EHLBECK

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB - W18

BORDEAUX, FRANCE

OCTOBER 1979

12. Sitzung

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1 LIST OF DELEGATES

AUSTRIA

E Armbruster	European Federation of Building Joinery Manufacturers, Wien

BELGIUM

L Montfort	Institute National du Logement, Brussels

CANADA

J D Barrett	Forintek Canada Corporation, Vancouver
C R Wilson	Council of Forest Industries, Vancouver

DENMARK

A Egerup	Technical University of Denmark, Lyngby
H J Larsen	Aalborg University Centre, Aalborg

EIRE

P R Colclough	Institute for Industrial Research and Standards, Dublin

FEDERAL REPUBLIC OF GERMANY

J Ehlbeck	Universitat Karlsruhe, Karlsruhe
H Kolb	Otto-Graf Institut, Stuttgart
K Möhler	Universitat Karlsruhe, Karlsruhe

FINLAND

U Saarelainen	Technical Research Centre of Finland, Helsinki

FRANCE

P E H Crubilé	Centre Technique du Bois, Paris

NETHERLANDS

J Kuipers	Stevin Laboratory, Delft
T A Van der Put	Stevin Laboratory, Delft

NORWAY

N I Bovim	Norsk Treteknisk Institutt, Oslo

POLAND

	W Marosz	Union of Building Joinery Industry, Warsaw
	W Nozynski	Centralny Osrodek Badowczo, Laskowa

SWEDEN

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

UNITED KINGDOM

	L G Booth	Imperial College of Science and Technology, London
	H J Burgess	Timber Research and Development Association, High Wycombe
	W W Chan	Consultant Engineer, Wembley
	W T Curry	Building Research Establishment, Princes Risborough
	R Marsh	C H Mitchell (Homes) Ltd, Epsom
(1)	J G Sunley	Timber Research and Development Association, High Wycombe
(2)	J R Tory	Building Research Establishment, Princes Risborough

INTERNATIONAL STANDARDS ORGANISATION

	Mrs A Sørensen	Secretariat ISO/TC 165, Denmark

(1) Co-ordinator and Chairman
(2) Technical Secretary

2 CHAIRMAN'S INTRODUCTION

Delegates were welcomed to the meeting by the Chairman and co-ordinator of CIB-W18, MR SUNLEY. The agenda was circulated and agreed.

3 CO-OPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: MRS SØRENSEN circulated paper CIB-W18/12-103-1 which contained the resolutions and other papers from the ISO/TC 165 meeting held in Ottawa, Canada in September 1979.

Resolution 10: PROF LARSEN agreed with DR KUIPERS that it would not be possible to add annexes to an international standard. The general joints testing would form one standard with others for particular forms of joints.

Resolution 11: PROF LARSEN reported that TC 139 were willing to produce a standard for testing structural plywood and he would propose a joint TC 139/ TC 165 working group for this work. He agreed with a proposal from DR BOOTH that TC 139 should be asked to circulate the RILEM testing standard as a first draft for comment.

DR KUIPERS said that he did not agree with the letter from Professor Noack, that stated that a purpose of the tests was to provide comparison with the results from small clear specimens.

Resolution 13: DR EHLBECK said that he would like to see British participation in the working group on timber classification and that the working group would not necessarily adopt the Australian system.

PROF LARSEN, answering a question from DR KUIPERS, said that although it was important to have a strength class system for timber, stresses for individual species would still be permitted.

Resolution 14: TC 165 would like to have a final draft of the CIB Structural Timber Design Code for circulation as a working paper before the end of 1980. This document would not necessarily have to be complete.

RILEM 3-TT: DR KUIPERS reported that in accordance with RILEM policy the 3-TT sub-group should be disbanded. However he is to propose to RILEM that the group be re-formed to draft standards for the testing of structural board materials. At the meeting immediately preceding the W18 meeting it had been agreed that Mr I D G Lee of the United Kingdom should produce a draft standard for the testing of particle board. It was the intention of 3-TT that this, and standards for other sheet materials such as hardboard and composite boards should bear a close similarity to the standard for plywood.

PROF LARSEN suggested that perhaps there should be one general testing standard for all sheet materials with annexes for tests applicable only to individual materials. There might also be a need for an ISO technical committee on structural wood-based sheet materials.

MR KUIPERS also drew the attention of delegates to the need for standards on prototype testing, structural testing and in-situ testing. He also stressed the need for standards on sampling and analysis to support the work that had already been completed by RILEM 3-TT.

It was agreed that the plywood sub-group of DR NOREN, DR BOOTH, DR KUIPERS and DR WILSON should be encouraged to continue its work on sampling and analysis. The formation of other sub-groups was deferred until later in the meeting.

EEC: MR SUNLEY told delegates that at a joint CEI-Bois/CIB-W18 meeting in June it had been agreed that the two organisations should produce a joint submission to EEC for Eurocode 5. The first formal meeting to discuss the drafting of the code would take place in Bruxelles on 29 October 1979.

PROF MONTFORT said that some chapters of Eurocode 1 had now been drafted. Eurocode 5 should follow the safety methods adopted for Eurocode 1 and be based on the CIB Code. However he pointed out that there could be difficulties in drafting Eurocode 5 since Sous-commission GLULAM did not accept the probabilistic design philosophy.

DR ARMBRUSTER said that there was a need for a simple code and this should permit deterministic design for an interim period before limit state design was introduced.

PROF LARSEN told members that the decision had been made that the Eurocodes should adopt the limit state design system. This would allow governments to introduce their own levels of safety and neither CIB-W18 or GLULAM could produce a code which defined safety levels. The first priority within EEC should be the harmonization of loading rules, continued PROF LARSEN.
He asked Dr Armbruster to explain to Sous-commission GLULAM that the fourth draft of the CIB Code, with the exception of Chapter 3, was equally applicable to deterministic or limit state design.

IUFRO S5.02: DR BARRETT told delegates of the long-term loading projects being undertaken by the University of British Columbia; Forest Products Laboratory, Madison and the Forintek Canada Corporation, Vancouver. He expressed the need for international participation and suggested that an international co-ordinating committee should be set up to provide comment before the start of the proposed test programme.

MR SUNLEY asked delegates to consider how they might contribute and to be prepared to discuss the project in detail at the next IUFRO meeting in Oxford, England 8-16 April 1980.

4 GLULAM STANDARD

DR ARMBRUSTER presented paper CIB-W18/12-12-1 'Glulam Standard Part 2: Glued Timber Structures: Rating (3rd draft)' and paper CIB-W18/12-12-2 'Glulam Standard Part 3: Glued Timber Structures: Performance (3rd draft)', explaining that the content of these standards was based on practical experience and not solely on theory.

PROF LARSEN did not accept that the content of the three parts of the GLULAM standard was appropriate to the drafting of the CIB Code. He agreed that sub-commission GLULAM should produce a production standard but this should not include stresses and could not form part of a design code.

It was agreed that these two papers should be considered as comments on the fourth draft of the CIB Code and further discussed at the next meeting.

5 CIB STRUCTURAL TIMBER DESIGN CODE

The following comments and amendments were made to paper CIB W18/12-100-3 'CIB Structural Timber Design Code (fourth draft)'.

Clause 2.1.1: The reference depth for glued laminated timber is to be retained as 300 mm. DR ARMBRUSTER had requested 500 mm.
Insert 'population' between 'as' and 'lower'.
Delete the words in brackets.
Small print text to be inserted to the effect that recommendations on board materials will be produced.,

Clause 2.2: Climate classes are to be changed to moisture classes and an example of the translation of moisture content to temperature and humidity will be provided.

Table 2.3a: Replace 'normal' by 'medium'.

Chapter 3: To be reduced to small print.
Structural or prototype testing is to be included as a permitted design method.
Reference to performance in fire is required.

Table 4.1.1: The first two lines below this table are to be in normal print.
A note is to be inserted that poles may be graded to the same strength classes.

Clause 4.7: This clause is to be revised to take account of International Standards and to permit other methods of protection.

Table 5.1.0a: Strength class SC 38 to be inserted.

Clause 5.1.1.0: Note required to avoid the inadvertent use of characteristic stresses in the equations.

Clause 5.1.1.6: DR BARRETT is to produce a proposal on shear and torsional stresses.

Clause 5.2.2: Replace 'Cambered' by 'Pitched and tapered'.

Clause 5.2.3: A note will be provided on the facing page about tapered curved beams.

Chapter 6.0: DR KUIPERS is to provide a clause on calculating the deformation at joints.

Clause 6.1.1.1: MR VAN DER PUT is to produce a paper recommending changes to this clause.

Fig 6.1.1.1b: End distances are to be increased to 15d.

Table 6.1.1.2: The values for round nails are to be reconsidered.

Clause 6.1.1.3: PROF LARSEN will redraft this clause.

Clause 6.1.2: Formula 6.1.2d is to be deleted.
The loading for dowels is to be increased.

Formula 6.1.3.2a: To be simplified.

Clause 6.1.4: To be revised.

Formula 7.1.1f: An alternative recommendation is to be produced from Australia.

Annexes: DR BOOTH and DR FOSCHI are to produce a joint paper on the design of thin-flanged beams.

PROF LARSEN undertook to consider and answer the comments received from Sous-commission GLULAM (paper CIB W18/12-100-1).

The secretary drew attention to paper CIB W18/12-100-2 'Comment on the CIB Code'.

6 STRENGTH CLASSES

MR TORY introduced 'Strength Classes for British Standard BS 5268' (Paper CIB-W18/12-6-2), explaining that the proposed revision of the British Code of Practice CP 112 into BS 5268 included the strength classes given in this paper.

PROF LARSEN said that the UK had spoilt the strength class system proposed for the CIB Code by bending the principle to fit commercially important grades.

MR SUNLEY replied that there was no point in introducing to any national code a strength class system which would not be used. It would be easy to have a class system for European timber and another for, say, North American use; but it would be difficult to provide a common system since each would be reluctant to accept the penalties involved.

PROF MONTFORT said that the basic requirement for international agreement was material classification. MR CURRY agreed, saying that grading would be the common factor, not stresses. MR CURRY saw the strength class system as an introduction to the decline of visual grading and to the optimisation in use of timber by the increasing use of machine grading. He pointed out how closely the British visual grades were to the ECE grades and offered to produce a paper for the next meeting comparing the UK system to that given in the CIB Code.

7 FIRE

MR SUNLEY introduced paper CIB W18/12-15-1 'British Standard BS 5268:Part 4: Fire Resistance of Timber Structures'.

It was agreed that a draft section on fire performance should be prepared for the next meeting, based on this British Standard.

8 TRUSSED RAFTERS

Paper CIB W18/12-14-1 'A Simple Design Method for Standard Trusses' was introduced by DR EGERUP. He said that he was trying to develop a design system for trusses that would permit any country adopting the system to insert it s own coefficients. Ideally it should take into account joint slip, plasticity and non-linearity.

DR CHAN offered to contribute to the work on the design method for trussed rafters.

9 JOINTS

DR EHLBECK briefly spoke on his paper 'Load-carrying Capacity and Deformation Characteristics of Nailed Joints' (CIB W18/12-7-1). He regarded the paper as an interim report identifying the parameters influencing joint performance. He is to produce rules for the CIB Code to permit the calculation of joint deformations.

PROF LARSEN presented paper CIB W18/12-7-2 'Design of Bolted Joints'. The purpose of the paper, he said was to explain how the rules contained in the CIB Code had been derived. Before the next meeting Prof Larsen hopes to carry out some test work to compare with the theoretical approach.

MR CURRY said that both this and the previous paper were very useful.theoretical approaches to the problems of joints but they showed how few test results were available to verify the theory.

Agreeing with Mr Curry, DR BOOTH suggested that part of the problem was that joint geometry was so varied. To adequately cover the subject there was a need for a co-ordinated program of joint testing.

Paper CIB W18/12-7-3 'Design of Joints with Nail Plates' was circulated by DR NOREN. He pointed out it was a joint proposal from the Nordic countries and if accepted would limit nail plate testing to the principal directions only. The Johansen and Bovim theory would permit the translation of these limited tests into the required strengths at angles to the principal axes.

DR KUIPERS drew attention to the conflicting test requirements of this paper and the RILEM test method for nail plates. He suggested that the RILEM method should be published but that if the Nordic proposals were accepted it could be necessary to amend the RILEM method at a later date.

MR BOVIM and DR NOREN asked for these proposals to be discussed in greater depth at the next meeting. They will then provide specific recommendations for the CIB code.

10 PLYWOOD

DR BOOTH reported that the plywood sub-group (Barrett, Booth, Ehlbeck, Norén and Wilson) had met to discuss the derivation of characteristic values, sampling and paper CIB-W18/12-4-1 'Procedures for Analysis of Plywood Test Data and Determination of Characteristic Values Suitable for Code Presentation'.

It had been agreed that Dr Norén and Dr Wilson should produce a further paper on sampling and analysis and the sub-group would meet again early in 1980. The group would require guidance on statistical methods to derive fifth percentiles with 75 per cent confidence limits, said Dr Booth; and they would also have to decide whether to recommend one particular method or to give a choice.

PROF LARSEN said that no method of analysis should be given since the appropriate distributional assumption could be different for each property and might vary with grading. Some element of judgement should be permitted.

MR SUNLEY considered it important that the sampling and analytical papers to which the Code would refer should contain some advice on which parametric or non-parametric methods were appropriate.

MR CURRY agreed with Mr Sunley, saying that some degree of standardisation was required.

PROF LARSEN, commenting on paper CIB W18/12-4-1, said that evaluation and presentation were being confused. He would have preferred the presentation to be in the form of load-carrying capacity rather than stresses and geometrical properties.

DR WILSON agreed that the presentation of load-carrying capacity could simplify codes but there were occasions when it was necessary to analyse data in greater depth.

11 OTHER BUSINESS

DR KUIPERS told delegates that two RILEM sub-groups were to be formed to consider structural testing and sheet materials. The membership of these two sub-groups was agreed as follows:

Structural Testing: Kuipers (chairman), Chan, Crubilé, Curry, Norén,
Nozynski (+ 1 German)

Sheet Materials: Kuipers (chairman), Booth, Chan, Curry, Lee, Mørkved
(+ 1 German)

MR SUNLEY closed the meeting and thanked MR CRUBILE and CHAMBRE DE COMMERCE ET D'INDUSTRIE DE BORDEAUX for their hospitality and for the facilities that had been made available to CIB W18.

12 NEXT MEETING

The next meeting of CIB W18 will take place during the week of 2-7 June 1980 in Otainiemi, Finland. The programme for that week is as follows:

Monday 2 June: RILEM 3-TT (members only;
chairman, Dr Kuipers)

Tuesday 3 June, morning: Trussed rafter sub-group
(chairman, Dr Egerup)

afternoon: plywood sub-group
Wednesday 4 June : CIB W18
Thursday 5 June : CIB W18
Friday 6 June : CIB W18, technical excursion
Saturday 7 June, morning: CIB W18/CEI-Bois discussion on Eurocode 5

The meeting-after-next has tentatively been arranged for Warsaw, Poland in May 1981.

13 PAPERS PRESENTED AT THE MEETING

CIB W18/12-4-1	Procedures for Analysis of Plywood Test Data and Determination of Characteristic Values Suitable for Code Presentation - C R Wilson.
CIB W18/12-6-1	Strength Classifications for Timber Engineering Codes - R H Leicester and W G Keating.
CIB W18/12-6-2	Strength Classes for British Standard BS 5268 - J R Tory
CIB W18/12-7-1	Load-carrying Capacity and Deformation Characteristics of Nailed Joints - J Ehlbeck
CIB W18/12-7-2	Design of Bolted Joints - H J Larsen
CIB W18/12-7-3	Design of Joints with Nail Plates - B Noren
CIB W18/12-12-1	Glulam Standard Part 2: Glued Timber Structures; Rating (3rd draft)
CIB W18/12-12-2	Glulam Standard Part 3: Glued Timber Structures; Performance (3rd draft)
CIB W18/12-14-1	A Simple Design Method for Standard Trusses - A R Egerup
CIB W18/12-100-1	Comment on the CIB Code - Sous-Commission 'GLULAM'
CIB W18/12-100-2	Comment on the CIB Code - R H Leicester
CIB W18/12-100-3	CIB Structural Timber Design Code (Fourth Draft)
CIB W18/12-103-1	ISO/TC 165: Ottawa, September 1979
CIB W18/12-16-1	British Standard BS 5268:Part 4: Fire Resistance of Timber Structures.

CURRENT LIST OF CIB-W18 TECHNICAL PAPERS

79.11.20

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
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979

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
16 Fire
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

11-1-1 Safety Design of Timber Structures - H J Larsen

TIMBER COLUMNS

SYMBOLS

PLYWOOD

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

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

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

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

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

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

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

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

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

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

STRESS GRADING

1-5-1 Paper 10 Quality specifications for sawn timber and precision timber - Norweigan 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

11-6-1 Evaluation of Lumber Properties in the United States - W L Galligan and J H Haskell

11-6-2 Stresses Perpendicular to Grain - K Möhler

11-6-3 Consideration of Combined Stresses for Lumber and Glued-laminated Timber (addition to Paper CIB-W18/9-6-4)

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

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

TIMBER JOINTS AND FASTENERS

LOAD SHARING

DURATION OF LOAD

TIMBER BEAMS

ENVIRONMENTAL CONDITIONS

5-11-1 Climate Grading for the Code of Practice - B Norén

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

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

LAMINATED MEMBERS

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

8-12-1 Testing of Big Glulam Timber Beams - H Kolb and P Frech

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

11-6-3 Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4) - K Möhler

12-12-1 Glulam Standard Part 2: Glued Timber Structures; Rating (3rd draft)

12-12-2 Glulam Standard Part 3: Glued Timber Structures; Performance (3rd draft)

PARTICLE AND FIBRE BUILDING BOARDS

7-13-1 Fibre Building Boards for CIB Timber Code - O Brynildsen

9-13-1 Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto and J Ehlbeck

9-13-2 The Structural Use of Tempered Hardboard - W W L Chan

11-13-1 Tests on Laminated Beams from Hardboard under Short- and Long-term Load - W Nozynski

11-13-2 Determination of Deformation of Special Densified Hardboard Under Long-term Load and Varying Temperature and Humidity Conditions - W Halfar

11-13-3 Determination of Deformation of Hardboard under Long-term Load in Changing Climate - W Halfar

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

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

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

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

10-14-1 Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon

11-14-1 Design of Metal Plate Connected Wood Trusses - A R Egerup

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

FIRE

12-16-1 British Standard BS 5268 The Structural Use of Timber: Part 4 Fire Resistance of Timber Structures

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)

11-100-1 CIB Structural Timber Design Code (Third Draft)

11-100-2 Comments Received on the CIB Code

11-100-3 CIB Structural Timber Design Code: Chapter 3

12-100-1 Comment on the CIB Code - Sous Commission Glulam

12-100-2 Comment on the CIB Code - R H Leicester

12-100-3 CIB Structural Timber Design Code (Fourth 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

11-102-1 Eurocodes - H J Larsen

INTERNATIONAL STANDARDS ORGANISATION

JOINT COMMITTEE ON STRUCTURAL SAFETY

CIB PROGRAMME, POLICY AND MEETINGS

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

CIB-W18/12-4-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PROCEDURES FOR ANALYSIS OF PLYWOOD TEST DATA AND DETERMINATION OF CHARACTERISTIC VALUES SUITABLE FOR CODE PRESENTATION

by

C R Wilson

Council of Forest Industries of British Columbia
CANADA

BORDEAUX, FRANCE

OCTOBER 1979

PROCEDURES FOR ANALYSIS OF PLYWOOD TEST DATA

AND

DETERMINATION OF CHARACTERISTIC VALUES

SUITABLE FOR CODE PRESENTATION

1. Background

In recent years the CIB W18 "Timber Structures" Committee has been working towards the development of a timber design code to provide a basis for the international use in the design of timber structures. As plywood is perhaps the most common internationally traded engineered wood based structural panel product, it was identified that there was an early need for internationally accepted sampling, testing and analysis procedures which would permit development of consistent design information for different plywoods.

Work is currently underway to develop an appropriate sampling document and a joint RILEM/CIB 3TT committee has developed a plywood testing document titled, "Testing Methods for Plywood in Structural Grades for Use in Load-bearing Structures."

This paper presents a proposal for CIB W18 consideration on

- the analysis of plywood test data resulting in four forms of presentation,

- the manner in which data from a limited number of constructions may be used to provide data for constructions (which have similar performance characteristics) not tested, and

- the adjustment of data from one form to another.

Though the procedures outlined below are applicable to any plywood test data, the illustration examples specifically reflect the RILEM/CIB 3TT test document.

2. Introduction

Current engineering practices present plywood engineering design properties at a preselected characteristic level (usually five percent exclusion limit) which is either at the ultimate level or at the working (permissible) level. These characteristic values are generally modified for load duration and for the working level are modified with a factor of safety.

At either the ultimate or working level, plywood data can be presented in design codes using a form of presentation based on

- carrying capacity,
- full cross section,
- effective cross section, or
- theoretical cross section.

For the most part, codes present the design data using one of the first three forms of presentation whereas researchers may use the theoretical cross section form where the need is to more accurately predict the actual rather than the apparent strength of panels.

This paper concentrates on the first three forms of presentation. The last form is only presented in an attempt to ensure compatibility of approach to the analysis of plywood data. It is not recommended that this last approach be used in the presentation of data suitable for design purposes.

3. Determination of Geometrical Properties

3.1 Carrying Capacity

If this form of presentation is to be used, no further analysis of test data is required once the characteristic capacity value (ultimate or working) has been determined.

Carrying capacity data can only be directly used in a code providing each construction and thickness has been tested. (In this regard, it is important to recognise that the construction giving the lowest test capacity in both the parallel and perpendicular direction must be tested.) Therefore to economically and realistically obtain design data information for a range of constructions not tested, the plywood test data must be further analysed using the effective cross section or theoretical cross section methods. Providing this additional analysis is undertaken, the following sections in this paper will suggest a way in which load carrying capacity data can be determined for constructions (which have similar performance characteristics) not tested.

In design codes, carrying capacity information is listed separately for each construction (or "minimum" construction) of each thickness. Therefore simple straightforward designs can be developed using the code information directly. If required, however, more sophisticated analysis cannot be undertaken by the designer unless additional lay-up information is also included in the code.

3.2 Full Cross Section

The full cross section method of analysis assumes a homogeneous cross section for both the parallel (0°) and perpendicular (90°) to face grain orientation. Therefore the overall panel thickness and the width of the plywood specimen are required to determine the geometrical properties. Appendix A indicates the areas used to determine the geometrical properties and presents an example calculation.

The limitations of direct use in codes of full cross section data are the same as those identified for direct use of carrying capacity data. Hence, once again, test data must be further analysed using the effective cross section or theoretical cross section methods.

In design codes, full cross section information requires listing of separate stress and moduli information for each construction (or "minimum" construction) of each thickness and geometrical properties of each thickness. Again, straightforward designs can be developed directly but information is not available for more sophisticated analysis.

3.3 Effective Cross Section

The effective cross section method of analysis (often referred to as parallel ply theory) assumes that only the plies with grain parallel to the principal stress contribute to the load carrying capacity and stiffness of the plywood. Appendix A indicates the areas used to determine the geometrical properties and presents an example calculation.

Some discussion related to the simplified assumption that the plies perpendicular to the principal stress do not contribute to the overall carrying capacity of the panel is warranted. The effective cross section method does provide a relatively straightforward way to determine the extreme fibre stress in the outermost load carrying ply. Hence, the method is particularly effective in permitting use of the data to make adjustments for constructions not tested. However, as the plies perpendicular to the direction of applied principal stress are assumed to carry no load, the method does not describe adequately some plywood constructions, particularly three ply plywoods.

In examining the figure of Appendix A dealing with effective cross section, attention should be given to the distance from the neutral axis to extreme fibre dimension, particularly for the 90° face grain orientation. To account for the fact that the outer ply does in fact carry some load and to bring the flexure data to an overall common bais, practice in North America has been to modify the flexure test data by employing an experimentally verified flexure factor. This flexure factor is used in order to account for differences in apparent values which result from ignoring plies perpendicular to face grain and to provide a common value for these properties. (At a later stage in this paper, an alternative to the use of a flexure factor is proposed.)

From the above it can be seen that the effective cross section approach permits an analysis of the data which provides a means to evaluate the stress level in the outermost load carrying plies. The value so obtained when combined with the appropriate geometrical property permits the calculation of load carrying capacity of the construction. In addition, providing that different constructions perform similarly to the one tested and that these different constructions conform to the same product standard, the procedure permits the realistic calculation of load carrying capacity of constructions and thicknesses of panels not tested.

Simple straightforward and sophisticated analysis can be done by the designer furnished with a limited number of common stress and moduli information and the appropriate (actual or "minimum") geometrical property of each thickness. Thus the effective cross section method simplifies the data required for inclusion in a design code.

3.4 Theoretical Cross Section

The theoretical cross section method of analysis assumes that the plies with grain parallel and perpendicular to the principal stress each contribute to the load carrying capacity and stiffness of the plywood. As the moduli of elasticity differ for the plies having face grain parallel to the principal stress as compared to the plies having face grain perpendicular to the principal stress, the theoretical cross section method is calculated in such a way so as to transform the cross section into an equivalent cross section of a single apparent modulus of elasticity. Appendix A indicates the areas used to determine the geometrical properties and presents an example calculation.

With the theoretical cross section method of analysis, the calculated extreme fibre stresses and moduli reflect the manner in which the forces are distributed in the test panel to a greater degree of accuracy than the effective cross section method. As a result, this method permits the researcher to more accurately predict the strength and stiffness of multi species and multi lay-up constructions. However, for codification (classification) purposes, the method introduces a degree of sophistication and complexity not generally required by the designer and hence will not be further discussed in this paper.

4. Representative Panel Test Results

The overriding principle of the RILEM/CIB 3TT test procedures is to provide a common basis on which to determine plywood mechanical properties. Providing the same sampling techniques are used and a consistent analysis of the test data is applied, then the resulting properties reflect in a compatible way the strength and stiffness of the plywood tested, provides strength and stiffness information of different plywoods relative to each other, and give the same degree of reliability in the design.

The RILEM/CIB 3TT test document outlines two different thickness measurement procedures. If the results are to be interpreted using the effective or theoretical cross section method, the individual ply thickness of each test piece is to be recorded. If, on the other hand, the way in which the results are to be interpreted is not known at the time of test, the total thickness of each test piece is to be recorded. In addition, for at least five test pieces from each panel, the thickness of each ply is to be recorded. This latter requirement has been included so that any data developed can be universally analysed at some future date.

Where the RILEM/CIB 3TT test standard requires two or more test pieces (for a given property) be obtained from a single panel, it is recommended that a single representative property be calculated for each panel prior to conducting an overall statistical analysis of the results.

(Note: At the time of writing, it was unclear which cutting schedule will be included in the final test procedure document. If two or more test pieces for a given test are obtained from a panel then the average of the test results reflects the representative panel property. If only one test piece for a given test is obtained from a panel then the result reflects the representative panel property.)

To illustrate, let us examine the bending strength case using the full cross section method and then the case where the effective cross section method is used.

For the full cross section method, the extreme fibre bending stress of each test piece is determined by dividing the maximum moment by the calculated section modulus using the overall measured thickness of the test piece. This paper recommends that the representative panel bending stress of the panel be then determined by averaging the individual bending stresses of the test pieces obtained from one panel.

If at a later date the data is to be re-interpreted using the effective and/or theoretical cross section methods, the extreme fibre bending stress of each test piece is determined by dividing its maximum moment by the average section modulus derived using the individual ply thicknesses of five test pieces. The panel representative bending stress is then determined by averaging the individual bending stresses of the test pieces obtained from one panel.

For the effective or theoretical cross section method, the extreme fibre bending stress of each test piece is determined by dividing its maximum moment by the calculated section modulus derived using the individual ply thicknesses of that test piece. The representative bending stress of the panel is then determined by averaging the individual bending stresses of the test pieces obtained from one panel.

Appendix B provides an example of how the test data and the physical thickness measurements are used to determine representative properties of a panel.

5. Representative Plywood Test Results

Once the individual representative panel properties are determined, representative characteristic plywood property values can be calculated using acceptable statistical techniques. Normal Gaussian statistics are generally acceptable and hence the following expressions should be used.

Mean $$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

Standard Deveiation $$s = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n - 1}}$$

Coefficient of Variation $$cv = \frac{s}{\bar{x}}$$

6. Representative Plywood Properties

6.1 Introduction

If all constructions have been tested then the representative plywood property values can be used directly for code purposes (modified as required for load duration and factor of safety). However, as many product standards permit a range of constructions and ply thicknesses for a given panel thickness, testing all constructions may prove impractical and the test data obtained from the test program must be in a form applicable to constructions not tested directly. Therefore some recommendations are required in order to provide for a grouping of plywood property values.

6.2 Grouping of Plywood Data

To determine plywood properties for a range of constructions and thicknesses, first, it is necessary to conduct some preliminary tests in order to determine what constructions perform similarly. The main test program can then be devised so as to ensure that property data will not be extrapolated to constructions that basically perform differently, e.g. three ply perpendicular to face grain results may be significantly different to five and more ply perpendicular to face grain results. Providing these differences are identified, the test program may be developed in such a way so as to reflect the common performance of each specific type. Note that by developing data in this way, it is not necessary to incorporate the flexure factor in any analysis of flexure data.

To determine a characteristic level of stresses (or moduli) common to several constructions, it is recommended that the preselected exclusion limit value be calculated for each construction tested and the same exclusion limit value be calculated for the group of constructions. If this grouped exclusion limit value does not exceed the exclusion limit value of the poorest construction by more than 10%, then the grouped exclusion limit value may be used for code inclusion purposes. If the exclusion limit value of the weakest construction is less than 90% of the grouped exclusion limit value, then the lowest exclusion limit value controls or the plywood having the lowest value should be eliminated from the group and the grouping analysis be recalculated.

7. Plywood Properties for Inclusion in Design Codes

Effective (or theoretical) cross section representative plywood properties controlling exclusion limit values can be directly included in engineering design codes providing appropriate minimum geometrical properties are also included.

In determining these minimum geometrical properties, several factors must be considered. These are

1. panel thicknesses as specified in the product standard and as measured by the plywood quality control requirements,

2. how the quality control thickness measurements relate to the thickness of the panels as tested, and

3. minimized constructions as permitted by the product standard.

Product standards specify minimum panel thickness and quality control procedures check panel thickness to ensure conformance to the product standard. These quality control measurements are usually done at the time of manufacture when the panels have a moisture content in the range of 0.04 to 0.06. The panels as tested according to the RILEM/CIB 3TT test standard have a moisture content of approximately 0.15 which reflects the approximate end use dry service conditions. Therefore if test data is to be used to predict the strength (or moduli) of similar constructions and grades not included in the overall test program, then it is recommended that (in addition to the test standard requirements) the overall panel thickness be measured at the time when normal quality control thickness measurements are made. Using this information and the minimum thickness specified in the product standards (quality control), geometrical properties can then be determined for a range of constructions which would reflect the panel thickness at the time of test conditions.

It is therefore recommended that as a starting point the effective cross section properties be determined by adjusting the minimum thickness of the standard to reflect the difference in overall thickness between the time of quality control check and end use (time of test). Minimized geometrical properties can then be calculated using this latter thickness and the ply thicknesses permitted in the product standard for each face grain orientation.

It is recognized that the above does not provide for a change in individual ply thickness (due to moisture content changes). However if this was done it would unnecessarily introduce a complication which is not warranted from a practical point of view.

Appendix C illustrates the recommended procedure to determine minimized effective cross section geometrical properties.

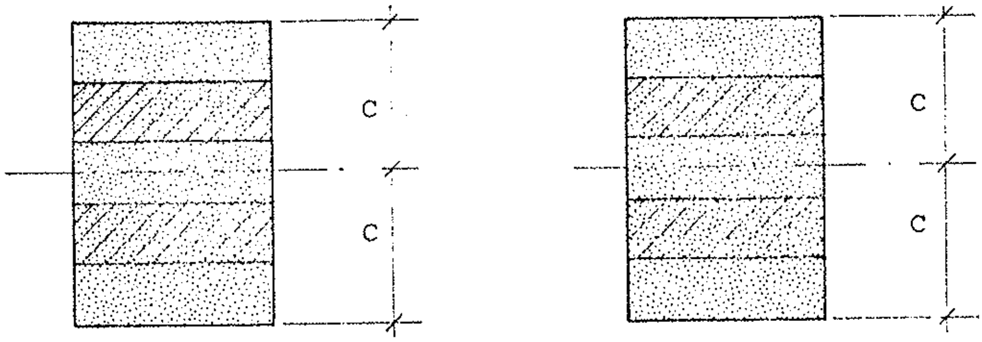
8. Adjustment of Plywood Properties to Any Form of Presentation

Providing data has been developed on the basis of effective cross section, then the information can be easily transformed to either the load carrying capacity or full cross section form of presentation for code purposes. This is done by determining the load carrying capacity by combining the effective cross section stresses (moduli) and related geometrical properties. Then, knowing the full cross section geometrical properties, the load carrying capacity information can be used to determine the full cross section stresses and moduli.

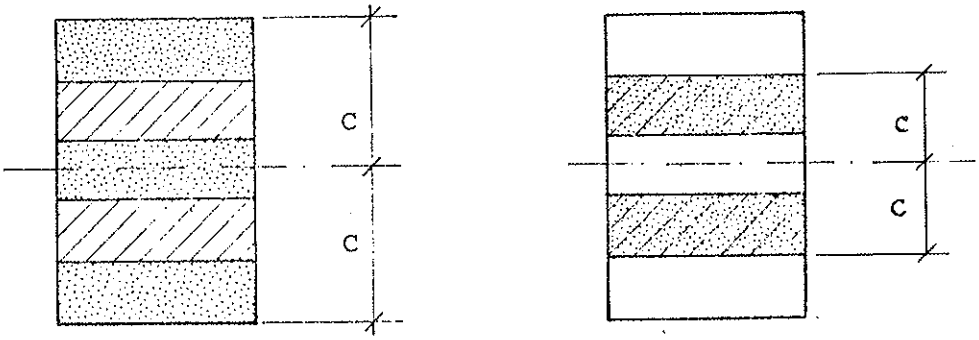
Appendix D illustrates the procedure of determining plywood properties for different forms of presentation.

AN EXAMPLE OF DETERMINING GEOMETRICAL PROPERTIES FOR PLYWOOD

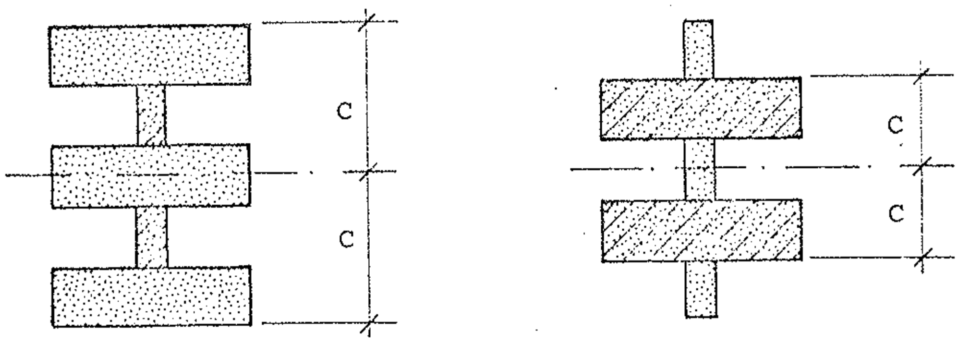
0° Face Grain Orientation | 90° Face Grain Orientation



Full Cross Section



Effective Cross Section

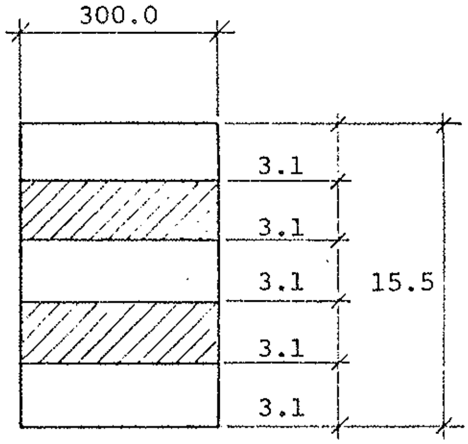


Theoretical Cross Section

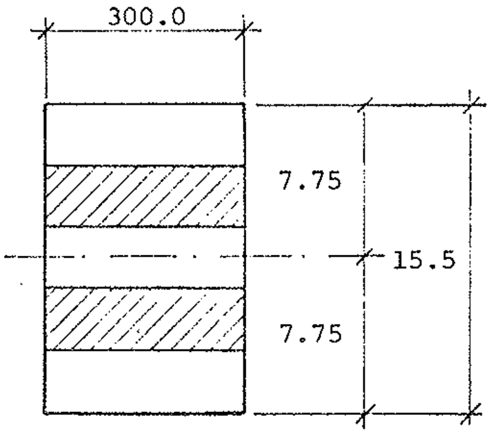
Area used to determine geometrical properties.

c Distance from neutral axis to extreme fibre.

Assumption



Full Cross Section Theory



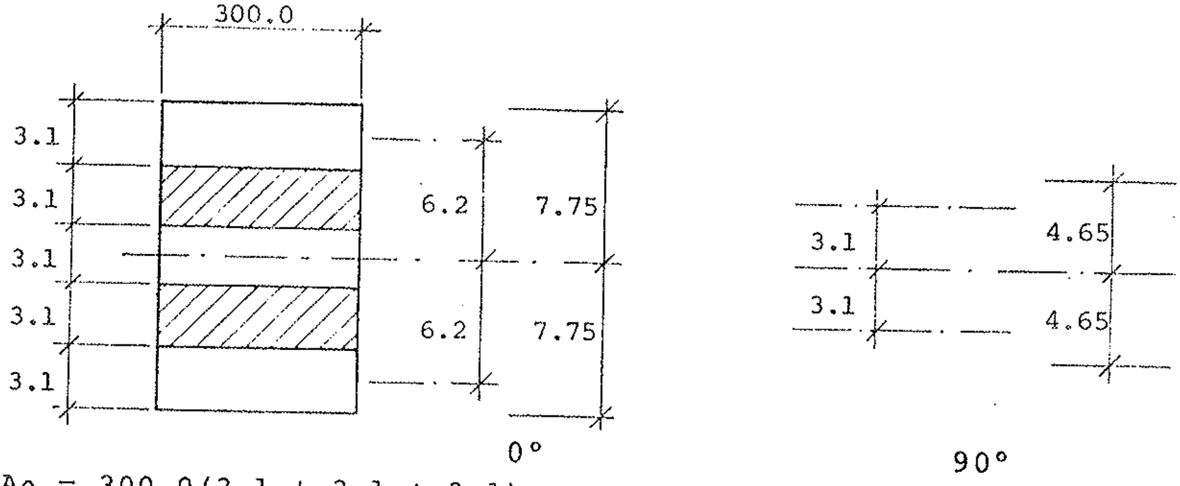
0° and 90°

$$A_0 = A_{90} = 300.0(15.5)$$
$$= 4{,}650 \text{ mm}^2$$

$$I_0 = I_{90} = \frac{300.0(15.5)^3}{12}$$
$$= 93{,}096.9 \text{ mm}^4$$

$$W_0 = W_{90} = \frac{93{,}096.9}{7.75}$$
$$= 12{,}012.5 \text{ mm}^3$$

Effective Cross Section Theory



$$A_0 = 300.0(3.1 + 3.1 + 3.1)$$
$$= 2,790.0 \text{ mm}^2$$

$$I_0 = \frac{300.0(3.1)^3}{12} + 300.0(3.1)(6.2)^2$$
$$+ \frac{300.0(3.1)^3}{12}$$
$$+ \frac{300.0(3.1)^3}{12} + 300.0(3.1)(6.2)^2$$
$$= 73,732.7 \text{ mm}^4$$

$$W_0 = \frac{73,732.7}{7.75}$$
$$= 9,513.9 \text{ mm}^3$$

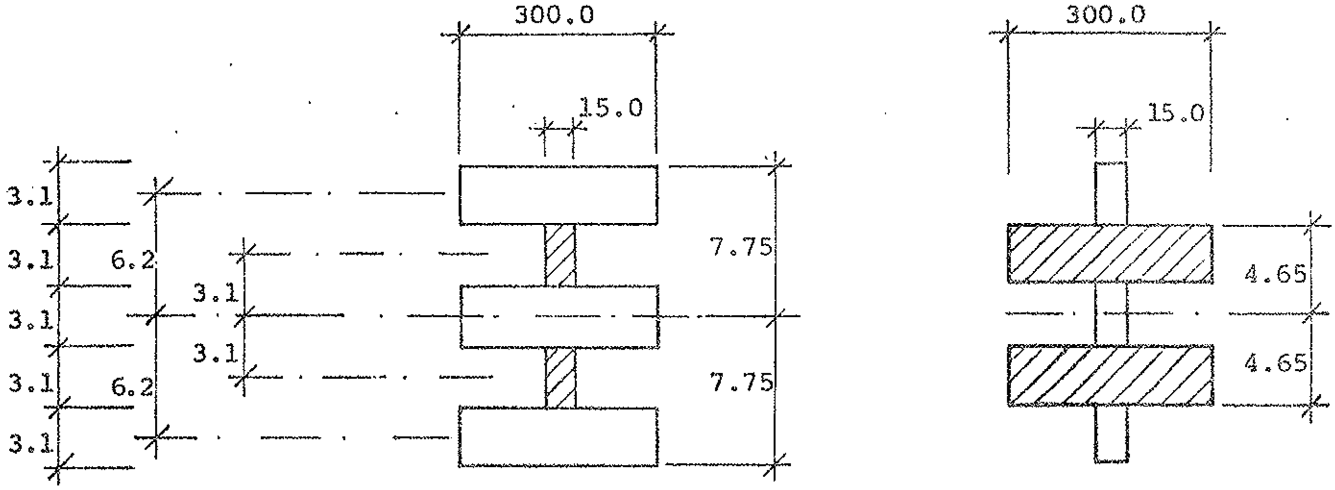
$$A_{90} = 300.0(3.1 + 3.1)$$
$$= 1,860.0 \text{ mm}^2$$

$$I_{90} = \frac{300.0(3.1)^3}{12} + 300.0(3.1)(3.1)^2$$
$$+ \frac{300.0(3.1)^3}{12} + 300.0(3.1)(3.1)^2$$
$$= 19,364.2 \text{ mm}^4$$

$$W_{90} = \frac{19,364.2}{4.65}$$
$$= 4,164.3 \text{ mm}^3$$

Theoretical Cross Section Theory

Assume $\frac{E_0}{E_{90}} = 20$



0°   90°

$$A_0 = 300.0(3.1) + 15.0(3.1) + 300.0(3.1) + 15.0(3.1) + 300(3.1)$$
$$= 2{,}883.0 \text{ mm}^2$$

$$I_0 = \frac{300.0(3.1)^3}{12} + 300.0(3.1)(6.2)^2$$
$$+ \frac{15.0(3.1)^3}{12} + 15.0(3.1)(3.1)^2$$
$$+ \frac{300.0(3.1)^3}{12}$$
$$+ \frac{15.0(3.1)^3}{12} + 15.0(3.1)(3.1)^2$$
$$+ \frac{300.0(3.1)^3}{12} + 300.0(3.1)(6.2)^2$$
$$= 74{,}700.9 \text{ mm}^4$$

$$W_0 = \frac{74{,}700.9}{7.75}$$
$$= 9{,}638.8 \text{ mm}^3$$

$$A_{90} = 15.0(3.1) + 300.0(3.1) + 15.0(3.1) + 300.0(3.1) + 15.0(3.1)$$
$$= 1{,}999.5 \text{ mm}^2$$

$$I_{90} = \frac{15.0(3.1)^3}{12} + 15.0(3.1)(6.2)^2$$
$$+ \frac{300.0(3.1)^3}{12} + 300.0(3.1)(3.1)^2$$
$$+ \frac{15.0(3.1)^3}{12}$$
$$+ \frac{300.0(3.1)^3}{12} + 300.0(3.1)(3.1)^2$$
$$+ \frac{15.0(3.1)^3}{12} + 15.0(3.1)(6.2)^2$$
$$= 23{,}050.8 \text{ mm}^4$$

$$W_{90} = \frac{23{,}050.8}{4.65}$$
$$= 4{,}957.2 \text{ mm}^3$$

AN EXAMPLE OF DETERMINING REPRESENTATIVE PANEL TEST RESULTS

Assumptions

- Bending strength tests conducted on two test pieces having the same face grain orientation from one panel.
- Ultimate moment capacity

  Test piece 1 = 415,800.0 N.mm

  Test piece 2 = 457,380.0 N.mm
- Test pieces have dimensional measurements similar to those identified in Appendix A.

Case I Full Cross Section

Prior to conducting tests, it is known that the results will be interpreted using the full cross section method. Therefore the overall thickness of each test piece has been recorded. In addition, to provide a means to evaluate the results at a later date using another method, the individual ply thickness of five test pieces from the panel have been also recorded.

Full Cross Section Evaluation

Assume

Test piece 1 $W_0 = 11,628.1\ mm^3$

Test piece 2 $W_0 = 12,403.1\ mm^3$

Evaluation of Test Pieces

$$f_b \text{ test piece 1} = \frac{415,800.0}{11,628.1} = 35.76\ N/mm^2$$

$$f_b \text{ test piece 2} = \frac{457,380.0}{12,403.1} = 36.88\ N/mm^2$$

Representative Panel Test Results

$$f_b = \frac{35.76 + 36.88}{2} = 36.32\ N/mm^2$$

Effective Cross Section Evaluation

Assume

Piece 1 $W_0 = 11{,}628.1\ mm^3$

Piece 2 $W_0 = 11{,}820.3\ mm^3$

Piece 3 $W_0 = 12{,}012.5\ mm^3$

Piece 4 $W_0 = 12{,}207.8\ mm^3$

Piece 5 $W_0 = 12{,}403.1\ mm^3$

Therefore the W_0 average of the five pieces is 12,014.36 mm^3.

Evaluation of Test Pieces

$$f_b \text{ test piece 1} = \frac{415{,}800.0}{12{,}014.36} = 34.61\ N/mm^2$$

$$f_b \text{ test piece 2} = \frac{457{,}380.0}{12{,}014.36} = 38.07\ N/mm^2$$

Representative Panel Test Results

$$f_b = \frac{34.61 + 38.07}{2} = 36.34\ N/mm^2$$

Case II Effective Cross Section

Prior to conducting tests, it is known that the results will be interpreted using the effective cross section method. Therefore the thickness of the plies of each test piece has been recorded.

Assume

Test piece 1 $W_0 = 9{,}209.5\ mm^3$

Test piece 2 $W_0 = 9{,}823.3\ mm^3$

Evaluation of Test Pieces

$$f_b \text{ test piece 1} = \frac{415{,}800.0}{9{,}209.5} = 45.15\ N/mm^2$$

$$f_b \text{ test piece 2} = \frac{457{,}380.0}{9{,}823.3} = 46.56\ N/mm^2$$

Representative Panel Test Results

$$f_b = \frac{45.15 + 46.56}{2} = 45.86\ N/mm^2$$

AN EXAMPLE OF DETERMINING MINIMIZED EFFECTIVE CROSS SECTION GEOMETRICAL PROPERTIES

ASSUMPTIONS

- 5 ply - 15.5 mm panel
- Product standard specifies

mimimum face/back ply thickness	2.4 mm
minimum interior ply thickness	2.4 mm
maximum face/back ply thickness	3.2 mm
maximum interior ply thickness	5.0 mm
nominal panel thickness	15.5 mm
negative tolerance	0.5 mm

- The average panel thickness of all 15.5 mm panels sampled for the test program at the time when normal quality control measurements are made (0.04 to 0.06 moisture content) is 15.2 mm and at the time the RILEM/CIB tests are undertaken (0.15 moisture content) is 15.8 mm.

EXAMPLE CALCULATION

1. Determine the increase in average thickness between the time normal quality control measurements are made and at the time the RILEM/CIB tests are undertaken.

RILEM/CIB test thickness	15.8
Quality control thickness	15.2
Increase in thickness between quality control and time of test	$\frac{15.8}{15.2} \simeq 1.04$

2. Determine the minimum panel thickness specified in the product standard.

Nominal panel thickness	15.5 mm
Negative tolerance	- 0.5 mm
Minimum panel thickness	15.0 mm

3. Determine the minimum panel thickness of the product standard at the time of RILEM/CIB test.

Minimum panel thickness	15.0 mm
Increase in thickness between quality control and time of test	x 1.04
Minimum panel thickness at time of test	15.6 mm

4. Derive construction with minimum geometrical properties parallel to face grain.

(a) Calculate the minimum thickness for all plies with grain parallel to the face grain.

Face	2.4 mm
Back	2.4 mm
Centre	2.4 mm
Total thickness of all plies with grain parallel to the face grain	7.2 mm

(b) Calculate the crossband thickness.

Minimum panel thickness ("at time of test")	15.6 mm
Total thickness of all plies with grain parallel to the face grain	- 7.2 mm
Total thickness of crossbands	8.4 mm
Crossband thickness (8.4 mm ÷ 2)	4.2 mm

(c) Describe the construction with minimized geometrical properties parallel to face grain.

Face	2.4 mm
Crossband	4.2 mm
Centre	2.4 mm
Crossband	4.2 mm
Back	2.4 mm
Panel thickness	15.6 mm

5. Derive construction with minimum geometrical properties perpendicular to face grain.

(a) Calculate the minimum thickness for all plies with grain perpendicular to the face grain.

Crossband	2.4 mm
Crossband	2.4 mm
Total thickness of all plies with grain perpendicular to the face grain	4.8 mm

(b) Calculate the maximum thickness for face grain and back plies.

Face	3.2 mm
Back	3.2 mm
Total thickness of face and back plies	6.4 mm

(c) Calculate the thickness of the centre ply.

Minimum panel thickness ("at time of test")	15.6 mm
Crossbands	- 4.8 mm
Face/back plies	- 6.4 mm
Centre thickness	4.4 mm

(d) Describe the construction with minimized geometrical properties perpendicular to face grain.

Face	3.2 mm
Crossband	2.4 mm
Centre	4.4 mm
Crossband	2.4 mm
Back	3.2 mm
Panel thickness	15.6 mm

6. Calculate geometrical properties for the minimized constructions (per metre width).

$A_0 = 7,200.0\ \text{mm}^2$ $\qquad A_{90} = 4,800.0\ \text{mm}^2$

$I_0 = 212,544.0\ \text{mm}^4$ $\qquad I_{90} = 57,792.0\ \text{mm}^4$

$W_0 = 27,249.2\ \text{mm}^3$ $\qquad W_{90} = 12,563.5\ \text{mm}^3$

AN EXAMPLE OF DETERMINING PLYWOOD PROPERTIES FOR DIFFERENT FORMS OF PRESENTATION

Assumptions

- Effective cross section theory has been utilized to analyse the test data and a characteristic level of stress/moduli common to several constructions identified. For this example, only the tension strength case will be examined and the stresses are

$$f_{t0} = f_{t90} = 25 \text{ N/mm}^2.$$

- Code values are required for the minimized constructions identified in Appendix C. Geometrical properties are

Effective Cross Section

$$A_0 = 7{,}200 \text{ mm}^2 \qquad A_{90} = 4{,}800 \text{ mm}^2$$

Full Cross Section

$$A_0 = A_{90} = 15{,}600 \text{ mm}^2$$

Example Calculations

0° Orientation

Effective Cross Section

$$f_{t0} = 25 \text{ N/mm}^2$$

$$A_0 = 7{,}200 \text{ mm}^2$$

Load Carrying Capacity

$$T_0 = 25(7{,}200)$$

$$= 180{,}000 \text{ N}$$

Full Cross Section

$$A_0 = 15{,}600 \text{ mm}^2$$

$$f_{t0} = \frac{180{,}000}{15{,}600}$$

$$= 11.54 \text{ N/mm}^2$$

90° Orientation

Effective Cross Section

$$f_{t90} = 25 \text{ N/mm}^2$$

$$A_{90} = 4{,}800 \text{ mm}^2$$

Load Carrying Capacity

$$T_{90} = 24(4{,}800)$$

$$= 120{,}000 \text{ N}$$

Full Cross Section

$$A_{90} = 15{,}600 \text{ mm}^2$$

$$f_{t90} = \frac{120{,}000}{15{,}600}$$

$$= 7.69 \text{ N/mm}^2$$

CIB-W18/12-6-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STRENGTH CLASSIFICATIONS FOR TIMBER ENGINEERING CODES

by

R H Leicester and W G Keating
Commonwealth Scientific and Industrial Research Organisation
AUSTRALIA

BORDEAUX, FRANCE
OCTOBER 1979

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by

R.H. Leicester and W.G. Keating
(CSIRO Division of Building Research,
Melbourne, Australia, Aug. 1979)

SUMMARY

This paper contains a general discussion on the use of strength classification systems in timber engineering Codes. In order to illustrate some of the more important concepts involved, it includes a detailed description of the classification systems used in Australian Codes. There is also discussion on aspects to be considered in the development of a classification system for an international Code.

1. INTRODUCTION

1.1 Definition of Classification Systems

In this paper the term "Classification System" refers to the use of a set of ordered values for design parameters. Examples of structural parameters that are suited to such a system are basic design stresses, elastic constants and connector strengths. Thus each design parameter tabulated within a classification system of a timber engineering Code is applicable not just to a specific timber, but rather to a complete class or group of timbers.

1.2 Motivation for the Use of Classification Systems

Where the structural species of timber utilised in a country are easily identifiable and few in number, say less than five, it may be appropriate that specific structural design properties be published for each of these species. However, in many countries, numerous species are utilised. For example Pong Sono (1974) has listed approximately 200 species of merchantable timber in Thailand. Espiloy (1978) notes that in the Philippines there are over 3000 timber species of which several hundred are probably potentially merchantable. In Australia about 80 species are used extensively and over 500 species have been classified for structural utilisation (Standards Association of Australia 1980). These species are often sold in mixtures, because of the practical difficulties associated with their identification and segregration.

Where numerous species are utilised, it is not feasible for timber engineering Codes to publish design information specific to each species. Rather, it is preferable to group structural design properties into a limited number of strength classes. In general each strength class will cover a large number of species and mixtures of species.

The use of a limited number of strength classifications is of considerable value to the designer as it enables him to specify timber by a strength classification rather than by species. He thus has a wider choice of timbers to choose from for any specific design. For the producer it assists in the utilisation of lesser known timbers which may be sparodic and regionally limited in their occurrence. For Code writers, strength classification systems are helpful in that their obvious rounding off of design properties alerts the design engineer to the degree of approximation inherent in design information. This is particularly true when strength classifications are applied to mixtures of species.

However, the greatest benefit from strength classifications is probably obtained in the production of building Standards. For example, in Australia, the Australian Standard 1684-1979, the SAA Timber Framing Code (Standards Association of Australia 1979), through a limited set of tables covering only seven strength classes, manages to present timber sizes for domestic construction applicable to unseasoned timber of several hundred species. As another example, the United Nations Development Organisation (1978), has in preparation a set of standard designs for wooden bridges which is based on only eight strength classes and yet is directly applicable to the commonly available timber species of India, South and Central America, West Africa, East Africa, the Pacific Region and South East Asia.

1.3 Limitations of Strength Classification Systems

The primary limitation of strength classifications is that they may lead to inefficiencies, usually of a minor nature, in the utilisation of some important timber species. *There is of course, no reason as to why structural timber Codes written primarily in a strength classification format should not be able to provide also for the use of special design recommendations for specific species.* This would normally be done for species that dominate the utilisation picture for a particular country, to an extent that is sufficient to justify the additional costs involved in obtaining specially for these species design information and also the associated building Standards, such as those giving framing sizes for domestic construction.

However, even for the extreme case of a single dominant species, the use of a classification system may well prove to be the most effective design Code format, as this will remove the necessity of changing all the associated building Standards each time the grading methods are altered, regional effects on timber properties are noted, or new research indicates that a change in design parameters is necessary.

1.4 Classification Systems for Non-Structural Properties

For countries utilising numerous species it is obviously useful to apply classification systems as a general means of tabulating information. In particular it is fairly common for information on density, shrinkage and durability to be tabulated in this manner.

1.5 Bibliography on Strength Classification Systems

Apart from Australian publications which will be mentioned in the course of this paper, there are also many other publications on strength classifications, but these are often difficult to find. Some useful publications are those prepared for ISO (Larsen 1978), Africa (Bolza and Keating 1972, Campbell and Malde 1970, Comben, Okigbo 1966, Ward 1974), Malaya (Burgess 1956, Engku Abdul Rahman bin Chik 1972), Singapore (Singapore Timber Standardisation Committee 1966), Philippines (Epsiloy 1978), Indonesia (Suparman Karnasudirdga et al 1978, Iding Kartasujana and Abdurahim Martawijaya), Laos (Timber Research and Development Association 1976), Papua New Guinea (Department of Forests 1973, Eddows 1977, Bolza 1975), Fiji (Department of Forestry 1968, 1970), Solomon Islands

(Forestry Division 1976, 1979), South East Asia (Bolza and Keating 1980), South America (Berni et al 1980), United Kingdom (British Standards Institution 1971) and Australia (Standards Association of Australia 1975, 1979, 1980).

In addition to strength classifications, many of these publications also contain classifications for density, shrinkage and durability.

1.6 Reference to Australian Standards

In this paper most of the methods described and Standards cited will be taken from Australian sources. This is primarily because the authors are most familiar with these sources, but also partly because these are fairly extensive as Australia has used strength classification systems now for more than 40 years.

It is of interest to mention that one particular method of entering the Australian system (Standards Association of Australia 1980) has now been applied to some 2000 species around the world including 700 African species (Bolza and Keating 1972), 500 South East Asian species (Bolza and Keating 1980), 100 South American species (Berni et al 1980) and 500 Australian species (Standards Association of Australia 1980). With the use of the related Australian Standards, such as AS 1720-1975, the SAA Timber Engineering Code (Standards Association of Australia 1975), it is possible to produce structural designs for all these timber species with respect to sawn timber, pole timber, glued laminated timber, plywood and the associated metal connectors.

From the bibliography cited in the previous sections, it would appear that the current Australian system of strength classification has been used in United Nations Development Organisation projects, in Kenya, Tanzania, Nigeria, Papua New Guinea, Fiji and the Solomon Islands. Some of the classification systems used in other countries are similar to or closely resemble the previous Australian system (Pearson et al 1962).

1.7 Outline of This Paper

The following is intended to explain the concept of strength classification systems. First the classification system used in Australia is described as an example to illustrate both the features of a classification system and the method of its use. This is then followed by a more general discussion on the development of a classification system for an international Code.

2. THE AUSTRALIAN CLASSIFICATION SYSTEMS

Australian classification systems have evolved over a period of about 45 years. The current forms are given in the Australian Standard 1720-1975, the SAA Timber Engineering Code (Standards Association of Australia 1975) and the basis of this method has been described by Pearson (1965) and Kloot (1973). The system is still undergoing minor modifications, and the following is from the current proposals for the revision of AS 1720-1975.

For obtaining design parameters, the following four strength classifications are used:

(i) Strength Groups for timber species and mixtures (S1-S7 and SD1-SD8)

A total of 15 strength groups related to the strength properties of small clear timber; seven groups for green timber and eight for seasoned timber.

(ii) Stress Grades for sawn and pole timbers (F2-F34)

A total of 12 stress grades related to the properties of structurally graded timber.

(iii) Stress Grades for plywood (F7-F34)

A total of eight stress grades for structurally graded plywood.

(iv) Joint Groups (J1-J4) and (JD1-JD4)

A total of eight joint strength groups; four for green timber and four for seasoned timber.

Tables 1-5 show some examples of design properties specified in terms of these strength classifications. It is to be noted that these tables cover the properties required for the design of sawn timbers, pole timbers, plywood and the associated metal connectors. Design properties for glued laminated timber could also be specified in terms of strength classifications such as the one proposed for the draft CIB-Structural Timber Design Code (1978). However, in AS 1720-1975, the design properties of glued laminated timber are obtained by applying suitable modification factors to the design properties for sawn timber.

TABLE 1

BASIC WORKING STRESSES (MPa) FOR COMPRESSION PERPENDICULAR TO GRAIN AND SHEAR AT JOINTS RELATED TO STRENGTH GROUPS

Strength Group		Compression perpendicular to grain (MPa)	Shear at joints details (MPa)
Green	Seasoned		
	SD1	10.4	4.15
	SD2	9.0	3.45
	SD3	7.8	2.95
S1	SD4	6.6	2.45
S2	SD5	5.2	2.05
S3	SD6	4.1	1.70
S4	SD7	3.3	1.45
S5	SD8	2.6	1.25
S6		2.1	1.05
S7		1.7	0.86

TABLE 2

BASIC WORKING STRESSES AND ELASTIC CONSTANTS
FOR SAWN TIMBER AND POLE TIMBER

Stress Grade	Type of stress				Modulus of elasticity (MPa)	Modulus of rigidity (MPa)
	Bending (MPa)	Tension parallel to grain (MPa)	Shear in beams (MPa)	Compression parallel to grain (MPa)		
F34	34.5	24.2	2.45	26.0	21500	1430
F27	27.5	19.3	2.05	20.5	18500	1230
F22	22.0	15.4	1.70	16.5	16000	1070
F17	17.0	11.9	1.45	13.0	14000	930
F14	14.0	9.8	1.25	10.5	12200	830
F11	11.0	7.7	1.05	8.3	10500	700
F8	8.6	6.0	0.86	6.6	9100	610
F7	6.9	4.8	0.72	5.2	7900	530
F5	5.5	3.9	0.62	4.1	6900	460
F4	4.3	3.0	0.52	3.3	6100	410
F3	3.4	2.4	0.43	2.6	5200	350
F2	2.8	2.0	0.36	2.1	4500	300

TABLE 3

BASIC WORKING STRESSES AND ELASTIC CONSTANTS FOR
STRUCTURAL PLYWOOD AT 12 per cent MOISTURE CONTENT

Stress grade	Stress					Modulus of elasticity	Modulus of rigidity
	Bending (MPa)	Tension (MPa)	Shear (MPa)	Compression in the plane of the sheet (MPa)	Compression normal to the plane of sheet (MPa)		
F34	34.5	24.2	2.30	26.0	10.4	21500	1075
F27	27.5	19.3	2.30	20.5	9.0	18500	925
F22	22.0	15.4	2.30	16.5	7.8	16000	800
F17	17.0	11.9	2.30	13.0	6.6	14000	700
F14	14.0	9.8	2.05	10.5	5.2	12200	625
F11	11.0	7.7	1.80	8.3	4.1	10500	525
F8	8.6	6.0	1.60	6.6	3.3	9100	455
F7	6.9	4.8	1.40	5.2	2.6	7900	345

TABLE 4

BASIC LATERAL LOADS FOR ONE NAIL
IN SINGLE SHEAR IN SIDE GRAIN

Joint Group		Basic lateral load per nail (N)						
Green	Seasoned	d=2.5	d=2.8	d=3.15	d=3.75	d=4.5	d=5.0	d=5.6
	JD1	435	530	650	885	1220	1460	1780
J1	JD2	330	400	490	665	915	1106	1346
J2	JD3	260	315	385	525	720	870	1060
J3	JD4	185	225	275	375	515	620	755
J4		130	160	195	265	365	440	540

d = nail diameter (mm)

TABLE 5

BASIC WITHDRAWAL LOADS FOR COACH SCREWS

Joint group		Withdrawal load (N/mm penetration of threaded portion)					
Green	Seasoned	d=6	d=8	d=10	d=12	d=16	d=20
J1		59	67	75	83	96	108
	JD1	74	84	94	104	120	135
J2		47	53	61	67	77	87
	JD2	59	66	76	84	86	108
J3		33	39	45	49	57	65
	JD3	41	49	56	61	71	81
J4		26	28	31	33	39	45
	JD4	32	35	39	41	49	56

d = shank diameter (mm)

3. METHODS FOR ENTERING THE AUSTRALIAN CLASSIFICATION SYSTEMS

3.1 Strength Groups

The method of determining the strength group for a particular species is described in a Miscellaneous Publication of the Standards Association of Australia (Standards Association of Australia 1980) and is summarised in Appendix 1. It is based primarily on the strength of small clear specimens of wood, but may also be derived (albeit more conservatively) from density values alone.

3.2 Stress Grades for Sawn Timber

In Australia stress grades are derived either through visual methods or through mechanical grading methods based on measurement of local stiffness. A third method, proof grading (Leicester 1979), is currently being assessed.

Tables 6 and 7 show the relationship between visual grade, strength group and stress grade. All three increment by a factor of 1.25 between each step and consequently they mesh together very effectively. Thus, for a specific stress grade, timber species from different strength groups may be interchanged.

3.3 Stress Grades for Pole Timber

The stress grades of pole timbers are considered to be related directly to the timber strength group as indicated in Table 8.

3.4 Stress Grades for Plywood

In the Draft Standard DR 78051, Draft Australian Standard Specification for Structural Plywood (Standards Association of Australia 1978), visual grading rules for plywood veneer are specified so that their strength is roughly 60 per cent of the clear wood strength when the maximum permissible defects occur. With this prerequisite satisfied, the stress grade for a plywood may be derived from <u>any one</u> of the following three parameters:

(i) strength group of the timber veneer,

(ii) the density of the veneers, or

(iii) the stiffness of the plywood sheets.

Table 9 shows the relationship between these parameters and the plywood stress grade.

3.5 Joint Groups

Joint groups are derived from density as shown in Tables 10 and 11. The basis for this has been described by Mack (1978).

3.6 Table of Strength Classifications

An example of a table of strength classifications is shown in Tables 12 and 13. As indicated previously, if it were required it would be possible to extend Table 13 immediately to all 2000 species of the world's timbers that have already been strength grouped according to the Australian system. It will be noted that for each type of strength classification, there should be an associated Standard that specifies the method for determining the correct classification.

TABLE 6

RELATIONSHIP BETWEEN STRENGTH GROUP,
VISUAL GRADE AND STRESS GRADE FOR UNSEASONED TIMBER

VISUAL GRADE*		STRESS GRADE						
Nomenclature	Per cent strength of clear material	S1	S2	S3	S4	S5	S6	S7
Structural Grade No.1	75	F27	F22	F17	F14	F11	F8	F7
Structural Grade No.2	60	F22	F17	F14	F11	F8	F7	F5
Structural Grade No.3	48	F17	F14	F11	F8	F7	F5	F4
Structural Grade No.4	38	F14	F11	F8	F7	F5	F4	F3

* Australian Standard AS 2082-1977, Visually stress-graded Hardwood for Structural Purposes, and AS 1648-1974 Visually Stress Graded Cypress Pine for Structural Purposes.

TABLE 7

RELATIONSHIP BETWEEN STRENGTH GROUP,
VISUAL GRADE AND STRESS GRADE FOR DRY TIMBER

VISUAL GRADE*		STRESS GRADE							
Nomenclature	Per cent strength of clear material	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Structural Grade No.1	75		F34	F27	F22	F17	F14	F11	F8
Structural Grade No.2	60	F34	F27	F22	F17	F14	F11	F8	F7
Structural Grade No.3	48	F27	F22	F17	F14	F11	F8	F7	F5
Structural Grade No.4	38	F22	F12	F14	F11	F8	F7	F5	F4

* Australian Standard AS 2082-1977, Visually Stress-grade Hardwood for Structural Purposes and AS 2009-1977 Visually Stress-graded Seasoned Australian Grown Softwood (Conifers) for Structural Purposes (Excluding Radiata Pine and Cypress Pine), AS 1490-1973 Visually Stress Graded Radiata Pine For Structural Purposes and AS 1648-1974 Visually Stress Graded Cypress Pine for Structural Purposes.

TABLE 8

CORRESPONDENCE BETWEEN STRENGTH GROUP AND
STRESS GRADE FOR ROUND TIMBERS GRADED TO AS 0117

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

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

TABLE 9

GRADING PARAMETERS FOR PLYWOOD STRESS GRADES

PLYWOOD STRESS GRADE	GRADING PARAMETERS*				
	Timber strength Group	Modulus of elasticity of plywood Sheet (MPa)	Minimum Air-Dry Density (kg/m^3)		
			Species Mean	Single veneers of plywood of a single species	Single veneers of plywood of mixed species
F34	SD1	21500	1200	1020	1110
F27	SD2	18500	1080	920	1000
F22	SD3	16000	960	820	890
F17	SD4	14000	840	700	780
F14	SD5	12500	730	620	680
F11	SD6	10500	620	530	570
F8	SD7	9100	520	440	480
F7	SD8	7900	420	360	390

* Only one of the five grading parameters need be used

TABLE 10

MINIMUM DENSITIES FOR JOINT STRENGTH GROUPS IN GREEN TIMBER

JOINT GROUP	J1	J2	J3	J4
BASIC DENSITY (kg/m^3)	750	600	475	380

TABLE 11

MINIMUM DENSITIES FOR JOINT STRENGTH GROUPS IN DRY TIMBER

JOINT GROUP	JD1	JD2	JD3	JD4
AIR-DRY DENSITY AT 12% MOISTURE CONTENT, (kg/m^3)	940	750	600	475

TABLE 12

STRENGTH CLASSIFICATIONS FOR UNIDENTIFIED SPECIES AND MIXTURES OF TIMBERS

SPECIES MIXTURE	STRENGTH GROUP(1)		STRESS GRADES FOR SAWN TIMBER(2)								STRESS GRADE FOR POLES(3)	STRESS GRADE FOR PLY-WOOD(4)	JOINT GROUP(5)	
			GREEN				SEASONED							
	Green	Seasoned	STR. No:1	STR. No.2	STR. No.3	STR. No.4	STR. No.1	STR. No.2	STR. No.3	STR. No.4			Green	Seasoned
Mixed Australian Hardwoods (excluding rainforest species) from S.A., Tas., Vic. and W.A.	S4	SD4	F14	F11	F8	F7	F22	F17	F14	F11	F17	F17	J3	JD3
Ash-type Eucalypts from N.S.W. Highlands	S4	SD4	F14	F11	F8	F7	F22	F17	F14	F11	F17	F17	J3	JD3
Non ash-type Eucalypts from Qld. and N.S.W.	S3	SD3	F17	F14	F11	F8	F27	F22	F17	F14	F22	F22	J2	JD2
Rainforest species	S7	SD7	F7	F5	F4		F11	F8	F7	F5	F8	F8	J4	JD4
Pinus species		SD7					F11	F8	F7	F5		F8		JD4
Softwood species (excluding pinus species)		SD8					F8	F7	F5	F4		F7		JD4
Imported softwoods	S6	SD7	F8	F7	F5	F4	F11	F8	F7	F5	F11	F8	J4	JD4

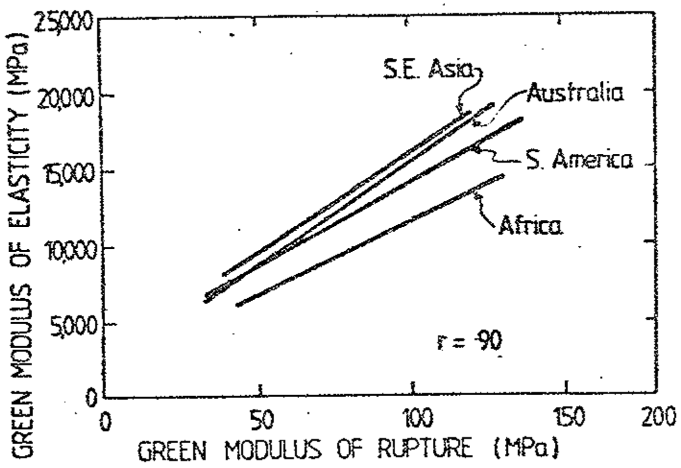
(1) For Strength Group Classification use Australian Standard.........
(2) For Stress Grade Classification use appropriate Australian Standards 2082-1977, 2009-1977, 1490-1973 and 1648-1974
(3) For Poles grade to Australian Standard 0117-1970
(4) For Plywood graded to Australian Standard.............
(5) For joint strengths assessed according to Australian Standard 1649-1974

TABLE 13

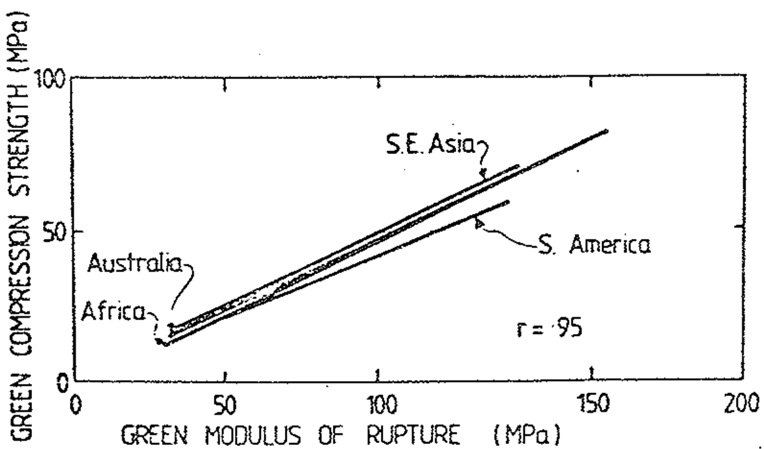
STRENGTH CLASSIFICATIONS FOR SELECTED SPECIES OF TIMBER

SPECIES	STRENGTH GROUP(1)		STRESS GRADES FOR SAWN TIMBER(2)								STRESS GRADE FOR POLES(3)	STRESS GRADE FOR PLY-WOOD(4)	JOINT GROUP(5)	
			GREEN				SEASONED							
	Green	Seasoned	STR. No.1	STR. No.2	STR. No.3	STR. No.4	STR. No.1	STR. No.2	STR. No.3	STR. No.4			Green	Seasoned
Ash, Mountain	S4	SD3	F14	F11	F8	F7	F27	F22	F17	F14	F17	F22	J3	JD3
Box, Brush	S3	SD3	F17	F14	F11	F8	F27	F22	F17	F14	F22	F22	J2	JD2
Fir, Douglas	S5	SD5	F11	F8	F7	F5	F17	F14	F11	F8	F14	F14	J4	JD4
Gum, Sydney Blue	S2	SD3	F22	F17	F14	F11	F27	F22	F17	F14	F27	F22	J2	JD2
Hardwood (Johnstone River)	S1	SD2	F27	F22	F17	F14	F34	F27	F22	F12	F34	F27	J1	JD1
Pinus, radiata	S6	SD6							F8	F7	F11	F11		JD4

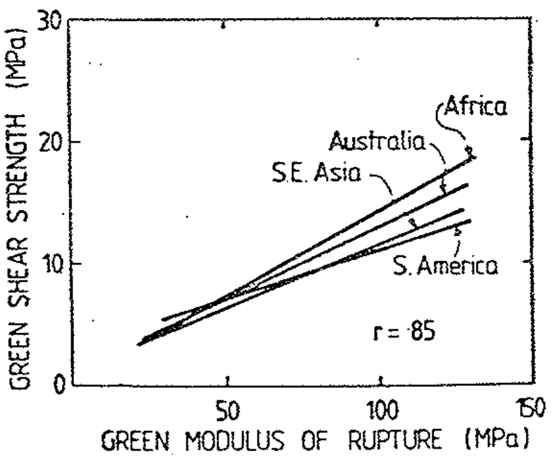
(1) For Strength Group classification use Australian Standard..........
(2) For Stress Grade classification use appropriate Australian Standards 2082-1977, 2009-1977, 1490-1973 and 1648-1974
(3) For Poles grade to Australian Standard 0117-1970 -
(4) For Plywood graded to Australian Standard...........
(5) For joint strengths assessed according to Australian Standard 1649-1974



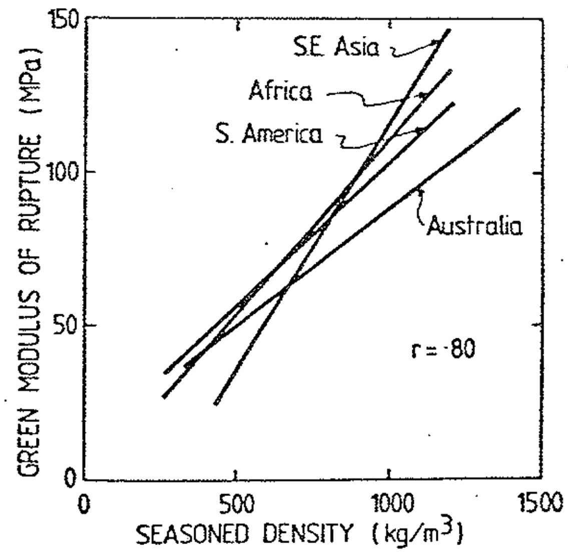
(i) Modulus of Elasticity



(ii) Compression Strength

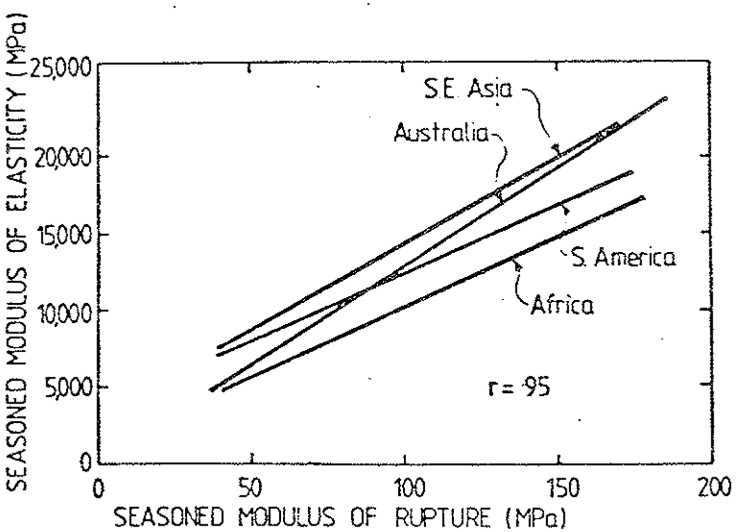


(iii) Shear Strength

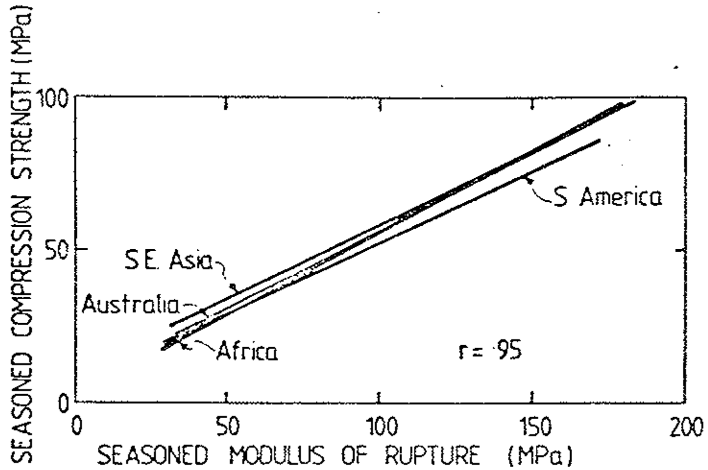


(iv) Modulus of Rupture

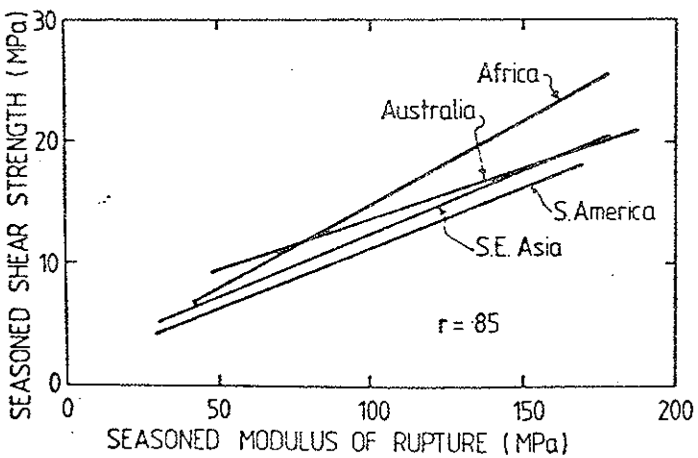
FIG 1. REGRESSION LINES FOR GREEN TIMBER



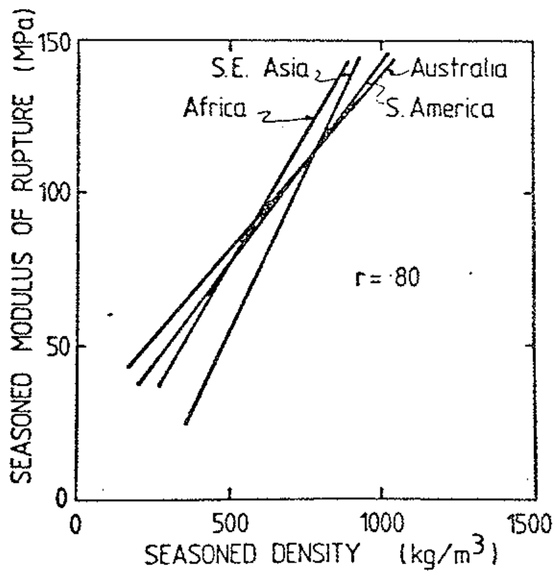
(i) Modulus of Elasticity



(ii) Compression Strength



(iii) Shear Strength



(iv) Modulus of Rupture

FIG 2. REGRESSION LINES FOR SEASONED TIMBER

4. STRENGTH CLASSIFICATIONS FOR AN INTERNATIONAL TIMBER ENGINEERING CODE

4.1 Applicability of the Australian Classification System

The Australian system has evolved through usage over several decades and has been found suitable for application by the United Nations Development Organisation and several countries. Consequently, the format should be useful as a general guideline for an international Code. Particularly successful features of the system are the wide range of species covered, the use of a geometric progression for the steps between various strength classes, and the facility to enter the system through several procedures, including procedures that may be used when there is limited technical information available.

The example of the Australian classification system also indicates an important point with respect to timber composites; namely that a decision must be made as to whether design properties for composites are presented as a function of strength classifications or whether the properties are to be computed from the design properties of solid timber. In the Australian system the choice was made to use a classification system for plywood, but to use a computation method for glued laminated timber. The use of classification systems with composites leads to similar advantages as those obtained through their use with solid timber, but they also lead to a considerable restriction in the range of possible design properties for composites.

Before the Australian or any other existing classification system can be modified for use in an international Code, two aspects should be investigated. The first is to determine whether the existing system has the accuracy necessary for an international Code. the Second is to determine the modifications necessary to extend it for international use. These two points will be discussed in the following sections.

4.2 Accuracy of Classification Systems

Most classification systems and the associated grading Standards have evolved over a period of time and their ranges of applications have been extrapolated with little assessment of their effectiveness. For comparison with other countries, the intended load factors implied in the use of Australian Standards are given in Appendix B.

Measurements by Madsen (1975, 1978), Madsen and Barret (1976) and Madsen and Nielsen (1976) of Canadian softwood timber strength at the 5-percentile level has shown that the influences of many material parameters such as small clear material strength, moisture content, lumber size and material defects do not follow some of the traditionally held views. Thus with the introduction of classification systems for structural timber Codes, it would be timely to initiate in-depth investigations on the effectiveness of the various methods used for entering these systems.

Apart from improving the accuracy of design parameters these investigations may lead to a considerable simplification in timber utilisation. For example, Madsen has noted that many of the expected effects on strength of forest location, visual grading rules, moisture content and even species differences do not occur and so may not be necessary to consider in entering a classification system.

One matter that would need to be resolved at the outset is the degree of accuracy that would be required in a classification system. Obviously it would be uneconomical to require from classification systems a degree of reliability that is considerably in excess of that obtained from the remainder of the design process.

In this context it is well to bear in mind that the design process can involve uncertainties that are of considerable magnitude (Pham and Leicester 1979). A simple example of this would be the uncertainties that occur in the design of the top-chord of a truss. Even if the basic strength of the lumber were correctly specified through a strength classification system, there still remains considerable uncertainty on the beam-column action of top chord members.

4.3 Extrapolation of Classification Systems

Some care is required in the translation of classification systems from one region of the world to another. One obvious source of difficulties is that the characteristics of structural timbers may differ. For example, the design properties related to the stress grades shown in Table 2 refer to Australian timbers visually graded according to Australian Standards. In other countries, the relationship between the various design properties may differ, either because of the use of different grading methods, or because the basic properties of the timber are different. As an example, Fig. 1 and 2 shows regression lines obtained from small clear samples of roughly 30 commonly available species from each of four regions around the world. The data is taken from the publications by Lavers (1969), Kukachka (1970), Wangaard (1952), Bolza and Kloot (1963 and 1975). While the regression lines for all regions are similar, there are some significant differences such as, for example, the relatively low modulus of elasticity of African timbers compared to the other three regions.

There are other factors to be considered in the translation of classification systems, such as the technological and socio-economic environment in which the timber engineering Code is to be used. For example, in Australia there has been a change from traditional practice and it is now considered that the use of certain metal connectors should be permitted in locations where natural defects occur in timber. Another example would be the acceptance of reduced levels of reliability in certain countries.

Probably a workable approach would be for ISO to specify the classification system and then to permit each region to choose or develop a method for entering the system.

5. CONCLUSIONS

The use of strength classification systems is essential for countries concerned with the utilisation of numerous timber species. This is particularly true for countries in which the species are used in mixtures, or in countries where there is little technical information on local timber properties.

The Australian classification system provides useful guidelines on the format that would be suitable for an international Code. This system has evolved through usage in Australia over a period of 40 years and has also been used in several other countries and a United Nations Organisation project. A method is available for entering the system when only limited technical data is available and some 2000 of the world's timbers have been classified in this way.

Because of regional differences, it may be necessary and desirable that methods for entering an international classification system be determined by and related to the requirements of each specific region. However, in the interests of facilitating international exchanges of technological information it is essential that the strength classifications themselves be defined in international Standards. These Standards should specify procedures for assessing any proposed methods of entering a classification system. In this regard, the technique of in-grade testing developed by Madsen (1978) would be particularly useful.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

Berni, C., Bolza, E. and Christensen, F.J. (1980). - Characteristics, Uses and Properties of 190 South American Timbers. (In Press).

Bolza, E. (1975). - Properties and Uses of 175 Timber Species from Papua New Guinea and West Irian. Commonwealth Scientific and Industrial Research Organisation, Division of Building Research Report 34, Australia.

Bolza, E. and Keating, W.G. (1972). - African Timbers - The Properties, Uses and Characteristics of 700 Species. Commonwealth Scientific and Industrial Research Organisation, Division of Building Research, Melbourne, Australia.

Bolza, E. and Keating, W.G. (1980). - Properties and Uses of Selected South East Asian, Pacific and Tropical

Bolza, E. and Kloot, N.H. (1980). - The Mechanical Properties of 174 Australian Timbers. Commonwealth Scientific and Industrial Research Organisation, Division of Forest Products Technological Paper No.25, Australia.

Bolza, E. and Kloot, N.H. (1975). - The Mechancial Properties of 81 New Guinea Timbers. Commonwealth Scientific and Industrial Research Organisation, Division of Building Research Technical Paper (Second Series) No. 11, Australia.

British Standards Institution (1971). - British Standard Code of Practice CP112: Part 2 metric units, 1971. British Standards Institution, U.K.

Burgess, H.J. (1956). - Strength Grouping of Malayan Timbers. Malayan Forester, Vol XIX, No. 1, 1956, pp. 33-36.

Campbell, P.A. and Malde, K. (1970). - Timber for Building in Tanzania. Ministry of National Resources and Tourism, United Republic of Tansania.

CIB - Working Group W18 (1978). - CIB Structural Timber Design Code (Third Draft). Sept. 1978.

Comben, A.J. ( ). - The Establishment of Provisional Stress Grades for Nigerian Timbers. Nigerian Journal of Forestry, pp. 34-42.

Department of Forests, (1972). - New Horizons, Forestry in Papua New Guinea, Dept of Forests, Papua New Guinea, Jacaranda Press, Australia.

Department of Forestry (1968). - Some Timbers of Fiji. Department of Forestry, Suva, Fiji.

Department of Forestry (1970). - Fiji Timbers and Their Uses. (Leaflet Series). Department of Forestry, Suva, Fiji.

Eddowes, P.J. ( ). - Species Notes on the Major Timbers of Papua New Guinea. Office of Forest Products Research Centre, Papua New Guinea.

Eddowes, P.J. (1977). - Commercial Timbers of Papua New Guinea - Their Properties and Uses. Forest Products Research Centre. Hebams Press, Port Morseby, PNG.

Engku Abdul Rahman Bin Chik (1972). - Basic and Grade Stresses for Strength Groups of Malaysian Timbers. The Malayan Forester, Vol. IIIV, No. 2, 1972, pp. 131-134.

Espiloy, Jr. E.B. (1978). - Strength Grouping of Philippine Timbers for Utilisation of Lesser-Known Species. Forest Products Research and Industrial Development Commission, National Science Development Board, Technical Note No. 187, March 1978.

Forestry Division (1976). - Solomon Island Timbers. Timber Booklet No. 1, Major Species. Forestry Division, Ministry of National Resources, Government Printing Office, Honiara, Solomon Islands.

Forestry Division (1979). - Solomon Island Timbers. Timber Booklet No. 2, Minor Species. Forestry Division, Ministry of National Resources. Government Printing Office, Honiara, Solomon Islands.

Iding Kartasujana and Abdurahim Martawijaya ( ). - Commercial Woods of Indonesia, Their Properties and Uses.

Kloot, N.H. (1973). - The Strength Group and Stress Grade Systems Forest Products Newsletter. Commonwealth Scientific and Industrial Research Organisation, Forest Products Laboratory. Sept.-Oct. 1973, pp.1-12.

Kukachka, B.F. (1970). - Properties of Imported Tropical Woods. USDA Forest Research Paper FPL 125, US Dept of Agriculture. Forest Products Laboratory, Madison, Wisconsin.

Larsen, H.J (1978). - Classification of Structural Timber. CIB Working Commission W18-Timber Structures. Perth Scotland, June 1978.

Lavers, G. (1969). - The Strength Properties of Timbers, Bulletin No. 50 (Second Edition, metric units) Forest Products Research. Ministry of Technology. HMSO, London.

Leicester, R.H. (1969). - Proof Grading, A Practical Application of Reliability Theory. Proceedings of Third International Conference on the Applications of Statistics and Probability in Soil and Structural Engineering. Sydney, Australia, Jan-Feb 1979, pp.263-277.

Mack, J.J. (1978). - The Grouping of Species for the Deisgn of Timber Joints with Particular Application to Nailed Joints. Commonwealth Scientific and Industrial Research Organisation, Division of Building Research Technical Paper (Second Series) No. 26, Australia.

Mack, J.J. (1979). - Australian Methods for Mechanically Testing Small Clear Specimens of TIMBER. Commonwealth Scientific and Industrial Research Organisation, Division of Building Research Technical Paper (Second Series) No. 31, Australia.

Madsen, B. (1975). - Moisture Content - Strength Relationship for Lumber Subjected to Bending. Canadian Journal of Civil Engineering. Vol. 2, No. 4, pp. 446-473.

Madsen, B. (1978). - In-Grade Testing, Problem Analysis. Forest Products Journal, Vol. 28, No. 4, April 1978, pp. 42-50.

Madsen, B. and Barret, J.D. (1976). - Time-Strength Relationship for Lumber. Structural Research Series, Report No. 13, Dept of Civil Engineering, University of British Columbia, Vancouver, British Columbia, Canada.

Madsen, B. and Nielsen, P.C. (1976). - In-Grade Testing. Size Investigation on Lumber Subjected to Bending. Structural Research Series, Report No. 15, Department of Civil Engineering, University of British Columbia, Vancourver, British Columbia, Canada.

Okigbo, L.C. (1966). - Derivation of Working Stresses for Nigerian Timbers (Unpublished). Imperial College of Science and Technology, London, U.K.

Pearson, R.G. (1965). - The Establishment of Working Stresses for Groups of Species. Commonwealth Scientific and Industrial Research Organisation, Division of Forest Products Technological Paper No. 35, Australia.

Pearson, R.G., Kloot, N.H. and Boyd, J.D. (1962). - Timber Engineering. Design Handbook, Jacaranda Press, Brisbane, Australia.

Pham, L. and Leicester, R.H. (1976). - Structural Variability due to the Design Process. Proceedings of the Third International Conference on the Applications of Statistics and Probability in Soil and Structural Engineering. Sydney, Australia, Jan-Feb. 1979, pp. 586-600.

Pong Sono (1974). - Merchantable Timbers of Thailand. Symposium on Research and Marketing of South East Asian Timbers and Timber Products, organised by the German Foundation for International Development (Berlin) and the Government of the Republic of the Philippines, Manila and Los Banos, Philippines, 19-23 November, 1974, pp. 120-165.

Singapore Timber Standardisation Committee (1966). - Code of Practice for the Use of Timber in Building Construction. Singapore Industrial Reserach Unit. Singapore.

Standards Association of Australia (1974). - Australian Standard 1649-1974, Determination of Basic Working Loads for Metal Fasteners For Timber. Standards Association of Australia, Sydney, Australia.

Standards Association of Australia (1975). - Australian Standard 1720-1975, SAA Timber Engineering Code (Metric Units). Standards Association of Australia, Sydney, Australia.

Standards Association of Australia (1977). - Australian Standard 2082-1977. Visually Stress-Graded Hardwood for Structural Purposes. Standards Association of Australia, Sydney, Australia.

Standards Association of Australia (1978). - Document DR 78051. Draft Australian Standard Specification for Structural Plywood. Standards Association of Australia, Sydney, Australia.

Standards Association of Australia (1979). - Australian Standard 1684-1979. SAA Timber Framing Code. Standards Association of Australia, Sydney, Australia.

Standards Association of Australia (1980). - Strength Grouping of Timber. Miscellaneous Publication No. 45, Standards Association of Australia (In Press).

Suparman Karnasndirdja, Bakir Ginoga and Osly Rachman (1978). - Strength Classification of Wood Based on the Relationship Between Modulus of Rupture and Other Strength Properties. Laporan Lembaga Penelitian Hasil Hutan (Forest Products Research Institute) No. 115, Indonesia (English Translation).

Timber Research and Development Association (1976). - Grouping of Laos Timbers for a Community Building System for Laos. Part 3 of a Report of the United Nations Industrial Development Organisation. U.K., Feb. 1976.

United Nations Industrial Development Organisation (In preparation). - Low Cost Modular Prefabricated Wooden Bridges. United Nations Industrial Development Organisation, Vienna, Austria.

Wangaard, F.F. (1952). - The Physical Properties of Tropical Wood. TAAPI, Vol. 35, No. 4.

Ward, D.M. (1974). - Grouping of Nigerian Woods, According to General Properties and End Use Performance. Federal Department of Forestry, Ibadan, July 1974.

Ward, D.M. et al (1974). - Grouping of Nigerian Woods, According to General Properties and End Use Performance. Federal Dept of Forestry, Ibadan, Nigeria.

8. APPENDIX A. METHODS FOR STRENGTH GROUPING TIMBER

The following describes briefly the method used in Australia for classifying timber according to strength groups S1-S7 and SD1-SD8 (Standards Association of Australia 1980). These groupings are based on properties measured in mechanical tests on small clear specimens of wood (Mack 1979). Depending on the data available, several methods of assessment are given. If data on modulus of rupture, modulus of elasticity and compression strength for more than five trees are available, then the average values are used to make a preliminary classification according to Tables 14 and 15. If all these three properties do not lead to the same preliminary classifications, then Rules are given for choosing the appropriate classifications. The basis of Tables 14 and 15 have been described by Pearson (1965) and Kloot (1973).

Another useful method of classification given in the same document is one that may be used when there is data only on density. For this case the classification is made as indicated in Tables 16 and 17. Obviously this method should and will lead to more conservative classifications than the method described previously which is based on structural properties.

TABLE 14

PRELIMINARY CLASSIFICATION VALUES FOR GREEN TIMBER

Property	Minimum Species Means						
	S1	S2	S3	S4	S5	S6	S7
Modulus of rupture (MPa)	103	86	73	62	52	43	36
Modulus of elasticity (MPa)	16300	14200	12400	10700	9100	7900	6900
Maximum crushing strength (MPa)	52	43	36	31	26	22	18

TABLE 15

PRELIMINARY CLASSIFICATION VALUES FOR SEASONED TIMBER

Property	Minimum Species Means							
	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Modulus of rupture (MPa)	150	130	110	94	78	65	55	45
Modulus of elasticity (MPa)	21500	18500	16000	14000	12500	10500	9100	7900
Maximum crushing strength (MPa)	80	70	61	54	47	41	36	30

TABLE 16

MINIMUM AIR-DRY DENSITY VALUES FOR ASSIGNING SPECIES TO GREEN STRENGTH GROUPS

Strength Group	S1	S2	S3	S4	S5	S6	S7
Air-dry density at 12% moisture content - kg/m^3	1180	1030	900	800	700	600	500

TABLE 17

MINIMUM AIR-DRY DENSITY VALUES FOR ASSIGNING SPECIES TO SEASONED STRENGTH GROUPS

Strength Group	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Air-dry density at 12% moisture content - kg/m^3	1200	1080	960	840	730	620	520	420

9. APPENDIX B. LOAD FACTORS IN AUSTRALIAN STANDARDS

9.1 General

Load factors cannot be considered in isolation from other factors (such as design loads) specified in design Standards and consequently some care must be exercised in comparing the load factors used in various countries. In Australia, basic design values of structural properties are obtained by applying load factors or material coefficients, to characteristic values obtained in short term laboratory tests that last roughly five minutes. The following equation describes the relationship between these three quantities:

basic design value = characteristic value/load factor

In design Codes it is stated that the design strengths for a five minute load duration are to be obtained by multiplying the basic design strength by a factor of 1.75. Hence the true load factors implied in the Australian Codes are roughly 1/1.75 = 0.57 times the nominal values of load factors given in the following sections.

9.2 Load Factors for Visually Graded Timber

For timber assessed through tests on small clear specimens (Mack 1979), the appropriate load factors used are given in Table 18.

9.3 Load Factors for In-Grade Tests on Structural Lumber

This refers to in-grade tests of lumber where each stick is tested at the worst defect and, in the case of bending tests, with that defect on the tension edge. The basic design stresses in bending B^* and tension T^* are given by

$$B^* = \frac{B_{0.05} \times 1.15}{1.75\ (1.2 + 1.4\ V_B)}$$

$$T^* = \frac{T_{0.05}}{1.75\ (1.2 + 1.4\ V_T)}$$

where $B_{0.05}$ and $T_{0.05}$ denote the 5-percentile strength values, and V_B and V_T are the coefficients of variation of the measured bending and tension strengths respectively. If tests are made only on a single population of timber for a particular species, then a contingency factor of 0.9 on B^* and T^* is used to allow for the occurrence of possible regional effects.

9.4 Load Factors for Mechanically Stress Graded Lumber

The basic design stress in bending is given by

$$B^* = B_{0.05}/2.35$$

9.5 Load Factors for Pole Timbers

Load factors for pole timbers assessed from mechanical tests on small clear specimens are taken to be the same as those for structural lumber as given in Table 18 with an effective grade factor of 0.94. No form factor relative to the use of a round section is to be used in design computations.

9.6 Load Factors for Plywood

Load factors for plywood assessed from mechanical tests on small clear specimens are taken to be roughly the same as those for structural lumber as given in Table 18 with the addition that the load factor for in-plane shear is taken to be 6.4 on the shear-block strength. Associated factors to account for the geometry of the plywood lay-up are given in AS 1720-1975.

9.7 Load Factors for Metal Connectors

The load factors specified in the Australian Standard 1649-1974 (Standards Association of Australia 1974) are given in Table 19.

TABLE 18

CHARACTERISTIC STRUCTURAL PROPERTIES AND LOAD FACTORS FOR STRUCTURAL LUMBER ASSESSED FROM TESTS ON SMALL CLEAR SPECIMENS

DESIGN PROPERTY FOR STRUCTURAL LUMBER	CHARACTERISTIC VALUE MEASURED ON SMALL CLEAR SPECIMENS	LOAD FACTOR
Tension strength	one-percentile of F_b'	3.17/GF
Bending strength	one-percentile of F_b'	2.22/GF
Compression strength parallel to grain	one-percentile of F_c'	1.67/GF
Compression strength	Mean limit of proportionality in compression perpendicular to the grain test	1.33
Shear strength of beams	mean F_v'	4.2/GF
Shear strength of joint details	mean F_v'	4.7
Modulus of Elasticity	mean	$(0.75/GF)^{0.5}$

Note 1 F_b', F_c' and F_v' are ultimate strengths in bending, compression and shear in tests on small clear specimens.

Note 2 GF = grade factor =

$$\frac{\text{bending strength of structural scantling containing maximum permissible defect}}{\text{bending strength of small clear specimen cut from scantling}}$$

The following are typical grade factors used for sawn timber in Australian grading rules:

- Structural Grade No. 1: GF = 0.75
- Structural Grade No. 2: GF = 0.60
- Structural Grade No. 3: GF = 0.48
- Structural Grade No. 4: GF = 0.38

TABLE 19

CHARACTERISTIC STRENGTH AND MATERIAL COEFFICIENTS FOR METAL FASTENERS ASSESSED FROM SHORT DURATION LABORATORY TESTS

TYPE OF LOAD	TYPE OF FASTENER	CHARACTERISTIC VALUE	LOAD FACTOR
All	All	Mean ultimate strength of fastener metal	2.0
All	All	Mean Yield of fasteners metal	1.67
Withdrawal	Nails	one-percentile of max. loads	2.0
Withdrawal	Screws	one-percentile of max. loads	2.5
Lateral	Nails, Screws, Staples	one-percentile of max. loads	4.15
		one-percentile of loads at slip of 0.4 mm	1.25
Lateral	Split rings bolts	one-percentile of max. loads	2.8
		average of max. loads	4.0
Lateral	Toothed plate	one-percentile of max. loads	2.5
		one-percentile of loads at slip of 0.8 mm	1.6
Lateral	Nailed plate	one-percentile of max loads	4.3
		one-percentile of loads at slip of 0.8 mm	1.6

Note 1 Where two sets of characteristic values and material coefficients are cited, the set to be used is that leading to the smaller design working load.

Note 2 Slip refers to displacement between the members connected

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STRENGTH CLASSES FOR BRITISH STANDARD BS 5268

by

J R Tory

Princes Risborough Laboratory

UNITED KINGDOM

BORDEAUX, FRANCE

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STRENGTH CLASSES FOR BRITISH STANDARD BS 5268

The British Code of Practice for structural timber, CP 112:Part 2, is to be revised to form part of a new British Standard, BS 5268. Included in the draft proposals for the new code is a series of strength classes to cover the range of softwoods and hardwoods used in the United Kingdom. This paper explains the procedure that was adopted to establish the provisional stresses for the classes. Only softwood properties were considered as those for hardwoods have yet to be formally placed before the code committees.

For some time there has been a significant body of opinion in the timber industry and in the code committees in the United Kingdom in favour of simplifying the specification of structural timber. At the moment the designer has more than 270 species/grade combinations from which to choose, each with different stress values. Although this number will be reduced by the new national grading standards[1,2] it will still remain at about 160 because of the need to provide for the use of imported timber graded to other national rules.

If simplification is to be achieved the only alternatives are to group species, or to develop a system of strength classes. In the current code CP 112:Part 2:1971 a system of softwood species groups is included, as reproduced in Table 1. This has the disadvantage that some grades within a species group can be of greater strength than inferior grades within a superior group and yet they may be excluded from use by the specification. A strength class system, taking into account both species and grade would avoid this problem. It also offers advantage with the increasing use of machine grading and the opportunity that affords for selecting timber to target stress values. Strength classes have therefore been proposed to supersede species groups.

A complete reappraisal of timber stresses has also been undertaken for the introduction of BS 5268 and it is these revised material properties which were considered to establish the strength class system and which are used in this paper. This revision of stresses showed that whether or not there was to be a strength class system, it would be possible to assign the same stresses to European redwood, European whitewood and British grown Douglas fir, Scots pine and larch. (It may be of interest to note that in the move towards international harmonisation the properties proposed for BS 5268 will be based on fifth percentile values rather than on first percentiles as in CP 112.)

Any system of species and/or grade grouping must inevitably introduce penalties for some of the constituent parts of the group. But to optimise on efficiency of use of timber it is important that the penalties be kept to a minimum, particularly for the high volume species. Initially an attempt was made to develop a rational set of class boundaries which would give constant percentage increments of span for the design of beams under uniformly distributed loads. And this initial attempt was naturally centred on the grade stresses for SS and GS redwood/whitewood, 7.4 and 5.1 N/mm^2. However such a rigid framework, coupled with the lack of flexibility in defining stresses for coarsely incremented visual grades, introduced unacceptable penalties for many of the species/grades. It was decided that a more subjective choice of class boundaries taking into account some of the commercial difficulties of operating such a system and giving preferential consideration to a small number of species/grades could produce a more acceptable system. The 'important' species/ grades were considered to be:-

European redwood/whitewood	-	SS GS
Hem-fir (Canadian)	-	Light framing; Construction and Standard
Spruce-pine-fir (Canadian)		Structural light framing; Select and No 2
		Joists and Planks; Select and No 2

In addition British grown Sitka spruce was given more prominence than its relatively small volume would justify. A listing of these primary species/grades, with strength properties ranked on the basis of bending stress, is given in Table 2. The stresses given for the Light Framing and Structural Light Framing grades are for nominal 2 x 4 inch (actual 38 x 89 mm) cross-section. The ranking, and therefore the initial class allocation, could equally well have been on the basis of modulus of elasticity or any other property but the use of bending stress is considered to produce the most viable system.

The choice of the number of strength classes was an arbitrary one; too many and the objective of rationalisation would have been partially lost; too few and the penalties for some of the species/grades would have become unacceptable. Examination of Table 2 suggested that a sensible division of the primary species/grades could be achieved by setting bending stress class limits at 7.3, 5.1 and 3.9 N/mm^2. These limits would produce reasonable increments between beam spans in design calculations and limit penalties to tolerable levels. The class limits for the other properties were taken as the minimum values within the classes except where the penalties incurred by the whole class would have been relatively large. For example: The inclusion of GS Sitka spruce in the class suggested by its bending stress would have reduced the class E(minimum) from 4100 to 3600; E(mean) from 7700 to 6700 and compression parallel to the grain from 4.8 to 4.1 N/mm^2. It was therefore preferable to move GS Sitka spruce to a lower class than to heavily penalise several other species/grades.

Consolidation of the class boundaries and the extension of them to provide classes C1, C2, C3, C4 and C5 was achieved by consideration of the more complete listing of softwood species and grades reproduced in Table 3. The proposed class stresses are given in Table 4; the values for classes C6, C7 and C8 being tentative figures to make provision for the inclusion of hardwoods. Since it is at present proposed that for all species, grades and classes tension stress will be taken as 60 per cent of the bending stress the tension stresses have not been included.

To reduce further some of the penalties inherent in this strength class system some marginal relaxations were permitted in assigning species/grade combinations to the classes. To assign a species/grade to a strength class the bending stress had to be greater than or equal to the class bending stress. In addition E(minimum), E(mean), compression parallel to the grain and shear had to exceed 95 per cent of the appropriate class value. No account was taken of compression perpendicular to the grain for the purposes of assigning a class. On this basis softwoods graded to the BS 4978 grades are assigned to the classes indicated in Table 4. Similar tables have been drawn up for timber graded to the North American rules and these will appear in BS 5268.

In establishing the strength classes no account was taken of the machine grades which are now extensively used in the United Kingdom. This was because of the improved flexibility of machine grading relative to visual grading. Changes in machine limits to grade for nominated bending strengths are easily introduced and the penalties to be incurred by machine graded timber may therefore be kept to a minimum. It is anticipated that if strength classes are accepted for BS 5268 the existing machine grades may be dispensed with in favour of grading directly for classes. This more precise selection will not be possible for visual grading.

Two disadvantages of the strength class system have become apparent in the discussion for the introduction of BS 5268. First; although the designer may in many cases specify timber more easily he must revert to the specification of individual species if he requires particular characteristics such as durability or amenability to preservative treatment. Second; the definition of joint performance for strength classes presents problems since some joints are dependent on a particular property which may be independent of class, and since any species may be graded to any class it could be considered that there is no variation in joint performance with class.

These problems are currently being considered by the British code committee. They must also consider whether the mixing of species and species combinations is to be permitted within individual specifications for strength classified timber.

It is recognised that the class stresses established for BS 5268 will not by themselves provide an internationally acceptable strength class system. The more important grades and species vary from country to country and naturally each country will wish to minimise the penalties of a system for their high volume material. A workable international system is more likely to evolve from a large number of classes, seperated by small penalty increments, from which each country would select the few classes appropriate to their species and grades. However, since the relativities between properties varies between species such a system will not necessarily be easily established.

REFERENCES

1 BS 4978: Timber Grades for Structural Use. British Standards Institution.

2 BS 5756: The Grading of Tropical Hardwoods for Structural Purposes. British Standards Institution.

TABLE 1: SOFTWOOD SPECIES GROUPS (CP 112:Part 2)

SPECIES GROUP	STANDARD NAME	ORIGIN
S1	Douglas fir - larch Douglas fir Pitch pine	Imported
	Douglas fir Larch	British
S2	Western hemlock Western hemlock (commercial) Hem-fir Spruce-pine-fir Parana pine Redwood Whitewood Canadian spruce	Imported
	Scots pine	British
S3	European spruce Sitka spruce	British
	Western red cedar	Imported

Table 2: PRIMARY SPECIES/GRADES: RANKED ON THE BASIS OF BENDING STRESS

SPECIES AND GRADE*	BENDING (N/mm^2)	E(MINIMUM) (kN/mm^2)	E(MEAN) (kN/mm^2)	COMPRESSION PARALLEL (N/mm^2)	SHEAR (N/mm^2)
HEM - FIR (CAN) SLF SEL	10.6	8.2	11.2	9.3	0.71
S - P - F (CAN) SLF SEL	9.2	6.5	9.4	8.3	0.68
HEM - FIR (CAN) J&P SEL	9.1	8.2	11.2	8.3	0.71
S - P - F (CAN) J&P SEL	7.9	6.5	9.4	7.3	0.68
RED/WHITEWOOD SS	7.4	6.5	9.8	7.3	0.82
HEM - FIR (CAN) SLF No 2	7.4	7.3	9.9	5.9	0.71
HEM - FIR (CAN) SS	7.3	7.8	10.6	7.8	0.71
S - P - F (CAN) SLF No 2	6.5	5.8	8.3	5.2	0.68
S - P - F (CAN) SS	6.3	6.2	8.9	6.9	0.68
HEM - FIR (CAN) J&P No 2	6.2	7.3	9.9	6.2	0.71
SITKA SPRUCE (HG)** SS	5.6	5.0	7.7	6.0	0.64
HEM - FIR (CAN) LF CNST	5.4	6.8	9.2	6.7	0.71
S - P - F (CAN) J&P No 2	5.4	5.8	8.3	5.5	0.68
RED/WHITEWOOD GS	5.1	5.6	8.5	5.0	0.82
HEM - FIR (CAN) GS	5.1	6.8	9.2	5.4	0.71
S - P - F (CAN) LF CNST	4.7	5.4	7.7	5.9	0.68
S - P - F (CAN) GS	4.4	5.4	7.7	4.8	0.68
SITKA SPRUCE (HG) GS	3.9	4.4	6.7	4.1	0.64
HEM - FIR (CAN) LF STD	3.0	6.3	8.6	5.5	0.71
S - P - F (CAN) LF STD	2.6	5.0	7.2	4.9	0.68

*Grades

1 Canadian and American

Structural Light Framing;	Select
	Number 1
	Number 2
	Number 3
Light Framing;	Construction
	Standard
	Utility
	Stud
Joists and Planks;	Select
	Number 1
	Number 2
	Number 3

2 United Kingdom, BS 4978 — Select Structural; General Structural

**HG = British grown.

SPECIES & GRADE	DRY (18%) STRESSES				
	BENDING	E(MINIMUM)	E(MEAN)	COMPRESSION PARALLEL	SHEAR
	(N/mm^2)	(kN/mm^2)		(N/mm^2)	
DOUG FIR-LARCH (CAN) SLF SEL	13.7	9.5	13.4	11.0	0.93
SOUTHERN PINE (USA) SLF SEL	12.8	7.5	11.7	10.7	0.98
DOUG FIR-LARCH (USA) SLF SEL	12.6	8.1	12.0	10.5	0.85
DOUG FIR-LARCH (CAN) SLF #1	11.7	9.0	12.7	8.8	0.93
DOUG FIR-LARCH (CAN) J&P SEL	11.7	9.5	13.4	9.7	0.93
SOUTHERN PINE (USA) SLF #1	10.9	7.1	11.1	8.5	0.98
SOUTHERN PINE (USA) J&P SEL	10.9	7.5	11.7	9.5	0.98
DOUG FIR-LARCH (USA) J&P SEL	10.8	8.1	12.0	9.3	0.85
DOUG FIR-LARCH (USA) SLF #1	10.7	7.7	11.3	8.4	0.85
HEM - FIR (CAN) SLF SEL	10.6	8.2	11.2	9.3	0.71
DOUG FIR-LARCH (CAN) J&P #1	10.0	9.0	12.7	8.8	0.93
HEM - FIR (USA) SLF SEL	10.0	6.2	9.9	8.5	0.68
DOUG FIR-LARCH (CAN) SLF #2	9.6	8.4	11.8	6.9	0.93
DOUG FIR-LARCH (CAN) SS	9.4	9.0	12.7	9.2	0.93
SOUTHERN PINE (USA) J&P #1	9.3	7.1	11.1	8.5	0.98
S - P - F (CAN) SLF SEL	9.2	6.5	9.4	8.3	0.68
HEM - FIR (CAN) J&P SEL	9.1	8.2	11.2	8.3	0.71
DOUG FIR-LARCH (USA) J&P #1	9.1	7.7	11.3	8.4	0.85
HEM - FIR (CAN) SLF #1	9.0	7.8	10.6	7.4	0.71
SOUTHERN PINE (USA) SLF #2	9.0	6.6	10.4	6.7	0.98
WESTERN WHITE (USA) SLF SEL	8.9	5.5	8.3	6.4	0.66
DOUG FIR-LARCH (USA) SLF #2	8.8	7.2	10.6	6.6	0.85
SOUTHERN PINE (USA) SS	8.8	7.1	11.1	9.0	0.98
DOUG FIR-LARCH (USA) SS	8.6	7.7	11.3	8.8	0.85
HEM - FIR (USA) J&P SEL	8.6	6.2	9.9	7.5	0.68
PARANA PINE SS	8.5	7.3	10.4	9.8	1.02
HEM - FIR (USA) SLF #1	8.5	5.9	9.4	6.7	0.68
DOUGLAS FIR (HG) SS	8.4	6.7	10.3	8.6	0.88
DOUG FIR-LARCH (CAN) J&P #2	8.0	8.4	11.8	7.3	0.93
S - P - F (CAN) J&P SEL	7.9	6.5	9.4	7.3	0.68
LARCH (J+E) (HG) SS	7.8	5.0	9.1	7.8	0.93
S - P - F (CAN) SLF #1	7.8	6.2	8.9	6.6	0.68
HEM - FIR (CAN) J&P #1	7.7	7.8	10.6	7.4	0.71
WESTERN WHITE (USA) J&P SEL	7.6	5.5	8.3	5.7	0.66
WESTERN WHITE (USA) SLF #1	7.5	5.2	7.9	5.1	0.66
SOUTHERN PINE (USA) J&P #2	7.5	6.6	10.4	7.2	0.98
RED/WHITEWOOD SS	7.4	6.5	9.8	7.3	0.82
HEM - FIR (CAN) SLF #2	7.4	7.3	9.9	5.9	0.71
HEM - FIR (CAN) SS	7.3	7.8	10.6	7.8	0.71
DOUG FIR-LARCH (USA) J&P #2	7.3	7.2	10.6	7.0	0.85
HEM - FIR (USA) J&P #1	7.3	5.9	9.4	6.7	0.68
CORSICAN PINE (HG) SS	7.0	5.4	8.9	7.3	0.83
DOUG FIR-LARCH (CAN) LF CNST	7.0	7.8	11.0	7.9	0.93
HEM - FIR (USA) SLF #2	7.0	5.5	8.8	5.3	0.68
HEM - FIR (USA) SS	6.9	5.9	9.4	7.1	0.68
S - P - F (CAN) J&P #1	6.7	6.2	8.9	6.6	0.68
DOUG FIR-LARCH (CAN) GS	6.6	7.8	11.0	6.4	0.93
S - P - F (CAN) SLF #2	6.5	5.8	8.3	5.2	0.68
SOUTHERN PINE (USA) LF CNST	6.5	6.1	9.6	7.7	0.98
EUROPEAN SPRUCE (HG) SS	6.4	5.6	8.3	6.2	0.77
DOUG FIR-LARCH (USA) LF CNST	6.4	6.6	9.8	7.5	0.85
WESTERN WHITE (USA) J&P #1	6.4	5.2	7.9	5.1	0.66
S - P - F (CAN) SS	6.3	6.2	8.9	6.9	0.68
HEM - FIR (CAN) J&P #2	6.2	7.3	9.9	6.2	0.71
WESTERN WHITE (USA) SLF #2	6.2	4.9	7.3	4.0	0.66

TABLE 3 (continued)

WESTERN RED CEDAR	SS	6.1	6.3	7.9	6.9	0.64
WESTERN WHITE (USA)	SS	6.1	5.2	7.9	5.3	0.66
SOUTHERN PINE (USA)	GS	6.1	6.1	9.6	6.2	0.98
PARANA PINE	GS	6.0	6.3	9.0	6.8	1.02
DOUG FIR-LARCH (USA)	GS	6.0	6.6	9.8	6.1	0.85
DOUGLAS FIR (HG)	GS	5.8	5.8	9.0	6.0	0.88
HEM - FIR (USA)	J&P #2	5.8	5.5	8.8	5.6	0.68
SITKA SPRUCE (HG)	SS	5.6	5.0	7.7	6.0	0.64
LARCH (J+E) (HG)	GS	5.5	4.3	7.9	5.4	0.93
HEM - FIR (CAN)	LF CNST	5.4	6.8	9.2	6.7	0.71
S - P - F (CAN)	J&P #2	5.4	5.8	8.3	5.5	0.68
DOUG FIR-LARCH (CAN)	SLF #3	5.3	7.3	10.3	4.2	0.61
WESTERN WHITE (USA)	J&P #2	5.2	4.9	7.3	4.3	0.66
RED/WHITEWOOD	GS	5.1	5.6	8.5	5.0	0.82
HEM - FIR (CAN)	GS	5.1	6.8	9.2	5.4	0.71
HEM - FIR (USA)	LF CNST	5.1	5.1	8.2	6.1	0.68
DOUG FIR-LARCH (CAN)	STUD	5.0	7.3	10.3	4.2	0.61
SOUTHERN PINE (USA)	SLF #3	5.0	5.7	9.0	4.1	0.64
CORSICAN PINE (HG)	GS	4.9	4.7	7.7	5.1	0.83
DOUG FIR-LARCH (USA)	SLF #3	4.9	6.2	9.2	4.0	0.56
HEM - FIR (USA)	GS	4.8	5.1	8.2	4.9	0.68
S - P - F (CAN)	LF CNST	4.7	5.4	7.7	5.9	0.68
SOUTHERN PINE (USA)	STUD	4.7	5.7	9.0	4.1	0.64
DOUG FIR-LARCH (CAN)	J&P #3	4.6	7.3	10.3	4.7	0.61
DOUG FIR-LARCH (USA)	STUD	4.6	6.2	9.2	4.0	0.56
EUROPEAN SPRUCE (HG)	GS	4.5	4.8	7.2	4.3	0.77
WESTERN WHITE (USA)	LF CNST	4.5	4.5	6.8	4.6	0.66
S - P - F (CAN)	GS	4.4	5.4	7.7	4.8	0.68
WESTERN RED CEDAR	GS	4.3	5.5	6.9	4.8	0.64
WESTERN WHITE (USA)	GS	4.3	4.5	6.8	3.7	0.66
SOUTHERN PINE (USA)	J&P #3	4.3	5.7	9.0	4.5	0.64
DOUG FIR-LARCH (USA)	J&P #3	4.2	6.2	9.2	4.4	0.56
HEM - FIR (CAN)	SLF #3	4.1	6.3	8.6	3.6	0.47
SITKA SPRUCE (HG)	GS	3.9	4.4	6.7	4.1	0.64
DOUG FIR-LARCH (CAN)	LF STD	3.9	7.3	10.3	6.5	0.93
HEM - FIR (CAN)	STUD	3.9	6.3	8.6	3.6	0.47
HEM - FIR (USA)	SLF #3	3.9	4.8	7.6	3.3	0.45
HEM - FIR (USA)	STUD	3.7	4.8	7.6	3.3	0.45
HEM - FIR (CAN)	J&P #3	3.6	6.3	8.6	3.9	0.47
S - P - F (CAN)	SLF #3	3.6	5.0	7.2	3.2	0.45
DOUG FIR-LARCH (USA)	LF STD	3.6	6.2	9.2	6.2	0.85
SOUTHERN PINE (USA)	LF STD	3.6	5.7	9.0	6.3	0.98
S - P - F (CAN)	STUD	3.4	5.0	7.2	3.2	0.45
HEM - FIR (USA)	J&P #3	3.4	4.8	7.6	3.6	0.45
WESTERN WHITE (USA)	SLF #3	3.4	4.2	6.4	2.5	0.44
WESTERN WHITE (USA)	STUD	3.3	4.2	6.4	2.5	0.44
S - P - F (CAN)	J&P #3	3.1	5.0	7.2	3.5	0.45
HEM - FIR (CAN)	LF STD	3.0	6.3	8.6	5.5	0.71
WESTERN WHITE (USA)	J&P #3	3.0	4.2	6.4	2.7	0.44
HEM - FIR (USA)	LF STD	2.8	4.8	7.6	5.0	0.68
S - P - F (CAN)	LF STD	2.6	5.0	7.2	4.9	0.68
WESTERN WHITE (USA)	LF STD	2.5	4.2	6.4	3.8	0.66
DOUG FIR-LARCH (CAN)	LF UTIL	1.8	7.3	10.3	4.2	0.61
DOUG FIR-LARCH (USA)	LF UTIL	1.7	6.2	9.2	4.0	0.56
SOUTHERN PINE (USA)	LF UTIL	1.7	5.7	9.0	4.1	0.64
HEM - FIR (CAN)	LF UTIL	1.4	6.3	8.6	3.6	0.47
HEM - FIR (USA)	LF UTIL	1.3	4.8	7.6	3.3	0.45
S - P - F (CAN)	LF UTIL	1.2	5.0	7.2	3.2	0.45
WESTERN WHITE (USA)	LF UTIL	1.2	4.2	6.4	2.5	0.44

TABLE 3: SOFTWOOD SPECIES/GRADES RANKED ON THE BASIS OF BENDING STRESS

Table 4

CLASS STRESSES AND MODULI OF ELASTICITY FOR THE DRY EXPOSURE CONDITION
(N/mm^2)

Strength class	Bending parallel to the grain	Tension parallel to the grain	Compression parallel to the grain	Compression perpendicular to the grain	Shear parallel to the grain	Modulus of Elasticity	
						Mean	Minimum
C1	2.5	1.5	3.3	2.1	0.47	6700	4400
C2	3.9	2.3	4.8	2.1	0.66	7700	4900
C3	5.1	3.1	5.0	2.3	0.68	8500	5600
C4	7.3	4.4	7.3	2.4	0.68	9400	6500
C5	10.8	6.5	8.8	3.3	0.89	11600	7100
C6	15.0	9.0	11.3	4.5	1.03	12300	7900
C7	21.4	12.8	14.1	6.3	1.22	14400	9000
C8	30.6	18.4	17.0	9.0	1.40	16200	10000

Table 5

SOFTWOOD SPECIES IN BS 4978 GRADES WHICH SATISFY THE REQUIREMENTS FOR STRENGTH CLASSES

Standard name	Strength class			
	C1	C2	C3	C4
Imported				
Parana pine			GS	SS
Redwood			GS	SS
Whitewood			GS	SS
Western red cedar	GS	SS		
Douglas fir-larch (Canada)			GS	SS
Douglas fir-larch (USA)			GS	SS
Hem-fir (Canada)			GS	SS
Hem-fir (USA)		GS	SS	
Spruce-pine-fir (Canada)		GS	SS	
Western white woods (USA)	GS	SS		
Southern pine (USA)			GS	SS
British grown				
Douglas fir			GS	SS
Larch	GS	SS		
Scots pine			GS	SS
Corsican pine		GS	SS	
European spruce	GS		SS	
Sitka spruce	GS	SS		

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WORKING COMMISSION W18 - TIMBER STRUCTURES

LOAD-CARRYING CAPACITY AND DEFORMATION CHARACTERISTICS OF NAILED JOINTS

by

J Ehlbeck

University of Karlsruhe
FEDERAL REPUBLIC OF GERMANY

BORDEAUX, FRANCE

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Load-Carrying Capacity and Deformation Characteristics of Nailed Joints

by

J. Ehlbeck

Introduction

Nailed joints are well-known since hundreds of years. New developments of numerous types of nails, wood-base materials (e.g. plywood, particleboard, fiberboard), other board materials (e.g. gypsumboard, asbestos-board) as well as new methods of assembling using nailing machines raise new questions on the scientific basis of judging the load-carrying capacity as well as the load-deformation behavior of nailed joints. This is of great importance in order to provide conditions and suppositions for the development in the field of mechanical fasteners at present and in future. Only if such basic information is available, the technical progress can be utilized by being translated into international and national timber design codes.

The manifold possibilities of application of nailed joints, e.g. in industrial as well as domestic buildings with different requirements of the deformations, in interior as well as exterior application with different climatic influences, or with small as well as large forces to be joined with different joint configurations, makes it necessary to study the basic performance of nailed joints using a "standard joint". The results must be accomplished and modified by systematic research of additional factors influencing the joint's performance, such as the influence of moisture content of the wood and the climatic changes of the environment as well as influences of long-term loading.

The Load-Slip Curve

In principle the load-slip curve, as presented in Fig. 1, describes the load-deformation characteristics of a nailed joint. The slip, s, is the relative movement of the members assembled. An initial slip, s_I, can be observed during the test performance and is caused by displacements of the nail shank driven between the wood fibers during the first loading. A certain portion of this initial slip may also be caused by adjustments in the test set-up and by other test errors. The curve is, however, non-linear from the very beginning, i.e. at low loads. A distinct proportional limit being a welcome help for stipulating design loads cannot be observed. This fact indicates the problem of defining a slip modulus to predict or calculate deformations, even in the low load range. The slope of the first part of the curve or origin tangent describes an initial slip modulus, k_o, which is a characteristic value of the load-slip curve,underestimating,however, the actual deformations. A secant slip modulus may exactly describe the deformation at a certain load level, overestimating, however, the slip below this load level. Taking into account that all material properties in wood construction are of great variability, it becomes obvious that slip moduli tabulated in timber design codes are only roughly estimated values. This should be realized when more and more sophisticated methods of analysis and design are used, which may not be justified if "constants" of great variability are introduced into the calculation.

An extremely important point of the load-slip curve appears to be the load level beyond which permanent joint creep occurs. This point of a certain "flow limit" is not yet well known. A definition was provided by NORÉN (1968) using that load beyond which it does not increase more than 10 % at a slip increment of 1 mm per member.

After plastic deformation of the nail shank, the nail is stressed in direction of the nail axis in addition to being loaded perpendicular to the nail axis. This effect can be observed from the load-slip curve where any inflection of the curve reflects a stiffening of the joint. The amount of this additional load-transmission capability depends primarily on the withdrawal resistance of the nail and the friction between the adjacent members which are pressed together. This phenomenon is called the "chain-effect" or "sag-effect". It is difficult to separate from the total load-transmission that part of the transmitted load which is the result of this "chain-effect". It depends on the friction between the adjacent members. The amount of this frictional force depends, on the other hand, on the pressure with which the members are forced together as well as on the coefficient of sliding friction decreasing with increase in slip and pressure. Nevertheless, the additional load-carrying capacity due to this "chain-effect" is of significance for nails providing a relatively high withdrawal resistance, i.e. for effectively plastic-coated or threaded nails.

The ultimate load, F_u, is observed only at a relatively large slip. It has been proposed that a practical and meaningful ultimate load be defined at that load where the slip between adjacent members does not exceed a certain maximum value, such as 7.5 mm. This definition may, however, not suit to the modern safety concepts.

As a distinct proportional limit is difficult to assign to the load-slip curve, an agreement may be reached to define this point (see Fig. 2) as being located at a small value of residual slip, $s_{ir,y}$. This proportional limit can be a basis for determining a secant slip modulus, k, for the lowest load range. Such a slip modulus varies with the stipulated residual slip value, $s_{ir,y}$, and may coincide with the flow limit discussed above.

Factors of Influence on Joint Performance

The performance of joints is influenced by a large number of variables. These variables make it complicated to predict the load-carrying capacity and the load-deformation behavior of the joint. It is not possible to describe the joint performance as indicated by the load-slip curve using a comprehensive mathematical formula including all factors of influence. Some of the most important factors are:

a) the material of which the nail is made and the nail's shape and dimensions:

   size, length, diameter,
   point, head, and thread dimensions,
   clean, smooth, or rough surface,
   plating, galvanizing, or other coating,
   nail stiffness and flexural properties;

b) the properties of the fastened and fastening members:
compressive and embedding strength,
elastic and creep moduli,
displacement modulus and elastic or plastic bearing constant,

(All these properties are related to density, grain direction and moisture content)
friction between fastener and its surrounding material,
relaxation ;

c) the joint configuration:
single and multiple nails per joint,
single, double, or multiple shear,
thickness of members,
distances of the fastener from the member sides and ends,
nail spacing,
predrilling,
nails driven into side-grain or end-grain wood,
depth of nail penetration,
clinching of protruding nail points;

d) the loading conditions:
static, repetitive, or dynamic loading,
short or long-term loading,
rate and range of loading,
time interval between nail driving and load application.

In order to describe the performance of nailed joints their load-carrying capacity as well as their deformation characteristics under service conditions must be considered. It would be unsufficient to take into consideration only the ultimate load or only the slip in the low load range for stipulating basic design values.

Load-Carrying Capacity

Using a "standard joint" which is a single-shear common round-wire nail joint with the nails driven into solid wood of equilibrium moisture content in normal climate (e.g. 20°C temperature and 65 % relative humidity) the load-carrying capacity is essentially controlled by

the embedding strength of the wood,
the bending resistance of the nail,

and the dimensions of the joint, i.e. the thickness of the members and the diameter of the nail.

The embedding strength, f_e, is defined as the ultimate pressure, p_u, per unit length of the nail, divided by the nail diameter, d_n:

$$f_e = \frac{p_u}{d_n} = \frac{F_u}{t \cdot d_n} \tag{1}$$

where t = member thickness and F_u = ultimate load.
This is a fictitious strength value; consequently varying with the member thickness and the nail diameter. The bending resistance of the nail is described by the yield moment, M_y, which is the product of the yield strength, f_y, of the nail material and the section modulus, W_p, when the nail is fully plastic:

$$M_y = f_y \cdot W_p \tag{2}$$

The mutual relationship of these parameters are decisive for the joint's ultimate load-carrying capacity. Neglecting the "chain-effect" and the load transmission by friction, which is difficult to analize because of the wood shrinkage during seasoning, MOELLER (1951) developed formulae predicting the ultimate load. He did not take into account any deformation characteristics. The validity range of these formulae, as shown in Fig. 3 for single-shear nailed joints (i.e."standard joints") indicates, that for small member thicknesses only the embedding strength controls the ultimate load, whereas the bending resistance of the nail is not used to advantage. Only with the member thickness, t_1, of the fastened member being

$$t_1 > 3.4 \sqrt{\frac{M_y}{f_e \cdot d_n}} \tag{3}$$

the load-carrying capacity reaches a maximum value, with

$$F_u = \sqrt{2 \cdot M_y \cdot f_e \cdot d_n} \tag{4}$$

With $W_p = d_n^3/6$ for circular cross-sections, Eq.4 reads:

$$F_u = \sqrt{\frac{f_y \cdot f_e}{3}} \cdot d_n^2 \tag{5}$$

Thus, the load-carrying capacity of a nailed joint can be determined by using a formula presented in the 4th.draft of the CIB Structural Timber Design Code of CIB-W18, June 1979, as f_y and f_e are dependent on the nail diameter, d_n:

$$F_u = k_{nail} \cdot d_n^{\alpha_{nail}} \qquad (6)$$

As a rule, $\alpha_{nail} \leqq 2$ for round-wire nails. For threaded nails this value must be investigated. k_{nail} is primarily dependent on the wood species, its properties, and the joint configuration.

In the German code DIN 1052, at present

$$k_{nail} = \nu \cdot \frac{500}{10 + d_n} \quad \text{and} \quad \alpha_{nail} = 2 \qquad (7a,b)$$

with ν = safety factor, d_n in mm. Thus, for common round-wire nails in European softwood the basic design load, F_d, is:

$$F_d = \frac{500}{10 + d_n} \cdot d_n^2 \quad \text{(in N)} \qquad (8)$$

with d_n in mm.

For nails with high shank withdrawal resistance, Eq.5 may be modified by a factor taking into account the additional load-transmitting capability caused by the "chain-effect".

Deformation Characteristics

It cannot be justified to disregard deformation during load transmission and to refer only to the ultimate load-carrying capacity in efforts to establish design loads. Under stipulated design loads for different types of nails as well as different member materials the deformation of the joints may be different and even of bad influence on the performance of the complete structure. Therefore, the load-slip behavior must be studied and limited slip values under service conditions must be laid down. Such values should, however, be differentiated according to the use and service conditions of the structure. Presuming a sufficient safety against the characteristic ultimate load the basic design loads can be modified for different deformation requirements.

Studies covering the load-slip behavior of nailed joints were performed based on different assumptions; yet an all-embracing theory has not been presented to-date. In the main, two kinds of investigation are known: the one using an elastic theory with the nail acting as a beam on elastic foundation, the other using empirical methods.

Elastic theory presumes a straight line of the load-slip relationship. Using the stiffness of the nail, $E_n \cdot I_n$, with E_n = modulus of elasticity of the nail and I_n = moment of inertia of the nail, and a displacement modulus, K, of the wood which is defined as the pressure per unit length of the nail divided by the displacement, δ, the slip modulus, k, (see Fig.2) can be calculated

$$k = \frac{F}{s} = \frac{K}{4 \cdot \lambda \cdot \psi} \tag{9}$$

with

$$\lambda = \sqrt[4]{\frac{K}{4 \cdot E_n \cdot I_n}} \tag{10}$$

λ is a characteristic value and ψ is a "specific slip", a function of λ and the geometric values of the joint, primarily the member thicknesses (see NORÉN, 1968). For large member thicknesses the specific slip approaches unity.

Hence, the non-linearity of the load-slip curve (see Fig.1) is a result of the non-linearity of the load-displacement relationship. The displacement modulus, K, decreases with increase in load, F. More research efforts need to be made to investigate this relationship for all types of materials used for nailed joints.

The empirical methods used in recent years refer either to the load-displacement relationship or directly to the load-slip curve in the lower load range. For certain joints the deformation characteristics are studied using multiple tests and best fitting mathematical formulae found by means of statistical procedures. The results of these studies available in literature include the disadvantage of referring to special joint configurations tested by making use of different test procedures. Consequently, they cannot be generalized indiscriminantly. On the other hand, these methods can deliver sufficient data to estimate secant slip moduli for different slip levels. A comparison of some empirical studies performed independently on each other indicate a certain relationship between secant slip moduli for different slip values, as is shown in Fig.4. A slip modulus, $k_{1.0}$, based on a 1.0 mm-slip, turned out to be approximately twice the slip modulus, $k_{2.5}$, derived from a 2.5 mm-slip, and so forth.

On the basis of this concept it is possible to develop design values (allowable loads) from the characteristic ultimate load and to stipulate corresponding slip moduli.

Conclusion and Recommendation

A reliable judgement of nailed joints has to take into consideration a plurality of influencing factors. One all-embracing formula to describe the joint's performance is not feasible. It is not reasonable to evaluate design data only on the basis of the ultimate load-carrying capacity. On the other hand, it is wrong to orientate design loads only by means of slip limitations, particularly because the service conditions of structures differ considerably. Consequently, for design codes both the ultimate characteristic load and the deformation characteristic under service conditions should be considered carefully.

The variety of nails developed to-date and the increasing number of building materials suitable to be assembled by nails call for intensified research efforts, especially to determine the material properties needed, such as the bending stiffness and flexural resistance of the nails as well as the displacement moduli and the embedding strenghts of the members. Such investigations must include all influencing parameters, e.g. density, grain direction, long-term loading etc.

Furthermore, nailed joints must be tested to determine additional factors, such as the "chain-effect" when using plastic-coated or threaded nails, creep influences, or repetitive and dynamic loading.

All research efforts should be carried out on the basis of an international coordination as well as on uniform methods of testing and analyzing test data. By this way may be developed both general but uniform design rules for international codes, flexible for further technical evolution, and specified rules in national codes taking into consideration regional conditions.

Literature Cited

1. CIB - W 18. 1979. Structural Timber Design Code, 4th Draft, June 1979

2. Fachnormenausschuß Bauwesen im Deutschen Normenausschuß. 1969. DIN 1052 Teil 1, Holzbauwerke, Berechnung und Ausführung

3. Möller, T. 1951. En ny metod foer beraekning av spikfoerband. Chalmers Tekniska Hoegskolars Handlingar No. 117. Göteborg, Sweden

4. Norén, B. 1968. Nailed Joints - Their Strength and Rigidity under Short Term and Long Term Loading. The National Swedish Institute for Building Research, Report No.22.

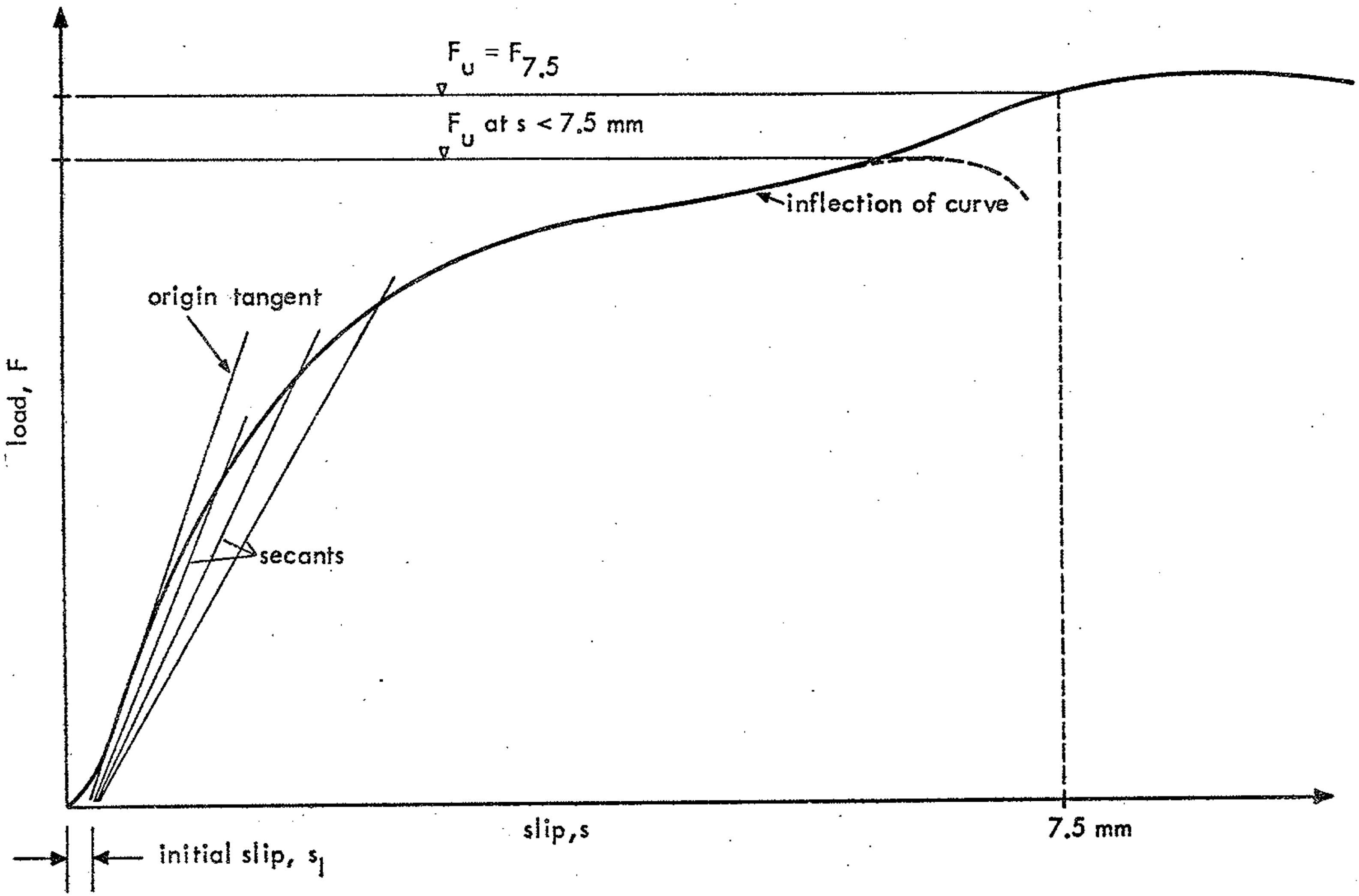

Fig. 1. - Load-slip curve for nailed joint.

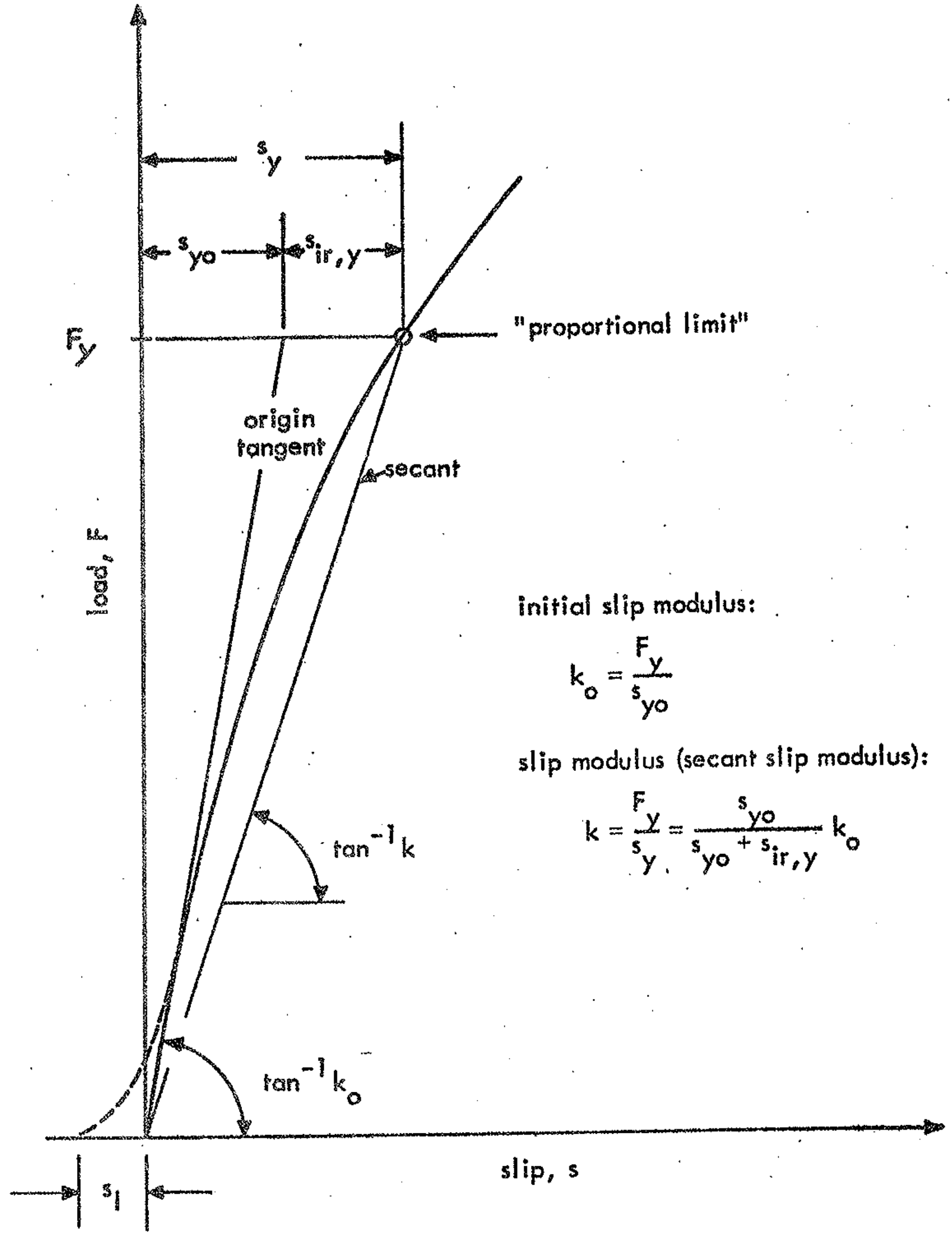

Fig. 2.- Definition of "proportional limit", F_y, of nailed joint.

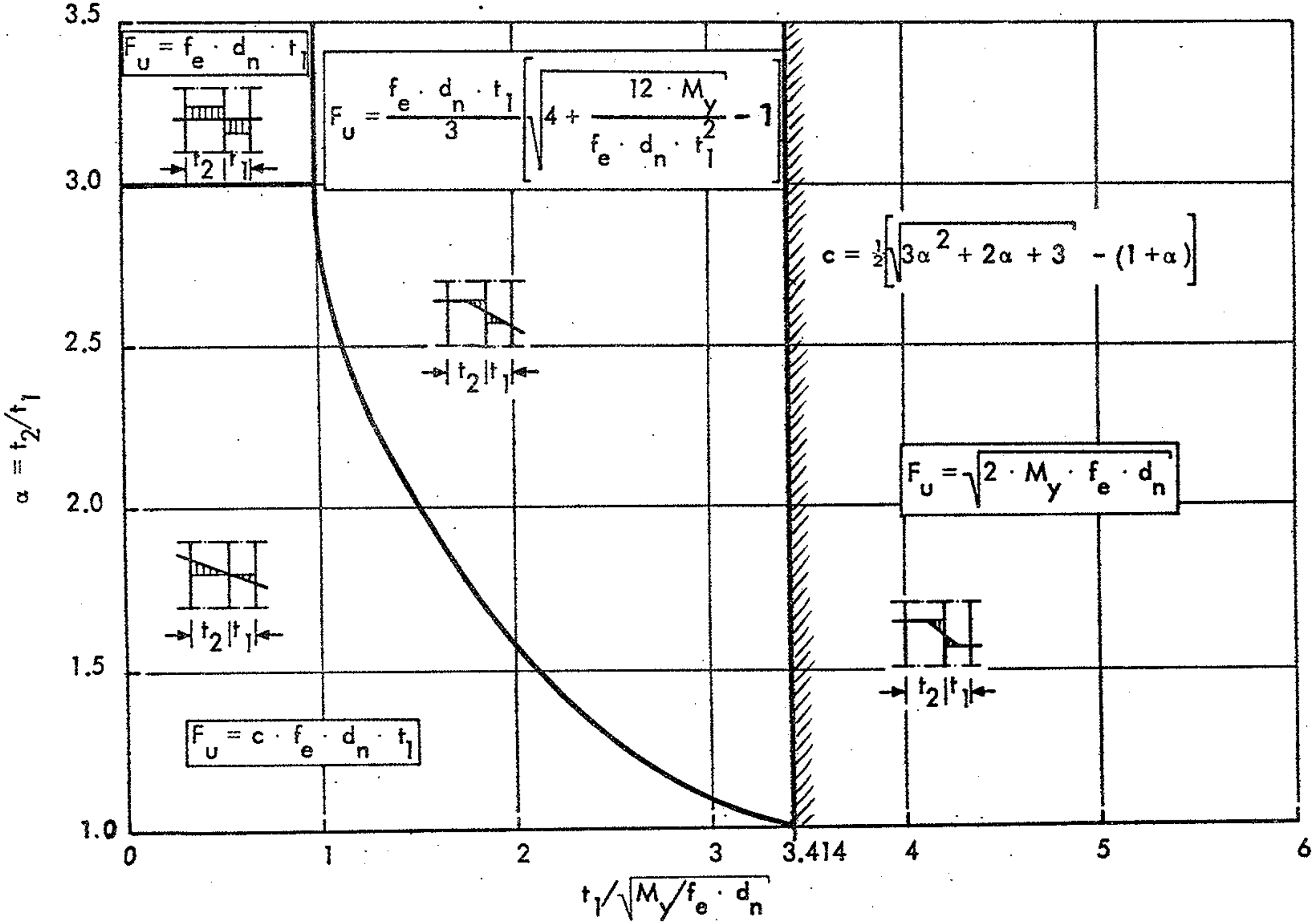

Fig. 3. – Validity range of formulae predicting ultimate load, F_u, of single-shear nailed joints (Moeller, 1951).

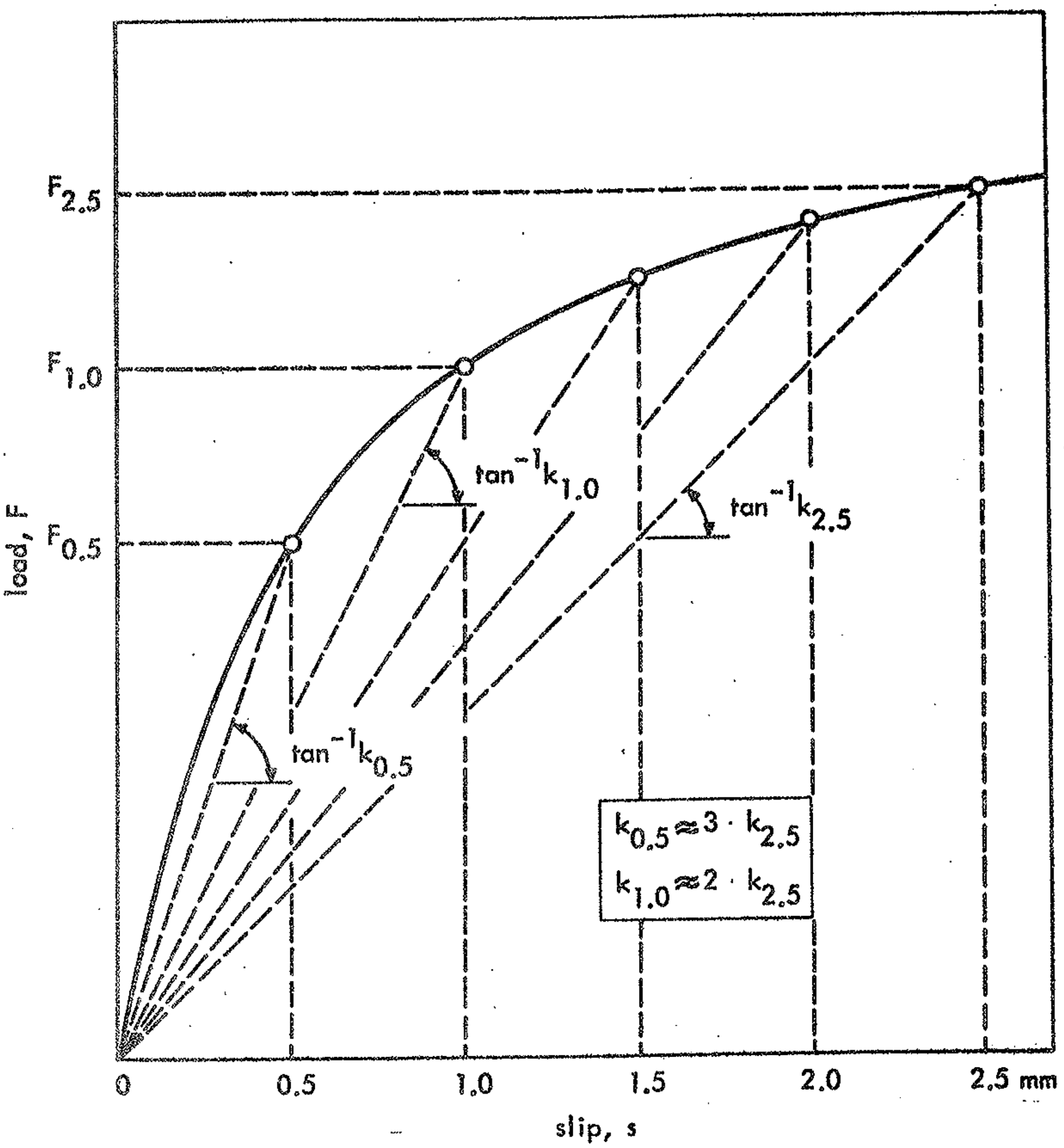

Fig. 4. – Schematic load-slip curve and secant slip moduli for different slip values.

CIB-W18/12-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF BOLTED JOINTS

by

H J Larsen

Aalborg University Centre
DENMARK

BORDEAUX, FRANCE
OCTOBER 1979

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1. INTRODUCTION

The theoretical background for the CIB-rules concerning laterally loaded bolts, screws and also nails is given, and the influence of different parameters are demonstrated. The theory was set up based on tests, but apart from them only a few test results are of use since they don't give the yield load and the material parameters needed, and moreover, they don't cover the range of basic variables.

The approximations given in the CIB-Code are mentioned and compared with the theoretical expressions.

2. THEORY

The rules in the fourth draft of CIB-Structural Timber Design Code [1] for laterally loaded bolts and screws are based on a theory originally proposed by Johansen [5] and later modified and extended by among others Möller [13] and Meyer [12].

The theory is based on the theory of plasticity and is thus aimed at the ultimate limit states and not the serviceability limit states. There is at present no theory for the prediction of the deformations in joints, and the necessary information is usually obtained by testing.

The basic assumptions are that the load-deformation curves are stiff-plastic for bending of the bolt as well as for the impression of a stiff bolt into the wood, cf. Figure 1.

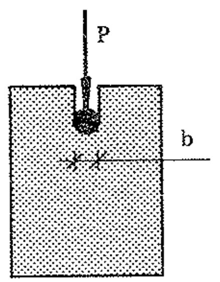


Figure 1

The yield moment of the bolt is

$$M_y = f_y W_p \tag{1}$$

where f_y is the yield stress and W_p the plastic section modulus. For a circular cross-section

$$W_p = d^3/6 \tag{2}$$

where d is the diameter.

The ultimate impression load $F = F_{ult}$ per unit width is written as

$$F_{ult} = f_{emb} \cdot d \tag{3}$$

where f_{emb} is a fictitious strength value called the embedding strength.

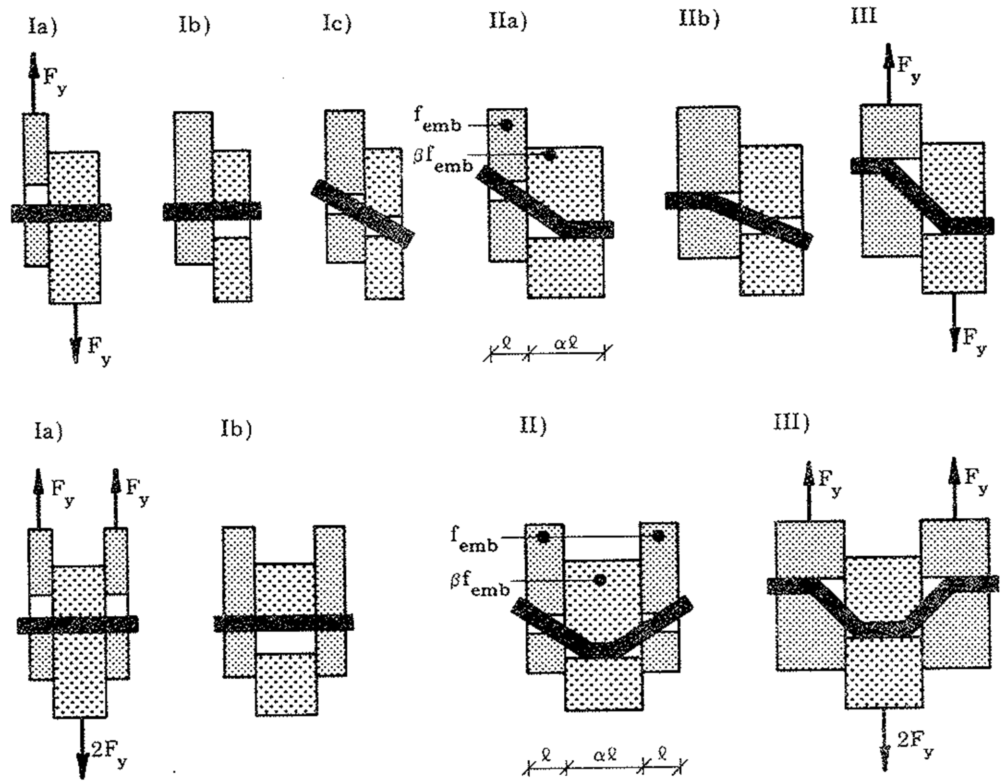


Figure 2

The possible failure modes for two-member joints and symmetrical three-member joints are shown in Figure 2.

The thicknesses are ℓ and $\alpha\ell$, and the embedding strengths f_{emb} and βf_{emb}, cf. Figure 2. For a two-member joint $\alpha \geq 1$.

The ultimate load-carrying capacities are given below. For a detailed derivation [7] is referred to. Note that the load-carrying capacity for a three-member joint is per shear.

$$\gamma = f_y / f_{emb} \tag{4}$$

Two-member joints:

$$\frac{F_y}{f_{emb}\,d\ell} = \min.\begin{cases} 1 & \text{Ia} \quad (5) \\ \alpha\beta & \text{Ib} \quad (6) \\ \dfrac{\sqrt{\beta + 2\beta^2(1+\alpha+\alpha^2)+\alpha^2\beta^3} - \beta(1+\alpha)}{1+\beta} & \text{Ic} \quad (7) \\ \sqrt{\dfrac{2\beta(1+\beta)}{(2+\beta)^2} + \dfrac{2\beta}{3(2+\beta)}\,\gamma\left(\dfrac{d}{\ell}\right)^2} - \dfrac{\beta}{2+\beta} & \text{IIa} \quad (8) \\ \sqrt{\dfrac{2\beta^2(1+\beta)}{(2\beta+1)^2}\,\alpha^2 + \dfrac{2\beta}{3(2\beta+1)}\,\gamma\left(\dfrac{d}{\ell}\right)^2} - \dfrac{\beta\alpha}{2\beta+1} & \text{IIb} \quad (9) \\ \dfrac{d}{\ell}\sqrt{\dfrac{2}{3}\,\gamma\,\dfrac{\beta}{1+\beta}} & \text{III} \quad (10) \end{cases}$$

Three-member joints (per shear):

$$\frac{F_y}{f_{emb}\,d\ell} = \min.\begin{cases} 1 & \text{Ia} \quad (11) \\ \alpha\beta/2 & \text{Ib} \quad (12) \\ \text{As formula (8)} & \text{II} \quad (13) \\ \text{As formula (10)} & \text{III} \quad (14) \end{cases}$$

The formulas are illustrated in Figures 3-8.

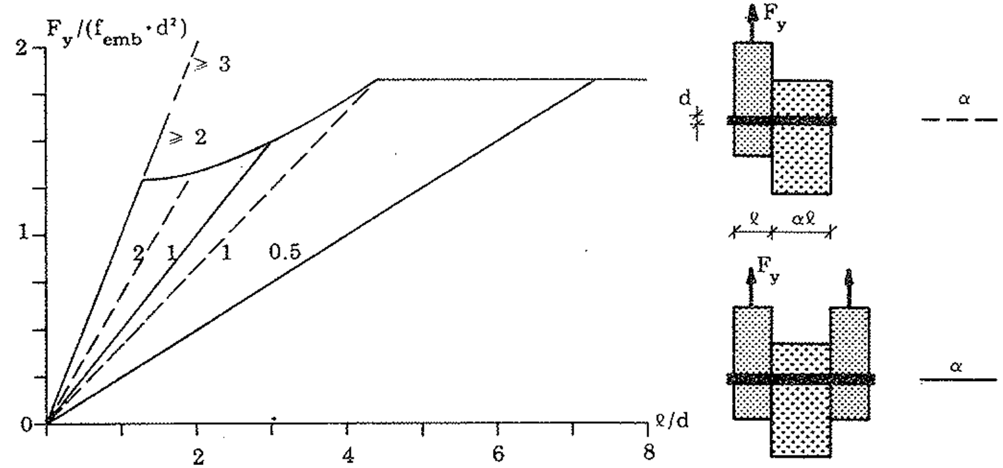


Figure 3

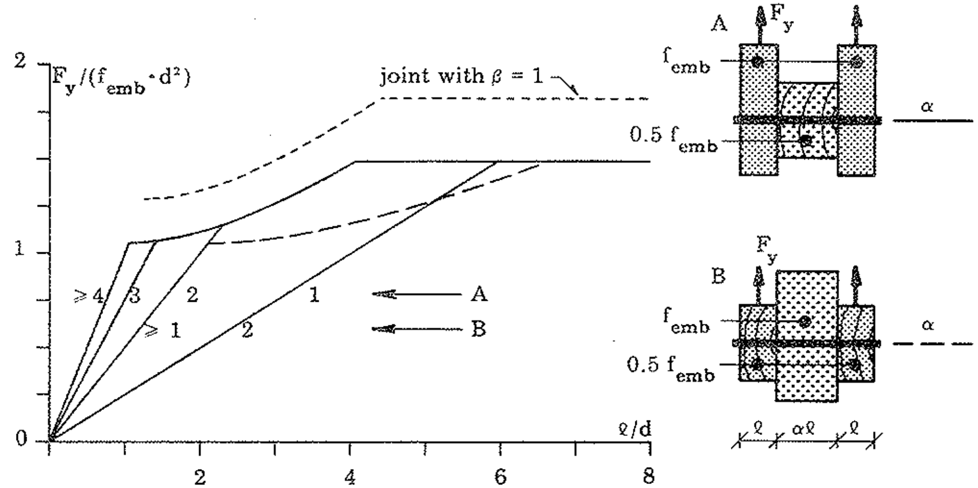


Figure 4

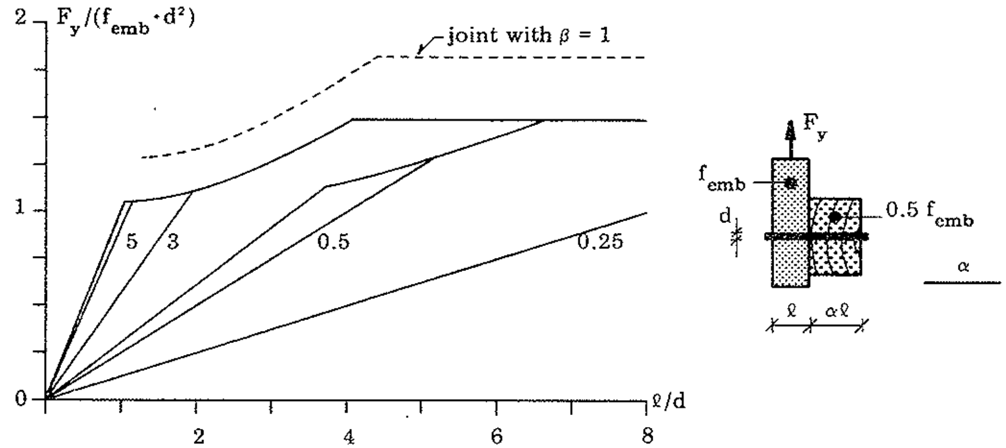


Figure 5

Figure 3 shows the general dependence of F_y on the bolt slenderness ratio ℓ/d in loading parallel to the grain in all members ($\beta = 1{,}0$). A value of $\gamma = 10$ has been assumed corresponding to ordinary bolts with $f_y = 240$ MPa and Nordic softwood with $f_{emb} \sim 24$ MPa. Figure 4-5 show F_y for load perpendicular to the grain in one of the members of two-member joints and in the side members or the middle member of a three-member joint, and parallel to the grain in the others. A value of $\beta = 0.5$ has been assumed. To make the values comparable a change of notations has been made for the case with load perpendicular to the grain in the side members.

Figure 6 shows the influence of the ratio γ for a three-member joint with $\beta = 1$. The ordinate in Figure 6a is $F_y/(f_{emb} d^2)$, in Figure 6b $F_y/(\sqrt{f_{emb} f_y}\, d^2)$. For small ℓ/d-values the load-carrying capacity is independent of γ, for slender bolts the load-carrying capacity for a given value of f_{emb} is proportional to $\sqrt{f_y}$.

In Figure 7 the theoretical values are compared with the design rules given in German Standard DIN 1052 for ordinary softwoods. The German permissible stresses have been transformed to characteristic values by multiplication by 3.0. The theoretical curves are drawn with $f_{emb} = 25.5$ and $\gamma = 15$.

In Figure 8 the theoretical values are compared with the values proposed by Trayer [15] based on tests. The comparison has been made for the basic three-member joint, i.e. a joint with $\alpha = 2$, and for γ-values of 20 and 10, corresponding approximately to the species groups 1 and 3 according to the Trayer-grouping.

In the timber codes in USA, e.g. [2], the load-carrying capacity is related to the capacity of the basic three-member joint. Three-member joints with other α-values are reduced to the basic case by disregarding the oversize of the outer members (for $\alpha < 2$) or the middle member ($\alpha > 2$). Except for $\ell/d < 1$ this rule is obviously on the safe side, for slender bolts no reduction is necessary.

A basic two-member joint, i.e. a joint with two equal members of thickness ℓ, is assumed to correspond to a basic three-member joint with a thickness ℓ' of the side members, cf. Figure 9. According to the above-mentioned rules $\ell' = \ell$ is assumed. This rule has recently been questioned by Wilkinson [17], who in accordance with the original Trayer-rules proposes $\ell' = 0.5\ell$. The correct answer according to Figure 3 is $\ell' = \ell$ for $\ell/d > \sim 4$ and $\ell' = \frac{2}{3}\ell$ for $\ell/d < \sim 2$.

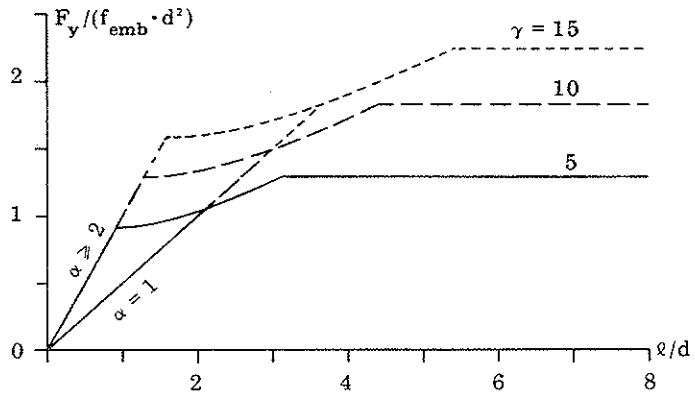
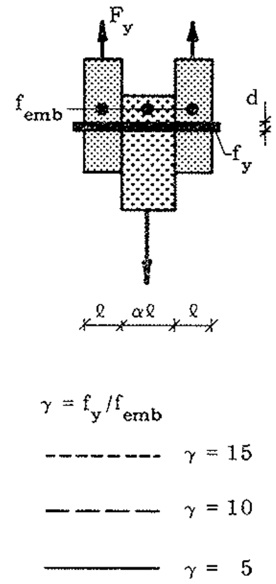


Figure 6a



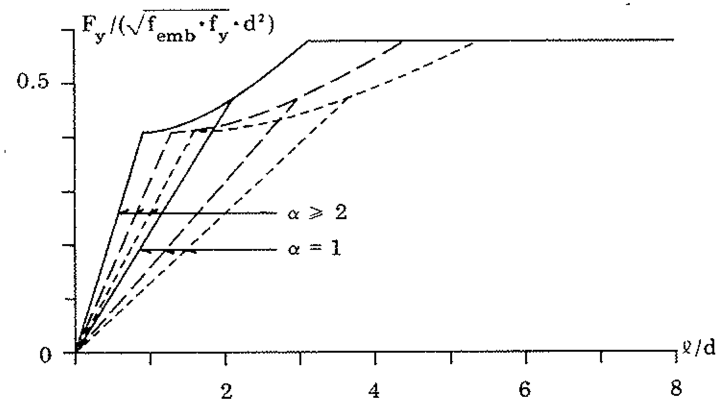


Figure 6b

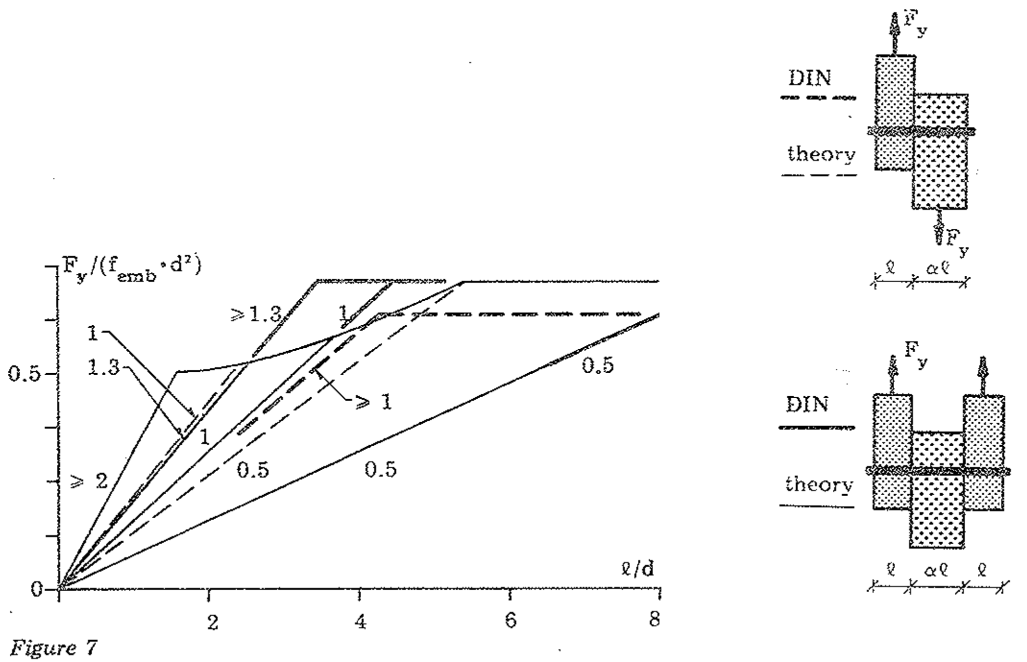


Figure 7

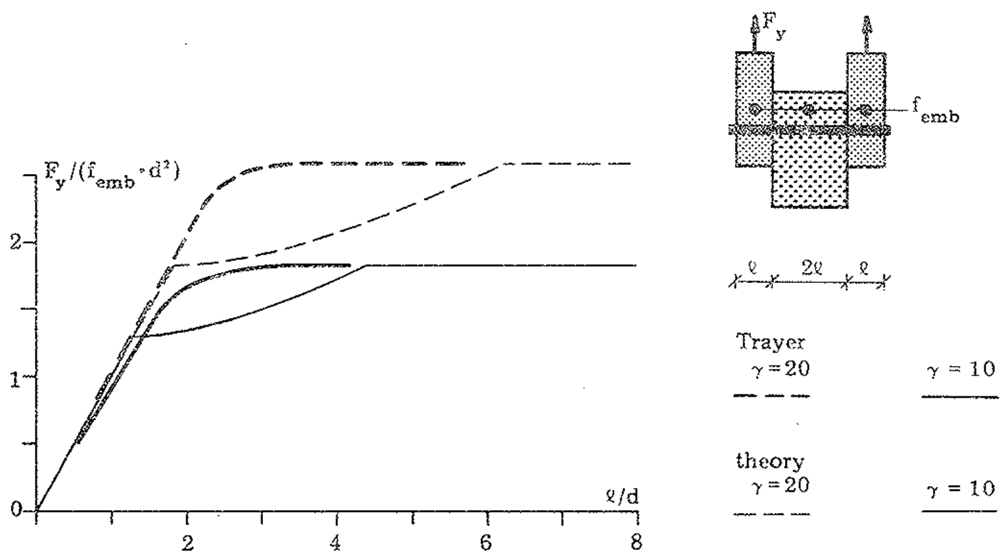


Figure 8

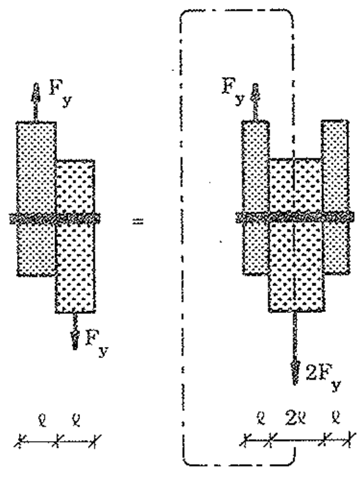


Figure 9

3. TEST

The number of tests that can be used to evaluate the theory is limited and it is recommended that a testing should be done on joints covering the range of the basic variables. Johansen based his theory on tests reported in [5]. Andersen & Granum [3] and Nielsen & Johansen [14] have investigated joints with bolts and different connectors but also a few series with bolts alone. Their conclusion is that there is good agreement between theory and test for small ℓ/d-ratios. For slender bolts the test values were higher - in some cases considerably higher - than the theoretical values due to influence from friction and from head and nut restraining the deformation of the bolt so failure mode III is found for too low ℓ/d-values.

Trayer made an extensive investigation [15] but since he only reports a proportional limit for the joints their use is limited in connection with a theory for the ultimate load-carrying capacity. His findings about the influence of bolt strength, compression strength of the wood and ℓ/d-ratios are generally in agreement with the theory. The problem for him and the US-rules based on his proposals is that he is not aware that some of his results are not general but only valid for certain ℓ/d-ranges.

The same applies to the tests reported by Wilkinson [17].

The theory has also been used for laterally loaded screws and nails and here a rather big test material exists in support of the theory, see among others Aune [4], Larsen [6], [7], Larsen & Reestrup [8], Mack [9], [10], [11], Meyer [12] and Van der Put [16].

4. CODE RULES

In the CIB Structural Timber Design Code approximative expressions are given:

$$F = \min. \begin{cases} 18\rho(k_1t_1 + k_2t_2)d & \text{(only for two-member joints)} \quad (15) \\ 35\rho k_2 t_2 d & \text{(only for three-member joints)} \quad (16) \\ 70\rho k_1 t_1 d & (17) \\ 42\sqrt{\rho}\, d^2 + 12\rho k_1 t_1 d & (18) \\ 75d^2 \sqrt{\rho}\, \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (19) \end{cases}$$

where

- t is timber thickness in mm
- d is the diameter in mm
- k is a factor, taking into consideration the influence of the angle between force and grain direction. For thick bolts ($d \geq 24$) and an angle of 90° $k \sim 0.5$
- ρ is the density.

In Figures 10 and 11 the theoretical expressions are compared with the approximative expressions for structural timber at least corresponding to SC 19 (i.e. ρ = 0.36). Where the curves coincide only the theoretical curves are shown.

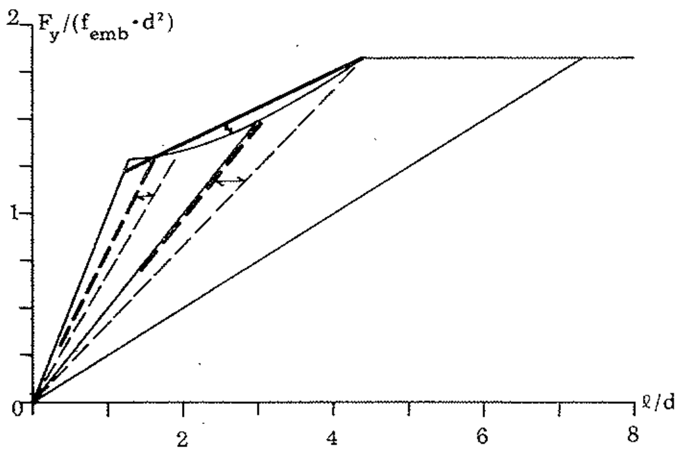


Figure 10. Joints as shown in Figure 3.

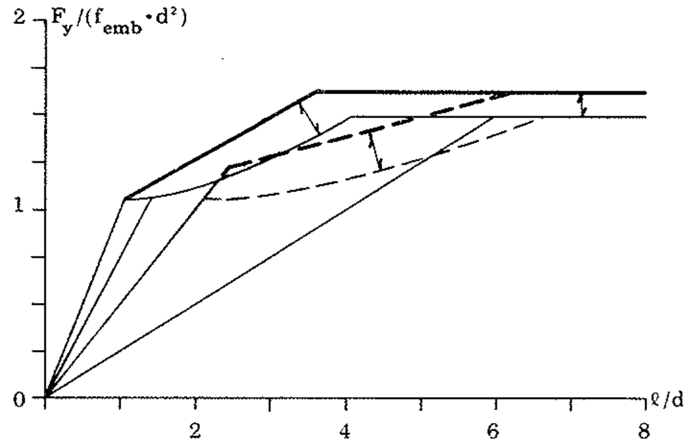


Figure 11. β = 0.5. Joints as shown in Figure 4.

The approximative formulas may seem a little complicated but are easy to use in practice. A simplification can be obtained by omitting formula (18) which could be justified by test result.

LITERATURE

[1] Conseil International du Batiment, Working Group 18: CIB-Structural Timber Design Code, 4. draft 1979.

[2] National Forest Products Association. National Design, Specification for Wood Wood Construction. 1977.

[3] Andersen, A. & Granum, H.: Forsøk med tømmerforbinderne Alligator, Bulldog, Rox og Stjerne. Norges Teknisk-Naturvidenskapelige Forskningsråd. Byggeteknisk Utvalg. Rapport Nr. 2, Oslo 1951.

[4] Aune, Petter: Trådstiftforbindelsers Bæreevne. Beregning og Forsøk (The Load Capacity of Nails, Theory and Tests). Norges tekniske Høgskole, Trondheim, 1966.

[5] Johansen, K. W.: Theory of Timber Connections. International Association for Bridge and Structural Engineering. Publ. Vol. 9, 1949.

[6] Larsen, H. J.: K. W. Johansen's Nail Tests. Bygningsstatiske Meddelelser. Copenhagen 1977.

[7] Larsen, H. J.: The Yield Load of Bolted and Nailed Joints. IUFRO-V, 1974. (Also in Report R52 from Structural Research Laboratory, Technical University of Denmark).

[8] Larsen, H. J. & Reestrup, V.: Tests on screws in wood. Bygningsstatiske Meddelelser, Copenhagen 1969.

[9] Mack, J. J.: The Strength of Nailed Timber Joints. I. In Messmate Stringybark. Commonwealth Scientific and Industrial Research Organization, Melbourne. Division of Forest Products Technological Paper No. 9, 1960.

[10] Mack, J. J.: The Strength of Nailed Timber Joints. II. Radiata Pine. As [9], but Paper No. 21, 1962.

[11] Mack, J. J.: The Strength and Stiffness of Nailed Joints under Short-Duration Loading. As [9], but Paper No. 40, 1966.

[12] Meyer, A.: Die Tragfähigkeit von Nagelverbindungen bei statischer Belastung. Holz als Roh- und Werkstoff, pp. 96-109. Bd. 15. 1957.

[13] Möller, T.: En ny metod för beräkning av spikförband. Chalmers Tekniska Högskolas Handlingar, No. 117, Göteborg 1951.

[14] Nielsen, T. F. & Johansen, M.: Forsøg med træforbindelser med bolte og mellemlæg. Statens Byggeforskningsinstitut. Copenhagen, Rapport 67. 1970. (Investigation of Timber Joints with Bolts and Connectors - with an English summary).

[15] Trayer, G. W.: The Bearing Strength of Wood under Bolts. United States Department of Agriculture. Technical Bulletin No. 332, 1932.

[16] Van der Put, T. A. C. M.: Proeven op stiftverbindingen in constructiehout. Stevin Laboratorium, Houtconstructies. Delft. Rapport 4-77-4. October 1977.

[17] Wilkinson, T. L.: Strength of bolted wood joints with various ratios of member thicknesses. Forest Products Laboratory, Madison. Research Paper FPL 314.

CIB-W18/12-7-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF JOINTS WITH NAIL PLATES

by

B Norén

Swedish Forest Products Research Laboratory
SWEDEN

BORDEAUX, FRANCE

OCTOBER 1979

SWEDISH FOREST PRODUCTS
RESEARCH LABORATORY
1979-10-09
BN/CE

DESIGN OF JOINTS WITH NAIL PLATES

B Norén Swedish Forest Products Research Laboratory

Introduction

For some time a small Nordic group has discussed guide lines for design of joints with nail plates in timber structures.
Members have been N-I Bovim, Norway, M Johansen, Denmark, B Norén, Sweden and T Poutanen, Finland.
Their draft proposal contain these sections:

1. Definition and symbols
2. Design calculations
3. Material
4. Manufacture and control
5. Testing of plates and joints

Section 5 is principally a reference to the testing standard proposed by NORDTEST and based on the RILEM/CIB recommendations.

The paper, presented here, is a rather free translation of section 2 in which is given rules for design calculations. The outline is kept from the proposal:

2.1 Model for calculation of joints
2.2 Design against plate failure
2.3 Design against plate grip failure
2.4 Design against wood failure
2.5 Rules for specific joints
2.6 Calculation of slip

However, the last subsection (2.6) has not been completed and is omitted here, also the rules for special joints (2.5) are incomplete.

The paper is presented to the CIB-W18 for information and discussion. The intention has not been to propose that it goes unabridged into the CIB code, section 6.1.5 Nail plates. In Sweden the proposal will be used as a base for revised "Approval Rules" concerning the design of nail plate joints.

1 DEFINITIONS AND SYMBOLS

1.1 Definitions

The structure consists of (wood) members and connections (joints) between the members. The presented design rules shall be applied within the connection boundaries which as a rule are specified cross sections of the members at some distance from their ends. The connections are designed to withstand the internal forces (axial and shear) and moments (bending and torsion) acting upon them at the boundary cross sections, as well as external forces (loads) within the boundaries. The design of the members is not dealt with here.

The forces and moments acting on the connections are determined by static calculation. For this calculation a static model of the structure is used, built of elements to describe the real members and imaginary elements to describe the behaviour of the connections.

Within the connections two types of joints (joint "lines") are defined: The plate/timber-joint (pt-joint) is the contact between the plate and the timber. The timber/timber-joint (tt-joint) is the direct joint between timbers. In general, the area of contact as well as the gap joint area are included in the definition.

1.2 Symbols

1.2.1 Directions

x- Joint direction

y- Direction of a perpendicular to the joint line in the centre of the plate sections. Positive direction is out from the studied member.

a- Specified principal direction of the plate. Positive direction is out from the studied member and part of plate.

b- Principal plate direction, perpendicular to the a-direction.

The a/b-cross is rotated the angle α from the x/y-cross. The origons of the two systems are coincident.

1.2.2 Angles

α Angle of x-direction to a-direction (y-direction to b-direction).

α_F Angle of x-direction to plate force (F_p).

α_R Angle of x-direction to joint force (R)

α_1 Angle of fibre direction to plate force (F_p)

α_2 Angle of plate force (F_p) to a-direction

α_3 Angle of fibre direction to a-direction

ϕ Angle of friction

ϕ Polar coordinate

1.2.3 Geometry

H Width (depth) of timber

B Thickness of timber

a Measure of plate in a-direction

b Measure of plate in b-direction

c Measure of plate

d Diagonal

d Distance from plate to end of joint

e Distance from plate to end of joint

l Length of plate section in joint line

(l Projection of plate/timber joint area)

r Division of l

h Length of timber/timber joint

$h = d + l + e$

e_{RC} Distance of force to centre of rotation

e_{CG} Distance of force to centre of gravity

A Joint area

ξ Length of constant stress

1.2.4 Forces and moments

R Connection force

M Connection moment

F_t Force on timber

F_p Force on plates

M_p Moment on plates

N Force-component in the y-direction

N_a, N_b Force -components in the a- and b-directions

Q Force-component in the x-direction (joint direction)

1.2.5 Stresses

σ_c (N/mm^2) Compression stress in wood

p or p_t (N/mm) Tension stress in plates

p_c Compression stress in plates

s (N/mm) Shear stress in plates

τ (N/mm^2) Shear stress in the plate/timber joint

1.2.6 Subscripts

p Plate

t Timber

t (Second subscript) tension

c Compression

a (Force or stress) in the a-direction

b (Force or stress) in the b-direction

D (Force or stress) design value

0 Fibre direction

90 Perpendicular to fibre direction

2 DESIGN CALCULATIONS

2.1 Model for calculation of joints

In the general case the forces are transmitted between the connected members partly by the plates through the plate/timber joints (pt-joints) and partly by compression and corresponding friction in the timber/timber joints (tt-joints). The proportions of these parts depend on the model chosen for the behaviour of the joints.
The model for calculation of joints should correspond reasonably with the model for the entire structure, especially with respect to deformations. Total correspondance will generally imply too complex models. The model must give forces and moments transmitted in the joints which are in equilibrium with the forces and moments acting at the connection boundaries, including possible loads. This demand is unconditional. The conditions for such equilibrium are demonstrated for a joint shown in Figure 2:1. The tt-joint AB and the corresponding cross-sections CD of the (two plates) are in the XZ-plane, the plates and member sides in planes parallel to the XY-plane.
The central line of the member shown in the figure cuts the joint perpendicular at E at a distance $(-X_S)$ from origon (0) which is central to the plate cross-sections CD. The force from connected members, acting on the member in E, is R and the bending moment is M (no torque). Part of that action takes place through the plates by the force F_p and moment M_p acting in 0, and the remaining part directly by a compression force $(-F_t)$ perpendicular to the tt-joint at a distance $(-X_T)$ from the y-axis and the correspondig friction force $(-\mu F_t)$.

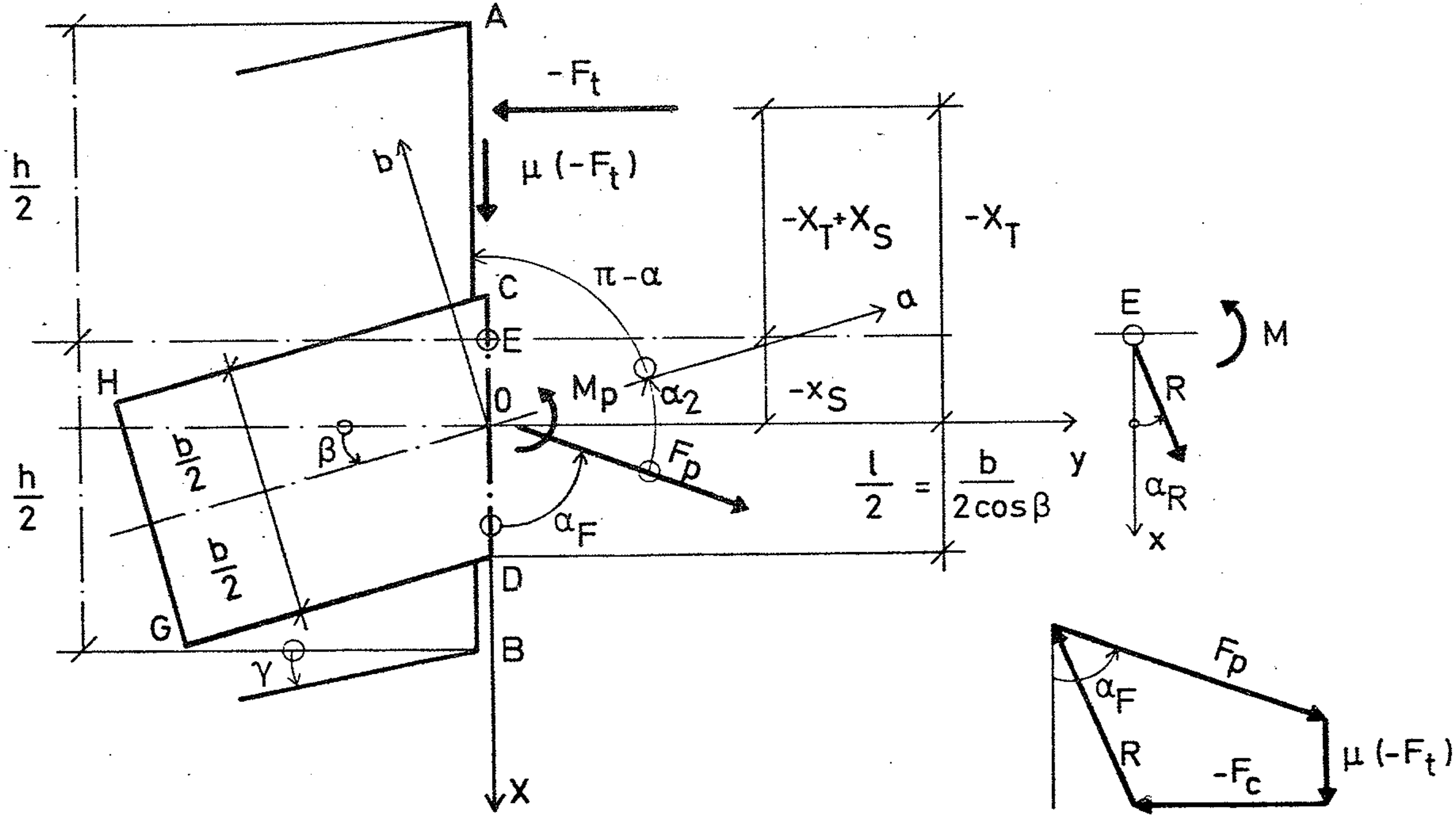


Figure 2:1 *Contact forces (t) and plate forces (p) generated by the force R and the moment M transmitted to the member.*

The following equations of equilibrium relate F_p, M_p and F_t to R and M:

$$F_p \sin \alpha_F + F_t = R \sin \alpha_R \tag{1}$$

$$F_p \cos \alpha_F - \mu F_t = R \cos \alpha_R \tag{2}$$

$$M_p - (-x_T) F_t = M - (-x_S) R \sin \alpha_R \tag{3}$$

Her α_R and α_F denote the angles of the forces R respectively F_p to the joint (x-axis). Five quantities (all but μ in the left terms are unknown. Hence, two additional equations are required). They are the "action-displacement" equations which in the model are replaced by equations appearing directly from assumptions on the distribution of the stresses on the wood member along AB and on the plates along CD.

When the capacity in the ultimate limit state is verified the model condition is simply that the force F_t, transmitted directly to the member, generates a constant compression stress σ_c (N/mm^2) over a length ξ_t of the tt-joint AB, while F_p generates a constant tension p (N/mm) in the cross section CD of each of the two plates. Other stresses in the y-direction (perpendicular to the joint) are not taken into account. Furthermore, the transmission of compression in the plates is neglected when there is contact between the timber members (6) and, naturally, the tt-joint is not supposed to transmit tension (7). With these assumptions the relations between forces and stresses in the joint section are

$$-F_t = \sigma_c \, B \, \xi_t = \sigma_c \, B(h + 2x_T - 2x_S) \tag{4}$$

$$F_p \sin \alpha_F = 2p \, \xi_p = 2p \, (b/\cos\beta - 2M_p/(F_p \sin\alpha_F)) \tag{5a}$$

$$F_p \cos\alpha_F = 2s\ \xi_s = 2sb/\cos\beta \tag{5b}$$

with the conditions

$$o \leq F_p \sin\alpha_F \leq pb/\cos\beta \tag{6}$$

$$0 \leq -F_t \leq \sigma_c Bh \tag{7}$$

The stress directions are perpendicular to the joint (compression in the timber, σ_c, N/mm^2, and tension in the plate, p, N/mm) respectively in the direction of the joint (shear in the plate, s, N/mm). It is purposeful to use these stresses if the criteria of failure and design values are based on them. So is the case in present Nordic codes. However, the design values will then be dependent on the angle to the fibre direction (σ_{cD}) and the angle to the principal directions of the plate (p_D and s_D). This complicates the prototype testing of the plates. Hence, in the following section the design conditions for the strength of the plate are based merely on design values in the a- and b- directions of the plate. F_p and α_F in equations (1) and (2) are replaced by the F_p-components N_a and N_b.

The following stresses are defined:

$$p_a = \frac{N_a}{2\xi_p \sin\alpha} \qquad \alpha = \pi/2 + \beta$$

$$p_b = \frac{N_b}{2\xi_p \cos\alpha} \qquad \xi_p = \frac{b}{\sin\alpha} - \frac{2M_p}{N_a \sin\alpha + N_b \cos\alpha}$$

$$s_a = \frac{N_a}{2b\cot\alpha}$$

$$s_b = \frac{N_b}{2b}$$

Design values for these stresses are given in the code or in certificates for plates of different types. There respective influence on the design of the joint appears from the design stipulations which are based on possible modes of failure in the plate depending on the pattern of holes.

The necessary equations when the design stipulations are given in terms of design values for the compression stress on the member perpendicular to the joint (contact area between members) and for stresses in the principal directions of the plates thus are:

$$N_a + F_t\ (\sin\alpha + \mu\cos\alpha) - R\cos(\alpha - \alpha_R) = 0 \qquad (1\ T)$$

$$N_b + F_t\ (\cos\alpha - \mu\sin\alpha) + R\ \sin(\alpha - \alpha_R) = 0 \qquad (2\ T)$$

$$M_p + X_T F_t - M - X_s\ R\sin\alpha_R = 0 \qquad (3)$$

Stresses (N/mm):

$$\sigma_c B = \frac{-\ F_t}{h + 2X_T\ -2X_s.} \qquad (4)$$

$$P_a = \frac{N_a/2}{b - 2M_p\sin\alpha/(N_a\sin\alpha + N_b\cos\alpha)} \qquad (5\ T)$$

P_b = ...

s_a = ...

s_b = ...

Limit values (design values) σ_{cD}, p_{aD} etc, are given for the stresses. In the general case σ_{cD} and at least one of the plate stress values are decisive for the joint design. When the stipulations include interaction between stresses, severel design values may have to be considered.

One complication, in spite of the assumptions of the block-distribution of stresses, is that the moment (M_p) and the forces (F_p or N_a and N_b) are implicated in the expression for the distribution length ξ_p. A simplification would be to assume that the compression stress (σ_c) in the tt-joint is constant until the neutral axis and that the same applies to the p-stresses in the plate to the limit set by the size of the plate. With this assumption, corresponding to full plastic response, ξ_p in (5) and (5T) is found from

$$\xi_p = b/\sin\alpha + d - \xi_t = b/\sin\alpha + d - (h + 2X_T - 2X_s) \qquad (8)$$

where $b/\sin\alpha$ (= $b/\cos\beta$) is the length of the plate cross section over the joint and d is the distance along the joint from the plate to the joint end at the compression side. A condition for applicability of (8) is $\xi_p \leq b/\sin\alpha$. When ξ_p reaches the limit value $b/\sin\alpha$, it means that the stress is constant all across the plate. After that the eq. (8) cannot be applied and need not be used anyway as the moment on the plate is zero ($M_p = 0$) which simplifies (5) and (5 T).

It should be observed that the eqs. (1) to (8) concern the conditions of the very joint and the cross sections of the plate along this joint. They apply irrespective of what the angles are between the joint and the principle directions of the plates or the fibre direction of the members. (Note that the quantity $b/\cos\beta$, introduced in (5) is nothing but the length of the part of the joint which is covered by the plates). These angles have no influence on the model, although they will be of great importance for the design values of the stresses or forces.

Also, the transformation to a new system of axis, by rotation the angle α from the x-y-system, make the transformation of (1), (2) and (5) into (1 T), (2 T) and (5 T) pure mathematical as long as the axis a and b are not referred to as the principle directions of the plates.

The conclusion is that the scarfe-joint of figure 2:2 and the two typical heel-joints of figure 2:3 can be treated by use of the equations given in section 2.1 and 2.2 with the introduced denotations.

In the example three cases of plate utilization are demonstrated. The fact that it is illustrated by a scarfe-joint is insignificant if the dimensions symbolized in the figure are regarded as belonging to the joint. If the joints in figure 2:3 or - for that matter - the joint shown in figure 2:1, are chosen for the illustration, nothing has to be changed in the following equations, except that b must be replaced by $\ell = b/\cos\beta = b\sin\alpha$ (or another symbol for the length CD).
In the example the simplifying assumption of "full plasticity" leading to (8) has been used. The components of the forces in the x- and y-directions have been denoted N respectively Q.

EXAMPLE

General

Symbols see figure 2:2

Forces affecting the joint:

$M = M \qquad N = R \sin \alpha_R \qquad Q = R \cos \alpha_R$

Forces, transmitted directly from timber to timber:

$N_t = \sigma_c B \xi_t \qquad Q_t = \mu N_t$

Forces transmitted by the plates:

$N_p = N + N_t \qquad Q_p = Q - Q_t$

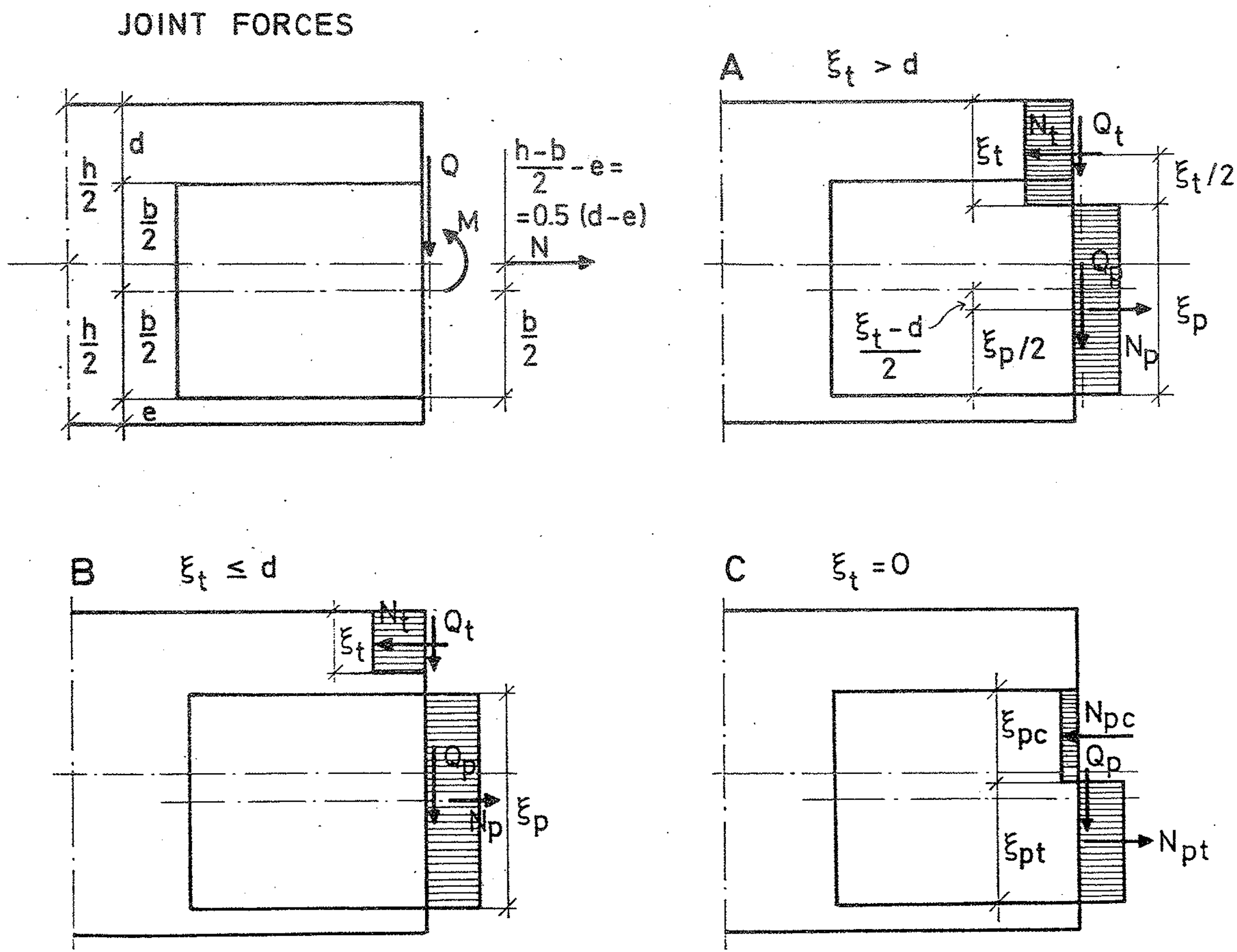


Figure 2:2 Distribution of forces in a lengthening joint.

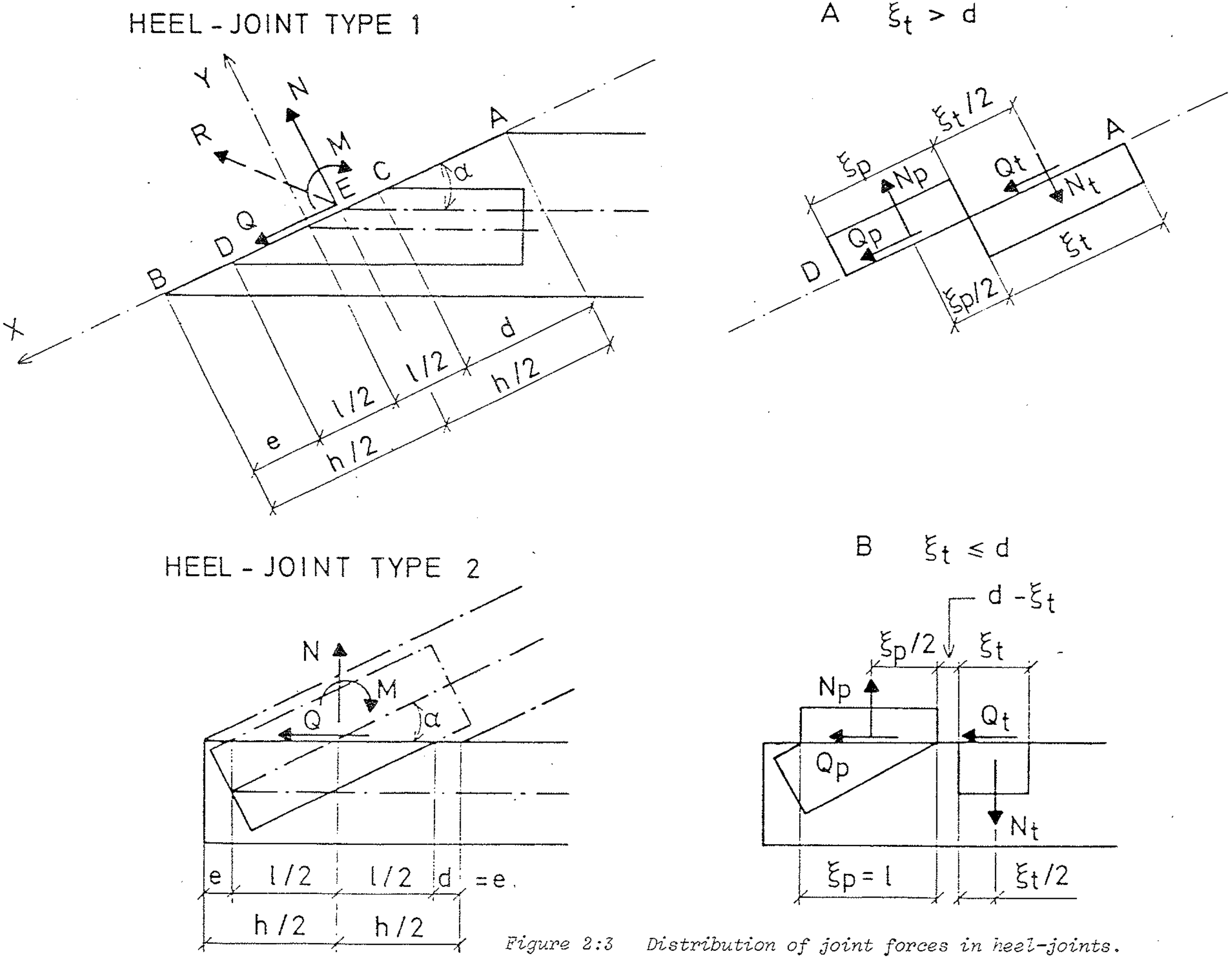


Figure 2:3 *Distribution of joint forces in heel-joints.*

Case A: Part of the plate utilized ($\xi_p < b$)

Moment on the plate:

$$M_p = 0,5(\xi_t - d)N_p \qquad (\xi_t \geq d)$$

which introduced in (3) gives:

$$\xi_t = \frac{2M + eN}{\sigma_c B(b + d) + N} \tag{1:A}$$

Case B: The plate utilized all along the joint ($\xi_p = b$)

$$M_p = 0 \qquad (\xi_t < d)$$

$$\xi_t = b/2 + d - \sqrt{(b/2 + d)^2 - \frac{2M - N(d - e)}{\sigma_c B}} \tag{1:B}$$

The cases A and B coincide when the tt-joint transmits a compression force

$$\sigma_c B d = \frac{2M - N(d - e)}{b + d}$$

The neutral axis and the stress limits then are at the edge of the plates ($\xi_t = d$)

Case C: No contact between the timbers.
The plate is stressed all along the joint (tension and compression)

$$N_p = N_{pt} - N_{pc} = N$$

$$Q_p = Q$$

Moment on the plate

$$M_p = 0,5(b - \xi_{pt})N_{pt} + 0,5(b - \xi_{pc})N_{pc} = M - 0,5(d - e)N$$

$$\xi_{pt} = (N_{pt}/N)b - 2M/N + d - e$$

$$\xi_{pc} = b - \xi_{pt} = -b\, N_{pc}/N + 2M/N + e - d \tag{1:C}$$

If $Q_p = 0$ ξ_{pc} can be expressed explicitely

$$-\xi_{pc} = \frac{2M - N(d - e)}{2p_{ac} b + N}$$

2.2 Design against plate failure

2.2.1 Design stipulations

The capacity of the plates to carry a force F_p and a moment M_p (evenly distributed on two plates) is verified with respect to the strength of the plate itself by the following condition (10):

$$\left(\frac{N_a}{N_{aD}}\right)^2 + \left(\frac{N_b}{N_{bD}}\right)^2 \leq 1 \qquad (10)$$

N_a and N_b are the components of F_p in the principle directions of the plate, see Figure 2.1:

$$N_a = F_p \cos \alpha_2 \qquad (11a)$$

$$N_b = F_p \sin \alpha_2 \qquad (11b)$$

in which $\alpha_2 = \pi/2 - \alpha_F + \beta$
is the angle between F_p and the a-direction.
N_a and N_b may be calculated from the model used in section 2.1.

N_{aD} and N_{bD} denote design values defined in the following section.

2.2.2 Design values for plate strength

As design value N_{aD} in (10) is taken the greatest (favourable) value of

$$N^p_{aD} = 2b_n p_{aD} \text{ and } N^s_{aD} = 2\ a\ s_{aD} \qquad (12a)$$

As design value N_{bD} in (10) is taken the greatest (favourable) value of

$$N^p_{bD} = 2a_n p_{bD} \text{ and } N^s_{bD} = 2b\ s_{bD} \qquad (12b)$$

In (12) p and s denote plate strength (N/mm) in tension (or compression) and in shear. The length of the cross section of the plate along the joint projected on the a-axis of the plate is denoted a and the projection on the b-axis is denotet b. It is assumed that the values a and b are based on effective plate dimensions, that is, actual dimensions must be reduced when the punched nail holes interfere with the plate edges.

The index n on a_n and b_n indicates that the plate is partly stressed. If the model introduced in section 2.1 is used, a_n and b_n are the projections in the principal directions of the plate of the stress distribution length ξ_p:

$$a_n = \xi_p \cos \alpha$$

$$b_n = \xi_p \sin \alpha = a_n \operatorname{tg} \alpha$$

where ξ_p is derived from (5a), (5 T) or (8). For example from (5a)

$$a_n = a - \frac{2M_p \cos \alpha}{F_p \sin \alpha_F} \qquad (14a)$$

$$b_n = b - \frac{2M_p \sin \alpha}{F_p \sin \alpha_F} \qquad (14b)$$

2.3 Design against grip failure

2.3.1 General method

The area of contact between a plate and the wood member (the plate/timber joint) is effective inside certain narrow zones along the timber edges. The width of the zone is supposed to be given in the plate type approval (generally 5 to 10 mm). By grip area (anchorage area) is here referred to effective area. It is denoted A or 2A to indicate the two plates, one on each face of the timber.

When the action on the plate at the section over the tt-joint is F_p and M_p (Figure 2:1), the reaction force and moment in the centre of rotation are

$$F_{RC} = F_p \qquad M_{RC} = M_p - F_p\, e_{RC} \qquad (15)$$

where e_{RC} is the distance from the force to the centre of the rotation of the plate in relation to the wood member.

The shear stress, generated by the force and the moment, may be assumed to have a constant value over the pt-joint area, corresponding to full plasticity. Hence, if the design value of shear stress is τ_D, the design value of the moment M_{RC} will be

$$M_{RCD} = 2\tau_D \int^{A} r_{RC}\, dA \qquad (16a)$$

The centre of rotation (RC) is established by iteration from the conditions for equilibrium

$$2\tau \int^{A} \sin\phi \, dA = F_p \cos\alpha_F \text{ and } 2\tau \int^{A} \cos\phi \, dA = F_p \sin\alpha_F \qquad (16b,c)$$

In (16) the polar coordinates of dA is (ϕ, r) and of RC (0,0).

2.3.2 Design stipulations

The condition for design against failure in the plate/timber joints is

$$\left(\frac{F_p}{F_{pD}}\right)^2 + \left(\frac{M_{RC}}{M_{RCD}}\right)^2 \leq 1$$

F_p is the force and M_{RC} the moment applied on the joint in the centre of rotation (15). The design value of M_{RC} is determined from (16a) and the design value of F_p from

$$F_{pD} = 2A\ \tau_D(\alpha) \qquad (18)$$

The design value of the shear stress, $\tau_D(\alpha)$, is given in certification (type-approval of the plate) as a function of the angles α_1, α_2 and α_3. In (16a) is used $\tau_D = \tau_D$ (0) if not said otherwise in the certificate.

2.3.3 Approximate method

The following condition for design is an acceptable approximation:

$$\left(\frac{F_p}{F_{pD}}\right)^2 + \left(\frac{M_{CG}}{M_{CGD}}\right)^2 \leq 1 \tag{19}$$

in which $M_{CG} = M_p - F_p\, e_{CG}$ (20)

where e_{CG} is the distance from the force F_p to the centre of gravity of the timber/plate joint (grip area).
The design values F_{pD} and M_{CGD} are defined by (18) respectively (21)

$$M_{CGD} = 2\, \tau_D\, A\, d/4 \tag{21}$$

The length d is calculated from

$$Ad/4 = \int^{A} r_{CG}\, dA \tag{22}$$

which can be differentiated by dividing A into a few ΔA concentrated to points. For a trapeziform joint-area, Figure 2:4, ΔA = A/2 can be assumed to be concentrated the distance d/4 from the centre of gravity, that is Ad/4 = 2(A/2)d/4 with d calculated from

$$d = 2\sqrt{z_a^2 + z_b^2} \tag{23}$$

$$z_a = \frac{1 + c/a + (c/a)^2}{1 + c/a} \cdot a/3 \tag{24a}$$

$$z_b = \frac{1 + 2c/a}{1 + c/a} \cdot b/3 \tag{24b}$$

In case c = 0 (triangle) $d = \frac{2}{3}\sqrt{a^2 + b^2}$

In case c = a (rectangle) $d = \sqrt{a^2 + b^2}$ i.e. d is the diagonal.

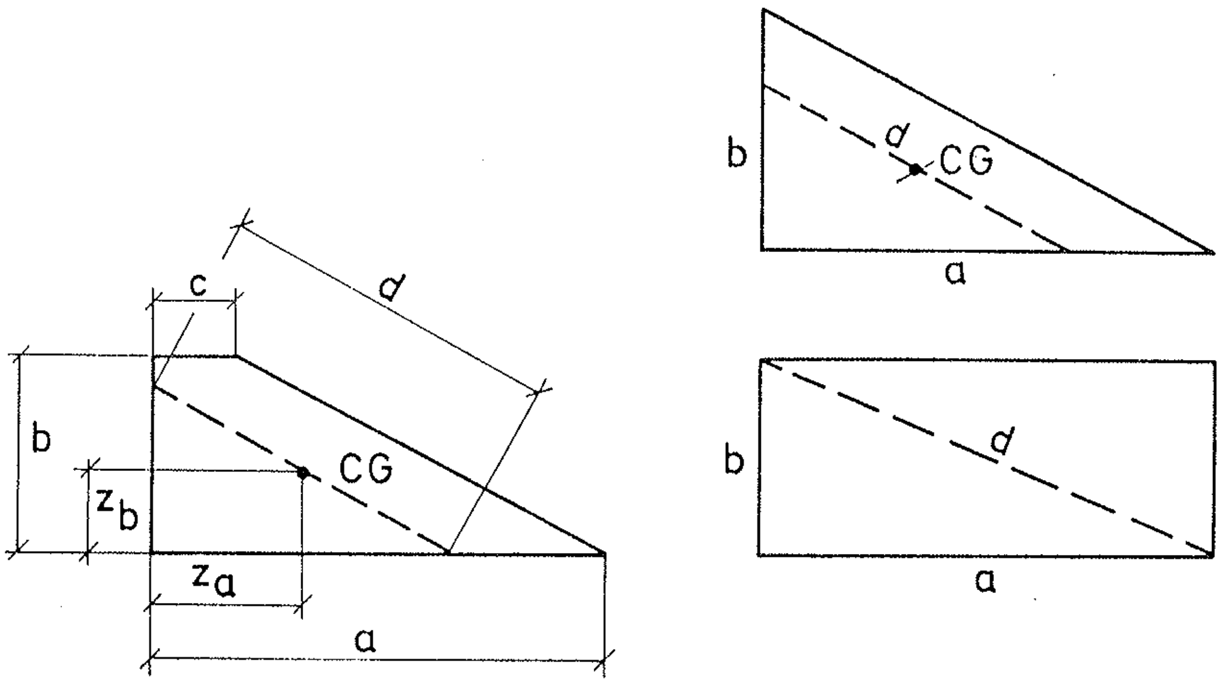


Figure 2:4 *Diagonal d in trapezium, triangle and rectangle.*

2.4 Design against wood failure

2.4.0 General

It must be verified that stresses (compression, tension, shear) generated in the wood within a connection do not exceed stipulated design values.

$$\sigma \leq \sigma_D \tag{25}$$

The influence on capacity of the angle of stress to the fibre direction (α_1) is presumed to be considered by the value of σ_D.

2.4.1 Compression

The stress at transmission of compression force between connected wood members (N_{tc}) - evenly distributed on timber thickness (B) and stressed part of the joint (ξ_t) - shall be proved not to exceed the design value for compression stress:

$$\sigma_c = \frac{N_{tc}}{B\ \xi_t} \leq \sigma_{cD} \tag{25a}$$

2.4.2 Tension

It must be verified that stresses within the connection generated in the fibre direction of the wood by the forces transmitted through the plates (F_p, M_p), do not exceed stipulated design values at combined tension and bending.

If the cross section area of the wood member is reduced by nails more than 15 %, the reduced area shall be used for the calculation of stress, otherwise the full section area may be used.

If the plate anchorage (effective plate/timber-joint area) penetrates less than 60 % of the width of the timber (H)

$$l_{A90} \leq 0{,}6\ H \tag{26}$$

it must also be verified that a simultaneous tensile force perpendicular to the fibre direction (F_{p90}) does not exceed the design value (F_{p90D}) defined by (27)

$$F_{p90} \leq F_{p90D} = \sigma_{t90D}\ B\left[l_{A0} + 0{,}5(l_1 + l_2)\right] \tag{27}$$

The equivalent length of constant stress, given within the brackets, is calculated as if the plane of failure is "through thickness (B)", parallel to the fibre direction and tangent to the anchorage area, see Figure 2:5. The lengths l_1 and l_2 are measured from the projection (l_{A0}) of the plate anchorage area to the timber end or similar intersection of the mentioned plane, an overruling stipulation beeing that l_1 or l_2 must not be assumed to be longer than four times the plate/timber-joint depth, l_{A90}.

Thus

$$l_1 \leq 4 l_{A90} \qquad l_2 \leq 4 l_{A90} \tag{28}$$

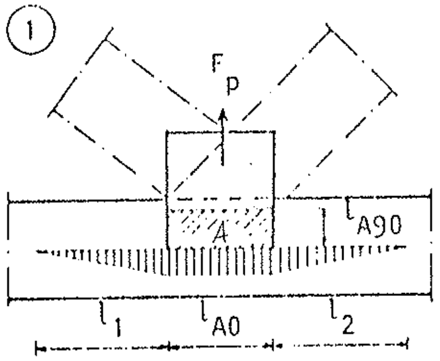
The definitions of l_{A0}, l_{A90}, l_1 and l_2 are examplified in Figure 2:5. The maximum plate force with respect to the design stress value is also given in the figure.

2.4.3 Shear

Shear stress capacity of the wood members within a connection may be verified by

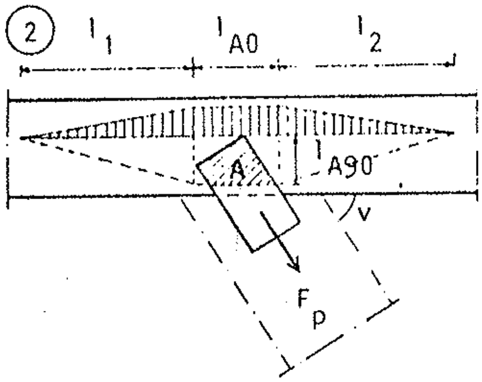
$$\tau_o \leq \tau_D = \tau_{oD} \left[1 + 2s_D \; r/(\tau_{oD} \; BH)\right] \tag{29}$$

In (29) τ_o (N/mm^2) is a mean value of shear stress parallel to the fibres calculated without regard to the contribution to shear rigidity of the plates and τ_{oD} is the stipulated design value for shear in the wood. The mean value of τ_o is determined over a length equal to the depth of the member (H). s_D (N/mm) is the design value of plate shear stress in that principal direction which is next to the shear direction. r is the part of the shear stress distribution length (H) which is covered by plates. B is the thickness of the timber.



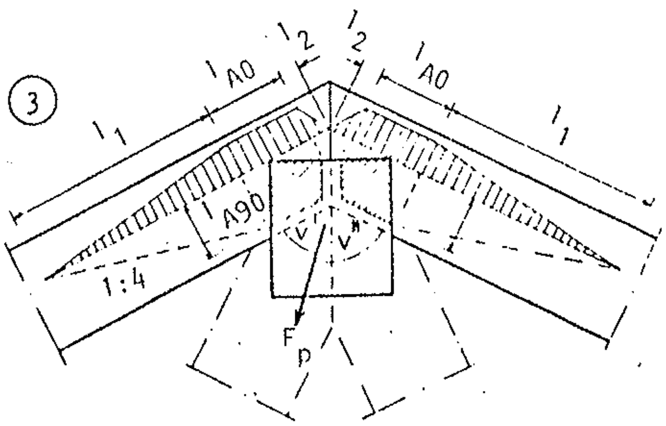
(1)(2) $l_1 = l_2 = 4\ l_{A90}$

(1) $F_p \leq \sigma_{t90D} B(l_{A0} + 4\ l_{A90})$



(2) $F_p \sin v \leq \sigma_{t90D}\ \cdot$

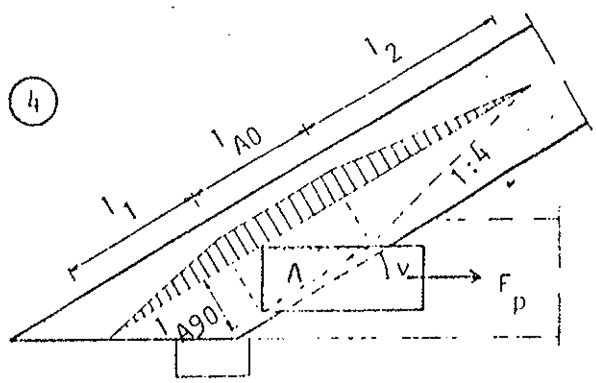
$\cdot\ B(l_{A0} + 4\ l_{A90})$



(3) $l_1 = 4\ l_{A90}$

$\frac{F_p}{2} \sin v \leq \sigma_{t90D}\ \cdot$

$\cdot\ B(l_{A0} + 2\ l_{A90} + 0{,}5\ l_2)$



(4) $l_2 = 4\ l_{A90}$

$F_p \sin v \leq \sigma_{t90D}\ \cdot$

$\cdot\ B(l_{A0} + 2\ l_{A90} + 0{,}5\ l_2)$

Figure 2.5 Fictitious distribution of stresses perpendicular to the fibre direction and corresponding design stipulation.

2.5 Rules for specific joints

2.5.1 Approximations

The model described in section 2.1 is based on the presumption that the cross section through the joint remains plane and on a stress distribution chosen without regard to the deformations. In reality the distribution of forces on the plate and the wood depends on the deformations. This is appreciated if a plate is considered which is cut in strips in the a-direction. The stiffness properties of such a plate will justify an assumption that the plate force F_p occurs in the a-direction, that is like F_p' in Figure 2:6, corresponding to $\alpha_2 = 0$ and $\alpha_F = \alpha$ in eqs. (1) and (2). The replacement of Fp to F_p' is made possible by the slip in the joint, which generates an additionel force F_{pg} in the plate and a contrary force F_{tg} in the timber/timber joint.

The force Fp is adjusted in relation to the anisotropy of rigidity of the plate. One may well assume an equivalent anisotropy of the strength with the result that the adjustment is favourable for the design against failure.

A consequent design method is to adopt certain values of α_F in the equations (1) and (2) which places the F_p in the joint direction ($\alpha_F = 0$) or in the a-direction ($\alpha_F = \alpha$) or along the b-axis ($\alpha_F = \alpha \pm \frac{\pi}{2}$). As a rule it follows from the joint forces and the position of the plates which alternatives that are relevant and favourable.

Further simplifications which often can be accepted is that the total length of the plate-section over the joint is utilized ($M_p = 0$) and that the contribution from friction between the members is neglected ($\mu = 0$).

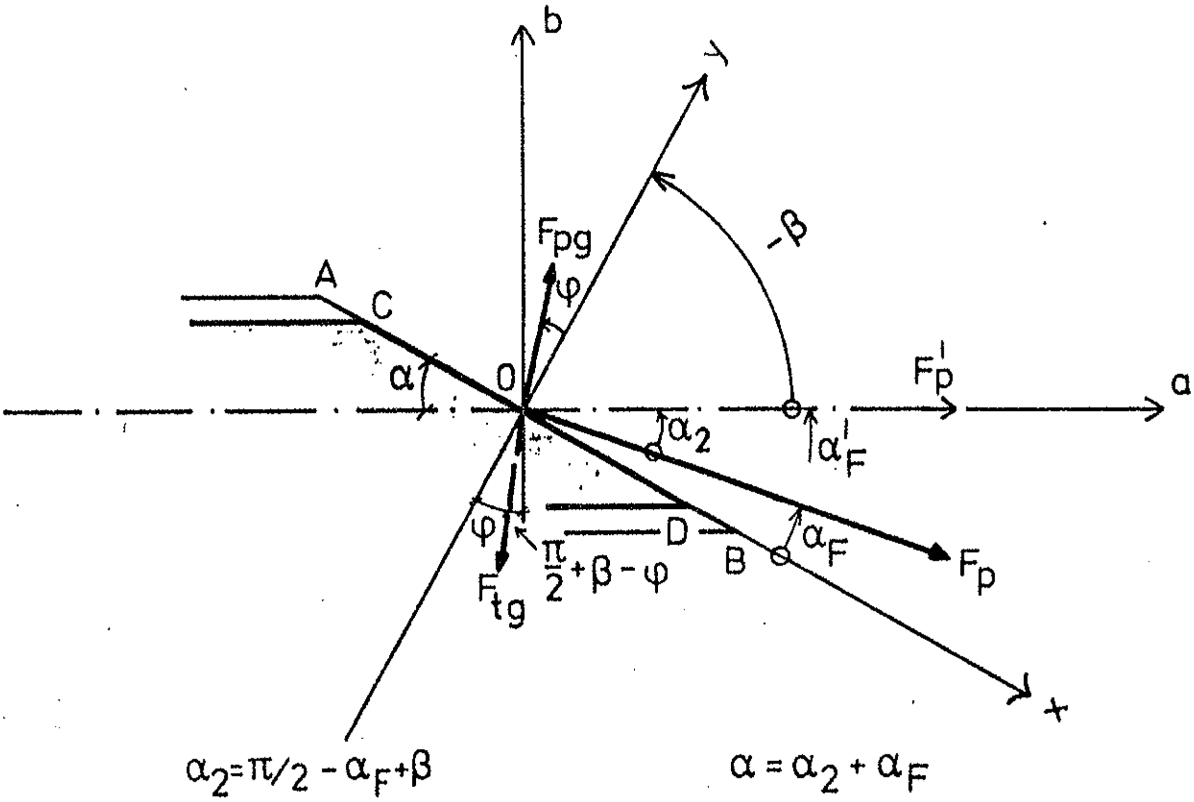


Figure 2.6 *Displacement of plate force (F_p to F_p') due to slip in the joint and generated forces. Friction is indicated by the angle ϕ.*

2.5.2 The heel-joint

The heel-joint in roof-trusses is one important object for application of design rules. There are several suggestions for the calculation. The following simple methods are quoted from proposals by Johansen and Källsner.

I Heel-joint as in Figure I

Presumptions

1) The a-axis of the plates is in the central plane of the bottom chord: $X_S = 0$

2) No friction: $\mu = 0$

3) The stress in the plate is constant over the joint cross section: $M_p = 0$

The equations of equilibrium are, cf eqs. (1), (2) and (3):

$$F_p \sin\alpha_F + F_t = R\sin\alpha_R = N \qquad (1S)$$

$$F_p \cos\alpha_F = R\cos\alpha_R = Q \qquad (2S)$$

$$F_t X_T = M \qquad (3S)$$

Design conditions are:

$$\frac{-F_t}{2(h/2 + X_T)\,B\sigma_C} \leq 1 \qquad (4S)$$

$$\left[\frac{F_p}{N_{aD}} \cos(\alpha-\alpha_F)\right]^2 + \left[\frac{F_p}{N_{bD}} \sin(\alpha-\alpha_F)\right]^2 \leq 1 \qquad (10S)$$

Solutions for $\alpha_F = \alpha$ and $\alpha_F = 0$ (or other relevant values of α_F) are checked with regard to (4S). The solution which is favourable for the plate design (10S) is chosen.

II Heel-joint as in Figure II

In principle this joint can be treated as the heel-joint, type 1. However, the conditions usually are such that the plate force may be assumed to lay in the joint direction. Thus the plate force F_p and timber force F_t are easily related to the joint forces.

$$F_p = Q \quad \text{respectively} \quad F_t = N$$

and the position of F_t is

$$X_T = M/F_t$$

In this case friction is simply considered by the alternative

$$F_p = Q + \mu F_t$$

2.5.3

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2.6 Calculation of slip

....

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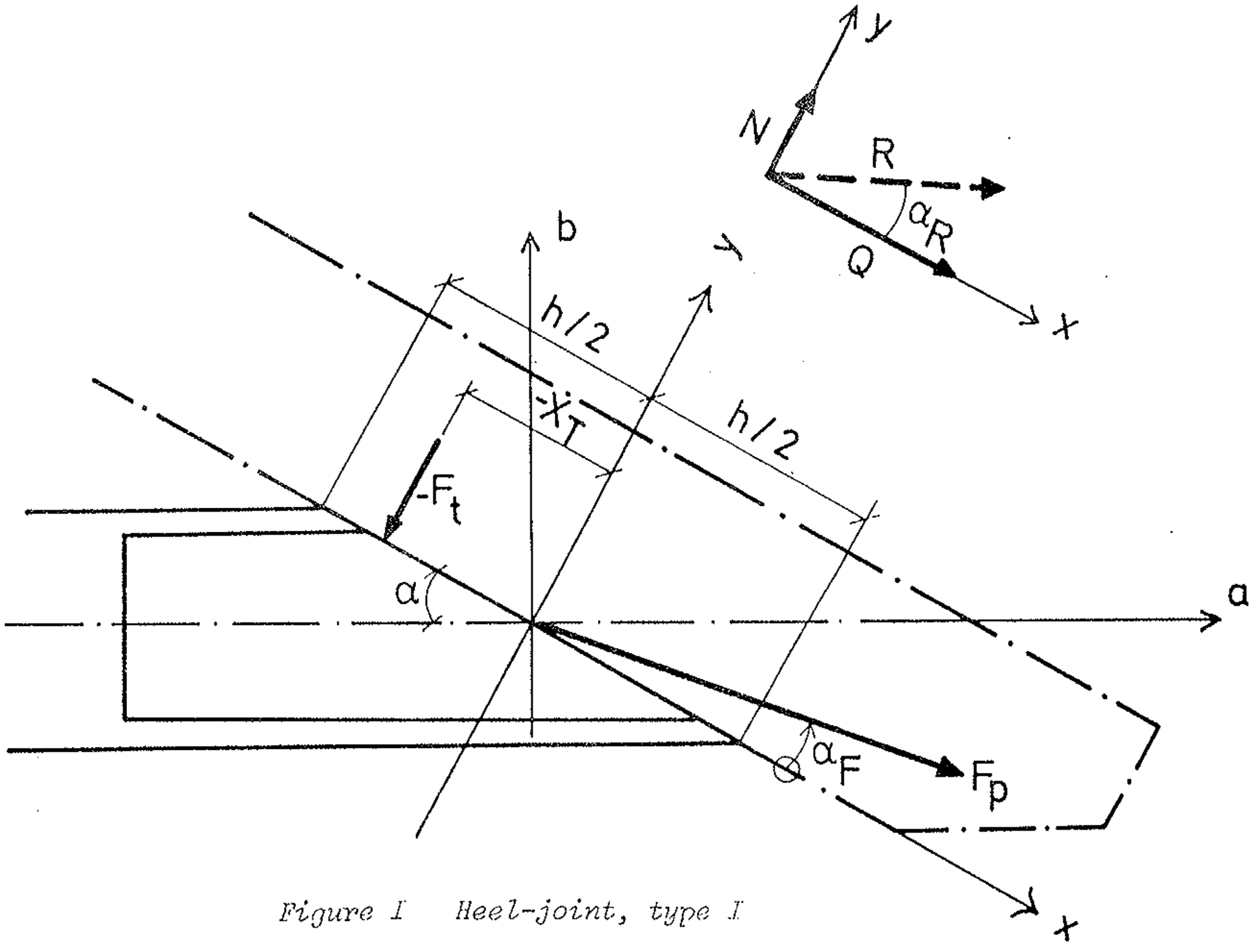

Figure I Heel-joint, type I

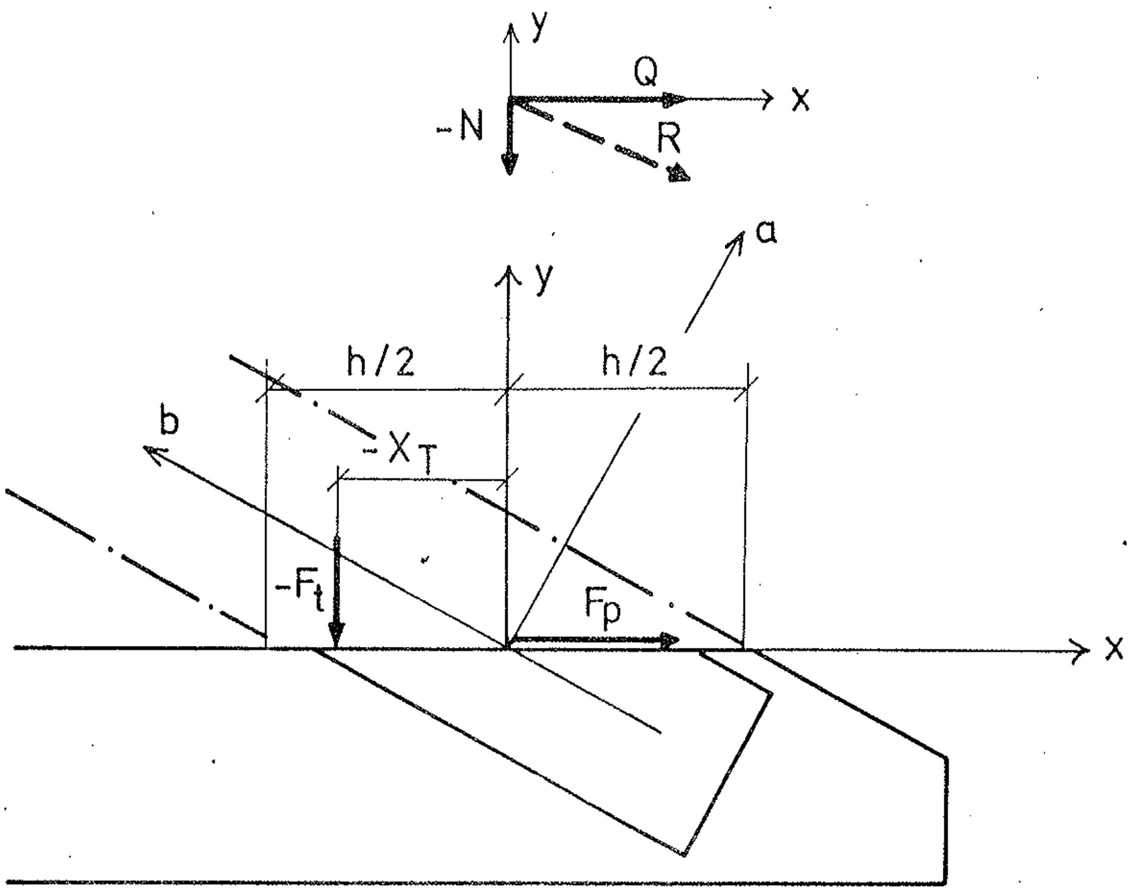

Figure II Heel-joint, type II

CIB-W18/12-12-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

GLULAM STANDARD PART 2

GLUED TIMBER STRUCTURES: RATING

(3rd draft)

BORDEAUX, FRANCE

OCTOBER 1979

FÉDÉRATION EUROPÉENE
DES SYNDICATS DE FABRICANTS DE MENUISERIES
INDUSTRIELLES DE BATIMENT
(F. E. M. I. B.)

« GLULAM »

SOUS-COMMISSION «GLULAM»
ARSENAL. OBJEKT 212
A-1030 WIEN
TELEFON 65 10 384
Austria / Autriche

July 1979

3th PROVISIONAL DRAFT

(to be used as working document only)

GLUED TIMBER STRUCTURES

RATING

PART 2

Note: At the session in Milan, on 27.10.1978, it was decided to divide Part 2 "RATING AND PERFORMANCE ", which had been intended till then, thus creating Part 2 " RATING" and Part 3 " PERFORMANCE ".
Notes within the text, marked by narrower lines and bordered by transversal dashes, serve as an explanation and are not subjekt of this GLULAM-Standard.
The results of the meeting of the Working-Group on 16.2.1979 as soon as of the meeting of GLULAM on 30.5.1979 in Frankfurt/M are worked in as much as possible

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PREFACE:

This GLULAM-STANDARD should give the base for determinations relating to timber, rating and performance of glued timber structures in the member countries of GLULAM. Furthermore, this standard should be the base for the intended CEN- and ISO- standards in the field of glued timber structures.
There are three parts of the GLULAM-STANDARD:

Part 1 : GLUED TIMBER STRUCTURES
REQUIREMENTS FOR TIMBER

Part 2 : GLUED TIMBER STRUCTURES
RATING

Part 3 : GLUED TIMBER STRUCTURES
PERFORMANCE

1) Scope:

This GLULAM-Draft, Part 2, applies to laminated or to cross-laminated load bearing structural elements made of sawn timber from coniferous species.

The requirements for timber which is glued are stated in Part 1.

The determinations and rules which are necessary for the manufacture of stuctural elements and for the performance of glued structures are subject of Part 3.

Deviations from the reqirements 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.

2) Rating:

2.1.1 Permissible stresses, dimensioning stresses, safety margins

---

Basic discussions concerning the system of contemplations of security at the present state:

a) Deterministic system (permissible stresses, presently usual system) A,B,F,I, FGR(BRD)

b) Limiting conditions (stochastical) W18 and ISO/TC 165

Determination of criteria for dimensioning on the basis of statistical data with the aid of coefficients

could not be ended in Milan 1978,Munich, March 1978 and Frankfurt/M, May 1979.

The delegates have the opinion, that the daterministic system for the next years will be kept up in the countries represented by them.

The stresses should be determined at or related to a wood moisture content of

15%.

The relation

$$\sigma_{d\alpha} = \frac{\sigma_{\parallel} \cdot \sigma_{\perp}}{\sigma_{\parallel} \cdot \sin^2\alpha + \sigma_{\perp} \cdot \cos^2\alpha}$$ CH

or $$\sigma_{d\alpha}' = \sigma_{d\alpha} - (\sigma_{d\parallel} - \sigma_{d\perp}) \cdot \sin\alpha$$ A, BRD, F, I, NL, W18

applies to compression sloping to the grain.

" F " suggests to take in supplementary charts

2.1.2 Tensile stresses perpendicular to the glueline

2.1.2.1 If tensile stresses perpendicular to the glueline cannot be avoided, they must not exceed

$$0.25\ \text{N/mm}^2.$$ 1)

2.1.2.2 If components of the beam are strained by additional stresses perpendicular to the grain or by shearing stresses, besides the regular stresses parallel to the grain, the condition

$$\left(\frac{\sigma_{\parallel}}{\sigma_{\parallel zul.}}\right)^2 + \left(\frac{\sigma_{z\perp}}{\sigma_{z\perp zul.}}\right)^2 + \left(\frac{\tau}{\tau_{zul.}}\right)^2 \leq 1$$ USA, FGR(BRD), W18, NL, F

In beams with variable height and with curved or broken axis tensile stresses perpendicular to the glueline occur, which should be considered when rating the dimensions. 2) 3)

Footnotes:

1) In unfavourable cases additional measures are necessary.

2) The tensile stresses perpendicular to the glueline in curved beams with constant height may be rated according to the turning direction of the moments using the formula

$$\sigma_{\perp} = \pm \frac{3\ M}{2\ R.b.\ h.}$$

Here the	M	moment of bending
Symbols	R	radius of bending in the axis of beam
have the	b	width of beams
meaning:	h	depth of beam

3) At laminated beams with variable height (saddle beams) it is recommended to rate the stresses by means of the formulae given in literature (e.g.Blumer H." Spannungsberechnungen an anisotropen Kreisbogenscheiben und Satteltr,ägern konstanter Dicke" and see CIB-W18, 3th draft, chapter 5, point 5.2.2)

2.2 Correction factors:

2.2.1 Height factor

Note: The presently proposed formulas are not yet guaranteed sufficiently through the test results. Therefore corresponding values or formulas will not be published until the presentation of extensive material.

2.2.2 Factor of curvature

With sharply curved beams, the influence of the bending stress within the lamellae must not be neglected.The permissible bendings stresses should be decreased because of the stresses occuring in the individual lamella, using the formula

$$k_{curv} = 1 - 2000 \left(\frac{t}{r} \right)^2$$ F,I,S, W18

or instead of this, because of the non-linear distribution of stresses, a decrease should be carried through using the formula

$$k = \frac{1}{1 + \frac{H}{2R}}$$ A, FGR(BRD), NL

Proposition of France: to admit charts

2.3 Moduli of elasticity and shearing:

Modul of elasticity ‖ to the grain	E Rating of deformation	Stress calculation according to theory 2th order
	12.0 kN/mm^2	9.6 kN/mm^2

$\perp$ to the grain	o.3 kN/mm^2
Modulus of shearing G	o.5 kN/mm^2
Modulus of torsion	o.9 kN/mm^2

2.4 Bowing and upper extrades:

Bowing must not exceed a measure which is depending on the purpose of the building.
For laminated wooden beams it is recommended the observance of a value of

$$v_{adm} = 1:3oo - 1/2oo.$$

Upper extrades, which have to be considered during manufacture, need to be stated in the plans.
For deformations, which are increasing under load depending on time (creep), this influence should be considered by means of the creep characteristic φ, depending on the degree of exploitation of the load bearing capacity, temperature and moisture content

$$v = v_o (1 + \varphi).$$

v_o = Initial deformation = elastic deformation
v_∞ = Final deformation = elastic and plastic deformation

3. References to other standards:

GLULAM-STANDARD	Part 1 :	GLUED TIMBER STRUCTURES REQUIREMENTS FOR TIMBER
GLULAM-STANDARD	Part 2 :	GLUED TIMBER STRUCTURES PERFORMANCE

CIB-W18/12-12-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTAION

WORKING COMMISSION W18 - TIMBER STRUCTURES

GLULAM STANDARD PART 3

GLUED TIMBER STRUCTURES: PERFORMANCE

(3rd draft)

BORDEAUX, FRANCE

OCTOBER 1979

FÉDÉRATION EUROPÉENE
DES SYNDICATS DE FABRICANTS DE MENUISERIES
INDUSTRIELLES DE BATIMENT
(F. E. M. I. B.)

« GLULAM »

SOUS-COMMISSION «GLULAM»
ARSENAL, OBJEKT 212
A-1030 WIEN
TELEFON 65 10 384
Austria / Autriche

J U L Y 1979

3th D R A F T   P A R T 3

Approved on the meeting 30.May 1979
Frankfurt/M

GLUED TIMBER STRUCTURES PERFORMANCE

Preface:

This GLULAM-STANDARD should give the base for determinations relating to timber, rating and performance of glued timber structures in the member countries of GLULAM. Furthermore, this standard should be the base for the intended CEN-and ISO-standards in the field of glued timber structures.

There are three parts of the GLULAM-STANDARD:

Part 1 : GLUED TIMBER STRUCTURES
REQUIREMENTS FOR TIMBER

Part 2 : GLUED TIMBER STRUCTURES
RATING

Part 3 : GLUED TIMBER STRUCTURES
PERFORMANCE

1) S c o p e:

This GLULAM- DRAFT applies to laminated or to cross-laminated load bearing structural elements made of sawn timber from coniferous species.
The requirements for timber which is glued are stated in Part 1.
The determinations or rules which are necessary for rating and dimensioning are subject of Part 2.

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.

2. Performance

2.1 General rules for design and installation of structural elements.

2.1.1 Structural elements should be designed avoiding stresses perpendicular to the gluelines as far as possible. In no case they are allowed to exceed 0.25 N/mm^2. See Part 2.

2.1.2 With curved beams the thickness of their lamellae should be determined by a ratio

$$r/t = 200 - 150$$

depending on their radius of curvature, whereby" t " should be determined in mm according to formula

$$t \leq 10 + 0.4 \left(\frac{r}{t} - 150 \right)$$

By observance of special production rules the values can be less than the limit values.

2.1.3 With openings in beams, the stress maxima occuring particularly in corners should be considered by design measures, for example,by gluing plywood to the respective places.

2.1.4 If the beam has step-like cut-outs at its ends, at the supports, the safe takeup of transversal and shearing stresses should be considered.See part 2.

2.1.5 In the final state as well as in the states of assembly sufficient precautions against upsetting should be provided by fastenings or similar measures.

If no additional precautions for stabilization are taken, the ratio $b : h \geq 1 : 10$ should not be exceeded.

b.......width of the beam
h ...... depth of the beam

2.2 End joints of lamellae:

Generally, end joints should be made by finger jointing. During manufacture, finger joints should be tested at random.
Within one lamella, the distance between the end joints should not be smaller than

1.5 m.

For the transmission of loads, butt end jointing is not permissible.

2.3 Arrangement of the lamellae:

(1) In general, the quality requirements for laminated beams which consist of individual components having smaller cross-sections, need to be related only to the unit as a whole but not to the individual parts. However, the components being situated in the tension zone must by themselves correspond to the chosen quality class.
In laminated beams, which are strained by bending this applies to all lamellae in the outer 1/8 of the depth of the beam and at least to the two outer lamellae of the tensile zone; just so the upper of the cross-section, however at least two lamellae, must be made of boards without end joints or of lamellae with finger joints.

End joints in the interior of a laminated structural element which is chiefly strained by bending or compression may be butt joints. These end joints in neighbouring lamellae should be staggered ad distances of at least 50 cm.

(2) Smallest number of lamellae :
The glued cross-section should be made of a least

4 lamellae.

2.4 Glues:

Only glues being adequate for this field of application and being tested by an authorized institution may be used.
The use of thermoplastic glues is not permissible.
The shear strength of the glue must be at least as high as that of the wood, and at least correspond to the instructions of DIN 68141 or AFNOR.

The glues should be resistent against moisture as well as against chemical or other impacts. Therefore, as a rule, synthetic resin glues should be used.

Note: A GLULAM-list of admitted glues should be added as enclosure and should be completed resp.corrected in regular distances.

2.5 Conditions of gluing:

2.5.1 Only well planed lamellae should be used, thickness deviations of up to 0.2 mm are tolerated. 1)

2.5.2 Intensity of pressure
The intensity of pressure should be evenly distributed as much as possible over the area which is glued and it should be as much as 0.6 N/mm^2. 2)

Footnotes: 1) Tests for the gluing of saw-rough timber are carried through presently. Corresponding determinations will be given, when sufficient results are available.

2) The intensity of pressure depends ~~on the~~ thickness of the lamellae, on the number of the lamellae, on the shape of the structural element (straight or curved) and on the glue used.

2.5.3 Pressing time:

The recommendation of the respective glue manufacturer as well as the climate (temperatur, humidity) in the gluing room should be considered.

2.5.4 Temperature and humidity:

The room temperature during pressing should be at least as high as 18° C. Prior to gluing, the wood which is glued should be conditioned to room temperature in time.

The relative humidity in the gluing room should not be less than 40% and not more than 80%.

With radio frequency gluing, the related rules should be obtained.

2.6 Supervision:

2.6.1 Supervision by the manufacturer himself:

The production of glued structural elements is only then permitted, when besides educated personnel corresponding equipments and instruments are available.

The gluings carried out should be plotted in data sheet (gluing records), where all the gluing work is recorded showing kind and amount of the same. The gluing records, which may be replaced by automatic records (EDV) as well, should comprise the following data:

date, building project, marking of the structural element,(project No.,current No.), amount of elements, wood species, wood quality, dimensions of the structural element or sketch of the same, moisture content of the wood, outdoor temperatur, conditioning time of the prepared timber and of the finish glued elements, start of glue spreading, start of pressing, pressing time, intensity of pressure, kinds of glues and hardeners used, gluing area per structural element, glue spread per area unit(g/m^2), signature of the supervisor.

Remark: model for gluing record is to append as appendix.

Temperature and relative humidity in the gluing room should be plotted by means of a climate-recorder.

For the supervision of the production shear-tests in the plain of the gluing area and strenght-tests for the fonger joints are recommended.

2.6.2 Supervision by competent authorities:

The production should be supervised and certificated by an authorized institution (associations of manufacturers' of glued structures, research institutes, etc.) using randomtests, or a supervision of this kind may be requested by the customer.

2.7 Pre-and post-treatment of glued structural elements:

If the structural elements have more severe service conditions due to moisture a suitable surface protection is needed but it must be warranted that it doesn't interfere with the glue.

A corresponding protection against humidity during storage and erection has to be provided.

In general, wood preservatives should be applied only after gluing, as far as no guarantee is given that no risk is involved if they are applied earlier.

2.8 Marking of glued structural elements:

Glued structural elements should be given durable markings showing manufacturer and date of manufacture.

3) Reference to other standards:

GLULAM- STANDARD	Part 1 :	GLUED TIMBER STRUCTURES REQUIREMENTS FOR TIMBER
GLULAM- STANDARD	Part 2 :	GLUED TIMBER STRUCTURES RATING.

CIB-W18/12-14-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

A SIMPLE DESIGN METHOD FOR STANDARD TRUSSES

by

A R Egerup

Technical University of Denmark
DENMARK

BORDEAUX, FRANCE

OCTOBER 1979

A SIMPLE DESIGN METHOD FOR STANDARD TRUSSES

BY

A.R. EGERUP

The Structural Research Laboratory
The Technical University of Denmark

Paper prepared for International Council for Building Research Studies and Documentation.

Working Commission W18 - Timber Structures.

FOREWORD.

At the last meeting of CIB W18 (Vienna Austria March 79) a paper: "Design of metal-plate connected wood trusses " was presented.The paper described some of the major differences in the design rules of various countries. These differences do not immediately seem significant, but a closer examination shows that they are in fact quite important.

However the differences arise from different ways of considering the same effect in the truss.

The conclusion was that most codes or recommendations could, with minor changes form a special CIB Trussed Rafter Code, which in principle could be followed by all countries.

This paper is meant as an outline of a possible common design procedure which could be universally followed.

BACKGROUND

The present design criteria for metal-plate-connected wood trusses in different countries have been developed and refined more or less according to the same principles. The level of sophistication is dependent upon experience, proto-type testing and analytical techniques.

Numerous existing trusses have been designed with very crude and simple methods and it is correct to say that no serious incidences of failure due to design method alone have taken place. The validity of the design is more a result of field experience and proto-type testing than actual well-proven theoretical design calculations, the design method has been calibrated to the experience. This applies to normally supported standard type trusses, but for individual trusses or other standards, e.g. where the support joint is moved further in on the lower chord (cantilevered trusses), the existing procedures which were perfectly valid for w-trusses are either unsafe or grossly over-dimensioned. Therefore there is a need for a general accurate and efficient truss design method.

At present only two methods of analysis exist

1. Pin-jointed analysis where the structure is modelled as a statically determinate structure assuming the members to be pin-connected.
   Axial forces in truss members are determined explicitly from an equilibrium equation.
   The moments are determined by moment coefficients depending on the type of loading and the number of panels normally based on an elastic solution for continuous beams over fixed supports.
   For manual calculation and computer analysis this is a very easy operation performed at a low cost and with high accuracy both manually and on the computer.

2. Frame analysis where the structure is considered as an indeterminate structure. This method considers a complex mathematical model of the truss with stiffness variation of the members (usually constant along each member).
   The partly rigid joints are modelled as small fictitious beam elements with stiffness and length more or less related to the physical model.
   Forces and deflections are determined by solving a great number of linear equations, which is expensive and time consuming on computer and impossible by manual calculation.
   The frame analysis is based on the theory of elasticity and produces a solution where the deformations are constitutive. This type of solution will normally be an upper bound solution and thus be conservative.
   Furthermore the moment distribution is very dependent on the mathematical model.

   There are two demands to be met in a truss design code:
   1. A simplified efficient design procedure which produces designs for ordinary standard truss configurations.

   2. A sophisticated computerized. method which produces accurate design for any timber truss, taking all significant effects from truss behaviour into account.

   The first method should be used as an every day design procedure which can be performed very easily at a low cost, either manually or on a small computer.The sophisticated computerized method should be used for individual structures and for calibration of the simplified procedure.

PROPOSAL FOR A SIMPLIFIED METHOD

A simplified method for standard trusses should produce the following parameters for use in the design criteria:

1. Axial forces.
2. Moments.
3. Buckling length
4. Deflections.

The axial forces in a truss are determined very accurately in the pin-jointed analysis, but moments are not determined very well in either the pin-jointed or the frame-analysis. The reason is the non-linear behaviour(plasticity)of the truss for even low load levels. However, test calculations and experience show that the moments can with reasonable accuracy be estimated by moment coefficients. [1)] These can among other things vary with the slope of the truss which can be taken into account if required.

The buckling length can also be given as a constant multiplied by the distance between panel points.

1) The moment coefficient is a factor c which multiplied by the simple moment M_o gives the moment in the required point $M = c \cdot M_o$ for a uniform distributed load w is $M_o = \frac{1}{8} w\, l^2$.

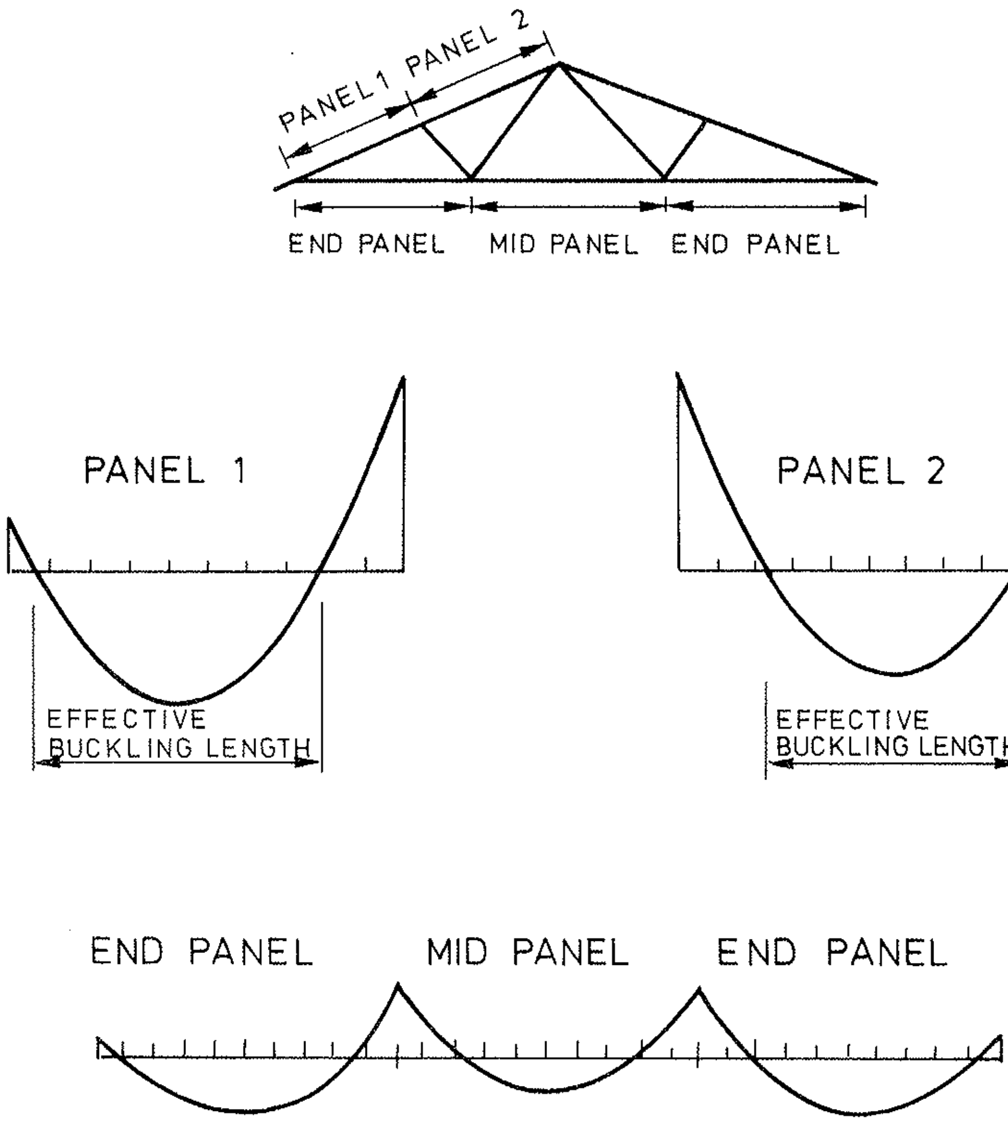


Fig. 1. Moment distribution in a W-truss.

Figur 1. shows the simplest example of a truss: the w-configuration. The moment distribution in the upper and lower chords is shown assuming uniformly distributed loads. The effective buckling length is calculated from the inflection points of the moment curve for a member. In the moment distribution it is assumed that both the support joint and the top joint are able to transfer moments.

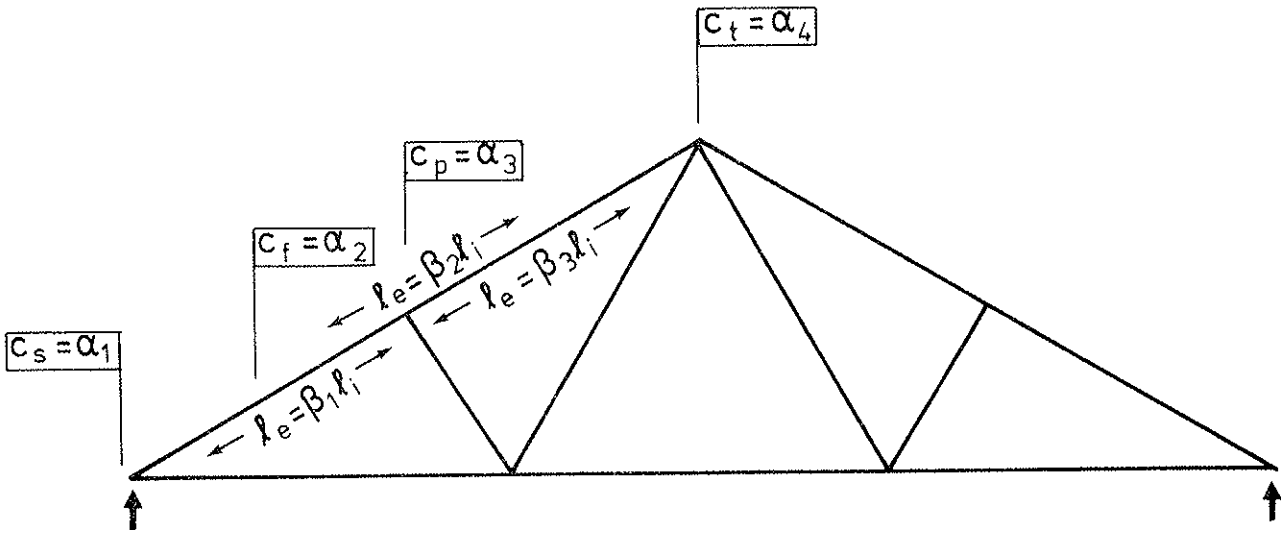


Fig. 2. Moment coefficients and effective buckling lengths.

In figure 2. moment coefficients in different parts of the truss are indicated as a variable α; the effective buckling length for compression members is also indicate as a variable β multiplied by the panel length.
The axial forces are determined from a pin-jointed analysis.

α and β are deliberately indicated as variables.
α and β are determined by calibration or judgement. For example α will depend on the plastic capacity of the timber and might vary from country to country according to the softness (ductility) of the timber.

A pure elastic solution will give the following α - values.

$\alpha_1 = 0.0$

$\alpha_2 = 0.070$

$\alpha_3 = 0.125$

$\alpha_4 = 0.0$

For example a plastic solution will give the following α- values

$\alpha_1 = 0.01$

$\alpha_2 = 0.083$

$\alpha_3 = 0.083$

$\alpha_4 = 0.01$

In the plastic solution it is assumed that the moment-capacity of the joints is approx 1o% of the moment capacity of the timber.

Other types of standard-configuration trusses (triangular-configurations) can be described with respect to moments and buckling length in a similar way.
The deflection criteria can be treated in a similar manner with a coefficient-system.

The method has the advantage that it can be used both manually and at low cost on a computer. Furthermore the method can generate limiting span-tables which must be part of the national code based on the rules in the CIB-code.

CIB-W18/12-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE:

from

Sous-Commission "GLULAM"

BORDEAUX, FRANCE

OCTOBER 1979

CIB-W18/12-100-1

FÉDÉRATION EUROPÉENE
DES SYNDICATS DE FABRICANTS DE MENUISERIES
INDUSTRIELLES DE BATIMENT
(F. E. M. I. B.)

« GLULAM »

To

International Council for
Building Research studies
and Documentation/W18-
Timber Structures

SOUS-COMMISSION «GLULAM»
ARSENAL, OBJEKT 212
A-1030 WIEN
TELEFON 65 10 384
Austria / Autriche

Wien,
Vienna 1th October 1979
Vienne

Ref.: Comment to the CIB-CODE 4th draft

The present 4th Draft CIB-W18 Code was discussed at the GLULAM-MEETING at Franfurt/M on 30th May 1979. It was decided to comment various paragraphs referring to glued timber structures:

ad Chapter 3: This chapter had been presented at the CIB-W18-meeting in March 1979; it is basing on the method of partial factors.

In the 2nd Draft FW, page 1, provivions had been made for using the deterministic system as well as the method of partial factors.

In the opinion of the delegates of the member countries of GLULAM it should be left for choice which system is used to rate structures. The introduction of a new concept of security can be done by the national authorities only. As regards the member countries of GLULAM it is to be assumed that the deterministic system will not be discarded at least within the next years. In most of the countries the standards for rating concrete, timber and steel have been revised and published recently. Also for this reason, an alteration is not to be expected in the next time. However, above all, it appears necessary to make safety considerations according to the same principle for all structural materials. In large structures made of timber, particularly in glued structures, parts from steel are used frequently. It would be intolerable to lay down

different principles for rating these two structural materials.
Other doubts were expressed as to the difficulty of the application of this proposed system in practice. It should be considered that the standards have to be applied in small plants as well, where no experts having the necessary knowledge of statistics and of the theory of probabilities are available. Hence the factors should be stated in so simplified a manner as to make them applicable in practice.
Further on, it was proposed to add exemples to the CIB-Draft as soon as security systems and factors will be stated conclusively, in order to facilitate the application of the standard.

<u>ad Chapter 5 : Paragraph 5.2.1 Height Factor</u>

For long time it is known that the strength of beams which are bearing bending loads is decreasing with increasing height of the beam. (Studies by Newlin and Traver 1924, further studies by Mollin, Cizek, A. Ylinen and others), however, with the usual dimensions of timber, this was of no practical importance.
By contrast, with high glued laminated timber beams, this influence has to be considered, whereby the way how to consider it still has to be decided.
In fact, in the standards of various countries, different figures are stated, as shown in Fig. 1 in the appendix.
Now the courses of the curves show very well that particularly with heights of more than 100 mm no decrease of the strength takes place, presumably due to an improving effect caused by the lamination.
The formula, comprised in the CIB-Code 4th Draft

$$c_H = \left(\frac{300}{H}\right)^{x} \quad \text{in mm}$$

is basing on the highly theoretical studies by B. Bohannan "Effect of size on bending strength of wood members", US Forest Service, Research Paper May 1966, in this studies

the experiments had been carried out using samples measuring 5 by 5 cm. It is our opinion that the results of this experiments are not applicable in practice of construction because it is impossible to extrapolate figures from small-scale experiments with samples sized 5 by 5 cm to 10 to 20- fold these dimensions.
The exponent $\varkappa$ depends on wood species and on grading; in former studies it had been stated as much as 1/9.
The height factor, having been calculated using this exponent $\varkappa$ is plotted in the enclosed Fig.1, where deviations chiefly occur in the range between H = 300 and H = 1000 mm.
It is the opinion of the delegates of GLULAM that in this matter a unification should be induced and that if a height factor should be considered at all, it should be applied only with heights from 500 mm on. The alteration of the formula for example to

$$k_H = \left(\frac{500}{H}\right)^{\varkappa}$$

with $\varkappa = 1/9$ would make a deviation of a few percent only, however, for tractical use, this would result in a simplification.

ad Paragraph 5.2.3 :

The Wilson-formula, initially provided for

$$k_{curv} = 1 - 2000\left(\frac{t}{r}\right)^2$$

was replaced by the modified Wilson-.Hudson- formula

$$k_{curv} = 0.76 + 0.001\,\frac{r}{t}.$$

In the usual range $\frac{r}{t} = 200 - 150$ both the formulas give practically identic figures.

The Wilson- formula is included in the standards of various countries, therefore it should be thought over whether or not it is possible to use this formula further on in its prevailing shape in order to facilitate the adaption of the national standards and codes.

For the rest we noted the following:

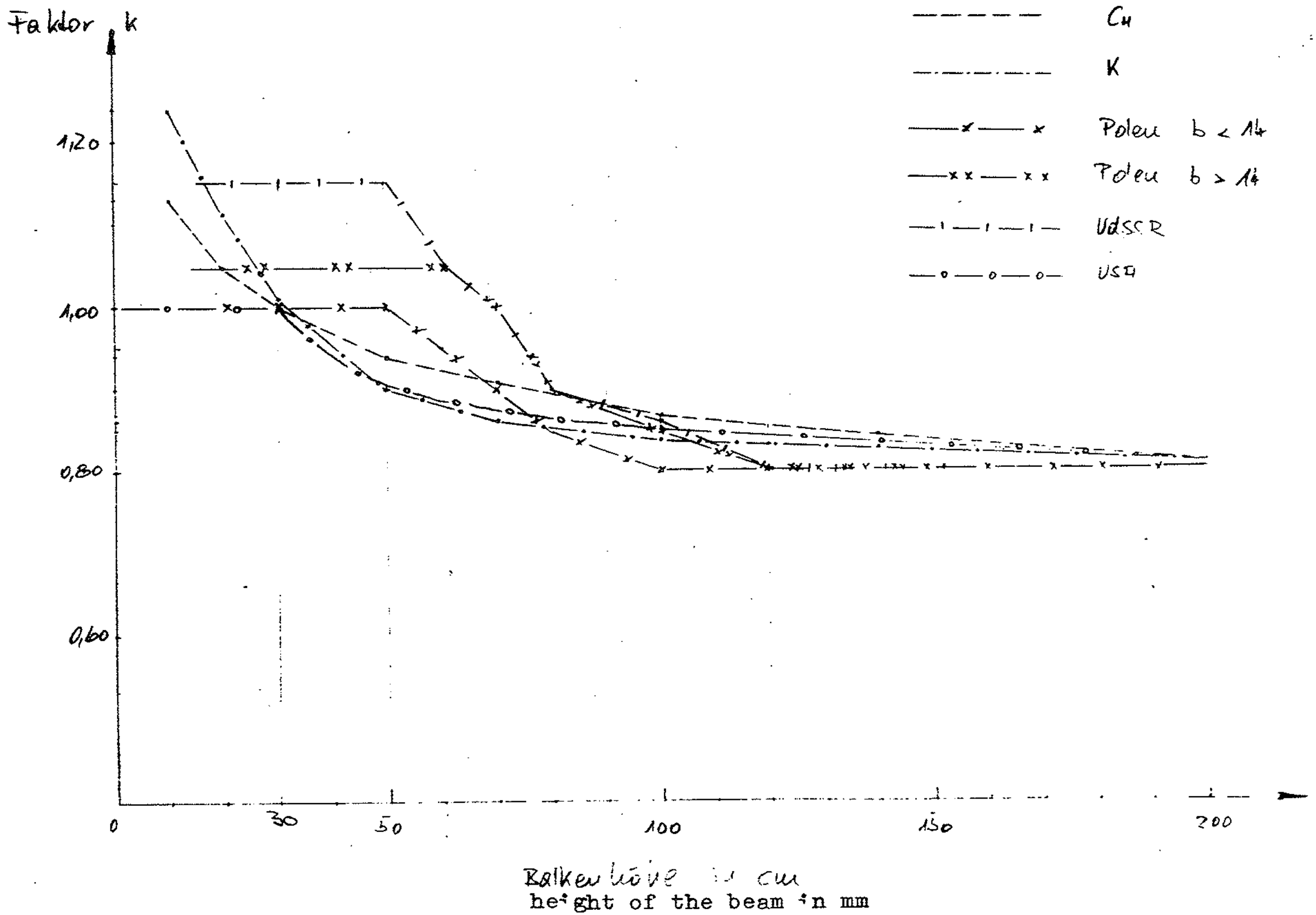
<u>ad Formula 6.1.3.1b :</u>

The formula shall be corrected:

$$F_{min} = 42.\sqrt{\rho}\,.\,d^2 + 12.\rho\,.k_1\,.\,t.\,d$$

with other words, in the second part of the formula " h " shall be replaced by " t " .

Appendix



Faktor k
1,20
1,00
0,80
0,60
0
30
50
100
150
200
CH
K
Polen b < 14
Polen b > 14
UdSSR
USA
Balkenhöhe in cm
height of the beam in mm

CIB-W18/12-100-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

by

R H Leicester

Commonwealth Scientific and Industrial Research Organisation
AUSTRALIA

BORDEAUX, FRANCE
OCTOBER 1979

CIB-W18 - TIMBER STRUCTURES; BORDEAUX, FRANCE

Dr R H Leicester of the Commonwealth Scientific and Industrial Research Organisation, Australia, wishes to draw the attention of the members of CIB-W18 in their deliberations for the CIB Structural Timber Design Code, to the following documents:

1 Lateral Deflection of Timber Beam-columns during Drying - R H Leicester, Wood Science and Technology Vol 5 (1971) No 3, pages 221-231.

2 A Rheological Model for Mechano-sorptive Deflections of Beams - R H Leicester. Wood Science and Technology Vol 5 (1971), pages 211-220.

3 Design of Bracing Systems for Timber Structures - R H Leicester CSIRO Lecture.

4 Buckling Strength of Slender Timber Members - R H Leicester. Extract from CSIRO course Lecture notes (1973).

5 Buckling Strength of Plywood Webs - R H Leicester. 17th Forest Products Research Conference (1975) CSIRO.

6 Contemporary Concepts for Structural Timber Codes - R H Leicester. Forest Products Reprint No 993 (1972).

7 Linear Fracture Mechanics Solutions for Zero and Right-Angle Notches - P F Walsh. CSIRO Division of Building Research Paper No 2 (1974).

8 Effect of Size on the Strength of Structures - R H Leicester. CSIRO Division of Building Research Paper No 71 (1973).

9 The Strength of Glued Lap Joints in Timber - P F Walsh, R H Leicester, A Ryan. Forest Products Journal Vol 23 No 5.

10 The Size Effect of Notches - R H Leicester. The Proceedings of the Second Australasian Conference on Mechanics of Structures and Materials (1969).

11 Fracture at Butt Joints in Laminated Pine - R H Leicester and P C Bunker. Forest Products Journal Vol 19 No 2.

12 Applications of Linear Fracture Mechanics in the Design of Timber Structures - R H Leicester. 1974 Conference of the Australian Fracture Group.

13 Applications of Fracture Mechanics in the Timber Engineering Code CA65 - R H Leicester. 16th Forest Products Research Conference (1973).

14 Fracture Strength of Wood - R H Leicester. First Australian Conference on Engineering Materials (1974).

15 The Fracture Strength of Wood - R H Leicester. CSIRO Lecture.

16 Fracture at Butt Joints in Laminated Pine - R H Leicester and P C Bunker. Forest Products Journal Vol 19 No 2 (1969).

17 Load-sharing Characteristics of Timber Structural Systems - G F Reardon and R H Leicester. Conference on Application of Probability Theory to Structural Design (1974).

18 Proof Grading, A Practical Application of Reliability Theory - R H Leicester. 3rd International Conference on Applications of Statistics and Probability in Soil Structural Engineering (1979).

19 Australian Standard 1649-1974. Determination of Basic Working Loads for Metal Fasteners for Timber.

20 Australian Standard 1720-1975. SAA Timber Engineering Code.

CIB-W18/12-100-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB STRUCTURAL TIMBER DESIGN CODE
(Fourth Draft)

BORDEAUX, FRANCE

OCTOBER 1979

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

WORKING GROUP W18
TIMBER STRUCTURES

CIB
STRUCTURAL TIMBER DESIGN CODE

Fourth draft, June 1979

FOREWORD

A first draft of the CIB Structural Timber Design 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 code was prepared and discussed by a Code Drafting Sub-Committee consisting of CIB members L. 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.

A second draft of the code was prepared by L. G. Booth, H. J. Larsen and J. R. Tory. This was discussed, except for Chapter 6, Mechanical Fasteners, at the CIB-W18-meeting in June 1978 and it was agreed that a third draft should be sent out for comments, among others from the members of ISO/TC 165-Timber Structures.

The third draft was discussed at the CIB-W18-meeting in March 1979 together with chapter 6 from the second draft and a draft for chapter 3, Basic Design Rules. Based on the discussion this fourth draft was prepared.

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.

Although chapter 3 is based on the partial factor method of design the remainder of this draft standard is equally applicable to either deterministic or partial factor methods provided material properties are given as characteristic values and suitable safety factors for strength and stiffness parameters are introduced to the design calculations.

Sections changed from third draft are marked with a vertical line of diamonds in the left-hand margin.

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.

LIST OF CONTENTS

The background for the CIB-Structural Timber Design Code is technical papers prepared for and discussed at meetings in CIB-W18 Timber Structures.

The papers are identified by CIB-W18 and a number a-b-c:

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

1. Princes Risborough, England, March 1973
2. Copenhagen, Denmark, October 1973
3. Delft, Netherlands, June 1974
4. Paris, France, February 1975
5. Karlsruhe, Federal Republic of Germany, October 1975
6. Aalborg, Denmark, June 1976
7. Stockholm, Sweden, Feb./March 1977
8. Bruxelles, Belgium, October 1977
9. Perth, Scotland, June 1978
10. Vancouver, Canada, August 1978
11. Vienna, Austria, March 1979

b denotes the subject 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

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

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

A current list of CIB-W18 technical papers from meeting 1-11 is given as an Annex to this code. Copies of individual papers are available from:

The Secretary, CIB-W18
Princes Ribourough Laboratory
Princes Risborough
Buckinghamshire HP17 9PX
United Kingdom

1. INTRODUCTION

1.1 Scope

The 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 containing 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.

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., and its integrity 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	° $(1° = \frac{\pi}{180})$
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. Attention is drawn to the special notations used in Annex 7A - B - C.

Main symbols

A	Area
E	Modulus of elasticity
F	Force
G	Shear modulus
I	Second moment of area (moment of inertia)
M	Moment, unless otherwise stated Bending moment
N	Axial force
V	Shear force Volume
a	Distance
b	Width
d	Diameter Side length for square nails
e	Eccentricity
f	Strength
h	Depth of beam
i	Radius of gyration
k	Factor, always with a subscript
ℓ	Span Length
r	Radius
t	Thickness
x y z	Coordinates
α	Angle
β	Factor
η	Factor
κ	Factor
λ	Slenderness ratio
μ	Ratio
ν	Poisson's ratio
ρ	Specific gravity
σ	Normal stress
τ	Shear stress
φ	Ratio

Subscripts

apex	At apex
axial	Axial
bearing	Bearing
bolt	Bolt
buck	Buckling
c	Compression
col	Column
con	Connector
crit	Critical
curv	Curvature
depth	Depth
E	Euler
e	Effective
f	Flange
head	Head (nail)
i	Inner
inst	Instability
m	Bending
max	Maximum
mean	Mean value
min	Minimum
nail	Nail
o	Outer
size	Size
t	Tension
tang	Tangency
thread	Threaded
tor	Torsion
v	Shear
w	Web
x	Related to the x-direction
y	Related to the y-direction Yield

Numbers 1, 2, . . . are used. The following have a special meaning:

0	In the fibre direction, parallel to grain
90	Perpendicular to the fibre direction, perpendicular to grain

1.5 Definitions (will be prepared at completion of the work)

The test climate and the climate in section 2.2 is chosen with regard to International Standard ISO 554, Standard atmospheres for conditioning and/or testing specifications.

The climate class grading is based on CIB-W18/5-11-1 with small changes as motivated in 6-11-1. The climate classes should not be confused with meteorological climates.

The load-duration grading is based on CIB-W18/3-9-1 and especially 7-9-1.

2. BASIC ASSUMPTIONS

2.1 Characteristic values and mean values

2.1.1 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 load duration of 3 to 5 mins. at a temperature of 20 ± 3°C and relative humidity of 0.65 ± 0.02. Where the characteristic values are estimated from a limited number of tests the estimate shall be made with a confidence level of 0.75.

The characteristic strength values are related also to

- a section depth of 200 mm for the bending strength of solid timber,
- a section depth of 300 mm for the bending strength of glued laminated timber,
- a volume of 0.02 m^3 for the tensile strength perpendicular to grain.

The characteristic 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 20 ± 3°C and relative humidity of 0.65 ± 0.02.

2.1.2 Mean values

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.

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 1

The climate class is characterized by a moisture content in the materials corresponding to a temperature of 20 ± 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 0.65.

: 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 moisture content in the materials corresponding to a temperature of 20 ± 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.

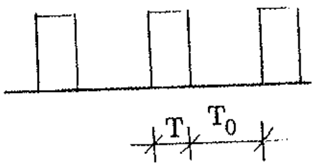
Climate class 3

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 class for a given load (specified by its time spectrum) depends on the properties of the material or the whole structure. For the intermittent load shown the effective loading time is equal to T if T_0 is long as compared to the recovery time for the material, and equal to the accumulated loading time if there either is no recovery or if T_0 is comparatively short.

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.

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

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 2.3b Examples of load-duration classifications for actions

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

Chapter 3 is new and is based on CEB, Volume I and General principles on reliability of structural design, May 1978 prepared by L. Östlund.

3. BASIC DESIGN RULES

3.0 General

Structures should be designed in such a way that there is a prescribed safety against the limit states described below being exceeded.

Furthermore, they should be designed in such a way that when exposed to fire they have adequate load-carrying capacity and integrity for a certain amount of time, out of regard for evacuation, limitation of flame spread and protection of firemen.

The main structure should normally be designed in such a way that it should not subsequently be damaged to an extent disproportionate to the extent of the original incident. This requirement may be achieved by

a) designing the structure in such a way that if any single load-bearing member becomes incapable of carrying load this will not cause collapse of the whole structure or any significant part of it, or

b) where necessary, ensuring (by design or by protective measures) that no essential load-bearing member can be made ineffective as a result of an accident.

3.1 Limit states

A structure, or part of a structure, is considered to have become unfit for its intended purpose when it reaches a particular state, called a limit state, in which one of the criteria relating to its load-bearing capacity or its conditions of service is infringed.

Limit states are classified into ultimate limit states and serviceability limit states.

3.1.1 Ultimate limit states

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

: Ultimate limit states may for example correspond to:
: - loss of static equilibrium of the structure, or part of the structure, considered as a rigid body (overturning),
: - rupture of critical sections of the structure due to exceeding the material strength (in some cases dependent on the load history) or by deformations,
: - loss of stability (due to among other things buckling),
: - unlimited slip of the whole structure or mutually between parts of it.

3.1.2 Serviceability limit states

Serviceability limit states are related to the criteria governing normal use.

: Serviceability limit states may for example correspond to:
: - deformations which affect the efficient use of a structure or the appearance of structural or non-structural elements,
: - excessive vibrations producing discomfort or affecting non-structural elements or equipment (especially if resonance occurs),
: - local damage (including cracking) which reduces the durability of a structure or affects the efficiency or appearance of structural or non-structural elements,
: - local buckling of thin plates (for example in thin webs or flanges) without rupture.
: - excessive impressions due to stresses perpendicular to the grain and not affecting the ultimate strength.

3.2 Actions and their combinations

3.2.1 Actions

It is assumed that actions are classified according to ISO . . . (DP 6116, under preparation) and that the following values are given for the individual actions

- characteristic value F_k
- combination value $\psi_0 F_k$

and furthermore, that the necessary information is given to assign the actions to one of the load-duration classes given in section 2.3.

A level 1-method according to CEB, Volume I has been chosen, see CIB-W18/11-1-1.

: It is assumed that the load values are given in separate documents, for example in the form of common load regula-
: tions or ISO standards.

: An action is an entity of
: - an assembly of concentrated or distributed forces acting on the structure (direct actions), or
: - the effect of imposed or constrained deformations in the structure (indirect actions)
: and due to the same cause.

: In addition to the duration classification (see 2.3), actions are also classified by ISO DS 0000 (at present Draft
: Proposal DP 6116) according to their variation in time as
: - permanent actions
: - variable actions
: - accidental actions

: The characteristic value of permanent actions from self-weight of the structure and weight of superstructure, etc.
: may be calculated from the intended values of the geometrical parameters and the mean unit weight of the material.

: For variable action the characteristic value is defined as the value which has a prescribed probability of not being
: exceeded in one year. When characteristic values cannot be determined from statistical data, as for example for
: actions from special equipment, the corresponding values may be estimated on the basis of available information.
: In some cases minimum characteristic values are prescribed by the competent public authority.

: For the combination value the factor ψ_0 takes account of the reduced probability of simultaneously exceeding the
: design values of several actions, as compared with the probability of the design value of a single action being ex-
: ceeded.

: For accidental actions the characteristic value is normally prescribed by the competent public authority.

3.2.2 Combinations of actions

In the ultimate limit states the following two types of combinations should be applied:

- ordinary combinations
- accidental combinations.

In the serviceability limit states the combinations of actions should be chosen with regard to the purpose of the actual calculation.

3.2.2.1 Ordinary combinations

Permanent action + one variable action with its characteristic value + variable actions with their combination values.

3.2.2.2 Accidental combinations

Permanent action + one accidental action + variable actions with their combination values.

3.3 Verification of design

The verification of the design should be made according to the method of partial coefficients.

In this method

- actions are expressed by design values F_d according to 3.3.1,
- strength parameters are expressed by design values f_d according to 3.3.2. Other relevant properties (e.g. modulus of elasticity in connection with instability design) are treated in a similar way,
- geometrical parameters are expressed by design values a_d according to 3.3.3.

: If the general condition for the actual limit state not being exceeded is expressed as
:
: $$\theta(F, f, a, \mu, C) \geqslant 0 \tag{3.3a}$$
:
: the design criteria will be
:
: $$\theta(F_d, f_d, a_d, \mu_d, C) \geqslant 0 \tag{3.3b}$$
:
: where

: F represents actions,
: f represents material properties,
: a represents geometrical parameters,
: μ are quantities covering the uncertainties of the calculation model,
: C are constants including preselected design constraints,
: $\theta()$ represents the limit state function, and
: subscript d denotes design value.

: The special problems in connection with soil mechanics are not treated in this code.

3.3.1 Design values of actions

The design value shall be obtained from the characteristic value or the combination value by multiplication by a partial coefficient γ_f:

$$F_d = \gamma_f F_k \tag{3.3.1a}$$

or

$$F_d = \gamma_f \psi_0 F_k \tag{3.3.1b}$$

The values of γ_f are prescribed by the relevant public authority.

: In CEB-volume II, concrete, the values for γ_f given in table 3.3.1a have been proposed.

: *Table 3.3.1a CEB-proposal*

action combination	action	γ_f	unfavourable effect	favourable effect
ordinary	permanent	γ_g	1.35	1.0
	variable	γ_q	1.5	not to be taken into account
accidental	permanent	γ_g	1.1	0.9
	variable	γ_q	1.0	not to be taken into account
	accidental	γ_a	1.0	not to be taken into account
serviceability	permanent	γ_g	1.0	
	variable	γ_q	1.0	

: In the NKB-proposal the values for γ_f given in table 3.3.1b have been proposed.

: *Table 3.3.1b NKB-proposal*

action combination	action	γ_f	unfavourable effect	favourable effect
ordinary	permanent	γ_g	1.0	0.8*)
	variable	γ_q	1.3	not to be taken into account
accidental	permanent	γ_g	1.0	1.0
	variable	γ_q	1.3	not to be taken into account
	accidental	γ_a	1.0	not to be taken into account
serviceability	permanent	γ_g	1.0	
	variable	γ_q	1.0	

: *) The factor should be applied to the total constant action, not to part of it.

: The design load combinations according to section 3.2.2 are thus for ordinary combinations

The examples are of a very general nature and should be adjusted to timber structures.

: $\gamma_{g,unf}F_{g,unf} + \gamma_{g,fav}F_{g,fav} + \gamma_q(F_{q,1} + \psi_{0,2}F_{q,2} + \psi_{0,3}F_{q,3} \dots)$ (3.3.1c)

:

: and for accidental combinations

:

: $\gamma_{g,unf}F_{g,unf} + \gamma_{g,fav}F_{g,fav} + \gamma_a F_a + \gamma_q(\psi_{0,1}F_{q,1} + \psi_{0,2}F_{q,2} + \psi_{0,3}F_{q,3} \dots)$ (3.3.1d)

:

:

3.3.2 Design values of strength parameters

The design values shall be obtained from the characteristic values, modified according to climate class and load-duration class, by division by a partial coefficient γ_m:

$$\gamma_m = 1.0 \tag{3.3.2a}$$

for serviceability limit states.

For the ultimate limit states γ_m is prescribed by the relevant public authority dependent on the failure classes given in table 3.3.2 a

Table 3.3.2a Failure classes

failure class	consequences
less serious	risk to life negligible and economic consequences small or negligible
serious	risk to life exists and/or economic consequences considerable
very serious	risk to life great and/or economic consequences very great

: The failure class for a given type of structures is normally prescribed by the competent public authority, e.g. in
: the form of a list of examples as given below. The authority may decide to refer all structures to the class serious.

: *Less serious:*
: - Buildings in which there are persons only occasionally, for example warehouses and sheds, if the buildings
: have no more than two stories and moderate spans.
: - Partition walls.
: - Lintels.
: - Roof and wall sheathing.

: *Serious:*
: - Buildings in which there are persons only occasionally, for example warehouses, if the buildings have more
: than two stories, or in one- or two-storey buildings with large spans (hall structures).
: - Buildings in which there are frequently many persons, for example dwelling houses, offices or factories, if
: the buildings have no more than two stories and moderate spans.
: - Building scaffolds and concrete forms.
: - Exterior walls.
: - Staircases.
: - Railings.

: *Very serious:*
: - Buildings in which there are frequently many persons, for example dwelling houses, offices, theatres, sports
: halls or factories, if the buildings have two or more stories or large spans (hall structures).
: - Grandstands.

: A structure is to be referred to a higher class than stated, if, in special cases, stricter requirements are made with
: regard to the risk of personal injury or the significance of the structure altogether as compared to what is normal
: for the actual type of structure.

: For structures where the surroundings are decisive for the class consideration should be given to future buildings,
: if any, in relation to the life time of the structure.

: For structures under erection, the class to which the structure is to be referred must be decided upon in each in-
: dividual case.

: In CEB-volume II, concrete, the values given in table 3.3.2b have been proposed.

: *Table 3.3.2b CEB-proposal for γ_m*

action combination	concrete	steel
ordinary	1.5	1.15
accidental	1.3	1.0

: According to the NKB-proposal:

$$\gamma_m = \gamma_{n1}\gamma_{m1} \tag{3.3.2b}$$

: where γ_{n1} is taken from table 3.3.2c and

: $\gamma_{m1} = 1.40$ for structures or structural members produced in a factory under special control arrangements
: and
: for structures or structural members where the characteristic values have been found by testing

: $\gamma_{m1} = 1.50$ in all other cases

: *Table 3.3.2c The partial coefficient γ_{n1}*

failure class	γ_{n1}
less serious	0.9
serious	1.0
very serious	1.1

: The value assumes a coefficient of variation not deviating from what is usual in timber structures, i.e. a coefficient of variation between app. 0.15 and 0.25.

: According to the NKB-proposal accidental action combinations are considered only for the failure class »very serious».

3.3.3 Design values of geometrical parameters

In most cases the geometrical parameters may be assumed as specified in the design. When the size of a geometrical parameter may have a significant effect on the structural behaviour or the resistance of the structure the design values a_d should be obtained from the characteristic value a as

$$a_d = a + \Delta a \tag{3.3.3a}$$

or

$$a_d = a - \Delta a \tag{3.3.3b}$$

where Δa takes account of the importance of variations in a and the given tolerance limits for a.

3.4 Analysis

For the ultimate limit states linear elastic, non-linear elastic and plastic theories may be applied according to the structural response of the structure or structural member to the actions. The characteristic values in chapter 5 are, however, derived from the test loads by the theory of linear elasticity and this theory should therefore also be used in the design of the members.

: It does not involve the requirement that the stress resultants in, for example, a lattice structure must be calculated under the assumption of linear-elastic behaviour but only that investigation of the individual members/cross-sections from the stress resultants found must be in accordance with the theory of elasticity, provided the strength values and simplified design methods used in the code are used directly.

For the serviceability limit states linear methods of analysis will usually be appropriate.

In the calculation of distribution of forces in statically indeterminate structures consideration should be given to unfavourable slip in joints etc.

The standard strength classes are introduced in CIB-W18/9-6-1.

REQUIREMENTS FOR MATERIALS

4.0 General

Strength and stiffness properties shall be based on tests for all actions to which the material may be subjected in the structure.

: This requirement does not prevent properties in some cases being based on comparisons with similar species or on
: well-established relations between the different properties.

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. The test specimens shall contain a grade-determining defect - preferably knots - in the zone with maximum force or bending moment.

4.1.1 Standard strength classes[1)]

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. Characteristic strengths and mean modulus of elasticity, in MPa

standard strength class		SC15	SC19	SC24	SC30	SC38
bending	f_m	15	19	24	30	38
bending	$E_{0,\,mean}$	6000	7200	8500	10000	12000
tension	$f_{t,0}$	6	9	16	20	25

: 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 the following standard
: grades: ECE6 : SC19 - ECE8 : SC24 - ECE10 : SC30.
: Annex 4 contains a survey of which national grades can be assumed to satisfy the requirements of the different
: standard grades. (Under preparation).

---

1) 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. The strength and stiffness values given are provisional.

The requirements apply to the glulam, not to the laminae. The additional tensile strength requirement for glulam made from more than one species is necessary to ensure a normal property-profile. The introduction of standard glulam strength classes does not prevent the introduction of other grades, e.g. grades with a higher $f_m/f_{t,0}$-ratio by using high-strength wood in the uttermost laminae.

4.2 Finger jointed structural timber

4.2.0 General

The manufacture of finger jointed structural timber should be according to rules and 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 $E_{0,mean}$ and the characteristic values for f_m and $f_{t,0}$ are not less than given in table 4.1.1.

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 $E_{0,mean}$ and 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 done according to rules and 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[1)]

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	38	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.

---

1) The specification of glued laminated members by strength classes (section 4.3.1 and table 5.2.0) has been included 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. The strength and stiffness values given are provisional.

CIB-W18 papers on sampling and evaluation are under preparation.

4.4 Wood-based sheet materials

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 and durability 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, bolts, 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 where surface corrosion will not significantly reduce the load-carrying capacity.

Table 4.7 Minimum protection against corrosion

climate class	nails, screws and bolts	other steel parts
1 2	none	galvanizing with a min. thickness of 20 μm[1)]
3	hot galvanizing with a minimum thickness of 70 μm	

1) In permanently heated buildings without artificial humidifying: none.

: 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 may have
: a corroding effect.

The modification factors correspond to the traditionally used reduction factors of $9/16 \sim 0.6$ for long-term load and 0.85 for exterior conditions. The reductions are probably less, especially for the low grades.

5. DESIGN OF BASIC MEMBERS

5.1 Solid timber members

5.1.0 Characteristic values

Characteristic values for the standard strength classes[1)] defined in section 4.1.1 are given in table 5.1.0a. 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 *Provisional*

		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}	375	450	525	600

* In rolling shear the shear strength may be put equal to $f_v/2$

Table 5.1.0 b Modification factors *Provisional*

values for	strength calculations		deformation calculations		
climate classes	1 and 2	3	1	2	3
permanent	0.55 (0.35)	0.45 (0.30)	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	0.9 (0.85)	0.75 (0.70)	1	0.8	0.7
instantaneous	1.1 (1.1)	0.95 (0.95)	-	-	-

Where a load case is composed of loads belonging to different load-duration classes the factors corresponding to the shortest load may be used.

Values in parentheses apply to tension perpendicular to grain.

: Attention is drawn to the fact that deformations 2-3 times those calculated with the values given may be ex-
: pected if green timber is allowed to dry under design load.

5.1.1 Tension members, compression members, beams and columns

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 be-

1) See footnote on page 4.1.

In cases where the influence of the size can be disregarded the conditions (5.1.1.1 a) and (5.1.1.1 b) can be generalized to an arbitrary angle, α, between stress and grain direction, viz.:

$$\sigma_t \leq \frac{1}{\sqrt{(\frac{\cos^2\alpha}{f_{t,0}})^2 + (\frac{\sin^2\alpha}{f_{t,90}})^2 + (\frac{\sin\alpha\cos\alpha}{f_v})^2}}$$

cf. formula (5.1.1.6 a).

The introduction of size factor for the two directions of practical interest has been found more important than having a general formula.

The values for $k_{size,90}$ are based on papers by J. D. Barrett (Wood and Fiber, Vol. 6, No. 2, 1974 and Canadian Journal of Civil Engineering, Vol. 2, 1975).

Together with the Hankinson formula this formula has been used for many years in many codes for designing traditional timber structures. No need has been felt for replacing it with a more sophisticated and more restrictive expression based on formula (5.1.1.6 a). The formula has been chosen in preference to the Hankinson formula due to its simplicity.

$k_{bearing}$ is based on a comparison of the rules in a number of codes, cf. CIB-W18/5-10-1, and on the work of G. Backsell (Swedish Institute for Building research, Report 12/66 1966).

tween the 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.

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 for nails and screws with a diameter of 5 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

$$k_{size,90} = \begin{cases} 1 & \text{for} \quad V \leqslant 0.02\ m^3 \\ (\frac{0.02}{V})^{0.2} & \text{for} \quad V \geqslant 0.02\ m^3 \end{cases} \qquad (5.1.1.1\ c)$$

for a volume of V uniformly loaded 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.

5.1.1.2 Compression without column effect

For compression at an 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 element in compression can carry the load in question. Refer to section 5.1.1.7.

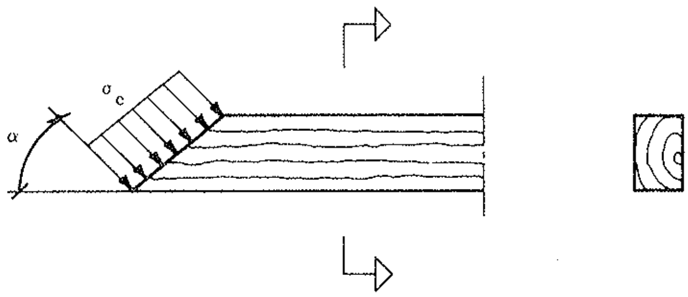


Fig. 5.1.1.2 a

For bearings on the side grain (α = 90°) formula (5.1.1.2 a) may be replaced by

$$\sigma_c \leqslant k_{bearing} f_{c,90} \qquad (5.1.1.2\ b)$$

A depth effect can be counteracted by stricter knot limitations for the bigger sizes. This may be the reason for the different traditions concerning depth factor, and it explains why it is impossible to give a general expression in this code.

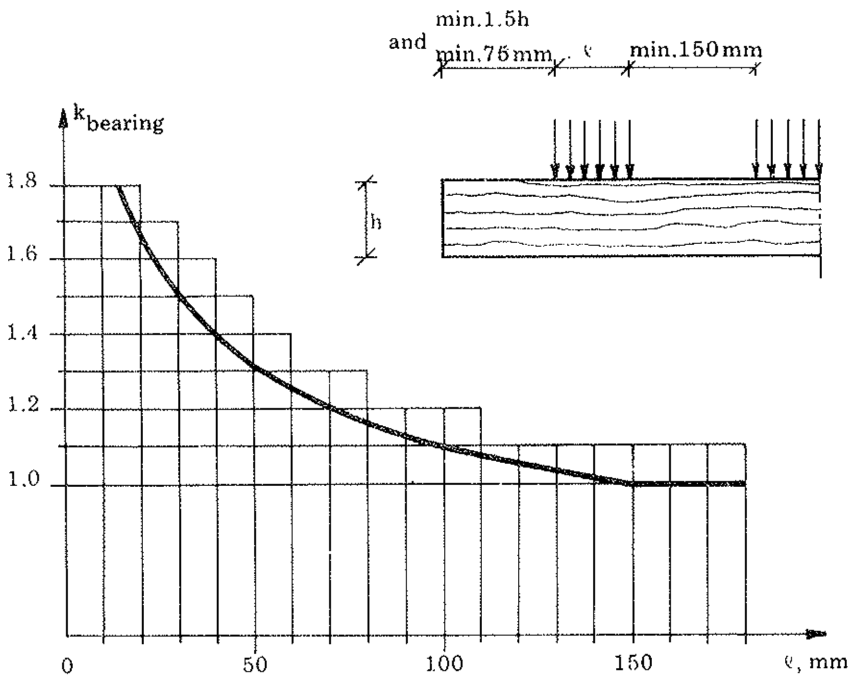
The limiting depth of 200 mm has been chosen on the basis of tradition in the countries where such a factor has been used.

At present the depth dependence for the ECE-grades is investigated. A provisional value of $\kappa = 1/9$ has been suggested.

Reference is also made to the survey in CIB-W18/5-10-1.

The rules concerning lateral instability are based on the work of Hooley & Madsen (ASCE Journal of the Structural Division, Vol. 70 (1964) ST 3) as described in CIB-W18/5-10-1.

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 $k_{bearing} = 1$.



$k_{bearing} = \sqrt[4]{150/\ell}$

$1 \leq k_{bearing} \leq 1.8$

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

The bending stresses, σ_m, calculated according to the theory of elasticity shall satisfy

$$\sigma_m \leq k_{depth} k_{inst} f_m \qquad (5.1.1.3\ a)$$

k_{dept} is a factor (≤ 1) taking into account the reduced strength of deep sections:

$$k_{depth} = \begin{cases} 1 & \text{for } h \leq 200 \text{ mm} \\ \left(\frac{200}{h}\right)^{\kappa} & \text{for } h \geq 200 \text{ mm} \end{cases} \qquad (5.1.1.3\ b)$$

The value of κ depends on among other things the wood species and the grading rules. Recommendations will be produced.

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.

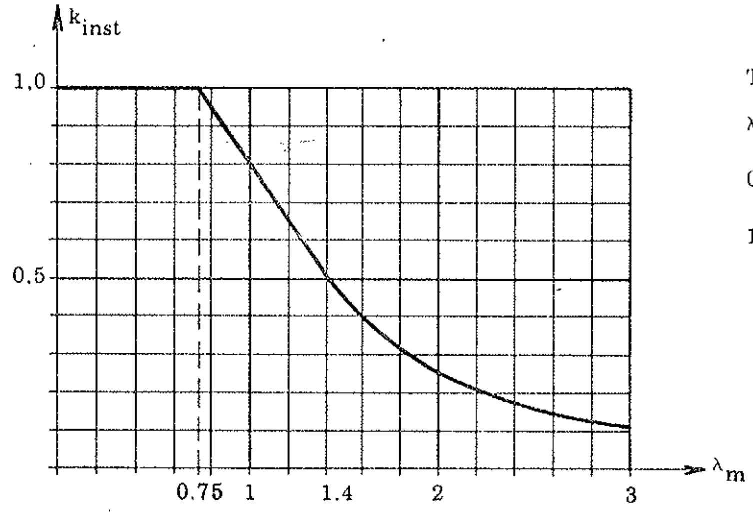
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

$$\lambda_m = \sqrt{f_m/\sigma_{m,crit}} \leq 0.75 \qquad (5.1.1.3\ c)$$

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

$\lambda_m < 0.75$; $k_{inst} = 1$

$0.75 < \lambda_m < 1.4$; $k_{inst} = 1.56 - 0.75\,\lambda_m$

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m^2$

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

$$\lambda_m = \sqrt{\frac{\ell_e h}{\pi b^2}\frac{f_m}{E_0}\sqrt{\frac{E_{0,mean}}{G_{mean}}}} \qquad (5.1.1.3\ d)$$

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	1.00
Simply supported, concentrated load at centre	0.85
Cantilever, uniform load	0.60
Cantilever, concentrated end load	0.85
Cantilever, end moment	1.00

The values apply to loads acting in the gravity axis. For downwards acting loads ℓ_e is increased by 2 h for loads on the top side and reduced by h for loads on the bottom side.

The reduction of the load near the supports is discussed in CIB-W18/9-10-1.

Further papers concerning size effect and the reduction for notched beams are under preparation and changes may be made.

5.1.1.4 Shear

The shear stresses, τ, calculated according to the theory of elasticity 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.

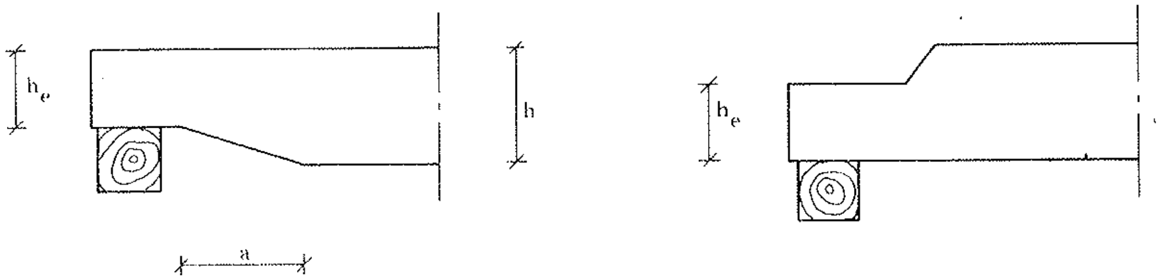


Fig. 5.1.1.4

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

$$\tau \leqslant \left(\frac{h_e}{h} + \frac{a}{3h}\right) f_v \qquad (5.1.1.4\ b)$$

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 and 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

At present no general theory of rupture exists, but only empirical or semi-empirical expressions for the most important practical cases, some of which are given below.

Plane stress

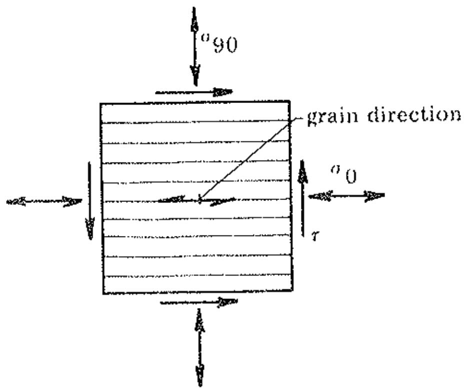
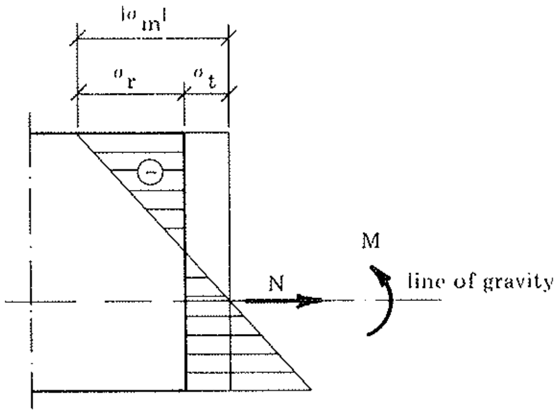


Fig. 5.1.1.6

The interaction formula is empirical and suggested by Norris, cf. Forest Products Laboratory, Madison, Report 1816, 1962. A more complicated formula

$$\left(\frac{\sigma_0}{f_0}\right)^2 - \frac{\sigma_0\sigma_{90}}{f_0 f_{90}} + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leqslant 1$$

given in the report mentioned is not supported by the tests described in CIB-W18/9-6-4, and CIB-W18 /11-6-3. Reference is also made to CIB-W18/11-10-1.



Formula (5.1.1.6c) (and (5.1.1.6e)) ensure that the resultant stress, σ_r, does not exceed f_m, which could be the case for unsymmetrical cross-sections if only (5.1.1.6b) (or (5.1.1.6d)) was used, since

$$\frac{\sigma_r}{f_m} = \frac{|\sigma_m|}{f_m} - \frac{\sigma_t}{f_m} = 1 + \frac{\sigma_t}{f_{t,0}} - \frac{\sigma_t}{f_m} \geqslant 1$$

for $f_{t,0} < f_m$.
See also CIB-W18/4-10-2.

(5.1.1.6f) is based on a paper by Möhler & Hemmer (Holz als Roh- und Werkstoff 35(1977)473-478).

The background for the column design is given in CIB-W18/2-2-1. The code format has emerged from discussion in CIB W18 and the work in connection with a revision of the British Code of Practice for The Structural Use of Timber.

The stresses shown in fig. 5.1.1.6 should - unless otherwise stated (see e.g. 5.1.1.2a) - satisfy the following condition:

$$\left(\frac{\sigma_0}{f_0}\right)^2 + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leqslant 1 \qquad (5.1.1.6\text{ a})$$

f_0 and f_{90} are chosen according to the sign of the σ_0 and σ_{90}, respectively. If σ_0 is a bending stress then $f_0 = f_m$.

Tension and bending
Only the case with tension in the grain direction is considered.

The stresses should satisfy the following condition

$$\frac{\sigma_t}{f_{t,0}} + \frac{\sigma_m}{f_m} \leqslant 1 \qquad (5.1.1.6\text{ b})$$

and in the parts of the cross-section, if any, where $\sigma_t + \sigma_m \leqslant 0$, furthermore

$$|\sigma_m| - \sigma_t \leqslant f_m \qquad (5.1.1.6\text{ c})$$

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

$$\frac{|\sigma_c|}{f_{c,0}} + \frac{|\sigma_m|}{f_m} \leqslant 1 \qquad (5.1.1.6\text{ d})$$

and in the parts of the cross-section, if any, where $\sigma_c + \sigma_m \geqslant 0$

$$\sigma_m + \sigma_c \leqslant f_m \qquad (5.1.1.6\text{ e})$$

: 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 should satisfy the following condition

$$\frac{\tau^2}{f_v} + \tau_{tor} \leqslant f_v \qquad (5.1.1.6\text{ f})$$

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 satisfy the following condition:

$$\frac{|\sigma_c|}{k_{col} f_{c,0}} + \frac{|\sigma_m|}{f_m} \frac{1}{1 - \frac{k_{col}}{k_E} \frac{|\sigma_c|}{f_{c,0}}} \leqslant 1 \qquad (5.1.1.7\text{ a})$$

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 $\lambda = \ell/i$, the material parameters and the initial curvature.

The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

$$e = \eta r_{core} \lambda \qquad (5.1.1.7\text{ b})$$

where r_{core} is the core radius.

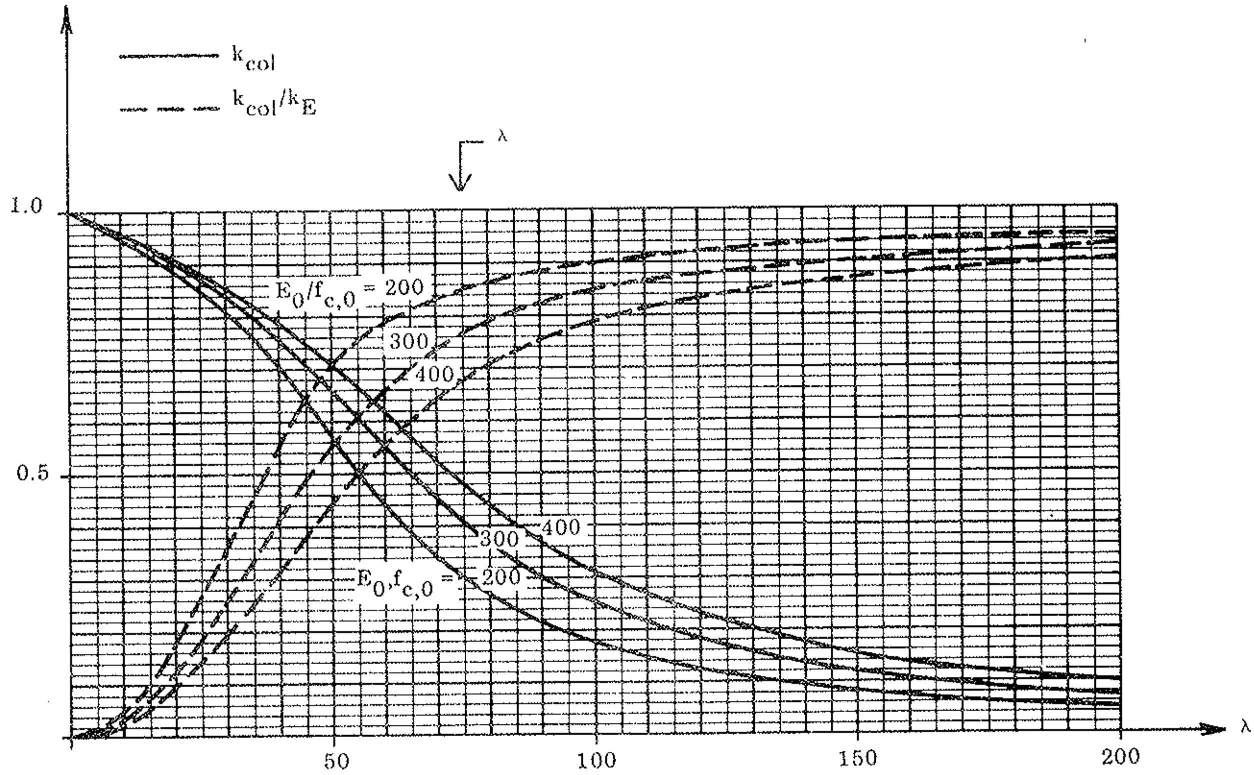
$$k_E = \frac{\sigma_E}{f_{c,0}} = \frac{\pi^2 E_0}{f_{c,0}\lambda^2} \qquad (5.1.1.7\ c)$$

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 against rotation	0.7
Restrained at both ends in position and one end against rotation	0.85
Restrained at both ends in position but not against rotation	1.0
Restrained at one end in position and against rotation and at the other end against rotation but not in position	1.5
Restrained at one end in position and against rotation and free at the other end	2.5

The slenderness ratio should not exceed 170, or for secondary members, 200.

Reference is made to CIB-W18/11-10-1.

5.1.2 Tapered beams

Relevant parts of section 5.2.2 may be applied.

The stresses in the outermost fibres taking into account the influence of the taper should satisfy (5.1.1.6a).

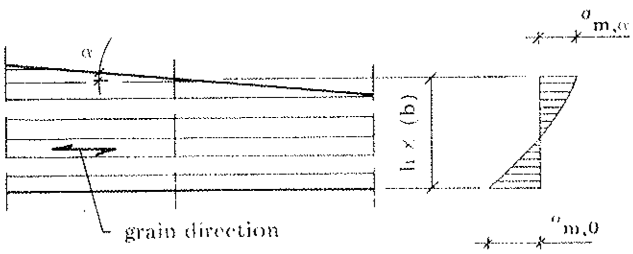


Fig. 5.1.2

: For a tapered beam with rectangular cross-section with the grain direction parallel to one of the contours and $\alpha \leq 20°$ the bending stresses in the outermost fibres can be calculated as

$$\sigma_{m,0} = (1 + 3.7 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.1.2\ a)$$

$$\sigma_{m,\alpha} = (1 - 3.7 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.1.2\ b)$$

: The stresses should satisfy the following conditions

$$\sigma_{m,0} \leq f_m \qquad (5.1.2\ c)$$

$$\sigma_{m,\alpha} \leq \frac{f_m}{\sqrt{1 + (\frac{f_m}{f_v} \tan\alpha)^2 + (\frac{f_m}{f_{90}} \tan^2 \alpha)^2}} \qquad (5.1.2\ d)$$

: where $f_{90} = f_{t,90}$ or $f_{90} = f_{c,90}$ depending on the sign of the stresses.

5.2 Glued laminated members

5.2.0 Characteristic strength and stiffness values

Characteristic values for the standard glulam strength classes[1)] 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 *Provisional*

		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}	600	750	750

* In rolling shear the shear strength may be put equal to $f_v/2$

1) See footnote on page 4.2.

The influence of notches has been found much more severe for glulam than for solid timber, and in some countries notching of glulam is not allowed. The expression has been proposed by Möhler, see CIB-W18/9-6-4.

The rules are in principle the same as in the Canadian Standard CSA-086/1977, but slightly simplified.

5.2.1 Straight beams and columns

Section 5.1.1 for solid timber applies except that formula (5.1.1.3b) should be replaced by

$$k_{depth} = \begin{cases} 1 & \text{for} \quad h \leqslant 300 \text{ mm} \\ \left(\frac{300}{h}\right)^{\kappa} & \text{for} \quad h > 300 \text{ mm} \end{cases} \qquad (5.2.1a)$$

and formula (5.1.1.4b) by

$$\tau \leqslant [1 - 2.8\,\frac{h - h_e}{h}(1 - \frac{a}{14(h - h_e)})]f_v \qquad (5.2.1b)$$

and notches with $h_e < 0.75h$ are not allowed.

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 perpendicularly to the grain satisfy the conditions 5.1.1.1 b, i.e.

$$\sigma_t \leqslant k_{size,90} f_{t,90} \qquad (5.2.2\text{ a})$$

with

$$k_{size,90} = \begin{cases} \frac{0.5}{V^{0.2}} & \text{for} \quad \text{uniformly distributed load} \\ \frac{0.35}{V^{0.2}} & \text{for} \quad \text{other loading} \end{cases} \qquad (5.2.2\text{ b})$$

$k_{size,\,90}$ should not be taken greater than corresponding to $V = 0.02\ m^3$.

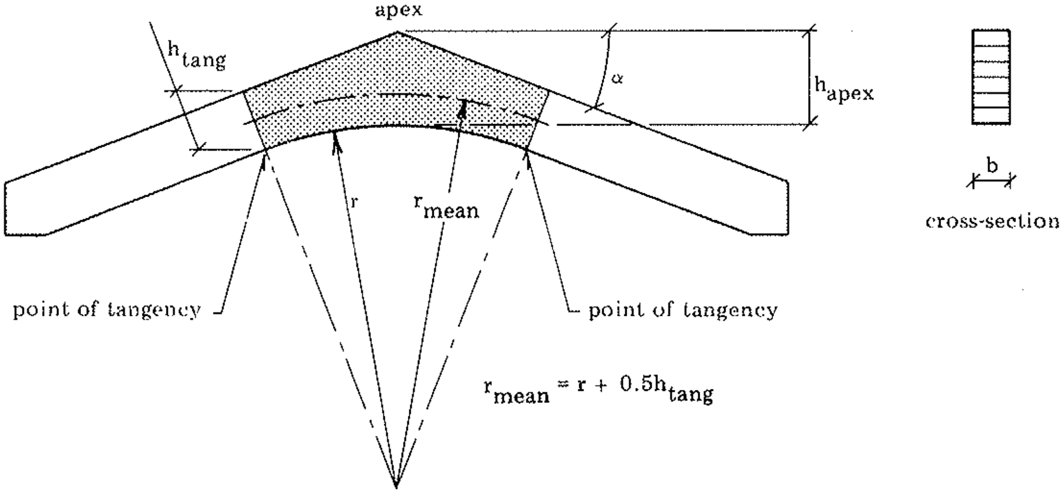


Fig. 5.2.2 a Double tapered curved beam

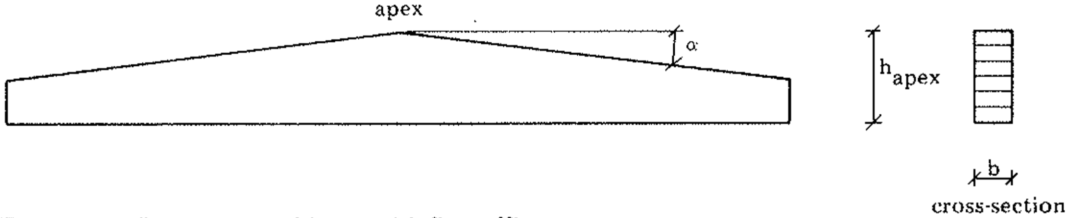


Fig. 5.2.2 b Double tapered beam with flat soffit

The background for the calculation of radial tensile stresses etc. is given by Foschi & Fox (See e.g. ASCE, Journal of the Structural Division, Vol. 76(1970) ST10.

The formula and the diagrams are based on papers by H. Blumer (among others Holzbau 6-8/1975) and Möhler & Blumer (Berichte aus der Bauforschung, Berlin 1974, Nr. 92).

For double tapered curved beams V is the beam volume in m^3 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 $V = 0.6\ bh^2_{apex}$.

Where there is an angle between the grain direction and the top or bottom the bending stress should satisfy the conditions in section 5.1.2.

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 $E_{0,mean}/E_{90,mean} = 30$.

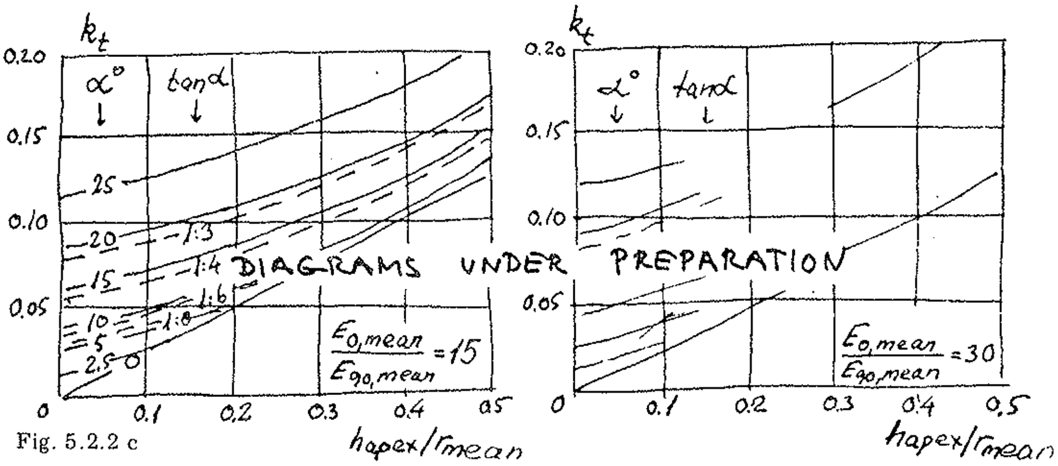


Fig. 5.2.2 c

The maximum bending stress in the apex cross-section occurs at the lower face and can be calculated as

$$\sigma_m = k_m \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2\ d)$$

where k_m is given in fig. 5.2.2 d for $E_{0,mean}/E_{90,mean} = 15$ and $E_{0,mean}/E_{90,mean} = 30$.

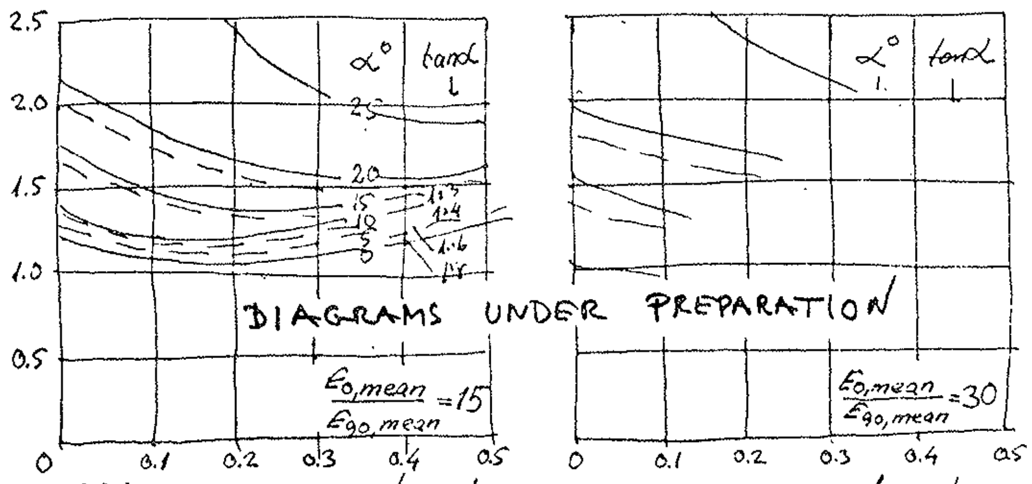


Fig. 5.2.2 d

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.

The formula is a simplification of the Wilson- and Hudson-formulas, see CIB-W18/5-10-1.

The values are suggested by K. Möhler, cf. CIB-W18/5-10-1.

5.2.3 Curved beams

This section applies to curves beams with constant, rectangular cross-section, see fig. 5.2.3 a.

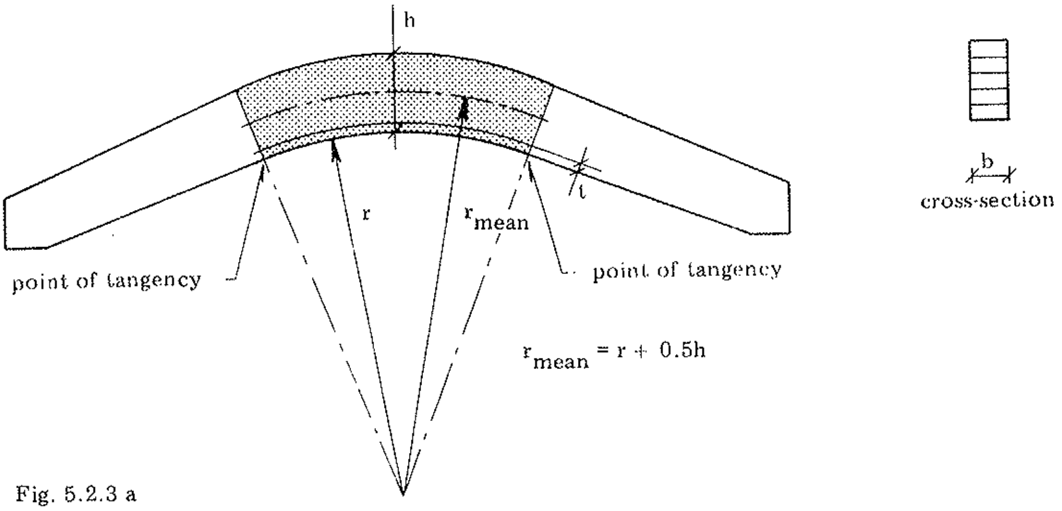


Fig. 5.2.3 a

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

$$k_{curv} = 0.76 + 0.001\frac{r}{t} \qquad (5.2.3\ a)$$

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.

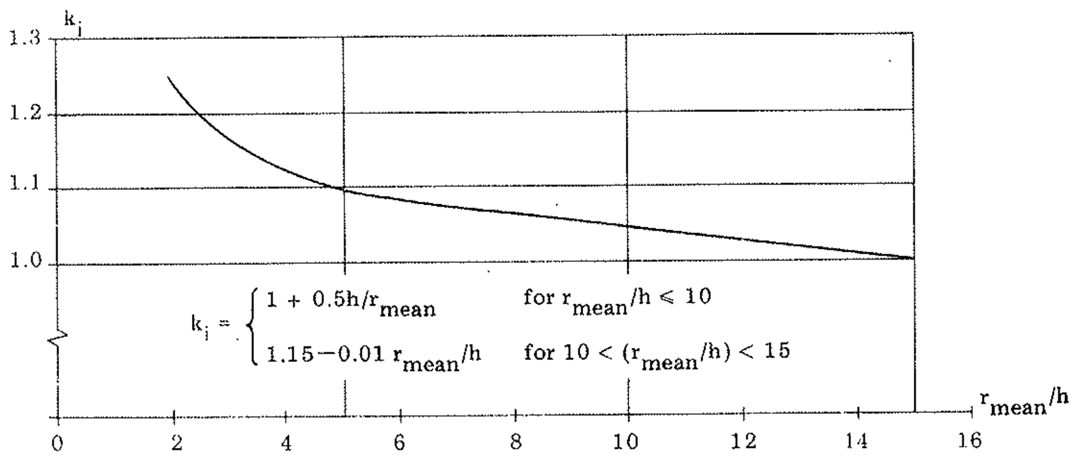


Fig. 5.2.3 b

This is a simplified version of the rules in Canadian Standard CSA-086-1977.

When the bending moments tend to reduce curvature (increase the radius) the tensile stresses perpendicular to the grain shall satisfy the condition

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.2.3 d}$$

where

$$k_{size,90} = \begin{cases} \dfrac{0.4}{V^{0.2}} & \text{for uniformly distributed load} \\ \dfrac{0.3}{V^{0.2}} & \text{for other loading} \end{cases} \tag{5.2.3 e}$$

V is the volume in m^3 of the curved part of the beam (corresponding to the shaded area in fig. 5.2.3 a). $k_{size,\,90}$ should not be taken greater than corresponding to $V = 0.02\ m^3$.

: The tensile stresses perpendicular to the grain in the curved part may be calculated as

:

: $$\sigma_t = \frac{1.5\ M}{r_{mean} bh} \tag{5.2.3 f}$$

Chapter 6 was not included in the third draft. The present text is a complete redraft of chapter 6 in CIB Timber Code, second draft, May 1978.

This will normally be on the safe side, since the joint strength depends on s_c^κ, where $0.5 \leqslant \kappa \leqslant 1.0$. A more refined system will be so difficult to use that it is not found justified.

Reference is made to CIB-W18 Proceedings - Meeting 11.

α_{nail} is in the range of 1.5 to 2.

To take into account the possibility of placing a nail in a knot or split most codes have a minimum requirement of at least 3-4 nails, even though 1 or 2 are in fact accepted in practice. Another rule has been discussed, namely the addition of one nail in joints where the calculation gives 1 or 2 nails, but the proposed test is more flexible.

6. JOINTS

6.0 General

When the joint is non-symmetric or where the load is eccentric consideration should be given to these factors by the determination of the behaviour of the fasteners as well as by the design of the members.

It should be taken into account that the load-carrying capacity of a multiple-fasterner joint will frequently be less than the sum of the individual fastener capacities and that some types of fluctuating load may cause a reduction, expecially when the stresses alternate between tension and compression.

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.

: Glue 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.

6.1 Joints with mechanical fasteners

6.1.0 Load-carrying capacities, general

The characteristic load-carrying capacity should be based on tests carried out in conformity with ISO/TC 165 DP 6891: Timber structures - Joints - Determination of strength and deformation characteristics of mechanical fasteners. Consideration should be given to the influence of drying after manufacture and to moisture variations in use.

For a number of fasteners characteristic load-carrying capacities under static load are given in section 6.1.1 - 6.1.4.

: In cases where the slip is important lower values should be used.
: Attention 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.

Where nothing else is stated the load-carrying capacities for the load-duration classes and climate classes defined in sections 2.2 and 2.3 are found by multiplication by the factors given in table 5.1.0 b.

6.1.1 Nails and staples

6.1.1.1 Laterally loaded nails

Timber-to-timber joints

The characteristic load-carrying capacity in N per shear plane can be determined by:

$$F = k_{nail}\, d^{\alpha_{nail}} \qquad (6.1.1.1\ a)$$

where d (in mm) is the diameter for round nails and the side length for square nails. The parameters k_{nail} and α_{nail} depend on, among other things, nail type and yield moment of the nails, wood species and grade (especially the density), the manufacture (e.g. preboring), and must be determined by tests.

If there are only one or two nails in a joint the values according to formula (6.1.1.1 a) are multiplied by 0.5.

For more than 10 nails in line the load-carrying capacity of the extra nails should be reduced by 1/3.

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 values can be used
: for Nordic softwood and other woods with corresponding properties
:
: $\alpha_{nail} = 1.7$
: $k_{nail} = 200\sqrt{\rho}$ (6.1.1.1 b)

where ρ is the specific gravity defined in section 2.1. No preboring is assumed.

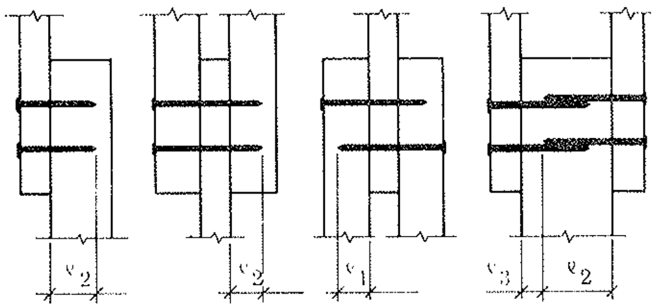


Fig. 6.1.1.1 a

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_2 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.

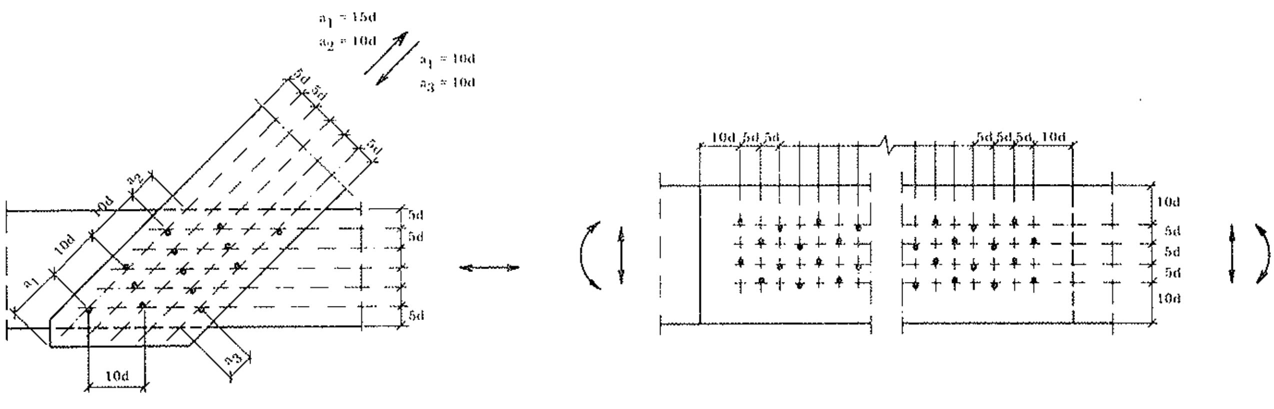


Fig. 6.1.1.1 b

Steel-to-timber joints

The recommendations for timber apply, but the load-carrying capacities for timber-to-timber joints may be multiplied by 1.25. Adequate strength of the steel plates is assumed.

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

2.5 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 *is applicable to all climate classes* for nailing perpendicular to the grain as in fig. 6.1.1.1 a and for slant nailing as in fig. 6.1.1.2 b. It 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 formulas (6.1.1.2 b - c) corresponding to the head being pulled through.

The length of the point is denoted ℓ_p.

$$F = \min \begin{cases} f_{axial} d(\ell - \ell_p) & & (6.1.1.2\ a) \\ f_{axial} dh + f_{head} d^2 & \text{for smooth nails} & (6.1.1.2\ b) \\ f_{head} d^2 & \text{for annular and spirally grooved nails} & (6.1.1.2\ c) \end{cases}$$

The parameters f_{axial} and f_{head}, depend, among other things, on type of nail, timber species and grade (especially density) and must be determined by tests. It is assumed that nailing is done in the green condition and that the test pieces are allowed to dry before testing.

For spirally or annularly threaded nails only the threaded part is considered capable of transmitting force.

For 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 normally be considered capable of transmitting force.

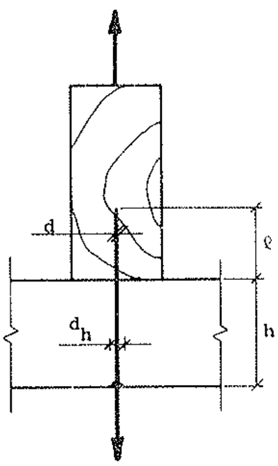


Fig. 6.1.1.2 a

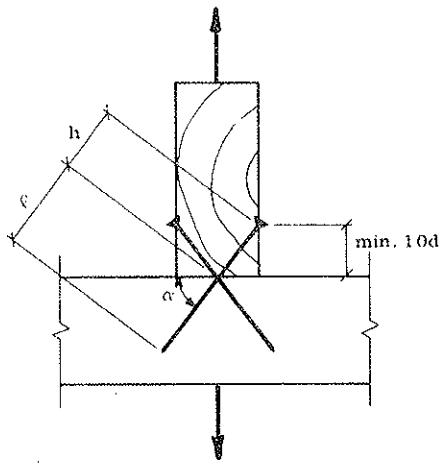


Fig. 6.1.1.2 b

: The distances for laterally loaded nails should be complied with and the distance to loaded edge by slant nailing
: should be at least 10d, see fig. 6.1.1.2 b.

Reference is made to CIB-W18/9-7-2.

: 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 $\rho \sim 0.36$ has been 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 threaded nails	- - -	to be determined by tests		- - -
annularly threaded nails	- - -	to be determined by tests		- - -

6.1.1.3 Staples

The rules for nailed joints apply. The angle between the crown and the grain direction should not be less than 30°.

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).

$$F = \min \begin{cases} 18\rho(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\text{ a}) \\ 35\rho\, k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\text{ b}) \\ 70\rho\, k_1 t_1 d & (6.1.2\text{ c}) \\ 42\sqrt{\rho}\, d^2 + 12\rho\, k_1 t_1 d & (6.1.2\text{ d}) \\ 75 d^2 \sqrt{\rho}\,\sqrt{(k_1 + k_2)/2}\,\sqrt{f_y/240} & (6.1.2\text{ e}) \end{cases}$$

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 $k_1 h_1 \leqslant k_2 h_2$.
For more than 4 bolts in line the load-carrying capacity of the extra bolts should be reduced by 1/3.

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

When the force acts at an angle to the grain it should further be shown that

$$V \leqslant \frac{2}{3} f_v b_e t \qquad (6.1.2\ f)$$

where V is the shear force produced by the bolts or dowels, t is the thickness of the member, and b_e is the distance from the loaded edge to the farthest point of the bolt, see fig. 6.1.2 a.

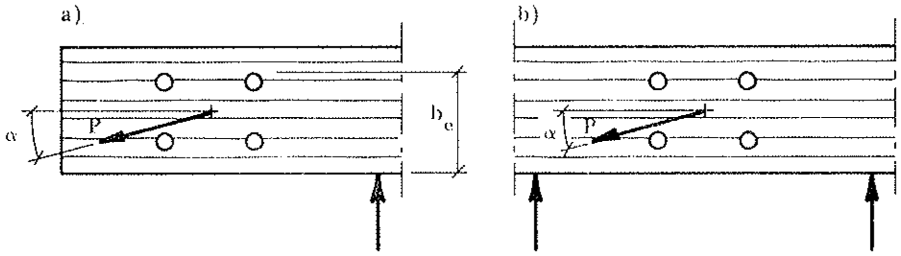


Fig. 6.1.2 a In case a) $V = P \sin\alpha$. In case b) $V = \frac{1}{2} P \sin\alpha$.

In multiple shear the load-carrying capacity can be found by considering the structure as a number of three-member joints.

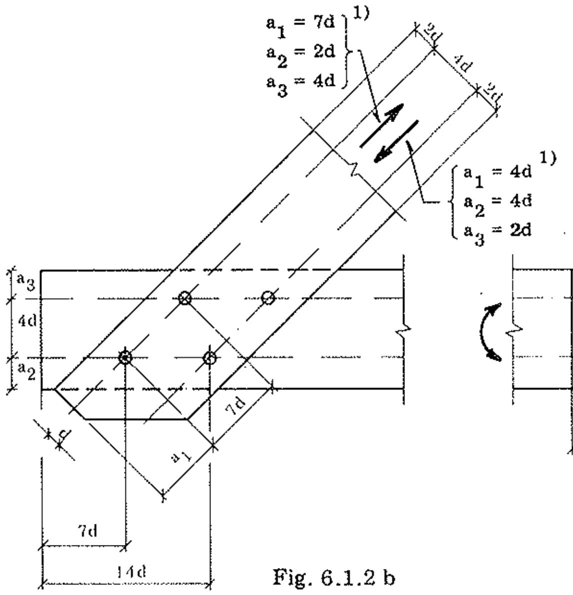
Where the side members are steel plates the loads calculated from the above formulas may be used with t_1 equal to t_2 equal to the thickness of the wood member.

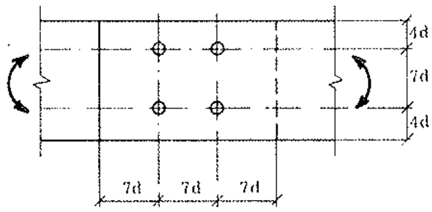
Where the middle member is a steel plate formula (6.1.2 b) is omitted and the values of the formulas (6.1.2 d) and (6.1.2 e) can be multiplied by 1.4.

For structural timber at least corresponding to SC19 (i.e. $\rho = 0.36$) the following is found by inserting into (6.1.2 a) - (6.1.2 e):

$$F = \min \begin{cases} 6.5(k_1 t_1 + k_2 t_2) d & \text{(only for two-member joints)} \quad (6.1.2\ g) \\ 12.5 k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ h) \\ 25 k_1 t_1 d & (6.1.2\ i) \\ 25 d^2 + 4.5 k_1 t_1 d & (6.1.2\ j) \\ 45 d^2 \sqrt{(k_1 + k_2)/2}\ \sqrt{f_y/240} & (6.1.2\ k) \end{cases}$$

Minimum distances are given in fig. 6.1.2 b.





1) The distance a_1 can be reduced to a minimum of 4d provided the load is reduced proportionally. If a yield strength f_y greater than 240 MPa is utilized a_1 should be increased by the factor $\sqrt{f_y/240}$.

Fig. 6.1.2 b

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).

Furthermore it should be verified that the condition (6.1.2 f) is satisfied.

It is assumed that:

- screws are screwed into pre-bored holes, see section 8.3,
- 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,

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.

: 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).

: The minimum distances are the same as for bolts (refer to section 6.1.2).

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. Adequate strength of the steel plates is assumed.

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.

It is assumed that the strength of the screw is adequate.

: For screws according to ISO 0000 the following can be assumed for structural timber at least corresponding to SC19

$$F = (30 + 12d)(\ell_t - d) \qquad (6.1.3.2\ b)$$

: It is assumed that the minimum distances and penetration lengths given for laterally loaded screws are complied with.

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 6.1.2. 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.

When a load is applied at an angle to the grain direction it should be shown that the condition (6.1.2 f) is satisfied. In this case b_e is the distance from loaded edge to farthest edge of the connectors, see fig. 6.1.4.

The rules for bolts should be complied with and the minimum distances between connectors should be sufficient to prevent splitting or shearing of the timber.

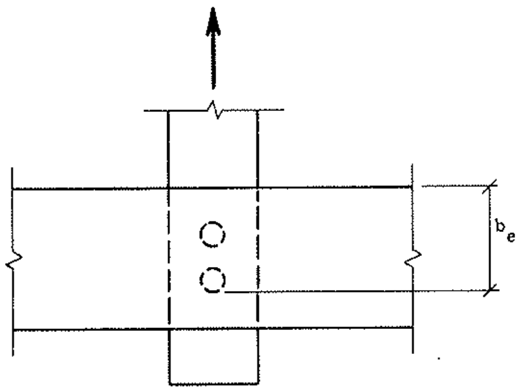


Fig. 6.1.4

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. in glued laminated timber 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 m².

The inclusion of limitations on $\sigma_{m,wc}$ and $\sigma_{m,wt}$ is discussed. According to established practice in USA and Canada they are disregarded in designing plywood beams. If the stresses are calculated according to the theory of elasticity they will for most plywood types be decisive.

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.

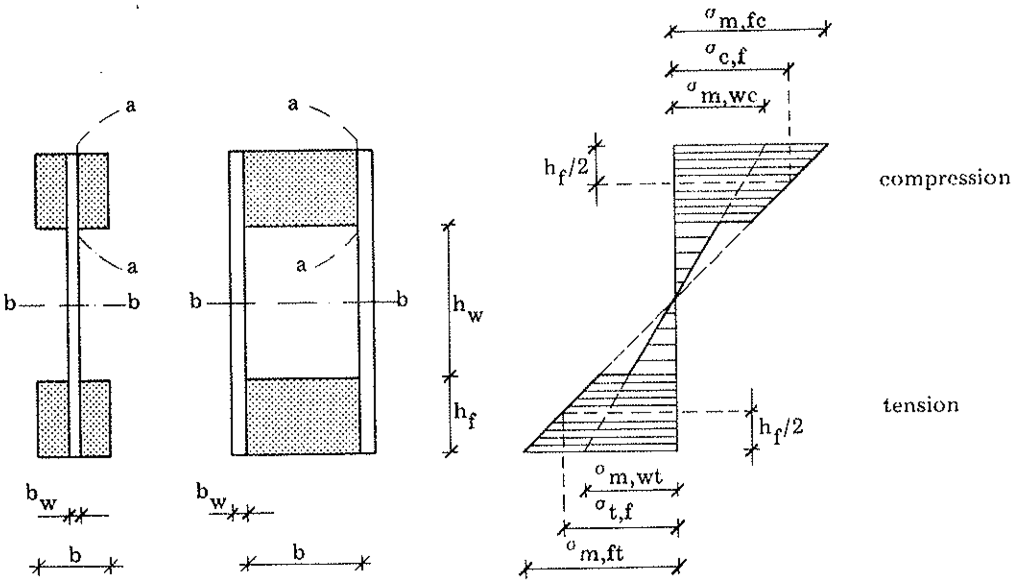


Fig. 7.1.1 a

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:

$$|\sigma_{m,fc}| \leqslant f_m \qquad (7.1.1\ a)$$

$$|\sigma_{c,f}| \leqslant k_{col} f_{c,0} \qquad (7.1.1\ b)$$

$$\sigma_{t,f} \leqslant f_{t,0} \qquad (7.1.1\ c)$$

$$\sigma_{m,ft} \leqslant f_m \qquad (7.1.1\ d)$$

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

$$|\sigma_{m,fc}| \leqslant k_{inst} f_m \qquad (7.1.1\ e)$$

where k_{inst} is determined according to section 5.1.1.3. This is on the safe side.

These rules are based on calculations according to the general method given below.

The method given is based on the theory of elasticity, see e.g. Halasz & Cziesielski (Berichte aus der Bauforschung, Heft 47) and CIB-W18/6-4-3.

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:

$$V \leqslant \begin{cases} f_v b_w (h_w + h_f) & \text{for} \quad h_w \leqslant h_{max} \\ f_v b_w h_{max} (1 + \dfrac{h_f}{h_w}) & \text{for} \quad h_{max} \leqslant h_w \leqslant 2h_{max} \end{cases} \qquad (7.1.1\ f)$$

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

Web	h_{max}
Plywood with $\varphi < 0.5$	$(20 + 50\,\varphi) b_w$
Plywood with $\varphi \geqslant 0.5$	$45\ b_w$
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:

$$\frac{\sigma}{\sigma_{crit}} + \left(\frac{\tau}{\tau_{crit}}\right)^2 \leqslant 1 \qquad (7.1.1\ g)$$

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

$$\sigma_{crit} = k_{buck,\sigma} \frac{\pi^2 \sqrt{(EI)_x (EI)_y}}{ta^2} \qquad (7.1.1\ h)$$

where $k_{buck,\sigma}$ for a number of cases is given in fig. 7.1.1 c and fig. 7.1.1 d.

τ_{crit} can be determined as

$$\tau_{crit} = k_{buck,\tau} \frac{\pi^2 \sqrt[4]{(EI)_x^3 (EI)_y}}{ta^2} \qquad (7.1.1\ i)$$

where $k_{buck,\tau}$ for pure shear is given in fig. 7.1.1 e.

The following notations are used

$(EI)_x$ 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.

$(GI)_{tor}$ is the torsional stiffness per unit length of the panel. For a homogeneous orthotropic panel, $(GI)_{tor} = Gt^3/3 + [\nu_{xy}(EI)_x + \nu_{yx}(EI)_y] \approx Gt^3/3$.

β_1 $= \frac{\ell}{a}\sqrt[4]{(EI)_x/(EI)_y}$. For an isotropic panel, $\beta_1 = \ell/a$.

β_2 $= 0.5\,(GI)_{tor}/\sqrt{(EI)_x(EI)_y}$. For an isotropic panel, $\beta_2 = 2G/E$.

a, ℓ, t see fig. 7.1.1 b.

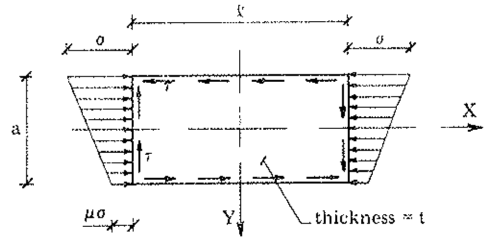


Fig. 7.1.1 b

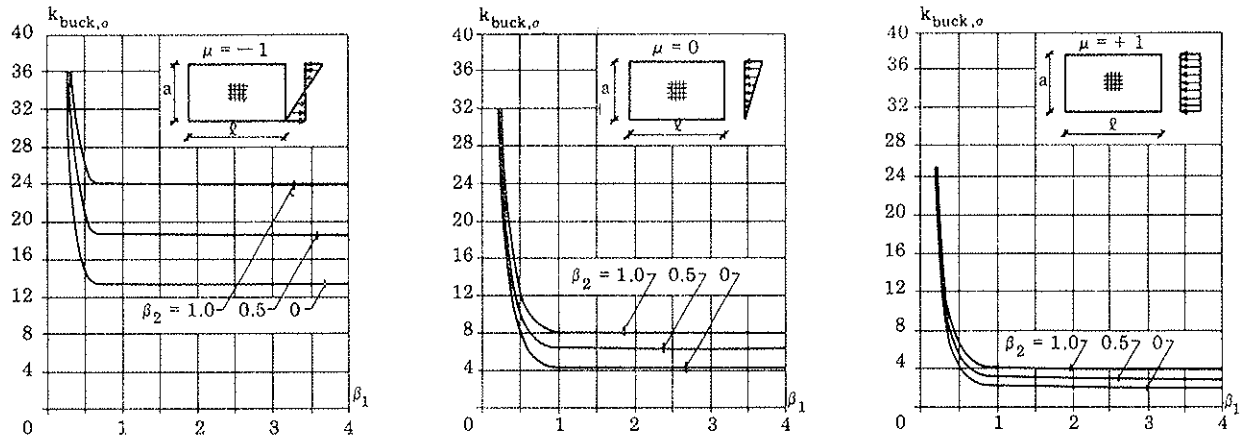


Fig. 7.1.1 c

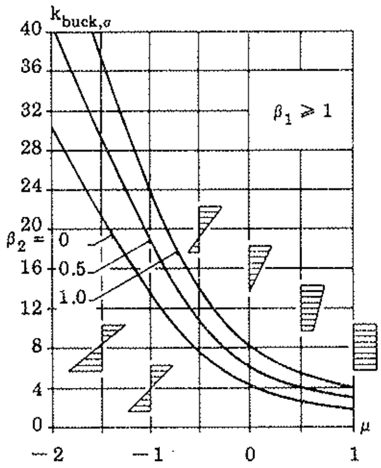


Fig. 7.1.1 d

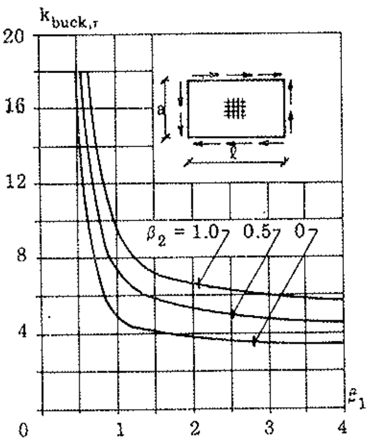


Fig. 7.1.1 e

In calculations of deflection the contributions from the shearing stresses in the webs should be taken into account.

The simple method has been shown by Booth to be satisfactory (IUFRO-Section 41, Madison, 1971). The effective width for uniform load for plywood corresponds approximately to $b_{f,e} = 0.15\ell$ (Möhler a.o. in Holz als Roh- und Werkstoff Nr. 21, 1963) but has been reduced to take into account the effect of uneven load distribution.

The limit b_{max} has been calculated according to the method given above.

The design of spaced columns and mechanically jointed components is discussed in CIB-W18/3-2-1.

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.

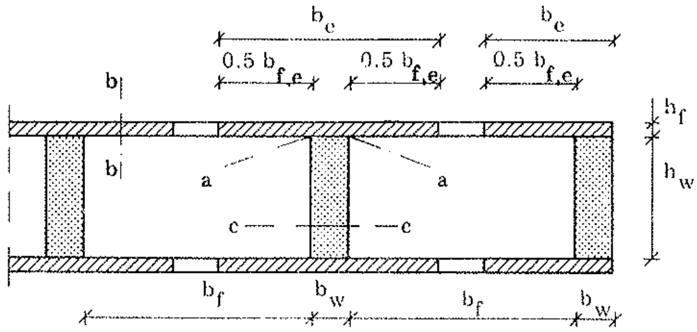


Fig. 7.1.2

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 a number of I-beams (taking the load on a width of $b_f + b_w$) and U-beams (taking the load on a width of $0.5\ b_f + b_w$) with an effective flange width, b_e, cf. fig. 7.1.2, where

$$b_e = b_{f,e} + b_w \tag{7.1.2 a}$$

or

$$b_e = 0.5\ b_{f,e} + b_w \tag{7.1.2 b}$$

respectively.

The free effective width, $b_{f,e}$ ($\leqslant b_f$), 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	$b_{f,e}/\ell$	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 Annex 7B and for lattice columns in Annex 7C.

The following guidelines were given in CIB-Timber Code, second draft:

> 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.

A sub-group was formed at the CIB-W18-meeting in Perth with the task of discussing these rules, especially with regard to the use of nail-plates.

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 $d < 5$ mm☆	$0.02\ E_0 d$
Round nails with $d > 5$ mm☆	$0.1\ E_0$
Bolts with toothed connectors	$1.3\ E_0$

E_0 is the modulus of elasticity of the timber in N/mm^2. 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 Annex 7A and for columns in Annex
: 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, slip and rotation in the joints, and rigidity of 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:
(under preparation)

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, 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, and to beams where lateral instability may occur; e.g. limiting bow to about 1/200 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 1 mm larger than the bolt. Washers with a side length of at least 3d and a thickness of at least 0.3d (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. The tolerances on the dowel diameter are − 0/+ 0.1 mm and the pre-bored holes in timber members 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 screw holes 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 might 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.

8.6 Surface treatment

Under preparation. Will contain recommendations on painting, staining, etc.

A more general method for types 2 and 3 allowing for unequal flange dimensions and different slip moduli in the interfaces is given in CIB-W18/11-4-1.

ANNEX 7A

MECHANICALLY JOINTED MEMBERS WITH I-, T- OR BOX CROSS-SECTIONS

1. SCOPE

Members with cross-sections as shown in fig. 1 are dealt with. The individual parts are full length and connected to each other by nails, bolts with toothed metal plate connectors or similar non-rigid fasteners.

A method is given to determine stresses, deflections and load on the fasteners of beams and the load-carrying capacity of columns, including the necessary requirements to the fasteners.

2. NOTATIONS

Reference is made to fig. 1. In all cases the Z-axis is a symmetry axis. For cross-sections of type 1 the Y-axis is a gravity axis, while for type 2 and 3 it is a symmetry axis.

For beams bending about the Y-axis is assumed.

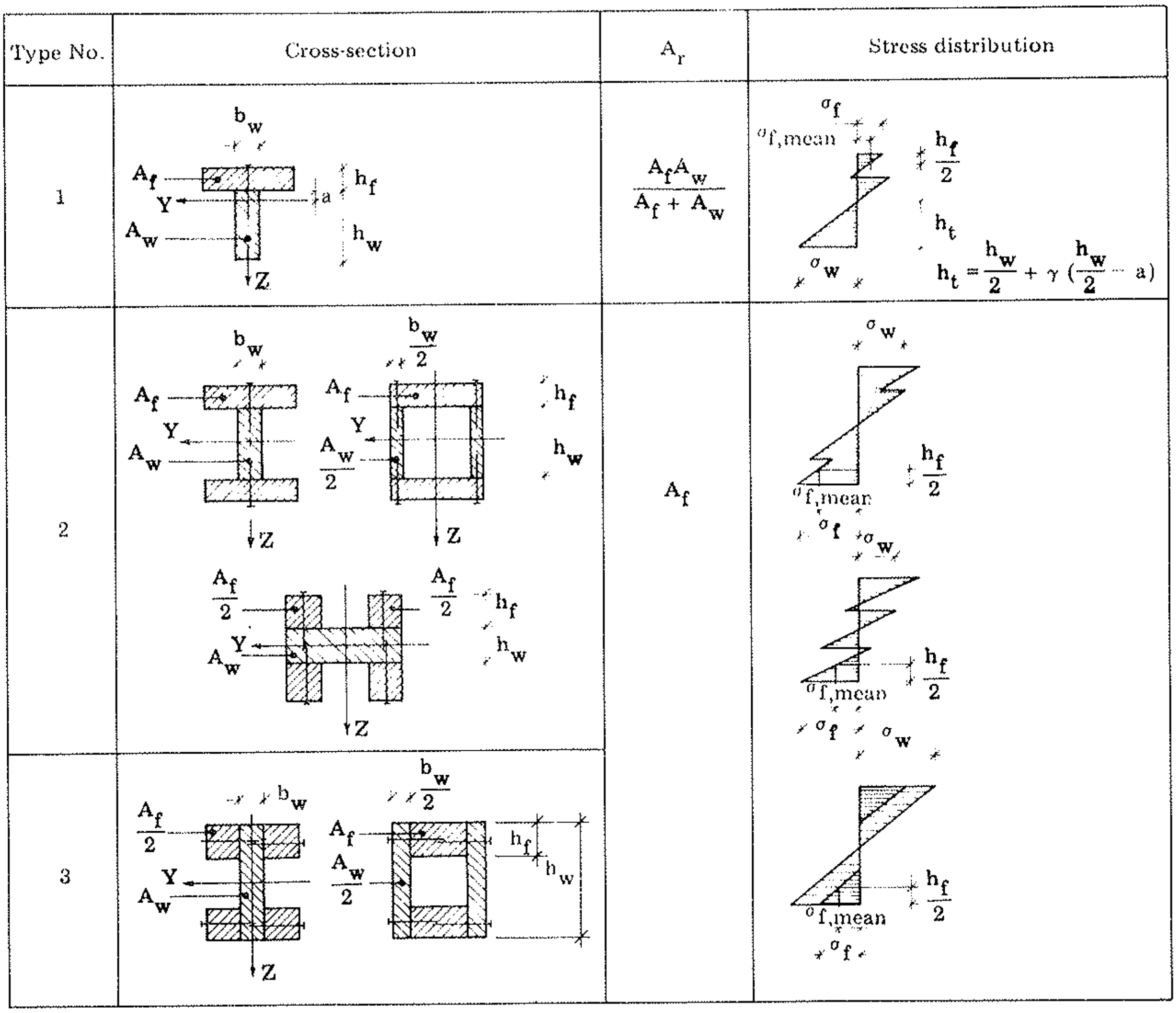


Fig. 1

A	Area	
	A_{tot}	Total area
	A_f	Flange area
	A_w	Web area
	A_r	See fig. 1
E	Modulus of elasticity	
F	Load on one fastener	
G	Shear modulus	
I	Moment of inertia (second moment of area)	
	I_{tot}	Total value calculated around the geometric gravity axis (Y-axis)
	I_{own}	Sum of moments of inertia for the individual parts around own gravity axis
	I_e	Effective moment of inertia, see formula (2)
K	Slip modulus, see section 3	
M	Bending moment (about Y-axis)	
P	Axial load on column	
	P_{crit}	Critical value
Q	Shear force (in the direction of the Z-axis)	
S	Static moment (first moment of area)	
	S_f	Static moment of flange about Y-axis
a	Distance of gravity axis from flange	
b	Width	
	b_w	Thickness of web
f	Strength	
	$f_{c,0}$	Compressive strength
h	Depth	
	h_f	Depth of flange
	h_w	Depth of web
	h_t	Depth of tension zone in web
k_{col}	Column factor	
ℓ	Span of beam or free length of column	
s	Spacing of fasteners (1/s is number of fasteners per unit length). If s is varying evenly in the longitudinal direction according to the shear force between s_{min} and $s_{max} \leqslant 4s_{min}$ an effective value of s equal to $s = 0.75\ s_{min} + 0.25\ s_{max}$ may be used.	
u	Slip between jointed members	
γ	Effectiveness factor, see section 4	
λ	Slenderness ratio	
	λ_e	Effective slenderness ratio
σ	Axial stresses	
	σ_f	σ in outermost fibre of flange
	$\sigma_{f,mean}$	σ in the middle of the flange
	σ_w	σ in outermost fibre of web
τ	Shear stress	

3. ASSUMPTIONS

The member is assumed either loaded in the Z-direction giving moments about the Y-axis, or with an axial load acting in the gravity axis.

The case where both bending moment and axial force are acting (beam columns) at the same time is only treated in a special case, cf. section 7.

The conditions are assumed linear-elastic and the following relation between the load on a fastener and the slip is assumed to apply.

$$F = Ku \tag{1}$$

All members are assumed to have the same modulus of elasticity, but the expressions may be extended to apply also to cross-sections where the cross-section members have different properties by transforming of the cross-section sizes in relation to their stiffness after the usual methods.

4. EFFECTIVE MOMENT OF INERTIA

The effective moment of inertia is determined by

$$I_e = I_{own} + \gamma(I_{tot} - I_{own}) \tag{2}$$

where

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} \frac{E}{K} s} \tag{3}$$

For beams with thin webs of plywood, particle boards or fibre boards, however,

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} \left(\frac{E}{K} s + \frac{E}{2Gb_w}\right)} \tag{4}$$

5. BEAMS

5.1. Calculation of stresses

Fig. 1 is referred to.

The stresses in cross-sections of type 1 are calculated from the following expressions:

$$|\sigma_w| = \frac{M}{I_e} h_t \tag{5}$$

$$|\sigma_{f,mean}| = \frac{M}{I_e} \gamma \left(\frac{h_f}{2} + a\right) \tag{6}$$

$$|\sigma_f| = |\sigma_{f,mean}| + \frac{M}{I_e} \frac{h_f}{2} \tag{7}$$

in cross-sections of type 2 from:

$$|\sigma_w| = \frac{M}{I_e} \frac{h_w}{2} \tag{8}$$

$$|\sigma_{f,mean}| = \frac{M}{I_e} \gamma \frac{(h_w + h_f)}{2} \tag{9}$$

$$|\sigma_f| = |\sigma_{f,mean}| + \frac{M}{I_e} \frac{h_f}{2} \tag{10}$$

and in cross-sections of type 3 from:

$$|\sigma_w| = \frac{M}{I_e}\frac{h_w}{2} \tag{11}$$

$$|\sigma_{f,mean}| = \frac{M}{I_e}\gamma\frac{(h_w - h_f)}{2} \tag{12}$$

$$|\sigma_f| = |\sigma_{f,mean}| + \frac{M}{I_e}\frac{h_f}{2} \tag{13}$$

5.2. Calculation of maximum shear stresses

For cross-sections of type 1 the maximum shear stresses occur where the stresses in the web are zero and can be calculated from

$$\max\tau = \frac{Qh_t^2}{2I_e} \tag{14}$$

For cross-sections of types 2 and 3 the maximum shear stresses occur in the middle of the web. For type 2 they can be calculated from

$$\max\tau = \frac{Q}{I_e b_w}\left(\gamma A_f \frac{(h_w + h_f)}{2} + \frac{1}{8}A_w h_w\right) \tag{15}$$

and for type 3 from:

$$\max\tau = \frac{Q}{I_e b_w}\left(\gamma A_f \frac{(h_w - h_f)}{2} + \frac{1}{8}A_w h_w\right) \tag{16}$$

5.3. Calculation of load on fasteners

The load per fastener can be determined from

$$F = \gamma\frac{QS_f}{I_e}s \tag{17}$$

5.4. Deflections

The deflections induced by the moment are calculated as usual, using the effective moment of inertia I_e.

6. CONCENTRICALLY LOADED COLUMNS

6.1. Load-carrying capacity

The load-carrying capacity corresponding to deflection along the Z-axis can be determined as

$$P_{crit} = k_{col}f_{c,0}A_{tot} \tag{18}$$

The columnn factor k_{col} is determined as for a corresponding column with rigid joints between the dross-section members, but the effective slenderness ratio:

$$\lambda_e = \ell\sqrt{\frac{A_{tot}}{I_e}} \tag{19}$$

is used.

For the T-cross-section and the I-cross-section, type 2, the load-carrying capacity for deflection in the Y-direction is found as the sum of the load-carrying capacity of the individual members, i.e. the stiffening effect that the members might have on each other is not taken into account.

6.2. Load on fasteners

The load on the fasteners can be calculated by eq. (17), assuming

$$Q = \begin{cases} \frac{P}{60}\,\frac{1}{k_{col}} & \text{for } 60 \leq \lambda_e \\ \frac{\lambda_e}{60}\,\frac{P}{60}\,\frac{1}{k_{col}} & \text{for } 30 \leq \lambda_e \leq 60 \\ \frac{P}{120}\,\frac{1}{k_{col}} & \text{for } \lambda_e \leq 30 \end{cases} \qquad (20)$$

7. COMBINED LOADS

In cases where small moments resulting from e.g. own weight are acting apart from axial load, the usual interaction formulas can be used for the stresses determined above.

8. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up timber columns. CIB-W18/3-2-1.

ANNEX 7B
SPACED COLUMNS WITH NAILED OR GLUED PACKS OR BATTENS

1. SCOPE

Columns as shown in fig. 1 are dealt with, i.e. columns with two or in certain cases three or four identical shafts jointed with packs or battens. The joints may be either nailed or glued or bolted with toothed metal plate connectors. Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio, whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

It is assumed that the construction rules given in section 3 are observed, and that the joints are designed for forces as stated in section 6.

Only concentrically loaded columns are dealt with.

2. NOTATIONS

Reference is made to fig. 1.

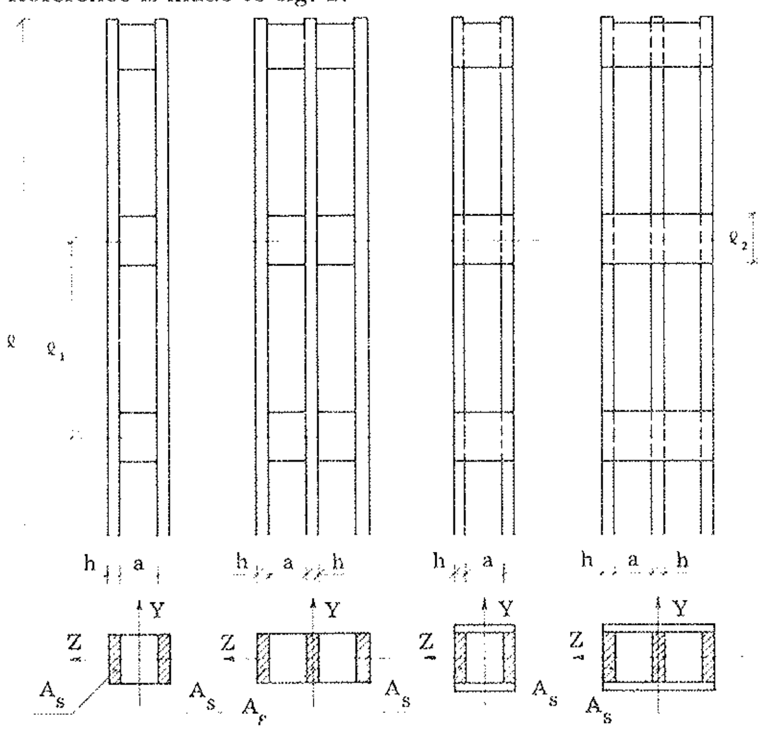


Fig. 1.

A	Area	
	A_s	Area of one shaft
	$A_{tot} = nA_s$	Total area
I	Moment of inertia (second moment of area)	
	I_s	I for one shaft about own gravity axis
	I_{tot}	Total moment of inertia about Y-axis
P	Column load	
	P_{crit}	Load-carrying capacity
Q	Shear force	

T_0	See fig. 3
a	Distance
	a — Free distance between shafts
	$a_1 = a + h$ See fig. 3
$f_{c,0}$	Compressive strength parallel to the grain
k_{col}	Column factor
ℓ	length
	ℓ — Free length of column
	ℓ_1 — Distance between midpoints of packs or battens
	ℓ_2 — Length of packs or battens
n	Number of shafts
λ	Slenderness ratio
	$\lambda = \ell \sqrt{\frac{A_{tot}}{I_{tot}}}$ Slenderness ratio for a solid column with the same cross-section
	$\lambda_1 = \ell_1 \sqrt{\frac{A_s}{I_s}}$ Slenderness ratio for the shafts
	λ_e — Effective slenderness ratio
η	Factor, see table 2.

3. ASSUMPTIONS

- The cross-section is composed of 2, 3 or 4 identical shafts.
- The Y- and Z-axes are symmetry axes.
- The number of free fields are at least 3, i.e. the shafts are at least jointed in the ends and in the third points.
- The free distance between the shafts is not greater than 5 times the lamella thickness ($a/h \leq 5$).
- The joints and packs and battens are designed for a shear force Q as stated in section 5
- The length of the packs should satisfy the condition $\ell_2/a \geq 1.5$.
- For nailed joints there should in each section be at least 4 nails or 2 bolts with metal plate connectors. For nailed joints at the ends apply that there should be at least 4 nails in a row in the longitudinal direction of the column.
- Battens should be made of structural plywood and their length satisfy the condition $\ell_2/a \geq 2$.
- The columns are solely subjected to concentrical axial loads.

4. LOAD-CARRYING CAPACITY

For deflection in the Y-direction the load-carrying capacity can be determined as the sum of the load-carrying capacity of the individual members.

For deflection in the Z-direction the load-carrying capacity are determined as

$$P_{crit} = k_{col} f_{c,0} A_{tot} \qquad (1)$$

The column factor k_{col} is determined as for solid columns, but the usual slenderness ratio

$$\lambda = \ell \sqrt{\frac{A_{tot}}{I_{tot}}} \qquad (2)$$

is replaced by the effective slenderness ratio

$$\lambda_e = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \qquad (3)$$

η is given in table 2.

Table 2: η

	Packs			Battens	
	glued	nailed	bolted*	glued	nailed
Long-term loading	1	4	3.5	2	6
Short-term loading	1	3	2.5	2	4.5

* with toothed metal plates.

5. SHEAR FORCES

The load on the fasteners and battens or packs can be calculated as stated in fig. 3, assuming

$$Q = \begin{cases} \frac{P}{60} \frac{1}{k_{col}} & \text{for } 60 \leq \lambda_e \\ \frac{\lambda_e}{60} \frac{P}{60} \frac{1}{k_{col}} & \text{for } 30 \leq \lambda_e \leq 60 \\ \frac{P}{120} \frac{1}{k_{col}} & \text{for } \lambda_e \leq 30 \end{cases}$$

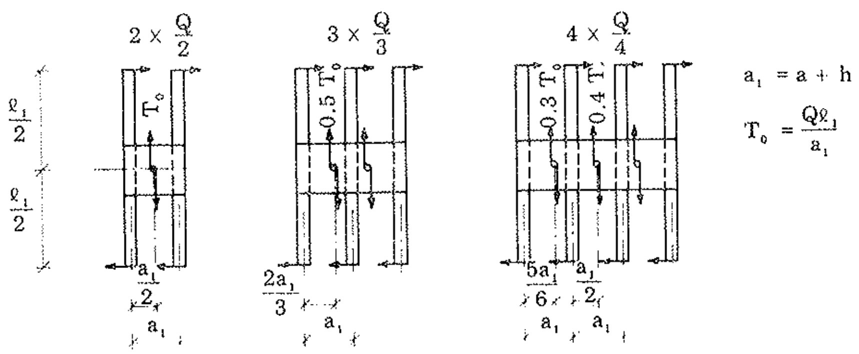


Fig. 2

6. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up columns. CIB-W18/3-2-1.

ANNEX 7C
LATTICE COLUMNS WITH GLUED OR NAILED JOINTS

1. SCOPE

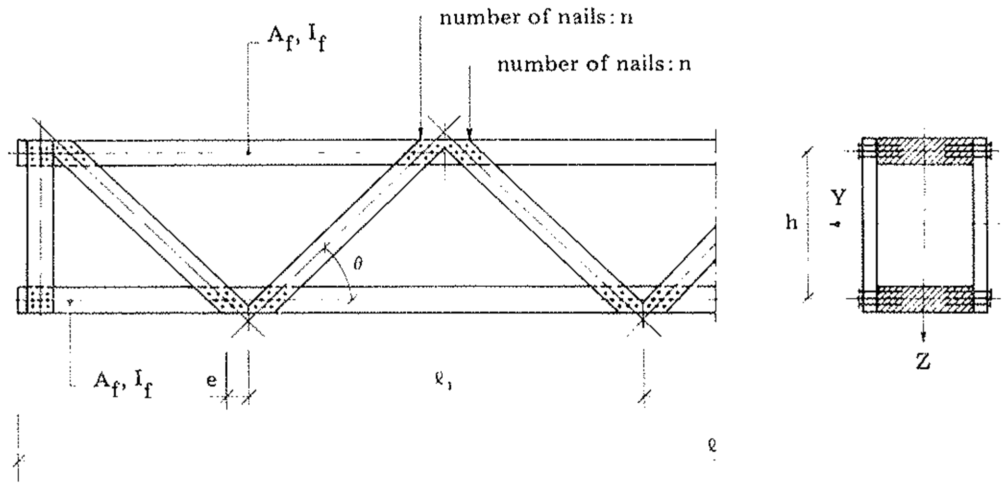
Lattice columns with N- or V-lattice and with glued or nailed joints are dealt with.

Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

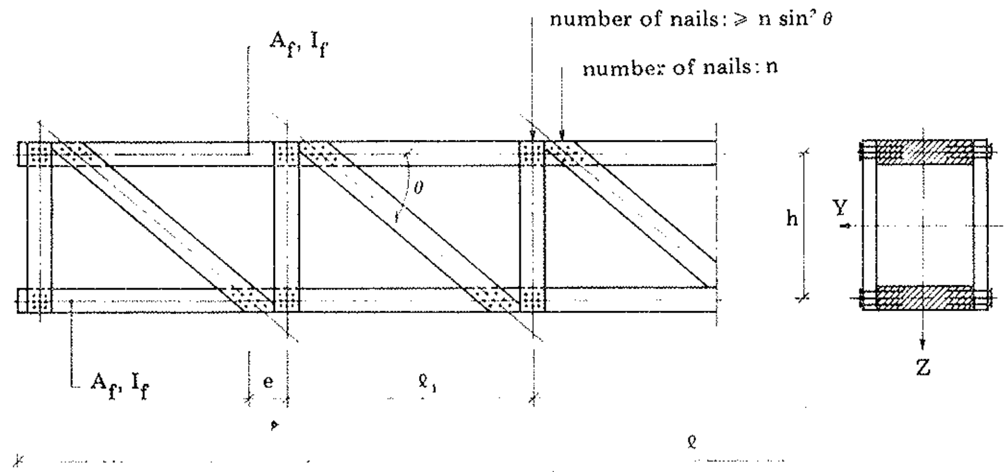
It is assumed that the joints are designed for forces as stated in sections 3 and 5.

2. NOTATIONS

Reference is made to fig. 1.



V-truss



N-truss

Fig. 1

A Area

- A_f Area of one flange

I Moment of inertia (second moment of area)

- I_f I for one flange about own axis
- I_{tot} Total value: $I_{tot} = 2I_f + \frac{1}{2} A_f h^2$

K Slip modulus for one nail, i.e. the force per nail that will cause a slip of 1

P Axial load on column

Q Shear force

e Eccentricity

$f_{c,0}$ Compression strength

h Depth of column (flange centre distance)

i_f Radius of gyration ($= \sqrt{I_f/A_f}$)

k_{col} Column factor

ℓ_1 Joint distance

n Number of nails per diagonal in a joint. If the diagonal consists of two or more pieces, n is the sum of nails, not the number of nails per shear face

θ Angle between a flange and a diagonal

λ Slenderness ratio

- λ_e Effective slenderness ratio
- λ_f Slenderness ratio of a flange (ℓ_1/i_f)

μ Parameter, see section 4.

3. ASSUMPTIONS

The structure is assumed to be symmetric about the Y- and Z-axes of the cross-section. However, the lattice of the two sides are allowed to be staggered the length $\ell_1/2$.

There should be at least 3 fields, i.e. $\ell >$ about $3\ell_1$.

The column should be designed for a shear force Q, as stated in section 5, i.e. the diagonals and joints should be designed for $Q/\sin\theta$.

In nailed structures at least 4 nails per shear should be used in each diagonal in each nodal point. At each end bracings should be used.

The slenderness ratio for the individual flange corresponding to the length ℓ_1 must not exceed 60, i.e.

$$\lambda_f \leqslant 60$$

Besides, it assumed that no local rupture is occurring in the flanges corresponding to the column length λ_1.

4. LOAD-CARRYING CAPACITY

The load-carrying capacity corresponding to deflection in the Y-direction is equal to the sum of the load-carrying capacity of the flanges for deflection in this direction.

For deflection in the Z-direction the load-carrying capacity is assumed to be equal to

$$P_{crit} = 2k_{col}f_{c,0}A_f$$

where the column factor k_{col} is determined as for a corresponding column of solid timber, but instead of the geometrical slenderness ratio of the column

$$\lambda = \ell\sqrt{\frac{2A_f}{I_{tot}}}$$

the effective slenderness ratio

$$\lambda_e = \lambda\sqrt{1+\mu}$$

is used, where μ is determined as stated below:

Glued V-truss:

$$\mu = 4\left(\frac{e}{i_f}\right)^2\left(\frac{h}{\ell}\right)^2$$

λ_e is not to be taken less than 1.05 λ.

Glued N-truss:

$$\mu = \left(\frac{e}{i_f}\right)^2\left(\frac{h}{\ell}\right)^2$$

λ_e is not to be taken less than 1.05 λ.

Nailed V-truss:

$$\mu = 25\frac{hEA_f}{\ell^2 nK\sin 2\theta}$$

Nailed N-truss:

$$\mu = 50\frac{hEA_f}{\ell^2 nK\sin 2\theta}$$

5. SHEAR FORCES

The column should be designed for a shear force Q given by

$$Q = \begin{cases} \dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 60 \leqslant \lambda_e \\ \dfrac{\lambda_e}{60}\dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 30 \leqslant \lambda_e \leqslant 60 \\ \dfrac{P}{120}\dfrac{1}{k_{col}} & \text{for } \lambda_e \leqslant 30 \end{cases}$$

6. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up columns. CIB-W18/3-2-1.

Current list of CIB-W18 technical papers

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

11-1-1 Safety Design of Timber Structures - H J Larsen

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 Källsner 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

PLYWOOD

STRESS GRADING

LOAD SHARING

DURATION OF LOAD

TIMBER BEAMS

ENVIRONMENTAL CONDITIONS

LAMINATED MEMBERS

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
11-6-3	Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4) - K Möhler

PARTICLE AND FIBRE BUILDING BOARDS

7-13-1	Fibre Building Boards for CIB Timber Code - O Brynildsen
9-13-1	Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto and J Ehlbeck
9-13-2	The Structural Use of Tempered Hardboard - W W L Chan
11-13-1	Tests on Laminated Beams from Hardboard under Short- and Long-term Load - W Nözyński
11-13-2	Determination of Deformation of Special Densified Hardboard Under Long-term Load and Varying Temperature and Humidity Conditions - W Halfar
11-13-3	Determination of Deformation of Hardboard under Long-term Load in Changing Climate - W Halfar

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
7-2-1	Lateral Bracing of Timber Struts - J A Simon
9-14-1	Timber Trusses - Code Related Problems - T F Williams
9-7-1	The Design of Truss-Plate Joints - F J Keenan
10-14-1	Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon
11-14-1	Design of Metal Plate Connected Wood Trusses - A R Egerup

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)

11-100-1 CIB Structural Timber Design Code (Third Draft)

11-100-2 Comments Received on the CIB Code

11-100-3 CIB Structural Timber Design Code: Chapter 3

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

11-102-1 Eurocodes - H J Larsen

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

1-105-1 Paper 1 A Note on International Organisations active in the Field of Utilisation of Timber - P Sonnemans

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

10-105-1 The Work 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

CIB-W18/12-103-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ISO/TC 165: Ottawa, September 1979

BORDEAUX, FRANCE

OCTOBER 1979

ISO/

our date	1979-07-10	our reference	3.8-55 HJL/jk
your date		your reference	

DIN, Deutsches Institut für Normung e.v.
Normenausschuss Bauwesen
Reichpietschufer 72-76
D-1000 Berlin 30
BRD

Dear Sirs,

Re: ISO/TC 165, Timber Structures.

To facilitate the discussion in September on the CIB Structural Timber Design Code I hereby send some comments on your letter dated 1979-05-15.

1. There is undoubtedly not only a depth effect but also a volume effect. It is however felt premature to include this in the code in general. The possibility of utilizing the effect in members with concentrated high stresses is being discussed in CIB-W 18.

2. The definitions on climate classes have been changed to make clear that they only define moisture content and not meteorological climate.

3. The load-duration classes normal and permanent are by nature different even though they have a common time limit.

4. The table could of course be omitted by reference to table 5.1.0a, but it is found convenient to have a complete description of the materials in chapter 4.

/2.

5. As above.
A production standard for glued laminated timber is under preparation in cooperation with Sous-Commission GLULAM.

6. The line on climate class 0 will be deleted. Your proposals concerning galvanizing do not correspond to to-day's practice.

7. Timber corresponding to the class SG 15 is in common use in many countries. A new class SG 38 is included in the fourth draft of the code. Density requirements, if needed, should be part of the grading rules necesarry to ensure conformity with the requirements of chapter 4.

8. To be discussed.

9. The complete equation would be

$$\sigma_{t,d} < k_{size,0}\, f_{t,0,d}$$

where $_d$ (denotes) design. Chapter 3 will include a statement saying that in all the formulas in section 5-7 the stresses are design values as well as the strength parameters, i.e. the stresses include load factors and the strength and stiffness values, safety factors and the modification factor from Table 5.1.0.b.

10. The section is being discussed and the rule will probably be changed.

11. The design rules for centrally loaded columns corespond to the DIN-rules used for many years but written in a general form to cope with different combinations of material properties.

Formula (5.1.1.7.a) can i many cases be replaced by $|\sigma_c|/k_{cal}\, f_{c,0}) + |\sigma_m|/f_m \leq 1$ and this could be mentioned in the comments.

12. In the fourth draft a value of $l_c/l = 2.5$ has been given (instead of 2.00). Your proposal seems better.

13. To be discussed in connection with the production standard for glulam.

14. To be discussed.

15. Yes, but in practice it is impossible in general to let the reduction factor depend on the type of rupture. For nail plates e.g. it is however possible, but they are not covered by the detailed rules of chapter 6.

The rules from table 5.1.0 h are assumed valid for joints too.

16. According to a German proposal the formula (6.1.1.1 a) will be changed to $F = k_{nail}d^{\alpha}$, which covers the DIN-formula by a suitable choice of α.

Your proposal concerning nails in boards corresponds to the assumption that a board with a thickness of t is equal to a timber member with the thickness:

- 1.5 t for particle board and semihard fibre board (where the 3. and 4. draft gives 1.0 t)
- 3.5 t for hard fibre board (where 3. draft gives 3.0 t and 4. draft 2.5 t).

The values can of course be changed if you have experimental and practical evidence of the soundness of your proposal.

17. To be discussed.
Since the values for improved nails are to be determined by tests for each make of nails no general definition is needed.

18. To be discussed.

19. The following requirement has been included: "It is assumed that the strength of the screw is adequate".
The correct formula (6.1.3.2 d) reads $F=(30 + 1 2d)(l_t-d)$.
The problem about d is to be discussed.

20. Clause 6.1.4 has no theoretical background but is only a simple method to take into account the fact that part of the load-carrying capacity is due to the bolt and thus depending e.g. on timber dimensions while part is due to the connector and practically independent hereof.

21. Proposals for simplifications will be appreciated but as far as I can see the rules given are simpler than the new DIN-rules.

22. The slip modulus for SC 19 is between 350 and 700 N/mm dependent on nail diameter. DIN 1052 gives 600 N/mm. (The correct expression for d < 5 mm is $0.02\ dE_0$).

23. Accepted in the 4. draft.

Your comments on ISO/TC 165 N 38 E will be discussed at the meeting in September.

Sincerely yours
for H. J. Larsen
Anne Sørensen
Anne Sørensen

Resolutions of TC 165 - meeting in Ottawa 1979-09-25/27

Resolution No. 9 (Ottawa No. 1)

The following revised scope was accepted:

Scope: Standardization concerning the design of load bearing structures of timber, wood products and appropriate related lignocellulosic fibrous materials, comprising technical requirements for design, for materials and for work of construction necessary to safeguard the design assumptions.

Resolution No. 10 (Ottawa No. 2)

The sceretariat was asked to make a second draft proposal of DP 6891 based on the comments made during the meeting. Following receipt of comments relative to the amended draft of DP 6891, this document shall be submitted to the Central Secretariat as a Draft International Standard.

Resolution No. 11 (Ottawa No. 3)

It was agreed that TC 165 should proceed with the work on Test Methods for Structural Timber. The latest version of the recommendation from RILEM 3TT/CIB W18 was adopted as Working Paper and should be circulatede for comments. Based on the comments received the secretariat, in cooperation with RILEM 3TT/CIB W18, should prepare a revised Working Paper. ISO/TC 55 should be informed and the members of TC 55/SC 1 invited to participate in the work.

Resolution No. 12 (Ottawa No. 4)

It was noted that ISO/TC 139 would produce standards for the testing of structural plywood based on RILEM 3TT/CIB W18 recommendations.

/2.

ISO/TC 139 should be asked to create a joint working group (TC 139 & 165) for this work at its next meeting (spring 1980).

Resolution No. 13 (Ottawa No. 5)

It was agreed to create a Working Group TC 165/WG 1 on Grouping of timber based on structural properties with the following scope:

1. The primary aim of the working group is to draft methods of grouping for sawn timber and joints based on the structural properties of the timber.

2. It is not the concern of the working group to discuss methods for the grading or the assessment of the strength of timber or the strength of joints. However, the working group should suggest acceptable methods of entry into the groups.

3. The methods proposed must be wide enough to enable the structural utilization of most of the merchantable timbers of the world.

4. Consideration should be given to the possibility that a new ISO committee on grouping be set up.

5. The working group is asked to produce a first rough draft for general discussion in 6-12 months.

6. The working group should further consider the possibility of applying methods of grouping to other structural timber products and components.

and the following members:

R. R. Leicester, Australia (convener)
P. Hoffmeyer, Denmark
J. Ehlbeck, Germany F. R.
Borg Madsen, Canada.
France would consider its participation.
Mexico hoped to be able to contribute.

/3.

Resolution No. 14 (Ottawa No. 6)

It was agreed that it was preferable to have the Structural Timber Design Standard as one document and that in addition simplified design methods should be given in annexes. CIB W18 should be asked to provide a target date for the production of a draft to ISO/TC 165. This draft is to be send out as an ISO/TC 165 Working Paper. A review period of at least six months should be allowed for comments.

DIN Deutsches Institut für Normung e.V. N 4

Normenausschuß Holz (NAHOLZ) ISO/TC 139/WG 6

ISO-Sekretariate für Holzwerkstoffe, Wohn- und Schulmöbel
TC 89, TC 136/SC 3, SC 5, SC 7, TC 139 und TC 151
CEN-Sekretariate für Spanplatten-Formaldehydbestimmung
TK 91 und Leitern TK 93

DIN Deutsches Institut für Normung e.V. · Kamekestr. 8 · D-5000 Köln 1

Mitglied der Internationalen Organisation für Normung (ISO)

To: ISO/TC 165 - Timber structures

From: D. Noack, Chairman of ISO/TC 139 - Plywood

Ihre Zeichen, Ihre Nachricht vom

Bei Antwort bitte angeben
Unsere Zeichen no-du

(02 21) 5713-1
Durchwahl 5713-523

Köln 1979-09-18

Subject: Report on activities of TC 139 related to the preparation of proposals of testing methods for plywood in structural grades for use in load-bearing structures

Ref.: Item 7, b) of the draft agenda of the Ottawa-Meeting of ISO/TC 165 as well as draft resolution 6 of TC's 165 London-Meeting in September 1977

According to resolution 6 of the London-meeting of TC 165, at which I had the honour to attend, it has been agreed regarding testing methods for structural plywood that TC 139 should be contacted in order to decide if the respective draft proposal of RILEM/CIB-3 TT - published by Professor Kuipers in Matériaux et Constructions, vol. 11, No. 62 - can be accepted as a basis for further international standardization and included in the programme of work of TC 139.

In December 1977 Professor Kuipers kindly sent me the manuskript ready for printing of this RILEM recommendation and in July 1978 I received a reprint of the draft by the technical secretary of CIB W 18. But because TC 139 did not meet during that time I could not discuss the subject within the plywood committee in 1978. The first opportunity to raise the question in TC 139 was a meeting of some working groups of TC 139, especially of the working group 6 on quality requirements of structural plywood, which took place only a few days ago on September 10 to 12, 1979, in Vienna/Austria.

In this meeting of TC 139 of structural plywood it has unanimously decided that:

1. The preparation of ISO Standards on test methods for the determination of physical and mechanical properties of structural plywood lies fully in the scope of ISO/TC 139.

Geschäftsräume: Kamekestr. 8 (am Friesenplatz) D-5000 Köln 1

Telex: 8 881 848 din d
Telegramm: DIN KÖLN

Bankkonten: Deutsche Bank AG
Depositenkasse Hohenzollernring
D-5000 Köln 1
BLZ 370 700 60 Konto-Nr. 2354223

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Zahlung bitte mit Vermerk: Für NAHOLZ

2. There is a need for the preparation of such test methods in order to judge the properties of structural plywood of different grades. These test methods seem especially necessary for comparative tests between medium-sized test pieces with natural and technical defects and the so-called small clear test pieces in order to determine the influence of the defects on the mechanical properties of structural plywood.

   The medium-sized test pieces should not be considered in relation to a continuous quality control of structural plywood in the factory which normally is carried out by using small clear test pieces.

3. The work of preparing relevant test methods for structural plywood shall be started by TC 139 immediately. This should favourably be done on the basis of the RILEM-draft 3 TT submitted by TC 165. This work will be carried out by Working Group 1 on testing methods for plywood of which I myself am the chairman.

   The test methods prepared should be as uniform as possible for all kinds of wood-based panel products (such as plywood, particle boards and fibre building boards) in order to enable direct comparison of board properties.

At the Vienna-Meeting of the plywood committee it has not been possible to go into technical details of different test methods, but it seems to be a fortune that the next general meeting of the Technical Committee on Plywood will take place in April/May 1980. In the mean time the relevant working party of TC 139 will be engaged in the subject by distributing the tentative recommendation of RILEM draft 3 TT-2 and by collecting comments regarding the proposed test methods. I hope that fast progress can be attained on the basis of the RILEM paper so that more concret results can be reported at the next meeting of TC 165.

One important point which has not been mentioned in the RILEM draft 3 TT-2 is a lower limit of the thickness of constructural plywood in relation to test methods. It will not be possible to apply all proposed test methods (for instance that of the bending test) also to very thin plywoods with thicknesses below 4 mm. We therefore have the question to TC 165 if a lower limit of the thickness of structural plywood should be considered, or will it be necessary to determine the mechanical properties of very thin plywood panels too. In this case special test methods may be necessary.

I deeply regret that - in contrary to my original intention and plans - I could not come to Ottawa in order to discuss this important item of test methods for structural plywood personally, but I hope that I could explain the present situation in this field with the foregoing report. I am sure that progress will be made during the next time and I am looking forward to a close successful cooperation between the Technical Committees 165 and 139 of ISO.

D. Noack

D. NOACK

NEDERLANDS NORMALISATIE-INSTITUUT
LID VAN DE INTERNATIONAL ORGANIZATION FOR STANDARDIZATION (ISO)
POLAKWEG 5 CORRESPONDENTIE-ADRES POSTBUS 5810, 2280 HV RIJSWIJK (ZH)

To the Secretariat of ISO/TC 165
"Timber Structures"
Dansk Standardiseringsraad
Aurehøjvej 12 and 15
Postbox 77
DK 2900 HELLERUP
Denmark

uw kenmerk your reference	uw brief van your letter of	ons kenmerk our reference	datum date
		ISO/TC 165/1859/PH/IL	1979-09-10

onderwerp subject

Dear Sirs,

With reference to point 9 of the agenda for the coming meeting of ISO/TC 165 in Ottawa we have pleasure to send you as annex 1 to this letter our proposal for a framework for a complete standard.
This framework can also be used if separate standards are preferred. In that case there are two main possibilities:
- each chapter is a separate standard, so that the coherence appears from a systematic numbering of the clauses;
- the paragraphes with the same last figure in their number form one standard, for instance:
  - 201.3.1 Nails: Material specifications;
  - 204.3.1 Nails: Characteristic strength;
  - 205.3.1 Nails: Deformations for different load duration classes;
  - 206.3.1 Nails: Design value for the strength etc.;
  - 601.3.1 Nails: Regulations in relation to the design and workmanship;
  - 701.3.1 Nails: Principles for basic testmethods etc.

With reference to point 12 of the agenda we enclose as annex 2 some comments on the content of the CIB-Timber Code.

We apologize for the late forwarding of these documents. We kindly ask you to make them available to the members of TC 165, especially to the delegates of the second meeting.

Sincerely yours,

(Ph.A. Willemsen)
Secretary building division

Annex: 2

telefoon	telegram-adres	telex	bankier	postrekening
(070) 90 68 00	Normalisatie Rijswijk	32173	Alg. Bank Nederland N.V. Rijswijk	25301

ISO/TC 165, Secretariat,
att.: H.J.Larsen
Dansk Ingeniørforening
Vester Farimagsgade 31,
DK 1606 Kbh.V Denmark

Dear Mr.Larsen,

In reply to your letter of 23.4.79 I can note the following:

a) My question concerned the safety factor of design stresses in Table 5.1.0a, which are connected with definite strength classe

b) About the problem of experimental base of strength classes you note 4.1.0 and thus short-time tests according to the rule of ISO/TC 165. Unfortunately I do not dispose of the latter as we as of test reports.

c) Though the machine-stress-grading is outside of the scope of C -W 18, the transition to characteristic strength of lumber sha not be avoid an examination in connection with the composition of Code.

d-e) As concerns the "climate classes" they do not foresee namely th m.c. of wood but only the relative air humidity.

f) Here were touched upon other points.

g) How about design stresses for timber logs?

I hope to hear from you further.

Yours sincerely

Yu.M.Ivanov

Annex 2 to our letter 1979-09-10

Comments from the Netherlands on

CIB Timber Code third draft - sept. 1978

General comments.

a) On the 'left' pages of the code, too less explanation, information or references have been given for a good insight of the principles and background of the code.

b) The limited range of strength classes mentioned in table 4.1.1. is too limited to cover the range of timbers used in the Netherlands.

c) The E values of table 5.1.0. for short-term loading is very low in comparison with those of the Dutch Timber Code NEN 3852.
For SC 15, for instance, $E_{o,mean}$ = 6,000 MPa.
For the "standaard bouwhout" of NEN 3852, which is comparable with SC 15, E_o = 10,000 MPa.
The E value for permanent loading and climate classes 1 and 2 amounts to:

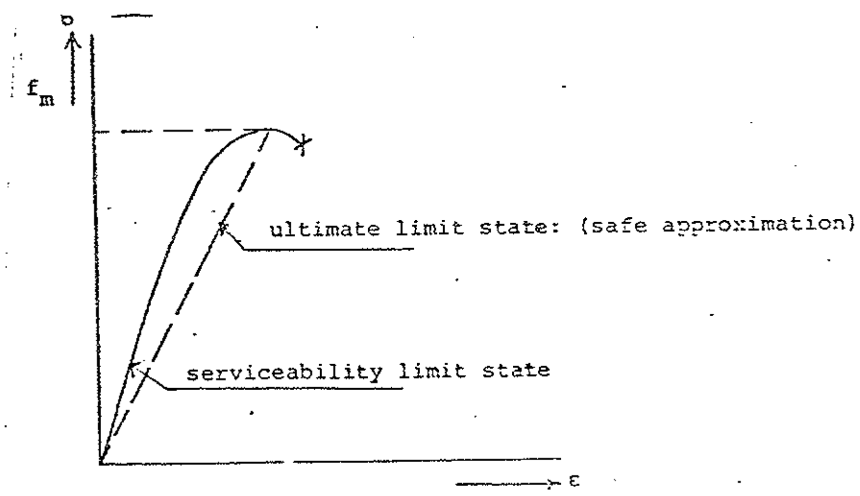
E_o = (0.6 à 0.7). 6000 = 3,600 à 4,200 MPa.

According to NEN 3852, including a creepfactor of 1.0:

E_o = 10,000/(1+1) = 5,000 MPa.

The C I B stiffness is considerable less than according to NEN 3852.

d) In table 5.1.0 two groups of E values have been given, for strength and for deformation calculations resp.
This splitting up is only acceptable if the corresponding limit state is dictated.
An explanation of the lower E value for strength calculations might be the non-linear stress-strain relation up to rupture of the timber (see figure below). In the ultimate limit state it is a safe approach to take a low value for the E in the linear elastic theory. However such information is missing in C I B code.
Furthermore for imposed deformations the resulting stresses will follow from the strains and the E value: a too low value of E may result in an unsafe structure.



Comments on chapter 6: Mechanical Fasteners.

e) The necessary information about force-deformation relation, is missing.

f) The reduction of the cross section by the presence of a fastener, should be taken into account for construction elements loaded bij tensile forces.

g) For each type of fastener corresponding minimum and maximum dimensions of timber should be given.

h) The splitting up of the load carrying capacity of a fastener comprising bolt and connector in a contribution from the bolt and from the connector separately is allowed for toothed plate connectors but not for split ring connectors.

i) It should be allowed to have smaller minimum distances between fasteners etc, bij taken into account reduction factors for the strength.

j) The caracteristic strength for nailed connections has been based on the strength of the whole connection.
For screws, bolts and dowels however, the strength of the connection has been based on the characteristic embedding strength of the timber and the yield point of the steel (thus not the ultimate strength), although the failure mechanism can be the same as for nails.
To our opinion the charateristic strength should be based on the strength of the whole connection for both nailed and other connections.

INDGÅET
22 MAJ 1979
J.nr. 3.8-55

14,10

15615 dansta dk
5449 iirs ei

good afternoon.

10th. may. '79. cn.

att: ms. anne soerensen. - secretariat of iso/tc 165.

i refer to final paragraph of your telex 10/5/79. doc. n 38e and 3rd draft of code for timber structures was received.

we have no written comment on n 38e.
we commented in february on 3 rd. draft of the code as follows.
1. clause 4.1.1
'' use ece recommended standard (ref tim/wp 3/ac 3/8) dated 24th. june 1977, as the reference to stress grades, and not the references contained in this clause,

2. clause 5.1.1.
'' change title to.
beams, colums, struts and ties or, alternatively, members in bending , tension and flexure''.

3. clause 7.3
national standards bodies should be free to specify their own standards. in ireland, irish standard 193 (p) was published in march 1978 called '' timber trussed rafters for roofs''.

regards.
n. v. torpey.
standards division.

5449 iirs ei
15615 dansta dk

DIN Deutsches Institut für Normung e.V.

NABau · Normenausschuß Bauwesen
Building Division of DIN

INDGÅET
21 MAJ 1979
J.nr. 3.3-5

DIN

Mitglied der Internationalen Organisat für Normung (ISO)

Normenausschuß Bauwesen · Reichpietschufer 72-76 · D-1000 Berlin 30

Secretariat ISO/TC 165
Dansk Ingeniørforening
Vester Farimagsgade 31
DK-1606 København V

Neue Anschrift ab 01. April 1979:
DIN
Deutsches Institut für Normung e.V
Normenausschuß Bauwesen
(NABau)
Burggrafenstr 4-10 · Postfach 1107
D-1000 Berlin 30
Tel.: (030) 2601-1 · Durchw. 2601 501

Ihre Zeichen, Ihre Nachricht vom	Unsere Zeichen, Unsere Nachricht vom	Telefon (030)	Berlin
	ISO 165/ack	26 22 031	1979-05-15

Subject: ISO/TC 165 "Timber structures"

Dear Sirs,

Please find enclosed the German comments on document "CIB Structural Timber Design Code, Third Draft, September 1978" of ISO/TC 165 and on document N 38 E.

Yours sincerely,
Building Division of DIN

Dr. Kiehl

Encl.

Geschäftsstelle: Reichpietschufer 72-76, D-1000 Berlin 30 · Telex: 1 85 413, 185413 idd
Kontoverbindungen: Bank für Handel und Industrie, BLZ 100 800 00, Konto Nr. 30 22 241 · Sparkasse der Stadt Berlin West, BLZ 100 500 00, Konto Nr. 0990002180 · Postscheckamt Berlin West, BLZ 100 100 10, Konto Nr. 208 76 108

Gleitende Arbeitszeit · Besuche und Telefonanrufe werden in der Kernarbeitszeit montags bis freitags von 9 bis 15 Uhr erbeten

German Comment on Document "CIB Structural Timber Design Code, Third Draft, September 1978" of ISO/TC 165

After revision of this document by our experts, we propose the following changes and supplements; at the same time we want to give general suggestion concerning the development of an international standard for timber structures.

1 Clause 2.1: For the bending strength of solid and laminated timber, the characteristic values are related only to a certain section depth. In our opinion there is also a dependency from the length and the cross sectional width of the mostly stressed part of the beam. For finger-jointed laminated beams this influence seems not to be so decisive as the weakness of the figure joint

2 Clause 2.2: The temperature according to the indicated climate classes must be defined more specifically, excluding possible misunderstandings. In this case the temperature should not be of interest, because facts are based on the moisture content of the timber, which results mostly from the relation humidity of the surrounding air. Temperatures between -25°C and +35°C can occur at every climate class.

3 Table 2.3a: The load duration classes "permanent" and "normal" should be put together, because exact separation seems to be impossible in relation to strength and deformation under loading times of more than 10^5h.

4 Table 4.1.1: This table should be eliminated , because the va s mentioned in there, are also mentioned in table 5.1.0a.

5 Table 4.3.1: This table should be eliminated, because the values mentioned in there are also mentioned in table 5.2.0.

The footnote of table 4.3.1 should be restricted to the tension zone only (instead of both parts). Appropriately, indications similar to those in DIN 1052 part 1, edition October 1969,

clause 11.5.2 and clause 11.5.6 should be included, i.e.:
"The quality timber requirements in DIN 4074, as far as glued construction members, which are built up from individual parts of smaller cross sections are concerned, refer in general only to the compound member and not to its individual parts. Such parts however, that are lateron working in the tension zone (boards) must on their own be of the prescribed quality grade. For glued laminated beams of rectangular cross sections that are subject to bending stresses, this applies to all lamellas in the range of 15% of the depth of the beam, however at least to the two outer lamellas of the tension side".

"For laminated beams subject to bending stresses, the top and bottom layers in at least 1/5 of the cross section depth, however at least two lamellas must be made of lamellas without joints or of lamellas with spliced or finger-jointed butts.

Joints in the interior of mainly bended or compressed laminated members can be butt-jointed. These joints must be staggered by at least 50cm in adjoining layers".

6 Table 4.7: Climate class 0 must be deleted in accordance with the indications given in clause 2.2. Furthermore a better protection against corrosion should be taken into consideration, i.e.: for "nails, screws and bolts" in climate class 2, for thin nails with a diameter of $\leq$ 3mm, there should be a protection by "galvanizing with a minimum thickness of 70 μm".

For other steel parts the following inclusion should be made for climate class 3: " galvanizing with a minimum thickness of 220 μm".

7 Table 5.1.0a: The indicated class SC 15 could be omitted, be cause the timber qualities given there, seem to be inappropria for timber structures; this class with the specifications give is not mentioned in ECE.

Furthermore the table should include the density for the singl classes.

In addition it is proposed to admit a higher quality class wit the indication SC 38, which would correspond to the class spec fication of SCL 38 (glued laminated members).

8 Clause 5.1.1.0: Concerning the application of nails and srew with a diameter $\leq$ 5mm, there is no clear definition concerning cases of prebored holes in the last paragraph. If the holes ar prebored, reductions in cross sectional area should be made in any case.

9 Clause 5.1.1.1: The equations given, do not consider the fac mentioned in table 5.1.0b. Instead of writing $f_{t,o}$ in equation 5.1.1.a, it would be better to write $f'_{t,o}$; another possibilit is the characterization of the factors in table 5.1.0b. This s be taken into consideration during the revision of all other e tions of chapter 5.

10 Clause 5.1.1.4: Equation 5.1.1.4 is too favourable for notc at the lower side of the beam (see equation 5.2.1b).

11 Clause 5.1.1.7: Especially the equations 5.1.1.7a and d see to be very complicated and not practical. Furthermore they see to simulate great accuracy. It would be better to give a mor simple equation.

12 Table 5.1.1.7: No ultimate value (e.g. 2.0) or ultimate va zone (e.g. 2.1 to 2.5) should be given. A value, e.g. $\geq$ 2.0 with a footnote, saying that the calculation shall be made acc

to the respective constructive data, would be better.

13 Clause 5.2.3: For European softwood, the thickness of the lamellas instead for usual cases is to be reduced for the relation $r/t < 200$, however the value should never be below $r/t < 150$, in order to avoid failure due to bending.

As intended for the future edition of the German standard, the thickness of the lamellas should not be greater than 33mm.

14 General note: It is intended to differentiate the single factors in a better way; e.g. k for the factors reducing the allowable stresses, and another symbol for the factors reducing the effective stresses.

15 Chapter 6 (reference is made to the second code draft) Table 6.0. There should be a footnote, indicating that the values mentioned are only valid in the case wood failure for wood joints using fasteners; the same applies for nail plates.
Furthermore other cases of failure should be mentioned especially.

In addition it has to be verified, whether the values are only valid for ultimate load, or whether the values are also valid for deformation (slip modulus) according to those stated in table 5.1.01

16 Clause 6.1.1.1: For nail joints, informations should be taken from the corresponding German standards DIN 1052 part 1, edition October 1969, and from the draft DIN 1052 part 3, edition February 1979. In DIN 1052 part 3 the following is mentioned for wood-based sheet materials:

"For lateral loading (shearing), the admissible load for load case I (see DIN 1052 part 1, edition October 1969) can be calculated , also in the case of nail joints with one shear plane only, of particle or fibre boards with solid timber by the equation

$$F_1 = \frac{500 \cdot d^2}{10 + d_n} \quad \text{in N, } (d_n = \text{nail diameter in mm})$$

provided that the thickness of the particle boards and the medium hard fibre boards is at least $4{,}5 \cdot d_n$. For the hard fibre boards the thickness has to be $2 \cdot d$. In case of thicknesses for particle boards and medium fibre boards between $3 \cdot d_n$ and $4{,}5 \cdot d_n$, the admissible load has to be reduced to $d/(4{,}5 \cdot d_n)$.

17 Clause 6.1.1.2: Based on the test results received in Germany the penetration depth of the nails should not be deducted by l_p in the equations 6.1.1.2a - c, if the minimum penetration depth is at least $4 \cdot d_n$.

The German standard considers the smooth nail unfavourable. It is only admitted for short term withdrawal loads by wind. Furthermore a definition for spirally grooved nails and annular grooved nails should be given. In our opinion the angle between the thread and the nail axis should not exceed 60° for these nail types.

18 Clause 6.1.2: Due to simplified application in practice, it is intended to develop only two equations showing safe results, instead of the five ones (6.1.2a - e) mentioned. The form in which the formulas are given seem to be too complicated and the expansion is too large.

The same applies to the comments of table 6.1.1.2 and to the comments in chapter 6.1.3.1.

19 Clause 6.1.3.2: The maximum penetration depth which should be allowed to take into account should be mentioned in addition to equation 6.1.3.2a.

Equation 6.1.3.2b should read (30 + 12) ... instead of (30 + 12,5). In both equations the diminution of the length of the point is not necessary, if the minimum penetration depth is at least 4d.

20 Clasue 6.1.4: The equation indicated, should be reconsidered. Apart from that, the evolution of the equation can not be recognized; some background informations seem to be necessary.

21 Clause 7: Simplified equations should be developped, see e.g. draft of DIN 1052 part 3. Furthermore reference is made to a publication of Prof. Dr.-Ing. Möhler, which will appear in "Holz als Roh- und Werkstoff" in June 1979.

22 Table 7.2: The values given for the "slip modulus" seem to be too high; they should be examined once again. Perhaps there are some mistakes in writing. For the design of ultimate load the slip modulus must be determined by ultimate load tests.

23 Clause 8.3: It is proposed that the diameter of the bolt hole should be 1mm larger (instead of 2mm) than the diameter of the bolt, see also DIN 1052 part 1.

German Comment on Document ISO/TC 165 N 38 E "Timber structures; joints; determination of strength and deformation characteristics of mechanical fasteners"

After revision of this document by our experts, we ask you for the following supplements and explanations.
Chapter 8.5 "Calculations", item 6 "Elastic slip": The value "2/3" is not visible from the arrangement of the values; we kindly ask you to explain this. The same applies to the equation for "elastic displacement" on the same page.

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CIB-W18/12-16-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

British Standard BS 5268: The Structural Use of Timber:
Part 4: Fire Resistance of Timber Structures.

BORDEAUX, FRANCE

OCTOBER 1979

BSI

BS 5268 : Part 4 : Section 4.1 : 1978
UDC 691.11 : 674.049.3 : 620.1.08

Code of practice for

The structural use of timber

Part 4. Fire resistance of timber structures

Section 4.1. Method of calculating fire resistance of timber members

Code de bonne pratique relatif à l'utilisation de bois de charpente
Partie 4. Résistance au feu des ouvrages en bois
Section 4.1. Méthode de calcul de la résistance au feu des pièces en bois

Richtlinien für Holz für Bauzwecke
Teil 4. Feuerbeständigkeit von Holzbauten
Abschnitt 4.1. Verfahren zur Berechnung der Feuerbeständigkeit von Holzelementen

British Standards Institution

Gr 3

Contents

This code of practice represents a standard of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from statutory and legal requirements.

Foreword

The code of practice CP 112 'The structural use of timber' has been allocated the new number, BS 5268, as part of the integration of work on codes and standards. The Parts of CP 112 that have been published are as follows.

Part 1 : 1967 Imperial units
Part 2 : 1971 Metric units
Part 3 : 1973 Trussed rafters for roofs of dwellings

Part 1 was withdrawn in 1974. Part 2 remains valid and will continue for a further limited period so that the permissible stress method of design can still be used. Part 3 is being revised. It is intended that the new BS 5268 'The structural use of timber' will have the following Parts.

Part 1 Limit state design
Part 2 Permissible stress design
Part 3 Trussed rafters for roofs
Part 4 Fire resistance of timber structures
  Section 4.1 Method of calculating fire resistance of timber members
  Section 4.2 Method of calculating fire resistance of timber stud walls and joisted floor constructions
Part 5 Preservation treatment for constructional timber (published 1977)
Part 6 Timber frame wall design

Part 2 of BS 5268 is in course of preparation and is a revision of CP 112 : Part 2 which it will supersede on publication. The recommendations of the forthcoming Part 1 of BS 5268 will entirely supersede those of Part 2 of BS 5268 after a limited number of years. Part 3 of BS 5268 is in course of preparation and is a revision of CP 112 : Part 3 which it will supersede on publication. Part 4 is a new Part consisting of two Sections (described below). Part 5 was published in 1977. Part 6 is a new Part in course of preparation.

Section 4.1 of BS 5268 : Part 4 gives information for the calculation of fire resistance. Such calculations are possible because, in fire, the behaviour of timber is predictable with regard to the rate of charring and loss of strength. It is also free from rapid changes of state and has very low coefficients of thermal expansion and thermal conductivity. Timber treatments including impregnation to retard the surface spread of flame should not be assumed to affect the charring rate. Section 4.2 will deal with timber stud walls and joisted floor constructions.

Fire resistance relates to complete elements of construction and not to individual materials; the appropriate test is described in BS 476 : Part 8. The stability (resistance to structural failure), integrity and insulation criteria may all be applicable and the performance of an element is expressed in terms of the periods of time that the appropriate criteria are satisfied.

The methods given in this code for assessing by calculation the fire resistance of timber members, in relation to stability criteria, use stress modification factors (see **5.1.2** and **5.2.2**) which have been arrived at empirically and checked against the results of a number of fire resistance tests conducted in accordance with the appropriate British Standard.

It is current practice during the fire resistance test in accordance with BS 476 : Part 8, in the case of compression members, to apply an axial load only, because the limitations of the existing test equipment preclude other loading arrangements. The information given in this Part of the code, dealing with the assessment of fire resistance of compression members, relates to fire resistance tests, where limited loading arrangements have to be used.

British Standard Code of practice for

The structural use of timber

Part 4. Fire resistance of timber structures

Section 4.1 Method of calculating fire resistance of timber members

1. Scope

This code of practice gives methods of assessing the fire resistance of flexural tension and compression members of solid or glued laminated timber and their joints.

2. References

The titles of the standards publications referred to in this code are listed on page 5.

3. Definitions.

3.1 For the purposes of this code the definitions given in BS 565, BS 4422 : Part 2 and in CP 112 : Part 2 apply, together with the following additional definitions.

residual section. The section of the uncharred timber that would be left after a given period of exposure to the fire conditions described in the test in BS 476 : Part 8, assuming a steady rate of charring, with allowance for accelerated charring at exposed arrises where recommended herein (see **4.3**).

3.2 In addition, the definition of **stability** used throughout Part 4 is based on that given in BS 476 : Part 8, which for the purpose of this code is taken as:

(a) the ability to sustain the applied load throughout the period of the fire test and,

(b) also, in the case of flexural members, ability to resist deflection during the fire test, to $\frac{\text{span}}{30}$

NOTE. It should be noted that this may differ from the normal structural engineering interpretation.

4. Behaviour of timber in fire

4.1 General. For the purposes of this code, charring can be assumed to occur at a steady rate in the fire resistance test described in BS 476 : Part 8. The timber beneath the charred layer does not lose significant strength because the thermal conductivity is low.

These characteristics make it possible to predict the performance in a fire resistance test of certain flexural tension and compression members thus reducing the need for testing.

The criteria of BS 476 : Part 8 are applicable to elements of building construction as follows.

(a) Flexural members (beams) : stability, (strength and deflection).

(b) Compression members (columns) : stability.

(c) Tension members : stability.

Where members are built into, or form part of, a fire resisting construction, the insulation and integrity requirements may also be applicable.

4.2 Resistance to charring

4.2.1 *Solid members.* Calculation of residual section of solid members should be based on the values given in table 1. These values should be modified in the case of fully exposed columns and tension members as set out in **5.2.2**(a) and **5.3.2**(a) respectively.

Table 1. Notional rate of charring for the calculation of residual section

Species	Charring in 30 min	Charring in 60 min
	mm	mm
(a) All structural species listed in table 1 of CP 112 : Part 2 : 1971, except those noted in items (b) and (c)	20	40
(b) Western red cedar	25	50
(c) Oak, utile, keruing (gurjun), teak, greenheart, jarrah	15	30

NOTE. Linear interpolation or extrapolation for periods between 15 min and 90 min is permissible.

Notional charring rates for particular species and longer periods of time not presently included in table 1 may be established by an appropriate authority.

4.2.2 *Glued laminated members.* The charring rates given in **4.2.1** may be applied to members laminated with the following thermosetting phenolic and aminoplastic synthetic resin adhesives: resorcinol-formaldehyde, phenol-formaldehyde, phenol-resorcinol-formaldehyde, urea-formaldehyde, and urea- melamine-formaldehyde. Where other adhesives are used, guidance should be sought from an appropriate authority.

4.2.3 *Finger joints.* Finger joints manufactured in accordance with the requirements of BS 5291 using adhesives specified in **4.2.2** may be considered to char at the rates given in table 1.

4.2.4 *Sections built up with metal fasteners* (see figure 2). The charring rates in **4.2.1** may only be applied to the section as a whole if metal fasteners on which the structural performance of the built-up member depends are fully protected from the effects of fire (see **5.4.2**). Where such protection is not given, local structural weaknesses may occur and the member can only be assessed for fire resistance by applying the residual section calculation, assuming charring on all faces of each component of the built-up member, or by conducting a fire resistance test.

4.3 Increased rate of charring on exposed arrises. Arrises will become progressively rounded during fire exposure. The radius of this rounding is equal to the depth of charring and the centre lies equidistant from the two aspect faces at a distance of twice the charring depth (see figure 1).

For periods of fire exposure not exceeding 30 min, where the least dimension of the rectangular residual section is not less than 50 mm, rounding is insignificant and may be disregarded.

5. Design considerations

5.1 Flexural members

5.1.1 *Stability criteria*

'(a) *Strength*. The residual section should be such that the member will support the appropriate loads that would be applied if the component were tested in accordance with the requirements of BS 476 : Part 8 to either the maximum permissible design load or the loads based on those which the member is required to support in normal service.'

(b) *Deflection*. The deflection under the appropriate design load should not exceed 1/30 of the clear span. Consideration should be given to the effect of deflection on the stability and integrity of other parts of the structure.

5.1.2 *Assessment of fire resistance*

(a) *Residual section*. The residual section should be computed by subtracting from the appropriate faces the notional amount of charring assumed to occur during the required period of fire exposure, making allowance for the rounding on the exposed arrises, where necessary.

(b) *Strength*. The load-bearing capacity of a flexural member should be calculated in accordance with normal practice, using the residual section and stresses of 2.25 × permissible long-term dry stresses given in CP 112 : Part 2, when the minimum initial breadth of the section is 70 mm or greater and 2.00 × permissible long term dry stress, when this dimension is less than 70 mm.

(c) *Deflection*. Deflections should be calculated using the residual section and the dry value of the modulus of elasticity taking the mean or minimum values, as used in the original design. The resulting deflection should not exceed the limit defined in **5.1.1**.

5.2 Compression members

5.2.1 *Stability criterion*. The residual section should be such that the member will support the appropriate loads such as would be applied if the component were tested in accordance with the requirements of BS 476 : Part 8, to either the maximum design compressive load or loads based on those which the member is required to support in normal service.

5.2.2 *Assessment of fire resistance*

(a) A column that is exposed to the fire on all faces (including a column which abuts on or forms part of a wall that does not have fire resistance, as in figures 3(b) and 4(b)), should be assumed to char equally on all faces during the whole period of fire exposures. To determine the residual section of such columns, the rates of charring given in table 1 should be multiplied by 1.25.

Where a column abuts on or forms part of a wall which provides fire resistance from either side not less than that of the column, charring on all faces is unlikely. Calculations should therefore be based on charring of the column occurring on the side of the wall on which 'the column has the greater surface exposure, using the' rates of charring given in table 1 (see figures 3(a) and 4(a)).

Care should be taken to ensure that the junctions between the wall and the column will be adequate as a barrier to fire so that the integrity of the construction is unimpaired.

Where a column abuts on or forms part of a wall, which is required to provide fire resistance from one side only (such as in an external wall) and which has fire resistance not less than the column, charring on the faces of the column which can be exposed to fire need only be considered and the rates of charring given in table 1 should be used. In establishing the vulnerable column faces, due regard should be given to the protection afforded by the walling materials.

Care should be taken to ensure that the junctions between the wall and the column will be adequate as a barrier to fire so that the integrity of the construction is unimpaired.

(b) No restraint in direction at the ends (as distinct from positional restraint) should be assumed in determining the effective length of residual column sections unless consideration of the residual joint (as indicated in **5.4**) shows that a degree of restraint would be provided.

(c) The maximum slenderness ratio based on the residual section should not exceed 250 (this limitation replaces those given in CP 112 : Part 2) and the stress modification factor for long-term loading for the slenderness ratio of the residual column should be derived from table 15 of CP 112 : Part 2 : 1971.

(d) The strength of a compression member should be calculated using the appropriate residual section, in accordance with CP 112 : Part 2 as modified by **5.2.2**(b) and **5.2.2**(c) with the compressive stress parallel to the grain of 2.00 × the permissible long-term dry stress.

(e) The strength of compression members subject to bending should be calculated in accordance with **3.14.3** of CP 112 : Part 2 using the stresses derived in **5.1.2**(b) and **5.2.2**(d) in place of the permissible stresses.

5.3 Tension members

5.3.1 *Stability criteria*. The residual section should be such that the member will support the appropriate loads.

5.3.2 *Assessment of fire resistance*

(a) To determine the residual section of a tension member the rates of charring given in table 1 should be multiplied by 1.25.

(b) The load-bearing capacity of a tension member should be calculated in accordance with normal practice using the residual section and a stress of 2.00 × permissible long term dry stress given in CP 112 : Part 2.

(c) The load-bearing capacity of a tension member subject to bending should be calculated in accordance with **3.15.2** of Part 2 of this code using the permissible stresses derived in **5.1.2**(b) and **5.3.2**(b).

5.4 Joints

5.4.1 *General*. The charring rates given in table 1 may be applied provided that in all cases the faces of the abutting pieces of timber are held in close contact and that special attention is paid to the placement or protection of metal fasteners and components (see **5.4.2** and **5.4.3**).

The methods of calculation given previously are directly applicable to the performance of individual flexural, tension and/or compression members. Junctions between members may be particularly vulnerable to the effects of fire and require special consideration. Where a compressive force is transferred by direct timber-to-timber bearing, the loss in strength of the joint is unlikely to be significant where members have been designed in accordance with the recommendations of this code.

However, where a structure is designed to have joints that transfer forces from one member to another, special account should be taken of the behaviour of such joints. An assessment should be made of the residual timber after the specified period, with particular attention to the effects of any metal connectors and the probability of rounding at abutting arrises (as indicated in **4.3**). In redundant structures, charring may alter the relative stiffness of various parts of the structure and result in a redistribution of forces, and account should be taken of complete or partial yielding of the joints as this may change the structural action. The structure with redistributed forces should be assessed for fire resistance as detailed in **5.1**, **5.2** and **5.3**.

5.4.2 *Metal fasteners.* Where any part of a nail, screw or bolt becomes exposed to heating during a fire, rapid heat conduction will lead to localized charring and loss of anchorage. Where this effect is likely to lead to the failure of a structural member which is required to have fire resistance, protection of the fastener should be provided by any one of the following methods.

(a) Ensuring that every part of the fastener is embedded in the timber so that it remains within the residual section as shown in figure 2. Any holes should be fully and securely plugged with timber glued in position. Advice on the use of alternative plugging materials should be sought from an appropriate authority.

(b) Covering the exposed part of the fastener with a suitable protecting material, e.g. timber, plasterboard, asbestos insulation board, or equivalent. Special attention should be paid to the fixing of such protection to ensure that it remains in position for the required period of fire resistance. Unprotected nails, screws or staples may be used in this case to fix this insulation.

(c) Any appropriate combination of the methods outlined in (a) and (b).

5.4.3 *Steel hangers for joists or beams.* Where steel hangers are fully protected for the required period of fire resistance either by a ceiling membrance or locally with a protecting material, they will be satisfactory in fire.

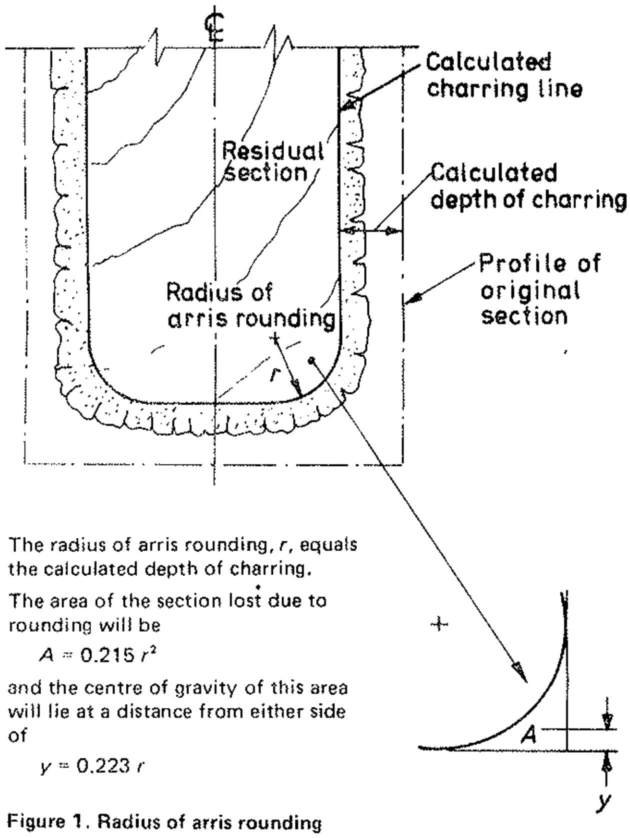
For floor construction up to and including 30 min fire resistance, joist hangers of the strap or shoe type, formed from 1 mm steel, may be used with ceiling construction which affords 20 min protection, e.g. 12 mm plasterboard.

For floor construction up to and including 30 min fire resistance, joist hangers of the substantial shoe type with gusset or strap bracing, formed from at least 3 mm steel, may be used without protection.

For 1 h fire resisting floors, a ceiling has to be used affording at least 45 min protection, e.g. 31 mm plasterboard.

5.4.4 *Metal plates and other metal connectors.* Metal connectors and metal connector plates may be used without restriction in trussed rafter construction when no fire resistance requirements exist. When a member incorporating exposed nail plates is required to have fire resistance, the provisions of **5.4.2** apply.

When the bolts of other types of metal connectors, e.g. toothed plates, split rings, etc., are likely to become exposed during a fire, additional protection as outlined in **5.4.2** should be provided. All other types of joints should be referred to an appropriate authority.



The radius of arris rounding, r, equals the calculated depth of charring.

The area of the section lost due to rounding will be

$A = 0.215\,r^2$

and the centre of gravity of this area will lie at a distance from either side of

$y = 0.223\,r$

Figure 1. Radius of arris rounding

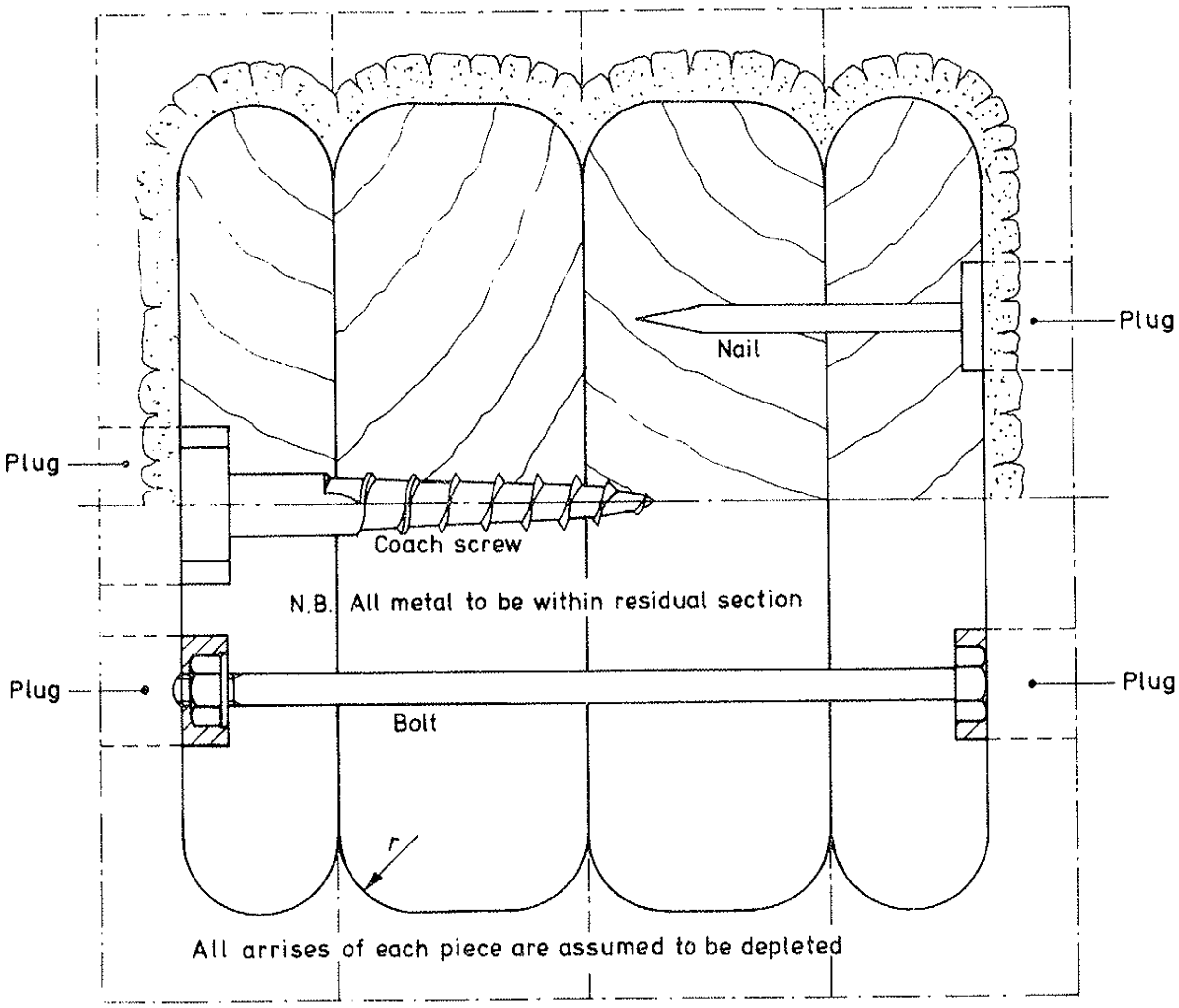
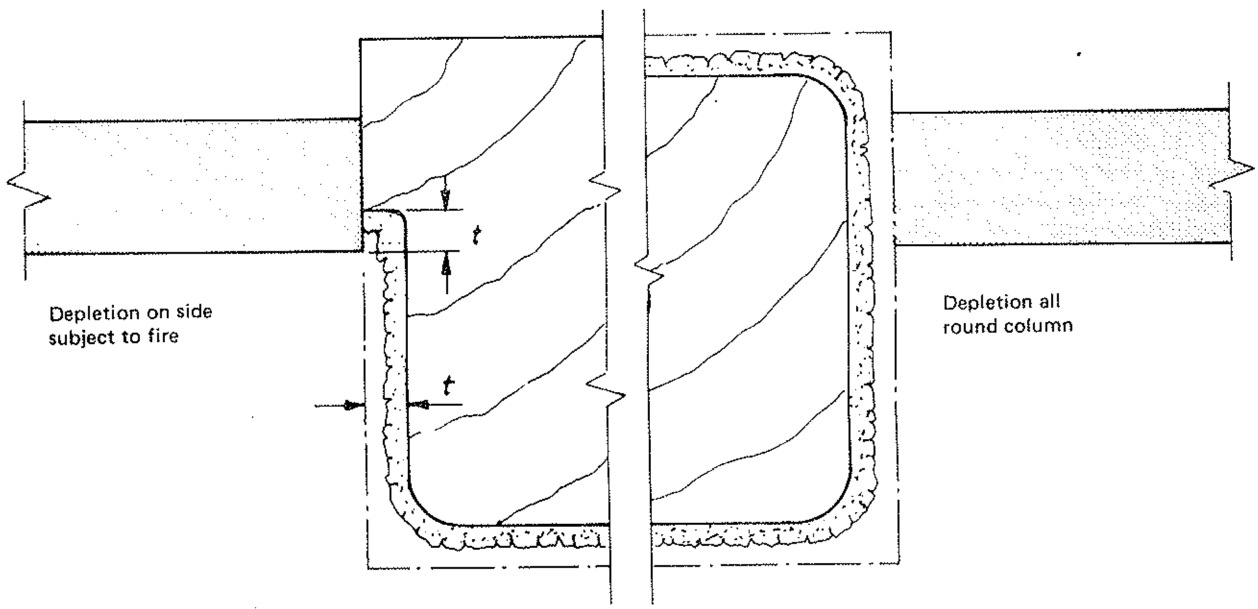


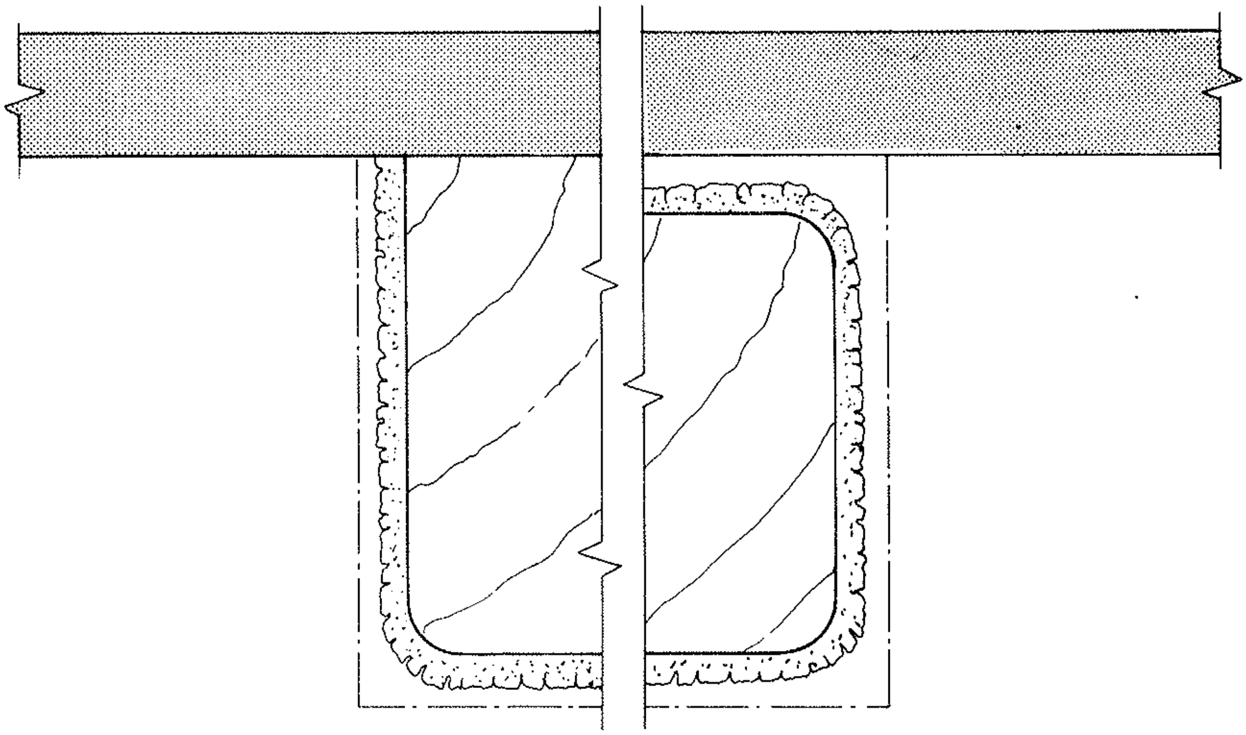
Figure 2. Sections built up with metal fasteners



(a) Wall having fire resistance not less than the column

(b) Wall having less fire resistance than the column

Figure 3. Columns built into walls



(a) Wall having fire resistance not less than the column

(b) Wall having less fire resistance than the column

Figure 4. Columns abutting on walls

Standards publications referred to

BS 476 Fire tests on building materials and structures
Part 8 Test methods and criteria for the fire resistance of elements of building construction

BS 565 Glossary of terms relating to timber and woodwork

BS 4422 Glossary of terms associated with fire
Part 2 Building materials and structures

BS 5291 Finger joints in structural softwood

CP 112 The structural use of timber
Part 2 Metric units

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Contract requirements
Attention is drawn to the fact that this British Standard does not purport to include all the necessary provisions of a contract.

Revision of British Standards
British Standards are revised, when necessary, by the issue either of amendment slips or of revised editions. **It is important that users of British Standards should ascertain that they are in possession of the latest amendments or editions.**

The following BSI references relate to the work on this standard:
Committee reference CSB/32 Draft for comment 75/10262

Amendments issued since publication

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British Standards Institution · 2 Park Street London W1A 2BS · Telephone 01-629 9000 · Telex 266933

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