is the modulus of elasticity of the timber in N/mm². d is the diameter in mm for round nails or the side length for square nails.

☆ For square nails 15% higher values are allowed.

: For beams a design method for a number of cross-sections is given in appendix 7A and for columns in appendix
: 7A-B-C.

7.3 Trusses

Trusses may be analysed as frame structures where the influence of initial curvature of the elements, eccentricities, deformations of elements and slip and rotation in the joints are taken into consideration in the determination of the resultant stresses.

As an alternative a simplified calculation after the following guidelines is permitted: The axial forces are calculated assuming hinges in all nodal points, and the moments in continuous members, if any, are assumed to lie between 80% and 100% of the simple moments (corresponding to hinges in both ends) dependent upon the degree of end-fixing and the support conditions. For non-continuous members the moments are assumed equal to the simple moments. The free column length is assumed between 85% and 100% of the theoretical nodal point distance dependent upon continuity and degree of restraint.

Appendices 7A, 7B, 7C have been presented as CIB Timber Standards 02, 03, 04 and are included in document CIB-W18/6-100-2.

8. CONSTRUCTION

8.0 General

The recommendations given in this chapter are necessary conditions for the applicability of the design rules elsewhere in this code.

Timber structures shall be so constructed that they conform with the principles and practical considerations of the design.

Materials for the structures shall be applied, used or fixed so as to adequately perform the functions for which they are designed.

Workmanship in fabrication, preparation and installation of materials shall conform in all respects to accepted good practice.

8.1 Materials

Timber and wood-based components of structural elements should not be unnecessarily exposed to climatic conditions more severe than those to be encountered in the finished structure. In particular they should not be subject to prolonged exposure to the weather or to conditions conducive to fungal or insect attack.

Timber which is damaged locally, crushed or otherwise misused should not be used for structural work.

Before construction timber should be seasoned as near as practicable to the moisture content appropriate to its climatic condition in the completed structure.

The limitations on bow in most national stress grading rules are inadequate for the selection of material for columns and beams where lateral instability may occur. Particular attention should therefore be paid to the straightness of columns; e.g. limiting bow to about 1/300 of the length. It may also be necessary to introduce more stringent limits on other particular members, e.g. twist for torsional members.

8.2 Machining

The size, shape and finish of all timber and other materials shall conform with the detailed design drawings and specifications for the structure.

The cutting of timber after preservative treatment should be avoided. However, when it is unavoidable, and exposure of untreated timber results, a liberal application of preservative should be made to the cut surfaces.

8.3 Joints

Fasteners should be placed in conformity with the drawings. The minimum distances given in section 6.1.1 - 6.1.4 should be complied with.

Wane, splits, knots or other defects are not allowed in joints to such a degree that the load-carrying capacity of the joints is reduced.

Unless otherwise specified nails should be driven in at right angles to the grain and to such depth that the surfaces of the nail heads are flush with the timber surface.

Unless otherwise stated slant nailing should be carried out in conformity with fig. 6.1.1.2 a.

Bolt holes may have a diameter not more than 2 mm larger than the bolt. Washers with a length and thickness of at least 3d and 0.3d, respectively (d is the bolt diameter), should be used under the head and nut.

Bolts should be tightened so that the members fit closely, and they should be tightened up if necessary when the timber has reached its equilibrium moisture content.

At least 2 dowels should be used in a joint. The minimum dowel diameter is 8 mm. Turned dowels should be used and the pre-bored holes in timber members should have a diameter which is 0.2 - 0.5 mm less than the dowel diameter while the pre-bored holes in steel plates should have the same diameter as the dowel. The dowels should be at least 2d longer than the total thickness of the joint.

Through the centre of each connector a bolt or screw for which the above rules are valid should be placed.

Connectors should fit tightly in the grooves.

When using toothed plates the teeth should be completely pressed into the timber. In smaller and lighter structures the bolt may be used for impressing provided it has at least 16 mm diameter. The washer should then have at least the same side length as the connector and the thickness should at least be 0.1 times the side length. It should be carefully checked that the bolt has not been damaged in tightening.

: Impressing should normally be carried out with special press tools or special clamping bolts with washers large and
: stiff enough to protect the timber from damage.

Lag coach screw holws shall be pre-drilled and treated as follows:

a. The lead hole for the shank shall have the same diameter as the shank and the same depth as the length of the unthreaded shank.

b. The lead hole for the threaded portion shall have a diameter determined by the characteristic density of the species or species group and by the length and diameter of the screw.
   : Recommendations on lead hole diameters will be provided.

c. Soap, or other non-corrosive lubricant (e.g. not ordinary petroleum) may be used to facilitate insertion of the screw.

d. Screws are to be inserted by turning with a suitable wrench, not by driving with a hammer.

8.4 Assembly

Assembly should be in such a way that unintentional stresses do not occur. Members which are warped, split or badly fitting at the joints should be replaced.

8.5 Transportation and erection

The over-stressing of members during storage, transportation and erection should be avoided. If the structure is acted upon or supported otherwise than in the finished building it must be proved that this is permissible and it must be taken into consideration that such action micht have a dynamic effect. In the case of e.g. framed arches, portal frames, etc., special care should be taken to avoid distortion in hoisting from the horizontal to the vertical position.

CIB-W18/9-102-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

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CHAPTER 27
'TIMBER CONSTRUCTION'

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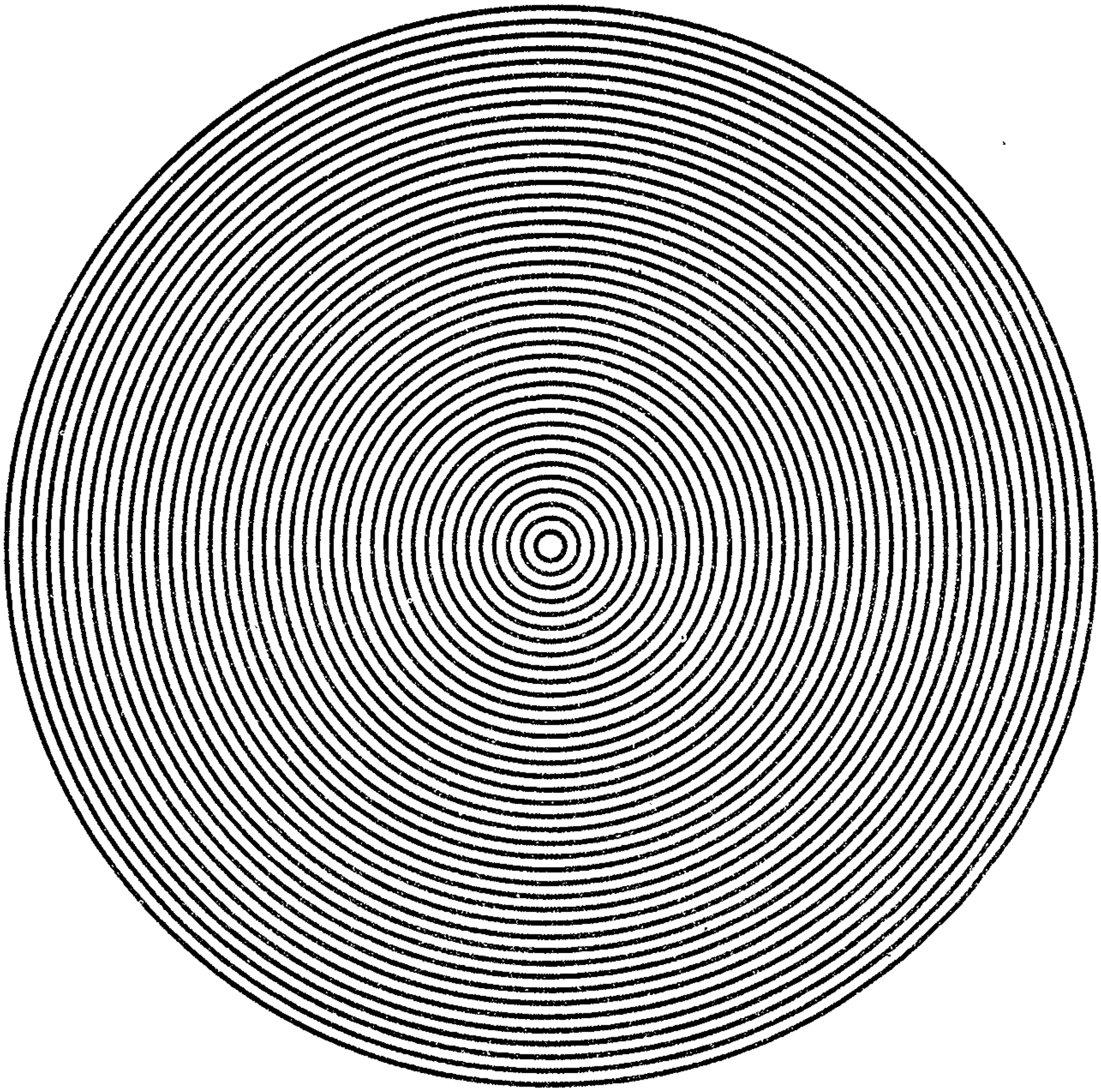
Chapter 27

'Timber Construction'



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TIMBER CONSTRUCTIONS

Code clauses are indicated by a shaded line to the left of the text.

:1 General Requirements for Timber Constructions

Timber constructions and their component parts must be designed and erected so as to ensure their long-term performance and structural integrity under loading and other conditions throughout their intended life. Timber and other timber products must be protected against rot and insect attack. Steel connectors must be protected against corrosion and glued joints must be made with a suitably durable adhesive.

:11 Grouping into climate classes

To consider climatic effects on timber constructions the constructions are grouped into climate classes. There are different requirements in different climate classes with regard to durability, also different values of the permissible stresses etc. Unless specifically mentioned Climate Class 0 should be taken as Class 1.

Climate Class 0 contains the following constructions:

Constructions indoors in permanently heated buildings not subject to humidity in the air.

Climate Class 1 contains the following constructions:

Ceiling joists and rafters in cool but ventilated attics above permanently heated rooms. External walls in permanent buildings protected by ventilated, impermeable claddings.

Climate Class 2 contains the following constructions:

Constructions in non-permanently heated but ventilated buildings, or rooms where high humidity is not generated, or storerooms i.e. holiday houses, cold garages, cold storage, farm buildings and crawl spaces. External roof boarding, scaffolding, formwork and similar temporary end uses.

Climate Class 3 contains the following constructions:
Constructions not protected from rain and wet conditions except scaffolding, formwork and similar temporary works (see Climate Class 2). Members in direct contact with the ground.

:12 Acceptable protection

Nails and screws in Climate Classes 1 and 2 (see :11) are normally accepted untreated.

A steel connector protected against moisture with a coat of zinc at least 25μm or similar corrosion protection is acceptable in Climate Classes 1 and 2. Hot dip galvanising is considered suitable for Climate Class 3 in accordance with Class A as described in SMS 2950 and with a zinc deposit of not less than 70 μm, or other suitable corrosion protection.

Glue assembly Class I is accepted for constructions in Climate Class 1. Class U accepted in all other climate classes and for constructions exposed to abnormally high temperatures (above 40°C) or in corrosive atmospheres.

:2 Requirements for Structural Safety

:21 Permissible stresses, modulus of elasticity and shearing modulus

:211 General

Permissible stresses in :212 - :214 are valid for static loads where there is no risk of rupture, deformation or any type of buckling.

Static loads are, according to Chapter 21, all loads except loads incurred by vehicles, cranes etc. Wind load may be regarded as static where there is no risk of sympathetic vibration.

:212 Construction timbers and glulam

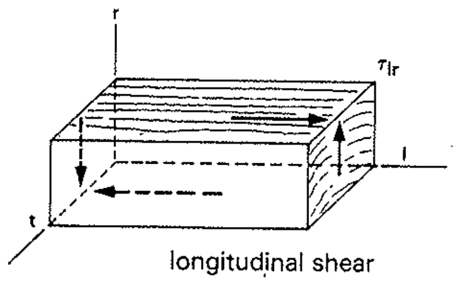
Permissible stresses and modulii of elasticity and shearing are given in Table 27 : 212a, under normal loading conditions, for structural timbers and glulam as per :411 and :412 respectively. The values given are applicable to Climate Class 1. To obtain the values in the other climate classes the values are multiplied by the factors in Table 27 :212b. The figures in Table 27 :212a should also be modified by the factors given in Table 27 :212c to take account of permanent and short-term loads.

The values in Table 27 :212a are applicable to T-timber of the grade specified. It is acceptable to use lower grades within the same structure provided the appropriate stresses are used for the design of these members and that they are clearly identified on the working drawings (see :4111).

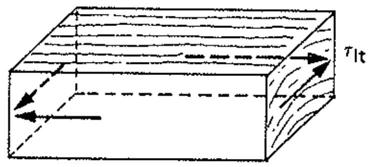
Table 27 :212a: Permissible stresses and modulii for structural and laminated timber of spruce and fir in Climate Class 1 — N/mm².

Stresses	Symbols	Structural timber			Laminated timber			
		T 30	T 20	Ö-virke	L 50*d)*	L 40	L 30	L 20
a) Bending on strong axis	σ_{bxa}	10	8	6	15	13	11	9
b) Bending on weak axis	σ_{bya}	10	7	5	11	9	9	9
Tension // to grain	σ_{tla}	9	6	3	10	8	8	6
Tension ⊥ to grain	$\sigma_{tra}, \sigma_{tta}$	0.2	0.2	0.2	0.3	0.2	0.2	0.2
Compression // to grain	σ_{cla}	9	7	5	12	12	10	8
Compression ⊥ to grain	$\sigma_{cra}, \sigma_{cta}$	2.0	2.0	2.0	2.5	2.5	2.5	2.0
c) Longitudinal shear	τ_{lra}, τ_{lta}	1.0	1.0	0.8	1.2	1.2	1.0	1.0
c) Cross shear	τ_{rta}	0.5	0.5	0.4	0.6	0.5	0.5	0.5
Modulus of elasticity // to grain	E_l	9 000	8 000	7 000	11 000	11 000	9 000	8 000
Longitudinal shear modulus	G_{lr}, G_{lt}	450	400	350	550	550	450	400

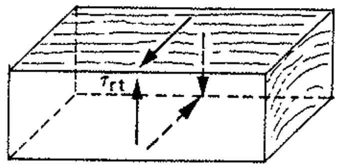
a) Applicable to horizontally laminated beams and square sections.
b) Applicable to vertically laminated beams.
c) For definitions of shear see Fig. 27 :212a.
d) This is not a readily obtainable grade.



longitudinal shear



longitudinal shear



cross shear

Fig. 27 :212a
Radial shear
Tangential shear
Cross shear

Table 27 :212b: Conversion factors for classes other than that shown in Table 27 :212a.

	Climate Class 2	Climate Class 3
Permissible stress	1.0	0.75
E and shear modulii	0.8	0.6

Table 27 :212c: Conversion factors for conditions of load other than normal.

	Permanent Load	Normal Load	Exceptional Load *a)*
Permissible stress	1.0	1.0	1.4
E and shear modulii	0.7	1.0	1.3

a) Includes load on scaffolds, formwork etc. if not expected to be for more than one weeks duration.

The bending stresses indicated in Table 27 :212a are valid for straight glulam components with rectangular cross-section. The values for bending should be multiplied by a factor Kr when the glulam components are curved with the moment axis parallel to the glue lines.

$$Kr = 1 - 15 \frac{t}{r} \qquad (27 :212)$$

Where:

t = thickness of laminate
r = the radius of bending at the centre of gravity of the laminates.

The values for T30 given in Table 27 :212a are also applicable to round timber as described in :4115.
Beech and oak, graded to the requirement for T30 (see :4111) can be designed using the stresses and modulii given in Table 27 :212a multiplied by a factor of 1.2. The factor can be increased to 1.5 for compression at right angles to the grain.

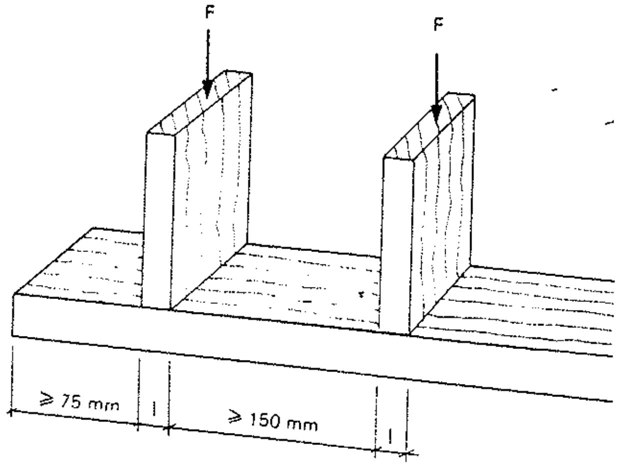


Fig. 27 :212b
Minimum dimensions for sole plates and studs.

Table 27 :212d: Conversion factors for sole plates and studs as shown in Fig. 27 :212b related to the values in 27 :212a.

Loading length l	10 mm	30 mm	50 mm	100 mm
Factor Ks	1.8	1.4	1.2	1,0

When loading takes place at an angle to the grain the normal and shearing stresses are calculated parallel with and at right angles to the grain direction, and they should be less than the permissible value. The resultant stress may only reach the highest permissible stress in the grain direction.

When several timber components act together higher allowable stresses can be justified, for example tongued and grooved boarding. The values in Table 27 :212a may be increased but not to more than the values for T30.

With local stress at right angles to the grain as in Fig. 27 :212b the permissible stress given in :212 may be multiplied by a factor K_s which is dependent on the bearing length in the grain direction as per Table 27 :212d, provided that deformation does not prejudice the construction's function and safety.

:213 **Structural plywood**

Permissible stresses and elastic and shearing modulii for normal loading conditions are given in Table 27 :213 for structural plywood, described in :413, of spruce or fir not less than 200 mm in width. For other loading conditions the values should be multiplied by the factors given in Table 27 :212c.

For a continuous glue joint between a deck of structural plywood and a joist with a width not exceeding 30 mm and designed as Fig. 27 :213b the permissible shear stress in the parallel plies (τ_{lra}) is doubled and the permissible shear stress in the perpendicular plies (τ_{rta}) is increased by 50%. This increase presupposes that the joint is not exposed to compression at right angles to the joint surface except the board's own weight and the weight of a light insulation.

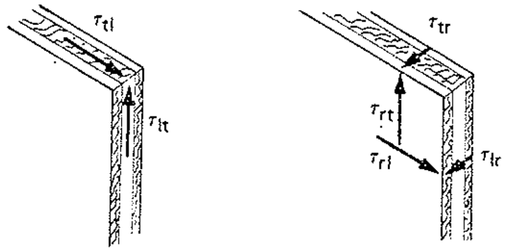


Fig. 27 :213a
Shear influence on plywood

Table 27 :213: Permissible stresses and modulii for structural plywood (N/mm²).
To obtain Climate Class 0 multiply values for Climate Class 1 by 1.1.

Stresses	Symbols	Climate Class 1			Climate Class 2		
		P 40	P 30	P 20	P 40	P 30	P 20
a) Bending	σ_{ba}	15.0	12.0	8.0	10.5	8.5	5.5
a) Tension // to grain	σ_{tla}	13.0	10.0	8.0	11.5	9.0	7.0
a) Tension ⊥ to grain	σ_{tra}	0.4	0.4	0.4	0.3	0.3	0.3
a) Compression // to grain	σ_{cla}	12.0	10.0	8.0	8.5	7.0	5.5
a) Compression ⊥ to grain	σ_{cra}	2.0	2.0	2.0	1.4	1.4	1.4
b) Panel shear	τ_{lta}	2.0	2.0	1.5	1.4	1.4	1.05
c) Rolling shear	τ_{rta}, τ_{lra}	0.6	0.6	0.6	0.4	0.4	0.4
a) Modulus of elasticity	E_l	9 000	9 000	8 000	6 000	6 000	5 500
b) Shear modulus	G_{lt}	450	450	400	300	300	270

a) Only veneers parallel to span are considered. The values are based on at least two parallel veneers. If only one reduce values by a half.
b) Shear stress is valid parallel or perpendicular to veneers. At 45° shear value may be doubled.
c) Values valid at all angles to grain.

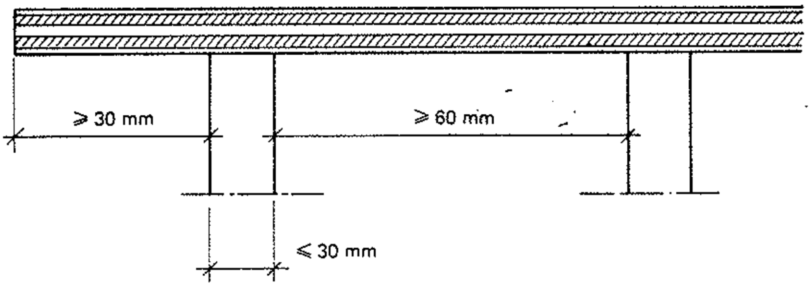


Fig. 27 :213b: Continuous glued joint between sheet material deck and ribs not less than 30 mm wide.

Table 27 :214: Permissible stresses and modulii for structural fibreboard and structural chipboard (N/mm²). To obtain Climate Class 0 permissible stresses in Climate Class 1 are multiplied by 1.1 and elasticity and shear modulii by 1.3.

	Stresses	Symbols	Fibreboard						Chipboard Climate Class 1		
			Climate Class 1			Climate Class 2			Thickness mm		
			O/T 50	Hard 35	Med. 13	O/T 50	Hard 35	Med. 13	9-13	16-19	22-25
	Bending	σ_{ba}	9.5	6.5	3.0	5.5	3.0	1.0	4.5	4.0	3.5
	Tension // to surface	σ_{tla}	5.5	5.0	1.4	3.5	2.0	0.8	2.0	1.6	1.3
	Tension ⊥ to surface	σ_{tra}	0.2	0.15	0	0.08	0.05	0	0.07	0.06	0.05
	Compression // surface	σ_{cla}	5.5	3.5	1.4	2.5	1.3	0.4	2.7	2.4	2.1
	Compression ⊥ surface	σ_{cra}	5.0	5.0	0.9	5.0	5.0	0.5	2.0	1.5	1.5
	Panel shear	τ_{lta}	3.5	3.0	0.8	2.0	1.3	0.6	1.5	1.3	1.1
a)	Rolling shear	τ_{rta}, τ_{lra}	0.35	0.25	0.04	0.3	0.2	0.04	0.25	0.2	0.2
	E bending	E_{lb}	2 700	1 300	700	1 800	900	350	1 900	1 500	1 200
	E tension and comp.	E_{lt}	2 700	1 300	700	1 800	900	350	1 500	1 200	1 000
	Shear modulus	G_{lt}	1 350	650	350	900	450	180	750	600	450

a) Increase for small loaded surfaces, see :213

:214 Structural fibreboard and structural chipboard

Permissible stresses and elasticity and shear modulii for normal conditions of load for structural fibreboard (K-board) as per :414 and for structural chipboard as per :415 are in Table 27 :214.

For exceptional cases of load the values are multiplied by 1.5 and for permanent load by 0.6.

For a continuous glued joint between a deck made of structural fibreboard or structural chipboard and a joist with a width not exceeding 30 mm, designed as per Fig. 27 :213b, the permissible shear stress as per Table 27 :214 may be doubled. This increase presupposes that the joint is not exposed to tension at right angles to the joint surface except by the board weight and the weight of light insulation.

:22 Permissible loads on joints

:221 General

The permissible loads given in :22 are valid for normal case of load in Climate Class 1 unless otherwise stated. For other Climate Classes or conditions of load the figures may be multiplied by the factors given in Table 27 :221.

Table 27 :221: Conversion factors for permissible permanent loads on joints in different materials and Climate Classes.

Material	Climate Class 1		Climate Class 2		Climate Class 3	
	Normal Load	Exceptional Load	Normal Load	Exceptional Load	Normal Load	Exceptional Load
Timber, plywood	1.0	1.4	1.0	1.4	0.75	1.05
Fibreboard	1.0	1.4	0.5	0.7	—	—
Chipboard	1.0	1.4	—	—	—	—

:222 Nail joints

:2221 Permissible lateral loading, timber to timber

A nail with a tensile strength (this is referring to the steel specification) in accordance with :42 which is driven perpendicular to the grain in spruce or pine, for normal conditions of loading, in Climate Classes 1 and 2, may be loaded per nail in accordance with Table 27 :2221, provided that the length of the nail (not counting the tip) in each piece of timber is at least 7d where d indicates the cross section of the nail (see sketch).

When the nail penetration is less than 7d, the permissible load values are reduced in proportion to the length of nail in the timber. The thickness of the timber must be at least 5d. Joints intended to transmit loads must have at least two nails.

The spacing of nails along the grain and the end distance must be at least 10d.

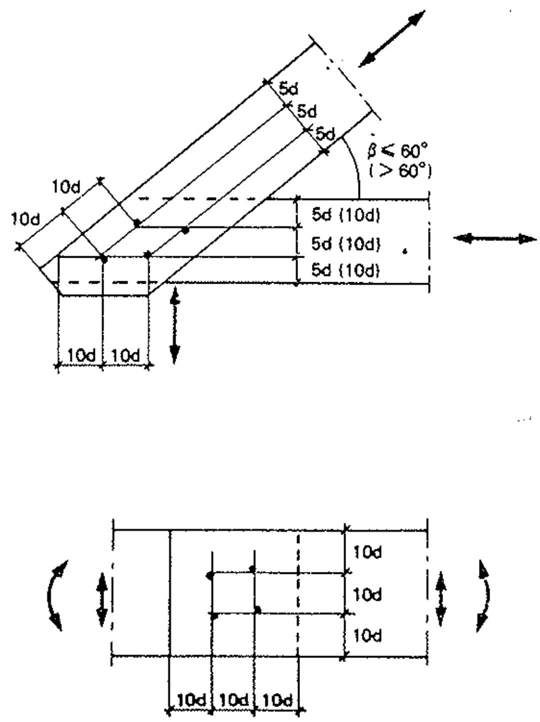


Fig. 27 :2221: Minimum space between nails and from nails to end and edges of timber.

Table 27 :2221: Permissible lateral loading on square nails and improved nails. The values must be multiplied by 0.8 for round wire nails.

Nail size			Permissible lateral load/nail N/nail	Nail size			Permissible lateral load/nail N/nail
Nail[a] Number	Dia. (mm)	Length[b] (mm)		Nail[a] Number	Dia. (mm)	Length[b] (mm)	
14	1.4	25	80	40	4.0	125	600
17	1.7	35	120	43	4.3	125	700
19	1.9	40	140	47	4.7	150	800
21	2.1	40, 50, 60	170	51	5.1	150	900
23	2.3	50, 60	200	55	5.5	175	1 000
25	2.5	60	250	60	6.0	200	1 100
28	2.8	75	300	65	6.5	225	1 200
31	3.1	75	350	70	7.0	250	1 400
34	3.4	100	450	80	8.0	300	1 700
37	3.7	100	500				

a) The nail number is ten times its diameter in millimetres.
b) Including the tip, the length of which is approx. 1.5d.

:2222 **Permissible lateral loading: sheet material or steel to timber**

In a joint between K board (see :413, :414 and :415) and a piece of timber of pine or spruce at normal loading conditions (these normal loading conditions are the same as the British 'long term loading', not the Canadian 'normal' loading) and Climate Class 1, the permissible lateral loading per nail is as given in Table 27 :2221, providing that the board thickness is at least 3.5d (or 1.8d for hardboard or oil-tempered board) and that the length in the timber is at least 7d. For thinner board the permissible lateral loading is reduced in proportion to the thickness.

The edge distances given in :2221 must be maintained but the spacing between nails may be reduced by 20%.

For 13 mm plasterboard nailed to timber with 35 × 17 nails under exceptional loading conditions and Climate Class 0, the permissible lateral force is 100N/nail. The edge distance should not be less than 15 mm and centres of nails not less than 100 mm.

For steel to timber joints the permissible lateral force is 1.25 times the permissible lateral load for timber to timber joints.

:2223 **Permissible withdrawal loading**

For axial loads in nails driven perpendicular to the grain the permissible load in withdrawal is equal to the minimum value given by formula 27 :2223.

$$F_a \leqslant \begin{cases} \mathcal{T}_b d\,(l - 1.5d) \\ \mathcal{T}_b d\,(b + l_f) \end{cases} \qquad (27\ :2223)$$

Symbols as in Fig. 27 :2223a. $\mathcal{T}_b$ and l_f are in Table 27 :2223.

Formula 27 :2223 may also be used for K-board nails fixed in timber.

Table 27 :2223 : Values for $\mathcal{T}_b$ and l_f in formula 27 :2223.

Nail type	$\mathcal{T}_b$ N/mm²	l_f a) $d_h \geqslant 2.5d$
Round wire	0.7	14d
Improved nails b)	2.0	5d

a) For $d_h < 2.5$ multiply l_f by $0.67\ (d_h/d - 1)$

b) Only the threaded portion of the nail should be assumed to transmit load.

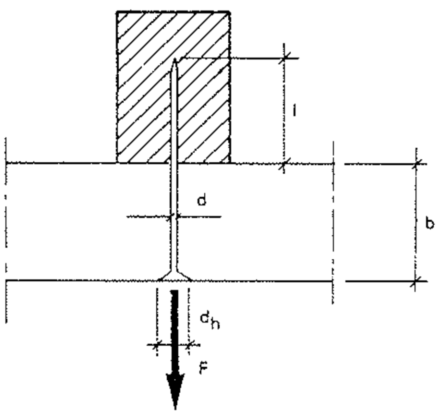


Fig. 27 :2223a Nail withdrawal.

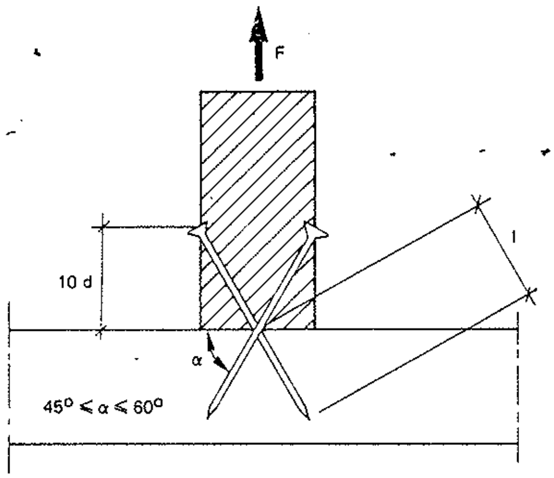


Fig. 27 :2223b Inclined nailing

For inclined nails as shown in Figure 27 :2223b assuming symmetrical nailing, the permissible load F per nail is calculated from formula 27 :2223 provided that F acts parallel with the direction of the nail straightening effect. With calculations of 'l' assume $\alpha = 45°$. For other angles nail holes must be started with a centre drill.

:223 **Nail plate joints**

Approved designs and calculation methods are indicated in 'Nail Plate Joints'. Statens Planverk approving rules No. 4 (1974).

:224 **Bolted joints without connectors**

When using material according to :42 and design as per :5 a lateral force is permitted in each member in a bolted joint equivalent to the lowest value obtained from the formula 27 :224.

The bolt is presupposed to be applied at right angles to the grain direction.

$$F_a \leqslant \begin{cases} 2.2\,(\mathcal{K}_{b1}\,b_1 + \mathcal{K}_{b2}\,b_2)\,d^{\,a} \\ 1.5\,\mathcal{K}_{b1}\,b_1\,d + 8.0\,d^2 \\ 9.0\,\mathcal{K}_{b1}\,b_1\,d \\ 4.5\,\mathcal{K}_{b2}\,b_2\,d^{\,b} \\ 12\,d^2\sqrt{\mathcal{K}_{b1} + \mathcal{K}_{b2}} \end{cases} \qquad (27\ :224)$$

a only valid for two member joints

b only valid for connectored joints

Where:

F_a	Permissible lateral force (N/joint)
b	Thickness of timber (mm)
d	Bolt diameter (mm)
$\mathcal{K}_b$	Factor as given in Table 27 :224

Index 1 signifies a side member and Index 2 an internal member. In a two member joint 1 and 2 represent the timber members, when the index is chosen so that: $\mathcal{K}_{b1} \cdot b_1 \leqslant \mathcal{K}_{b2} \cdot b_2$

The distance between bolts in the grain direction must be at least 7d and across the grain direction at least 4d and in other directions at least 5d.

The distance in the grain direction from a bolt to the end of the timber must be at least 7d when a component of the force in the bolt is along the grain in the direction of the end (the loaded end), in other cases at least 4d.

The distance across the grain from a bolt to edge of the timber must be at least 4d when there is a component of the force in the bolt across the grain in the direction of the edge (loaded edge), in other cases at least 2d. See Fig. 27 :224.

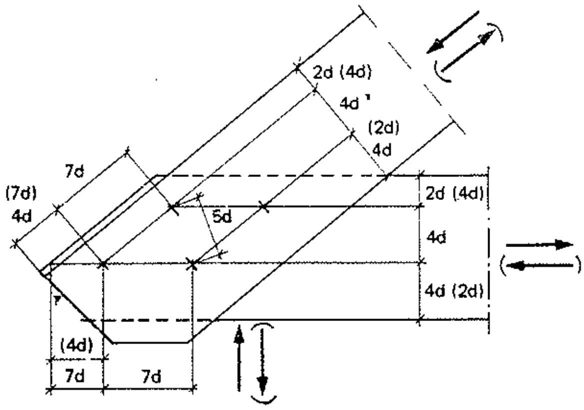


Fig. 27 :224 — The figure shows the minimum spacing and edge distances for bolts without connectors. The figures in brackets signify loaded distances for alternative load directions.

Table 27 :224 — Factor Kb in the formula 27 :224.

Angle of load to grain	K_b for bolt diameter (mm) [a] d = 6	d = 12	d >25
0°	1.00	1.00	1.00
30°	1.00	0.89	0.80
45°	1.00	0.80	0.67
60°	1.00	0.72	0.57
90°	1.00	0.66	0.50

a) interpolate to obtain values between d.

:225 **Bolted joints with Connectors**

The permissible lateral force can be calculated as the sum of the permissible lateral force for bolts as per :224 and the permissible lateral force for connectors provided that the bolted joint with connectors as defined in :42 fulfil the requirements of :5 and the following stipulations regarding minimum distances and minimum timber thickness.
The minimum distance for joints with spacers is indicated in Table 27 :225.

Table 27 :225 — Minimum distance for joints with connectors.

Distance	Toothed plate Round	Toothed plate Square	Others
Centre to centre	1.25d [a]	1.5d [a]	1.75d [a]
Centre to edge in grain direction	1.25d	1.5d	1.75d
⊥ to grain direction	0.6d	0.7d	0.8d

[a] d = indicates the diameter of round connector or edge of square connector.

The minimum thickness of timber edge member is, for toothed plate connectors (see Page 66 Träbyggnadsord-lista) 1.5b, where b is the plate's maximum depth; for only one sided toothed plates double the depth. The minimum timber thickness for an internal member at a toothed plate connector is 1.5b and for other connectors 2b.

Permissible loads for timber connectors can be obtained from general approval certificates from Statens Planverk.

:226 **Wood screw joints**

:2261 **General**

If the material is as specified in :42 and design is carried out in accordance with :5 the permissible lateral forces in wood screws inserted perpendicular to the grain are given in :2262 and :2263.

:2262 **Permissible lateral forces**

If fulfilling the assumptions a) - c) below it is permissible to have a lateral force in two member joints of fir and spruce equal to the lowest value according to Formula 27 :2262.

a) The timber thickness 'b' must be at least 2d
b) The length of penetration must be at least 8d
c) Minimum distances as per :224 where d is the screw's shank diameter.

$$F_a \leqslant \begin{cases} 4 K_b\, b\, d \\ 16\, d^2 K_b \end{cases} \qquad (27\ :2262)$$

Where:

- F_a Permissible lateral force (N)
- K_b Factor as per Table 27 :224
- b Thickness of head-side member (mm)
- d Shank diameter of screw (mm)

:2263 **Permissible withdrawal forces**

Permissible withdrawal forces are calculated using Formula 27 :2263 for screws with a shank diameter in excess of 3 mm and minimum distances as defined in :224.

$$F_a = T_b(d + 2.5)l_{gf} \qquad (27\ :2263)$$

- F_a Permissible withdrawal load (N)
- l_{gf} Length of thread penetration (mm)
- d Shank diameter of screw (mm)
- T_b Permissible stress
  - T_b = 4N/mm² for fir and spruce
  - T_b = 6N/mm² for beech and oak

:227 **Glued joints**

Drawings must show the assumed gluing class and stress grade, for example 'Glue-joint U6'.

The type of glue should be chosen with regard to climatic conditions to which the construction is expected to be exposed (see :12).

The following permissible shear stresses are applicable to the glue joint. Where the relevant permissible stress for the material is lower then the lower stress must be used.

For stress grade 6 a shear stress of 1.2N/mm² is permissible.
For stress grade 3 a shear stress of 0.6N/mm² is permissible.

:2271 For more information on glued timber constructions see :412 and for glue see :42.

:2272 Finger jointed construction timbers are accepted as mentioned below on the assumption that the joints are constructed and controlled in such a manner as recommended by Planverket.

1. Buildings with not more than two storeys or 10m height.
   a) Columns
   b) Bending or tension members in a beam or framework when the centre distance between beams or the frameworks is not greater than 1.3m and at least 3 act together so that failure of one member would not lead to a collapse of the structure as a whole.
2. Buildings with more than 2 storeys.
   a) Roof structure propped from timber beams or slab of other material.
   b) In an infill wall that carries only its own weight plus windload and other lateral load as per assumptions in 1b).

Finger jointed timber must not be used in scaffolding or other constructions exposed to impact forces.

:2273 Nail or screw glued joints are permitted in Class 3 and 6 stress grades: Other forms of joint are only permitted in Class 6.

The choice of glue and distance between fixings is made by the manufacturer basing his judgement on previous tests and does not necessarily have to be mentioned on the drawing.

:3 Design Considerations

:31 Conditions for calculation

When calculating stresses and deformations of timber constructions it is assumed that there is correlation between bending stress and modulus of elasticity in timber materials. Displacements of mechanical joints are taken into consideration.

Where members are joined by connectors of the same or differing types the force distribution must be determined with regard to the deformation of the timber components as well as the stiffness and rigidity of the joints.

:311 In permanent buildings mechanical joints may be calculated in the following way. The joint is designed for moment calculated by the elastic theory, but the whole construction should be designed so that the moment in the joint does not exceed 0.8 of the moment by elastic theory.

:312 The maximum moisture movements for timber in Climate Classes 1 or 2 are assumed to be 0.4 mm/m in the grain direction and 10 mm/m at right angles to the grain.

Maximum moisture movement in the primary direction in unrestrained structural plywood in Climate Classes 1 or 2 is assumed to be 0.5 mm/m. If the cross-sectional area of veneers in the primary direction exceeds 60% of the board's cross section 0.4 mm/m is assumed in that direction and 0.6 mm/m at right angles.

The maximum moisture movement of unrestrained structural fibreboard in Climate Classes 1 or 2 is assumed to be 1.3 mm/m for hard or medium density fibreboard, 1.0 mm/m for an oil-tempered board and for chipboard in Climate Class 1, 1.2 mm/m.

Design of floor shutterings in accordance with "Dimension tables for traditional floor shutterings" Building Research Report R15/1970 is accepted as satisfying SBN 75.

:32 Columns

For columns the permissible compression stress parallel to grain given in Table 27 :212a should be reduced by multiplying the factor k_l as per Table 27 :32. The factor k_l is a reduction factor based upon the slenderness ratio $\lambda = l/i$ where 'l' is the column's effective length and 'i' the relevant radius of gyration.

The values in Table 27 :32 assume that the initial bow in the unloaded column does not exceed 1/300 of its effective length.

:321 For single and fully load-sharing columns the effective length is based upon the support and position of the column as well as its cross-sectional properties and the load. In such cases calculations as per Chapter 157 in the manual Bygg 1A, 1971 are permitted. Fully fixed end conditions are normally not possible. For cases without fully fixed ends it is acceptable to use the recommendations contained in StBK-N1 "Steel Building Regulations 70" 33 :357.

Table 27 :32 — Reduction factors for compression.

Slenderness ratio λ	Factor K_l	Slenderness ratio λ	Factor K_l
20	1.00	100	0.28
30	0.91	110	0.23
40	0.81	120	0.20
50	0.72	130	0.17
60	0.63	140	0.14
70	0.53	150	0.12
80	0.44	160	0.11
90	0.35	170	0.10

:322 The definition of the slenderness ratio given in Chapter 363 in the manual Bygg 3, 1969 is accepted to be used for a simple column where the members are intermittently glued or nailed together.

:323 Where a joint occurs at right angles in a column and is fully braced or strutted to prevent movement at the joint, no more than two thirds of the calculated compression is permitted through the joint.

The following Formula 27 :323a should be used for columns having an assymetric cross section subject to bending and direct compression.

$$\frac{\sigma_n}{\sigma_{na}} + \frac{\sigma_{bx}}{\sigma_{bxa}} + \frac{\sigma_{by}}{\sigma_{bya}} \leqslant 1 \quad (27\ {:}323a)$$

Where:

$\sigma_n, \sigma_{bx}, \sigma_{by}$ axial stress, bending about strong axis and bending about weak axis respectively.

$\sigma_{na}, \sigma_{bxa}, \sigma_{bya}$ Permissible compression stress and bending stresses in the two directions respectively.

For a column σ_{na} is calculated as per :32.

For bending in the strong direction when there is a risk of twisting σ_{bxa} is calculated by :331.

For inclined loading without axial force and without risk of bending, design may be carried out according to 27 :323b

$$\frac{\sigma_{bx}}{\sigma_{bxa}} + \frac{\sigma_{by}}{\sigma_{bya}} \leqslant 1.2 \quad (27\ {:}323b)$$

Where the following should be valid

$\sigma_{bx} \leqslant \sigma_{bxa}$ and $\sigma_{by} \leqslant \sigma_{bya}$

Symbols as 27 :323a

:33 Beams

:331 Lateral restraint

For beams that are symmetrically loaded and subject to bending the permissible bending stress σ_{bxa} (as per Table 27 :212a) should be modified to take into account the risk of twisting by multiplying with Factor κ_v as per Table 27 :331. In the table κ_v is indicated as a function of the beam's slenderness expressed as:

$$\alpha = \sqrt{\frac{3\sigma_{ba}}{\sigma_{el}}} \quad (27\ {:}331a)$$

where σ_{el} is the calculated tension stress according to the elastic theory. The values in Table 27 :331 presuppose that the beam has an initial unloaded bow of less than 1/300 of the distance between lateral supports.

Calculation of σ_{el} can be carried out, for example, with the rules given in StBK-K2 'Elastic stability, lateral torsional buckling, 1973,' where permissible values of stress and elastic modulus are given in :212.

Table 27 :331 — Reduction factors for lateral torsional buckling.

Slenderness	Factor κ_v
$\alpha \leqslant 0.6$	1
$0.6 < \alpha < 1.4$	$1.37 - 0.61\,\alpha$
$\alpha > 1.4$	$\frac{1}{\alpha^2}$

For a straight beam with solid rectangular cross section loaded in the strong direction and end supported to prevent twisting or lateral deflection it is permissible to calculate the slenderness factor α as follows:

$$\alpha = 0.07\frac{\sqrt{h\,l}}{b} \quad (27\ {:}331b)$$

Symbols

b the width of the beam
h the depth of the beam
l the length of the beam

For beams loaded by secondary beams that are preventing lateral deflection of the compression edge 'l' may be calculated as the distance between the secondary beams.

For trussed beams with supports designed to restrain the ends of the beam lateral stability calculations should be carried out using Formula 27 :331b but 'l' should be taken as twice the beam length.

For a beam with I -cross section and with flanges of structural timber or laminated timber the bending stresses need not be reduced provided that lateral restraints are provided to the compression flange at centres not in excess of 10 times the width of the flange.

For longer centres of lateral restraints the compression flange of the beam may be considered as a column spanning between restraints and the actual compression stress limited as described in :321.

:332 Shearing stresses on ends of beams

When calculating shearing stresses in a beam resting on its bottom edge the load on the top edge may be ignored if it occurs within a distance from the support equivalent to the depth of the beam.

For a beam notched at the support the reduced capacity that is caused by the notch over and above the reduction due to the reduced cross section is given by the permissible shear stress as in Table 27 :212a modified by the Factor κ_u as per Table 27 :332. The symbols are explained in Fig. 27 :332.

Table 27 :332 — Reduction factor for notch at support.

a as per 27 :332	Factor κ_u
$a < 3\,(h - h_1)$	$\frac{h_1}{h}\left(1 + \frac{a}{3h_1}\right)$
$a \geqslant 3\,(h - h_1)$	1.0

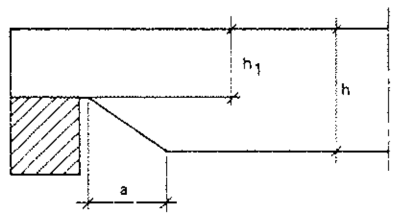


Fig. 27 :332
Notch at support.

:333 **Beam with structural timber flanges and webs of sheet materials**

The design method for I-beams with timber flanges and webs of a sheet material is based upon Formula 27 :323a, where σ_n is calculated — deformation due to slip in mechanical fasteners being considered — as the average stress on the flange and σ_{bx} as the difference between edge and average stress. Furthermore, consideration must be given to the differences in elastic modulus between the two materials.

Reduction of permissible stresses with regard to buckling of the web is not necessary if the clear distance between the flanges does not exceed the highest value h_o in Table 27 :333 and the beam is equipped with stiffeners at supports and under concentrated loads.
In the table η symbolises the ratio between the bending stiffness for a piece of plywood at right angles to the beam's axis and the gross bending stiffness.

The bearing area should be chosen sufficiently large that the end load can be transferred to the web without the permissible stress at right angles to the fibre direction for the flange timber as per Table 27 :212a being exceeded. Web stiffeners should be fixed tightly between the upper and lower flanges. Any gap should not exceed half the thickness of the web.

Table 27 :333 Limits of h_o for sheet material web.

Sheet material ratio	Height of web h_o
K-plywood	
$\eta < 0.33$	$(22 + 70\,\eta)\,t_w$
$\eta \geqslant 0.33$	$45\,t_w$
K-board	$35\,t_w$

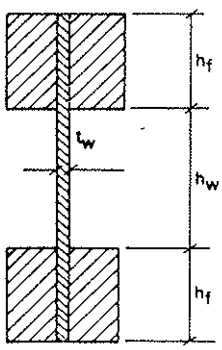


Fig. 27 :333 — Beam with timber flanges and sheet material web.

At web height between h_o and $2h_o$ buckling of the web must be checked using the formula for lateral force, V_a, 27 :333. For deeper webs than $2h_o$ special investigation of the problem is required.

For $h_w \leqslant h_o$

$$V_a \leqslant \tau_{lta}\, t_w\,(h_w + h_f)$$

For $h_o \leqslant h_w \leqslant 2\,h_o$

$$V_a \leqslant \tau_{lta}\, t_w\, h_o\left(1 + \frac{h_f}{h_w}\right) \qquad (27\ :333)$$

τ_{lta} as per Table 27 :213 and 27 :214 respectively
h_o as per Table 27 :333

Other symbols as Fig. 27 :333.

:334 **Stressed skin panel**

For a flange made of sheet material glued to a rib as per Fig. 27 :334 it is accepted, without further investigation that effective width b_e is the lowest value according to Table 27 :334, when the actual flange width is b_f.
For a compression flange where b_f is greater than twice b_e consideration should be given to buckling and a special investigation is needed.

When designing note the differences between the elastic modulii of the ribs and flanges.

There is considered no risk of buckling of the rib if the height is less than $h_o/2$ with h_o as per Table 27 :333 and the beam has cross noggings at the supports. If the ribs are stiffened :333 may be used for buckling design considerations.

Table 27 :334 Effective width b_e

Skin material	b_e relative to Shear	b_e relative to Buckling[a)]
Plywood with face grain // to span	$0.14\,L$[b)]	$25\,t_f$
⊥ to span	$0.14\,L$	$30\,t_f$
Fibreboard, Chipboard	$0.30\,L$	$30\,t_f$

a) Only compression skin.

b) Where L is the design clear span or the distance between points of zero moment where continuity occurs.

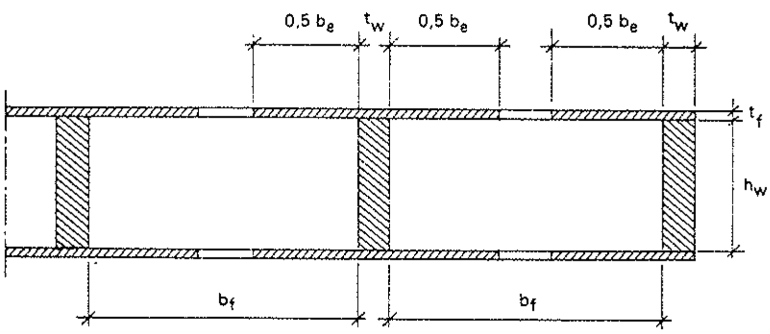


Fig. 27 :334 — Stressed skin panel.

:34 **Frameworks**

:341 Axial loads in a framework may be calculated on the basis of pin jointed connections.

When a difference occurs between this theoretical procedure and the real centre of gravity lines of the members, due regard should be paid to eccentricity in the joints.

An acceptable alternative is to calculate the framework as an elastic frame construction with both members and the joint deformations being considered.

:342 Handling stresses are accepted when attention is paid to each member joint being designed to carry a load in the direction of the plane of the frame of not less than 1kN for beams spanning up to 12m and in other cases at least 2kN.

:4 **Assumptions regarding materials**

:41 and :42 lists the assumptions for such materials and products for which those permissible stresses and forces indicated in :21 and :22 are relevant.

:41 **Timber materials**

:411 **Structural timber**

:4111 **T-Timber**

All structural timber in stress grades T30 and T20, sawn and planed, is accepted if sorted and T-marked as per "Instructions for sorting and grading of T-timber", issued by the T-Timber Association and authorised by Planverket.

The T-marking should be carried out on the flat side of the timber. The T-mark should be stamped on the extreme ends of the timber and carry the mark appropriate to the grade selected.

Where for a distance at one or both ends not exceeding 20% of the overall length the timber is below grade, the position of the T-mark must indicate the length of timber of that grade and the end pieces must not be below T20 for a T30 length nor below Ö-Virke for a T20 length.

T-timber forming part of a manufactured component does not require to be marked in the manner described above, if the element is clearly visible and carries the T-marking, manufacturing mark and manufacturing number. Factories which mark in this way must have been registered by the T-Timber Association.

:4112 **Ö-Virke (other structural timber)**

Structural timber in stress grade Ö-Virke is sawn or planed timber of fifth quality or better as per 'Sorting of sawn timber of fir and spruce' issued by the Association of Swedish Sawmillmen 1965, with the following requirements:

a) The timber must not contain pronounced compression wood or cross grain and must not have a slope of grain more than 1 in 5.

b) Knots on an edge may not be bigger than 80% of the edge width. For sawn timbers smaller than 50 × 100 mm, size of knots on face or edge may not be larger than 50% of the face or edge.

c) The timber must fulfil the tolerance for sawn timber as per SIS 23 27 11 and planed timber as per SIS 23 27 12.

The responsibility for sorting the Ö-Virke as per a) and b) rests with the factory or site supervisor.

:4113 **Machine stress graded structural timber**

Structural timber in stress grades T30, T20 and Ö-Virke, machine stress graded, is permissible if sorted and controlled in a manner accepted by Planverket. Machine stress graded T30 and T20 timber must be marked with the stress grade and the letter M, machine number, the control institution's number, the manufacturer's name and Planverket's control marking. Machine stress graded Ö-Virke is marked in a similar way except the stress grade need not be noted other than by the letter M.

Machine stress graded structural timber included in a manufactured constructional unit need not be marked as indicated above, providing the unit is clearly marked with M, the machine number, the control institution's mark, the manufacturer's name and Planverket's control mark. Factories marking thus are registered by Planverket.

:4114 **Finger jointed structural timber**

Structural timber in stress grades T30, T20 and Ö-Virke, finger jointed with glue class U is permitted provided it is manufactured and controlled and marked in a manner authorised by Planverket. Finger jointed structural timber should be marked in a durable way on one of the faces close to each joint or at intervals not exceeding 1.5m along the timber. The marking should consist of Planverket's control mark, a mark to identify the manufacturer (and the factory if the manufacturer has more than one production unit), a time code indicating the year of production, gluing week, and the stress grade (i.e. T30, T20, Ö, T30M, T20M, or ÖM). (Note that there is now no requirement to mark with an F.) when the timber is machine stress graded the marking must include the number of the grading machine as allocated by Statens Planverk.

Finger jointed structural timber can only be used in situations where failure of a joint will not cause considerable damage or risk of progressive collapse as per :2272.

:4115 **Round timber**

Round timber is permissible and should be regarded as stress grade T30 provided it contains no loose rot or deep insect attack. Round timber in permanent constructions must be free of bark.

:412 **Glued timber constructions**

Glulam is a glued timber component made up of at least four laminates with the grain direction oriented in the direction of the length of the component.

Glulam is designed in stress grades L50, L40, L30 and L20. L marked glulam (Limträ) produced, tested and marked according to the regulations issued by Swedish glulam control is approved by Planverket.

Glued timber constructions that cannot be called glulam as defined above are accepted if they are manufactured to standards accepted by Planverket and have been tested in a manner directed by Planverket (see 'Glued timber construction. Production and testing'. Statens Planverk approval rules 1975 :6).

Glued timber constructions should be marked with glue class (I or U) and when glue nailing, glue stress (6 or 3) as well as the manufacturer's code, time code and the marking of Planverket.

Glue nailing and glue screwing is glueing where the required pressure is obtained by nails or screws respectively.

Glue joints are produced in stress grades 6 and 3, where the figures indicate the shear strength of the joint in N/mm² as defined in ASTM D905-49.

:413 K-Plywood (Structural plywood)

Structural plywood (K-plywood) is produced in stress grades P40, P30 and P20. U-glued plywood is acceptable to Planverket provided it is tested in a manner laid down by them (see 'Timber based board materials. Production and Testing'. Statens Planverk authorising rules 1975 :5). Structural plywood should be marked with stress grade, manufacturer's code and date. Plywood produced in any other manner should be judged by testing as per the methods indicated in the above authorising rules.

:414 K-board (Structural fibreboard)

Structural fibreboard (K-board) is stress graded 50 (oil tempered), 35 (hard) and 13 (medium board). A board is accepted that is manufactured and tested in a manner laid down by Planverket (see Planverket's authorising rules 1975 :5). Structural fibreboard should carry Planverket's test marking, quality and manufacturer's marking as well as date.

:415 Structural chipboard

Structural chipboard (K-chipboard) is accepted provided it complies with the requirements for Class 1 in SIS 23 48 01 and flooring chipboards as described below. Structural chipboard should be tested in a manner accepted by Planverket and marked with K- and SIS-marking, SIS number, class, thickness, manufacturing code and date (see Planverket's approval rules). Chipboard for flooring must comply with the requirements laid down by Planverket (see Planverket's approval rules 1975 :5). Flooring chipboard must be tested in a manner prescribed by Planverket and marked 'floor', Planverket's test mark, thickness, manufacturing code and date.

:42 Connectors

Laterally loaded wire nails are accepted if the tensile strength is not less than 40 (20-d) N/mm², where d is the nail's diameter in mm.

Bolted joints in stress grade 4.6 as per SMS 2265 are permissible and nuts in stress grade 4 as per SMS 2268.

Washers are permissible in a bolted joint without connectors if the washer dimension (diameter or length of edge) is at least 3d and the thickness 0.3d where d is the bolt diameter. For a bolted joint with connectors the size of the connectors must be at least 4d and the thickness at least 0.4d.

Wood screws as per SMS R 1573-1575 and SMS 1576 are permissible.

Metal plate gussets, connectors and other types of mechanical timber connectors are judged by Planverket at the same time as general approval where testing is carried out to "Design by Strength Testing" Statens Planverk approval rules 1975 :4.

Glues to be used in glulam are approved by the Swedish Glulam Control and other glued constructions are approved by the Swedish Forest Products Research Laboratory.

:5 Workmanship

The Instructions a) - e) below must be followed if one designs using the permissible stresses in :2.

a) Regarding workability of nail plates see 'Nailplates' Statens Planverk approval rules No. 4 (1974).

b) Timber may not have wane where it is jointed to other timbers, or loose knots or similar defects to such an extent that the joint's strength is likely to be impaired.

Timber that splits on nailing for example must be rejected. Timber for constructions in Climate Classes 1 and 2 should not be jointed with mechanical fasteners if its moisture content is in excess of 22%.

c) Nails should be driven at right angles to the grain unless otherwise stated on the drawings. Nails should be hammered home until the nail head is level with the timber surface.

Where angled nailing is mentioned on the drawings nailing as per Fig. 27 :2223b is required.

20% deviation from the minimum distance indicated in the design is permissible, providing that the total number of nails per stated joint surface is not increased. If no indication of nail distance is shown on the design :2221 should be followed.

d) Bolt holes are made so that the bolt is a tight fit with the timber. The drill diameter should not exceed the bolt diameter.

Tightening of bolts should be carried out when the timber is dry. If bolt positions, dimensions of connectors etc. are not indicated on the drawing, :224 and :225 should be followed.

e) Holes for wood screws should be drilled so that there is a tight fit between the shank and the hole, and for the threaded portion the diameter of the drill should be 0.8 - 0.9 times the diameter of the root of the thread.

Hammering of wood screws is not permissible. If the screw positions are not indicated on the drawing :226 is to be followed.