INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB - W18

MEETING EIGHTEEN

BEIT OREN

ISRAEL

JUNE 1985

CONTENTS

1. LIST OF PARTICIPANTS

BELGIUM

M G R	Verhorst	American Plywood Association, Berchem

CANADA

B	Madsen	University of British Columbia, Vancouver
C	Stieda	COFI of British Columbia, N. Vancouver

DENMARK

M	Johansen	Danish Building Research Institute, Horsholm
H J	Larsen	Danish Building Research Institute, Horsholm
H	Riberholt	Technical University of Denmark, Lyngby

FEDERAL REPUBLIC OF GERMANY

H	Brüninghoff	Bergische Universität - Gesamthochschule Wuppertal
J	Ehlbeck	Universität Karlsruhe, Karlsruhe
P	Glos	Universität München, München

FINLAND

J	Kangas	Technical Research Centre of Finland, Espoo
T	Poutanen	Tampere

IRELAND

P R	Colclough	Institut für Ind. Res. + Stand., Dublin

ISRAEL

U	Korin	Building Research Station, Haifa

ITALY

A	Ceccotti	Università Degli Studi Di Firenze, Firenze
A	Vignoli	Università Degli Studi Di Firenze, Firenze

NETHERLANDS

J	Kuipers	Stevin Laboratory, Delft

NORWAY

E	Aasheim	Norsk Treteknisk Institutt, Oslo
P H	Leirtun	Norges Byggstandardiseringsrad, Oslo
T O	Ramstad	Norwegian Building Research Inst., Oslo

SWEDEN

J	Brundin	Swedish Inst. f. Wood Techn. Res., Stockholm
B	Edlund	Chalmers University of Technology, Göteborg
B	Källsner	Swedish Inst. f. Wood Techn. Res., Stockholm
B	Norên	Swedish Inst. f. Wood Techn. Res., Stockholm
B	Thunell	Royal Inst. of Techn., Wood Techn. and Processing, Stockholm

SWITZERLAND

U A	Meierhofer	EMPA, Dübendorf

UNITED KINGDOM

H J	Burgess	TRADA, High Wycombe
R F	Marsh	Ove Arup and Partners, Londen
I	Smith	TRADA, High Wycombe
J G	Sunley	TRADA, High Wycombe
J	Tory	Princes Risborough Lab., Aylesbury

UNITED STATES OF AMERICA

D H	Brown	American Plywood Association, Tacoma
E G	Stern	Virginia Polytechnic Inst. + State University, Blacksburg

2. CHAIRMAN'S INTRODUCTION

MR. SUNLEY said the CIB had accepted his proposal to hand over to DR. STIEDA as the new coordinator of W18. He expressed his thanks to those who had assisted over the last 12 years, in particular the different secretaries, the host countries and those who had attended meetings, especially individuals who had made notable contributions.

He felt very real results had been achieved in the CIB Code, the liaison between people of different countries and the collection of published papers. These would form an impressive background for the international harmonisation which he felt must happen, and he emphasised the reputation built up by CIB-W18 and its potential in future developments.

DR. STIEDA then took over as chairman of the meeting.

3. COOPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: MR. LARSEN reported on the Corsica meeting of May 1984, saying certain test standards had now become draft proposals and been sent out for comment. New work being taken up included performance requirements for glulam and safety in relation to biodeterioration, and the ECE recommendations on grading and finger joints were to be appraised for presentation as international standards. The next meeting would take place at TRADA in the third week of March 1986.

RILEM: PROFESSOR KUIPERS said the nail joints test method had been agreed for printing as a final recommendation and the methods for staples, wood-based boards and structures as tentative recommendations. The work of 57 TSB was now concluded.

There were suggestions for a new committee to consider in-grade test methods and also a probabilistic approach to potential bio-degradation in structural timber. A discussion of the need for a standard on in-grade testing in addition to the existing ISO standard concluded that a paper was needed for the next meeting to assist in resolving the question.

Several of those present agreed with a suggestion by MR. LARSEN that a Code section on earthquake design was desirable, but added that research was needed before test methods could be established to yield data that might be required and that a paper on the topic should be invited for the next meeting.

MR. SUNLEY said it was unclear what would be done in the durability study. PROFESSOR KUIPERS said three or four people could consider what was needed; if the proposal did not develop in a promising way, the topic could be dropped. It was decided that an outline should be provided for the next meeting to assess whether W18 could make a contribution.

CEI-Bois/FEMIB: MR. RIBERHOLT introduced his paper CIB-W18/18-12-1 'Report on European glulam control and production standard', saying the object of the European control bodies who had attended two meetings had been to establish common rules for control of production and approval of the product. Draft proposals were expected about the end of 1985.

Answering MR. TORY, he said their relationship with ISO standards would be considered very shortly and that quality control would be included.

IUFRO S5.02: Reporting on the Mexico meeting of December 1984, PROFESSOR MADSEN said only about 19 people attended but the meeting was successful. The Proceedings would be available shortly. He drew attention to the Pacific Timber Engineering Conference held at Auckland University in May 1984. The Proceedings of the Conference may be obtained from the Institution of Professional Engineers New Zealand, Box 12241, Wellington (NZ $ 62.50 including postage). A Timber Engineering Seminar in South Africa was also mentioned; this was held at the Council for Scientific and Industrial Research, Box 395, Pretoria, in April 1985.

IABSE: PROFESSOR EDLUND reported on the Congress held in Vancouver in September 1984. He felt the small attendance of timber specialists might reduce the emphasis on timber at the next Congress in Helsinki in 1988. Four papers had been presented by PROF. GUTKOWSKI, MR. GEHRI and DR. GIRHAMMAR, a couple of posters on timber bridges were presented (deck systems and inspection respectively), and among others a TRADA paper on mechanical fasteners had appeared.

EEC EUROCODES: MR. LARSEN listed the structural Eurocodes in preparation, saying he was in a coordination group formed by representatives of the different drafting panels. Model chapters were being prepared for all materials but there would be some variations for the different materials. A common chapter not subject to changes was also being produced.

MR. SUNLEY said five individuals were employed as consultants for Eurocode 5 (Brüninghoff, Ehlbeck, Crubilé, Larsen, Sunley). They would produce a document by October 1985 for circulation to the ten member states. There would be a year for comments, and these would be considered by a new committee which would produce a final document for adoption, perhaps alongside national codes.

Answering DR. STIEDA, MR. LARSEN said that common reliability was not being sought for all materials but that calibration would lead to member sizes similar to those now being used.

It was agreed that MR. SUNLEY would continue to represent CIB-W18 in Eurocode meetings. DR. STIEDA said the American Society of Civil Engineers was producing information on limit state design, and those interested could contact Dr. Joe Murphy. Referring to the main CIB organisation, MR. LARSEN gave information on the joint committee on structural safety which had developed from a CIB working commission, and described the CIB Congress to be held in Washington DC, 21-26 September 1986. MR. SUNLEY said a Congress paper on the general activities of W18 had been requested.

LOAD DURATION GROUP: Under this Agenda item added by the chairman, PROFESSOR GLOS reported the activities of a number of cooperating institutions in France, Holland, Denmark, Germany, Sweden and the United Kingdom under the EEC 'wood as a renewable material' project. Final reports would be produced for the end of 1985 and an additional programme was to be planned.

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

MR. SUNLEY said this sub-group had been established at the request of MR. BECKETT and its future was not clear. DR. STIEDA added that W18 was prepared to help with advice and PROFESSOR EHLBECK suggested writing to the institutes involved to offer them the W18 Proceedings.

5. TRUSSED RAFTER SUB-GROUP

MR. LARSEN said an Annex on the design of "W" trussed rafters had been produced for the Eurocode and ISO, and papers extending to other configurations would be produced for the next meeting.

6. SAMPLING SUB-GROUP

PROFESSOR GLOS said a paper to be presented would form a first working draft for Annex 41. He thought the work should extend to data analysis, which would require an additional Annex. Considerable work had been done already by DR. NOREN and others. The present studies dealt with the sampling of structural materials, and the earlier work should be added for the Annex.

7. TIMBER FRAME HOUSING SUB-GROUP

MR. SUNLEY said terms of reference had been drawn up (last page of Karlsruhe Proceedings) but there had been difficulty in finding a chairman. MR. TORY suggested that a UK wall design standard presently being prepared might be of interest elsewhere. MR. SUNLEY said recent delegations from France and Holland had been keenly interested in UK practice and MR. VERHORST said he had joined the group from Holland, which arose from government pressure to draw up adequate specifications.

PROFESSOR EHLBECK said the group could cater also for seismic design. MR. RAMSTAD asked if special design rules were needed for timber frame houses and MR. SUNLEY replied that additional information was needed on racking resistance and the interaction between adjacent panels. DR. NOREN thought that if something was missing from the Code then it should be added.

It was concluded that by next year's meeting opinions should be developed on whether the efforts to establish a sub-group should be continued or not.

8. TROPICAL TIMBERS

MR. SUNLEY said the proposed sub-group arose from correspondence with CIB headquarters in which he had suggested that it was appropriate for W18 to deal with the structural use of tropical timbers. PROFESSOR STERN suggested inviting Dr. de Freitas to be chairman of the sub-group.

The chairman concluded that the CIB could be advised that a framework had been created for such a group and that W18 would give all possible assistance.

9. TRUSSED RAFTERS

MR. AASHEIM presented the paper by himself and N.I. Bovim, CIB-W18/18-7-6 'The strength of nail plates', saying it was based on work in Norway several years ago which had not until now been reported. DR. NOREN said the work was related to other CIB papers and formed part of the background to Nordic group discussions on trussed rafter joint design. He thought he could prepare a summary from the existing summary in Swedish as a possible Annex. MR. POUTANEN supported the method presented as safe and simple.

Introducing his paper 18-14-1 'Simplified calculation method for 'W' trusses (Part 2)' MR. KALLSNER said it was a less complex version of a paper presented at the Rapperswil meeting. In answer to questions by PROFESSOR STERN, the form of construction chosen for the eaves joint was explained.

Paper 18-14-2 'Model for trussed rafter design' was introduced by the author MR. POUTANEN. He said the paper showed that eccentric connections have a large effect on moments transferred; by adjusting eccentricities, beneficial effects could be achieved to lead to economical structures.

At a later stage of the meeting further papers relating to trussed rafters were presented by MR. KANGAS, paper 18-7-4 'A detailed testing method for nail plate joints' and paper 18-7-5 'Principles for design values of nail plates in Finland'. The author described a modified method for the plate shear test and PROFESSOR KUIPERS suggested that a proposal could be made for amendment of Annex 3TT-1A based on the research performed.

After discussion of the Code needs, the hope was expressed that the Scandinavian group would produce an Annex based on the Eurocode draft and another giving a simple guide to connection design for the next meeting.

10. STRUCTURAL STABILITY

MR. CECCOTTI presented paper 18-15-1 'Full-scale structures in glued laminated timber, dynamic tests: theoretical and experimental studies', by himself and MR. VIGNOLI. Answering MR. MEIERHOFER, he said the work had been done to satisfy the building control authorities. The effect of dead load was only minor, and as timber structures were not well known in Italy, the authorities had required this type of test. In response to a question by MR. MARSH he said the test was more economical than a static load test. Special instruments were needed but the loading was small.

11. SIZE EFFECTS

Paper 18-6-4 'Size effects in timber explained by a modified weakest link theory' by PROFESSOR MADSEN was presented by the author, who said the work indicated that strength was related to the volume of material that is highly stressed. He found the depth effect was not significant in bending but existed in tension. The breadth effect seemed to be an inverse relation, but this was complicated by the effect of grading rules. MR. RIBERHOLT said the effect found was not necessarily a depth effect but might arise from random positioning of defects, also that types of extreme distribution other than Weibull could be tried. The author agreed that further investigations and contributions by others would be valuable.

12. CODES AND STANDARDS

Presenting paper 18-1-1 'Notes on the development of a UK limit states design code for timber' by A.R. Fewell and C.B. Pierce, MR. TORY said the first part seemed the more important as a strictly probabilistic method was not practical and calibration must be applied. He questioned the need for a consequence factor, which however was supported at the meeting and MR. LARSEN said provision was made for such a factor in Eurocode No.1.

MR. LARSEN introduced his paper 18-1-2 'Eurocode 5, timber structures' saying the dividing factor of 1.4 perhaps led to an unrealistic factor for long term load; it was now proposed to change to 1.3 for normal structures. There was an extensive discussion of the proposals, including questions on the factors for load duration and creep. The author proceeded to deal with shear design, responding to a question by PROFESSOR GLOS that the more complex formula would not be applicable for solid beams of normal size. MR. SUNLEY explained the application of strength class design values in the case of glulam beams, saying the actual figures were based on those applied in the United Kingdom and could be changed if necessary after comment from other countries.

Paper 18-6-6 'Partial safety coefficients for the load-carrying capacity of timber structures' by B. Noren and J. O. Nylander was presented by DR. NOREN. Answering DR. STIEDA he said the proposed method was being considered for the future Swedish code which was now circulating for comment. PROFESSOR EHLBECK asked if the materials factor used was similar to the Eurocode 5 value, and MR. LARSEN replied that it differed because a factor of 1.35 was applied to dead load in the Eurocode loading.

PROFESSOR MADSEN presented paper 18-2-1 'Column design methods for timber engineering' by A.H. Buchanan, K.C. Johns and himself. He concluded that the proposals should be examined and considered for incorporation in the CIB Code. He felt it was important for a model code to be based on the actual performance of the material. MR. LARSEN said an underlying theory including deflection would be needed to form a basis for a Code proposal. After discussion the Chairman concluded that the meeting was not in a position to say that a new formulation was needed, and that any new proposal would be for study by experts in the field.

Paper 18-102-1 'Antiseismic rules for timber structures: an Italian proposal' by G. Augusti and A. Ceccotti was presented by the latter. Answering DR. SMITH who pointed out that a lot of related work had been done by New Zealand workers, MR. CECCOTTI said the New Zealand proposals were very detailed; in Italy the rules were needed only for certain types of timber structures. DR. STIEDA said there was a real need for the type of work displayed, to contribute towards filling a gap in the existing Code.

13. JOINTS

DR. STERN presented paper 18-7-1 'Model specification for driven fasteners for assembly of pallets and related structures' by himself and W.B. Wallin. Answering DR. KORIN he said a coating was required only for staples which have plain shanks; the coating must improve the withdrawal strength by thirty per cent.

Two papers were introduced by DR. SMITH: paper 18-7-2 'The influence of the orientation of mechanical joints on their mechanical properties' by I. Smith and L.R.J. Whale and paper 18-7-3 'Influence of number of rows of fasteners or connectors upon the ultimate capacity of axially loaded timber joints' by I. Smith and G. Steck. He said papers on these topics had been requested at the previous meeting and concluded the presentation by pointing out that the paper on multiple fasteners showed the need for further research of various kinds. MR. LARSEN said the recommendations on this topic would be borne in mind for the future revision of the CIB Code and for Eurocode 5.

PROFESSOR KUIPERS introduced his paper 18-9-1 'Prediction of creep deformations of joints' and answered a number of questions about the type of fastener, the environment of the tests and the influence of moisture content variations.

14. BRACING

The Chairman announced that DR. BRUNINGHOFF had prepared paper 18-15-2 as an Annex on bracing as requested at the last meeting, and this would be included in the Proceedings for comment.

15. TIMBER STRESSES AND GROUPING

The Chairman referred to three papers by R.H. Leicester: paper 18-6-1 giving a comment on the two following, paper 18-6-2' Configuration factors for the bending strength of timber' and paper 18-6-3 'Notes on sampling factors for characteristic values'. PROFESSOR GLOS said the last of these dealt with the derivation of characteristic values from small samples and could be better titled. He felt the results would be valuable for use in quality control and data analysis.

MR. RIBERHOLT presented his paper 18-6-5 'Placement and selection of growth defects in test specimens'. He said the paper gave rise to questions and he would like the opinions of others on these. After discussion and following a suggestion by MR. LARSEN, there was a measure of agreement that for the time being tests should be made with the worst defect in the zone of maximum stress but the characteristic strength should take the higher value corresponding to random placement.

16. SAMPLING

Paper 18-17-1 'Sampling of timber in structural sizes' by P. Glos was introduced by the author, who said it provided a first working draft for Annex 41. It was agreed to put the paper on the Agenda for thorough discussion at the next meeting.

17. BEAM DESIGN

Note was taken of paper 18-10-1 'Submission to the CIB-W18 committee on the design of ply-web beams by consideration of the type of stress on the flanges' by J.A. Baird. MR. LARSEN said that he did not agree with the design method proposed and that he would write to Mr. Baird explaining the reason.

Paper 18-10-2 'Longitudinal shear design of glued laminated beams' by R.O. Foschi was also noted in the absence of the author. The paper gave a further explanation of design recommendations in the Canadian Code CSA-086 and was accompanied by a paper reprinted from the Canadian Journal of Civil Engineering, Vol.4, No.3, pages 363-370, 1977.

18. OTHER BUSINESS

There was strong support for a suggestion by MR. LARSEN to hold an international symposium on timber structures in two or three years' time. It was left to DR. STIEDA to set up a small group to make preliminary arrangements. MR. SUNLEY reminded the meeting that a great deal of work would be involved for the host country, with financial implications also.

The Chairman expressed the gratitude of members for the excellent arrangements made by DR. KORIN and the opportunity he had given them to appreciate the culture and history of Israel as well as providing first class facilities for the technical meetings.

19. NEXT MEETING

Those present were very pleased to accept an Italian invitation to hold the next meeting in Florence. It was agreed that this would be a joint meeting with IUFRO S5.02 and would take place in early September 1986 near the time of the IUFRO Congress in Yugoslavia.

20. PAPERS PRESENTED AT THE MEETING

CIB-W18/18-1-1	Notes on the Development of a UK Limit States Design Code for Timber - A R Fewell and C B Pierce
CIB-W18/18-1-2	Eurocode 5, Timber Structures - H J Larsen
CIB-W18/18-2-1	Column Design Methods for Timber Engineering - A H Buchanan, K C Johns, B Madsen
CIB-W18/18-6-1	Comment on Papers: 18-6-2 and 18-6-3 - R H Leicester
CIB-W18/18-6-2	Configuration Factors for the Bending Strength of Timber - R H Leicester
CIB-W18/18-6-3	Notes on Sampling Factors for Characteristic Values - R H Leicester
CIB-W18/18-6-4	Size Effects in Timber Explained by a Modified Weakest Link Theory - B Madsen and A H Buchanan
CIB-W18/18-6-5	Placement and Selection of Growth Defects in Test Specimens - H Riberholt
CIB-W18/18-6-6	Partial Safety-Coefficients for the Load-Carrying Capacity of Timber Structures - B Norén and J-O Nylander
CIB-W18/18-7-1	Model Specification for Driven Fasteners for Assembly of Pallets and Related Structures - E G Stern and W B Wallin
CIB-W18/18-7-2	The Influence of the Orientation of Mechanical Joints on their Mechanical Properties - I Smith and L R J Whale
CIB-W18/18-7-3	Influence of Number of Rows of Fasteners or Connectors upon the Ultimate Capacity of Axially Loaded Timber Joints - I Smith and G Steck

CIB-W18/18-7-4	A Detailed Testing Method for Nailplate Joints - J Kangas
CIB-W18/18-7-5	Principles for Design Values of Nailplates in Finland - J Kangas
CIB-W18/18-7-6	The Strength of Nailplates - N I Bovim and E Aasheim
CIB-W18/18-9-1	Prediction of Creep Deformations of Joints - J Kuipers
CIB-W18/18-10-1	Submission to the CIB-W18 Committee on the Design of Ply Web Beams by Consideration of the Type of Stress in the Flanges - J A Baird
CIB-W18/18-10-2	Longitudinal Shear Design of Glued Laminated Beams - R O Foschi
CIB-W18/18-12-1	Report on European Glulam Control and Production Standard - H Riberholt
CIB-W18/18-14-1	Simplified Calculation Method for W-Trusses (Part 2) - B Källsner
CIB-W18/18-14-2	Model for Trussed Rafter Design - T Poutanen
CIB-W18/18-15-1	Full-Scale Structures in Glued Laminated Timber, Dynamic Tests: Theoretical and Experimental Studies - A Ceccotti and A Vignoli
CIB-W18/18-15-2	Stabilizing Bracings - H Brüninghoff
CIB-W18/18-17-1	Sampling of Timber in Structural Sizes - P Glos
CIB-W18/18-102-1	Antiseismic Rules for Timber Structures: an Italian Proposal - G Augusti and A Ceccotti

21. CURRENT LIST OF CIB-W18 PAPERS

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

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

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden; February/March 1977
8 Brussels, Belgium; October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979
13 Otaniemi, Finland; June 1980
14 Warsaw, Poland; May 1981
15 Karlsruhe, Federal Republic of Germany; June 1982
16 Lillehammer, Norway; May/June 1983
17 Rapperswil, Switzerland; May 1984
18 Beit Oren, Israel; June 1985

b denotes the subject:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing
9 Duration of Load
10 Timber Beams
11 Environmental Conditions
12 Laminated Members

13 Particle and Fibre Building Boards
14 Trussed Rafters
15 Structural Stability
16 Fire
17 Statistics and Data Analysis
100 CIB Timber Code
101 Loading Codes
102 Structural Design Codes
103 International Standards Organisation
104 Joint Committee on Structural Safety
105 CIB Programme, Policy and Meetings
106 International Union of Forestry Research Organisations

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

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

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

LIMIT STATE DESIGN

1-1-1 Limit State Design - H J Larsen

1-1-2 The Use of Partial Safety Factors in the New Norwegian Design Code for Timber Structures - O Brynildsen

1-1-3 Swedish Code Revision Concerning Timber Structures - B Norén

1-1-4 Working Stresses Report to British Standards Institution Committee BLCP/17/2

6-1-1 On the Application of the Uncertainty Theoretical Methods for the Definition of the Fundamental Concepts of Structural Safety - K Skov and O Ditlevsen

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

18-1-1 Notes on the Development of a UK Limit States Design Code for Timber - A R Fewell and C B Pierce

18-1-2 Eurocode 5, Timber Structures - H J Larsen

TIMBER COLUMNS

2-2-1	The Design of Solid Timber Columns - H J Larsen
3-2-1	The Design of Built-Up Timber Columns - H J Larsen
4-2-1	Tests with Centrally Loaded Timber Columns - H J Larsen and S S Pedersen
4-2-2	Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns - B Johansson
5-9-1	Strength of a Wood Column in Combined Compression and Bending with Respect to Creep - B Källsner and B Norén
5-100-1	Design of Solid Timber Columns (First Draft) - H J Larsen
6-100-1	Comments on Document 5-100-1, Design of Solid Timber Columns - H J Larsen and E Theilgaard
6-2-1	Lattice Columns - H J Larsen
6-2-2	A Mathematical Basis for Design Aids for Timber Columns - H J Burgess
6-2-3	Comparison of Larsen an Perry Formulas for Solid Timber Columns - H J Burgess
7-2-1	Lateral Bracing of Timber Struts - J A Simon
8-15-1	Laterally Loaded Timber Columns: Tests and and Theory - H J Larsen
17-2-1	Model for Timber Strength under Axial Load and Moment - T Poutanen
18-2-1	Column Design Methods for Timber Engineering - A H Buchanan, K C Johns, B Madsen

SYMBOLS

3-3-1	Symbols for Structural Timber Design - J Kuipers and B Norén
4-3-1	Symbols for Timber Structure Design - J Kuipers and B Norén
1	Symbols for Use in Structural Timber Design

PLYWOOD

2-4-1	The Presentation of Structural Design Data for Plywood - L G Booth
3-4-1	Standard Methods of Testing for the Determination of Mechanical Properties of Plywood - J Kuipers
3-4-2	Bending Strength and Stiffness of Multiple Species Plywood - C K A Stieda

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

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

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

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

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

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

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

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

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

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

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

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 Norén

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

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

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

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

9-6-2	Code Rules for Tension Perpendicular to Grain - H J Larsen
9-6-3	Tension at an Angle to the Grain - K Möhler
9-6-4	Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler
11-6-1	Evaluation of Lumber Properties in the 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) - K Möhler
12-6-1	Strength Classifications for Timber Engineering Codes - R H Leicester and W G Keating
12-6-2	Strength Classes for British Standard BS 5268 - J R Tory
13-6-1	Strength Classes for the CIB Code - J R Tory
13-6-2	Consideration of Size Effects and Longitudinal Shear Strength for Uncracked Beams - R O Foschi and J D Barrett
13-6-3	Consideration of Shear Strength on End-Cracked Beams - J D Barrett and R O Foschi
15-6-1	Characteristic Strength Values for the ECE Standard for Timber - J G Sunley
16-6-1	Size Factors for Timber Bending and Tension Stresses - A R Fewell
16-6-2	Strength Classes for International Codes - A R Fewell and J G Sunley
17-6-1	The Determination of Grade Stresses from Characteristic Stresses for BS 5268: Part 2 - A R Fewell
17-6-2	The Determination of Softwood Strength Properties for Grades, Strength Classes and Laminated Timber for BS 5268: Part 2 - A R Fewell
18-6-1	Comment on Papers: 18-6-2 and 18-6-3 - R H Leicester
18-6-2	Configuration Factors for the Bending Strength of Timber - R H Leicester
18-6-3	Notes on Sampling Factors for Characteristic Values - R H Leicester
18-6-4	Size Effects in Timber Explained by a Modified Weakest Link Theory - B Madsen and A H Buchanan

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WORKING COMMISSION W18 - TIMBER STRUCTURES

NOTES ON THE DEVELOPMENT OF A UK LIMIT STATES DESIGN CODE FOR TIMBER

by

A R Fewell and C B Pierce
Princes Risborough Laboratory
United Kingdom

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

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1 INTRODUCTION

Through the requirements of the Eurocodes it is likely that a limit states partial coefficients design format will become an option for timber in the UK. The benefits of such a change are unclear but perhaps the major one is that by using a format common to all materials across Europe (and other countries) timber design procedures will become less specialised and will be understood and used by an increasing number of designers. Often quoted benefits such as rationalisation of probabilities of failure are likely to be initially very limited because of the current state of knowledge.

The problem that we are confronted with is how timber design can accommodate a limit states format with partial coefficients consistent with values for other materials, whilst at the same time both adhering strictly to the Eurocode definitions and achieving acceptable design solutions. Whilst reliability estimates of these solutions should be examined and compared between components and materials it would be pointless to start with the objective of rationalising reliability because too many assumptions would need to be made. This would result in meaningless reliability estimates and make any resulting radical changes in design solutions unacceptable. The one thing we know for sure is that current design solutions are generally acceptable, and we can use these to estimate the partial coefficients and some of the parameters for which we have little or no data. This would in effect be adopting a similar approach to that used in the UK for the Eurocodes for concrete and steel.

This paper firstly illustrates by a simple calibration example, the extent of the problem of using a limit states design procedure for timber that aims to use partial coefficients consistent with other materials and at the same time maintain current design solutions. It then reviews and comments on various suggestions for the inclusion of factors which might alleviate the problem. Lastly comments are made on the inclusion of reliability concepts and the presentation of design values in structural codes.

PRL59-84

But $f_{LS} = \frac{f_K \times 0.563}{\gamma_M}$ where f_K is characteristic strength (for test duration loads)

γ_M is the material partial coefficient

and 0.563 is a factor to adjust f_{LS} to long duration loads

$$\therefore \quad \gamma_M = \frac{f_K \times 0.563\ (Q + G)}{f_p(\gamma_Q Q + \gamma_G G)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This assumes that Q and G are the same in both cases and also that in the limit state equation the safety element for geometrical properties Δa is zero. This last assumption is reasonable because the negative tolerance on timber cross section dimensions is very small (1 mm) and is only allowed on up to 10 per cent of pieces.

Values for loads and load partial coefficients

There are currently two proposals for γ_Q and γ_G. The first from the Eurocode draft is $\gamma_Q = 1.5$ and $\gamma_G = 1.35$. The second from the UK concrete code CP 110 is $\gamma_Q = 1.6$ and $\gamma_G = 1.4$. Both sets of values are included in this analysis to give some indication of the sensitivity of γ_M to changes in γ_Q and γ_G. It should be pointed out that these values have been derived somewhat arbitrarily and are not based on research data.

Equation (1) for γ_M derived above is applicable to floor joists, ceiling joists, roof joists and purlins as given in schedule 6 of the Building Regulations. For rafters it is not possible to obtain an explicit equation for second moment of area because secondary bending is involved. The values of Q and G used in the design of members are as follows:

vary γ_M for size and would make stress adjustments for member size the same in the intended limit states code of practice BS 5268:Part 1 as in Part 2. The determination of values for f_K/f_p for bending and tension strength are shown in Figure 1 for visual grades (taken from ref 3) and Figure 2 for machine grades. Although machine and visual grades have the same permissible stresses, machine grades will have lower f_K values due to the smaller safety factor applied as a result of their greater grading accuracy. γ_M values are intended to reflect the uncertainties of material properties and will therefore differ for machine and visual grades.

It can be seen from Figure 1 that for bending and tension strength of visual grades

$$\frac{f_K}{f_p} = \frac{1}{0.724} \times \frac{1}{0.563} = 2.45$$

Similarly from Figure 2 for machine grades

$$\frac{f_K}{f_p} = \frac{1}{0.8} \times \frac{1}{0.563} = 2.21$$

But it must be remembered that the f_K values are based on 300 mm depth or width and are not the f_K values in Fewell[3].

γ_M values were calculated from equation 1 using the values for f_K/f_p and applied loads given above. Similar calculations were carried out for other properties and the range of γ_M for various ratios of Q/G is illustrated in Figures 3 and 4. Figure 5 shows the variation in γ_M for the members in the Building Regulations schedule 6 span tables. From these figures appropriate values of γ_M were determined and are listed in Table 2. In a comprehensive code calibration exercise γ_M would also be influenced by its effect on volume timber consumption.

3 THE CONSIDERATION OF ADDITIONAL FACTORS AND INFLUENCES

Three of the basic components of the design equation (1) namely, the actions, the characteristic stresses and the action partial coefficients need to be examined to determine if modifications are justified which will result in more realistic γ_M values. Suggested modifications are now considered under separate headings to assess their justification and the likely extent of any benefit to be obtained.

It may be thought that this exercise is just playing with numbers and the actual values are irrelevant. Whilst there is an element of truth in this in that the factors and coefficients must tie up with current design solutions, they do have clear definitions which cannot be completely ignored. Having to isolate all the parameters in the design equation will provide us with a better understanding of their effects, which in the long term can only improve the economy and reliability of timber structures.

a. Comparisons between lightweight and heavyweight materials.

The overall factor on the load side of the design equation has a value of $(\gamma_G G + \gamma_Q Q)/(G + Q)$. Because γ_G and γ_Q have different values the overall factor varies with the proportions of G and Q and consequently will be influenced by the weight of the construction material. Figure 6 was drawn using the equation given above and the alternative values for γ_G and γ_Q used previously in this paper.

Figure 6 also shows the range of Q/G for timber, taken from Table 1, and for concrete and steel from a personal communication from Judge[5] giving details of floor and roof beams. It can be seen that there is generally a higher overall load factor on timber than the heavier materials although this is only around 2 to 7%. It has been suggested that a load reduction factor should be included which would allow for this effect and at the same time increase γ_M. However a principle of the partial coefficient approach is that where relatively higher proportions of imposed loads, which are less predictable, exist the overall load factor is effectively increased. It would be wrong therefore to counteract this effect with a lightweight material factor.

subjected to the maximum bending moment would have a probability of more than 95% of being stronger than the characteristic value. Appropriate factors are therefore required which will increase timber MOR and MOE values to achieve consistency with other materials.

Data stored in PRL's computer data bank were analysed to determine such factors using the Cook-Bolinder grading machine measurements which are closely related to strength and the MOE values listed in BS 5268. In this way lower fifth percentile values of bending strength for a random test position within each piece could be compared to the test value which has the assumed weakest cross-section at the position of maximum bending moment. The data comprised two samples of Canadian spruce-pine-fir (38 x 184) and one sample of European whitewood (50 x 100). Each sample was graded into SS and GS sub-samples and the resulting mean factors were 1.12 for MOR and 1.04 for MOE. However this approach to determining the required factors is not entirely satisfactory and there was considerable variation between sub-samples. A programme of tests using a matched distribution approach is being proposed and this should lead to more reliable values which it is estimated will probably be in the range of 1.10 to 1.25 for MOR and 1.00 to 1.08 for MOE and may well vary with grade. These factors are much less than some researchers have assumed but it should be noted that Leicester[6] uses a factor of 1.15 to adjust MOR test results with the assumed worst defect at the position of maximum bending moment and the worst edge in tension to an MOR value with random defect positions.

Whilst the above factor will have the desired effect of increasing γ_M by around 15 to 20% this may be counteracted by the possible need to provide a stricter definition of the characteristic stress. The CIB Structural Timber Design Code[7] defines the characteristic stress as a lower fifth percentile with a 75% lower confidence bound. This lower condfidence bound is not yet mentioned in the Eurocodes already drafted but the intention is that the CIB Code will be used as a basis for the Eurocode for timber. The UK characteristic stresses are determined assuming a 3 parameter Weibull distribution without a lower confidence bound. Kroll[8] has now produced a computer programme which for the Weibull distribution enables the fifth percentile to be determined with a 75% lower confidence bound. It seems that this will have the effect of reducing characteristic stresses by around 1 to 3 % although its inclusion would be unjustified if it was not also a requirement for other materials.

In Australia the specified duration of the maximum load for floor joists has just been decreased from 50 years to 5 months which increases permissible bending stresses by a factor of 1.4. This may be extreme but it is an example of a country moving in the direction proposed above.

For strength properties other than bending even less evidence exists on the effect of load duration. Currently the same factors are used for all strength properties but that is probably an over-simplification. It would be convenient if the effect of load duration was less for strength properties other than bending. This would then counteract the adjustment due to test method (described under c) for bending strength.

4 THE PRESENTATION OF STRESSES AND PARTIAL FACTORS IN BS 5268:PART 1

The permissible stress design code BS 5268:Part 2 already has a profusion of grade stress tables which, it has been said, will be confusing to many designers unfamiliar with the document. To list different partial factors and grade stresses for visual and machine grades will only serve to extend these tables and complicate design procedures where the method of grading to be used is uncertain. There is also the problem of strength classes which include both types of grading and will add a further complication.

The easiest way to overcome the above problem would be to tabulate values of f_{LS} (where $f_{LS} = 0.563\ f_K/\gamma_M$) and therefore remove the need to list separate values for machine and visual grades. A precedent for this type of presentation already exists in the draft code for steel BS 5950. This would still require the same amount of information to be tabulated as in BS 5268:Part 2 but the only way of reducing this further would be to delete stresses for individual grades and species and include only strength class information.

5 RELIABILITY ANALYSIS

The use of classical reliability models to represent the design process completely disregards about 90% of all failures[13,14]: these occur as a result of poor workmanship, abuse or gross human error. As with any model the quality of the result is strongly dependent upon the quality of the input data and the accuracy with which the model represents reality. In representing loads and resistances with particular types of idealised statistical distributions the analyst often has to make decisions based on

The load distribution was assumed to have 20 per cent coefficient of variation (V_S) throughout, though of course this could easily be put into the analysis as a further variable. A simple assumption was made for the duration of load effect in that the strength of every piece is reduced by a constant modification factor regardless of quality, the value being taken from the Madison curve (Figure 7).

Based on a single sample of 164 pieces of 50 x 150 mm Swedish redwood/whitewood, Figure 8 shows the relationship between log p_F (or the safety index β) and the level of applied load for both short term (5 minutes) and long term (50 years) periods. The following points emerge:

i With all models p_F increases (β decreases) as the load and strength distributions become "closer" (ie as $\overline{n}$ becomes smaller) but both the level of p_F and its rate of increase vary enormously depending on the model. However, the N/N, E_1/N and N/W2 models produce answers that are generally quite similar to each other but the LN/LN, N/W3 and E_1/W3 models appear to give very much smaller p_F values.

ii The N/N and E_1/N models produce almost identical probability of failure values in this example and indeed they cannot be distinguished in Figure 8. This is perhaps a surprising result that is due to the positioning of the failure distribution relative to the distributions of R and S. Figure 9 shows that the failure integral may lie almost entirely within the main body of the resistance or applied stress distributions or anywhere between them depending on $\overline{n}$ and the coefficients of variation V_R and V_S. When V_R is large (as in timber) the p_F distribution is governed by the body of the S distribution and the tail region of S is unimportant. However, it does mean that p_F is very sensitive to the shape of the lower tail of the R distribution. These observations would not necessarily be applicable to a material with low V_R - see Figures 9b and 9c.

When the 3-parameter Weibull distribution is fitted to the same sample of strength data a fairly high location parameter ε is estimated, and since the p_F distribution must lie above ε, it lies entirely within the R distribution. Consequently for a high value of ε the value of p_F from the N/W3 and E_1/W3 models will be very much smaller than that from the corresponding N/N and E_1/N models. Hence when R is represented as a as indicators of population values. The strength values of 21 samples of redwood/whitewood of varying size and grade were fitted by 3-parameter Weibull distributions. As might be expected intuitively, the value of the estimated location parameter β decreases as the sample size n increases (Figure 12). There will probably be a limiting value of β which represents the stress above which pieces will not be broken by routine handling operations. This relationship has a very major effect on the behaviour of the N/W3 and E_1/W3 models for reliability work. Figure 13 shows that as the location parameter decreases the probability of failure at design stress approaches a value of about 10^{-3} under long-term load assumptions; this seems to be regardless of the shape and scale parameters except that variation in these parameters would contribute to the sampling error either side of $p_F = 10^{-3}$.

Consequently this analysis has an important bearing on the first analysis, which compared the different models. The fact that the N/W3 model appeared to give far lower p_F values than the N/N and N/W2 models is not a true model difference but very largely an effect of sample size; a similar comment applies to the E_1/W3 model in relation to E_1/N. When the largest sample available (n = 220 for 38 x 150 mm S10 grade material) was put into the N/W3 and E_1/W3 reliability models, Figures 10 and 14 show that the resulting probabilities of failure were much nearer to the N/N, E_1/N and N/W2 models. It also implies that if small samples were used the resulting p_F values from the N/W3 and E_1/W3 models could be ridiculously small. The analysis shows that it would be unwise to use sample sizes less than about 200 for predicting reliability estimates of populations.

The above discussion also raises the question as to whether the current practice of estimating 5th percentile characteristic stresses of populations from small samples using the 3-parameter Weibull is satisfactory: the true population value of ε is likely to be very small and therefore either the 2-parameter Weibull or the 3-parameter Weibull with an arbitrarily low value of ε may be more applicable. Ideally more research is required here to compare the efficiency of the two models in estimating 5th percentiles from small samples.

The choice of distributional types for a reliability model should be made primarily on an understanding of the material behaviour and the nature of the safety of particular elements. These methods must not be used indiscriminately (or on a large scale), but they could be useful in comparing safety levels of existing and proposed design methods as long as the comparisons are confined within particular element types of the same material so that the same set of assumptions is valid throughout.

CONCLUSIONS

1. This paper illustrates by a calibration example the major problem of adopting a limit states format for timber design with partial coefficients consistent with values for other materials, whilst at the same time adhering to the Eurocode definitions and achieving acceptable design solutions.

2. By carrying out research and including adjustments to the design parameters for (a) methods of testing and determining characteristic stresses, and (b) the effects of load duration, the above problem can possibly be overcome whilst at the same time achieving a better understanding of the various factors which influence the design of timber members. This greater understanding should in the long term help to improve the economy and reliability of timber structures.

3. The use of a consequence factor to benefit timber in its major use of low rise housing and a factor to reduce the higher overall load factor for timber compared with concrete and steel are difficult to justify.

4. In order that the different f_K and γ_M values for machine and visual grades do not increase the complexity of the stress tables over those given in BS 5268:Part 2, it is recommended that f_{LS} should be tabulated and not f_K and γ_M separately. A precedent for this type of presentation already exists in the draft code for steel BS 5950.

5. Whilst initially partial coefficients could be determined by calibration to current design solutions it would be advantageous in the long term to rationalise the notional probabilities of failure (p_F) for various components by the use of reliability studies. From analyses investigating the effect on p_F of various distributional models for load and resistance in the design of a simple beam the following conclusions are apparent.

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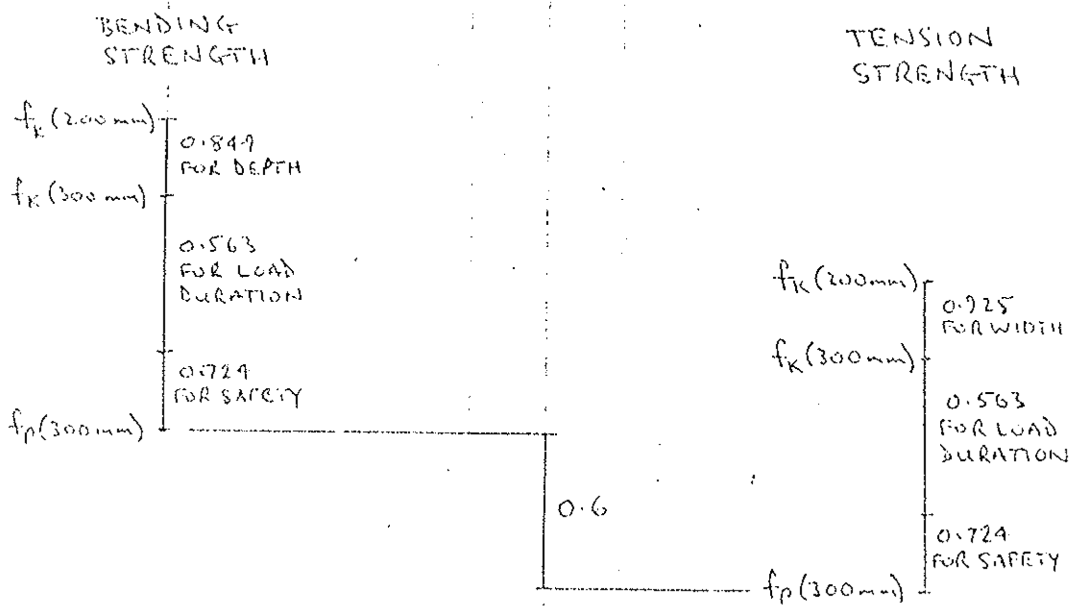

FIG 1 DIAGRAM OF STRESS FACTORS FOR VISUAL GRADES

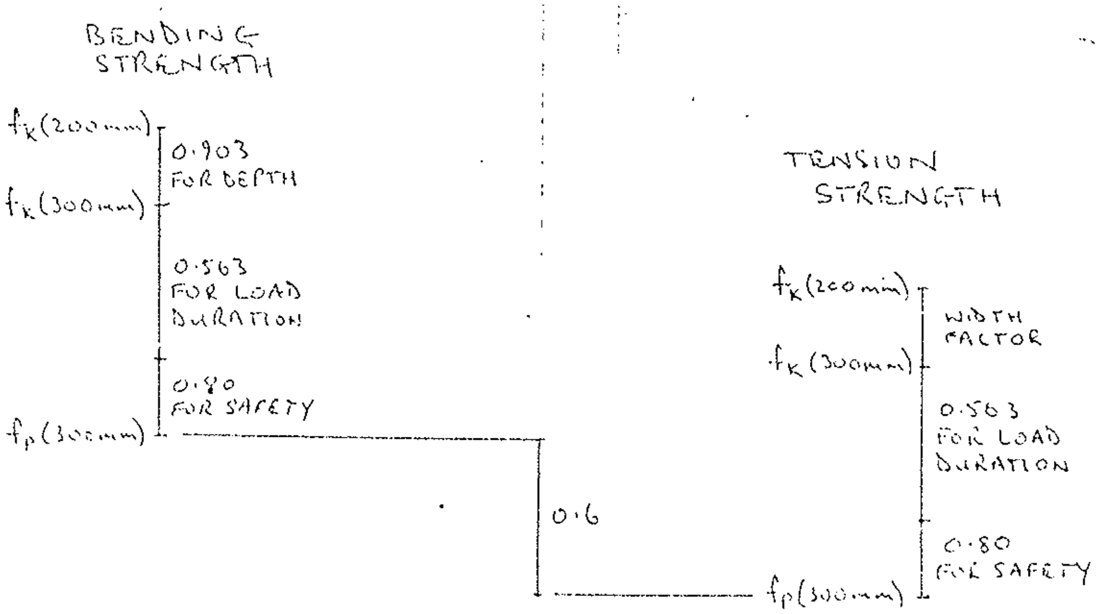

FIG 2 DIAGRAM OF STRESS FACTORS FOR MACHINE GRADES

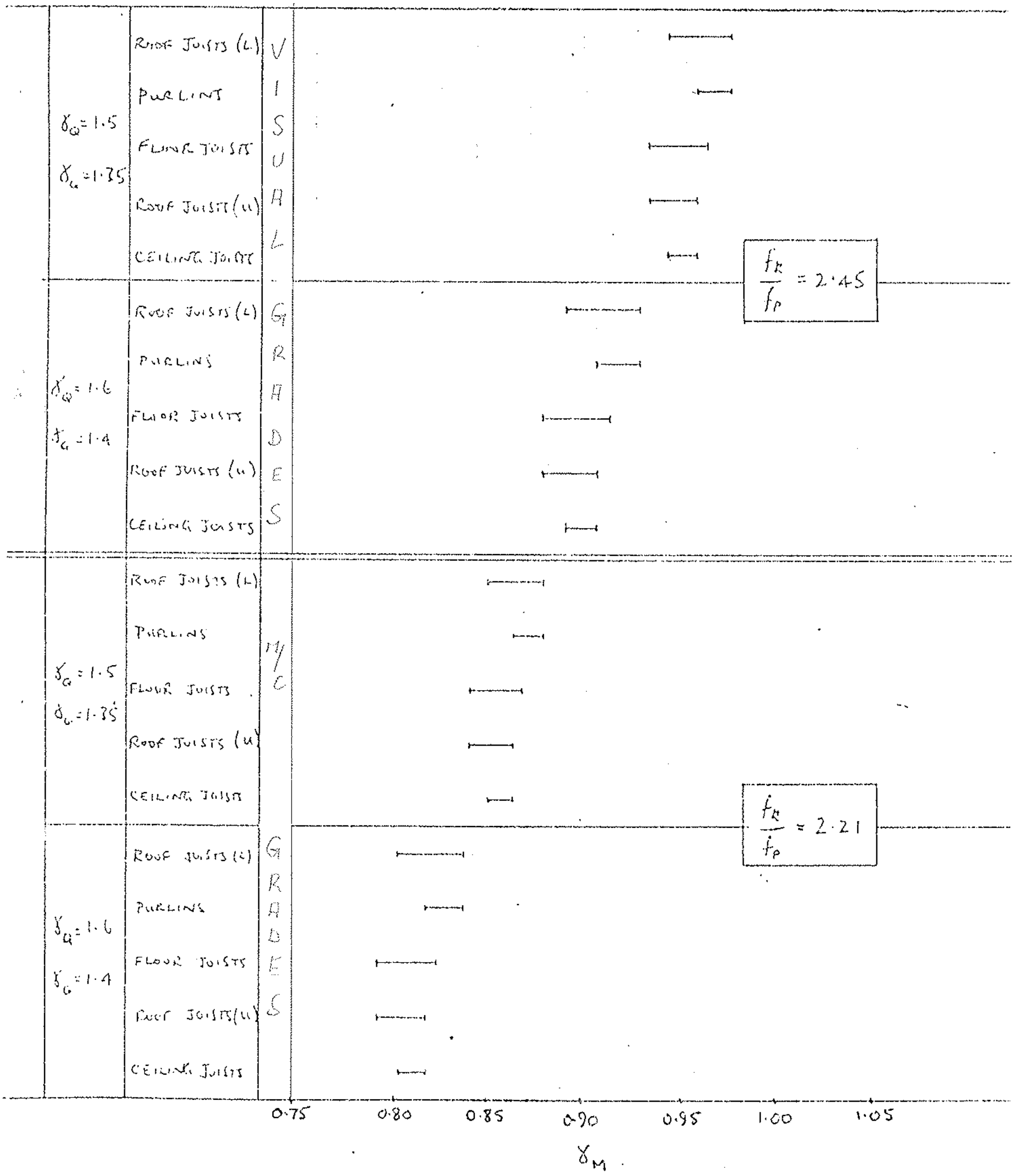

FIG 5 VALUES OF γ_M FOR BENDING

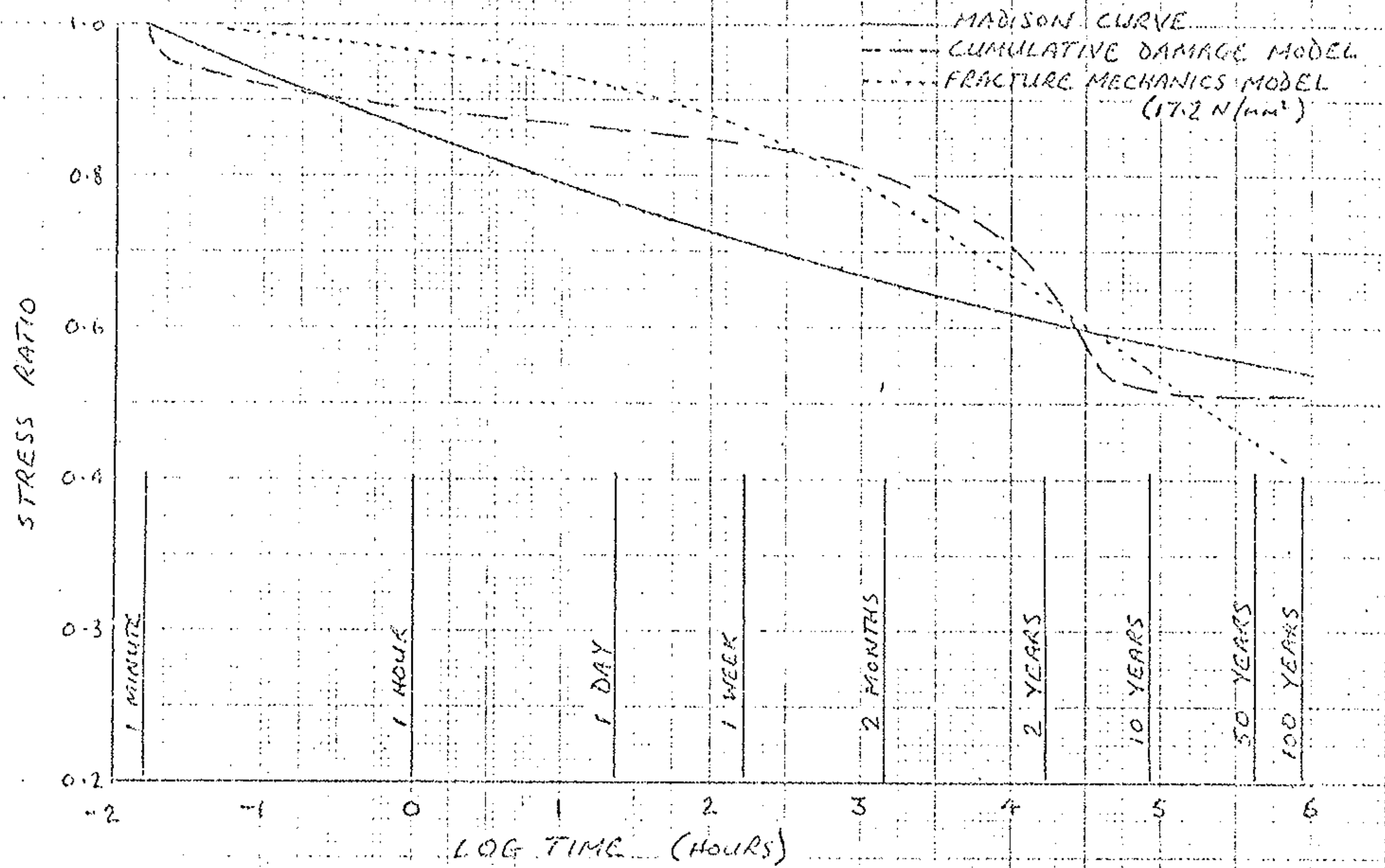

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FIG 7. DURATION OF LOAD MODELS FOR BENDING STRENGTH

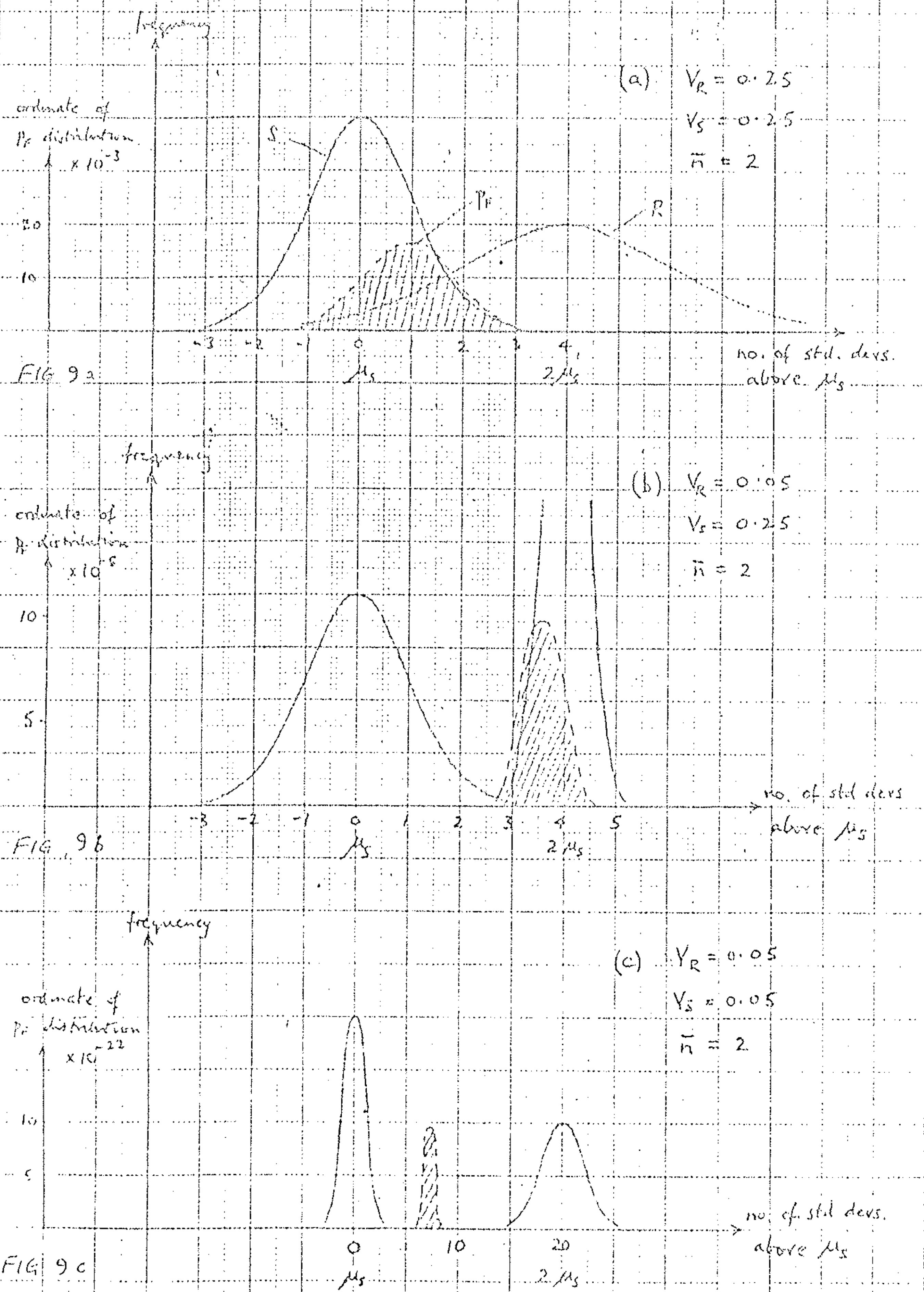

FIG 9. RELATIVE POSITION OF P_f DISTRIBUTION FOR N/N MODEL

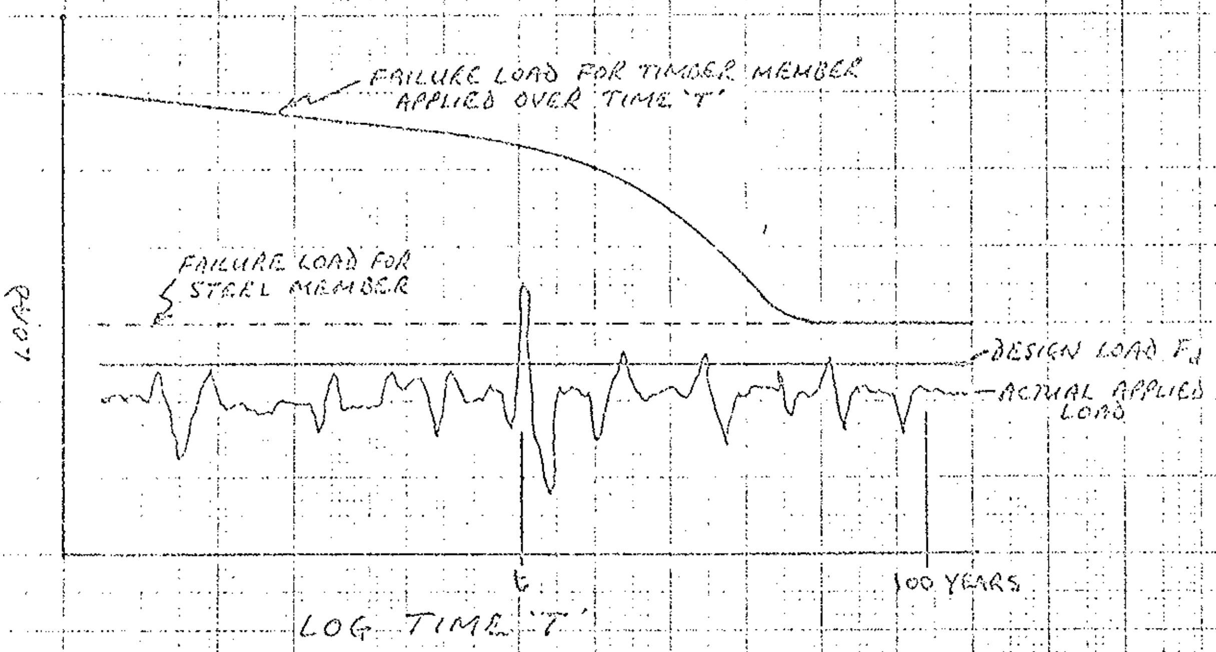
FAILURE LOAD FOR TIMBER MEMBER APPLIED OVER TIME 'T'
FAILURE LOAD FOR STEEL MEMBER
LOAD
DESIGN LOAD F_d
ACTUAL APPLIED LOAD
t
100 YEARS
LOG TIME 'T'

FIG 11

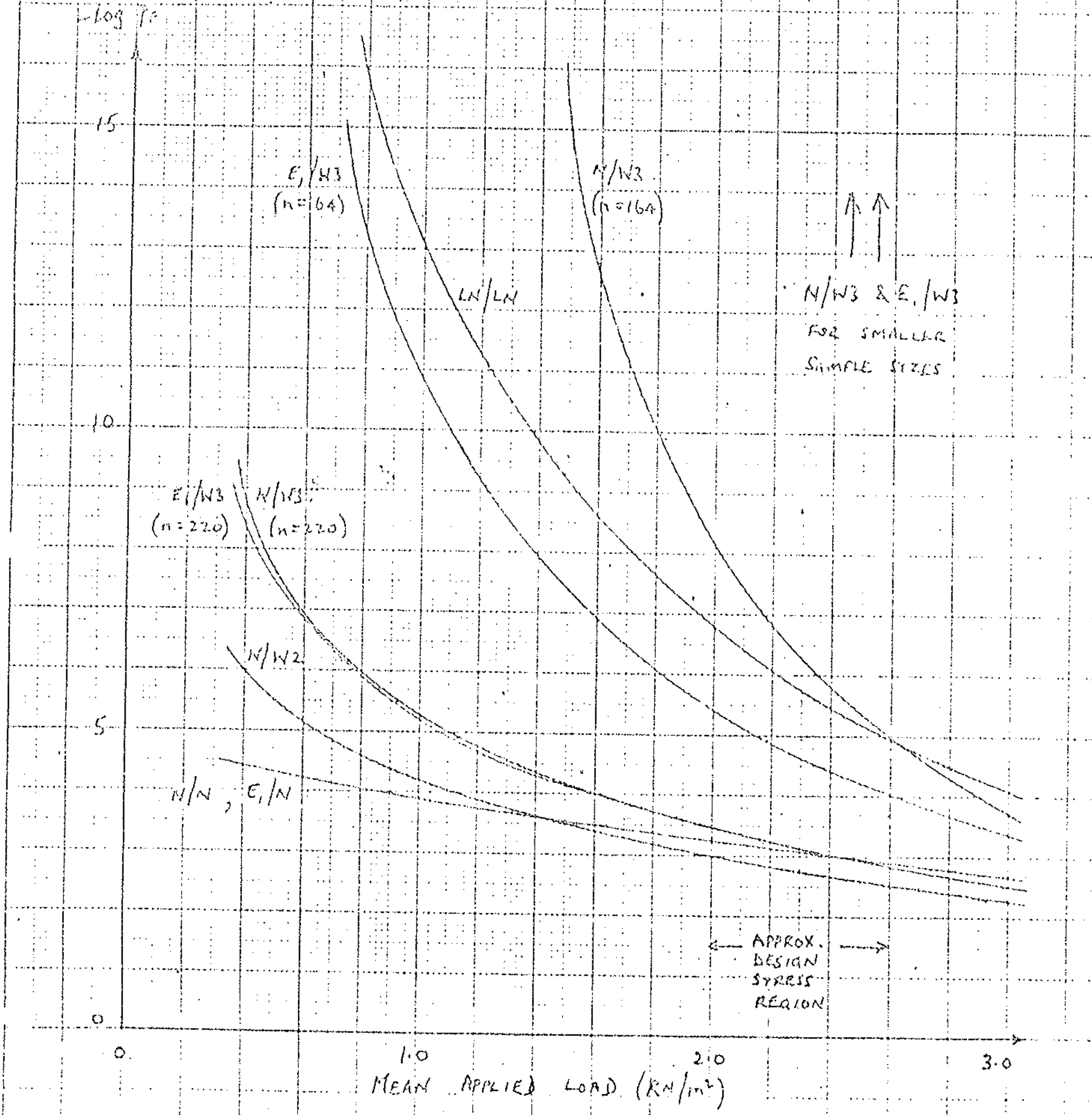

FIG 14 COMPARISON OF LONG-TERM FAILURE PROBABILITIES SHOWING HOW E1/W3 AND N/W3 ARE AFFECTED BY SAMPLE SIZE

CIB-W18/18-1-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

EUROCODE 5, TIMBER STRUCTURES

by

H J Larsen
Danish Building Research Institute
Denmark

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

1. INTRODUCTION

The European Community is drawing up a series of codes for building structures. These codes - Eurocodes - are intended to establish a set of common rules as an alternative to the differing rules in force in the various member states.

The European complex consists of

Eurocode No. 1	Common unified rules for different types of construction and materials
- No. 2	Concrete structures
- No. 3	Steel structures
- No. 4	Composite structures (steel concrete)
- No. 5	Timber structures
- No. 6	Masonry structures
- No. 7	Foundations
- No. 8	Structures in seismic zones

Eurocode No. 1, 2 and 3 have been published, and the member states have been asked to comment on them before the end of September 1985.

Eurocode No. 4 is available in an English version.

A drafting panel consisting of

Heinz Brüningshof
Philippe Crubilé
Jürgen Ehlbeck
Hans Jørgen Larsen (chairman)
John Sunley

has been asked to prepare a draft for Eurocode 5 before the end of October 1985.

A first draft for the Eurocode has been prepared, based on CIB Publication 66, CIB - Structural Timber Design Code. It has, however, been necessary to draft a number of supplementary sections and also to make changes in the content and the way of presentation in some other sections to make the code complete and coherent.

In the following some of these supplements and changes are presented as a basis for discussion in CIB W18, viz the safety system and partial coefficients, material specifications, tension perpendicular to the grain and shear design.

The Eurocode text is in two levels, Principles, which specify the requirements and the criteria to be fulfilled by the structure, and Rules for Application (identified by : in the margin), which are intended to be examples on acceptable methods of satisfying the Principles.

2. NOTATION

In the following only general symbols are given. Other symbols are defined when used.

Main symbols

R	resistance
S	action effect
Q	load
V	volume
f	strength value
g	dead load
q	variable load
γ	partial coefficient
σ	stress
τ	shear stress
ψ	combination factor

Subscripts

c	compression
d	design
dis	distribution
ef	effective
k	characteristic
m	bending
	material property (on γ)
max	maximum
t	tension
v	shear
vol	volume
wei	Weibull
0	in the fibre direction, parallel to grain
90	perpendicular to grain

3. SAFETY SYSTEM AND PARTIAL COEFFICIENTS

The safety system is a partial coefficient method based on a limit state philosophy, see e.g. ISO/DIS 2394, General Principles on reliability for structures.

In brief the design format for a strength problem is as follows:

A structure is loaded by dead load g and n variable load. For each variable load it is assumed that the characteristic value (e.g. corresponding to a return period og 50 years) and the combination value are known. The characteristic values are q_1, ... q_n. The combination values are given as $\psi_{1,1}q_1$, ..., $\psi_{1,n}q_n$. The factors ψ_1 are about 0,5 - 0,7.

The design load effect S_d is calculated for the design load combination:

$$S_d(\gamma_g G + \gamma_q(q_1 + \sum_{i\geq 2} \psi_i q_i)$$

The design resistance is calculated from the characteristic resistance R_k (e.g. corresponding to the 5-percentile) as

$$R_d = R_k/\gamma_m.$$

γ are partial coefficients (γ_g and γ_q: load factors. γ_m: material factor).

It is then required that

$$S_d \leq R_d$$

As tentative values for the Eurocodes

$$\gamma_g = 1.35$$

and

$$\gamma_q = 1.50$$

have been proposed. Corresponding to these values the drafting panel for Eurocode 5 has proposed

$$\gamma_m = \cancel{1.4} \quad 1, \quad 1,3$$

This value has been fixed by comparison with the present safety level in the codes in France, Germany and Denmark and with the γ_m values proposed in Eurocode 2 and 3 for concrete and steel.

The value should be evaluated together with the long term factors given in table 3.1.4a on page 6.

4. MATERIAL SPECIFICATIONS

The drafting panel has set up the following principles for the inclusion of materials.

A very large number of timbers and wood based panel materials are available to the EEC countries for structural use. However, only those materials which satisfy the following conditions are included:

- Substantial quantities are likely to be available to the EEC countries
- The materials are produced (graded or manufactured) to an acceptable available laid down standard
- The mill or factory producing the material is subjected to acceptable independent third party quality surveillance and the product marked accordingly
- Sufficient test or design information is available to enable characteristic strength values to be allocated to the material.

Also, proprietary jointing devices have not been included. It is assumed that such materials obtain Agrément or other acceptable certification.

The first draft for the sections on structural timber is given below. It is very preliminary, and a first discussion of it will take place in May.

It is based on strength classes along the same lines that has been followed by ISO in ISO TC 165/DP 8972 Timber Structures - Solid timber and poles - Structural grouping. But in stead of the strict geometrical scale of the ISO proposal (characteristic bending strength in MPa: 12 - 15 - 19 - 24 - 30 - 38) the Eurocode classes have in the central area been based on the strength profiles of the most commonly used species and grades (characteristic bending strength in MPa: 12 - 15 - 21,5 - 28,5 - 38), see table 3.1.3b.

The modification factor in table 3.1.4a should as mentioned above be seen in connection with the choice of safety factors.

3.1.2 Strength classes

A series of strength classes C 1 - C 8 are described in table 3.1.2.

The bending strength applies to a beam with a depth of 200 mm. The tension strength applies to a member with a width of 200 mm.

Table 3.1.2. Strength classes. Characteristic values in MPa

MPa		C1	C2	C3	C4	C5	C6	C7	C8
		Strength class							
bending	$f_{m,k}$	12.0	15.0	21.5	28.5	38.0	48.0	60.0	75.0
tension									
- par. to grain	$f_{t,0,k}$	7.5	9.5	13.5	18.0	24.0	30.0	38.0	48.0
- perp. to grain	$f_{t,90,k}$	0.3	0.3	0.4	0.5	0.6	0.8	1.0	1.0
compression									
- par. to grain	$f_{c,0,k}$	14.5	16.5	19.0	26.0	30.0	38.0	48.0	60.0
- perp. to grain	$f_{c,90,k}$	6.5	6.5	7.0	8.0	11.0	13.0	15.0	18.0
shear	$f_{v,k}$	1.9	1.9	2.4	3.0	3.8	4.8	6.0	6.0
Modulus of elasticity, mean	$E_{0,mean}$	8000	9000	10000	12000	13500	17000	22000	27000

3.1.3 Species and grades

A species and grade can be assigned to a strength class in accordance with table 3.1.3 a.

Table 3.1.3 a. Classification rules

grade classification for each property $f_{m,k}$	$f_{t,0,k}$	$f_{c,0,k}$	$E_{0,mean}$	resultant assigned strength class
C_x	C_x	C_x	C_x	C_x
C_x	C_x	C_{x-1}	C_{x+1}	C_x
C_{x+1}	C_x	C_{x-1}	C_{x-1}	C_x

C_x means that the characteristic value is greater or equal to the required characteristic value given in table 3.1.2.

The strength classes are independent of species and grades. For some applications it may however be necessary to specify a particular species from within a strength class to take account of particular characteristic; e.g. natural durability, ease of preservation, adhesives and fasteners.

Table 3.1.3 b gives species and grades for some softwoods meeting the requirements of classes C1 - C4 given in table 3.1.2. Classes C5 - C8 will usually comprise relatively dense hardwoods.

Table 3.1.3 b. Strength class for some softwoods

	strength class			
	C1	C2	C3	C4
European				
whitewood, redwood		S6	S8	S10
North American				
douglas fir-larch, hem-fir		S6	S8	S10
spruce-pine-fir		S6	S8	S10
douglas fir-larch, hem-fir		No. 1, No. 2	Sel.	
spruce-pine-fir		No. 1, No. 2	Sel.	
western whitewood (USA)	No. 1, No. 2	Sel.		
southern pine (USA)		No. 1, No. 2 No. 3		Sel.

S6, S8, S10: Grades according to ECE (1982)
No. 1, No. 2, No. 3, Sel(ect): Joist and plank grades according to NLGA 1979 or NGRDL 1975.

3.1.4 Characteristic strength values

For structures in moisture class 1 and with very short-term loads the characteristic strength values given in table can be used. For beams a depth of 200 mm is assumed and for tension members a width of 200 mm. If the depth or width respectively is h the bending strength or tension strength should be multiplied by k_h

$$k_h = \left(\frac{200}{h}\right)^{0.2} \qquad (3.1.4\ a)$$

For other cases than moisture class 1 and very short-term load the characteristic values from table 1 - modified for depth and width other than 200 mm - should be multiplied by the factor $k_{mod,1}$ given in table 3.1.4

Table 3.1.4 a. Modification factor $k_{mod,1}$ to strength values

	tensile strength *perp.* to grain and shear strength		other strength properties	
load-duration class	moisture class 1 and 2	moisture class 3	moisture class 1 and 2	moisture class 3
long-term	0.60	0.50	0.85	0.70
medium-term	0.75	0.60	0.90	0.70
short-term	0.90	0.70	0.95	0.75
very short-term	1.00	0.80	1.00	0.80
instantenous	1.20	1.00	1.20	1.00

If a load combination consists of actions belonging to different load-duration classes $k_{mod,1}$ should be chosen corresponding to the action with the shortest duration, e.g. for dead load and a short term-load k_{mod} corresponding to short term-load is used.

For instability calculations (lateral instability, columns, buckling) the ratio $E_{0,d}/f_{c,0,d}$ can be taken from table 3.1.4 b. The table is based on the assumption that there is no safety differentation between short and slender columns.

Table 3.1.4 b. The ratio $E_{0,d}/f_{c,0,d}$ for use by instability calculations

strength classe	C1	C2	C3	C4	C5	C6	C7	C8
$E_d/f_{c,0,d}$	370	350	330	310	310	310	310	310

3.1.5 Characteristic stiffness values

For structures in moisture class 1 and with short-term loading deflections in serviceability states can be calculated with $E_{0,mean}$ from table 3.1.4.

For other cases $E_{0,mean}$ should be divided by k_{creep} given in table 3.1.5.

The shear modulus G_{mean} can be taken as $E_{0,mean}/16$.

The modulus of elasticity perpendicular to the grain $E_{90,mean}$ can be taken as $E_{0,mean}/30$.

Table 3.1.5. Modification divisor k_{creep} to stiffness values

	moisture class		
load-duration class	1	2	3
long-term	1.5	2.0	3.0
medium-term	1.2	1.5	2.0
short-term very short-term instantaneous	1.00	1.25	1.5

If a load combination consists of actions belonging to different load-duration class the deflection are calculated separetely for the different action with the appropriate creep factor.

3.3 Glued laminated timber

3.3.1 The workmanship should be of such quality that the glued joints maintain the required integrity and strength throughout the expected life of the structure.

For the adhesives section 3.6 apply.

Glued laminated timber should be manufactured in accordance with ISO BBBB.

3.3.2 Grades

Glued laminated timber should be produced to the same strength classes as timber (section 3.1.2) and sections 3.1.3 - 3.1.5 apply.

The strength and stiffness values should be determined either by testing in accordance with section 3.1.1 or by the use of appropriate calculation models for determining the strength and stiffness from the properties of the lamellae.

: It can be assumed that glue laminated timber of strength class C 3, C 4 and C 6 can be obtained by using timber grades C 2, C 3 and C 5 in accordance with table 3.3.2. It is assumed that the lamellae are finger jointed in accordance with 3.2 to the same strength class as the timber.

: Table 3.3.2. Strength class of glued laminated timber dependent on lamella quality

	strength class of glued laminated timber		
	C 3	C 4	C 5
Tensile members			
- all lamellae	C 2	C 3	C 4
Bending and compression members			
- outer lamellae[1)]	C 2	C 3	C 4
- inner lamellae	C 1	C 2	C 3

: 1) Outer lamellae are the lamellae in the extreme sixth of the depth on either side. There should, however, be at least two outer lamellae.

5. TENSION PERPENDICULAR TO THE GRAIN

The general rules for tension perpendicular to the grain are as follows.

5.1.3 Tension perpendicular to the grain

The following condition should be satisfied:

$$\sigma_{t,90,d} < k_{vol}\, k_{dis} f_{t,90,d} \qquad (5.1.3\ a)$$

where k_{vol} (≤ 1) takes into account the effect of the size of the stressed volume V, and k_{dis} the effect of the distribution of the stresses.

k_{vol} and k_{dis} can be determined under the assumption of a 2-parameter Weibull distribution of $f_{t,90}$.

For a stressed volume V k_{vol} can be taken as

$$k_{vol} = \left(\frac{V_0}{V}\right)^{1/k_{wei}} \qquad (5.1.3\ b)$$

where $V_0 = 0.02\ m^3$ and $k_{wei} = 5$ can be assumed for most softwood species.

For a uniformly stressed volume $k_{dis} = 1$.

For a volume V with a varying stress $\sigma_{t,90} = \sigma\,(x,y,z)$ with a maximum value of σ_{max}

$$k_{dis} = \left(\frac{1}{V}\int_V \left(\frac{\sigma(x,y,z)}{\sigma_{max}}\right)^{k_{wei}} dV\right)^{1/k_{wei}} \qquad (5.1.3\ c)$$

As compared to the CIB code the basis for the volume factor has been given explicitely (a 2-parameter Weibull distribution), and rules for the general case with a variable stress has been given.

This makes it possible for the user to calculate the distribution factor for other cases with tension perpendicular to the grain, i.e. for curved beams, where the Eurocode rules are as follows:

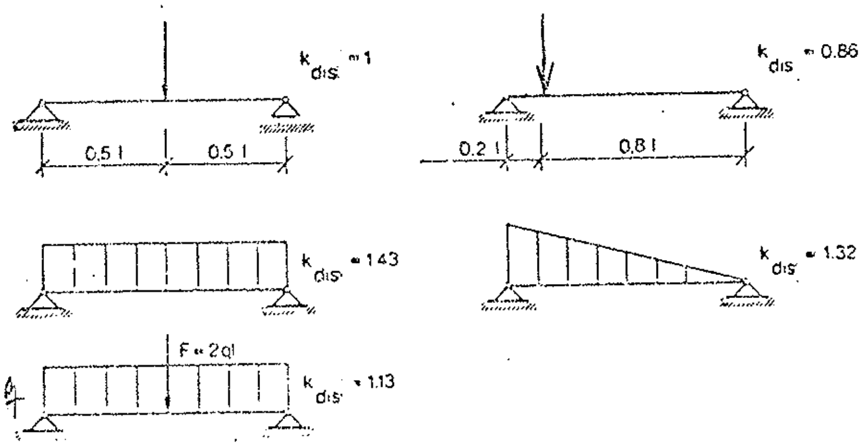

Figure 5.1.7 b. Examples on $k_{dist,v}$

In relation to these rules it has been necessary also to link the definition of the characteristic shear strength to a specified strength distribution volume. In accordance with the tensile strength perpendicular to the grain a uniformly stressed volume of 0,02 m^3 has been chosen.

That V_o in (5.1.7c) is 0,025 m^3 and not 0,02 m^3 is because of the parabolic stress distribution in the beam.

The correct limit for the use of (5.1.7b) is of course 0,025 m^3 for a concentrated load in the middle.

To use the limit 0,1 m^3 corresponds to a distribution factor of $4^{0,2}$ = 1,32, a value to be compared with k_{dis} = 1,43 for a uniformly distributed load.

5.1.12 Curved beams

This section applies to simply supported curved beams with constant, rectangular crosssection, see fig. 5.1.12.

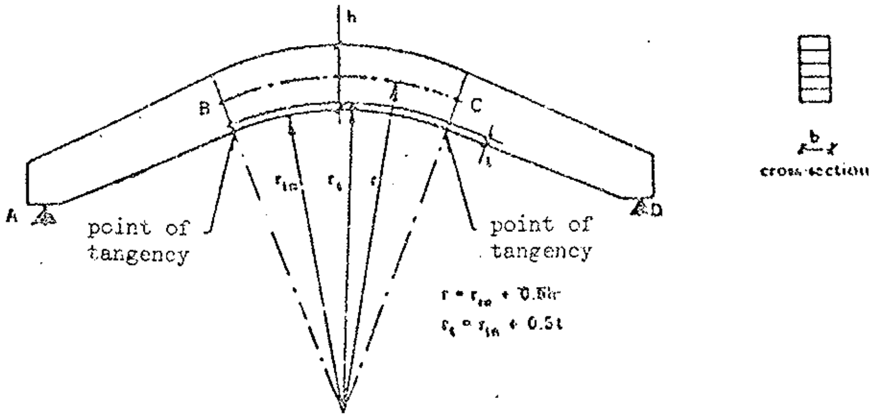

Figure 5.1.12. Curved beam. The shaded area is used by calculation kvol,4

When the bending moments tend to reduce the curvature (increase the radius) the tensile stresses perpendicular to the grain should satisfy the condition

$$\sigma_{t,90,d} \leq k_{vol} k_{dis} f_{t,90,d} \qquad (5.1.12\ a)$$

where k_{vol} takes into account the effect of the size of the loaded volume and k_{dis} of stress distribution on the strength. k_{vol} can be determined under the assumption of a 2-parameter Weibull distribution of $f_{t,90}$.

The volume factor in (5.1.12 a) can determined from

$$k_{vol} = \left(\frac{V_0}{V}\right)^{1/k_{wei}} \qquad (5.1.12\ h)$$

with $V_0 = 0.02\ m^3$ and $k_{wei} = 5$ for most softwood species.

V is the volume of the curved part of the beam (corresponding to the shaded area in figure 5.1.12 a.)

The distribution factor can be determined from

$$k_{dis} = \begin{cases} 1{,}4 \text{ for uniformly distributed load} & (5.1.12\ i) \\ 1{,}0 \text{ for other loading} & (5.1.12\ j) \end{cases}$$

Since

$$k_{dis}, k_{vol} = \frac{1{,}4 \cdot 0{,}02^{0,2}}{V^{0,2}} = \frac{0{,}65}{V^{0,2}}$$

these rules are in principle the same as formula (5.2.2e) in the CIB code.

6. SHEAR DESIGN

The Canadian Code for Engineering Design of Wood (Canadian Standard CSA CAN 3-086-M80) gives as the first code design rules for shear taking into account the influence of the stressed volume and the distribution of the shear stresses.

The Canadian rules are a little complicated and not easily comprehensible. The following simplified presentation has therefore been chosen:

5.1.7 Shear

The total effective design load $Q_{ef,d}$ on the beam should satisfy the following condition

$$Q_{ef,d} \leq k_{vol,v} k_{dis,v} \cdot \left(\frac{4}{3}\, b\, h\, f_{v,d}\right) \qquad (5.1.7\ a)$$

For beams with a volume of 0,1 m^3 or less (5.1.7 a) may be replaced by

$$\tau_d \leq f_{v,d} \qquad (5.1.7\ b)$$

$k_{vol,v}$ takes into account the effect of the size of the stressed volume, and $k_{dis,v}$ the effect of the distribution of shear forces. They can be determined assuming a 2-parameter Weibull distribution of f_v

For a beam with the volume V, $k_{vol,v}$ can be taken as

$$k_{vol,v} = \left(\frac{V_0}{V}\right)^{1/k_{wei}} \qquad (5.1.7\ c)$$

where V_0 = 0.02 m^3, and k_{wei} = 5 can be assumed for most softwood species.

For a beam with length l and varying shear force $V_d = V_d(x)$, $k_{dist,v}$ can be calculated from

$$k_{dis,v} = \frac{0.5\, Q_{ef,d}}{\left(1/l \int_l V_d^{k_{wei}}\, dx\right)^{1/k_{wei}}} \qquad (5.1.7\ d)$$

Examples on $k_{dis,v}$ are given in figure 5.1.7 b. Further examples are given in Annex 101.

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COLUMN DESIGN METHODS FOR TIMBER ENGINEERING

by

A H Buchanan, K C Johns, B Madsen
University of British Columbia
Canada

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COLUMN DESIGN METHODS FOR TIMBER ENGINEERING

Andrew H. Buchanan[1]

Kenneth C. Johns[2]

Borg Madsen[1]

[1]Department of Civil Engineering, University of British Columbia, Vancouver, B.C. V6T 1W5, (604)228-2637

[2]Faculté des Sciences Appliquées, Université de Sherbrooke, Sherbrooke, Quebec. J1K 2R1, (819)565-4411

ABSTRACT

Combined bending and axial loading is often encountered in lumber and timber members. Existing design methods are based on studies carried out many years ago, and are no longer appropriate because they do not recognize that wood with defects behaves as a non-linear ductile material in compression, and as an elastic brittle material subject to size effects in tension.

This paper summarizes the findings of a comprehensive investigation into the behaviour of lumber subjected to eccentric axial loading carried out at two Canadian universities. The study included analytical modelling, and an extensive experimental program using full size lumber.

The results of the investigation have been used in this paper to propose improved design methods, using design charts and approximate formulae for in-plane behaviour. The discussion is extended to general loading cases and biaxial behaviour. Input information required for the design process is also discussed.

INTRODUCTION

Combined bending and axial loading is encountered in many types of timber structures. The most common are columns, chord and web members of trusses, and wall studs. Others include arches, domes and rigid frame structures.

Current design methods, developed in the 1920's, are not appropriate for modern timber engineering design. Major deficiencies include the following:

- they are mainly based on concentric loading tests on small clear wood specimens,
- they assume that realistic strength properties can be obtained from tests on small clear specimens,
- they assume that a member loaded in combined bending and compression will fail when a limiting elastic compression stress is reached. The possibility of a tension failure is ignored,
- non-linear stress-strain behaviour in compression is not considered,
- the effect of stressed volume on the strength in tension is not recognized,
- they do not recognize that knots and material variability make it impossible to load a member with perfectly concentric axial loads,

This paper presents improved design methods which overcome the deficiencies listed above. The improved methods, based on a comprehensive experimental and analytical research program, are much more appropriate for engineering use.

Previous Studies

The strength of lumber subjected to combined bending and axial loading has not received much attention. Most conventional design methods were derived using the results of tests on small clear specimens. Existing design methods will be discussed later in this paper.

In recent years it has been recognized that commercial quality lumber and timber behave quite differently from small clear wood specimens in many respects. This has led to the development of "in-grade testing", where the weaker portion of large samples of full size lumber of certain sizes, grades and species are tested to failure. Extensive in-grade testing has been carried out in bending and in axial tension throughout Canada (Madsen and Nielsen 1978a,b), and the results of these tests are being incorporated into design codes.

Very little in-grade testing has been performed on members subjected to combined bending and axial loading. A few tests under combined axial tension and bending have been described by Senft and Suddarth (1970) and Senft (1973). Larsen and Theilgaard (1979) tested members with combined axial loads and transverse loads to verify their theory for beam-column behaviour. Malhotra (1982) tested columns with small end eccentricities, and compared the results to several older formulae. Zahn (1982) tested short lengths of lumber with eccentric compression loads to obtain moment-curvature information for a simulation model. None of the above studies have led to specific proposals for improved design methods.

Presentation

The objectives of this paper are to summarize the results of a major experimental and analytical investigation into the strength of lumber under combined bending and axial loading, and to use those results to propose realistic design methods.

The paper begins by describing the research program. The behaviour of eccentrically loaded columns is reviewed, followed by a summary of existing design methods. Proposed new design methods are introduced, using both design charts and approximate formulae to consider in-plane behaviour. The discussion is then extended to more general loading cases and biaxial behaviour. Finally a description of the input information required for these design methods is given.

RESEARCH PROGRAM

Description

The proposals made in this paper are the outcome of a comprehensive research program conducted at two Canadian universities over a period of several years.

In the early 1970's the concept of in-grade testing was developed by Madsen at the University of British Columbia, and considerable testing was carried out on lumber loaded in bending, axial tension, axial compression, and shear. As an extension to this in-grade testing program, Johns of the University of Sherbrooke collaborated with Madsen in 1980, and they initiated a project for in-grade testing of lumber under eccentric axial compression loading.

When the first phase of testing was completed, Buchanan developed the basic concepts of a material strength model which was able to explain the behaviour observed in the experimental results. Results of the research program to that stage were reported by Johns and Buchanan (1982).

The research program was then expanded to include further in-grade testing of lumber under eccentric axial compression and tension, and two strength models were developed, calibrated and verified. All of this research was carried out on a co-operative basis with Madsen and Buchanan working at the University of British Columbia, with Johns and Bleau working at the University of Sherbrooke. Testing was carried out at both universities and the test results were shared. Full descriptions of the research results are given in theses by Bleau (1984) and Buchanan (1984a).

Strength Model

In the analytical parts of this research program, two similar strength models have been developed to explain the observed behaviour. Both models are based on the stress-strain relationship for wood shown in figure 1, using the following basic concepts.

1. Strength of members subjected to combined bending and axial loading can be predicted from the behaviour of similar members loaded in axial tension and axial compression.
2. Plane sections are assumed to remain plane.
3. Timber stressed in tension behaves in a linear elastic manner until brittle fracture occurs at a limiting tension stress.

4. Timber stressed in compression behaves in a non-linear ductile manner. The stress-strain curve has a long falling branch after maximum compression stress has been exceeded.
5. For both axial tension and axial compression members, maximum attainable stresses decrease as member length or member cross section is increased. These effects are referred to as "length effects" and "depth effects", respectively.
6. The maximum attainable tension stress also depends on the proportion of the cross section subjected to tension. This phenomenon results in the modulus of rupture being considerably higher than the axial tension strength.
7. The models do not consider torsional or out-of-plane deformations, duration of load effects, or the possibility of shear failures. Some of these factors will be discussed separately, later in this paper.

The main difference between the two models is that when considering instability effects, the model described by Buchanan (1984b) uses a step-by-step procedure to develop column deflection curves, whereas that described by Bleau (1984) uses energy considerations in a finite element formulation. Both models make good predictions of the experimental results.

For the rest of this paper, output from Buchanan's model (referred to as the "strength model") will be used as the basis for design proposals, although similar proposals could have been made using Bleau's model or experimental results directly.

RESEARCH RESULTS

Experimental Results

This section describes results of testing carried out on 38x89 mm (nominal 2x4 inch) spruce-pine-fir (SPF) lumber. The lumber was purchased as "Number Two and Better" grade in 4.9 m (16 ft) lengths. Average moisture content at the time of testing was 10%. Similar results were obtained for 38x140 mm (nominal 2x6 inch) material.

Eccentric axial compression tests were carried out in a special testing machine which provided equal end eccentricities about the strong axis, with lateral restraint to prevent any twisting or out-of-plane buckling. Five lengths of lumber were tested in this way with up to five eccentricities at each length.

Test results for eccentric axial loading of the shortest length, 450 mm (18 in) long, are shown on figure 2. Each point represents the axial load and mid-span moment at failure, for one board. All failures at this short length were material failures. It can be seen that the strength of this lumber varies considerably, particularly in tension.

For each end eccentricity, the mean, 5th and 95th percentile values have been determined, and these have been plotted on figure 3. Additional points on the axes of figure 3 show the results of axial tension tests and compression tests (vertical axis) and two bending tests (horizontal axis).

Comparison with Model

The curves on figure 3 are output from the strength model. Each curve originates from axial tension and axial compression test results on the vertical axis, these two points having been used as input. The solid curve in figure 3 is output from the model, calibrated to the mean test results. The curves at the 5th and 95th percentiles have been obtained using the same parameters and the resulting fit is seen to be reasonably good.

The curve at the 5th percentile illustrates that the bending moment capacity can be increased by application of a moderate axial compression load. This behaviour, which is very similar to reinforced concrete, occurs because weak lumber loaded in bending fails in the tension zone. An axial compression force suppresses this tension failure and hence increases bending strength. This behaviour is not recognized by design codes which conservatively predict decreasing bending moment capacity with any increase in compression loading, as shown by the straight dotted line in figure 3.

Figure 4 is an interaction diagram of axial load and end moment for members 38x89 mm, 1.8 m long (nominal 2x4 in, 6' long). The points represent test results at the 5th percentile, mean and 95th percentile levels, obtained from a large number of individual test results similar to the top half of figure 2. The lines in figure 4 are predictions from the strength model, including instability effects. The model is seen to give a good prediction of strength, somewhat conservative at the 95th percentile level. Similar results have been obtained from other lengths. Increasing member length leads to decreasing strength, mainly due to instability resulting from increased slenderness, but also due to size effects which affect material strength properties.

The model has only been calibrated and verified using two cross section sizes of lumber, but it is believed to be applicable to timber members of any size.

COLUMN BEHAVIOUR

Concentric Loading

The mode of failure of a concentrically loaded column depends on its slenderness. The strength of a short column is governed by the compression strength of the material. A slender column fails in an instability mode, at a critical load which is proportional to the modulus of elasticity and inversely proportional to the square of the length.

For a column of intermediate length there is a transition between these two types of failure, as shown in figure 5, in which case the load capacity depends on both the strength and stiffness of the material. It has already been pointed out that for a material such as lumber, it is impossible to apply concentric loads with no eccentricities.

Eccentric Loading

The behaviour of an eccentrically loaded compression member will be reviewed in this section, Consider the member with equal end eccentricities as shown inset in figure 6, as the axial load P is increased to failure. If the end eccentricity is a distance e, the moment at the ends of the member will always be P times e. The moment at mid-span will be $P(e+\Delta)$ where Δ is the mid-span deflection.

Figure 6(a) is an interaction diagram of axial load vs. moment. The outer curved line is the ultimate interaction diagram representing material failure. An interaction diagram of this shape is obtained from experimental and analytical studies on lumber at the weak end of the strength distribution, as discussed earlier in the paper. As an example of typical loading, line 0-A shows the load path for axial load and end moment as the axial load P is increased. The corresponding load path for mid-span moment is shown by the curved line 0-B. The horizontal distance between lines 0-A and 0-B represents the amount by which the initial moment, P times e, has been magnified to $P(e+\Delta)$. In this case the member fails at an axial load P_1 when the mid-span load path 0-B intersects the material strength interaction diagram at point B. This is described as a material failure.

If the same member is loaded with a smaller eccentricity, the load path for end moments is shown by line 0-C, and the load path for mid-span moments by line 0-D. In this case an instability failure occurs when the axial load reaches a maximum value P_2. The mid-span moment at failure is shown by point D, which is inside the material strength curve. If the member were loaded with a system under load control (for example, gravity loads) to load P_2, deformations would increase rapidly and a material failure would follow immediately. If the member were loaded under conditions of controlled displacement, the load path shown by the extension of the line 0-D could be followed to eventual material failure at point E.

If this process is repeated many times for the same member using a full range of eccentricities, figure 6(b) can be produced. The solid line is the same ultimate interaction diagram. The dotted line P_u-D-B-M_u is the locus of points such as B and D in figure 6(a), representing combinations of axial load and mid-span moment (or magnified moment) just causing failure. For low axial loads this line co-incides with the ultimate interaction diagram, indicating material failures. For higher axial loads instability failures occur. The chain-dotted line P_u-C-A-M_u is the locus of points such as A and C in figure 6(a), representing combinations of axial load and end moment (or unmagnified moment) just causing failure. For any axial load the horizontal distance between the two dotted lines represents the moment magnification due to member deformations at failure.

Three-Dimensional Presentation

The concentric column curve of figure 5 can be related to the interaction diagrams of figure 6(b) in three-dimensional sketch form. Figure 7 shows the relationship between axial load, moment and slenderness.

The curve on the axial load - slenderness plane is the column curve for concentric loading. The curve on the axial load - moment plane is the ultimate interaction diagram for a cross section; the outer solid curve in figure 6. Curves parallel to this plane are interaction diagrams such as the curve P_u-D-B-M_u in figure 6(b), for increasingly slender columns.

EXISTING DESIGN METHODS

Existing design methods for combined bending and axial loading will be discussed below with reference to three major North American design codes.

Canadian Code

The current Canadian Code (CSA 1980) specifies allowable design stresses which are intended to provide a certain factor of safety against failure under maximum anticipated working loads.

For concentrically loaded columns the code uses the three ranges of slenderness illustrated in figure 5. For short columns the code specifies an allowable stress derived from testing of small clear compression specimens. For long columns the allowable stress is calculated from the traditional Euler formula. For columns of intermediate length the code specifies a fourth power parabolic transition curve between short and long column behaviour. This transition curve is based on studies in the United States in the 1920's (Newlin and Trayer 1925; Newlin and Gahagan 1930).

These code provisions for concentric loading are based on correct principles, but improvements could be made in two areas. Firstly, in-grade testing of full size lumber in axial compression (restrained to prevent buckling), often produces very different design stresses than those obtained from small clear specimen testing. Design stresses should therefore be based on in-grade test results to give more accurate designs. Secondly, the three-part formula for three ranges of slenderness could be replaced by a single continuous formula for all slenderness ratios. The transition curve for intermediate length columns is slightly on the unsafe side of the load capacity predicted by the strength model.

Figure 8 shows a comparison of the strength model prediction, the Code formula, and a simple approximate curve adapted from Neubauer (1973) as

$$[1] \qquad \frac{P_u}{P_a} = \frac{1}{1 + \frac{F_c}{E} \frac{(L/d)^3}{40}}$$

where P_u is the concentric axial load capacity of a slender column, P_a is the axial load capacity of a short column, F_c/E is the ratio of axial compression strength to modulus of elasticity of the material and L/d is the ratio of effective length to corresponding cross sectional dimension of the column. This curve will be referred to later when discussing new design methods.

For combined bending and axial loading the Canadian Code specifies a simple failure criterion based on a linear interaction between the axial load capacity of a concentrically loaded column and the moment capacity in bending alone. The formula is

$$[2] \qquad \frac{P/A}{F_a} + \frac{M/S}{F_b} < 1$$

where P is the axial load, A is the area of cross section, F_a is the allowable axial stress under concentric loading for the particular slenderness ratio, M is the magnified bending moment which includes that due to axial loads, S is the section modulus and F_b is the allowable stress in bending.

Problems with this approach are that the code gives no guidance for calculating the moment resulting from column deflections, and the interaction formula is very conservative for lumber, as shown for short lengths on figure 3. This conservatism arises because the formula ignores the possibilities of ductile yielding in the compression zone or failure in the tension zone, both of which are observed in tests.

NFPA Code

A design code widely used in the United States is that produced by the National Forest Products Association (NFPA 1982), referred to as the NFPA Code. This code is also based on allowable stresses.

The requirements for concentrically loaded columns are essentially the same as in the Canadian Code. For combined bending and axial loads an appendix to the NFPA Code provides a general formula which includes both eccentric axial loads and transverse loads. This formula is based on the work of Newlin (1940) later explained by Wood (1950). Its derivation assumes sinusoidal deflected shape, linear elastic behaviour, and failure at a limiting compression stress with no consideration of failure in the tension zone.

These assumptions which need to be reconsidered in the light of more recent information produce very conservative results compared with those obtained in the current study.

British and several European codes are based on very similar assumptions to the NFPA Code, although the code formulae are quite different.

Ontario Highway Code

A recently produced code is the Ontario Highway Bridge Design Code (OHBDC 1982) referred to as OHBDC. This code is written in a limit states format and its requirements for compression members are quite different from previous codes.

The main change is the absence of a requirement for concentrically loaded columns, recognizing that it is essentially impossible to load a timber compression member with zero end eccentricity. The following formula (the same as used in reinforced concrete design codes) is specified for all compression members

$$[3] \qquad \frac{P}{P_a} + \frac{C_m}{1 - P/P_e}\frac{M}{M_u} < 1$$

where P is the axial load, P_a is the compression capacity of a short column, M is the unmagnified moment and M_u is the moment capacity of a cross section.

The moment M is required to be the actual end moment, or the moment due to a minimum end eccentricity of 0.05 times the cross section dimension whichever is greater. An initial out-of-straighness of 1/500 times the effective length is also required to be considered.

The term $1/(1-P/P_e)$ is a factor representing moment magnification due to column deflections, with P_e being the Euler buckling load. This magnification factor is a good approximation to actual behaviour of a linear elastic member but underestimates moment magnification if any non-linear material behaviour occurs in compression.

The term C_m is a factor to allow for unequal end moments, given by

$$[4] \qquad C_m = 0.6 + 0.4\, M_2/M_1 > 0.4$$

where M_1 and M_2 are the larger and smaller end moments, respectively. The ratio M_1/M_2 is positive if the member is in single curvature, negative for double curvature.

Equation 3 assumes that all failures are material failures, with a linear interaction between axial load and moment; instability failures are only considered indirectly in that the magnification factor can only be used for axial loads less than the Euler buckling load.

The OHBDC code format is a major improvement over previous codes, but it is based on incomplete assumptions which produce very conservative results in some cases and unconservative results in others.

NEW DESIGN METHODS

In this section of the paper several alternative design methods will be presented. We will initially restrict ourselves to in-plane behaviour of eccentrically loaded columns with equal end eccentricities. Extrapolation to other loading cases and biaxial behaviour will follow. Lateral torsional buckling is included under biaxial behaviour. With reference to the three-dimensional presentation of figure 7, all of the methods to be discussed below use axial load-moment interaction diagrams rather than axial load-slenderness curves. This choice has been made because all members will be subjected to some moments (as recognized by the OHBDC code), and this presentation makes it easier to understand what is happening to the members.

All of the design methods proposed in this paper consist of checking procedures. The structural designer first calculates the axial load and bending moment that the proposed member is required to resist, then uses one of the proposed "design methods" to check whether a trial section has the required capacity.

Design Approaches

There are two basic approaches to design of eccentrically loaded compression members, depending on how the moment magnification resulting from member deformations is handled.

Approach 1 considers interaction diagrams for unmagnified moments, such as the curve P_u-C-A-M_u in figure 6(b). In this case the designer is not required to make any estimate of moment magnification.

Approach 2 considers interaction diagrams for magnified moments, such as the curve P_u-D-B-M_u in figure 6(b). In this approach (which is similar in concept to steel and concrete codes), the designer is required to calculate magnified moments as part of the design process.

Within each of these two basic approaches, two design techniques will be considered.

Technique A is based directly on the results of the research program using a series of design charts obtained as output from the strength model.

Technique B uses approximate formulae to describe the results of the research program. These formulae, which are derived from the strength model output, are generally conservative, but may be easier to use in some cases.

Method 1A: Design Charts for Unmagnified Moments

Figure 9 shows a typical design chart which can be used directly for design of eccentrically loaded columns. The outer curve of figure 9 is the ultimate interaction diagram for the material, corresponding to the outer curve in figure 6(b) or the 5th percentile curve in figure 3. The inner curves in figure 9 are interaction diagrams of axial load vs. unmagnified moment, for several values of slenderness ratio.

This chart has been non-dimensionalized so that it can be used for members of different cross sections and different strengths. It is important to recognize that this chart has been constructed for a material with certain strength properties, as shown by the ratios in the upper right corner. Other charts can be produced for different ratios of strength properties.

Before using the chart, the design values of axial load and moment must be non-dimensionalized. The axial compression force P is non-dimensionalized to P/F_cA, where F_c is the code-specified axial compression stress for a concentrically loaded restrained column of the same length and A is the cross section area. Bending moment, M, is non-dimensionalized to M/F_bS where F_b is the code-specified bending stress and S is the section modulus.

The chart can also be used for combined bending and axial tension. The design axial tension force, T, is non-dimensionalized to T/F_cA.

Method 1B: Approximate Formulae for Unmagnified Moments

Re-examination of the design chart in figure 8 suggests that many of the curves can be approximated by straight lines, leading to the simple design formula

$$[5] \qquad \frac{P}{P_u} + \frac{M}{M_u} < 1$$

To calculate the concentrically loaded column capacity, P_u, the column formula in the existing code could be used, although the simpler expression given in equation 1 is preferable.

Figure 10 compares this formula with the design chart. The solid curves are the same as shown in figure 9. The dotted lines have been produced by equation 5. The formula is a reasonable fit for all cases except short columns, $L/d < 15$, in which case it becomes very conservative. Equation 5 is an empirical expression which just happens to work because the shape of the mid-span moment interaction diagram, reduced by the moment magnification factors, produces an interaction diagram for end moments consisting of nearly straight lines. It is a coincidence that any designers who have been incorrectly using the current code requirements without making allowance for second order deflections, have been producing accurate designs according to equation 5.

The few unsafe regions visible on figure 9 could be removed by replacing P_u with $0.9P_u$ in equation 5, but this has not been considered necessary.

For combined axial tension and bending, the lower half of figure 9 can be conservatively approximated by the dotted straight line shown on figure 10, which is also given by equation 5, replacing P with T to produce

[6] $$\frac{T}{T_u} + \frac{M}{M_u} < 1$$

Method 2A: Design Charts for Magnified Moments

This method is very similar to method 1A, the major difference being that the design charts, such as shown in figure 11, represent combinations of axial load and mid-span (or magnified) moment at failure.

To use this chart the designer must calculate the maximum moment in the member, taking second order deformations into account. The traditional method of doing this has been to magnify the initial moment by a magnification factor, F, given by

$$[7] \qquad F = \frac{1}{1 - P/P_e}$$

where P_e is the Euler buckling load for the member. However, both the experimental and analytical results of the present study indicate that moment magnification at failure is considerably greater than given by equation 7, because of non-linear behaviour of wood in compression. To avoid unsafe designs, more accuracy can be obtained with an empirical modification to equation 7

$$[8] \qquad F = \frac{1 + P/P_e}{1 - P/P_e}$$

which gives amplification nearly the same as equation 7 at low axial loads, but twice that value as P approaches P_e. These two expressions are compared in figure 12 for a typical case with slenderness ratio L/d = 20. The outer curve P_a-M_u is the ultimate interaction diagram for material strength. The curve P_u-B-M_u represents combinations of axial load and unmagnified moment, P times e, at failure, and the curve P_u-D-M_u represents corresponding combinations of axial load and magnified moment, $P_a(e+\Delta)$, at failure. The two dotted curves show magnified moments given by equations 7 and 8 above. The curve given by equation 8 is seen to give the better approximation to curve P_u-D-M_u.

To use the design chart of figure 11, the designer takes the design values of axial load and moment, magnifies the moment with equation 8, non-dimensionalizes both axial load and moment, then checks the chart to see if the selected member is satisfactory. For combined axial tension and bending the design chart can be used directly, as before. The curves in the tension region of figures 9 and 11 are identical.

As before, the chart has been non-dimensionalized so that it can be used for any cross section size, but again several charts will be necessary to cover a suitable range of material strengths ratios.

It may be that some designers have access to structural analysis computer programs which include deflections due to second order effects. In this case an expression for F is not required, but the designer should be aware that moment magnification in a wood column at failure may be considerably greater than that predicted by linear elastic analysis.

Method 2B: Approximate Formulae for Magnified Moments

The curves of figure 11 can be approximated by straight lines, as shown in figure 13. The sloping lines all originate at the point marked B, and the shaded areas indicate the extent to which the lines are conservative.

A design formula using these lines is given by

$$[9] \qquad \frac{P}{P_u} + \frac{FM}{BM_u} \leq 1, \qquad\qquad FM \leq M_u$$

where F is the magnification factor given by equation 8. B is a number which reflects the shape of the interaction diagrams. A semi-empirical expression for B is

$$[10] \qquad B = 1.35\ F_c/F_t, \qquad B > 1$$

Equation 9 can be used directly, given design values of P and M, to check whether a selected member has sufficient strength.

Comparison of Proposed Design Methods

A comparison of the four methods described above will be made here. The two basic approaches (unmagnified vs. magnified moments) will be compared first, followed by a comparison of the two techniques (charts vs. formulae).

The main advantage of using unmagnified moments (Approach 1) is that there is no need to calculate a magnification factor, as the effects of magnification are included in the design charts or formulae. This also eliminates the problem of accidentally omitting or under-estimating the magnification factor. Approach 1, used with design charts, is an "exact" method, to the extent that the strength model is accurate in predicting actual behaviour. The three other proposals involve empirical approximations to output from the strength model.

Design based on magnified moments (Approach 2) has several advantages; the designer is given a better understanding of real column behaviour at ultimate loads, and this approach is similar to steel and concrete code approaches. A secondary advantage of making the magnification factor a separate item in the calculations is that it can be easily modified, if further research suggests that this is necessary for long duration loads or other effects. The major disadvantage of the magnified moment approach is that an approximate empirical expression must be used to calculate the magnification factor. A secondary disadvantage is the difficulty in verifying this approach experimentally, because it is difficult to measure the mid-span failure moment accurately.

When comparing the design chart technique with the approximate formulae, the formulae are less accurate, because they are very conservative approximations to the charts in some cases. However, the formulae are much easier to incorporate into design codes, and they can be more easily altered to allow for lateral torsional buckling.

The best solution may be a two-level code format, where the code specifies approximate equations, but allows the alternative use of

> "a rational design method which includes the effects of second order deflections, non-linear stress-strain behaviour in

compression and size effects resulting from brittle fracture in tension".

The design charts could then be included in a commentary or design manual, and could be used for large volume applications or other special situations.

For the charts to be used accurately, it may be necessary to produce quite a number of them, to cover all possible types of material. The actual number depends on the range of materials to be included and on the degree of accuracy required. The number could be 20 or more.

MORE GENERAL LOADING CASES

The presentation of proposed design methods has so far been restricted to in-plane behaviour of eccentrically loaded members with equal end eccentricities. In this part of the paper we will consider minimum eccentricities and the effects of unequal end moments and transverse loads on in-plane behaviour. We will then extend the presentation to biaxial behaviour.

Minimum Eccentricity

It is impossible to load a timber member with perfectly concentric axial loads. Wood is a material with considerable variability, and any variation of modulus of elasticity within a cross section will introduce bending into the member. Similarly, any out-of-square ends or small misalignment of connections will introduce bending. Another source of bending is from initial out-of-straightness about either principal axis ("crook" or "bow" in the board).

As described above, the OHBDC code specifies a minimum end eccentricity of 0.05 times the relevant cross section dimension, and an initial out-of-straightness of 1/500 times the effective length. These values have not been investigated here, but they appear to be reasonable, so it is recommended that they be included in any new design method, to produce minimum moments about both principal axes

Unequal End Moments and Transverse Loads

No experimental or analytical studies have been carried out on timber columns with unequal end eccentricities. However, many studies have been carried out on steel and concrete, leading to wide acceptance of the equivalent moment method which is believed to be suitable for timber, and which has been included in the OHBDC code.

For unequal end moments, the member can be designed for an equivalent uniform moment given by $C_m M_1$ where C_m is defined in equation 4 and M_1 is the larger end moment. When transverse loads also occur, the equivalent moment is given by $C_m M_1 + M_c$ where M_c is the bending moment at mid-height caused by the transverse loads. The equivalent moment can be used directly in either the design charts or the formulae. If end eccentricities are not equal, the load capacity of the member may be governed by the strength of the cross section at one end. Using design charts, the design combination of axial load P and maximum end moment M_1 should be inside the curve for $L/d = 0$ on figure 9 or figure 11. If formulae are being used, an approximation to this curve is given by

$$[11] \qquad \frac{P}{P_a} + \frac{M_1}{BM_u} < 1, \qquad\qquad M_1 < M_u$$

where all the terms are as defined previously.

There are many possible loading cases, some of which may be difficult to analyse, although no more so than for other materials. A considerable amount of engineering judgement will often be necessary, and a more accurate second order analysis may sometimes be appropriate.

BIAXIAL BEHAVIOUR

Most structural members are not constrained to deform in one plane. Even if a member has applied moments about one axis, it may fail due to buckling about the other axis, or lateral torsional buckling. Very few investigations have been carried out on timber or lumber subjected to combined axial loads and biaxial bending. The suggestions in this section have been made by extending in-plane behaviour to the more general case, using methods previously developed for steel. In the following discussion it is assumed that the x-axis is the strong axis for any cross section.

Loading Categories

In order to keep the design process as simple as possible, and accepting that much more study is required in this area, we suggest that in-plane design methods can be used for certain categories of columns subjected to biaxial loading. Obviously, in-plane design methods can be used for members that are constrained to deform in one plane. Examples are wall studs or top chords of trusses with continuous plywood sheathing or closely spaced bracing members.

For members loaded in bending about the weak axis, with no major bending about the strong axis, in-plane design methods can again be used because such members will only fail in the weak direction.

The same argument can be extended to members with only nominal moments about both axes. For example, web members of pinned-jointed trusses may have nominal moments (due to end eccentricity and out-of-straightness) about both axes, but it appears reasonable to design such members for in-plane behaviour in only the more slender direction.

The final category is for members which have significant moments about the strong axis, and which are free to deform in any direction. Examples are web members of rigid-jointed trusses, or top chords of trusses which have no plywood sheathing. In these cases lateral torsional buckling must be considered, together with bending about the weak axis due to accidental or applied moments.

Interaction Between x-axis and y-axis Bending

To use the results of an in-plane investigation for predicting biaxial effects, it becomes necessary to know the nature of the interaction between x-axis and y-axis bending strengths, for both short and long columns, with varying amounts of axial load.

We first make the assumption that this interaction is linear, as shown in figure 14. Non-linear interaction will be considered later. The linear interaction assumption which is probably conservative, is made for both short columns and long columns, as is often done in steel and concrete codes [8]. It is described by

$$[12] \qquad \frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} < 1$$

where M_x and M_y are the applied moments and M_{ux} and M_{uy} are the moment capacities for the particular level of axial load, all for the respective axes.

Design for Biaxial Loading using Formulae

The design formulae previously presented can be easily modified to incorporate biaxial behaviour, using the linear interaction assumption. The formula for unmagnified moments (Method 1B) equation 5 becomes

$$[13] \qquad \frac{P}{P_u} + \frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} < 1$$

where M_x and M_y are the unmagnified moments about the x and y axes. The term P_u is the concentric axial load capacity of the member, considering buckling about the most slender axis.

M_{ux} and M_{uy} are the moment capacities for bending about the respective axes, with no axial load.

Using the formula for magnified moments (Method 2B) equation 9 becomes

$$[14] \qquad \frac{P}{P_u} + \frac{F_x M_x}{BM_{ux}} + \frac{F_y M_y}{BM_{uy}} < 1, \qquad \frac{F_x M_x}{M_{ux}} + \frac{F_y M_y}{M_{uy}} < 1$$

where the magnification factor is given by

$$[15] \qquad F_x = \frac{1 + P/P_{ex}}{1 - P/P_{ex}}$$

where P_{ex} is the Euler buckling load considering slenderness about the x-axis. The term F_y is obtained in the same way, considering the y-axis. All other terms are as described in the discussion of equations 9 and 13.

If end moments are unequal, a strength check should be made at the ends of the member, using

$$[16] \qquad \frac{P}{P_a} + \frac{M_{1x}}{BM_{ux}} + \frac{M_{1y}}{BM_{uy}} < 1, \qquad \frac{M_{1x}}{M_{ux}} + \frac{M_{1y}}{M_{uy}} < 1$$

where P_a is the axial load capacity of a short column, and M_{1x} and M_{1y} are the respective maximum end moments.

Design for Biaxial Loading Using Charts

To use either of the design chart methods for biaxial loading, it becomes necessary to use the chart twice, once for each principal direction, then to check that the biaxial condition is safe, using equation 12. In is case the term M_{ux} in equation 12 is the moment capacity from the chart, as a function of the axial load and the slenderness ratio in the x-direction. M_{uy} is the equivalent value in the y-direction.

Once again, if unequal end moments or transverse loads exist, M_x and M_y can be modified as described previously, and a strength check at the ends can be made using the charts for $L/d = 0$ and equation 12.

Lateral Torsional Buckling

As mentioned previously, when an unrestrained beam or beam-column is loaded in bending about its strong axis, it may fail by lateral torsional buckling before the in-plane bending strength is achieved.

This only occurs with cross sections that are significantly stiffer about one axis than the other. The reduction in bending moment capacity due to lateral torsional buckling can be calculated using the Canadian Code requirements for glulam members. Those requirements are based on the work of Hooley and Madsen (1964). This check should be made for all unrestrained members with bending about the strong axis.

However, the reduced bending moment capacity cannot simply be plugged into the proposed design methods because the shape of the interaction diagram between axial load and bending moment will change when the bending moment capacity is governed by lateral buckling.

In this case, any axial load will reduce the mid-span moment capacity, producing quite different interaction diagrams from those illustrated earlier in the paper.

To partially overcome this problem, we can use results obtained for steel members, where the following interaction formula was found to be "quite satisfactory in both the elastic and inelastic ranges" (Johnston 1976).

[17] $$\frac{P}{P_{uy}} + \frac{F_x M_x}{M_{uxt}} < 1$$

where P_{uy} is the axial load capacity considering buckling about the y-axis, M_{uxt} is the bending moment capacity about the x-axis, reduced to allow for lateral torsional buckling and F_x is as given by equation 15. This formula implies a linear interaction between axial load and magnified bending moment.

Using design methods for magnified moments, this behaviour can be easily introduced into equation 14 by simply replacing BM_{ux} with the reduced moment capacity. If design charts are being used, equation 17 can be used instead of figure 11 for the x direction.

It is much more difficult to incorporate this complication into either of the design methods using unmagnified moments, without further study.

Non-Linear Interaction

The interaction between x-axis and y-axis bending strength has received considerable attention for steel members (Chen and Atsuta 1976) where it is recognized that the linear interaction formula (equation 12) is very conservative.

A simple non-linear interaction formula proposed for steel codes is

$$[18] \qquad \left(\frac{M_x}{M_{ux}}\right)^{\alpha} + \left(\frac{M_y}{M_{uy}}\right)^{\alpha} < 1$$

where α is an exponent describing the shape of the interaction diagram. For steel the value of α varies from 1.4 to 2 or more, depending on the level of axial load and the shape of the cross section.

This approach has been investigated for timber columns, as described in the Appendix. A suggested value of α, for both short and long columns, can be given by

$$[19] \qquad \alpha = 1.3 + \frac{P}{2P_a}$$

where P is the axial load and P_a is the axial load capacity of a short column.

To use the non-linear interaction formula with the design chart methods, it is a simple matter to use equation 18 in place of equation 12.

With the approximate formulae methods, an extra calculation must be made before using equation 18. For unmagnified moments, equation 5 must be re-arranged to give the moment capacities associated with axial load P, for both the x and y directions.

$$[20] \qquad M_x = M_{ux}\,(1 - P/P_{ux}), \quad M_y = M_{uy}\,(1 - P/P_{uy})$$

Similarly for magnified moments, equation 9 must be rearranged to give

$$[21] \qquad M_x = M_{ux}\,\frac{B}{F_x}\,(1 - P/P_{ux}), \quad M_y = M_{uy}\,\frac{B}{F_y}\,(1 - P/P_{uy})$$

and in each case the non-linear interaction given by equation 18 can be checked.

The use of a non-linear interaction formula such as equation 18 could lead to more efficient designs in cases where biaxial bending is combined with axial loading.

INPUT INFORMATION

Appropriate input information is essential if any of the proposed new design methods are to lead to efficient designs. Four material properties are required for the member being considered:

1. Bending strength
2. Axial compression strength (as a short column)
3. Axial tension strength
4. Modulus of elasticity

Each of these properties should be derived from in-grade testing of members of the same size, length, species, and grade as the member being designed. They should all be lower 5th percentile values, modified as necessary with safety factors or resistance factors.

In-grade test results are available for most sizes, species and grades of Canadian lumber in bending and in axial tension (Madsen and Nielsen (1980a,b). Compression results are expected to be available in the near future.

For larger beam and timber sizes, only limted bending strength results are available. Until further testing is carried out (which will be difficult for members of large cross sections, particularly in tension), strength properties will have to be extrapolated from smaller members.

Length Effects

In-grade testing is usually carried out at only one length, for each cross section size. Recent research indicates that strength is length dependant, and that length effects can be quantified using

$$[22] \qquad \frac{x_1}{x_2} = \left(\frac{L_2}{L_1}\right)^{1/k_1}$$

where x_1 and x_2 are the strengths of lengths L_1 and L_2, respectively (at any level in the distribution of strength) and k_1 is a length effect parameter. Typical values are $k_1 = 13$ in axial compression and $k_1 = 4$ to 6 in bending and in axial tension, the lower value being for weaker material (Buchanan 1984a). The value of $k_1 = 4$ corresponds to a reduction factor of 0.84 for doubling length.

Design stresses from the code should be corrected for length before using any of the proposed design methods. Length effects will be discussed more fully in a subsequent paper (Madsen and Buchanan 1984).

Moisture and Load Duration Effects

The effect of moisture content should be considered by using appropriate input values. At the low end of the strength distribution of lumber, compression strength is significantly affected by moisture content, whereas bending and tension strengths are not. Any changes in modulus of elasticity tend to be offset by simultaneous changes in cross section dimensions.

This paper has not considered the effects of long duration loading. It may be appropriate to use a conservative value of modulus of elasticity to reduce the possibility of failure by creep buckling. Alternatively some appropriate change could be made to the moment magnification factor. More research is required in this area.

Safety Factors

Determination of safety factors (for an allowable stress code) or resistance factors (for a limit states code) is beyond the scope of this paper. However, this is an important area that is receiving increasing attention.

For a code in a limit states format, it becomes necessary to determine values of resistance factor that produce designs which have an acceptable reliability index (or probability of failure). It may be necessary to have different resistance factors for bending, tension, compression and instability.

For design using design charts, the resistance factors can either be applied to code-specified material properties before design, or they can be incorporated into the design charts when they are prepared.

CONCLUSIONS

Four alternative design methods have been proposed for lumber and timber members subjected to combined bending and axial loading. These methods are based on the results of experimental and analytical studies into the strength of lumber under eccentric axial loading.

The proposed design methods reflect the behaviour of real materials more accurately than existing methods. They offer potential for more efficient and economical design of certain types of timber structures.

This study has identified many areas with potential for further research, the most important being the biaxial behaviour of slender timber members at ultimate loads.

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APPENDIX

INTERACTION BETWEEN X-AXIS AND Y-AXIS BENDING

Short Columns

For a short column of a linear elastic material with no size effects over the cross section, it can be shown that the interaction between x-axis and y-axis bending is the linear relationship given by equation 12.

Timber columns do not fall directly into this category for two reasons:

1. At low axial loads (or in axial tension) load carrying capacity is governed by stresses in the tension zone, where behaviour is linear elastic but strength is affected by size effects. Extreme fibre stresses at failure tend to increase as the highly stressed proportion of the cross section decreases.
2. At high axial loads much of the cross section becomes "plastic" as ductile compression yielding occurs.

The effects of this behaviour on the biaxial strength of a short column have been investigated by calculating axial load - moment interaction diagrams for the actions at several angles through a square cross section, then calculating $M_x - M_y$ interaction diagrams for various levels of axial load. Calculations were made at 15° intervals for typical material having axial tension capacity 0.7 times axial compression capacity, and a stress - strain relationship such as that shown in figure 1.

Figure A1 shows the resulting $M_x - M_y$ interaction diagram, ignoring the effects of cross section size effects. The curve for zero axial load is a straight line between points marked 1.0 on each axis. As the axial load increases to $0.3P_a$, the straight lines move away from the origin, following the pattern shown in figure 3. For higher axial loads, the bending moment capacity decreases and the lines become more and more curved. The curves for high axial loads are very similar to those shown by Chen and Atsuta (1976) for a rectangular steel cross section assuming fully plastic behaviour. Note that the interaction diagrams become curved when non-linear compression behaviour occurs. For lumber which is weak in tension there is no non-linear compression behaviour does not occur until significant axial loads are applied. On the other hand, small clear wood specimens which are strong in tension would experience compression yielding at zero axial load, producing curved interaction diagrams at low axial loads.

Returning to the material that is weak in tension, if size effects over the cross section are introduced, the straight lines for low axial loads become curved as shown in figure A2. Behaviour for high axial loads is not affected; these curves look slightly different because the moment value that they have been normalized to has also changed.

The size effect has been quantified using the following assumptions: The cross section consists of a large number of elements of variable strength. The material is perfectly brittle such that failure of any one element will cause failure of the cross section. The element strengths can be described by a two-parameter Weibull distribution with a shape parameter of $k_3 = 8.0$. This value is typical of the test results for uniaxial behaviour (Buchanan 1984a).

To compare the shapes of the curves in figure A2, they have been non-dimensionalized in figure A3.

Following the work of Chen and Atsuta (1976) the non-linear interaction formula of equation 18 has been fitted to these curves (by measuring the radial distances for the 45° loading case). The resulting values of α have been plotted against axial load in figure A4. The dotted curve shows that a reasonable approximation to this behaviour can be obtained using

$$[A1] \quad \alpha = 1.3 - \frac{P/P_a}{2\ \ell n(P/Pa)}$$

an expression almost identical to that proposed by Chen and Atsuta for steel members.

For cross sections that are not square, the strength model in its present form can only be used to calculate actions about principal axes. However it is expected that the results presented for square sections will be similar for other rectangular sections.

Long Columns

The behaviour of long columns subjected to axial compression and biaxial compression is very complex because buckling may take place about either principal axis, or in torsion, or as a combination of these. In the absence of any studies of this behaviour in timber members, the results of structural steel research will again be used.

Referring to figure A4, Chen and Atsuta found that the dotted curve from equation A1 predicted short column behaviour, but that a straight line was more appropriate for long columns. The straight dotted line on figure A4 follows the trend observed for steel, its formula being

$$[A2] \quad \alpha = 1.3 + \frac{P}{2P_a}$$

This expression is recommended for use until further research is performed. It should be noted that this straight line is very close to the curves, for $P/P_a < 0.5$, so a simple conservative approach would be to use equation A2 for both short and long columns. Note that for axial tension, P should be taken as zero in either of equations A1 or A2.

LIST OF SYMBOLS

A = area of cross section

B = axis intercept for interaction diagram

C_m = equivalent moment factor

d = depth of cross section

E = modulus of elasticity

F = moment magnification factor

F_a = allowable stress in short column

F_b = allowable stress in bending

F_c = axial compression strength as short column

F_t = axial tension strength

k_1 = length effect parameter

L = effective length of member

M = applied moment

M_1 = larger end moment

M_2 = smaller end moment

M_c = mid-span moment from transverse loads

M_u = bending moment capacity

M_{uxt} = moment capacity about x-axis, reduced for torsional buckling

P = applied compression force

P_a = axial compression capacity as short column

P_e = Euler buckling load

P_u = axial compression capacity of slender column

S = section modulus

T = applied tension force

T_u = axial tension capacity

x = strength

α = exponent for non-linear interaction

Subscripts "x" and "y" refer to actions about the x- and y-axes, respectively.

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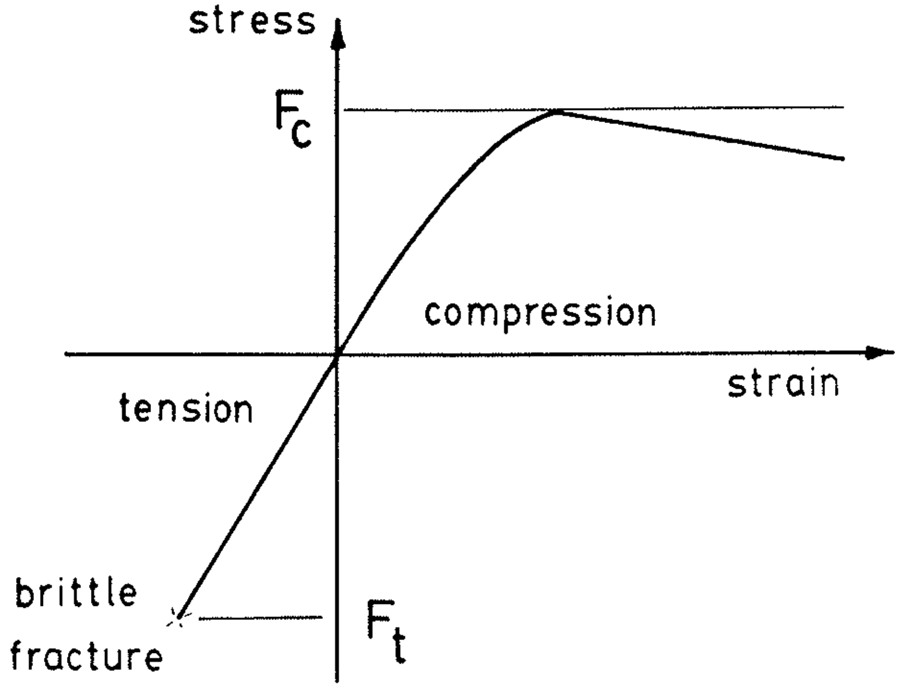
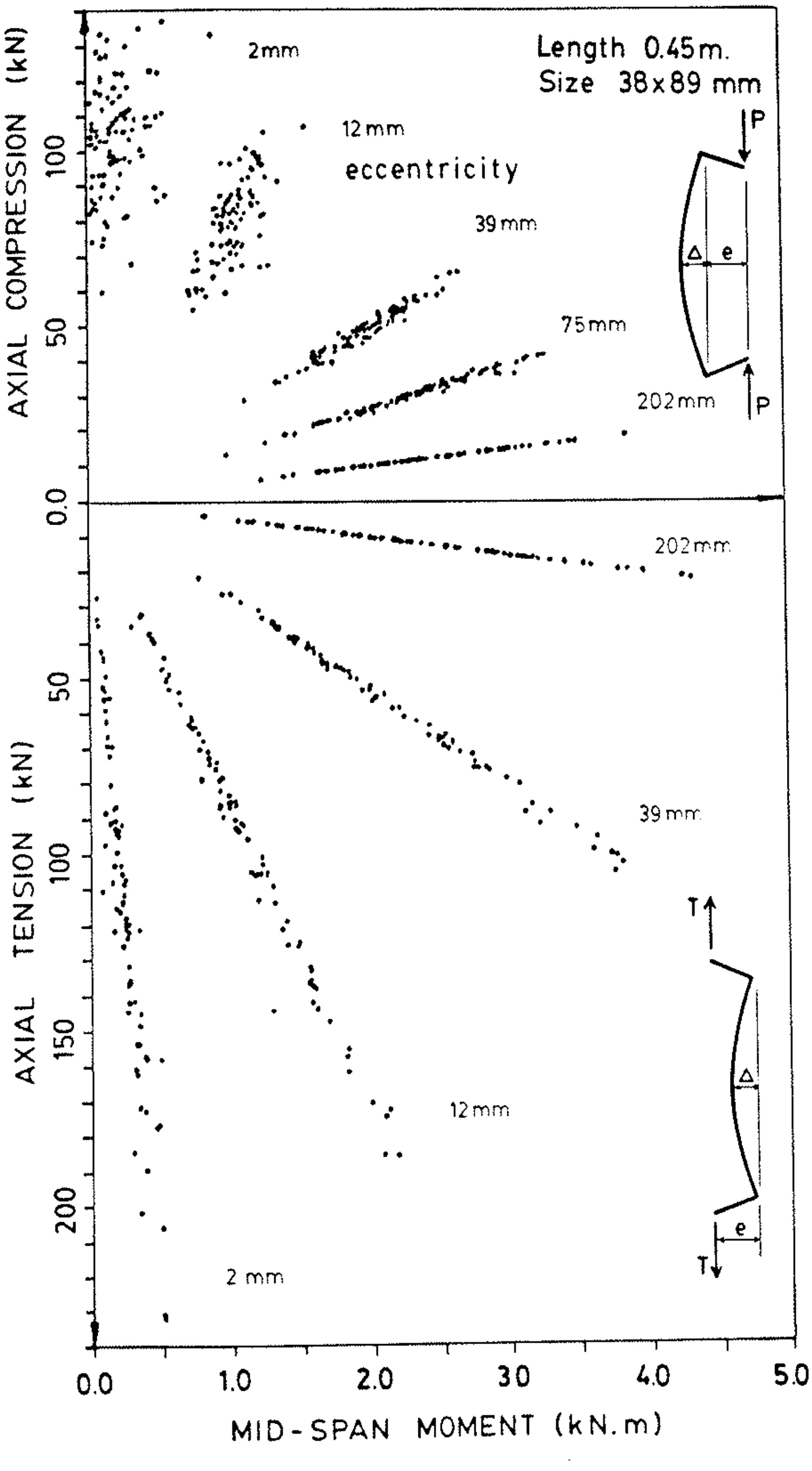
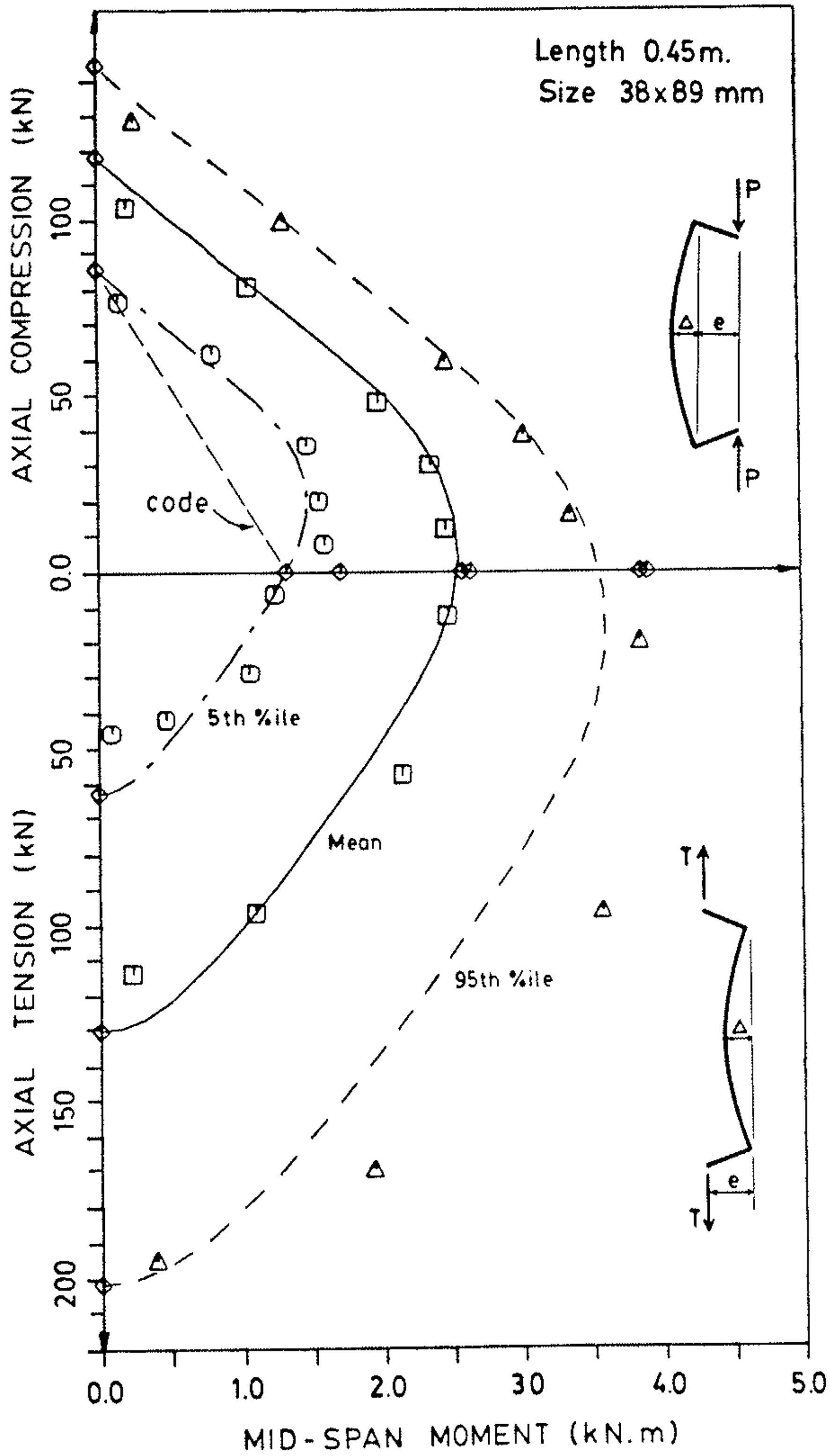
The authors wish to acknowledge the assistance of Mr. Raymond Bleau, and the financial support of the Natural Sciences and Engineering Research Council, the National Lumber Grades Authority, and the Izaak Walton Killam Trust.



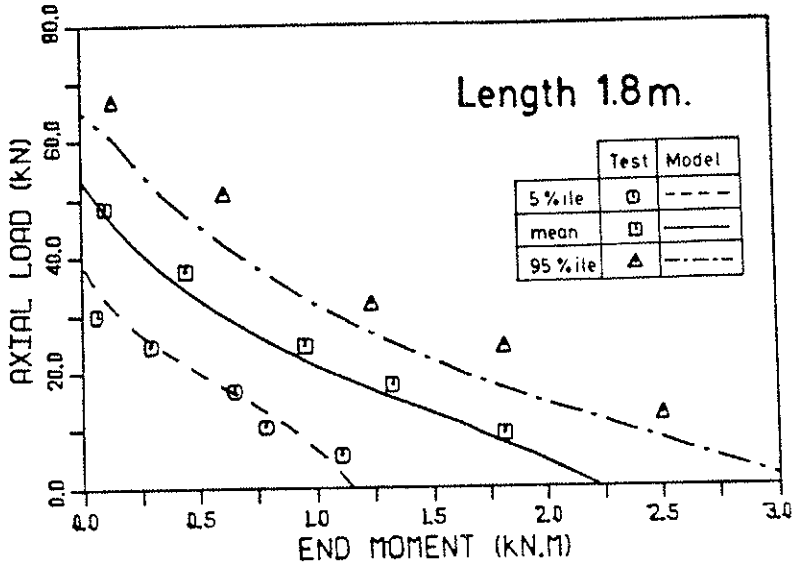
Buchanan, Johns & Madsen   Figure 1



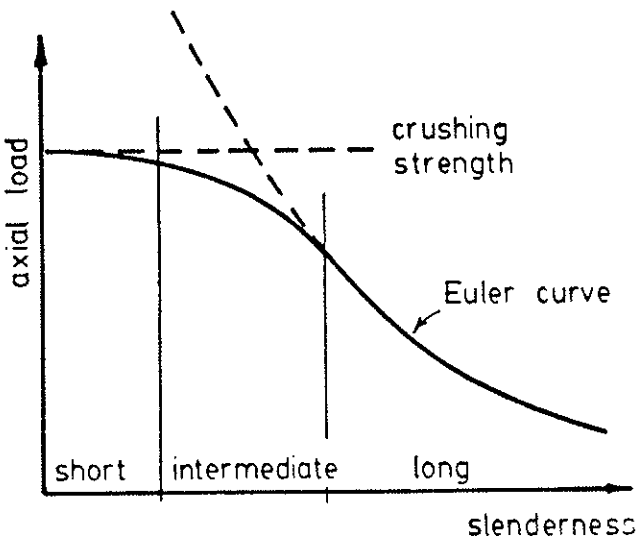
Buchanan, Johns & Madsen   Figure 2



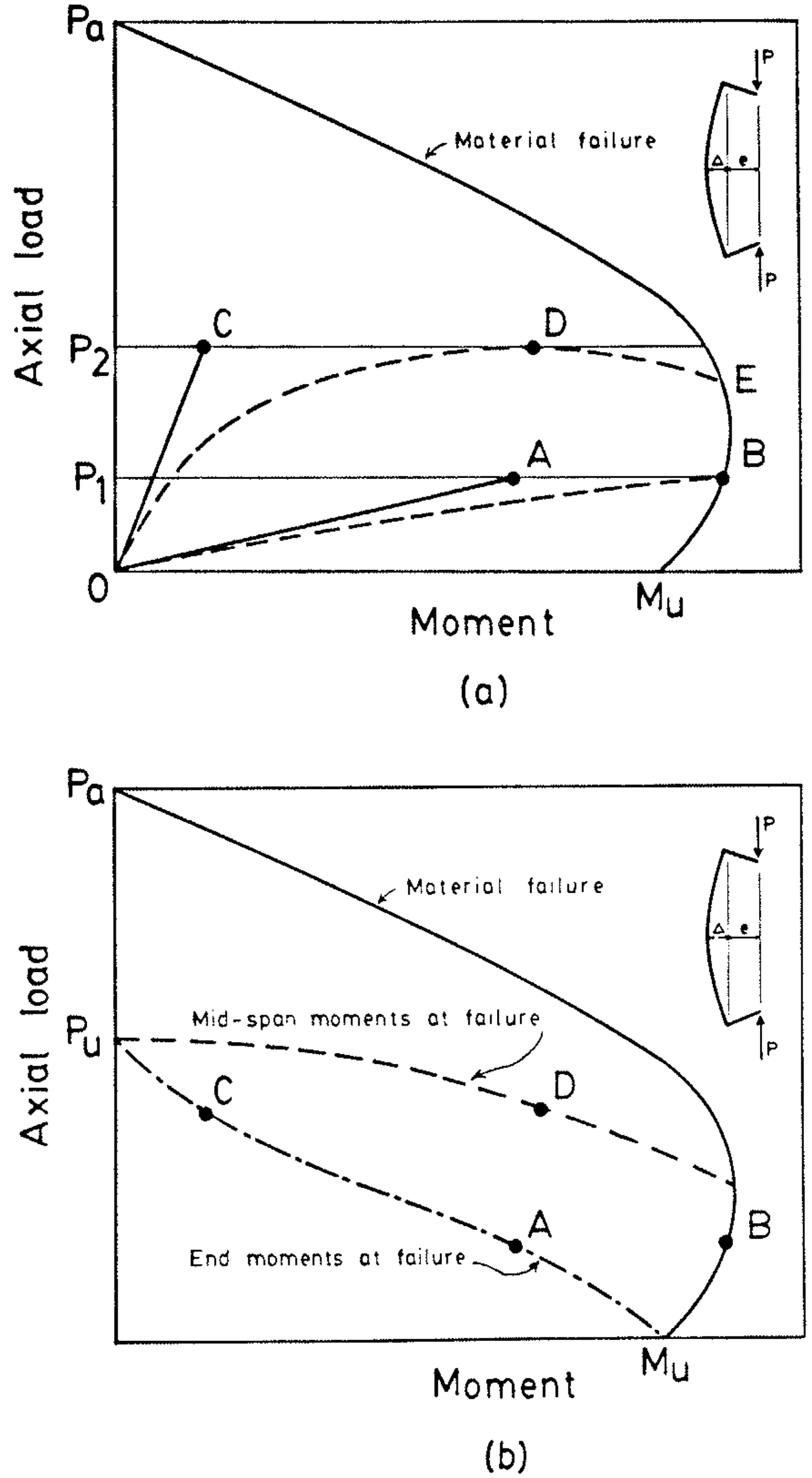
Buchanan, Johns & Madsen Figure 3



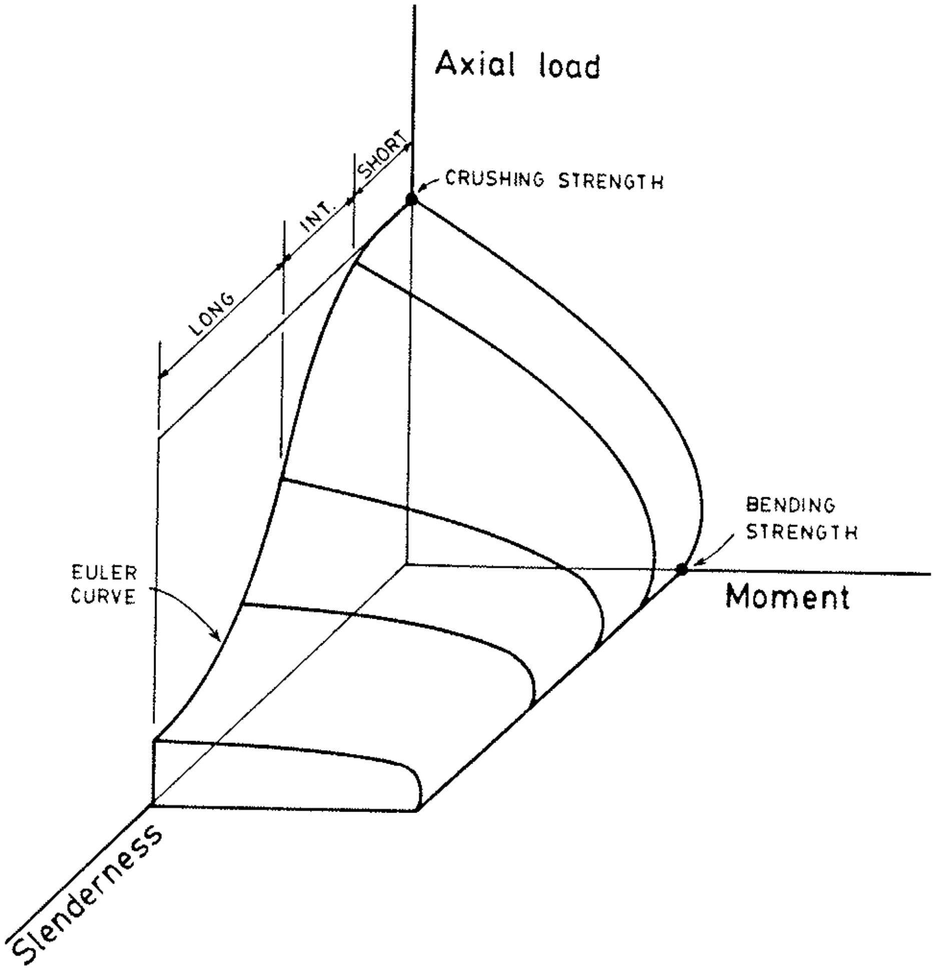
Buchanan, Johns & Madsen   Figure 4



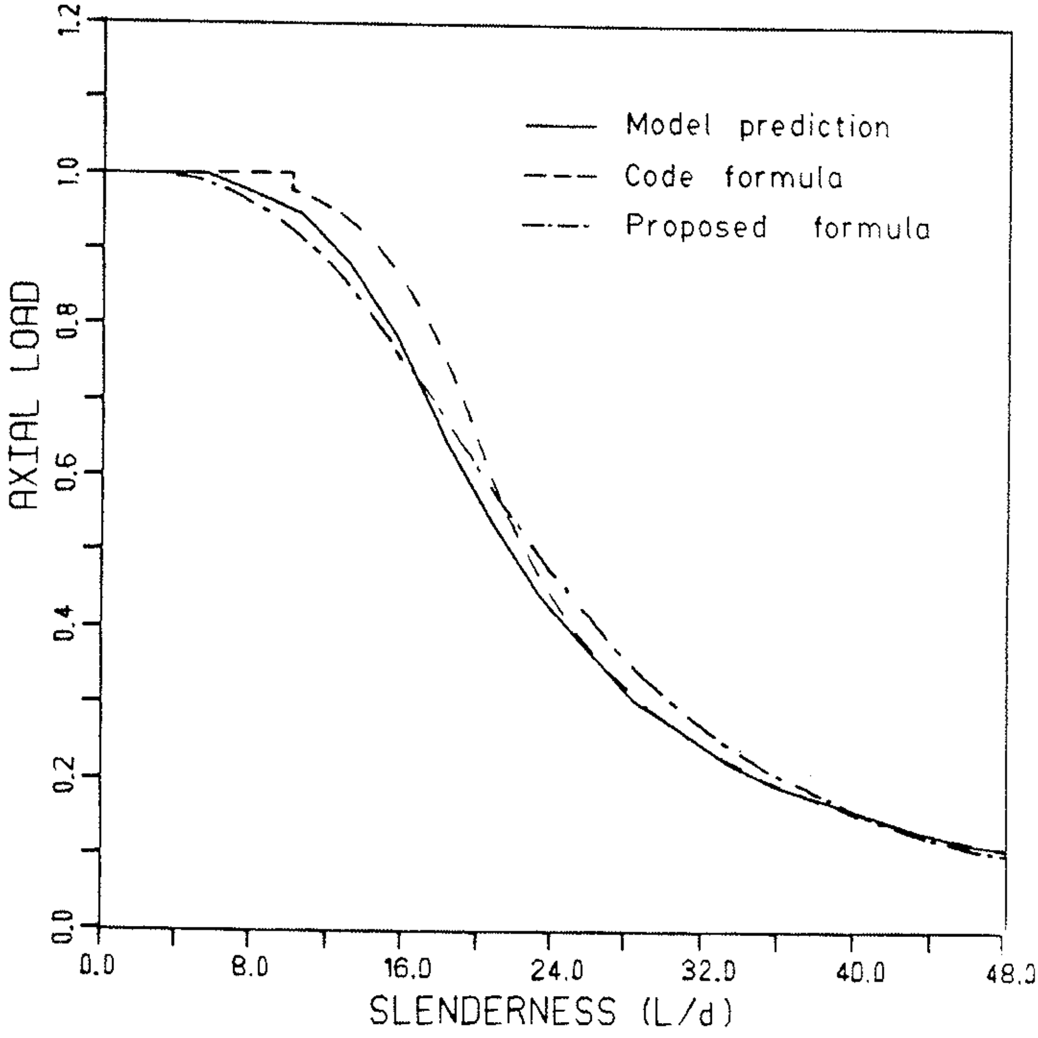
Buchanan, Johns & Madsen   Figure 5



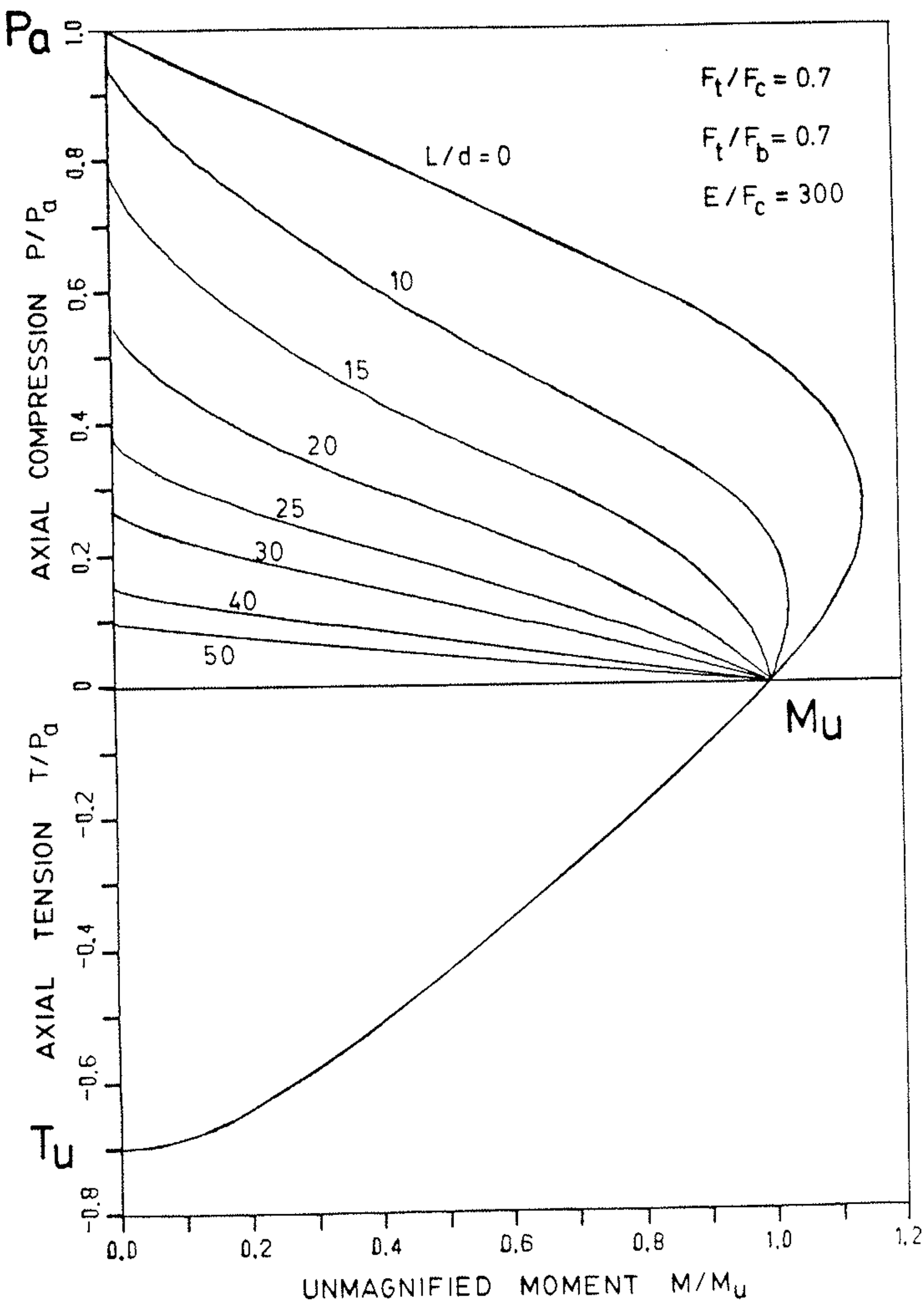
Pa
Material failure
P
P
Δ
e
Axial load
C
D
P2
E
A
B
P1
0
Mu
Moment
(a)
Pa
Material failure
P
P
Δ
e
Axial load
Pu
Mid-span moments at failure
C
D
A
B
End moments at failure
Mu
Moment
(b)



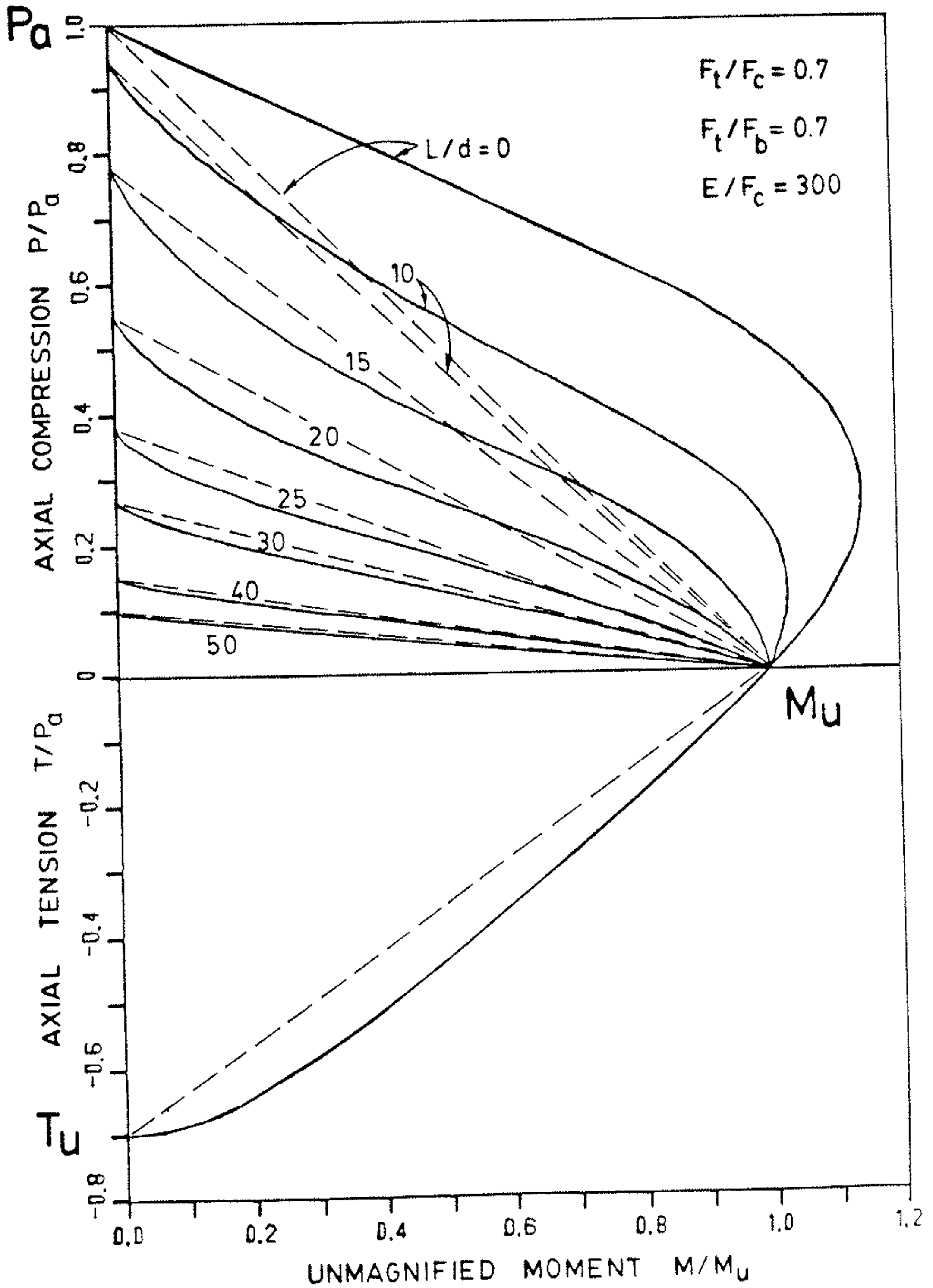
Buchanan, Johns & Madsen Figure 7



Buchanan, Johns & Madsen Figure 8



Buchanan, Johns & Madsen Figure 9



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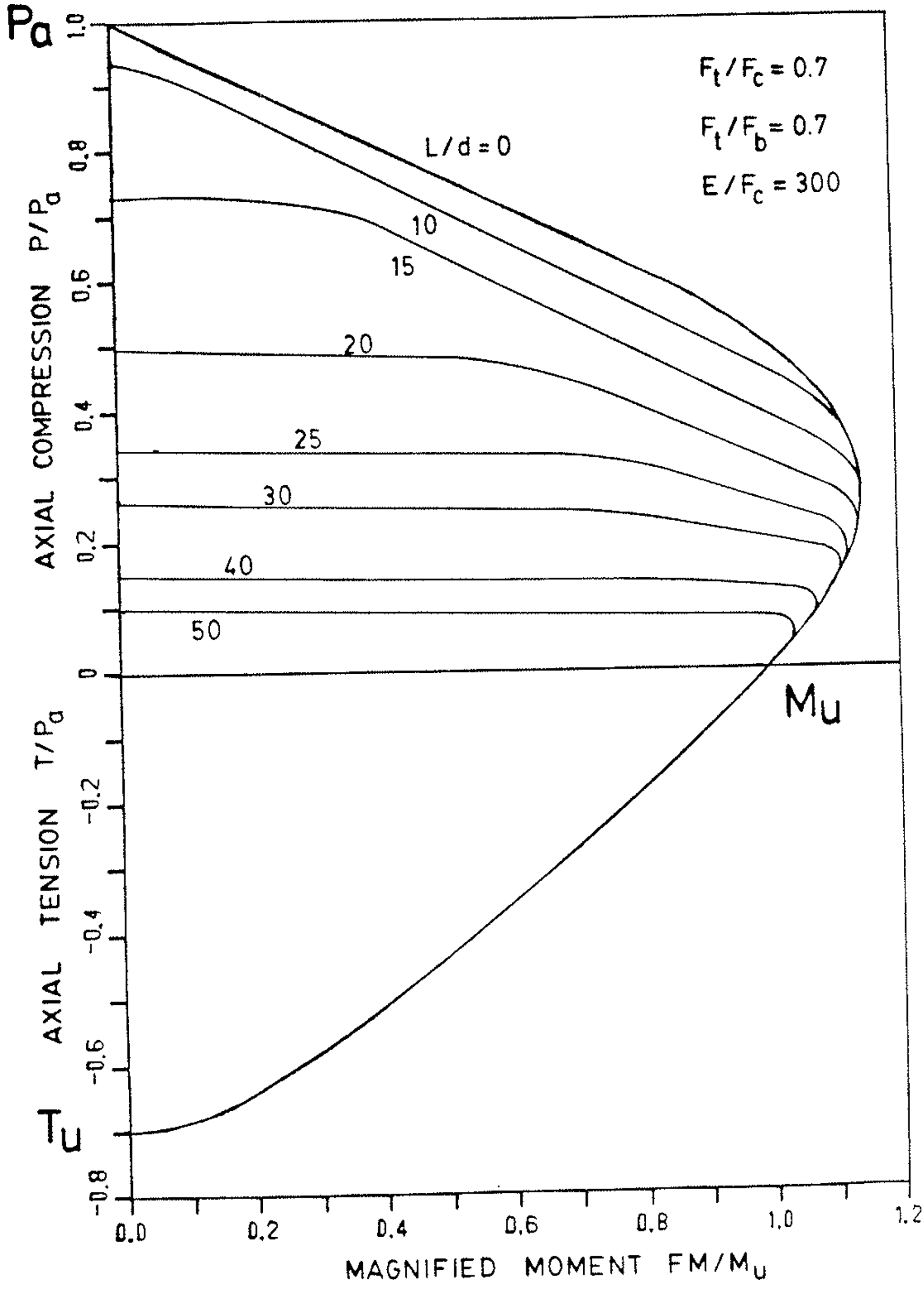
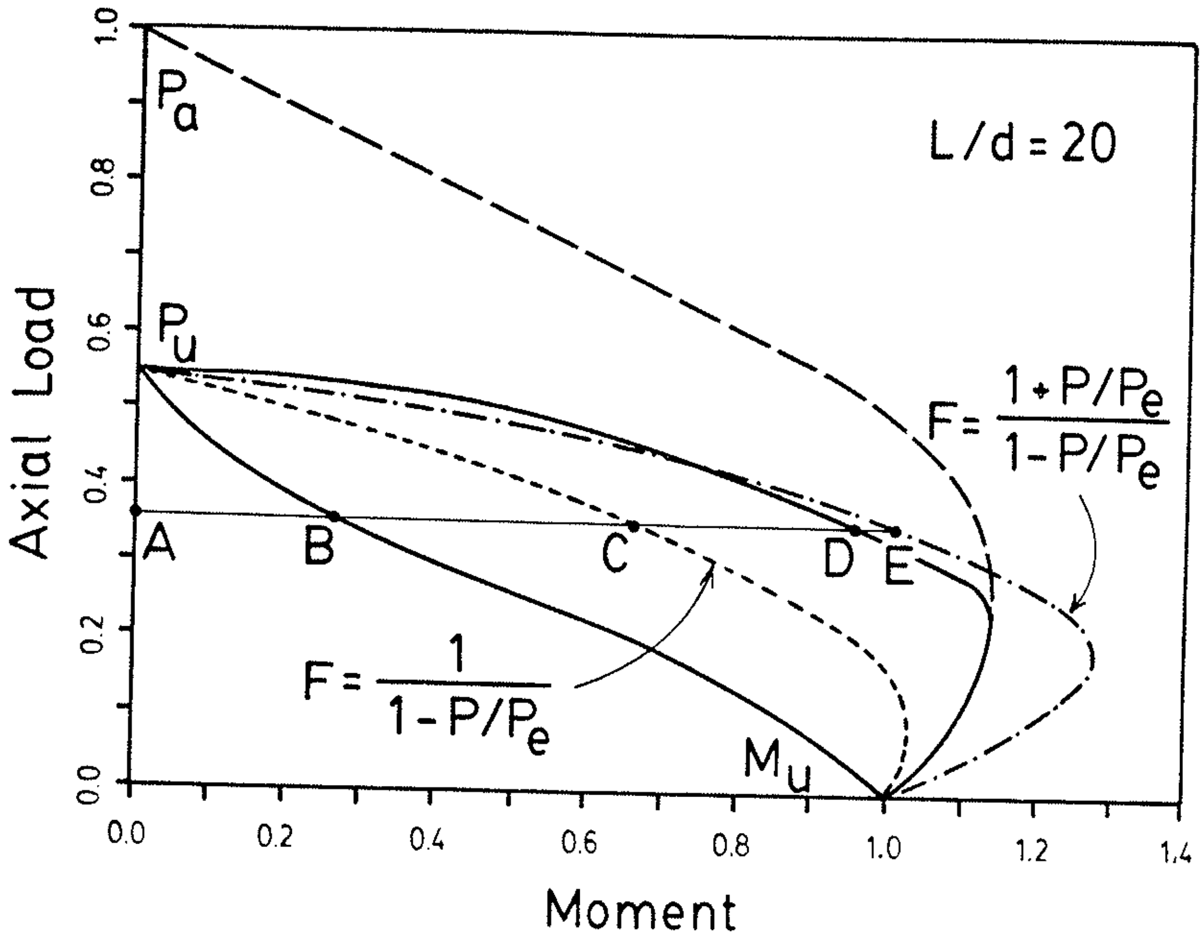
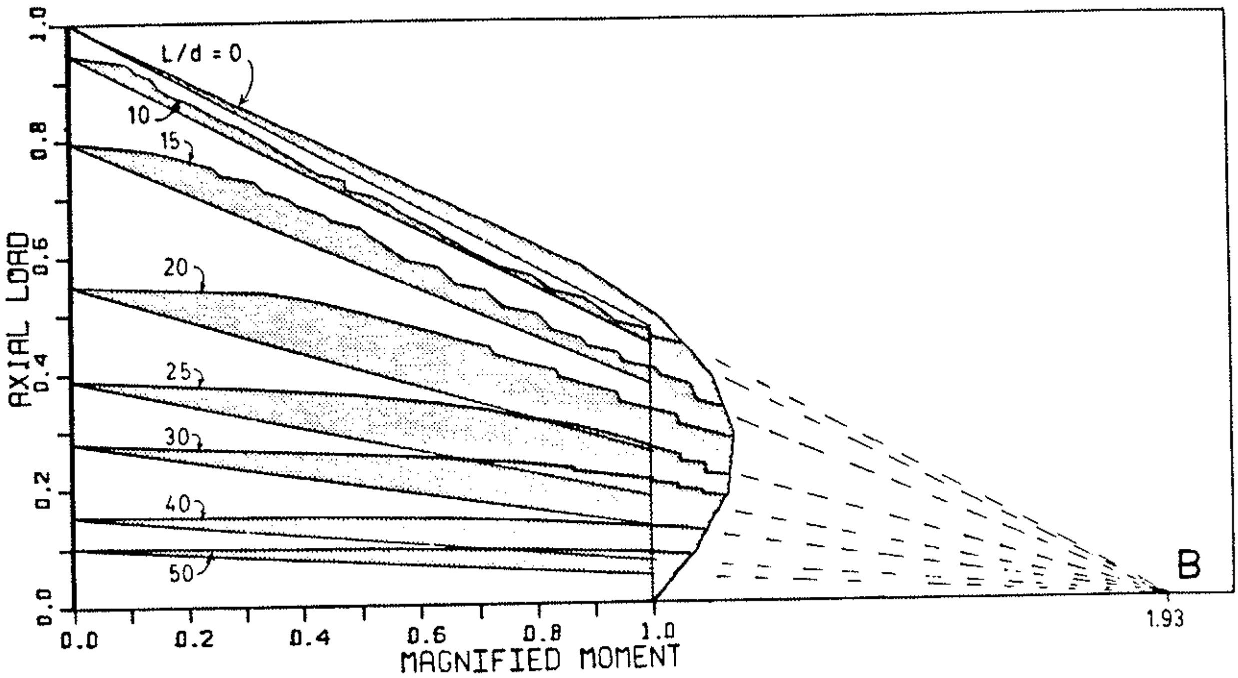


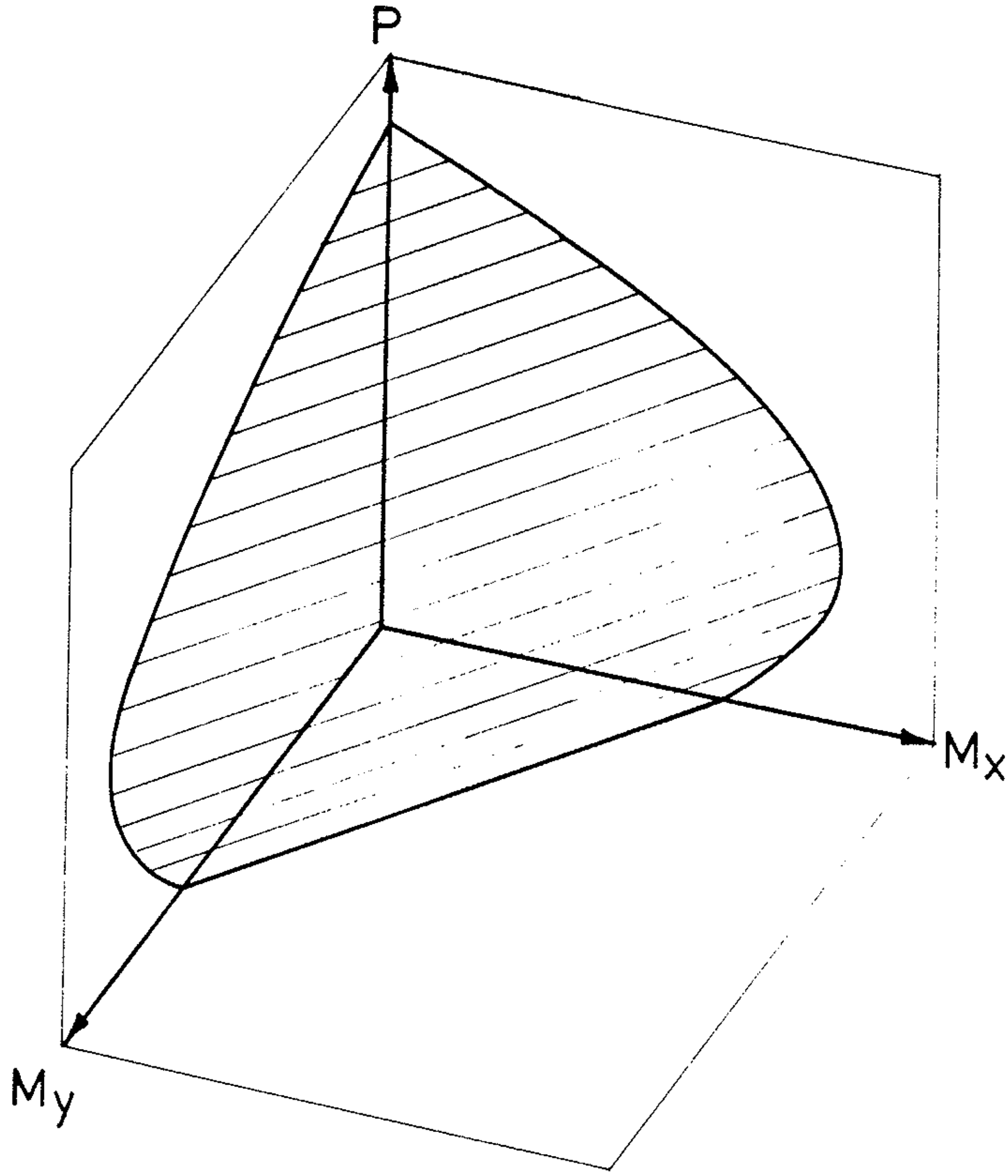
Buchanan, Johns & Madsen   Figure 11



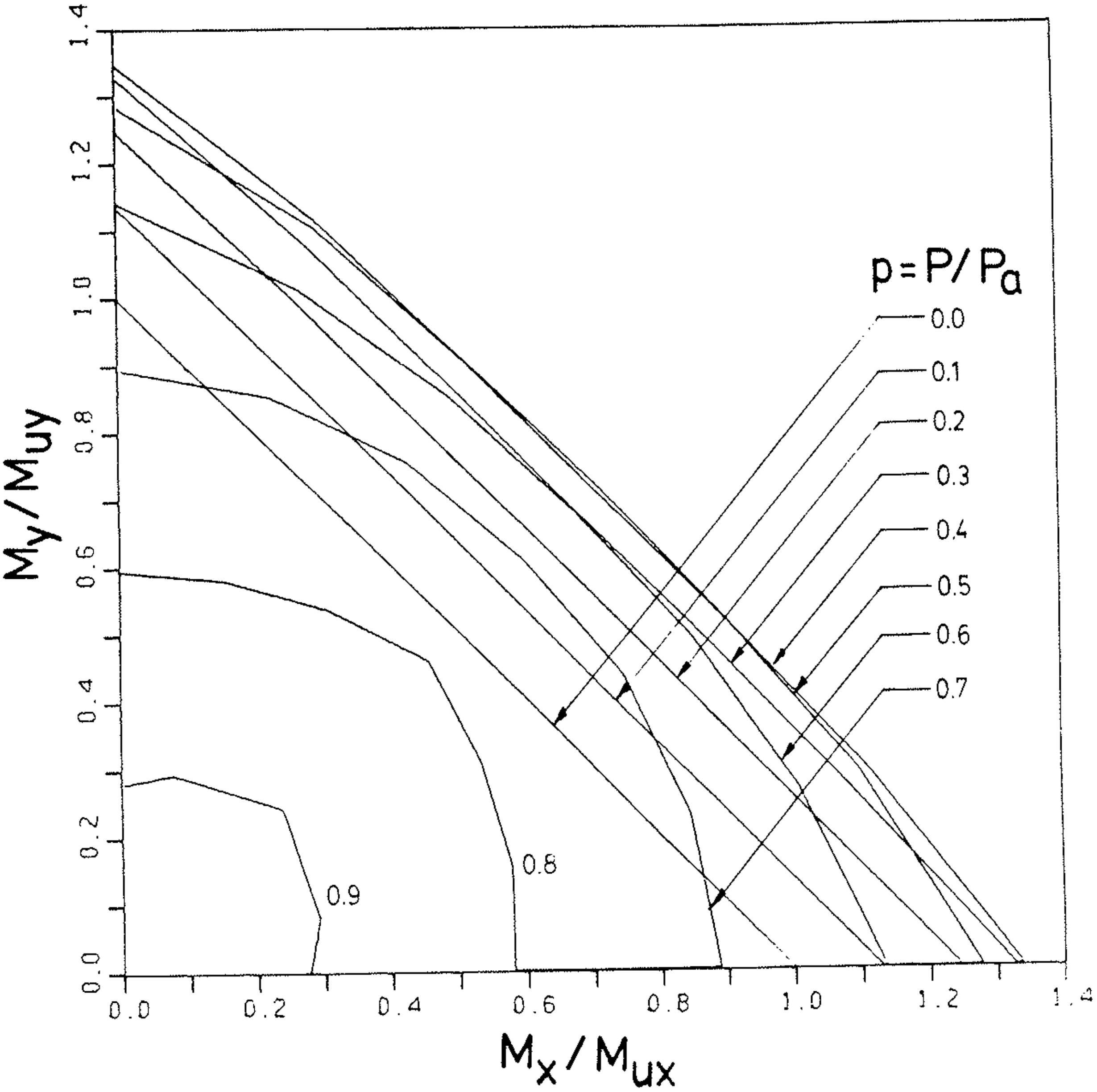
Buchanan, Johns & Madsen Figure 12



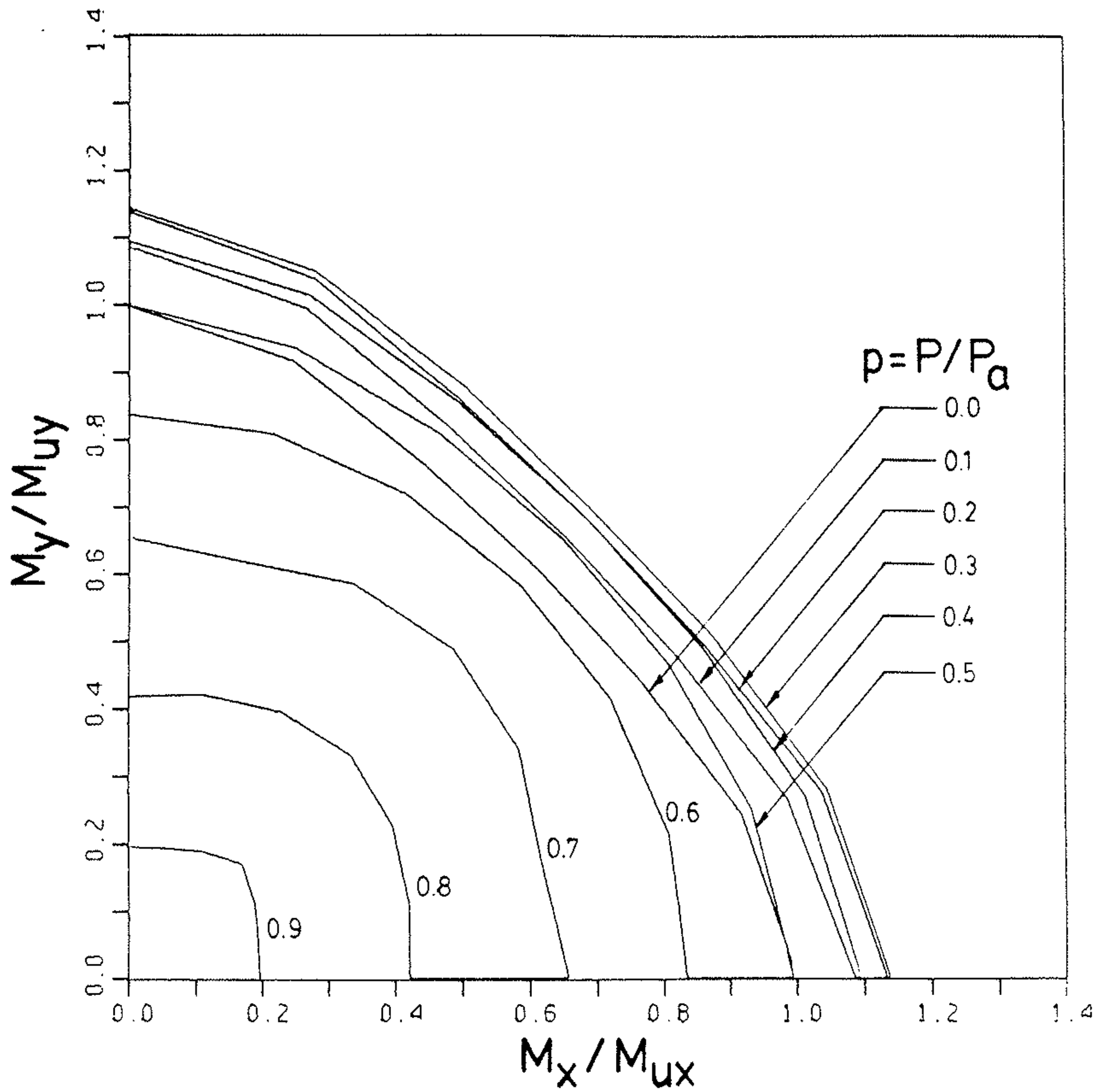
Buchanan, Johns & Madsen Figure 13



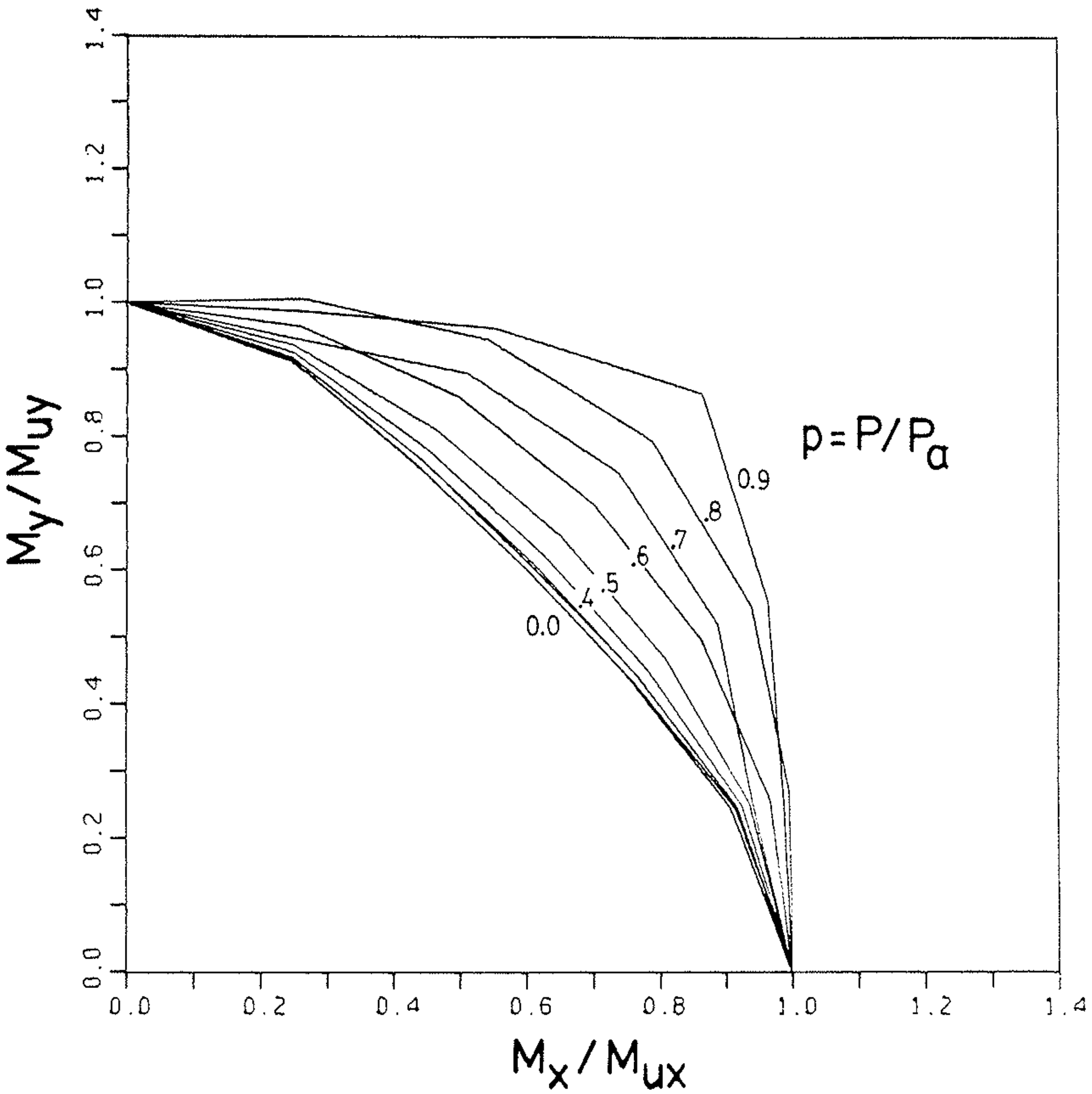
Buchanan, Johns & Madsen   Figure 14



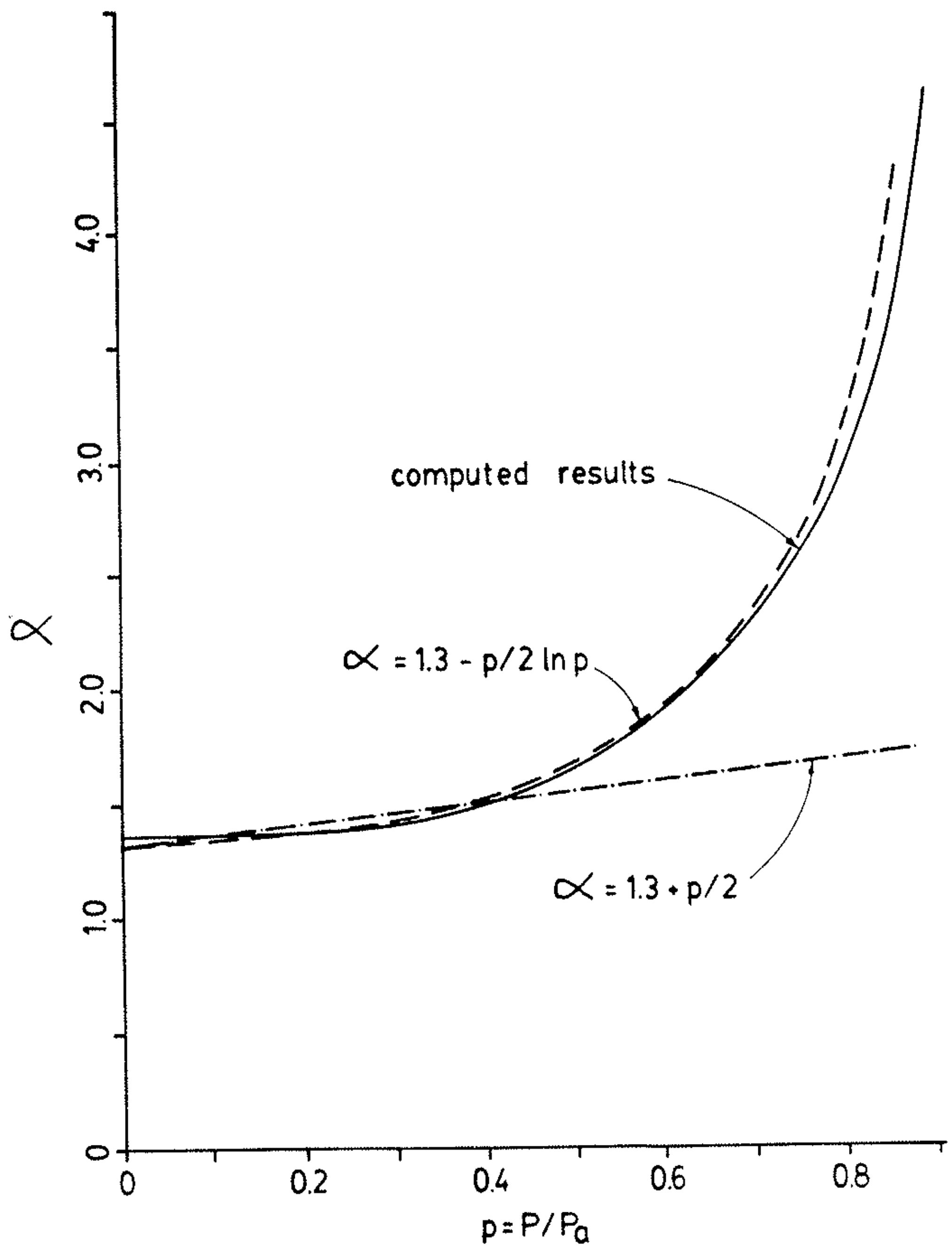
Buchanan, Johns & Madsen   Figure A1



Buchanan, Johns & Madsen   Figure A2



Buchanan, Johns & Madsen Figure A3



Buchanan, Johns & Madsen   Figure A4

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON PAPERS: 18-6-2 AND 18-6-3

by

R H Leicester
CSIRO
Division of Building Research
Australia

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

Comment on Papers:

'CONFIGURATION FACTORS FOR THE BENDING STRENGTH OF TIMBER'

and

'NOTES ON SAMPLING FACTORS FOR CHARACTERISTIC VALUES'

by R H LEICESTER

Parts of these two papers are relevant to the topic of in-grade testing. Equations (4), (7) and (8) in the paper 'Notes on sampling factors for characteristic values' shows the reqired penalty in characteristic value due to the fact that the tested samples may be of limited size N.

Furthermore, in the paper 'Configuration factors for the bending strength of timber', equation (3) together with the data shown in Figures 4(a) and 5(a) provide an idea of the effect of bias in testing; thus for the case of biased testing the basic requirement to estimate the five percentile is replaced by the necessity to estimate the 0.05 α_{BA} percentile. Note that $\alpha_{BA} \geqslant 1.0$.

From the form of equation (8) in the paper 'Notes on sampling factors for characteristic values' it is seen that for a given penalty in characteristic value due to limited sample size we have roughly $N_p \simeq$ constant, i.e. $N\alpha_{BA} \simeq$ constant. Hence, as an example, if $\alpha_{BA} = 2$, the required sample size in biased testing is reduced by a factor of 2 relative to that required for unbiased testing (for a given acceptable penalty).

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CONFIGURATION FACTORS FOR THE BENDING STRENGTH OF TIMBER

by

R H Leicester
CSIRO
Division of Building Research
Australia

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CONFIGURATION FACTORS FOR THE BENDING STRENGTH OF TIMBER

by

R.H. LEICESTER

B.Eng.(Western Australia), M.Sc.(Western Australia), Ph.D.(Illinois)

Senior Prinicpal Research Scientist, Division of Building Research,

CSIRO, Melbourne, Australia

SYNOPSIS

Timber is a heterogeneous material containing dispersed defects. As a result, the measured bending strength of timber depends on many test factors such as the length of the beam, the method of loading, and any bias used in selecting the test beam. The influence of these various factors are described in terms of two 'configuration' factors. The paper uses simple statistical concepts together with an examination of characteristics measured by direct test to provide an insight into these factors. It is concluded that the broad features of configuration factors are adequately described by simple statistical models, but that more sophisticated models are desirable for practical purposes.

INTRODUCTION

Timber is a heterogeneous material whose bending strength varies not only from stick to stick but also along each stick. As a result, the statistical distribution of bending strength of a given population of timber will depend not only on the properties of the parent population, but also on the manner in which it is sampled and tested.

In the following, some of the factors that affect the measured bending strength are discussed in terms of simple statistical models. Further insights are obtained through consideration of experimental data obtained in a limited set of pilot studies.

DEFINITION OF CONFIGURATION FACTORS

Two types of configuration factors will be used. They will be denoted by the symbols 'α' and 'β'. The definitions of these factors are illustrated in Figure 1, which shows the cumulative distribution functions p_A and p_B of the strengths A and B. In formal terms, the configuration factors are defined by

$$\alpha_{BA}(x) = \ln[1-p_B(x)]/\ln[1-p_A(x)] \qquad \ldots (1)$$

$$\beta_{AB}(x) = x_A(p)/x_B(p) \qquad \ldots (2)$$

where $x_A(p)$ and $x_B(p)$ denote the magnitudes of the p-percentiles of A and B respectively.

It is of interest to note that for small values of $p_A(x)$ and $p_B(x)$, equation (1) may be written

$$\alpha_{BA}(x) \approx p_B(x)/p_A(x) \qquad \ldots (3)$$

It can be shown that if both A and B are Weibull distributions having the same coefficient of variation V, then both $\alpha_{BA}(x)$ and $\beta_{AB}(x)$ are constants with the following values,

$$\beta_{AB} = \bar{A}/\bar{B} \approx \alpha_{BA}^{V} \qquad \ldots (4)$$

where $\bar{A}$ and $\bar{B}$ denote mean values of A and B respectively.

It is to be noted that for the case $\bar{A} > \bar{B}$ it follows that $\alpha_{BA} > 1$ and $\beta_{AB} > 1$. It should also be noted that from the definitions of α and β it follows that for the case of three strength distributions A, B and C the relationship between configuration factors is

$$\alpha_{CA} = \alpha_{CB}\,\alpha_{BA} \qquad \ldots (5a)$$

$$\beta_{AC} = \beta_{AB}\,\beta_{BC} \qquad \ldots (5b)$$

Both the configuration factors α and β are of practical interest. The factor β at the five percentile level is used in design codes to specify the effects of various parameters on the characteristic strength. The configuration factor α is of value for proof grading and in-grade testing specifications. For example, the test conditions for in-grade testing may be chosen so as to produce a biased strength that is lower than the target in-situ strength; as a result the five percentile in-situ strength will correspond to the 0.05α percentile value of the test strength, and since $\alpha > 1$ the 0.05α percentile value can be estimated with a greater accuracy than the five percentile value for a given sample size.

FACTORS FOR LOADING CONFIGURATIONS

In the interpretation of test data, one of the most important factors to consider is the effect of loading configuration. It is shown in the Appendix that for the case of symmetrically loaded beams as shown in Figure 2, a modification of the conventional Weibull theory of strength leads to

$$\alpha_{BA} = [L_B + (K_B/V\rho)]^{\rho}/[L_A + (K_A/V\rho)]^{\rho} \qquad \ldots (6)$$

and

$$\beta_{AB} = [L_B + (K_B/V\rho)]^{\rho V}/[L_A + (K_A/V\rho)]^{\rho V} \qquad \ldots (7)$$

where

$$\rho = (1-r)^{0.5} \qquad \ldots (8)$$

in which the dimensions L and K are the total and mid-span lengths as shown in Figure 2, and r is the correlation coefficient for matched pairs of timber beams, each pair being cut from the same stick of timber.

For the case of $r \to 1$, equations (6)-(8) lead to $\alpha_{BA} \to 1.0$ and $\beta_{AB} \to 1.0$; for the case of $r \to 0$, equations (6)-(8) lead to the solution given by the conventional Weibull theory of strength[5].

EXPERIMENTAL DATA

Material

The data to be presented are from tests on 45 x 100 mm seasoned radiata pine (Pinus radiata) and on 45 x 90 mm unseasoned messmate (Eucalyptus obliqua). The test beams were cut from sticks that were 4.3 metres long and had being graded according to the Australian standards[3,4] AS 1490-1973 and AS 2082-1977 respectively. The radiata pine was a knotty material comprising a mixture of the F7 and F8 stress grades described in AS 1490, the F7 stress grade being a nominal 38 percent structural grade with an allowable centre face knot size equal to half the face width. The messmate was generally clear material. comprising the structural grades Nos.1 and 2 described in AS 2082, the No.2 grade being a nominal 60 percent grade.

Test Method

Roughly 200 sticks of each species were selected. These were divided into four similar groups labelled I, II, III and IV. Each stick was cut in half, and from the centre of each half a beam of length 20D was cut, where D denotes the face width of the timber and the depth of the beams. The beams were tested over a span of 18D to measure their bending strength.

Beams cut from the sticks in groups I and II were tested in third point loading, i.e. K/L = 1/3 according to Figure 2, and beams cut from the sticks in groups III and IV were tested with centre point loading, i.e. K = 0.

For beams cut from the sticks in groups I and III, the edge placed in tension was chosen to correspond to the edge along which the grade controlling defect was located. For beams cut from the sticks in groups II and IV the edge placed in compression was chosen to correspond to the edge along which the grade controlling defect was located.

TEST RESULTS

Effect of Loading Configuration

Figure 3 shows the measured configuration factors for the two types of loading applied. The population A is taken to be bending strength of the 200 beams with centrepoint loading cut from sticks in groups III and IV, and configuration B is taken to be the bending strength of the 200 beams with third point loading cut from the sticks in the groups I and II.

By comparing the strengths of the pairs of beams cut from the same sticks in groups I and II it was found that for the radiata pine a suitable choice of statistical parameters was V = 0.35 and r = 0.1. From equations (6) and (7) this leads to the predicted configuration factors α_{BA} = 1.93 and β_{AB} = 1.26. Similarly for messmate it was found that V = 0.15 and r = 0.5 which lead to α_{BA} = 2.73 and β_{AB} = 1.16.

It is seen that the estimates of configuration factors based on equations (6) and (7) are reasonably good for average conditions but do not fit well for the lower percentile values that are of considerable practical interest. One reason for this could be that the characteristics of the lower tails are determined by discrete defects, whereas the theory is based on the assumption of continuously varying strength.

Effect of Testing Bias

Two types of testing bias are frequently introduced into in-grade tests. The first is to to place the assessed weakest defect at the point of maximum moment. This is equivalent to stressing most of the stick to the maximum moment, an effect which can be predicted by use of equations (6) and (7). To check the ability of the equations to predict this type of effect, the effect of doubling the loading length was examined and the results are shown in Figure 4.

In Figure 4 the population A is taken to be the strength of all beams from groups I and II, and population B is taken to be the strength of the weakest beam from each stick. From equations (6) and (7) and the parameters V and r previously mentioned the expected configuration factors are α_{BA} = 1.93, β_{AB} = 1.26 for radiata pine and α_{BA} = 1.63, β_{AB} = 1.08 for messmate. Again it is to be noted that the simple model fits the data well in broad terms, but not in detail.

In addition to selecting the weakest defect, a further bias is sometimes introduced into testing. This is to deliberately place the worst defect on the tension edge in a strength test. The effect of this additional bias is illustrated by the test data shown in Figure 5. Here the population A is taken to be made up of the strength of the weakest beam in each stick of groups I and II, and population B is taken to be the strength of the weakest beam in each stick of group I only.

It should be noted that the assumption of obtaining a higher failure rate if the worst defect is always placed in the tension edge would be that the configuration factor α_{BA} is bounded by $1 < \alpha \leq 2$.

Effect of Stick Length

The effect of doubling the length of stick that is graded is simulated in Figure 6. Here the population A represents the strength of all beams from the sticks in groups I and II and population B represents the strength only of the beams located in the half of the stick that contains the grade controlling defect.

It is reasonable to assume that the weakest beams in A and B are in fact the same ones and accordingly since there are twice as many beams in population A as compared with B then the value of α_{BA} should tend to the value of 2.0 for small percentiles. This is in line with the graphs in Figure 6a.

CONCLUDING DISCUSSION

Some insight into the configuration factors associated with the measurement of bending strength has been obtained through examination of the test data given in Figures 3-6 and the application of simple statistical concepts such as the use of equations (6) and (7) which were developed to predict the effect of the loading configuration. These studies indicate that configuration factors encountered in typical practical situations are sufficiently large that they should not be ignored; the various factors studied can be cumulative in the manner indicated by equations (5a) and (5b).

Although the effectiveness of the test data cited is limited by the small size of the samples used, they are sufficient to demonstrate that while the simple statistical models used are adequate to describe the broad features of configuration factors, they are not accurate enough to predict the magnitudes of the low percentiles with an accuracy sufficient for practical applications. This applies particularly to the accuracy of the relationship between α_{BA} and $\hat{\rho}_{AB}$ given in equation (4).

It is apparent then, that to make further progress it will be necessary to test larger samples of timber and also to develop statistical models of strength that can describe the characteristics of timber that contains dispersed isolated defects.

ACKNOWLEDGEMENTS

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APPENDIX

DERIVATION OF FACTORS FOR LOADING CONFIGURATIONS

For a Weibull distribution of the random variable A, the cumulative distribution function $p_A(x)$ may be written[1]

$$p_A(x) = 1 - \exp\{\phi(x/\bar{A})^m\} \qquad \ldots \text{(A1)}$$

where $\phi = \Gamma^m [1 + (1/m)]$, and to a good approximation

$$m \approx V^{-1.09} \qquad \ldots \text{(A2)}$$

For a rough approximation equation (A2) may be written

$$m \approx 1/V \qquad \ldots \text{(A3)}$$

where V is the coefficient of variation of A.

In terms of this approximation for the parameter m, the configuration factors derived by Bohannon[2] for beams loaded as shown in Figure 2 are given by

$$\beta_{AB} = \bar{A}/\bar{B} = [L_A + (K_A/V)]^V/[L_B + (K_B/V)]^V \qquad \ldots \text{(A4)}$$

An examination of the derivation of equation (A4) reveals that it is based on the assumption that the strengths of timber between any two points within a stick are uncorrelated. It can be shown that, if there is some degree of correlation, then equation (A4) should be replaced by

$$\beta_{AB} = \bar{A}/\bar{B} = [L_A + (K_A/V_w)]^{V_w}/[L_B + (K_B/V_w)]^{V_w} \qquad \ldots \text{(A5)}$$

where V_w is the within-stick coefficient of variation.

To derive V_w, the strengths of two beams cut from the same stick will be considered. These strengths will be denoted by A_1 and A_2 and will be written in the form

$$A_1 = A_o\ A_{w1} \qquad \ldots \text{(A6)}$$
$$A_2 = A_o\ A_{w2} \qquad \ldots \text{(A7)}$$

where A_{w1} and A_{w2} represent the components of strength that vary within a stick, and A_o is the component of strength that varies from stick to stick. The random variables A_o, A_{w1} and A_{w2} are assumed to be uncorrelated.

Taking the natural logarithms of A_1 and A_2, equations (A6) and (A7) lead to

$$T_1 = T_o + T_{w1} \qquad \ldots \text{(A8)}$$
$$T_2 = T_o + T_{w2} \qquad \ldots \text{(A9)}$$

where the logarithms of A are denoted by T. From Equations (A8) and (A9) it follows that

$$\text{var}(T_1 - T_2) = \text{var}(T_1) + \text{var}(T_2) - 2r_{T1T2}\ \{\text{var}(T_1)\ \text{var}(T_2)\}^{0.5} \qquad \text{(A10)}$$

and

$$\text{var}(T_1 - T_2) = \text{var}(T_{w1} - T_{w2})$$
$$= \text{var}(T_{w1}) + \text{var}(T_{w2}) \qquad \ldots \text{(A11)}$$

where var( ) denotes the variance and r_{T1T2} is the correlation coefficient between T_1 and T_2.

Introducing the following approximations

$$\mathrm{var}(T_1) = \mathrm{var}(T_2) \cong V^2 \qquad \ldots \text{(A12)}$$

$$\mathrm{var}(T_{w1}) = \mathrm{var}(T_{w2}) \cong V_w^2 \qquad \ldots \text{(A13)}$$

$$r_{T1T2} \cong r_{A1A2} = r \qquad \ldots \text{(A14)}$$

into equations (A10) and (A11) leads to

$$V_w^2 = V^2(1-r) \qquad \ldots \text{(A15)}$$

Substitution of (A15) into (A5) leads to equation (7). Use of the approximation $\beta_{AB} \cong \alpha_{BA}^{V}$ from equation (4) then leads to equation (6).

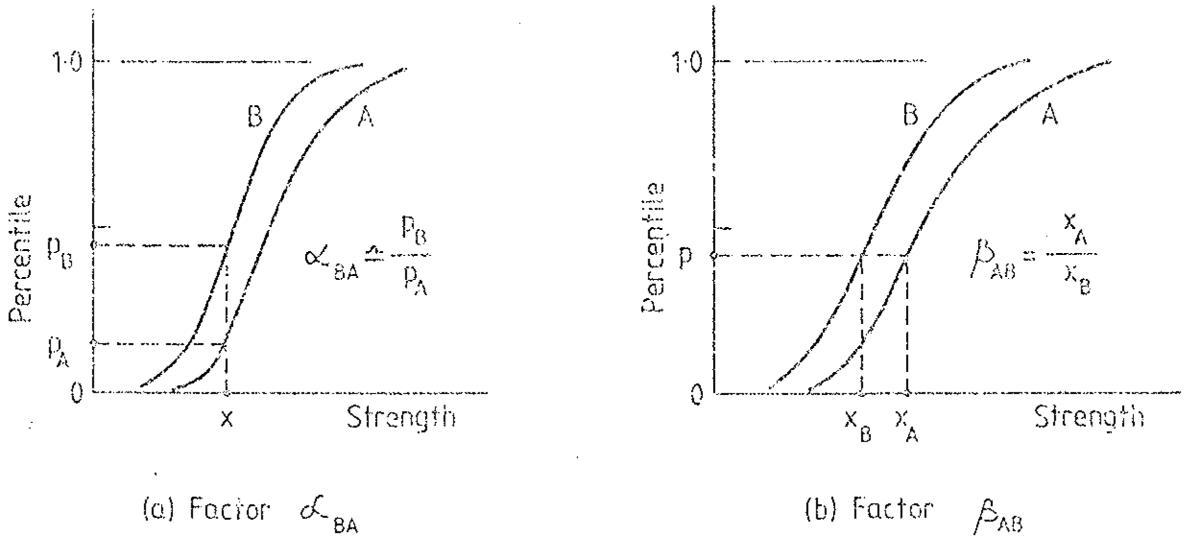


Figure 1. Illustration of the definitions of the configuration factors α and β

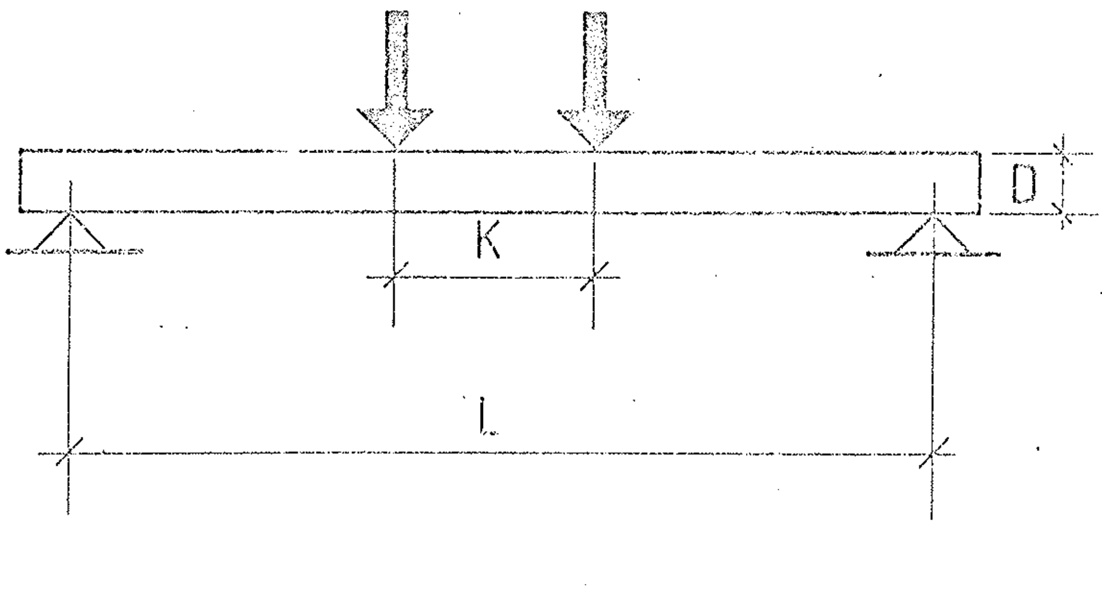
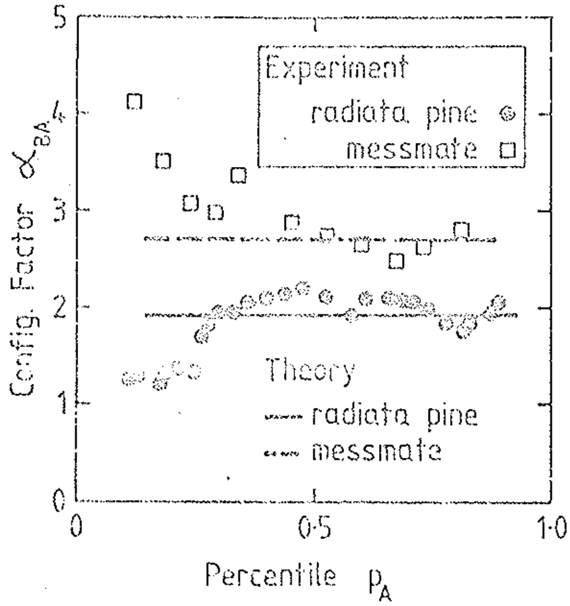
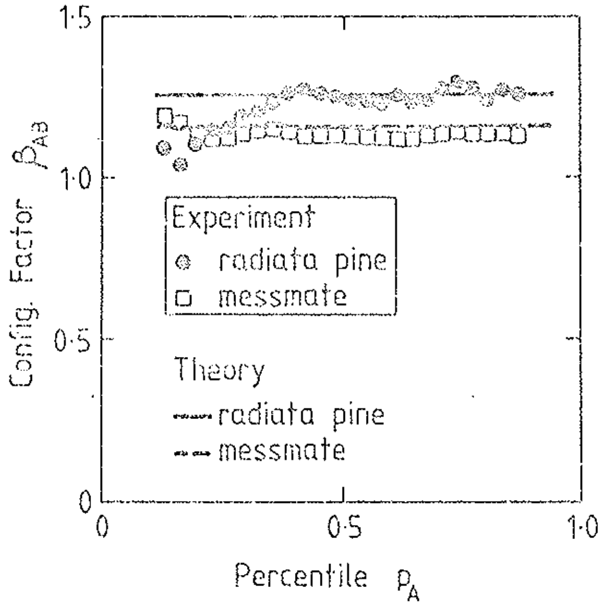


Figure 2. Notation for loading method

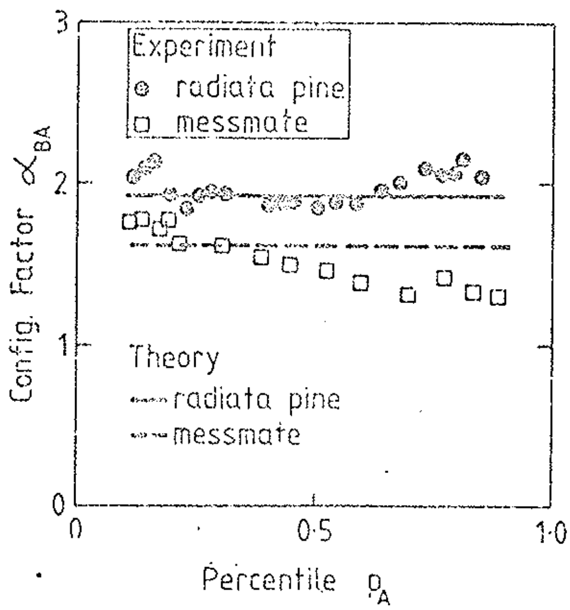


(a) Factor α_{BA}

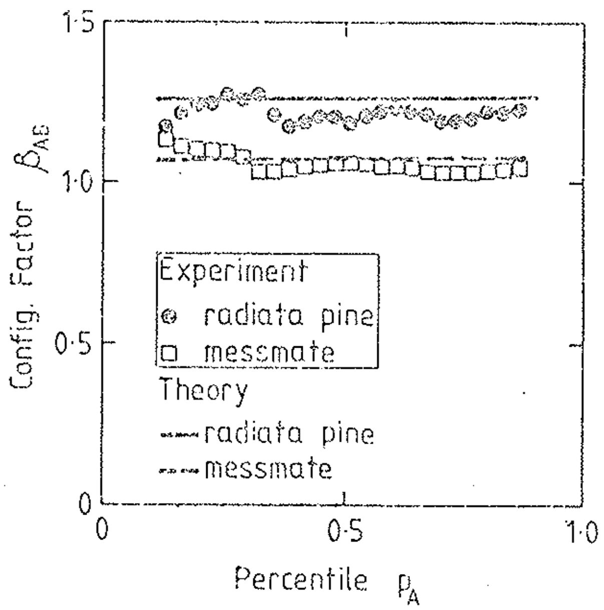


(b) Factor β_{AB}

Figure 3. Configuration factors for loading method
Configuration A: centre point loading
Configuration B: third point laoding



(a) Factor α_{BA}



(b) Factor β_{AB}

Figure 4. Configuration factors for effect of loaded length
Configuration A: loaded length = L
Configuration B: loaded length = 2L

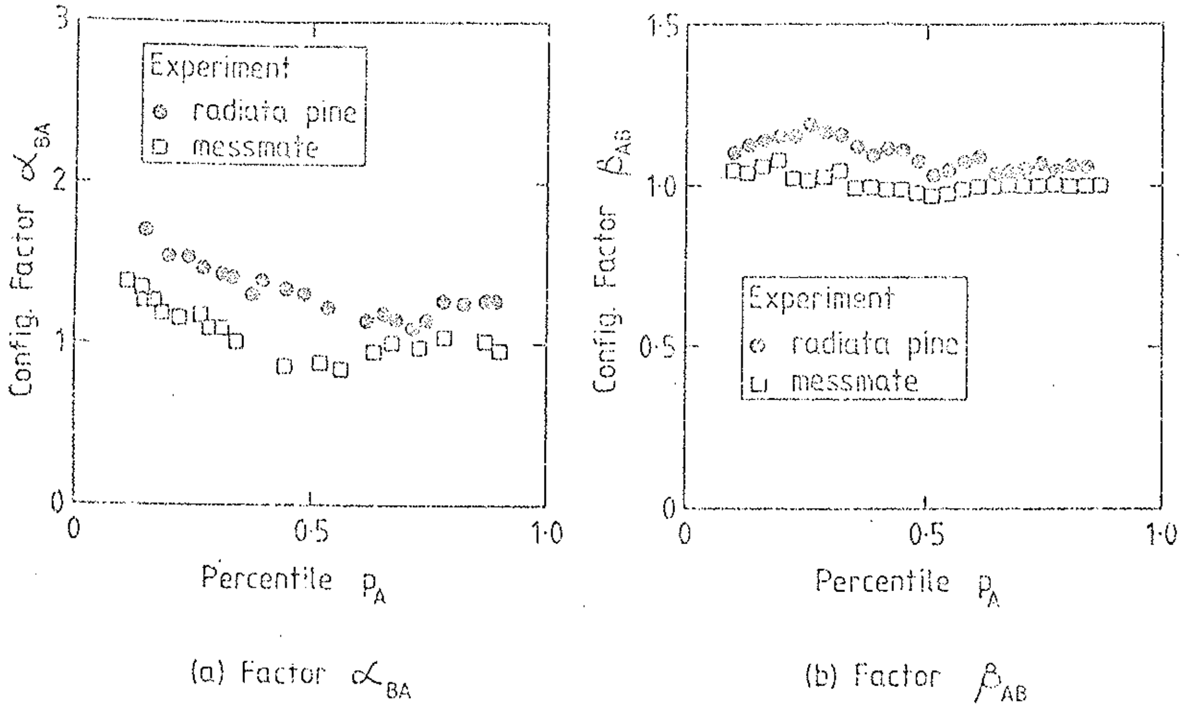


Figure 5. Configuration factors for the effect of edge bias
Configuration A: grading defect on tension edge
Configuration B: grading defect on random edge

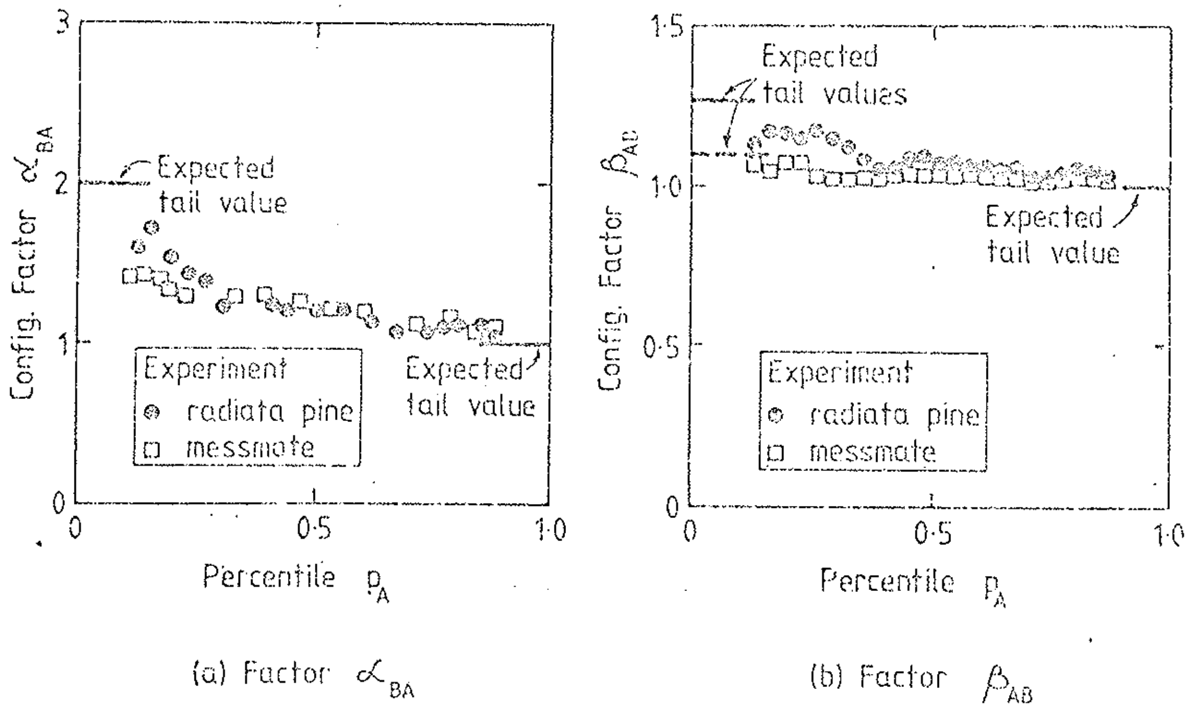


Figure 6. Configuration factors for the effect of stick length
Configuration A: stick length is full stick
Configuration B: stick length is half stick

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

NOTES ON SAMPLING FACTORS FOR CHARACTERISTIC VALUES

by

R H Leicester
CSIRO
Division of Building Research
Australia

MEETING EIGHTEEN
BEIT OREN
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JUNE 1985

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NOTES ON SAMPLING FACTORS FOR CHARACTERISTIC VALUES

by

R H LEICESTER

CSIRO Division of Building Research

(March, 1985)

1. SMALL SAMPLES ($N \leqslant 10$)

$$R_k = R_{min}/LF \tag{1}$$

where

R_k = required characteristic value

R_{min} = minimum value in a sample of N structural elements

LF = load factor

Load factor is derived from

$$LF = \left\{\frac{\ln\ (1\text{-confidence})}{N\ \ln\ (1\text{-percentile})}\right\}^V \tag{2}$$

where V is the estimated coefficient of variation of the structural elements.

For a characteristic that is a five percentile value estimated with a 75 per cent confidence, equation (2) leads to

$$LF = [27/N]^V \tag{3}$$

[Note: Equations (2) and (3) are essentially exact for all N (i.e. from $N=1$ to $N=\infty$) for Weibull distributions of strength, and are conservative for other practical distributions of strength].

[Note: This formulation is useful for application in a prototype testing situation where only a few structures or structural elements, sometimes only one, may be tested].

2. LARGE SAMPLES ($N \geqslant 30$)

2.1 Simple Formulation (5 percentile, 75% confidence)

Rank the data and measure off $R_{k(est)}$, the data value of the five percentile. Compute V_{sam}, the coefficient of variation of the sample. Hence compute

$$R_k = R_{k(est)} \left[1 - 2.7\, V_{sam}/(N)^{\frac{1}{2}}\right] \tag{4}$$

where R_k is the five percentile value estimated with 75 per cent confidence.

[Note: Equation (4) is exact for Weibull distributions and conservative for other practical distributions of strength].

2.2 Complex formulation (5 percentile, 75% confidence)

In addition to $R_{k(est)}$ and V_{sam}, this formulation requires the computation of $\bar{R}$, the mean value of R and K_{skew}, the coefficient of skewness. From these compute R_k as follows,

$$V_o = (K_{skew} + 0.95)/3 \qquad (5)$$

$$\sigma_o = \bar{R}(1 - V_{sam}/V_o) \qquad (6)$$

$$R_k = \sigma_o + (R_{k(est)} - \sigma_o)\ [1 - 2.7\ V_o/(N)^{1/2}] \qquad (7)$$

Here again R_k is the five percentile estimated with 75 per cent confidence.

[Note: Equation (7) is essentially exact i.e. within 5%, for all practical distributions of strength].

2.3 <u>Formulation for the general case</u> (p-th percentile, 75% confidence)

If we wish to estimate the p-th percentile with 75% confidence, then the constant 2.7 in equations (4) and (7) are replaced by $0.6/p^{1/2}$, for example equation (4) becomes

$$R_k = R_{k(est)}\ [1 - 0.6\ V_{sam}/(Np)^{1/2}] \qquad (8)$$

where $R_{k(est)}$ here denotes the data value of the p-th percentile.

[Note. Equations (4) and (7) may be extended for other levels of confidence].

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIZE EFFECTS IN TIMBER EXPLAINED BY A MODIFIED WEAKEST LINK THEORY

by

B Madsen
University of British Columbia
Canada
and
A H Buchanan
New Zealand

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

SIZE EFFECTS IN TIMBER EXPLAINED

BY A

MODIFIED WEAKEST LINK THEORY

by

BORG MADSEN

Professor

Department of Civil Engineering, University of British Columbia.

and

ANDREW H BUCHANAN

Consulting Engineer

Christchurch, New Zealand.

ABSTRACT

Size effects in timber have been included in the design process for some of its minor strength properties (shear, tension perp.).This paper demonstrates that size effects should also be considered when designing with its major properties (tension, bending and compression).

Using a modification to the weakest link theory, reflecting the anisotropic nature of timber, good agreement have been obtained between tests and the theory. The theory enables us to compare tests with different spans and/or load configuration with an understanding not previously possible.

Results from several large testing programs have been used to quantify some of the necessary size effect parameters. A simple design method for bending members, taking length effects and load configuration effects into account, has been proposed.

Information for tension members is less comprehensive ; nevertheless a tentative suggestion for the design of those members have been included.

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LIST OF ILLUSTRATIONS

SIZE EFFECTS IN TIMBER EXPLAINED

BY A

MODIFIED WEAKEST LINK THEORY

I INTRODUCTION

FAILURE MODES

Wood stressed in tension generally follows a linear elastic stress-strain relationship, and fails with a sudden brittle fracture. In compression, on the other hand, wood exhibits non-linear behaviour with a large amount of ductile yielding. This behaviour is illustrated in Fig. 1.

In bending, behaviour can be predicted from the tension and compression properties, but the failure mode depends on the relative values of tension and compression strenghts.

For this reason the failure mode for lumber in bending is very different from the failure mode of wood. (In this paper a destinction is made between "wood" in the sense of clear defect-free wood and "lumber" or "timber" which contains natural growth characteristics such as knots). Straight grained wood is stronger in tension than in compression so failure in bending occurs in the compression zone where small visible wrinkles develop prior to final failure. This leads to a somewhat ductile behaviour. Lumber on the other hand often contains knots in the tension zone making it much weaker in tension than in compression. This causes the failure to occur in the tension zone (with a brittle fracture) before any ductile compression yielding develops. The failure initiation often occurs where the fibres adjacent to knots are at an angle to the edges of the board, in which case a combination of localized tension perpendicular to the grain stresses and shear stresses causes crack growth resulting in sudden failure.

IN-GRADE TESTING

The strength properties for lumber and timber have traditionally been derived from tests of small clear defect free wood specimens of the species in question. These test values have then been corrected for moisture content, duration of load, and other effects to obtain allowable stresses to be used for structural design purposes.

Because the failure modes for wood and lumber are very different, the use of the small clear specimen approach as the basis for design stresses is slowly being abandoned in favour of the more realistic "In-Grade testing" approach. Here large representative samples of full size specimens of lumber are tested to a predetermined proof load level which will break 10 to 15% of the sample, allowing the shape of the lower tail of the strength distribution to be established in an economical manner. The lower tail of the distribution can be used to calculate a suitable characteristic value for design purposes. The major Canadian commercial species, grades and sizes have been tested in this manner and strength values for bending and tension are now incorporated into CSA-086-84.

SIZE EFFECTS

An important finding from the In-Grade testing program was that deep bending members and wide tension members have less strength than their smaller counterparts of the same species and grade. The strength differences due to size were found to be up to 50%, so size effect factors had to be incorporated into the CSA-086-84 code reflecting the experimental observations.

These size effects are related to the brittle fractures which occur when lumber fails in a tension zone.

The tension members were all tested at a constant length of 3.0 m so the size effects result from varying the width of the members.

The bending members were all tested at a constant span to depth ratio of 17:1, so the strength differences may be the result of varying beam depth, or length or both.

The existance of a size effect associated with brittle fracture in wood beams has been reported earlier by Bohannan (1966) who conducted tests with laminated wood beams. His findings were incorporated in the design procedure for laminated beams in the United States but were not adopted in Canada. For lumber a depth effect was recognized to the extent that the design values for 38 x 89 mm material were slightly higher than for deeper members of the same species and grades.

OTHER WEAKEST LINK EFFECTS

The brittle nature of failure in lumber, associated with the weakest link concept, suggests that in addition to depth effect, the strength of lumber should depend on the length of the member, the load configuration, and the type of loading. These subjects will be addressed in this paper.

The weakest link concept has previously been incorporated in CSA-086-84 for checking tension perpendicular to the grain stresses in tapered glulam beams and also for checking shear stresses in very large glulam beams.

SUMMARY

Since brittle fracture behaviour apparently governs the strength of lumber it is important to understand the phenomena so it can be incorporated properly in the design process.

This paper presents a review of brittle fracture theory and describes testing programs conducted in an attempt to quantify the magnitude of the different weakest link effects. The data are compared with theory and suggestions are made for incorporating these effects into the design procedure for lumber.

II WEAKEST LINK THEORY

HISTORICAL DEVELOPMENT

Conventional brittle fracture theory (or "statistical strength theory") has been developed on the basis of the weakest link concept first proposed by Pierce (1926) who studied cotton yarns and Tucker (1927) who studied concrete. Major developments of the theory were made by Weibull (1939 a,b) who verified his results with tests on many different brittle materials, but apparently not wood. Weibull showed how the strength of a weakest link system can be explained by a cumulative distribution of the exponential type, and how the strength depends on the volume of the test specimen for uniform or varying distributions of stress within the specimen.

Johnson (1953) improved the theoretical basis for Weibull's theory, recognizing that the exponential type of distribution proposed by Weibull is the appropriate extreme value distribution for the weakest element in a member.

BRITTLE AND DUCTILE BEHAVIOUR

Most materials can be classified as being either "brittle or "ductile", depending on the type of failure that they exhibit. Brittle materials fail very suddenly, without warning signs such as large increases in deflections. Glass and ceramics are typical brittle materials. Ductile materials (including many metals and plastics) fail in a more gradual manner with a large amount of yielding before final failure.

To further examine the difference between brittle and ductile materials, it is useful to consider a member which conceptually consists of a large number of small elements. The strength of the individual elements varies according to a strength distribution.

If the member is subjected to a uniform stress (as occurs in, for instance, tension members) and the stress is increased gradually, failure of any one of the elements will cause a redistribution of stresses within the member. If the member is made of brittle material and the failed element loses all of its strength suddenly, there will be an instantaneous increase in stresses in the adjacent elements. There may also be further stress increases due to developing stress concentrations in the vicinity of the failed element. These stress increases make it highly probable that the strength of adjacent elements will be exceeded in which case the fracture will propagate suddenly through the member, causing immediate and total failure. A material of this type is called "perfectly brittle" material and its strength is governed by the strength of its weakest element.

In a ductile material, on the other hand, an element will yield and still carry a portion of the load so there is no sudden increase in the stress in the adjacent elements. A "perfectly ductile" material has unlimited capacity to undergo deformations after the maximum strength has been reached.

Most real materials exhibit behaviour somewhere between the two extremes of perfectly brittle and perfectly ductile behaviour. Lumber is interesting because it is somewhat ductile in compression while it has a brittle behaviour in tension. This behaviour is illustrated by the typical stress strain curves for lumber shown in Fig. 1.

The equations used in this paper will be based on an assumption of perfectly brittle bahaviour. The mathematical theory for this behaviour is described in Appendix A.

MANIFESTATIONS OF BRITTLE FRACTURE PHENOMENON

The brittle fracture phenomenon affects material strength in several different ways. The most important ones are :

1. Large members tend to fail at lower stresses than smaller members loaded in a similar manner.
2. For members of a given size, failure stresses decrease as the highly stresses portion of the member increases.

3. Failure stresses depend upon the nature of the applied loading (load control vs deformation control).

4. The above effects are greatest for materials with a large amount of variability in strength.

Items 1,2 and 4 above are size effects which will be discussed further in this paper. Item 3 has been described by Madsen and Mindess (1984), who demonstrated that lumber in bending is apparently weaker when subjected to a load controlled bending regime than a deformation controlled loading regime. It was observed that the strength difference between the two loading regimes was large (about 30%) at the weak end of the strength distribution where brittle fracture modes predominate but that the strength difference disappeared at the strong end of the distribution where more ductile compression failure occur. .

APPLICATION OF BRITTLE FRACTURE THEORY TO WOOD

The first study in which the Weilbull brittle fracture theory was applied to wood was reported on by Bohannan (1966). He studied clear wood beams and found that for geometrically similar beams the strength was proportional to the depth of the beam to the power 1/9, this being the result of a depth effect and a length effect of a similar magnitude. He found that strength was not affected by beam breadth.

Barrett (1974) used the weakest link theory in tension perpendicular to grain studies which were subsequently used to explain cracking phenomenon which occurred in pitched cambered beams and a design method based upon that work is included in CSA-086.

Foschi and Barrett (1975) also applied the weakest link theory to the shear strength of glulam beams and developed a design formula which was incorporated into CSA-086.

Buchanan (1983) used brittle fracture theory to relate the strength of wood in bending to the strength in axial tension. For clear wood he found that the effect of varying length was approximately the same for the two testing modes but that the effect of varying depth was much greater for tension members than for bending members.

BRITTLE FRACTURE THEORY MODIFIED FOR TIMBER

In the above mentioned studies it was assumed that wood could be regarded as "perfectly brittle" material and that the weakest link theory could be applied in its simple two-parameter form.

Some modifications to the theory is necessary to allow for different magnitudes of size effects with length, depth and breadth.

It is even more difficult to apply brittle fracture theory to lumber, because lumber tends to have a small number of large defects compared to wood with a large number of small defects.

Results of lumber testing show size effects of different magnitudes in bending and in tension. Size effects are also observed in axial compression, even though compression failures are relatively ductile. These effects are less in compression than in axial tension or bending but they are still significant.

As with other materials, size effects in lumber are related to variability in strength properties within each member. For more homogeneous materials such as cement, metals or even concrete, the variability in material properties tends to be the same in all directions within a member and between members. The size effects can therefore be quantified with a single size effect parameter.

However lumber is very different because it is anisotropic. A failure in the clear wood portion would be governed by a flaw (weakest link) on the microscopic level. It may be a weak cell wall or a deflective connection between cells. Superimposed on that system of flaws are the effects of the grain deviations around knots. They represent a set of flaws on the macroscopic level.

While it may be reasonable to assume that the flaws on the microscopic level are randomely distributed throughout the volume of a member, the flaws on the macroscopic level have a distribution along the length of the member which is different from its distribution within a cross section. Furthermore, the effects of the flaws are different depending upon the direction of the stresses. For these reasons it is advantageous to use a number of different parameters to quantify the different size effects, at least initially.

Another assumption commonly used by others, and followed in this paper is that the lower bound for the strength of a flaw is zero. This leads to the two-parameter Weibull distribution formulation. This assumption implies zero strength for infinite size which may be appropriate for the lower grades of lumber but may well be unreasonable for the highest grades. The assumption is, if anything, conservative and is considered to be reasonable provided that the theory is not extrapolated far beyond the range of the test data.

SIZE EFFECTS

The size effects to be considered in this study are described below. The formulae shown for quantifying the effects have been developed in Appendix A using the weakest link theory with the assumption of zero strength for infinite size.

a) Length effect

The term length effect refers to the phenomenon that when boards of different lengths are tested under similar loading condition, long boards tend to be weaker than shorter boards. Using brittle gracture theory this phenomenon can be described by :

$$\frac{x_1}{x_2} = \left(\frac{L_2}{L_1}\right)^{1/k_I} \qquad (1)$$

where x_1 and x_2 are the strength of members of length L_1 and L_2 respectively, k_1 is the length effect parameter. The change in strength for doubling the length can be obtained by setting $L_2/L_1 = 0.5$. The resulting expression has been plotted in Fig. 2. Here the ordinate is the remaining strength in a piece of length 2L as a proportion of a piece of length L. The abscesia is the k_1 value as well as the coefficient of variation (c.v) for a two parameter Weibull distribution. If k_1 is greater than 10 (c.v = 12) the strength is only reduced slightly. However, as k_1 becomes smaller (c.v larger) the effect of doubling length becomes severe and for $k_1 = 2.5$ (c.v = 0.42) only 75% of the strength remains.

The parameter k_1 tends to have similar values in bending and tension. Axial compression tests do not generally result in brittle fracture but the weakest link description can nevertheless be applied because the failure still occurs in the weakest cross section. The length effect factor k_1 will have quite different values in tension and compression.

b) Load Configuration Effect

The "load configuration effect" describes the phenomena that the strength of a bending member depends on the proportion of the members length subjected to high stresses.

For example, a single concentrated load gives higher failure stress than two concentrated loads spread apart.

The load configuration effect can be quantified by converting the specific type of loading to an equivalent length and inserting in (1). The formula developed in Appendix A is :

$$L_e = \frac{1 + \frac{a}{L} k_1}{1 + k_1} L \qquad (2)$$

where L_e is the equivalent length and a is the distance between two concentrated loads. Determination of the value of k_1 is covered later in this paper.

c) Depth Effect

The term "depth effect" is used where beams of the same length but different beam depths show different strengths. The formula derived from brittle fracture theory is :

$$\frac{x_1}{x_2} = \left(\frac{d_2}{d_1}\right)^{1/k_2} \qquad (3)$$

where x_1 and x_2 are failure stresses of members of depth d_1 and d_2 respectively. k_2 is the depth effect parameter and it cannot be assumed that it has the same value as k_1. Determination of the value of k_2 is covered later in this paper.

d) Stress Distribution Effect

The "stress distribution effect" describes the phenomenon that strength depends on the distribution of stresses at a cross section. For example, the failure stress in bending is usually much higher than the failure stress in axial tension. In a beam it is only the outer fibres which are subjected to high stresses and the probability of a flaw occuring here is less than had the whole cross section been subjected to the same high stress. Again using brittle fracture theory the failure stress in the extreme tension fibre f_m can be related to the failure stress in an axial tension test f_t by :

$$f_m = \left(\frac{1 + k_3}{c}\right)^{1/k_3} f_t \qquad (4)$$

where c is the neutral axis location (as a proportion of the beam depth, measured from the tension face), and k_3 is the stress-distribution parameter. In theory the value of k_3 should be the same as k_2 but in practice these two parameters may be different..

For the common case of bending with the neutral axis at mid-depth, equation (4) becomes :

$$f_m = 2\,(1 + k_3)^{1/k_3}\, f_t \qquad (5)$$

and f_m is now the modulus of rupture.

To obtain f_m from f_t or vise versa, the value of k_3 would obviously be needed. In practice however, the values of f_m and f_t are each obtained independently, so the value of k_3 can be calculated from them, but it is not normally required in design.

e) Breadth Effect

The brittle fracture theory suggests that failure stress should decrease with an increase of the breadth of the member. The formula would be :

$$\frac{x_1}{x_2} = \left(\frac{b_2}{b_1}\right)^{1/k_4} \qquad (6)$$

where x_1 and x_2 are the failure stresses of members of breadth b_1 and b_2 respectively. k_4 is the breadth effect parameter.

However the experimental work of Bohannan (1966) could not verify the existence of a breadth effect. Tests by Madsen and Stinson (1982) showed that for beams with the same depth, but of varying breadth, and tested over the same span the strength increased as the breadth was increased. This is opposite to the predictions of the brittle fracture theory. A satisfactory explanation for this anomaly has not been found but it could, in part, be associated with the grading rules. The breadth effect is not considered further in this paper.

f) Length effect for Members in Series

All of the discussion of size effects, to this point, has been concerned with strength of populations of single members. These size effects have occured as a result of the variability in the strength of the constituent elements within each member.

For certain structural elements (bottom chords of long span trusses for example) several members are connected in series. A length effect will be present here too, this time, as a result of the variability in strength between boards.

This multiple-member length effect can be quantified as before, re-arranging equation 1 to give :

$$R_n = n^{-1/k_5} \tag{7}$$

where R_n is the reduction factor for n members in series and k_5 is the multiple member parameter. The value of k_5 can be obtained from the coefficient of variation (c.v) of test results using :

$$k_5 = c.v^{-1.22} \tag{8}$$

The formulation assumes that the distribution of member strength can accurately be modelled by a two-parameter Weibull distribution. If a three-parameter distribution is more appropriate, equation 7 and 8 will be conservative and a more accurate expression is :

$$R_n = a + (1 - a)n^{-1/k_5} \tag{9}$$

where k_5 is the shape parameter of the three-parameter Weibull of member strength, and "a" is the location parameter expressed as a ratio of the strength at the quantile of interest, q.

$$a = \frac{x_o}{x_o + m\,(-\ell n\,(1 - q))} \tag{10}$$

x_o and m are the location and scale parameters, respectively, of the distribution.

g) Effects of Grading Rules

When investigating size effects in graded lumber, it is very difficult to seperate the effect of member size from the effect of grading rules.

Consider for example the effect of member depth. Some defects are limited to a certain size, independent of member depth, while others are permitted to be a certain percentage of the depth, so strength differences between different sizes can be affected by the way in which grading rules control such defects.

A similar problem exists with member length. When lumber of a certain grade is cut into short pieces to investigate length effects, many of the short pieces become of a higher grade than the original piece because there may be only one limiting defect in a given long board. Conversely, if a defect (such as wane) is limited to some percentage of the member length, then cutting into smaller lengths may down-grade a board. For these reasons, investigations into size effects using commercial lumber must be reported and interpreted carefully. In cases where the size effect is found to be independent of strength, the same size effect factor can of course be applied to all the grades. In cases where the size effect factor appears to be strength dependent, a different appropriate value of k could be chosen for each grade using the appropriate characteristic strength of the grade.

SUMMARY

To this point, the paper has reviewed the conventional size effect theory for perfectly brittle materials. The theory has been modified and improved for application to lumber, with size effects in different directions receiving seperate consideration. The above will be used as a base for interpretating available test results.

III EXPERIMENTAL WORK

Three test programs will be described in this section of the paper, all designed to elucidate different aspects of the size effect phenomena caused by brittle fracture in bending of lumber. They are :

a) Size investigation of lumber (1976)

b) Load configuration effects (1983)

c) Size effect study, In-Grade testing program (1978)

The studies a and c have been described earlier and only an outline of these tests is presented here. However, study b has not been reported on previously and is therefore described in greater detail. Experimental work on tension properties will be covered later.

a) Size Investigation of Lumber (1976)

These tests were conducted by and reported on by Madsen and Nielsen(1976). That report merely describes the test methods and draws experimental conclusions without attempting to identify the possible underlying causes for the findings.

Fig. 3 shows the extent of the tests conducted.

Four differents sizes of lumber (38 x 89, 38 x 140, 38 x 184 and 38 x 235 mm) were tested at different span-to-depth ratios as shown. The sample size used in each test was about 100. The material used was Hem-Fir (H-F) of "No. 2 and Better Grade". It was kiln dried and cut to the required length. About half of the lumber was of select structural (SS) grade. The data have been analysed according to grade (SS and No 2) but that division caused a sample size of about 50 which is marginal for material with this degree of variability. Some material of No 3 grade was included in the test samples but it was eliminated in the data analysis. The data for the whole sample (No. 2 and Better Grade) is more conclusive. All the tests were conducted with the loads at the third points. From these tests it is possible to estimate the length effect separately from the depth effect.

b) Load Configuration Effects (1983)

The objective of this test was to investigate the load configuration effects for both simple support conditions and "fixed ends". The tests are described in more detail since the data have not been reported elsewhere.

Testing Program

Bending tests were conducted using matched samples of one cross section size of lumber (38 x 140 mm) under several loading conditions. These included four span to depth ratios, three load configurations and two different end conditions.

Most of the tests were carried out at span-to-depth ratios of 22:1 and 11:1. However, supplementary tests were performed at 17:1 in order to tie into existing in-grade test results and also at a 30:1 ratio to investigate very long beams.

The three load configurations consisted of a central point load, two symmetrical loads L/4 apart and two symmetrical loads L/3 apart, L being the span. Most of the loading conditions described were tested twice, once with simple supports and once with the beam ends restained to prevent rotation ("fixed ends").

A sample size of about 100 boards was used for each test condition resulting in a total of approximately 3 200 boards. All the tests were carried to failure.

The testing program is illustrated in Fig. 4.

Material

The lumber was of the spruce-pine-fir (S-P-F) species group. This is the most voluminous species group and therefore of greatest importance to the Canadian economy. As mentioned, only one size (38 x 140 mm) was used but both select structural grade and No 2 grade material was obtained because the size effects were believed to be grade dependent.

The material was obtained in two lengths 3.0 m and 4.27 m, the shorter boards intended for span-to-depth ratios of 11:1 and 17:1, the longer for 22:1 and 30:1. Two tests were conducted at one span using the two different lengths of boards to see if there was a substantial strength difference due to the length of the boards. No significant difference was found and the tests could therefore be compared without adjustment for the length of the original board.

It was intended that No 2 grade material should consist of pieces which had been downgraded due to the presence of knots or slope of grain, but the sawmill was unable to supply sufficient material of that specification and ordinary No 2 grade material was received. Many of the pieces had been downgraded due to wane and other defects which are not believed to affect strength. The select structural grade did not cause supply problems.

Upon arrival at the laboratory the long span edgewise modulus of elasticity (MOE) was established for each piece. The pieces were then sorted into samples of 100 using the MOE value as a strength indicator. This was done in order to obtain similar strength distributions for each sample. Each sample was then randomly assigned to one of the test configurations. The material had moisture content in the range of 11% to 14%.

Testing

A Tinius Olson test frame was used. It was equipped with a MTS actuator operating in the load controlled mode. The rate of loading was selected such that failure would take place within one minute. For the tests with simple end conditions the bearing plates could rotate and one could additionally translate in the direction of the board. The load was applied through plates which could both rotate and translate. The bearing plates were large enough to prevent indentation due to compression perpendicular to the grain forces.

For the fixed end tests special shoes were built to clamp the ends. One could translate along the length of the piece without rotating while the other was stationary. After some tests had been conducted it was decided to improve the fixed end grips so that the developing end moments could be recorded. This was accomplished by incorporating load cells as shown load cells as shown in Fig. 5.

The in-grade testing equipment developed at U.B.C. (Madsen and Nielsen 1978a) was also used for some tests in order to tie these new tests to previous work.

Axial tension tests at two different lengths were also carried out using the in-grade tension testing machine developed at U.B.C. (Madsen and Nielsen, 1978b).

Additional tests were conducted using the same material to investigate the effect of load control vs. displacement control on strength. This work, reported by Madsen and Mindess (1984) has been referred to earlier in this paper.

Data

During the testing the following information was recorded and placed in the data acquistion system :

Dimensions

Moisture content

Grade determining defect

Grain orientation

Mode of failure

Type of defect

Location of failure

Failing load

End moments

c) In-Grade Testing (1978)

While this program reported on by Madsen and Nielsen (1978a) was primarily concerned with obtaining characteristic strength values for lumber of different species and grades, some tests were included to illustrate the size effect. It is that portion of the testing program which is of interest for this paper.

Lumber of 38 x 89, 38 x 140, 38 x 184 and 38 x 235 mm size was tested for the three major Canadian species Douglas-Fir (D-F), H-F and S-P-F. The tests were performed on four grades : SS, No 1 grade, No 2 grade and No 3 grade. For each sample consisting of about 200 specimens the 5th percentile was obtained using a proof loading method whereby only 10 to 15% of the pieces failed.

All the tests were conducted with the loads applied at the third points of the span. The span to depth ratio was 17:1 for all the tests. The extent of the testing dealing with size effect is shown in Fig. 6.

The 1978 tests can provide no independent information on values of k_1 and k_2. However, once k_1 has been determined from the 1976 and 1983 results, the 1978 results can be used to estimate values of k_2.

IV DATA ANALYSIS FOR BENDING

In this section of the paper, the results of the above experimental test programs are analysed to quantify the length effect parameter k_1 and the depth effect parameter k_2.

Values of k_1 will be investigated first, using the following data :

1) 1976 test results for different spans, all with loads at the third points of the span.

2) 1983 test results, looking at the effect of doubling the length but keeping the load configuration constant.

3) 1983 test results again, this time looking at the effect of different load configurations on a constant span.

4) 1978 test results for different sizes tested at a constant span to depth ratio, making the assumption that there is no depth effect. This assumption will be discussed later.

The values of k_1 vary considerably. Reasons for that will be discussed later.

<u>Length Effect</u>

This section describes how the data were analysed and will be followed by a discussion once all the data have been presented.

The most comprehensive data available is that from 1976. Here different sizes were tested at different span to depth ratios as shown in Fig. 3. The theory tells us that the logarithm of strength should be linear related with logarithm of length and that the slope of the line is $-^1/_{k_1}$. Such plots were made separately for each size and the values of k_1 and as well as the correlation coefficient (r) are listed in Table I. The calculations were done for the 5th and 50th percentiles to determine whether k_1 changes with strength. Most of the values are based upon four data points (different span to depth ratios) only the 38 x 235 mm size had 3 data points. The analysis was done for Select Structural grade alone, No 2 grade, and all grades combined. The k_1 values range from 3.1 to 10.5, most being between 3.5 and 6.5.

The 1983 tests also provide information on the length effect. Here sets of geometrically similar load configurations have been tested at two different spans and they can therefore provide direct estimated of the k_1 value. Three tests were conducted at spans of 1540 mm and 3080 mm using the following load configurations :

a) Single concentration load in centre of span

b) Two concentrated loads $^L/4$ apart

c) Two concentrated loads $^L/3$ apart.

From these tests, doubling the span for each load configuration, 12 estimates of k_1 were obtained (6 for the 5th percentile and 6 for the 50th percentile) as shown in Table II. With one exception, all the values range from 2.9 to 7.8.

It is also possible to get estimates of the k_1 value from the different load configuration tests conducted at the same span. According to the theory an effective length can be calculated by formula (2) for two different load configurations and a comparison of these effective lengths can be used to estimate k_1. An iterative method was used. An initial value of k_1 was chosen and the effective lengths established from (2) and inserted into (1). The process was repeated until the value of k_1 was the same in the two formulae. The results are shown in Table III. In some cases the process did not converge and an estimate could not be found. If the estimate was less than unity or greater than 10, the estimate was disregarded when calculating averages.

The differences in load configurations between $^L/4$ and $^L/3$ is not large and the slope of the line is therefore not well defined as indicated in the table. However, the final averages are consistent with those obtained above.

The 1978 In-Grade tests were conducted with a constant span to depth ratio, with both depth and length being varied simultaneously. The tests were originally interpretated as exihibiting a depth effect (which has been introduced into the CSA-086-1984 design code). However, the observed strength difference between the different depths could just as well have been ascribed to a length effect but that was not considered at the time. Let us make the assumption that all the strength differences are a length effect. Then using formula (1) in conjunction with (3) it is possible to find a value k_1, such that k_2 (the depth effect) tends to infinity or stated differently, that the slope in a plot of strength vs. depth becomes zero. The values from that exercise are shown in Tabel IV.

Variability in Results

The estimates of k_1 (summarized in Table V) vary considerable and contributing reasons for the variability are :

a) Lumber strength is highly variable. In view of the large number of factors affecting lumber strength one would expect some difficulty in making precise estimates of a statistical phenomenon such as size effects.

b) The sample sizes are relatively small particulary when the tests are broken down into the two seperate grades (Select Structural Grade and No 2 Grade) so the confidence in the estimates on the 5th percentile is less than desirable.

c) The weakest edge was placed in the tension zone in the 1976 tests while in the other tests a random placement was used.

d) Material from three species groups was used, obtained from different suppliers.

e) The 1976 tests were loaded using a constant rate of deformation while the 1983 tests were load controlled.

Despite these shortcomings it is still possible to establish some important trends.

Depth effect versus length effect

The question of whether the observed strength differences at constant span to depth ratio are caused by a depth effect (as presumed in earlier studies) or by a length effect alone is central to the further interpretation of the data and will therefore be addressed first.

Only the 1976 data can provide direct information on that question. The test data (5th & 50th percentiles for the combined grades) are shown in Fig. 7. All the tests were conducted using third point loads. The figure shows clearly that data points from the three larger sizes ie. 140, 184 and 235 mm are intermingled and that no consistent depth effect exists amongst these sizes.

The data points for the 89 mm size seem to be located slightly below the rest of the data cluster. Two of the points (1.06 and 3.35 mm) however, could well belong to the data cluster formed by the other data points and the diviation of the two intermediate points could be caused by chance.

On the other hand the difference in the 89 mm data could be real since that material is often cut from a different place in the log than the rest of the sizes. For the purpose of this analysis it is judged that the data points for the 89 mm size can be considered to belong to the same data cluster formed by the other sizes and it is, with the above caviat, concluded that a significant depth effect does not exist. The observed differences in strength is therefore attributed to a length effect.

Strength Dependency

To investigate whether the length effect is strength dependent, the k_1 values calculated for the 5th, 50th and 95th percentiles were compared. In general the k_1 values for weak material were less than for stronger material. The spread in the data was large, with a correlation coefficient of 0.7, but the slope was distinct and indicates that the length effect is strength dependent. This observation is reinforced by the notion that the strength of timber is governed by brittle type failures in the weak end of the strength distribution while compression failures are dominating for stronger material.

The strength dependency of the length effect was observed over a wide range of strength (15 - 45 MPa). The differences between the 5th percentiles for the different grades is smaller (15 - 24 MPa) and the strength dependence not as pronounced. The average k_1 value for Select Structural grade is 3.7 while it is 3.4 for No 2 grade. It can therefore be concluded that the grade dependency is small.

Species Dependency

The 1976 tests were conducted with Hem-Fir and the average values for k_1 is 4.6. The 1983 test conducted with S-P-F shows an average value of 3.3. The In-Grade tests also show differences in k_1 for the different species and additionally the values are generally lower than observed with the laboratory tests except for No 2 Grade Hem-Hir. The value of $k_1 = 8$ is much higher than the rest. The Hem-Fir In-Grade test values are based upon only four experiments so the confidence may not be high. One of the four values (140 mm size) is in fact abnormally high.

Similarly the k_1 value for Select Structural grade for Douglas Fir is very low in the In-Grade tests. In that case the 89 mm tests give 5th percentile values higher than observed from other tests with 89 mm material.(H-F & S-P-F).

There is a distinct possibility that the k_1 factors could be species dependent. This could well be related to the observation that lumber from some species such as Douglas Fir is characterized by few but large knots while other species, notable spruce produce lumber with many but smaller knots. This could influence the degree of brittleness and thus the k_1 values.

Influence of Structural Redundancy

The tests conducted with "fixed end conditions" were not included in the initial data analysis because the failures are more complicated. The stress distribution along the length of the board is not well known due to the structural indeterminancy. Full fixity at the ends was not attained because local compression prependicular to the grain deformation occured at the support points despite the long bearing plates used. The deformations and therefore the rotation were most severe for the stronger boards. Information on the magnitude of the end moments was obtained in six of the tests by inserting a load cell in the clamps.

The mode of failure was different for the fixed ended tests in that the failures often appeared to develop in two or more places simultaneously and the boards scattered into 3 or more pieces.

Despite these limitations it is still possible to make some useful observations from the tests.

The effective length L_e was calculated for all the tests (fixed ended as well as simply supported) and graphs of logarithm of strength vs. logarithm of L_e were prepared for the two grades each for the 5th percentile and the 50th percentiles. Fig. 8 is an example. The effective length was calculated from a formula developed by A I Johnson :

$$L_e = \frac{(L - a)(L + a)}{2L\ (k_1 + 1)} + \frac{(L - a)^2}{2L\ (k_1 + 1)} \quad \frac{(L - a)}{(L + a)} + a\,\frac{(L - a)}{(L + a)} \qquad \text{(ii)}$$

L = span

a = distance between two symmetrically placed loads

k_1 = length effect parameter

The formula presupposes a perfectly brittle material behaviour. The value used for k_1 was 3.5.

For the No 2 Grade shown in Fig. 8 very good correlation coefficients were obtained for both the simple support conditions and the "fixed ended" tests. However, the select structural grade did not produce equally good results for the "fixed ended" tests. The correlation coefficient was about 0.7 and the slope for the fixed ended points was less.

Further the slope diminished with increased strength. It would thus appear that the brittle fracture phenomena is less pronounced where structural redundancy exists and that the length effect is less severe.

It might be speculated that the more knotty material (No 2 Grade at the 5th percentile range) behaves more like a brittle material because the knot frequency is sufficiently high to influence the strength even at the short spans, while the Select Structural Grade has a knot frequency which will only cause brittle failures to occur at the longer spans.

V DESIGN CONSIDERATIONS

a) General

The present CSA-086 code (1984) operates with a depth effect for the design of sawn bending members. The designer has to calculate the bending moment and then guess a size in order to obtain the strength to be used to calculate the required section modulus. If his initial guess is not correct he has to make a new guess and try again.

The current strength properties were obtained by testing representative material at a span to depth ratio of 17:1, which is reasonable because most beams will have span to depth ratios in the range of 12 - 25.

However, this paper demonstrates that both a length effect and a load configuration effect exist and that they are of sufficient magnitude to be of importance for design. To merely superimpose these effects on the present depth concept would lead to a very complicated design procedure. It may be useful therefore to take a different approach.

It is suggested that a standard beam configuration should be adopted as the base for referencing strength. A 3.0m beam with loads at the third points would be suitable. The 3.0 m span is suggested because it is roughly at the middle of most applications for sawn lumber and adjustments to other lengths are therefore minimized. The third point loading is suggested because it gives a distribution of moments close to that of a uniformely distributed load. It should be noted that the standard beam is suggested for representing strength only and it is not necessary to conduct all bending tests with that particular span and load configuration since a conversion to the standard can be made.

Fig. 9 shows an example of converting test results to the proposed standard beam configuration using the modified weakest link theory and $k_i = 3.5$. Column 1 shows the original 5th percentile values obtained from the tests, representing our present interpretation. The values vary from 23.2 MPa to 14.5 MPa. However, by applying the proposed load configuration effect the values in Column 2 emerges. The values for the 1.55 m span are now very close to each other and so are the values for the 3.1 m span. Applying the adjustment for length effect we obtain the numbers shown in Column 3. The values for the different tests are now very similar giving us an uniformity between the results not previously attainable.

The code would publish design values for the relevant species groups and grades. In addition a set of conversion factors would be provided to encompass depth, length and load configuration effects. The designer knows the length and load configuration when he establishes the moment and he merely has to look up one adjustment factor which he applies to the published strength in order to obtain the required section modulus.

b) Precission

While the proposed design method is conceptionally much simpler than what is used today it could become complicated if the precision of the k_i values has to be high. As indicated the k_i values may be both grade dependent and species dependent and this could lead to a cumbersom set of adjustment factors.

All the tests taken together represent a wide variety of loading conditions from very short spans to very long spans each with several loading configurations. It is therefore possible to assess the effect on an overall basis.

Two extreme cases are investigated.

i) The k_1 values calculated and shown in Tables I - IV. This set would be the "best possible" case because the k_1 values were calculated for the particular test. It also represents the most complicated case and will be used as a bench mark to which the other case can be compared.

ii) A single value of $k_1 = 3.5$. This would be the simplest solution from a design point of view.

All the test results are now converted to the standard beam configuration using k_1 values from i and ii.

Before combining the test series it is however necessary to eliminate the variability caused by different 5th percentile for the different species. This is accomplished by normalizing the individual tests data to the mean 5th percentile for the series.

The normalized values are ranked and the distribution of all the tests established. Fig. 10 shows case i as a solid line and case ii as a dotted line. The differences between the two cases is not great leading to the conclusion that for engineering purposes it is sufficiently precise to use a single value $k_1 = 3.5$.

Fig. 11 compares the present manner of interpretating test data (solid line) with the suggested method incorporating size effects (dotted line). In an "ideal world" all the points should be located on a vertical line through $x = 1$. However, variability caused by : small sample sizes, test method, method of interpretation etc., causes deviations from that line.

The improved method of interpretation reduces the overall variability to about half.

c) Design Method

The Code should provide strength data (allowable stress or characteristic values) for relevant species groups and grades, just as it has done in the past. However, the value would be defined as the strength of the mentioned standard beam configuration (3.0 m span ; 1/3 point loading). In addition the code would contain a table or graphs quantifying the length adjustment factor. A suggested layout is shown in Fig. 12. A variety of load configurations are shown together with values of the length adjustment factor for different spans. The table has been prepared for a value of k_1 = 3.5 and similar tables could be produced for other values of k_1 should that be deemed necessary. The basic formulae are also shown for use for other cases than those covered in the Fig. 12.

The designer faced with a specific case will know the span and load configuration and he can calculate the moment. With that information he can look up the appropriate length effect factor which is to be multiplied with the strength of the selected species and grade chosen for the job.

The section modulus can be established from :

$$S = \frac{M}{f_B \times K_L}$$

f_B = strength

K_L = length adjustment factor

The proposed method is rather simple and accounts for depth effect, length effect and load configuration effect in a single operation.

VI DATA ANALYSIS FOR AXIAL TENSION

Depth Effect

Axial tension testing of a large number of pieces of 38 mm thick lumber of different depth (width) but constant length has identified a very significant depth effect.

Madsen & Nielsen (1978b) obtained consistant results for several species and grades with an average k_2 value of 5.2 at the 5th percentile. The design stresses in the Canadian code are based upon these tests.

Johnson and Kunish (1975) found an even greater depth effect in a more limited series of tests with commercial sizes of lumber. These results are in sharp contrast to the bending results presented above where no significant depth effect occurs.

The reasons for this difference in behaviour between tension and bending members are not clear. They probably include different failure modes, different stressed volumns, strain energy and fracture mechanics effects. Some of these items have been discussed by Buchanan (1983).

Length Effect for Single Members

Very little data is available on length effect in tension. As part of the 1983 bending test program described above, axial tension tests were carried out on boards of two lengths of No 2 Grade S-P-F lumber of size 38 x 140 mm.

The long (3.0 m) boards had only 67% of the strength of the short (0.91 m) boards, which gives a length effect parameter of $k_1 = 3.0$.

In another study, Buchanan (1984) tested similar 38 x 89 mm boards 2.0 m and 0,91 m long. The strength difference was less in this case, giving a value of $k_1 = 8.3$.

In the absence of any other test data, it appears that the value of $k_1 = 3.5$ obtained for bending could also be used for tension members.

Length Effect for Members in Series

The theory for length effect for members in series has been given earlier.

Tension strength is a very variable property, with coefficient of variation up to 40% often being recorded. Using equation 8 this gives a value of $k_5 = 3.1$ and hence $R_5 = 0.8$ for two members in series.

When a three parameter Weibull distribution is fitted to tension strength result typical values are $k_5 = 2.7 - 2.9$ and $x_o = 0.6$ times the 5th percentile value. These numbers are, for the time being, suggested as suitable for code purposes, giving R_n values as shown in Table VI using $k_5 = 2.8$.

VII SUMMARY AND CONCLUSIONS

Various size effects observed in lumber have been explained by a modified weakest link theory. The modifications to the traditional weakest link theory consist of assigning different size effect parameters to the different directions of the anisotropic material.

Some of the salient parameters have been quantified based upon analyses from several large testing programs.

While the estimates of the size effect parameters varied considerably a sensitivity analysis showed that the value of the length effect parameter $k_1 = 3.5$ gave sufficient precision for engineering purposes.

The concept allows comparisons of test with different spans and/or load configuration to be made resulting in greater conformity between them than previously possible.

A standard beam configuration (3.6 m span, 1/3 point loading) is suggested to be used as the base for referencing bending strength of lumber.

A design method is proposed for inclusion in timber design codes which takes into account both length effect and load configuration effect in a simple manner.

Tentative suggestions are also made for design of tension members but further work in this area is urgently needed.

ACKNOWLEDGEMENT

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TABLE I

k_1 VALUES OBTAINED FROM SLOPE ANALYSIS
(SLOPES IN BRACKETS)

1976 DATA SPECIES : HEM-FIR

$f_{0.05}$		SS GRADE		SS + No 2		No 2 GRADE	
SIZE	N	r	k_1	r	k_1	r	k_1
38 x 89	4	0.82	3.1 (0.323)	0.80	3.1 (0.323)	0.60	4.5 (0.222)
38 x 140	4	0.67	6.5 (0.154)	0.79	4.5 (0.222)	0.96	5.5 (0.182)
38 x 184	4	0.67	4.1 (0.244)	0.64	3.5 (0.286)	0.77	5.3 (0.189)
38 x 235	3	0.99	4.7 (0.213)	0.93	4.0 (0.250)	0.25	10.5 (0.095)
AVERAGE SLOPE			0.234		0.270		0.198
k_1			4.3		3.7		5.1

$f_{0.50}$		SS GRADE		SS + No 2		No 2 GRADE	
SIZE	N	r	k_1	r	k_1	r	k_1
38 x 89	4	0.88	7.8 (0.128)	0.92	5.0 (0.200)	0.76	6.6 (0.152)
38 x 140	4	0.88	6.2 (0.161)	0.77	5.3 (0.189)	0.32	15.8 (0.063)
38 x 184	4	0.93	4.6 (0.217)	0.83	4.4 (0.227)	0.58	6.1 (0.164)
38 x 235	3	0.52	8.2 (0.122)	0.86	6.5 (0.154)	0.73	6.3 (0.159)
AVERAGE SLOPE			0.157		0.193		0.158
k_1			6.4		5.2		6.3

TABLE II

k_1 VALUES OBTAINED BY SLOPE ANALYSIS

1983 Data   Species : S-P-F   Size : 38 x 140 mm

	MID		$L/4$		$L/3$	
	$f_{0.05}$	$f_{0.50}$	$f_{0.05}$	$f_{0.50}$	$f_{0.05}$	$f_{0.50}$
SS GRADE	2.9 (0.345)	6.5 (0.154)	6.8 (0.147)	7.8 (0.128)	$k_1 > 10$	4.9 (0.204)
No 2 GRADE	3.6 (0.278)	3.2 (0.313)	4.0 (0.250)	3.7 (0.270)	2.9 (0.345)	3.7 (0.270)

SS $f_{0.05}$: $\overline{1/k_1} = 0.246$; $k_1 = 4.1$

SS $f_{0.50}$: $\overline{1/k_1} = 0.162$; $k_1 = 6.2$

No 2 $f_{0.05}$: $\overline{1/k_1} = 0.291$; $k_1 = 3.4$

No 2 $f_{0.50}$: $\overline{1/k_1} = 0.284$; $k_1 = 3.5$

TABLE III

k_1 VALUES OBTAINED FROM LOAD CONFIGURATIONS

1983 DATA SPECIES : S-P-F SIZE : 38 x 140 mm

		L = 1540 mm $^{L}/_{d}$ = 11:1			L = 3075 mm $^{L}/_{d}$ = 22:1		
		MID → $^{L}/_{3}$	$^{L}/_{3}$ → $^{L}/_{4}$	MID → $^{L}/_{4}$	MID → $^{L}/_{3}$	$^{L}/_{4}$ → $^{L}/_{3}$	MID → $^{L}/_{4}$
$f_{0.05}$	SS GRADE	$k_1 < 1.0$	$k_1 < 1.0$	5.4 (0.185)	7.2 (0.139)	$k_1 < 1$	$k_1 > 10$
	No 2 GRADE	2.8 (0.357)	1.0 (1.000)	4.3 (0.236)	1.3 (0.769)	$k_1 < 1$	6.3 (0.159)
$f_{0.50}$	SS GRADE	7.6 (0.132)	$k_1 > 10$	6.3 (0.158)	4.7 (0.213)	$k_1 < 1$	8.8 (0.114)
	No 2 GRADE	5.0 (0.200)	9.5 (0.105)	4.0 (0.250)	7.3 (0.137)	$k_1 > 10$	6.6 (0.152)

SS $f_{0.05}$: $\overline{1/k_1} = 0.162$; $k_1 = 6.2$

SS $f_{0.50}$: $\overline{1/k_1} = 0.154$; $k_1 = 6.5$

No 2 $f_{0.05}$: $\overline{1/k_1} = 0.504$; $k_1 = 2.0$

No 2 $f_{0.50}$: $\overline{1/k_1} = 0.169$; $k_1 = 5.9$

TABLE IV

k_1 VALUES OBTAINED BY SLOPE ANALYSIS

1978 IN-GRADE TEST DATA

	DOUG-FIR		HEM-FIR		S-P-F	
	SS	No 2	SS	No 2	SS	No 2
$f_{0.05}$	1.6 (0.625)	2.3 (0.434)	2.4 (0.417	8.0 (0.125)	3.2 (0.313)	2.8 (0.337)

TABLE V

SUMMARY OF k_1 VALUES

$f_{0.05}$	DOUG-FIR		HEM-FIR		S-P-F	
	SS	No 2	SS	No 2	SS	No 2
LENGTH EFFECT			4.3 (0.234)	5.1 (0.198)	4.1 (0.246)	3.4 (0.291)
LOAD CONFIG.					6.2 (0.162)	2.0 (0.504)
IN-GRADE	1.6 (0.625)	2.3 (0.434)	2.4 (0.417)	8.0 (0.125)	3.2 (0.313)	2.8 (0.357)

AVERAGES : FOR $f_{0.05}$

HEM-FIR	:	$1/k_1 = 0.244$	$k_1 = 4.1$
S-P-F	:	$1/k_1 = 0.312$	$k_1 = 3.2$
DOUG-FIR	:	$1/k_1 = 0.523$	$k_1 = 1.9$
SS	:	$1/k_1 = 0.274$	$k_1 = 3.7$
No 2	:	$1/k_1 = 0.295$	$k_1 = 3.4$

	DOUG-FIR		HEM-FIR		S-P-F	
	SS	No 2	SS	No 2	SS	No 2
LENGTH EFFECT			6.4 (0.157)	6.3 (0.158)	6.2 (0.162)	3.5 (0.284)
LOAD CONFIG.					6.5 (0.154)	5.9 (0.169)

TABLE VI

REDUCTION FACTORS FOR TENSION MEMBERS IN SERIES

n	1	2	3	4	5
R_n	1.0	0.91	0.87	0.84	0.80

APPENDIX A - BRITTLE FRACTURE THEORY

Theory for Uniform Stress Distribution

For a relatively simple explanation of conventional brittle fracture theory, consider several identical members subjected to uniform axial tension stress.

Assume that the material is "perfectly brittle" such that the strength of a member is determined by the strength of its weakest element. An alternative conceptual approach which produces the same results, is to consider a homogeneous brittle material containing a large number of defects with a statistical distribution of size. In this case the strength of the member is determined by the size of the largest defect.

If each member consists of a large number of brittle elements, selected at random from a parent population of elements with a cumulative distribution function (C.D.F.) of strength given by a Weibull distribution.:

$$F(x) = 1 - \exp\{ -(\frac{x - x_o}{m})^k \} \tag{A1}$$

where x is the strength, x_o is a lower limit or minimum strength called the "location parameter", m is a "scale parameter" with the same units as x, and k is a dimensionless "shape parameter" which reflects both the skewness and the spread of the distribution.

If each member contains n elements, it can be shown (Bury, 1975) that the c.d.f. of the weakest element in each member (and hence the c.d.f. of member strengths) is given by :

$$F(x) = 1 - \exp\{ -n(\frac{x - x_o}{m})^k \} \tag{A2}$$

where m in equation A1 has been replaced by $mn^{-1/k}$

The simplest example of a "weakest link" material is a chain, in which failure of the weakest link causes failure of the chain. If a chain factory produces links which have a distribution of strength described by equation A1, then a number of chains, each containing n links, will have a distribution of strength given by equation A2.

Equation A2 can be rearranged to give the strength at any quantile q in the distribution

$$x_q = x_o + mn^{-1/k}\{ -\ell n\,(1-q) \}^{1/k} \tag{A3}$$

(For example q = 0.5 would give the median or 50th percentile strength).

Now consider two members of different sizes containing n_1 and n_2 elements. Equation A3 for each member can be combined to give the ratio of strength of the two sizes at any quantile q

$$\frac{x_q\,(n_1)}{x_q\,(n_2)} = \frac{x_o + mn_1^{-1/k}\{ -\ell n\,(1-q) \}^{1/k}}{x_o + mn_2^{-1/k}\{ -\ell n\,(1-q) \}^{1/k}} \tag{A4}$$

If the location parameter x_o is assumed to be zero, as is often done, the three-parameter model described above reduces to a two-parameter model and equation A4 is greatly simplified to

$$\frac{x_1}{x_2} = (\frac{n_2}{n_1})^{1/k} \tag{A5}$$

where x_1 and x_2 are the strengths of members of size n_1 and n_2, respectively. It can be seen in this case that relative size effects can be quantified by the shape parameter k, and the ratio of sizes, regardless of the quantile, or the actual values of n_1 and n_2. A log-log plot of strength against volume becomes a straight line of slope -1/k as shown in Figure A1.

In some cases it may be a poor assumption to use the two-parameter form of the Weibull distribution, particularly when extrapolating to large volumes, because this assumption implies strength reducing to zero as the volume becomes infinitely large.

Theory for Variable Stress Distribution

The above development has assumed that the member is subjected to a uniform distribution of stresses. In the more general case where stresses vary within a member, equation A3 can be written as :

$$F(x) = 1 - \exp\{-\frac{1}{V_1}\int_V \left(\frac{x - x_o}{m}\right)^k dV\} \qquad (A6)$$

where V is the volume of the member, and V_1 is a reference volume associated with the scale parameter m. For the two parameter case, the integral in this equation can be evaluated for any non-uniform stress distribution and the result expressed as

$$F(x) = 1 - \exp\{-\frac{V_e}{V_1}\left(\frac{x}{m}\right)^k\} \qquad (A7)$$

where V_e is equivalent stressed volume.

Values of V_e can be used to predict size effects using equation A5, with n_1 and n_2 replaced by V_{e1} and V_{e2} respectively. As an example of the calculation of V_e, for a beam of span L with two symmetrically placed loads, distance a apart, and neutral axis at mid-depth, the integral in equation A6 can be evaluated over the tension region (assuming brittle fracture in tension only) to give and equivalent volume of :

$$V_e = V\frac{1 + \frac{a}{L}k}{2(k+1)^2} \qquad (A8)$$

which is seen to be a quite simple proportion of the total volume V, of the member.

For an axial tension member the equivalent volume V_e in equation A7 is the total volume of the member, so the relative strengths of bending and axial tension members can also be compared using equation A5. The equivalent stressed volume of a bending member is much less than the total volume of the member stressed in axial tension, so tension stresses at failure are greater in bending tests than in axial tension tests.

Coefficient of Variation

The coefficient of variation of the Weibull distribution is given by :

$$cv = \frac{\{\Gamma(1+2/k) - \Gamma^2(1+1/k)\}^{1/2}}{x_o/m + \Gamma(1+1/k)} \qquad (A9)$$

where Γ is the gamma function.

For the two-parameter model with $x_o = 0$, the coefficient of variation becomes a function only of the shape parameter k. A simple but accurate approximation is

$$cv = k^{-0.922} \qquad (A10)$$

Because the coefficient of variation is a function only of k, strength tests of members of different sizes should all have the same coefficient of variation, and the k value obtained from equation A10 should be the same as that obtained in a log-log plot of strength against volume. The same should also apply for a comparison of bending tests and axial tension tests.

For this ideal case of a perfectly brittle material following a two-parameter Weibull distribution it would be possible, in theory, to carry out only one test series on members of only one size, and to predict the strength of other sizes on the basis of a k value obtained from equation A10. Unfortunately this is not possible because neither clear wood nor lumber with defects behave exactly according to this model for tension stresses parallel to the grain, as described in the body of the paper.

Load configuration

The conventional brittle fracture theory developed to this point can be used to predict the relative strengths of beams with different load configurations.

The relative strengths can be obtained using equation A5, and the equivalent of equation A8 for different load configurations. Fig. A2 (from Johnson 1953) shows how the relative strength depends on the coefficient of variation (and hence on k, from equation A10) for seven different configurations. The effect of load configuration increases significantly as the coefficient of variation increases.

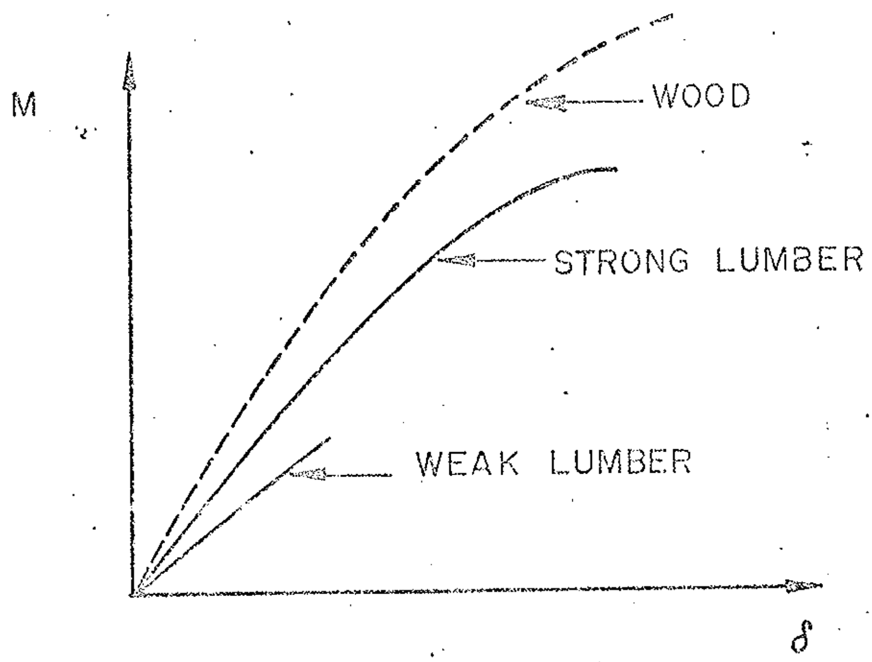


FIG 1. SCHEMATIC MATERIAL BEHAVIOUR

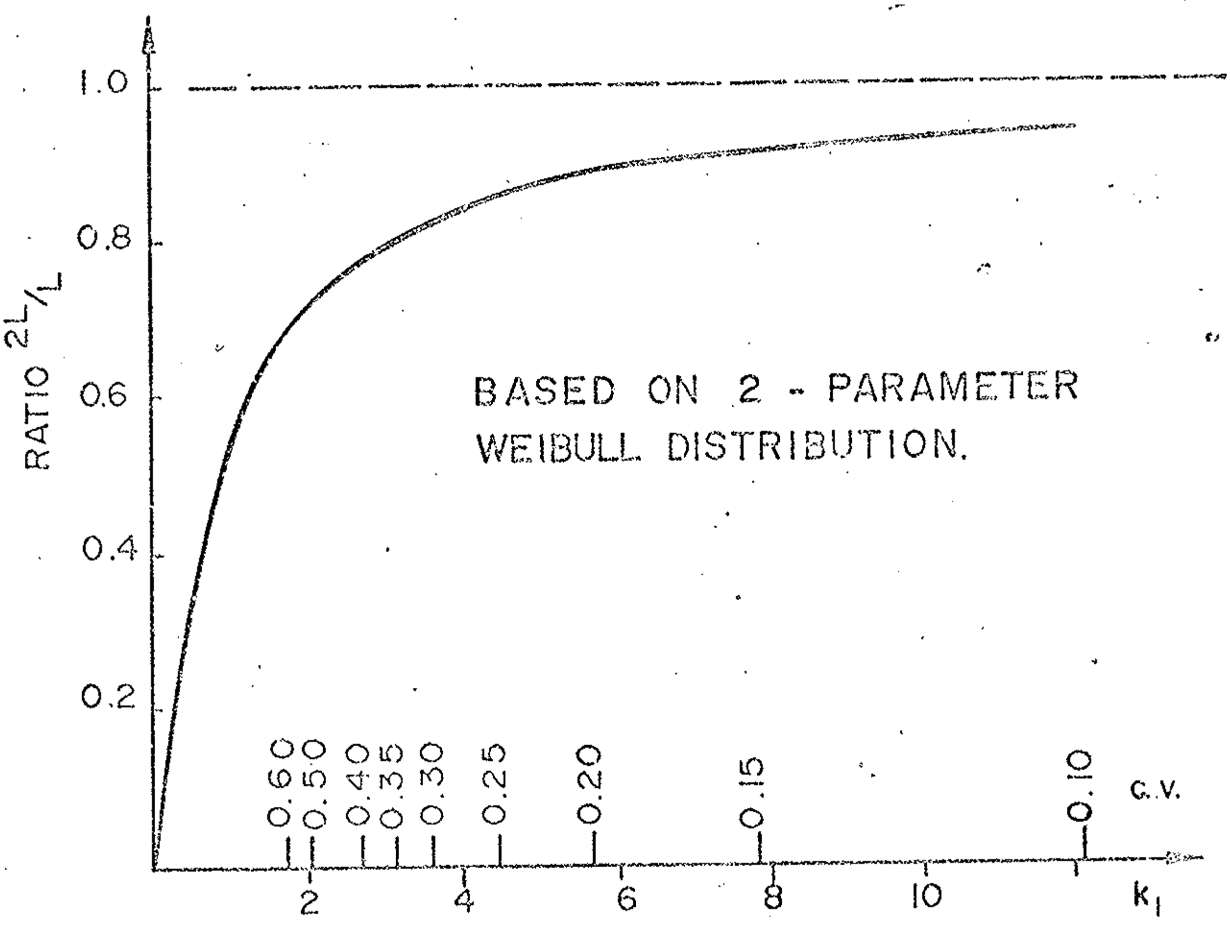
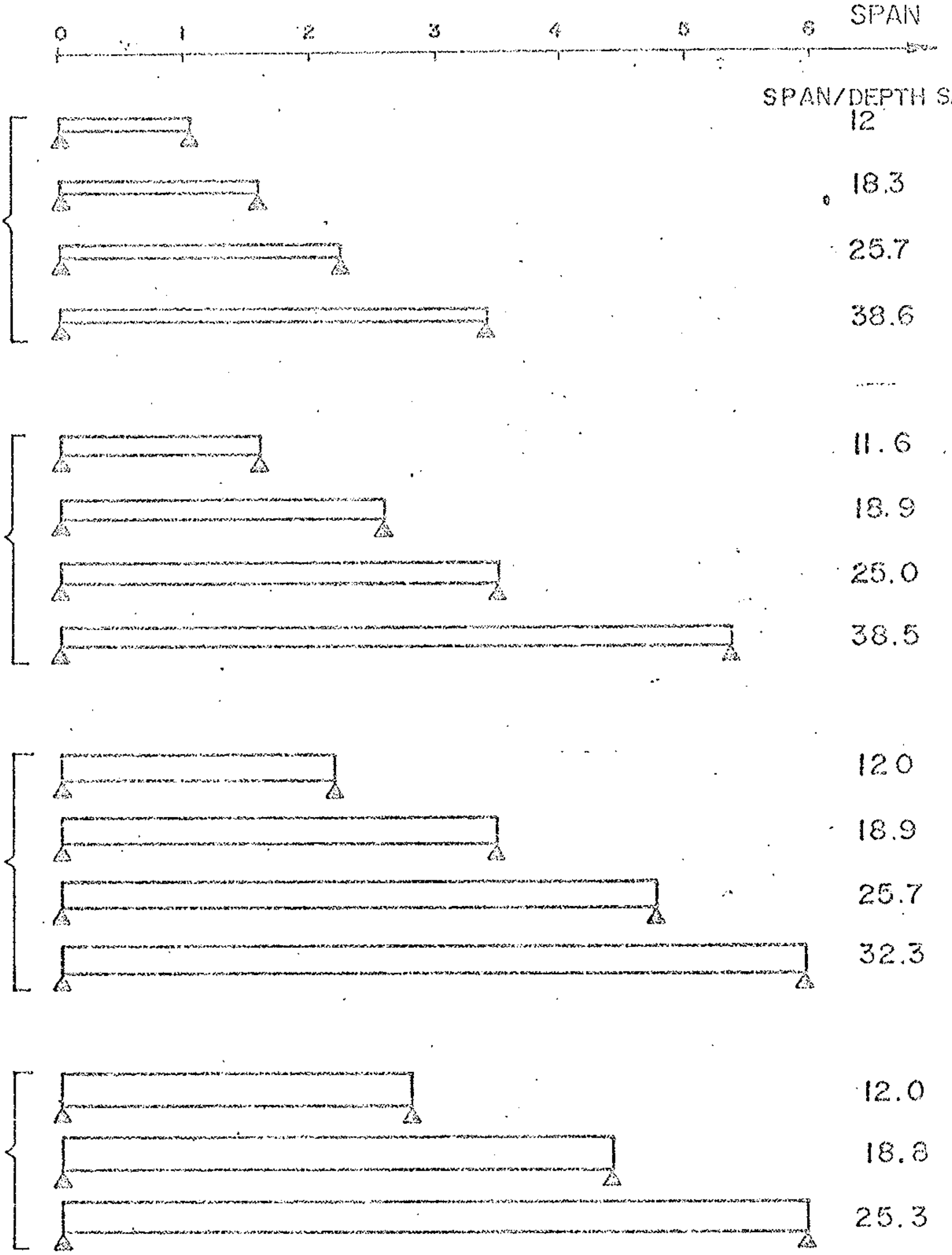


FIG 2. STRENGTH AFTER DOUBLING LENGTH
AS FUNCTION OF k_1

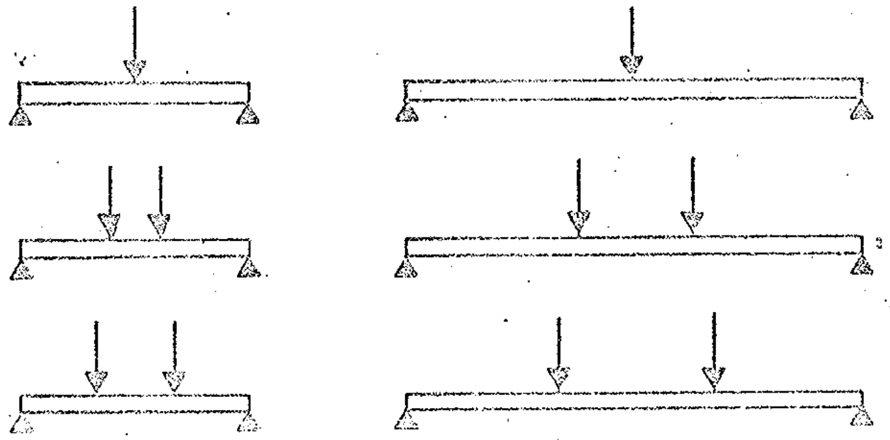
1976 SIZE EFFECT DATA



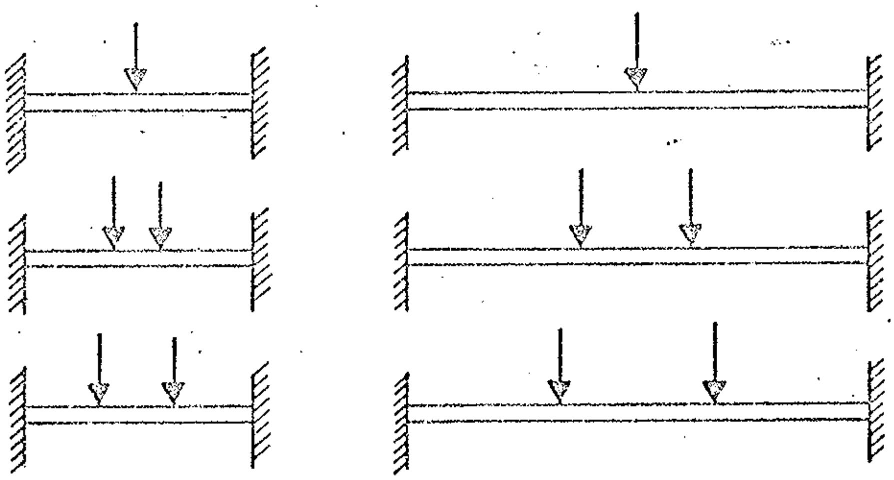
DEPTH mm	SPAN/DEPTH	SAMPLE SIZE
89	12	88
	18.3	93
	25.7	92
	38.6	83
140	11.6	95
	18.9	93
	25.0	90
	38.5	91
184	12.0	98
	18.9	98
	25.7	96
	32.3	97
235	12.0	96
	18.8	101
	25.3	99
	TOTAL	1418

FIG 3 1976 TEST SERIES. HEM - FIR

1983 LOAD CONFIGURATION EFFECT



SIMPLE SUPPORT



FIXED ENDS

SPAN:	1.54m	3.08m	
SPAN/DEPTH:	11	22	
GRADES:	SS	SS	
	#2	#2	
N =	100 ±	100 ±	TOTAL 3872
SIZE	38x140	38 x 140	

FIG 4 1983 TEST SERIES S. P. F.

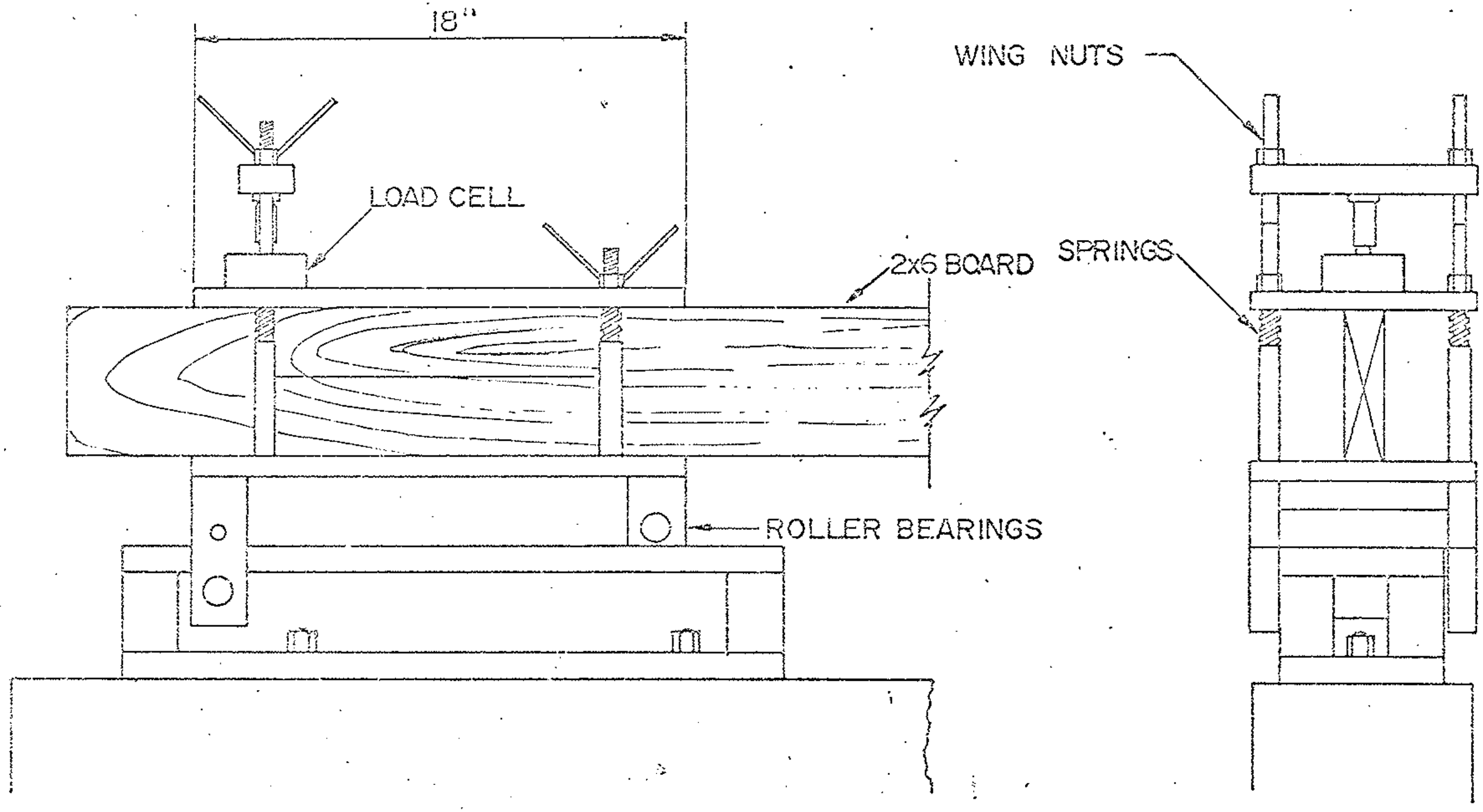


FIG. 5 FIXED END SUPPORTS

1978 IN - GRADE TESTS

SIZE EFFECT

SPAN/DEPTH: CONSTANT 17:1

SAMPLE SIZE: N = 200

NUMBER OF TESTS

		S-P-F		HEM-FIR		DOUG-FIR	
DEPTH mm	SPAN mm	S.S	2	S.S	2	S.S	2
89	1511	4	6	1	1	2	3
140	2375	4	6	1	1	2	2
184	3130	4	6	1	1	2	3
235	3994	2	2	1	1	2	3

TOTAL NUMBER OF SPECIMENS: 12000 ±

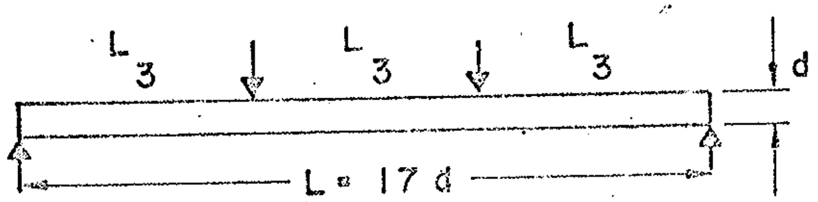


FIG. 6 1978 TEST SERIES. MIXED SPECIES

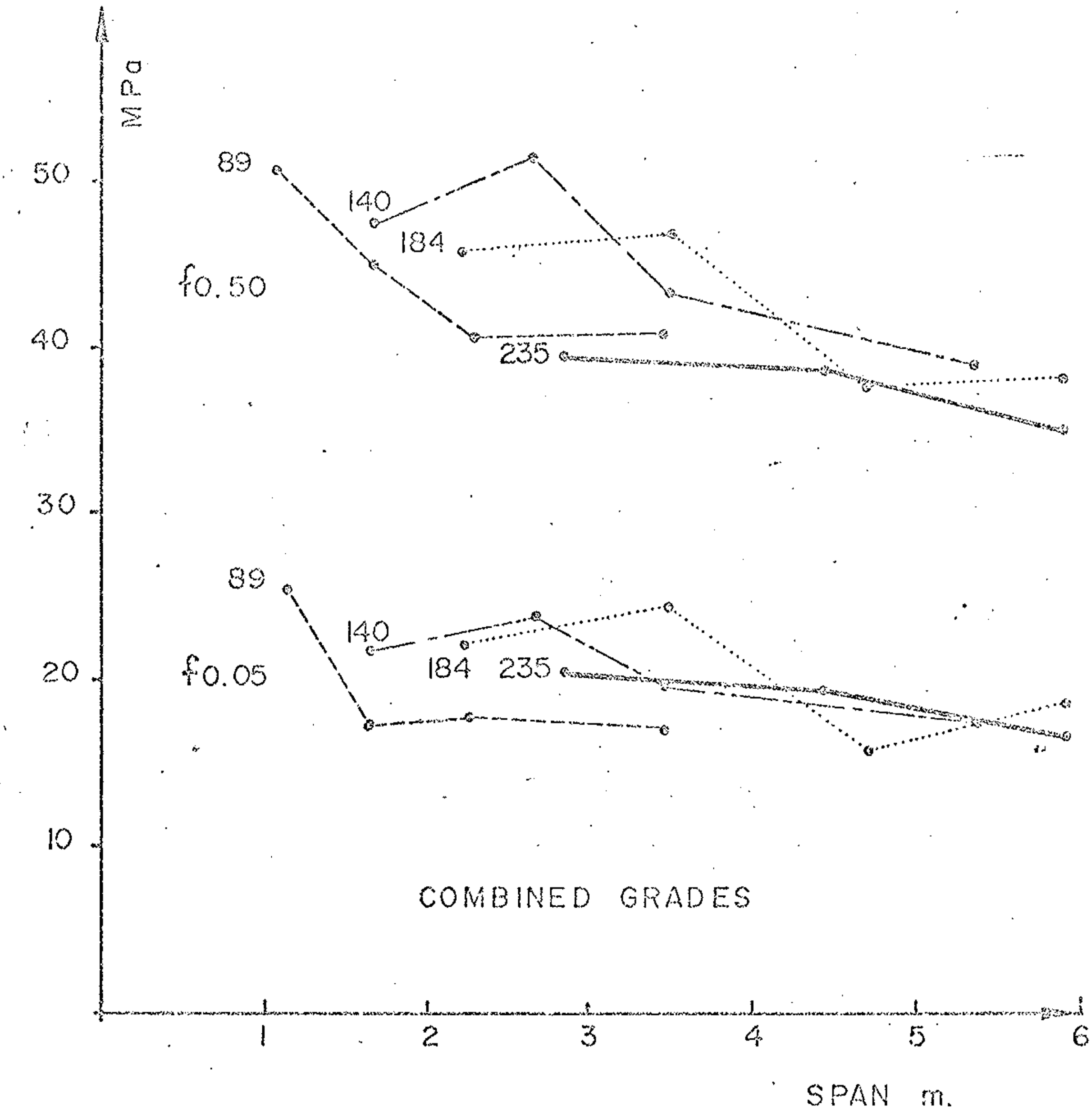


FIG 7. 1976 TEST RESULTS

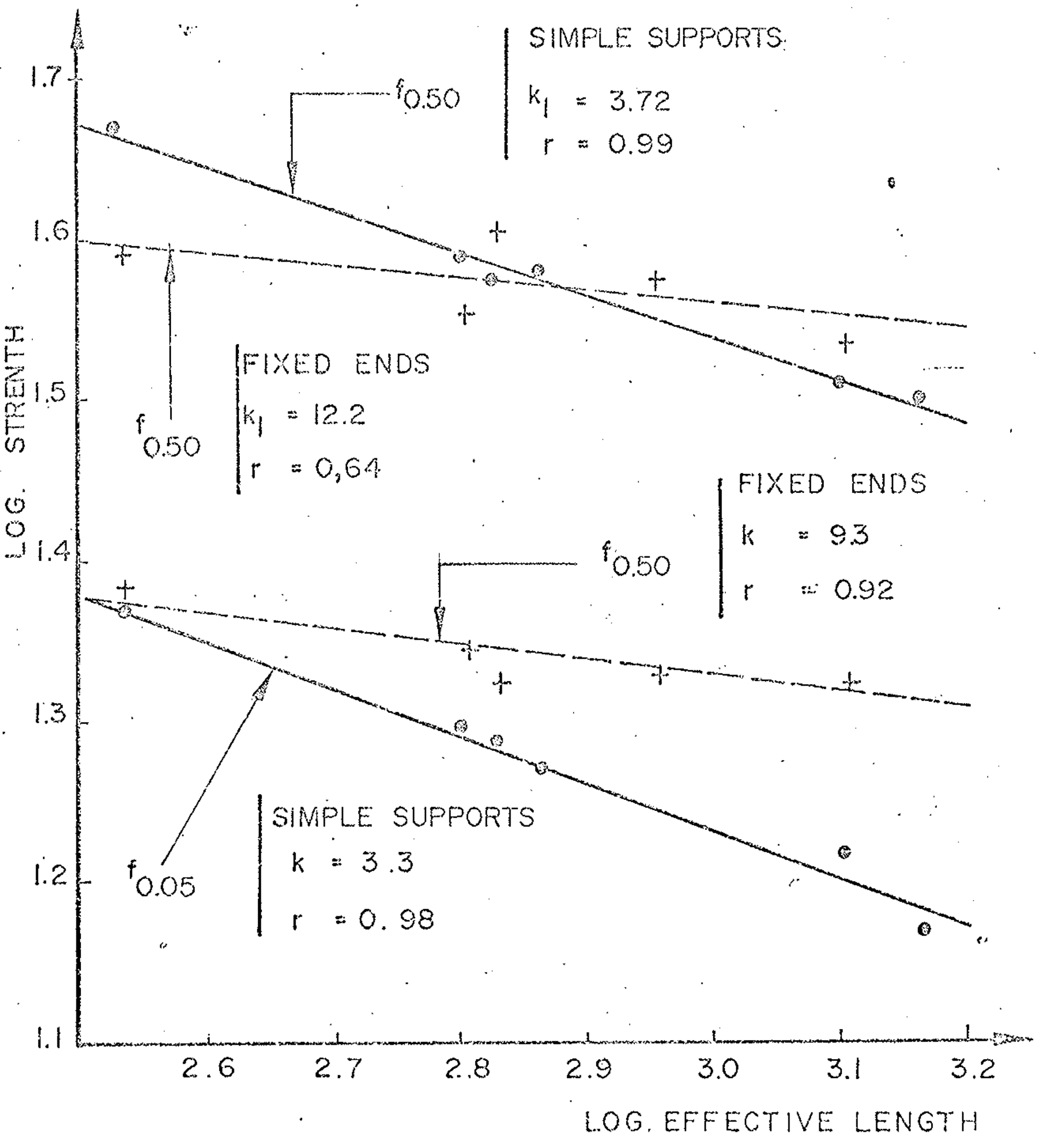


FIG 8. EFFECT OF REDUNDANCY
#2 GRADE

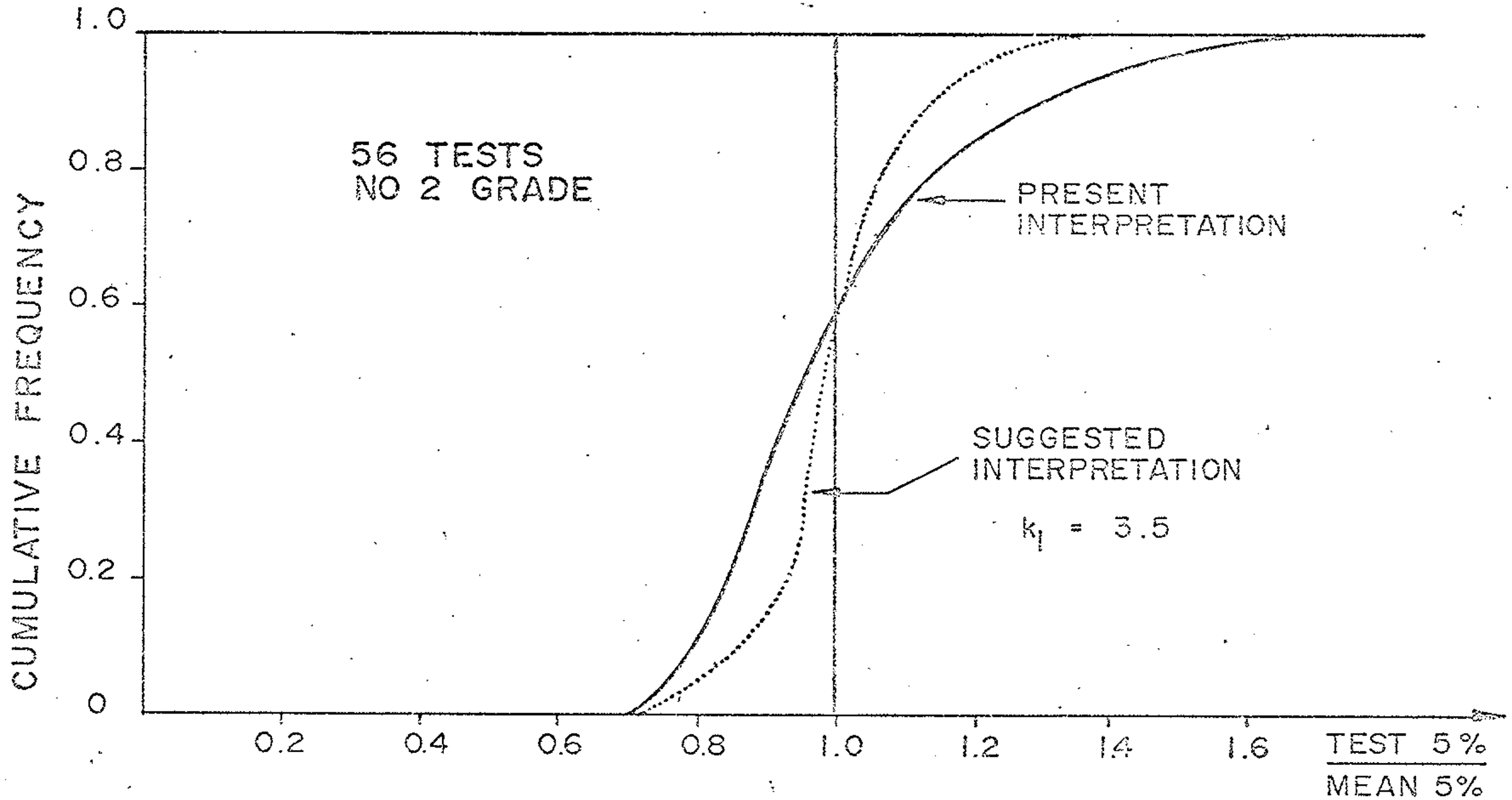


FIG II. VARIABILITY OF NORMALIZED
TEST DATA

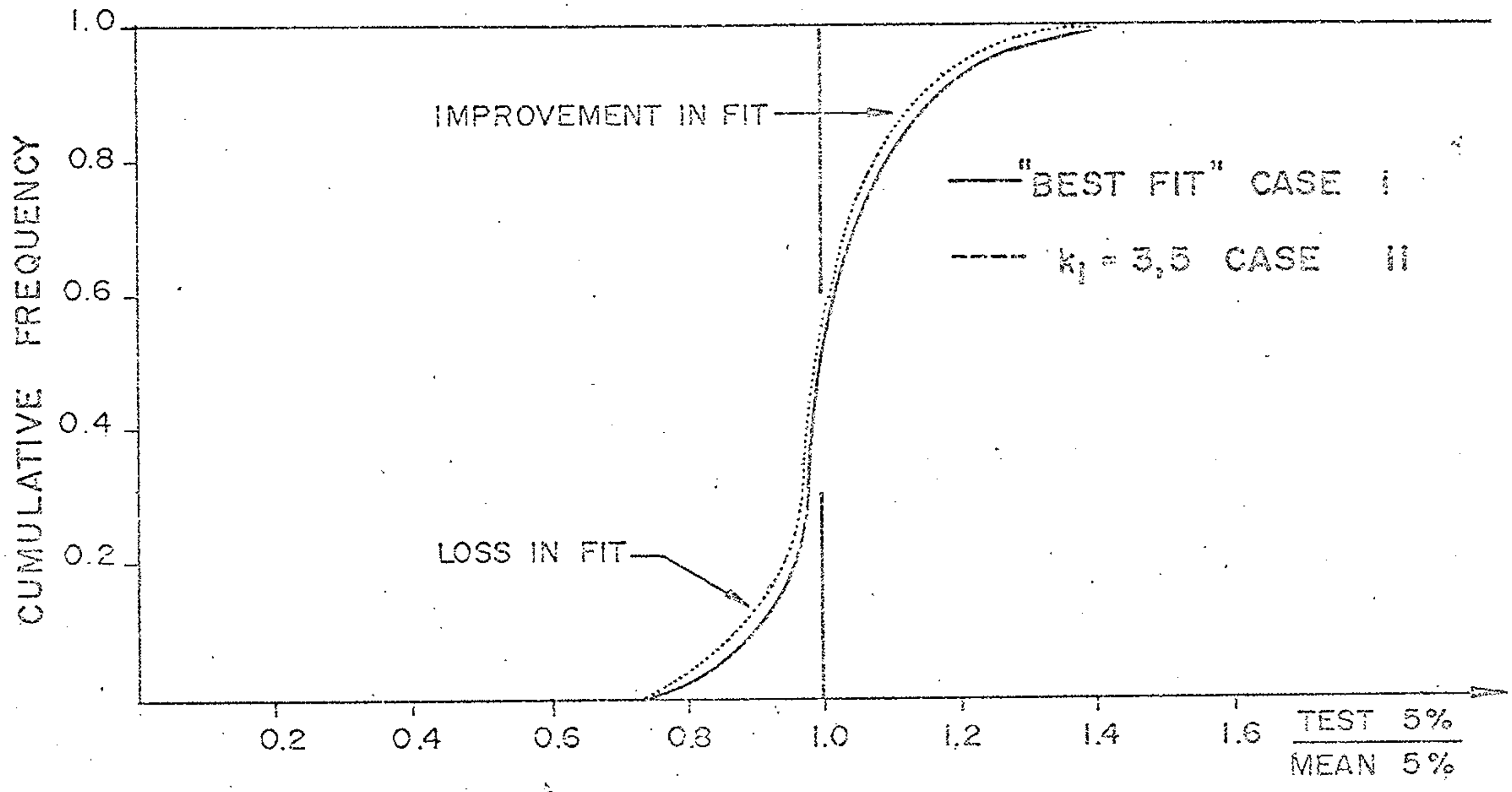
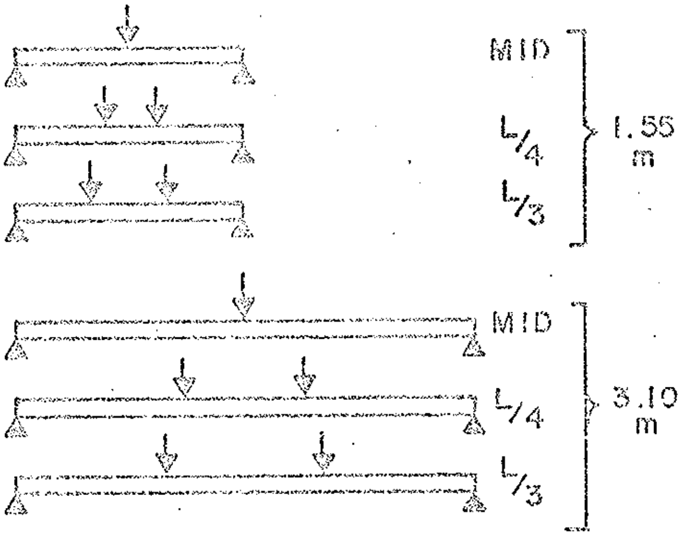


FIG. 10 COMPARISON OF "BEST FIT" WITH k_1 = 3.5

EXAMPLE: S-P-F

38 x 140 mm



COL 1 TEST RESULTS	COL 2 CORRECT FOR LOAD. CON	COL 3 CORRECT FOR LENGTH
23.23	18.60	15.32
19.60	18.80	15.48
18.38	18.38	15.40
19.13	15.32	15.43
16.46	15.79	15.90
14.50	14.50	14.60

$k_f = 3.5$

FIG. 9 COMPARISON OF TEST RESULTS

SPAN m		L/4	L/3	M
1.0	1.71	1.43	1.37	1.11
2.0	1.40	1.17	1.12	0.91
3.0	1.25	1.04	$\boxed{1.00}$	0.81
4.0	1.15	0.96	0.92	0.75
5.0	1.08	0.90	0.86	0.69
6.0	1.02	0.86	0.82	0.67
7.0	0.99	0.83	0.79	0.64

FIG. 12 LOAD & LENGTH FACTORS

CIB-W18/18-6-5

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PLACEMENT AND SELECTION OF GROWTH DEFECTS IN TEST SPECIMENS

by

H Riberholt
Technical University of Denmark
Denmark

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

PLACEMENT AND SELECTION OF GROWTH DEFECTS IN TEST SPECIMENS

Preface

During the investigation of the significance of the placement of defects in test specimens it was revealed that the strength depends on the definition of the structural element for which it is applicable.

In this paper there has been defined the strength of two different elements:

The strength of a cross section in a beam element of wood

The strength of a structural element, typical a beam or a bar.

Further, some aspects were found in the definition of the strength of elements which can be characterized as unclear or may be arbitrary. A discussion should be initiated on how the tests can be performed, and how the test results can be transformed, so that they are applicable for the modelling of real structures. This paper treats some of the aspects which should be taken up in the discussion.

1. Recommendation

Due to the fact that the strength of the cross sections varies along the length of boards, planks or lumber the placement of growth defects will influence the result of strength tests.

It is recommended that the placement of growth defects is done according to the table below. The recommendation is intended used for coniferous wood.

The selection of the worst growth defect, i.e. the weakest cross section, in a test specimen is done by an estimation. This can be based either on visual judgement or on non-destructive measurements.

Test type	Placement of the worst growth defect
Bending	In the moment span and in the tension side
Compression parallel to the grain	At the middle of the length of the test specimen
Tension parallel to the grain	At the middle of the length of the test specimen

It is recommended that the growth defects are selected so that the size of the defects belong to the whole interval between two grade limits. This should reflect praxis, and it would be conservative to tend to have defects close to the maximum allowable limit.

For shear and compression or tension perpendicular to the grain it is recommended to use specimens either with small growth defects or made of clear wood.

The load should be applied so that the loose spring wood will become decisive for the strength. Figure 1.1 shows the recommended load directions.

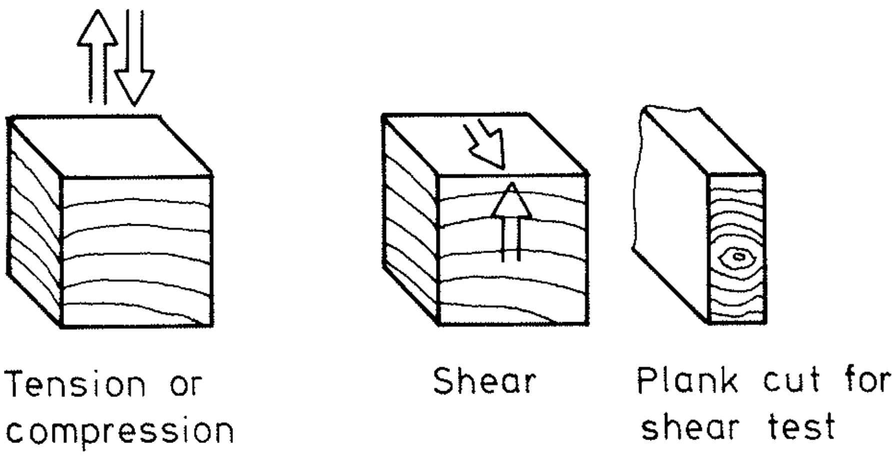


Figure 1.1 Direction of applied stress or force in test specimens.

2. Comments

There is given some comments to the recommendations. Further, some methods are given for how some of the strength results should be transformed in order to be applicable to structural elements.

2.1 Bending

The intention with the recommendation is to obtain the lowest bending strength.

2.1.1 Defect placed randomly in the compression or tension zone

If the growth defect is placed randomly in the tension or compression side, then the bending strength, the 5th percentile, will be increased.

The largest obtainable increase in the 5% percentile for a Normal Distributed bending strength can easily be evaluted. It must be valid for the weakest 10 percent that, if the defect is put in the compression side then the strength must be higher than the 10th percentile for the bending strength of test specimens with the weakest defect in the tension side. The 5th percentile strength for a randomly placed defect simply becomes the 10th percentile strength for the growth defect in the tension side. Figure 2.1 shows the shift in strength.

The assumption mentioned above does not hold fully in reality, and therefore the increase will not be so large. Figure 2.1 illustrates this.

Meanwhile it is recommended that provided the bending tests are conducted with the estimated worst defect in the tension side, then the 5th percentile could be increased according to the curve in figure 2.1 if the percentile shall be used for beams with defects placed randomly in the tension or compression zone.

This increase will always appear, unless one have the seldom situation that the moment in the beam changes sign.

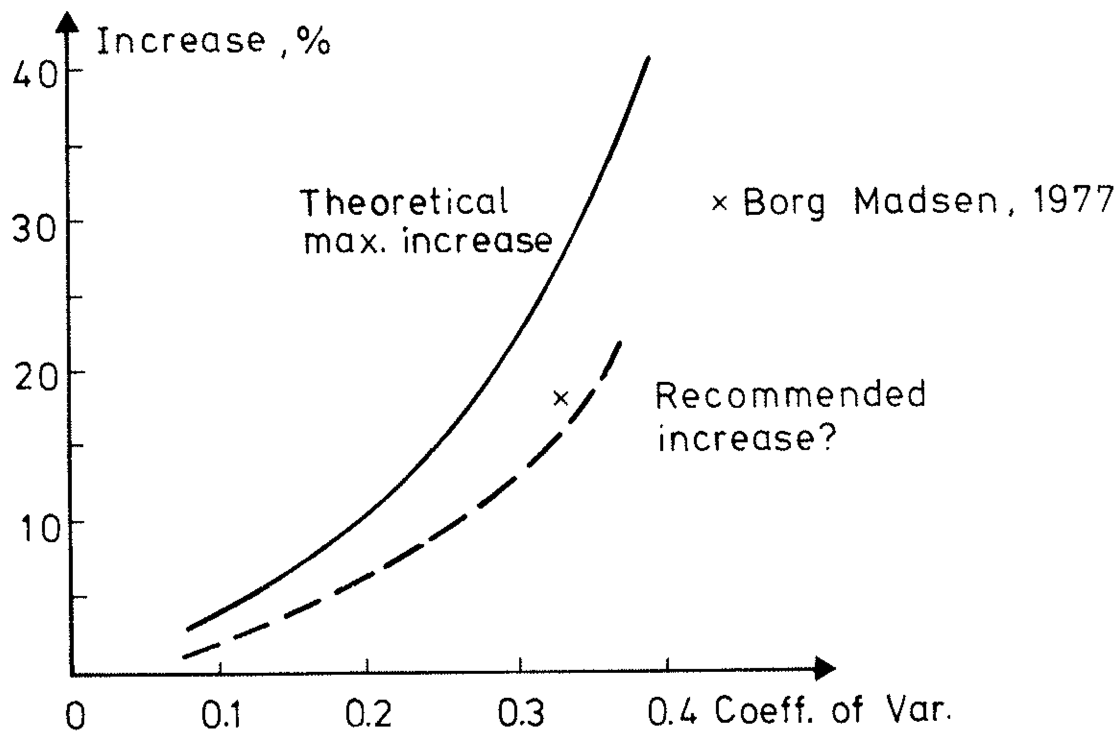


Figure 2.1 Theoretically largest and recommended increase in the 5th percentile of the bending strength by having the defect placed randomly instead of only in the tension side. Further some increases based on measurements.

With this testing and interpretation method one will have the advantage of preserving the continuity with previous testing methods.

2.1.2 Defects placed randomly along the length of the beam

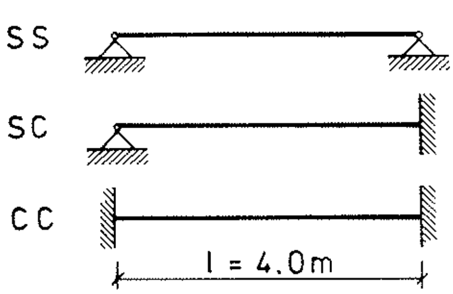
For a timber beam the classifying defect may occure randomly along the length of the beam. The worst defect does not necessarily occur at the maximum moment. The influence of this on the 5th percentile of the bending strength can be evaluated based on assumptions, which for example are given in [Riberholt, 1980].

The change in strength is substantially. It can be illustrated by an example from [Riberholt, 1980].

In this the occurence of defects along the beam axis is described by a Poisson process with constant intensity λ . The strength of the cross section with a random defect (not the worst defect) is described by a Normal Distribution Funktion F_d and the strength in between is given by another Normal Distribution Function F_c , (c for "clear wood").

Figure 2.2 shows the different distribution functions F_{beam} expressed in the maximum moments.

$$F_{beam}(M_{max}) = P\{\text{rupture for max. mon.} = M_{max}\}$$



$\lambda = 2.0\ m^{-1}$
Strength distributions:
F_c: Normal distr.
$\mu(M) = 5.0$ kNm
$\sigma(M) = 1.0$ kNm
F_α: Normal distr.
$\mu(M) = 2.5$ kNm
$\sigma(M) = 0.5$ kNm

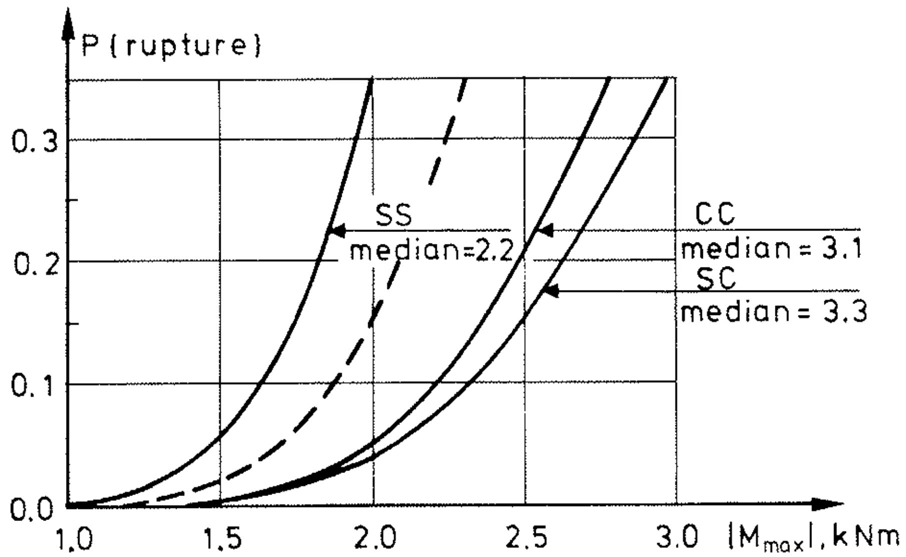


Figure 2.2 Cumulative distribution functions for timber beams with different supports. The dotted line represents F_d .

Figure 2.2 shows that the 5th percentile for M_{max} in the beams with 1 or 2 clamped supports is approximately one third larger than for the simple supported beam. The reason is that for these beams the likelyhood for coincidens of max. moment and max. defect is smaller.

This increase will depend on the structure of which the timber beam is a part. Both the length (size) and the moment distribution will influence the value of the increase.

Meanwhile, one could imagine that conservative increase factors could be established for the case of strength control at moment peaks. By the evaluations of the increase factors it should be taken into account that the form of the moment curve may change during the life time of the structure.

2.1.3 Size effect

For a timber beam with a certain cross section the bending strength will decrease for increasing length. This is due to the fact that for a long beam there will be present more weak cross sections than in a short beam.

It is expected that this effect is of the same magnitude as that of defects in the tension side or random placement.

This type of size effect can be combined with the observation that for similar cross sections but of different size the larger will have the smallest modulus of rupture. The combined effect will be of a relatively large magnitude.

But it could also be that the origine of the two effects is the same. The cross sectional size effect could be due to the fact that the number of tested cross sections with defects is larger in beams with large cross sections. As far as the author knows, no report has given the answer to this question.

2.2 Compression parallel to the grain

Due to the short length of the test specimen one does normally only measure the strength of one cross section with a defect. If the defect is selected as the most weakening of all in a piece of lumber, then one will approximately get the distribution function for the weakest cross section in a column.

It is expected that the position of the defect in the cross section does not give a significant effect on the compression strength as in bending tests.

In timber columns the random placement of the defects along the axis will have an effect similar to that found for beams. Again the 5th percentile could be increased, when the position of the defects along the axis is more or less random, compared with the situation where there is coincidens of the weakest cross section, maximum lateral deflection and maximum moment from a may be lateral load.

2.3 Tension parallel to the grain

The tested length of a piece of lumber should contain the estimated weakest cross section. Thereby one will obtain the same statistic information as described for compression.

Otherwise the comments for compression are applicable to tension.

Since it is expected that the length/size is specially pronounced for the brittle tension rupture, it is suggested to discuss strength in dependence of the definition of the structure or structural element.

For example a bottom chord in a truss. If the tension strength is connected to a bay one could use a higher 5th percentile than if it was connected to the whole bottom chord. If one arbitrarily defined that the tension strength should be the 5th percentile for the whole bottom chord then the measured tension strength should be reduced according to this definition.

2.4 Shear and compression or tension perpendicular to grain

The recommended selection of test specimens have been based on the assumption that growth defects can have a reinforcing effect.

For compression there is certainly a stiffness increase and thereby an increase in strength.

For tension and shear it is often seen that the rupture surface follows the annual rings i.e. the loose spring wood. But on the other hand strength reducing cracks and splits do often originate in growth defects as knots. Opinions or test reports which ilucidate this are inquired.

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Riberholt, H., 1980. Strength distribution of timber structures. Measured variation of the cross sectional strength of structural lumber. IUFRO Wood Engineering Group Meeting, Oxford, UK. 1980, pp. 30-32.

CIB-W18/18-6-6

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PARTIAL SAFETY-COEFFICIENTS FOR THE LOAD-CARRYING CAPACITY OF TIMBER STRUCTURES

by

B Norén
Swedish Institute for Wood Technology Research
Sweden
and
J-O Nylander
The National Board of Physical Planning and Building
Sweden

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

B Norén and J-O Nylander: *Partial safety-coefficients for the load-carrying capacity of timber structures.*

Introduction

The Swedish regulations for the change-over to partial safety coefficients at design of building structures is supposed to be complete during 1986. Principles, classifying of loads and load combinations and the values of the partial coefficients have been analyzed and discussed for more than ten years. A volontary code has been in force since 1980 [1], but the code of practice for timber structures, including the values for partial coefficients on resistance, has only recently been presented for comments (1985).

During this process the effect on volume of wood and on the design of joints has been checked on timber structures of different kind. Thus, eventually, design in accordance with the (1985) proposed code has been compared to design in accordance with the present code (SBN 1980).

The result of such comparisons no doubt had important influence on the revision of the rules. However, the new code involves a number of changes among which the differentiating of the safety factor is only partly responsible for possible deviations from traditional design, Also, the aim of the new code, a better prediction of probability of failure and a more uniform safety between structures of different kind, has necessarily involved occasions where the new code resulted in a heavier design. Still for medium sized timber structures in general and structures for the framework of houses in particular the ambition has been to reduce the cost rather than to increase it.

General rules

In the following the partial coefficients will be expressed in terms of the traditional single safety factor as well as in terms of the safety-index β used in a higher-level-method. This might be of some help in establishing the values of the safety coefficients whether one prefers to base the judgement on experiance rather than adjusting them to demands for uniform safety or safety index.

Still, background and limits for what can be done merely through the choice of the partial factors for the resistance (the "material"), is here assumed to be the appreciation of the General Rules [2, 3].

It is characteristic for the Nordic and probably for the majority of national rules, that there are General Rules indifferent to material and - as supplement - special rules ("technology rules") for structures of the principle building materials (steel, concrete, wood, etc) and possibly for structures for specific purpose. The authors of these application codes are generally tied to the general rules with respect to partial coefficients on loads (γ_g, γ_q) and to the partial coefficient for safety-class (γ_n), while their freedom is less limited with regard to the coefficients on the resistance side. However, the general rules recommend certain principles and stipulate values of the safety-index β to which the total effect of the partial factors should preferably correspond [4].

Partial-coefficient-method versus traditional method

The criteria for design will always be that the resistance (R) exceeds the effect of loading:

$$R - S \geq 0$$

At the traditional method of design this condition may be simplified by

$$\varphi_k f_k / n - (g_k + q_k) \geq 0 \qquad (0:1)$$

Here f_k is the characteristic value of strength and g_k and q_k the characteristic values of load-effect.

The factor φ transforms f to comparability with g and q. It includes geometric parameters, duration factors etc.

The single safety factor (n) in the traditional method must compensate for the stocastic variation in f, g, q and φ.

$$n = \varphi_k f_k / (g_k + q_k) \qquad (0:2)$$

The values in different codes has not always been possible to compare directly in the past due to, among other things, that the fractiles used for the variables deviate.

At the Partial-Coefficient-Method (Level-1-Method) the safety factor is differentiated with respect to the deviation of the different variables. Additionally, a partial factor (γ_n) is separated to consider the demand for increased safety due to consequences of failure.

The condition of the traditional ("Level 0") method (0:1) is thus generalized at the Level-1-Method:

$$\frac{\varphi_k f_k}{\gamma_m \gamma_n} - (\gamma_g g_k + \gamma_q q_k) \geq 0 \tag{1:1}$$

or at the limit

$$\gamma_m \gamma_n = \varphi_k f_k / (\gamma_g g_k + \gamma_q q_k) \tag{1:2}$$

The elimination of $\varphi_k f_k$ in (0:2) and (1:2) gives the ratio of the product $\gamma_m \gamma_n$ and the "traditional" safety factor n (provided that the characteristic values are based on the same fractile in both cases):

$$\gamma_m \gamma_n = n \frac{1 + \mu_k}{\gamma_g + \mu_k \gamma_q} \quad \text{where } \mu_k = q_k / g_k$$

is the ratio of characteristic values for variable load to permanent load.

EXAMPLE

The Swedish general code stipulates n = 1.8 at the traditional (Level 0) method and the partial safety factors $\gamma_g = 1.0$ and $\gamma_q = 1.3$ at the Level-1-Method. Consequently, if the aim is that the design should remain the same when the safety factor of the Level-0-Method is differentiated as in the Level-1-Method, the condition for the product $\gamma_m \gamma_n$ is

$$\gamma_m \gamma_n = 1.8 \frac{1 + \mu_k}{1 + 1.3 \mu_k}$$

For timber structures in ordinary houses, using $\gamma_n = 1.1$ (Safety Class 2) and $\mu_k = 4$ (20 % permanent load), the equation gives $\gamma_m = 1.32$. The conclusion has been that higher γ_m-values than 1.3 for timber structures in Safety Class 2 should be avoided.

Deriving the value of γ_m from the Level-2-Method

In the same way as the Partial-Coefficient-Method (Level-1-Method) was compared with the traditional single-safety-factor-method (Level 0), it can be compared with the internationally recognized semiprobabilistic method, the Level-2-Method. However, as this method is based on the mean values of the variables, not on fractile values as the other two, it is practical to transform (1:2) into

$$\gamma_m \gamma_n = \frac{\varphi_m f_m}{g_m} \Psi \tag{1:3}$$

Assuming the normal distribution for g (and q) and lognormal distribution for f and φ

$$\Psi = \frac{\exp(-k_\varphi \delta_\varphi)\ \exp(-k_f \delta_f)}{\gamma_g(1+k_g \delta_g) + \mu_m \gamma_q (1+k_q \delta_q)} \tag{1:4}$$

Here δ is the coefficient of variation, k a factor depending on what fractile is used and μ_m is the ratio of mean values of variable to permanent load.

In order to correlate the design limit expressed by (1:3) and (1:4) with the design limit found from the Level-2-Method, the relation of mean values in (1:3) may be written

$$\frac{\varphi_m f_m}{g_m} = 1/\Omega \tag{2:2}$$

Thus the product $\gamma_m\gamma_n$, sized by applying the Level-2-Method, is

$$\gamma_m\gamma_n = \Psi/\Omega \tag{2:3}$$

The expression for Ω is analogous to that of Ψ:

$$\Omega = \frac{\exp(-\alpha_\varphi\beta\ \delta_\varphi)\ \exp(-\alpha_f\beta\ \delta_f)}{1 - \alpha_g\beta\delta_g + \mu_m(1 - \alpha_q\beta\delta_q)} \tag{2:4}$$

By integrating φ and f into one variable for resistance, $R = \varphi f$, using $\delta_R{}^2 = \delta_\varphi{}^2 + \delta_f{}^2$, and transformning g and q similarily into one variable S for load-effect, (2:4) is transformed for application of the Level-2-Method:

$$\Omega = \frac{\exp(-\alpha_R\beta\delta_R)}{1 + \alpha_S\beta\ \delta_S} \qquad \text{where } \alpha_R = -\cos\psi \text{ and } S = \sin\psi$$

The safety-index β and the angle ψ in the sensitivity factors α_R and α_S are explained in the brief description of the Level-2-Method to Figure 1.

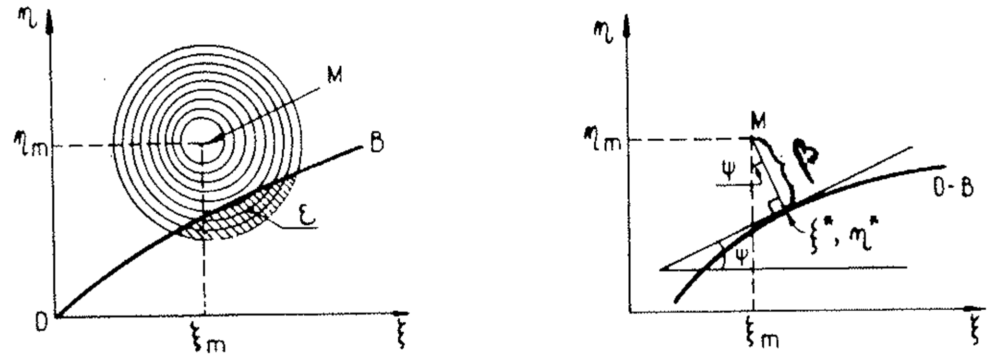

Figure 1. The starting point for the Level-2-Method, used for calibrating of γ_m, is the distribution of possible combinations of variables important for effect of load and the resistance of the structure. After certain transformation of the variables into two mutually independent variables $S(\xi)$ and $R(\eta)$ for load effect resp. resistance, the frequence (ζ) is represented by the surface generated by rotating a normal-distribution-curve (with standard deviation = 1) around the axis (M) perpendicular to the ξ-η-plane and representing the mean values. Various ζ-levels of this surface are shown by the circles in the figure to the left)

The border between survival and failure

$$\theta[S(\xi),R(\eta)] = 0 \qquad (2:1)$$

is represented by the screen OB perpendicular to the ξ-η-plane. The shortest distance from the axis M to the screen (β) represents the safety index. The direction of the screen is defined by the angle ψ.

The probability of failure is

$$\varepsilon = \Phi(-\beta)$$

If the convex screen OB is replaced by the touching plane, ϕ is the function for normal distribution.

Proposal for γ_m in the Swedish code

In order to establish a proper value of the factor γ_m for the application of the partial-coefficient-method in the new Swedish code, a number of calculations of

$$\gamma_m \gamma_n = \Psi / \Omega$$

has been carried out. Here is merely quoted one example which concerns timber structures for housing or similar structures in Safety Class 2 for which $\gamma_n = 1.1$ is stipulated.

Assumptions on deviations (coefficients of variation) are $\delta_\varphi = 0.12$ and $\delta_f = 0.2$ giving $\delta_R = 0.234$ and - from the general loading code - $\delta_g = 0.05$ and $\delta_q = 0.40$. Further, at calculating Ψ, the coefficients k = 2.06 (0.98 percentile) and 1.65 (0.05 percentile) have been used for load effect respectively resistance.

The full curve in Figure 2 shows the values of the product $\gamma_m \gamma_n$ which should be used in the partial-coefficient-method (Level-1-Method) if aimed at complete agreement with the (Level-2-Method) at the safety index $\beta = 4.3$ applied in Safety Class 2. Correspondent curves (dashed) are shown for $\beta = 3.8$ and 4.8, stipulated for Safety Classes 1 and 3.

Obviously the partial-coefficients should preferably not be constants but related to the proportions of permanent load (gravity load) and variable load. In the figure these proportions are expressed in terms of the ratio of variable mean load to permanent mean load (μ_m) as well as in terms of characteristic values:

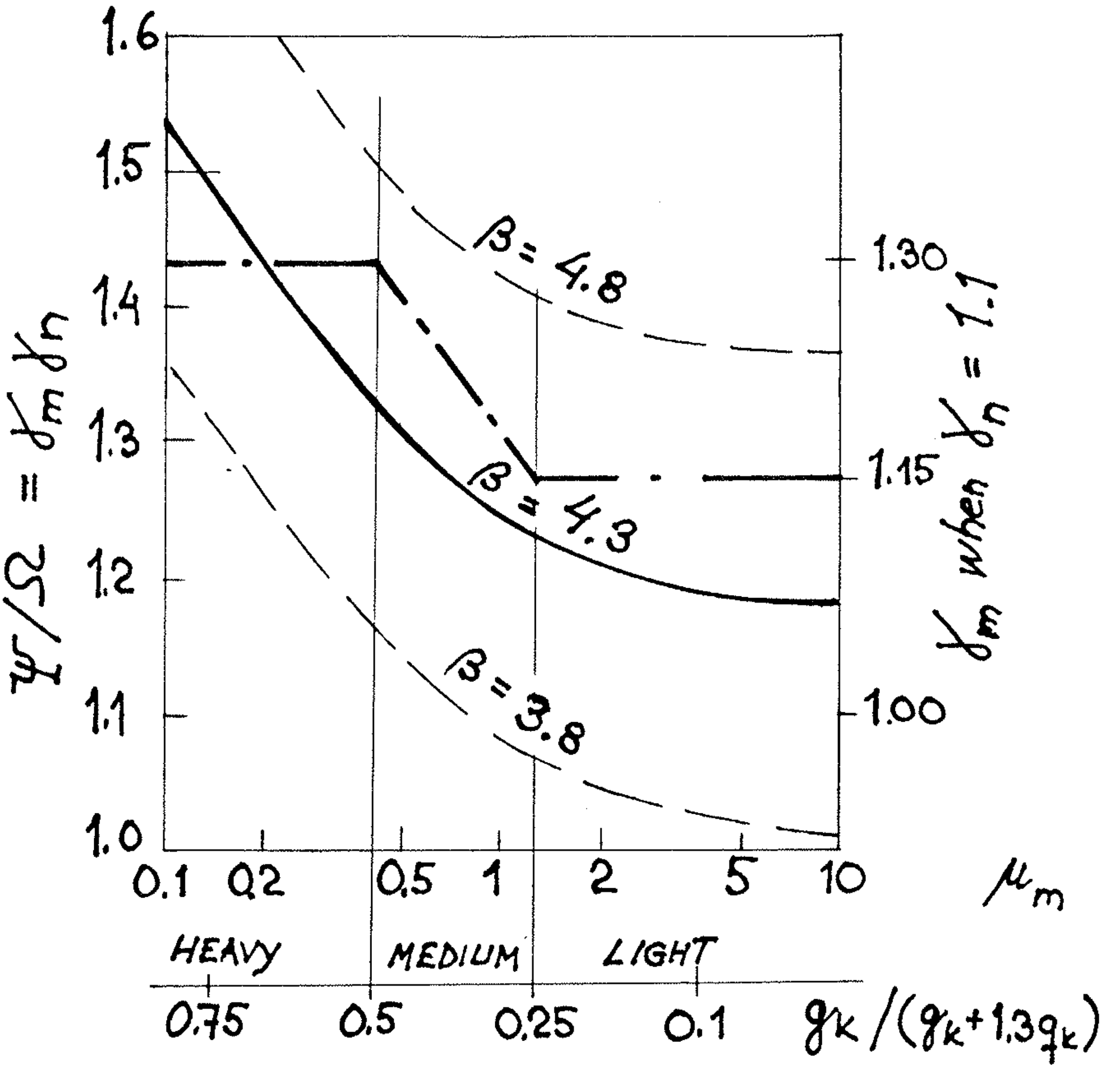

Figure 2. Calibrating the product $\gamma_m \gamma_n$ for ordinary timber structures to the safety index (β) of the Level 2-Method. The broken curve is a proposed approximation for Safety Class 2.

$$\mu_k = \frac{1.0g_k}{1.0g_k + 1.3q_k}$$

In the Swedish proposal the β = 4.3 curve is approximated to the curve of straight lines shown in the figure, at γ_n = 1.1 corresponding to γ_m = 1.15 for "light" structures, increasing linearly between μ_k = 0.25 and μ_k = 0.5 to γ_m = 1.30 for "heavy" structures.

References

1 SBN avd 2A Bärande konstruktioner (Swedish Building Code, Section 2A Load-bearing Structures) Statens Planverk PFS 1979:7.

2 Recommendation for Loading and safety regulations for structural design. The Nordic Committee on Building Regulations NKB-Report No 36.

3 AK 79/81 Allmänna regler för bärande konstruktioner. (General design rules). The National Board of Physical Planning and Building Report 50, 1982.

4 General principles on reliability for structural design. IABSE report 35, Zürich 1981.

CIB-W18/18-7-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

MODEL SPECIFICATION FOR DRIVEN FASTENERS FOR ASSEMBLY OF PALLETS AND RELATED STRUCTURES

by

E G Stern and W B Wallin
Virginia Polytechnic Institute and State University
U.S.A.

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

MODEL SPECIFICATION FOR DRIVEN FASTENERS FOR ASSEMBLY OF PALLETS AND RELATED STRUCTURES

By

E. George Stern, Earle B. Norris Research Professor Emeritus of Wood Construction
Virginia Polytechnic Institute and State University, Blacksburg, Virginia 24061

Walter B. Wallin, Research Forest Products Technologist, U.S. Forestry Sciences Laboratory, Princeton, West Virginia 24740, and Adjunct Professor of Forest Products, Virginia Polytechnic Institute and State University, Blacksburg, Virginia

FOREWORD

The fasteners used for the assembly of pallets and related structures are also used for the assembly and erection of such timber structures as timber trusses, timber-framed buildings, pole-type structures, etc. For such purposes, these fasteners are called "truss nails", "framing nails","pole barn nails", etc. These structures are exposed to the elements (winds, storms, waves, shocks, etc.). Therefore, these fasteners have to withstand similar forces to which pallets are exposed.

In the light of this, the specifications covering pallet nails are also applicable to those nails which are sold as truss nails, framing nails, pole barn nails, etc.

The test procedures employed provide data which indicate whether the fasteners can, under given conditions, (1) be driven and (2) transmit potential forces without excessive joint deformation and, particularly, fastener bending.

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in Haifa (Beit Oren), Israel, June 3 to 7, 1985

MODEL SPECIFICATION FOR DRIVEN FASTENERS FOR ASSEMBLY OF PALLETS AND RELATED STRUCTURES

By E. George Stern, Earle B. Norris Research Professor Emeritus of Wood Construction Virginia Polytechnic Institute and State University, Blacksburg, Virginia 24061

Walter B. Wallin, Research Forest Products Technologist, U.S. Forestry Sciences Laboratory, Princeton, West Virginia 24740, and Adjunct Professor of Forest Products, Virginia Polytechnic Institute and State University, Blacksburg, Virginia

SYNOPSIS

A descriptive and performance specification was drafted covering nails and staples used for the assembly of wooden pallets and related structures in the U.S.A. This specification is based on recent technological developments in this field and emphasizes the benefits of the use of fastener withdrawal and shear indexes as a means of rating fastener performance.

INTRODUCTION

This specification is to provide manufacturers and users of pallet nails and staples with a detailed description of common, driven, regular and stiff-stock steel as well as hardened-steel fasteners used for the assembly of expendable and returnable wooden pallets and related structures employed in materials handling and shipping. Such pallet fasteners which do not conform to this specification shall be considered special fasteners. These specials include such standardized fasteners having features different from those descibed in this specification, such as heads, points, and threads of different designs.

Inspection and MIBANT testing (2.3) of the pallet fasteners described in this specification shall be considered a routine procedure. All other testing required shall be covered specifically in the purchase order.

The standard fasteners are number-coded for easy reference by pallet-fastener manufacturers and pallet specification writers as well as ordering by pallet manufacturers. The performance index estimated and presented for each standard fastener should be helpful in the fastener selection as the optimum one for a given end product.

The fastener information presented includes the input required for the fasteners used in the Pallet Design System (5.5), the first-generation reliability-based design procedure for wooden stringer pallets.

The listing, coding, and indexing of the standard pallet fasteners in this specification should result in their preferred use and reduce the proliferation of special fasteners to the benefit of all involved.

This specification may be incorporated in related documents by reference. Any required deviation from this specification shall be noted prominently in order to reduce to a minimum any possible misinterpretations.

This specification may involve hazardous materials, operations, and equipment. This specification does not purport to address all the safety problems associated with its use. It is the user's responsibility to consult and establish appropriate safety practices and determine the applicability of regulatory limitations.

FASTENERS COVERED

Standard Pallet-Assembly Nails

The standard, helically threaded, stiff-stock and hardened steel, pallet-assembly nails are described in Table 1. They have filleted flat heads, are pointless or provided with short (blunt) or medium diamond or chisel points (not wider than their wire diameter), have four thread flutes and 60 to 67-deg thread angles. Their fastener withdrawal and shear indexes, FWI and FSI, with respect to their withdrawal resistance, FWR, and their shear resistance, FSR, respectively, are based on the following relationships (4.2, ref.3):

$$FWI = 221.24\ (WD)\ (27.15\ (TD - WD)\ (H) + 1)$$
$$FSI = 263{,}260\ (WD)^{1.5} / (3M + 40)$$

(a listing of the symbols used is given at end of paper)

The relationships of the nail-design variables are indicated in Figs. 1 to 5.

Standard Pallet-Assembly Staples

The standard, bright or coated, plain-shank, regular-stock steel, ½-in. (13-mm) wide, pallet-assembly staples are described in Table 2. They have two, equal-length, flattened legs connected by the flattened staple crown. The legs are provided with short (blunt) chisel points. Their fastener-quality indexes, FWI and FSI, with respect to their withdrawal resistance, FWR, and shear resistance, FSR, respectively, are based on the following relationships (4.2, ref. 3):

$$FWI = 221.24\ (1.273\ (WW + WT)$$
$$FSI = 263{,}260\ (0.8476\ (WW + WT))^{1.5} / (3M + 40)$$

For any coating to be given consideration in determining joint effectiveness, the increase in delayed withdrawal resistance of coated staples, driven into green wood and tested after its seasoning to 12-pct moisture content, shall be at least 33 pct above that of identical bright staples. If the coating is more effective, its benefit can be prorated in determining the fastener's FWI.

Standard Pallet-Mat Assembly Nails and Staples

Standard, bright and coated, plain-shank, regular-stock steel nails and staples for pallet-mat assembly are described in Table 3. The nails have filleted flat heads and short (blunt) diamond or chisel points. The staples have two, equal-length, flattened legs, connected by the flattened staple crown, and short (blunt) chisel points. To allow for at least 3/16-in. (5-mm) clinching, the fastener length shall be equal to the thicknesses of the fastened and fastening members plus 3/16(+) in. (5 mm) less the depth of countersinking.

PERFORMANCE CRITERIA

The performance criteria, in lb,* are based on the 5-pct exclusion level and are determined as follows (4.2, 3rd ref.):

Fastener withdrawal resistance values, FWR:

$$FWR = 222.2\ (FWI)\ G^{2.25}\ P\ /\ (MC - 3)$$

Fastener head-pull-through resistance values, FHR:

For nails: $FHR = 1{,}250{,}000\ (HD^2 - WD^2)\ T\ G^{2.25}\ /\ (MC - 3)$

For staples: $FHR = 1{,}591{,}550\ \ CL\ \ WW\ \ T\ \ G^{2.25}\ /\ (MC - 3)$

Fastener shear resistance values, FSR:

$$FSR = 61.926\ FSI\ G\ T\ C\ /\ (MC - 3)$$

---

* 1 lb = 4.45 N

ORDERING INFORMATION

Orders for the pallet-assembly fasteners listed in this specification shall include the following information:

	EXAMPLE:
Quantity (weight or number of packaged fasteners)	50 lb bulk
Type and size, in in.	Pallet nail: $2\frac{1}{2} \times 0.120$
Material, grade, and finish	Steel, hardened, bright
Shank-thread description, in in.	Helical, $1\frac{3}{4}$TL, 0.142 TD, 60 TA
Head (crown) and point description, in in.	0.28 flat; pointless
MIBANT angle, in deg	16
FWI and FSI performance indexes	108 FWI; 124 FSI
NWPCA code number	10AA
Special information required	--------

Standard pallet-assembly fastener-package labels shall provide the required label information in a uniform manner and sequence, as indicated in Appendix X 1.

MATERIALS AND MANUFACTURING

The steel used for making the fasteners shall be made by any suitable means and the method of fastener manufacture shall be at the manufacturer's discretion to provide a product of specified properties. This specification covers the following three steel grades:

Regular-stock steel -- This grade is customary if no special mechanical properties are specified.

Stiff-stock steel -- This grade is stiffer than regular stock and results in higher resistance to buckling and bending and lesser MIBANT angles than regular stock.

Hardened steel -- This grade denotes heat-treated and tempered fasteners (Note 1), which provide higher resistance to buckling and bending and lesser MIBANT angles than regular-stock and stiff-stock fasteners.

NOTE 1 -- Warning: The driving of hardened-steel fasteners into certain materials is potentially hazardous because of the possibility of fastener breakage and flying particles. Hardened-steel fasteners should not be driven without appropriate precautions, the use of safety glasses and/or other equivalent protection.

Coatings and finishes are applied to fasteners as specified. The fastener dimensions apply to the fasteners prior to any coating or finishing.

Galvanized coating shall meet the requirements of weight and adherence given in ASTM A 153, Class D (2.6). The coating may be applied by the hot-dip, tumbling or mechanical galvanizing process. Galvanized wire for forming into steel staples shall be coated in accordance with ASTM A 614, Class 1 (2.7). Supplementary chromate or dichromate treatment may be used, to enhance corrosion resistance, subject to prior agreement between vendor and purchaser.

Coated fasteners shall be uniformly covered with a natural resin (cement) or conversion coating (thermo-plastic or thermo-setting polymer). Other coatings may be used subject to prior agreement between vendor and purchaser. If test verification of the effectiveness of the coating is required, it shall provide a minimum increase of 33 pct in delayed withdrawal resistance of the fastener when driven into green wood and tested after its seasoning to 12-pct moisture content in accordance with ASTM D 1761 (2.8). None of the coatings shall be tacky or gummy under ambient conditions normally encountered.

Helical shank deformations of the fasteners listed in this specification shall be applied by threading, in line with ASTM F 547 (2.1). However, other methods of applying shank deformations may be used where they are shown to be suitable for the fastener application, subject to prior agreement between vendor and purchaser.

The helical threads shall extend from the nail point to at least two-thirds of the nail length. The thread flanks on the nail-head side shall be almost perpendicular to the nail axis and the thread flanks on the point side shall be tapered toward the point. Both flat-bottom and round-bottom threads are suitable.

PHYSICAL REQUIREMENTS

The nails described in this specification shall be sufficiently ductile and tough to provide an average MIBANT angle, in line with ASTM F 680 (2.3), for 25 random samples of the lot under scrutiny, ranging from 8 to 28 deg for hardened steel nails, 29 to 46 deg for stiff-stock steel nails, and 47 deg and beyond for regular-stock steel nails which may be clinched, provided that not more than 8 pct of the nail heads or shanks fail partially or completely during impacting. In borderline cases, retesting is permitted. Because of the different wear condition of different MIBANT devices, correction factors may have to be applied which relate to the condition of the device in use.

The mechanical properties of the wire may be used in guiding in its selection. Thus, stiff-stock wire shall have a minimum tensile strength of 120,000 psi (827 Pa).

When MIBANT or equivalent data cannot be made available, stiff-stock steel nails shall have a minimum hardness of HR C 24, as determined by conversion of tensile strength to hardness in accordance with SAE J 417b (6.1); while hardened steel nails shall have a minimum hardness of HR C 37 unless specified otherwise.

DIMENSIONS AND TOLERANCES

The nominal fastener dimensions are given in Tables 1 to 3. The tolerances provided for in this specification are applicable to hammer and machine-driven nails and tool-driven staples. The tolerance for tool-driven nails shall be mated to the requirements of the driving equipment, subject to prior agreement between vendor and purchaser.

The nail length shall be measured from the underside of the head to the tip of the point. the nail-length tolerances shall be ± 0.078 in. (2 mm) for lengths from 1.00 in. (25 mm) up to and including 2.50 in. (64 mm) and 0.118 in. (3 mm) for lengths over 2.50 in. (64 mm). The length of tool-driven staples shall be measured from the top of the staple crown to the tip of the staple point. The staple-length tolerances shall be +0.016 and -0.062 in. (+0.5 and-1.6 mm).

The wire diameter of nails shall be measured away from the gripper marks and prior to the application of any coatings and finishes. The diameter tolerances shall be ±0.002 in. (0.05 mm). The leg width and thickness of tool-driven staples shall be measured prior to the application of any coating or along that leg portion which is not coated. The tolerances of the leg thicknesses and widths of tool-driven staples shall be as is indicated in the following tabulation:

Nominal Wire Diameter In.	Mm	Thickness Range In.	Mm	Width Range In.	Mm
0.062	0.16	0.054 to 0.058	0.14 to 0.15	0.062 to 0.066	0.16 to 0.17
0.072	0.18	0.064 to 0.068	0.16 to 0.17	0.072 to 0.076	0.18 to 0.19
0.080	0.20	0.070 to 0.078	0.18 to 0.20	0.082 to 0.090	0.21 to 0.23

The thread-crest diameters of threaded nails shall be the mean diametral dimension along the deformed portion of the shank. The tolerance shall be +0.004 and -0.000 in. (+0.10 and -0.00 mm).

The tread-angle tolerance of helically threaded nails shall be ±2 deg with the plane perpendicular to the nail axis.

- The nominal head diameters shall be based on the mean of the major and minor head diameters. The tolerance shall be ±10 pct of the nominal head diameter in individual measurements and the differences in major and minor head diameters shall not exceed 10 pct of the nominal head diameters. The shape of the heads may deviate from the round to permit tight nail collation, provided the bearing area of the head is not decreased by this change.

WORKMANSHIP

The fasteners covered in this specification shall be substantially clean and free of foreign material, corrosion products, and fine metal slivers. Cohered fasteners shall be properly aligned in their assembled form. Machine-quality fasteners shall be substantially free of foreign matter which could clog tools and machines.

TESTING

When required in the purchase order that tests be conducted on the performance of randomly selected fasteners of the lot involved, the test methods to be used shall be in accordance with ASTM F 680 (2,3) except that the MIBANT angle test shall be the governing test in preference of the conventional bend test and the Rockwell hardness test.

When required in the purchase order, the manufacturer shall furnish a certified test report which covers the fasteners in the lot (Note 2) involved.

NOTE 2 -- A lot shall consist of fasteners of a single shipment of the same type, grade, and finish taken from the same manufacturing lot.

REJECTION AND REHEARING

Fasteners that fail to conform to the requirements of this specification may be rejected. Rejection shall be reported promptly to the vendor in writing. In the case of dissatisfaction, the vendor may make a claim for a rehearing. Under these rehearing procedures, the lot shall be reinspected in the presence of representatives of both vendor and purchaser, unless special arrangements were negotiated at the time of ordering.

A random sample shall be obtained from the lot for examination as follows:

Test Type	Sample Size	Acceptance Number for Each Test (Note 3)
Non-destructive test	100	8
Destructive test	30	2
Functionally defective (Note 4)	100	2

NOTE 3 -- Acceptance number is the maximum number of defectives that may occur for acceptance of the lot.

NOTE 4 -- A functionally defective lot includes defects such as duds, lack of coating, incorrect coating or wrong grade. If the acceptance number is exceeded for any of the characteristics subject to rehecring, the lot represented by the sample shall be rejected.

The responsibility for the performance of the required inspection and testing in line with this specification is that of the fastener manufacturer, unless specified otherwise in the purchase order. The fastener manufacturer may use his own or any other suitable facilities for the performance of inspection and testing, unless specified otherwise in the purchase order. The purchaser shall have the right to perform any of the inspections and tests, where such inspections and tests are considered necesssary to ensure that the fasteners conform to the specified requirements.

PACKAGING

The fasteners shall be packaged in accordance with the supplier's standard practice which is acceptable to the carrier at the lowest rate, unless specified otherwise in the purchase order. Containers and packaging shall comply with the Uniform Freight Classification Rules (7.1) or National Motor Classification Rules (7.2). Marking of the containers for shipment of the fasteners shall be in accordance with Appendix X 1, unless specified otherwise in the purchase order.

TERMINOLOGY

The terminology used in this specification is in accordance with 2.1, 2.2, and 3.4. The following additional definitions are applicable:

Staple-wire thickness -- Staple leg dimension measured in direction parallel to crown.

Staple-wire width -- Staple-leg dimension measured in direction perpendicular to crown.

Staple-crown length -- Dimension between staple legs ~~and~~ along crown.

Staple width, staple-crown width -- Overall width of staple including that of both staple legs.

Helix -- Single thread crossing along shank of threaded fastener.

MIBANT angle -- Bend angle formed by nail shank or staple legs when deformed as cantilever using machine and method described in 2.3.

SYMBOLS USED IN SPECIFICATION

WD Wire diameter of fastener, in in.

WW Width of rectangular wire, in in.

WT Thickness of rectangular wire, in in.

TL Thread length, in in.

TA Thread angle, in deg

TD Thread-crest diameter, in in.

HD Head diameter, in in.

CL Staple-crown length, in in.

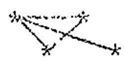
H Number of helixes per inch of thread length

P Penetration of threaded shank portion of fastener in fastening member, in in.

M MIBANT angle of fastener, in deg

C Number of fastener couples (single staple is one fastener)

Fasteners per joint:	2	3	4	5	6
Fastener couples per joint:	1	3	4	5	6

Example.- For 4-nail joint, use four couples as follows: 

G Oven-dry specific gravity

MC Moisture content during assembly, in pct of dry weight (limited to 28 pct for green wood)

T Thickness of fastened member, in in. (limited to maximum of $\frac{3}{4}$ in. in computation of head-pull-through resistance)

FWI Fastener withdrawal index

FSI Fastener shear index

FWR Fastener withdrawal resistance, in lb

FHR Fastener head pull-through resistance, in lb

FSR Fastener shear resistance, in lb

APPLICABLE DOCUMENTS

1 NWPCA Specifications and Standards (NWPCA, 1619 Massachusetts Avenue, N.W. Washington, D.C. 20036)

1.1 Specifications and Grades for Hardwood Warehouse, Permanent or Returnable Pallets, 1962. Amendments, February, 1969; August, 1974; and February, 1977.

1.2 Specifications and Grades for Warehouse, Permanent or Returnable Pallets of West Coast Woods, 1962. Amendments, February, 1969; August, 1974; and February, 1977.

1.3 Specifications and Grades for Warehouse, Permanent or Returnable Pallets of Southern Pine, 1974. Amendment, February, 1977.

1.4 Specifications for Softwood Plywood Pallets, PP 61-80, February, 1980.

1.5 NWPCA Logo-Mark Hardwood Pallet Standards, March, 1982.

1.6 NWPCA Logo-Mark West Coast Pallet Standard, March, 1982.

1.7 NWPCA Logo-Mark White Woods Pallet Standard, March, 1983.

2 ASTM Standards (ASTM, 1916 Race Street, Philadelphia, Pa. 19103)

2.1 Standard Definitions of Terms Relating to Nails for Use with Wood and Wood-Base Materials, ASTM F 547-77(84).

2.2 Standard Definitions of Terms Relating to Collated and Cohered Fasteners and Their Application Tools, ASTM F 592-80(84).

2.3 Standard Methods of Testing Nails, ASTM F 680-80(84).

2.4 Standard Specification for Driven Fasteners: Wire Nails, Spikes, and Staples, 8th Draft (Working Document, Not for Publication), October, 1982.

2.5 Standard Methods of Test for Pallets and Related Structures Employed in Materials Handling and Shipping, ASTM D 1185-85.

2.6 Standard Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware, ASTM A 153-78.

2.7 Standard Specification for Zinc Coated (Galvanized) Carbon Steel Wire, ASTM A 641-71a.

2.8Standard Methods of Testing Metal Fasteners in Wood, ASTM D 1761-74.

2.9 Standard Methods of Test for Rockwell Hardness and Rockwell Superficial Hardness of Metallic Materials, ASTM E 18-79.

3 ANSI/ASME Standards (ASME, 345 East 47th Street, New York, N.Y. 10017)

3.1 Pallet Definitions and Terminology, ANSI MH1.1.2, 1978.

3.2 Pallet Sizes, ANSI MH1.2.2, 1975.

3.3 Procedures for Testing Pallets, ANSI MH1.4.1, 1977.

3.4 Glossary of Terms in Pallet Field, Draft (Working Document, Not for Publication), March, 1984. Proceedings of '85 International Physical Distribution Conference, Tokyo, Japan, May 13 - 15, 1985.

4 U. S. Government Publications (General Services Administration, Washington, D.C. 20405) and Reports (U. S. Forestry Sciences Laboratory, P. O. Box 152, Princeton, W. Va. 24740)

4.1 Nails, Brads, Staples and Spikes: Wire, Cut and Wrought, Federal Specification FF-N-105B, March 17, 1971. Amendments, December, 1972, and October, 1974.

4.2 Wallin, Walter B., and Kenneth R. Whitenack. Life Expectancy and Cost per Use Analysis of Wooden Pallets, Skids, Etc., Forestry Sciences Laboratory, January, 1981. Also, Life Expectancy and Cost per Trip Analysis for Wooden Pallets and Related Structures, Ibidem, December, 1981. Also, Durability Analysis for Wooden Pallets and Related Structures, Ibidem, August, 1982.

4.3 Wallin, Walter B., and Kenneth R. Whitenack. Pallet Joint Shear Resistance, the F(2) Factor, Pallet Enterprise 1(4): 20-21, May/June, 1982. Also, Application of Joint Performance Criteria to Pallet Design, Ibidem, 1(5): 24-27, July/August, 1982. A lso, Fastener Equivalence Guides for Wooden Pallets, Ibidem, 1(6): 25, 27-29, September/October, 1982.

5 VPI&SU Bulletins and Reports (VPI&SU William H. Sardo Jr. Pallet and Container Research Laboratory, Blacksburg, Va. 24061)

5.1 Stern, E. George. Nails -- Definitions and Sizes, A Handbook for Nail Users, Bulletin No. 61, September, 1967.

5.2 Stern, E. George. MIBANT Test Criteria for Pallet Nails. Bulletin No. 115, December, 1972.

5.3 Stern, E. George. MIBANT Tests for Pallet Staples, Bulletin No. 149, April, 1977.

5.4 Stern, E. George, Walter B. Wallin, and Kenneth R. Whitenack. Pallet Rigidity, A Major Indicator of Pallet Durability, Unpublished Report, March, 1983. Proceedings of Symposium on Forest Products Research International, Pretoria, South Africa, April 22 -26, 1985.

5.5 White, Marshall S. Pallet Design System, PDS 1.0, Unpublished Report, June 26, 1984.

6 SAE Standard (SAE, 400 Commonwealth Drive, Warrendale, Pa. 15096)

6.1 Hardness Tests and Hardness Number Conversions, SAE J 417b, 19 .

7 Transportation Standards (UCC, 202 Union Station, Chicago, Il. 60606 and NMFTA, 1616 P. Street, N.W., Washington, D.C. 20036)

7.1 UCC. Uniform Freight Classification Rules.

7.2 NMFTA. National Motor Classification Rules.

Table 1

Standard, Helically Threaded, Stiff-Stock and Hardened-Steel, Pallet-Assembly Nails with Filleted Flat Head, Pointless or with Short (Blunt) or Medium (< 5/32-In.) Diamond or Chisel Point (not wider than wire diameter), and with Four Thread Flutes and 60 to 67-Deg Thread Angle

Code	Length In. (see Fig. 1)	Wire Diam. In.	Crest Diam. Stand. In. (see Fig. 2)	Crest Diam. Min. In.	Thread Angle Stand. Deg (see Fig. 4)	Thread Angle Max. Deg	Head Diam. In. (see Fig. 3)	Quality Index (FWI) with Respect to Withdrawal Resistance	Quality Index (FSI) with Respect to Shear Resistance for Given MIBANT Angle Deg: Stiff-Stock Nails 29	34	40	46	Hardened-Steel Nails (see Fig. 5) 8	12	16	20	24	28 Deg
1AA	1½	0.105	0.124	--	60	--	0.25	94	70	63	56	50	140	118	102	90	80	72
1AB			0.124	--	--	67	0.25	76	70	63	56	50	140	118	102	90	80	72
1BA			--	0.120	60	--	0.25	81	70	63	56	50	140	118	102	90	80	72
1BB			--	0.120	--	67	0.25	66	70	63	56	50	140	118	102	90	80	72
2AA	1¾	0.105	0.124	--	60	--	0.25	94	70	63	56	50	140	118	102	90	80	72
2AB			0.124	--	--	67	0.25	76	70	63	56	50	140	118	102	90	80	72
2BA			--	0.120	60	--	0.25	81	70	63	56	50	140	118	102	90	80	72
2BB			--	0.120	--	67	0.25	66	70	63	56	50	140	118	102	90	80	72
3AA	1¾	0.112	0.132	--	60	--	0.28	100*	78	70	62	55	154	130	112	99	88	80
3AB			0.132	--	--	67	0.28	80	78	70	62	55	154	130	112	99	88	80
3BA			--	0.128	60	--	0.28	87	78	70	62	55	154	130	112	99	88	80
3BB			--	0.128	--	67	0.28	70	78	70	62	55	154	130	112	99	88	80
4AA	2	0.105	0.124	--	60	--	0.25	94	70	63	56	50	140	118	102	90	80	72
4AB			0.124	--	--	67	0.25	76	70	63	56	50	140	118	102	90	80	72
4BA			--	0.120	60	--	0.25	81	70	63	56	50	140	118	102	90	80	72
4BB			--	0.120	--	67	0.25	66	70	63	56	50	140	118	102	90	80	72
5AA	2	0.112	0.132	--	60	--	0.28	100*	78	70	62	55	154	130	112	99	88	80
5AB			0.132	--	--	67	0.28	80	78	70	62	55	154	130	112	99	88	80
5BA			--	0.128	60	--	0.28	87	78	70	62	55	154	130	112	99	88	80
5BB			--	0.128	--	67	0.28	70	78	70	62	55	154	130	112	99	88	60
6AA	2	0.120	0.142	--	60	--	0.28	108	86	77	68	62	171	144	124	109	98	88
6AB			0.142	--	--	67	0.28	87	86	77	68	62	171	144	124	109	98	88
6BA			--	0.137	60	--	0.28	92	86	77	68	62	171	144	124	109	98	88
6BB			--	0.137	--	67	0.28	75	86	77	68	62	171	144	124	109	98	88
7AA	2¼	0.112	0.132	--	60	--	0.28	100*	78	70	62	55	154	130	112	99	88	80
7AB			0.132	--	--	67	0.28	80	78	70	62	55	154	130	112	99	88	80
7BA			--	0.128	60	--	0.28	87	78	70	62	55	154	130	112	99	88	80
7BB			--	0.128	--	67	0.28	70	78	70	62	55	154	130	112	99	88	80
8AA	2¼	0.120	0.142	--	60	--	0.28	108	86	77	68	62	171	144	124	109	98	88
8AB			0.142	--	--	67	0.28	87	86	77	68	62	171	144	124	109	98	88
8BA			--	0.137	60	--	0.28	92	86	77	68	62	171	144	124	109	98	88
8BB			--	0.137	--	67	0.28	75	86	77	68	62	171	144	124	109	98	88
9AA	2½	0.112	0.132	--	60	--	0.28	100*	78	70	62	55	154	130	112	99	88	80
9AB			0.132	--	--	67	0.28	80	78	70	62	55	154	130	112	99	88	80
9BA			--	0.128	60	--	0.28	87	78	70	62	55	154	130	112	99	88	80
9BB			--	0.128	--	67	0.28	70	78	70	62	55	154	130	112	99	88	80
10AA	2½	0.120	0.142	--	60	--	0.28	108	86	77	68	62	171	144	124	109	98	88
10AB			0.142	--	--	67	0.28	87	86	77	68	62	171	144	124	109	98	88
10BA			--	0.137	60	--	0.28	92	86	77	68	62	171	144	124	109	98	88
10BB			--	0.137	--	67	0.28	75	86	77	68	62	171	144	124	109	98	88
11AA	2½	0.135	0.159	--	60	--	0.30	120	103	92	82	73	204	172	148	130	117	105
11AB			0.159	--	--	67	0.30	96	103	92	82	73	204	172	148	130	117	105
11BA			--	0.154	60	--	0.30	103	103	92	82	73	204	172	148	130	117	105
11BB			--	0.154	--	67	0.30	84	103	92	82	73	204	172	148	130	117	105
12AA	2¾	0.120	0.142	--	60	--	0.28	108	86	77	68	62	171	144	124	109	98	88
12AB			0.142	--	--	67	0.28	87	86	77	68	62	171	144	124	109	98	88
12BA			--	0.137	60	--	0.28	92	86	77	68	62	171	144	124	109	98	88
12BB			--	0.137	--	67	0.28	75	86	77	68	62	171	144	124	109	98	88
13AA	2¾	0.135	0.159	--	60	--	0.30	120	103	92	82	73	204	172	148	130	117	105
13AB			0.159	--	--	67	0.30	96	103	92	82	73	204	172	148	130	117	105
13BA			--	0.154	60	--	0.30	103	103	92	82	73	204	172	148	130	117	105
13BB			--	0.154	--	67	0.30	84	103	92	82	73	204	172	148	130	117	105
14AA	3	0.120	0.142	--	60	--	0.28	108	86	77	68	62	171	144	124	109	98	88
14AB			0.142	--	--	67	0.28	87	86	77	68	62	171	144	124	109	98	88
14BA			--	0.137	60	--	0.28	92	86	77	68	62	171	144	124	109	98	88
14BB			--	0.137	--	67	0.28	75	86	77	68	62	171	144	124	109	98	88
15AA	3	0.135	0.159	--	60	--	0.30	120	103	92	82	73	204	172	148	130	117	105
15AB			0.159	--	--	67	0.30	96	103	92	82	73	204	172	148	130	117	105
15BA			--	0.154	60	--	0.30	103	103	92	82	73	204	172	148	130	117	105
15BB			--	0.154	--	67	0.30	84	103	92	82	73	204	172	148	130	117	105
16AA	3½	0.135	0.159	--	60	--	0.30	120	103	92	82	73	204	172	148	130	117	105
16AB			0.159	--	--	67	0.30	96	103	92	82	73	204	172	148	130	117	105
16BA			--	0.154	60	--	0.30	103	103	92	82	73	204	172	148	130	117	105
16BB			--	0.154	--	67	0.30	84	103	92	82	73	204	172	148	130	117	105

* Basis of comparison.

Table 2

Standard, Bright and Coated, Plain-Shank, Regular-Stock Steel, $\frac{1}{2}$-In. Wide, Pallet-Assembly Staples with Two, Equal-Length, Flattened Legs and Short (Blunt) Chisel Points

Code	Length In.	Nominal Wire Diameter In.	Cross-Section Thickn. In. (Used for Determination of FWI and FSI Only)**	Cross-Section Width In. (Used for Determination of FWI and FSI Only)**	Finish Bright	Finish Coated *	Quality Index (FWI) with Respect to Withdrawal Resistance Bright	Quality Index (FWI) with Respect to Withdrawal Resistance Coated*	Quality Index (FSI) with Respect to Shear Resistance for Given Computed MIBANT Angle, Deg for Bright or Coated Fasteners 85	115	150 Deg
1	2	0.062	0.055	0.061	x	--	33	--	--	--	17
2					--	x	--	50	--	--	17
3		0.072	0.067	0.073	x	--	39	--	--	28	--
4					--	x	--	59	--	28	--
5		0.080	0.075	0.080	x	--	44	--	42	--	--
6					--	x	--	65	42	--	--
7	$2\frac{1}{4}$	0.062	0.055	0.061	x	--	33	--	--	--	17
8					--	x	--	50	--	--	17
9		0.072	0.067	0.073	x	--	39	--	--	28	--
10					--	x	--	59	--	28	--
11		0.080	0.075	0.080	x	--	44	--	42	--	--
12					--	x	--	65	42	--	--
13	$2\frac{1}{2}$	0.072	0.067	0.073	x	--	39	--	--	28	--
14					--	x	--	59	--	28	--
15		0.080	0.075	0.080	x	--	44	--	42	--	--
16					--	x	--	65	42	--	--
17	3	0.072	0.067	0.073	x	--	39	--	--	28	--
18					--	x	--	59	--	28	--
19		0.080	0.075	0.080	x	--	44	--	42	--	--
20					--	x	--	65	42	--	--
21	$3\frac{3}{4}$	0.072	0.067	0.073	x	--	39	--	--	28	--
22					--	x	--	59	--	28	--
23		0.080	0.075	0.080	x	--	44	--	42	--	--
24					--	x	--	65	42	--	--

* Increase in delayed withdrawal resistance of coated staples, driven into green wood and tested after its seasoning to 12-pct moisture content, shall be at least 33 pct above that of identical bright staples. If the coating is more effective, its benefit can be prorated in determining its FWI (see 5.2.2).

** For permissible range, see 10.3.

Table 3

Standard, Bright and Coated, Plain-Shank, Regular-Stock Steel, Nails and Staples for Pallet-Mat Assembly: Nails with Filleted Flat Heads and Short (Blunt) Chisel Points and Staples with Two, Equal-Length, Flattened Legs Connected by Staple Crown and Short (Blunt) Chisel Points and ½-in. Wide

NAILS

Code	Length In.	Wire Diameter In.	Head Diameter In.
1	1	0.086	0.19
2		0.091	0.22
3		0.099	0.25
4		0.105	0.25
5	1¼	0.086	0.19
6		0.091	0.22
7		0.099	0.25
8		0.105	0.25
9	1½	0.086	0.19
10		0.091	0.22
11		0.099	0.25
12		0.105	0.25
13	1¾	0.086	0.19
14		0.091	0.22
15		0.099	0.25
16		0.105	0.25
17	2	0.086	0.19
18		0.091	0.22
19		0.099	0.25
20		0.105	0.25

STAPLES

Code	Length In.	Nominal Wire Diameter In.
1	1	0.062
2		0.072
3		0.080
4	1¼	0.062
5		0.072
6		0.080
7	1½	0.062
8		0.072
9		0.080
10	1¾	0.062
11		0.072
12		0.080
13	2	0.062
14		0.072
15		0.080

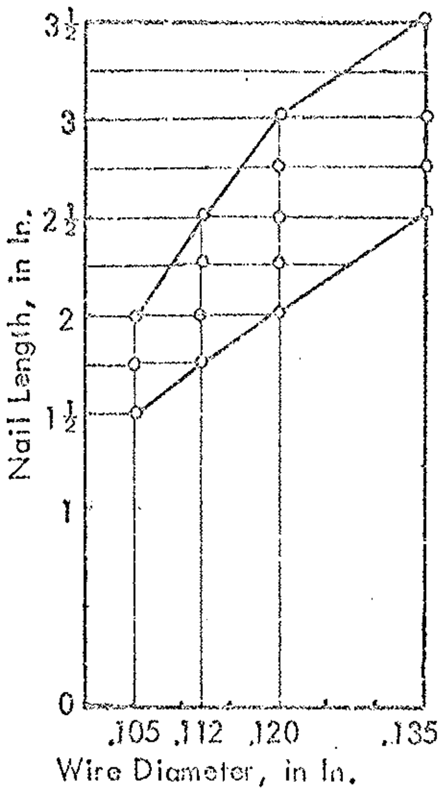


Fig. 1.- Standard pallet-assembly nails, helically threaded, stiff-stock or hardened steel.

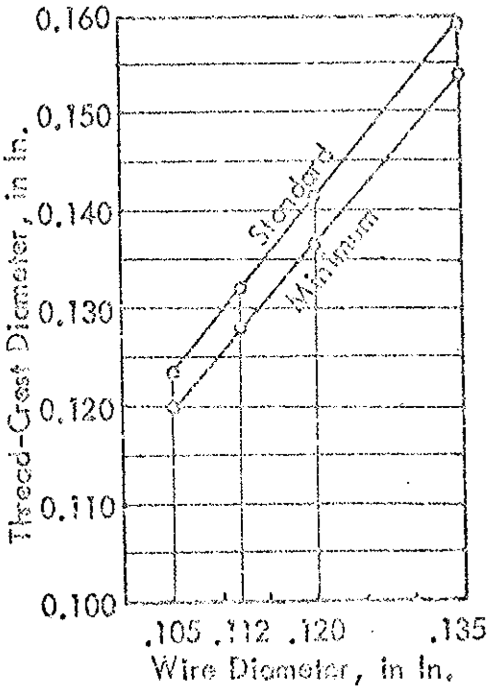


Fig. 2.- Relationship between thread-crest diameter and wire diameter of standard pallet-assembly nails.

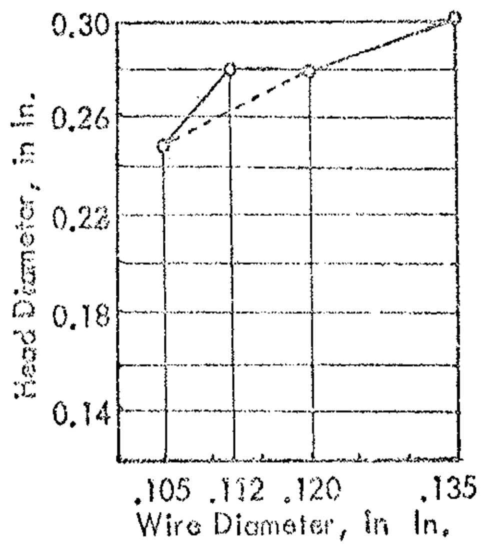


Fig. 3.- Relationship between head diameter and wire diameter of standard pallet-assembly nails.

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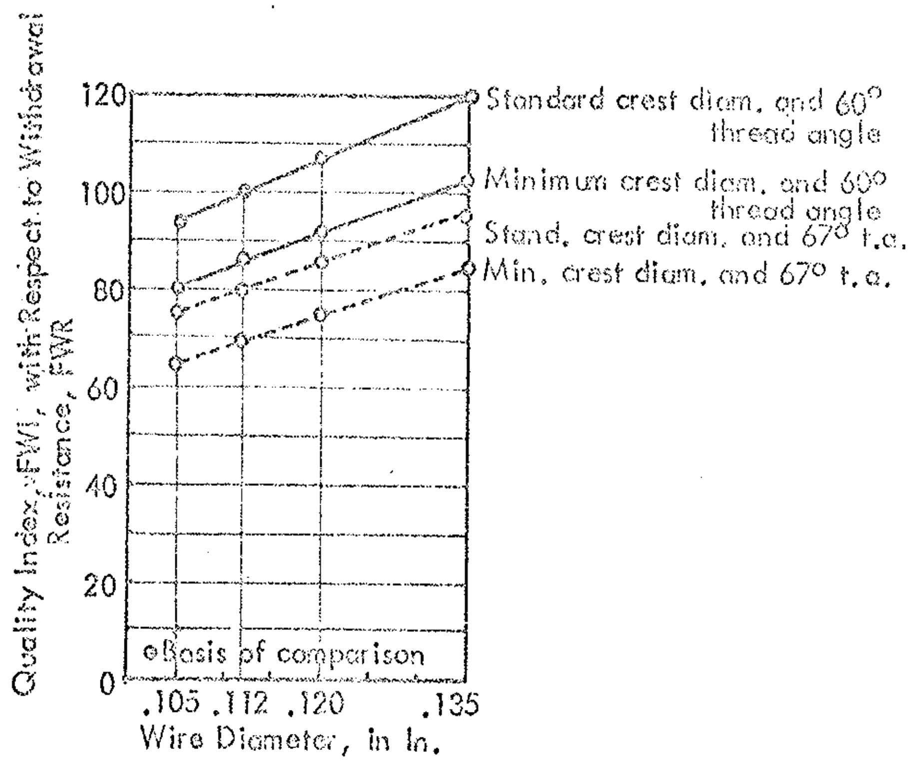


Fig. 4.- Relationship between quality index with respect to withdrawal resistance and wire diameter, thread-crest diameter, and thread angle of standard pallet-assembly nails.

84070905

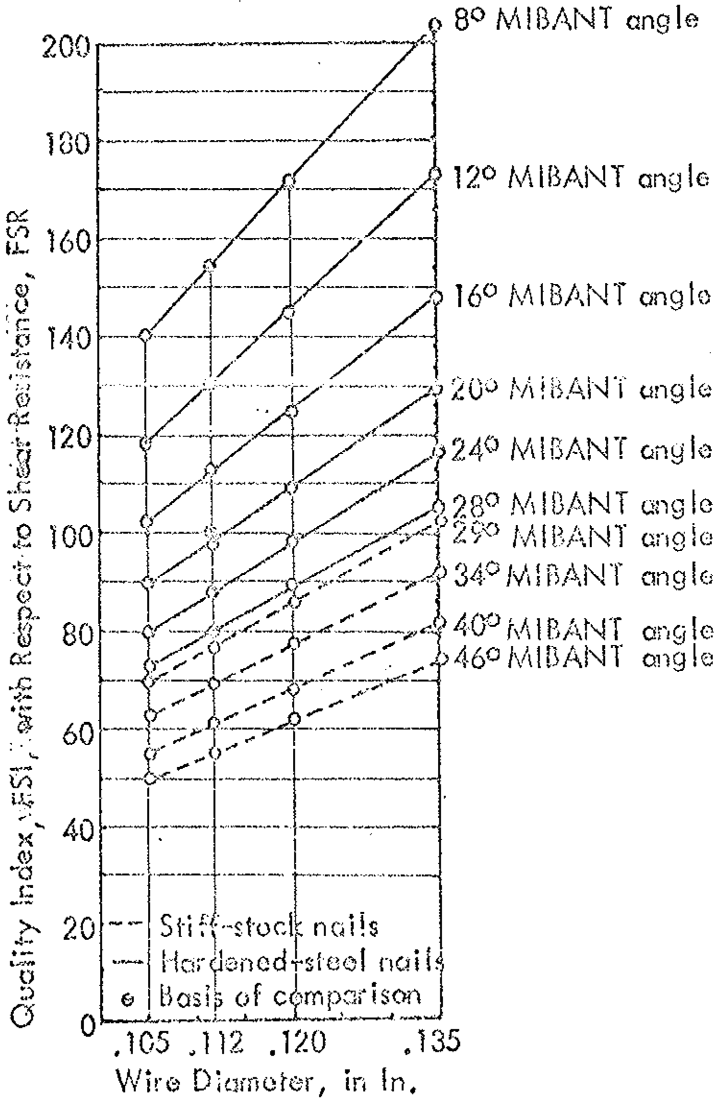


Fig. 5.- Relationship between quality index with respect to shear resistance and wire diameter and MIBANT angle of standard pallet-assembly nails.

APPENDIX X 1

Standard Fastener Package Labels

1. NAILS Quantity: ..... lb

Manufacturer: ....
Manufacturer's Code: .... Lot: ....

Type: Pallet Nail ....
Length: .... in. Wire Diameter: .... in.

Metal: Regular Steel .... Stiff-Stock .... Hardened Steel ....
Finish: Bright .... Galvanized .... Coated (Type) ....

Shank Deformations: Helically Threaded
Thread Type: Heli cal ....
Length: .... in. Crest Diameter: .... in.
Number of Flutes: 4
Angle: .... deg Helixes per Inch: ....

Head Type: Flat .... Head Diameter: .... in.
Point Type: ....

MIBANT Angle: .... deg Performance Index FWI: ....
NWPCA Code No.: .... Performance Index FSI: ....

2. STAPLES Quantity: ....

Manufacturer: ....
Manufacturer's Code: .... Lot: ....

Type: Pallet Staple ....
Length: .... in. Width: ...., in.
Nominal Wire Diameter: .... in. Leg Cross-Section: ...... x ...... in.

Metal: Regular Steel Finish: Bright ..... Galvanized ..... Coated (Type) .....
....

Crown Type: Flattened
Point Type: Chisel

MIBANT Angle: .... deg Performance Index FWI: ....
NWPCA Code No.: .... Performance Index FSI: ....

CIB-W18/18-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE INFLUENCE OF THE ORIENTATION OF MECHANICAL JOINTS ON THEIR MECHANICAL PROPERTIES

by

I Smith and L R J Whale
TRADA
United Kingdom

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

INTRODUCTION

Following some discussion at the last CIB-W18 meeting on how test specimens should be 'orientated' during static bending tests on structural size members, Dr. Smith was asked to consider the question of test specimen orientation in relation to mechanical joints.

It is usually assumed by those conducting tests on mechanical joints and by those using the resulting test data to establish design resistances, that joints are made in such a way that the fastener(s) is located in clear wood, i.e. features such as knots, fissures and sloping grain are not included in the area of the joint. Joint resistance is characterised using tests on axially loaded joints with a small number (one to four is typical) of fasteners per joint. For such joints orientation of test specimens refers to:

(i) whether the applied forces produce compressive or tensile loading, and

(ii) the angle to the grain at which the fastener(s) load each of the jointed members,

(iii) for solid timber members the positioning of the fastener(s) relative to the growth rings in the radial-tangential plane.

(Strictly speaking item (iii) constitutes part of the procedure for sampling materials but by common usage it is often incorporated as part of the testing procedure).

Through study of typical types of joints this paper examines the relative importance of various factors which define the orientation of a mechanical timber joint. The related topic of what level of distinction should be made in design codes between different jointing situations is also addressed.

TENSION JOINTS v COMPRESSION JOINTS

Fig. 1 shows a schematic representation of compression and tension embedment specimens. The stress distributions for tension embedment specimens and compression embedment specimens are clearly quite different. It is only in situations where the width of the specimen is very large compared with the fastener diameter, e.g. nails in plywood, that tension embedment characteristics will tend to approach those for compressive embedment situations. In general it is to be expected that both stiffnesses and ultimate strengths for joints loaded in tension will be different from those for nominally identical joints loaded in compression.

On the basis of recent tests conducted by TRADA:

(a) Laterally loaded nailed joints with members loaded parallel to grain have a ratio of initial slip modulus in tension to initial slip modulus in compression in the order of 0.80 to 1.30. The ratio varies depending upon factors such as nail diameter and timber species.

(b) Laterally loaded nailed joints with members loaded parallel to grain have a ratio of ultimate capacity in tension to ultimate capacity in compression in the order of 1.00 to 1.05.

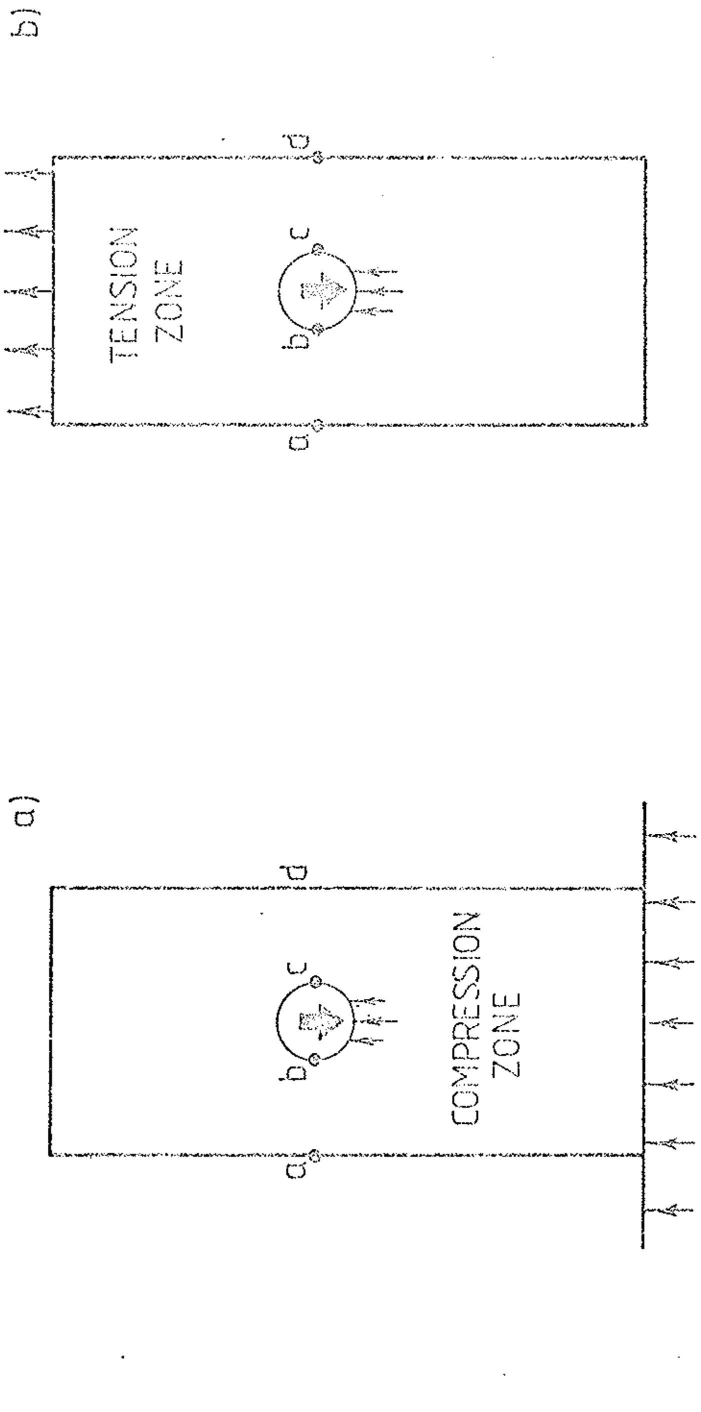
(c) Laterally loaded bolted joints with members loaded parallel to the grain have a ratio ofslip modulus in tension to initial slip modulus in compression in the order of 0.9 to 1.40. The ratio varies depending upon factors such as nail diameter and timber species.

(d) Laterally loaded bolted joints with members loaded parallel to grain have a ratio of ultimate capacity in tension to ultimate capacity in compression in the order of 1.00 to 1.10.

These results apply to joints with an end distance of not less than seven times the bolt diameter or twenty times the nail diameter, and an edge distance of not less than 1.5 times the bolt diameter or five times the nail diameter. Lower tensile than compressive strengths may well be observed for joints with end and/or edge distances less than these.

It seems probable that laterally loaded joints with connectors of the split-ring or shear-plate types will have greater differences in ultimate tensile and compressive capacities than the differences observed for joints wit nails or bolts.

Fig. 1



Tension embedment specimen

Compression embedment specimen

INFLUENCE OF ANGLE TO THE GRAIN AT WHICH THE FASTENERS LOAD EACH OF THE JOINTED MEMBERS

Owing to differences in embedment characteristics for fasteners loading timber at different angles to the grain, both stiffnesses and strengths of joints are functions of the direction to the grain at which the fastener(s) loads each joint member. Embedment tests at TRADA have shown that:

(1) for nails the initial embedment stiffness for loading perpendicular to the grain is in the order of 0.65 to 0.90 of that for loading parallel to the grain,

(2) for bolts the initial embedment stiffness for loading perpendicular to the grain is in the order of 0.35 to 0.45 of that for loading parallel to the grain,

(3) for nails the ultimate embedment strength for loading perpendicular to the grain is in the order of 0.75 to 1.15 of that for loading parallel to the grain, and

(4) for bolts the ultimate embedment strength for loading perpendicular to the grain is in the order of 0.40 to 0.60 of that for loading parallel to the grain.

Within the ranges quoted, the actual ratio of a perpendicular to grain property to the equivalent parallel to grain property varies depending upon factors such as fastener diameter and timber species.

Although variables such as member thicknesses and material properties for the fastener(s) also influence joint characteristics it is obvious that the directions of applied forces relative to joint member axes can significantly influence a joint's stiffness and strength properties. As an illustration consider the two types of nailed joints shown in Fig. 2 (type N1) and Fig. 3 (type N2) and the two types of bolted joints shown in Fig. 4 (type B1) and Fig. 5 (type B2). The following results were obtained:-

1. $\dfrac{\text{Mean initial slip modulus for type N2 joints}}{\text{Mean initial slip modulus for type N1 joints}} = 0.855$

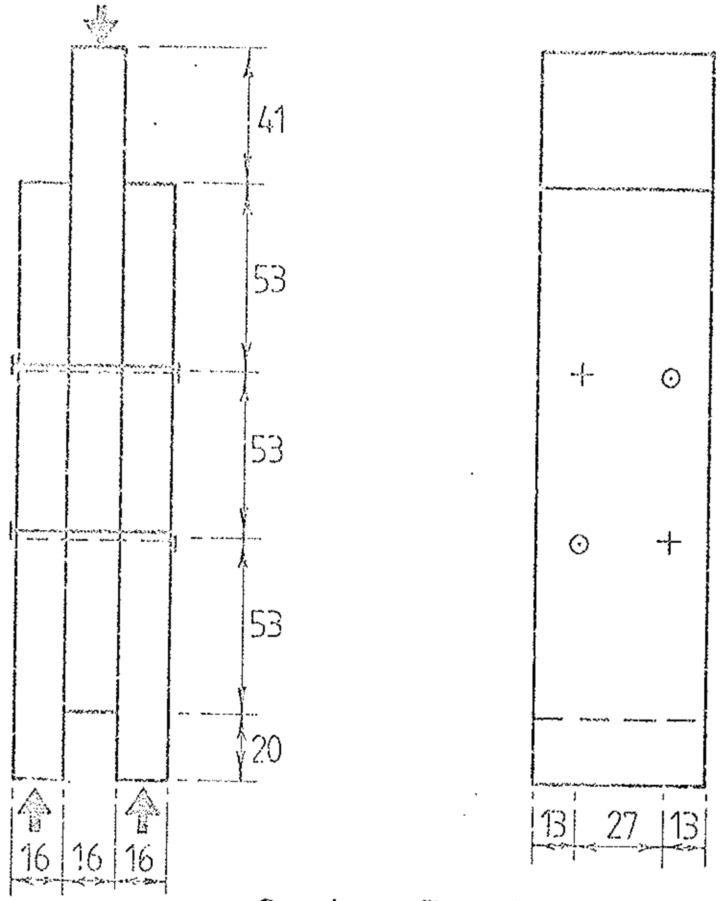
2. $\dfrac{\text{Mean initial slip modulus for type B2 joints}}{\text{Mean initial slip modulus for type B1 joints}} = 0.457$

3. $\dfrac{\text{Mean ultimate capacity for type N2 joints}}{\text{Mean ultimate capacity for type N1 joints}} = 1.087$

4. $\dfrac{\text{Mean ultimate capacity for type B2 joints}}{\text{Mean ultimate capacity for type B1 joints}} = 0.741$

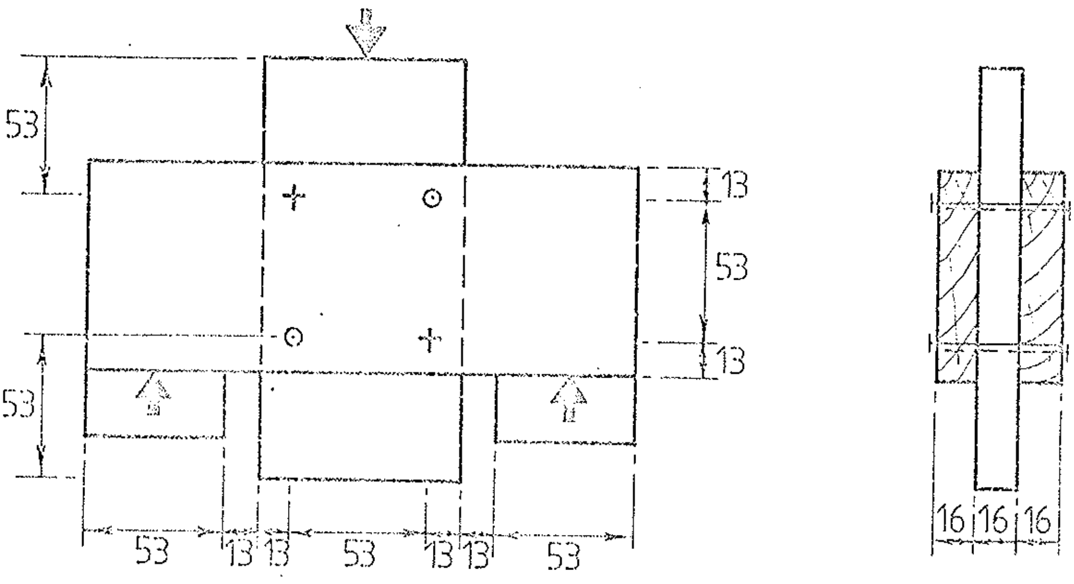
NAILED JOINTS

Fig. 2



Specimen Type N1

Fig. 3



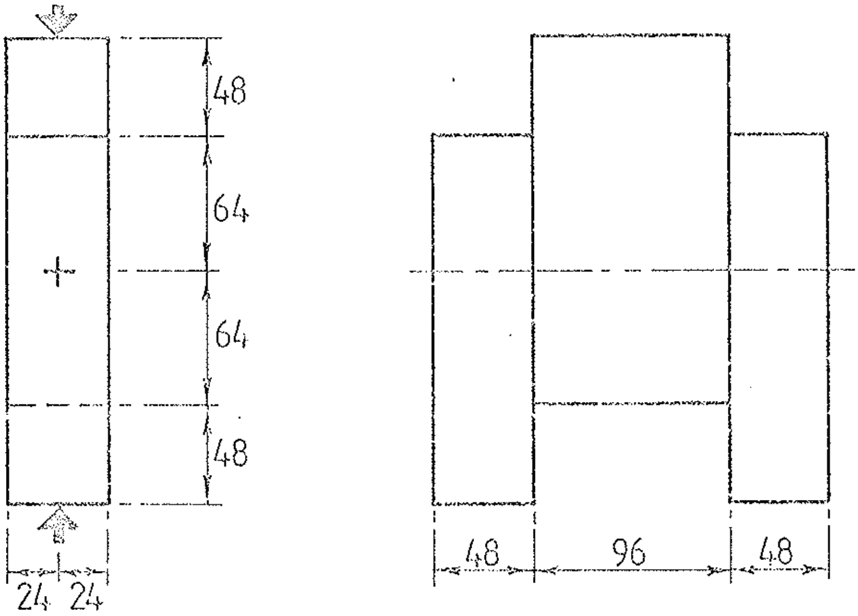
Specimen Type N2

Notes:-

1. Timber members planed all round.
2. Nails: 50mm long x 2·65mm diameter round plain head bright wire nails to BS 1202: Part 1: 1974
3. Nails driven without pre-bored lead holes. 0·5mm shims placed between members when the nails are driven and removed prior to testing.
4. Relative displacement of adjacent joint members in the direction of the applied load relates to the level of the centre of the nail group.
5. Test speed corresponds to a cross-head movement of 2·5mm per minute.

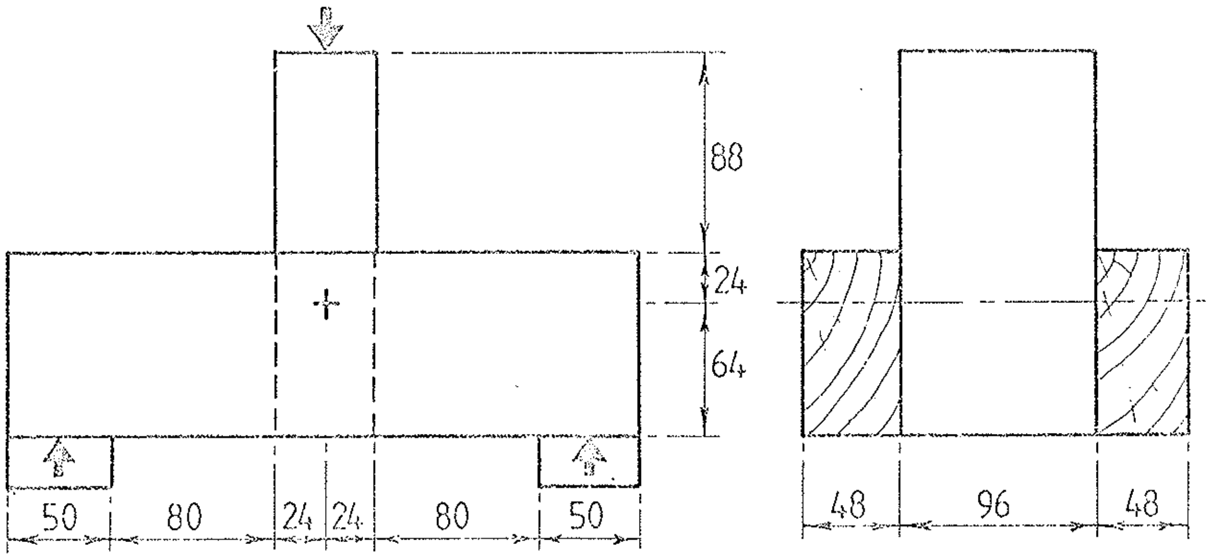
Fig. 4

BOLTED JOINTS



Specimen Type B1

Fig. 5



Specimen Type B2

Notes:-

1. Timber members planed all round.
2. Bolts: Metric Black Hexagon Head, M16, strength grade 4·6, to BS 4190:1967.
3. Washers: 48 x 48 x 4mm thick mild steel.
4. The joint is made finger tight prior to testing.
5. Relative displacement of adjacent joints timbers in the direction of the applied load relates to the level of the bolt axis.
6. Test speed corresponds to a cross-head movement of 2·5mm per minute.

INFLUENCE OF ORIENTATION OF FASTENER OR CONNECTOR IN RADIAL - TANGENTIAL PLANE

The question of whether or not the orientation of a fastener or connector relative to the growth rings in the radial-tangential plane significantly influences its stiffness and strength characteristics under lateral loadings, depends upon factors such as: the type of fastener, dimensions of the fastener and jointed members and natural features of the wood such as the width of the growth rings and relative cleavage strengths in different planes.

Available data seems to indicate that orientation of fasteners or connectors in the radial-tangential plane:

(i) has relatively little influence under lateral loadings on the stiffness characteristics of joints with fasteners such as nails or bolts or joints with shear plate connectors, (this may not be true for joints with punched metal plates, toothed-plate or split-ring connectors).

(ii) has only a small influence under lateral loadings on the strength characteristics of joints with fasteners such as nails or bolts, and,

(iii) can have a significant influence upon the ultimate capacities of laterally loaded joints with connectors of the shear-plate and split-ring types.

EFFECT OF INCLUDING 'UNCLEAR' TIMBER IN THE AREA OF A JOINT

As was previously mentioned joint properties are usually characterised by tests on specimens that are made in such a way that fasteners are located in 'clear' wood. In real structures however, even when correctly stress graded timber is used, fasteners may be inserted near to or coincident with features such as knots and sloping grain. There is little data in the literature from which reliable estimates can be made of the influences of various 'grade' features on the strengths of timber joints.

It seems obvious that for certain types of joints, e.g. laterally loaded joints with large bolts or split-ring connectors, that features such as splits and sloping grain will produce significant losses in ultimate capacities compared with those for similar joints in clear wood. Results of some recent embedment tests at TRADA illustrate that for other situations, e.g. laterally loaded joints with nails or small diameter bolts, inclusion of grade features in the area of a joint does not necessarily produce significant losses in stiffnesses or strengths. Table 1 shows results from compression tests on 6.5mm (3 ply) French pine plywood embedment specimens with 3.75mm common wire nails loading the plywood parallel or perpendicular to the grain of the face veneers. (Each plywood specimen was tested parallel to the grain of the face veneers, then using a new nail location it was tested perpendicular to the grain of the face veneers). Each of the original twenty specimens were cut from a different sheet of plywood. After testing, five of the specimens were judged to have produced invalid results. In four instances this was because features such as knots or voids were found in the centre veneer, when the specimen was cut up following testing. In the remaining specimen (number 19) the result was invalid because an incorrect test method was used. Five re-tests were made using pieces of material side matched to the original specimens. In general there was no significant influence upon calculated statistics for stiffness or strength from substituting results for re-tests in lieu of original values, especially for strength. Bearing in mind that in laterally loaded nailed joints, forces are usually distributed between a number of fasteners, it seems unlikely that localised features such as knots and sloping grain will seriously influence the strength or stiffness characteristics of such joints. End splits are likely to significantly reduce a joint's resistance whatever type of fasteners are used.

From the above, there is clearly a need for an improved understanding of the effect of including unclear timber in the area of a joint. This would lead not only to better assignments of characteristic resistances but also to better specification of code clauses dealing with workmanship.

SENSITIVITY OF WOODEN STRUCTURES TO VARIATIONS IN LOAD - DEFORMATION CHARACTERISTICS OF MECHANICAL TIMBER JOINTS

Theoretical studies have been conducted at TRADA to assess the sensitivity of a range of wooden structural components to variations in the load-deformation characteristics of mechanical joints, (1, 2, 3). Based on these studies it is concluded for components such as built-up beams, stressed skin panels, trusses and racking panels that provided loads at given joint deformations are specified to a reasonable precision, (in the order of $\mp$ 20% of their true values), then deformations, member stresses and joint forces can be predicted to within at least 10% of their true values. It should be noted that this pre-supposes that a component's behaviour can be accurately modelled using available methods of structural analysis.

On the basis of the above it is recommended that differences in joint stiffness and strength properties of less than 20% should be neglected when catagorising jointing situations for the writing of structural timber design codes.

Typical embedment results – 6·5 mm (3 ply) French pine plywood with 3·75 mm common wire nail

Replicate	Loading parallel to grain of face veneers		Loading perpendicular to grain of face veneers	
	$K_{initial}$ (kN/mm)	P_{ult} (kN)	$K_{initial}$ (kN/mm)	P_{ult} (kN)
1	7·40	1·580	23·76	1·720
2	17·20 (11·64)	1·578 (1·364)	7·80 (15·60)	0·934 (1·334)
3	20·96	1·660	22·02	1·630
4	14·48	1·392	19·86	1·450
5	13·60	1·634	9·80	1·264
6	12·60	1·226	13·12	1·452
7	12·88	1·156	13·58	1·410
8	10·50	1·276	16·44	1·246
9	14·60	1·340	14·10	1·024
10	20·64 (11·04)	1·286 (1·190)	18·20 (14·20)	1·296 (1·214)
11	8·08 (14·96)	1·012 (1·018)	17·40 (18·94)	1·302 (1·326)
12	13·24	1·390	11·58	1·206
13	13·76	1·272	16·92	1·294
14	9·68	1·140	10·20	1·180
15	15·04	1·232	16·00	1·288
16	12·84	1·504	10·04	1·300
17	12·72 (13·80)	1·350 (1·250)	16·80 (10·76)	1·632 (1·436)
18	17·52	1·252	17·20	1·700
19	26·28* (15·94)	1·614* (1·242)	26·20* (4·60)	1·694* (1·188)
20	20·08	1·342	22·00	1·420
Mean	14·096 [13·828]	1·349 [1·323]	15·622 [15·036]	1·355 [1·354]
S.D.	3·771 [3·145]	0·172 [0·163]	4·393 [4·720]	0·206 [0·174]
C.V. (%)	26·75 [22·74]	12·72 [12·33]	28·12 [31·39]	15·21 [12·82]
5%tile	7·893 [8·655]	1·066 [1·055]	8·395 [7·271]	1·016 [1·068]

Key: $K_{initial}$ = initial stiffness, P_{ult} = ultimate capacity

( ) signifies a repeat observation,

[ ] signifies value calculated substituting results from retests for original values,

* signifies that the value is excluded from the statistical analysis.

CONCLUSIONS

1. For mechanical timber joints 'orientation' refers to:
   (i) whether the applied forces produce compressive or tensile loading, and

   (ii) the angle to the grain at which the fastener(s) loads each of the jointed members,

   (iii) for solid timber members the positioning of the fastener(s) relative to the growth rings in the radial-tangential plane.

2. Because stress fields in the vicinity of fasteners are quite different for nominally identical joints subjected to compressive and tensile loadings, it is to be expected that in general stiffness and strength properties will differ for tension and compression joints. This does not of necessity imply that distinctions should be made between tension and compression joints during a design process. The need for distinctions is dominantly a function of the type of fasteners or connectors used and the materials used for the joint members.

3. For laterally loaded nailed joints it is not necessary to distinguish between different joint 'orientations' in ultimate capacity calculations or when assigning stiffness properties within structural analyses.

4. For laterally loaded bolted joints there is need to distinguish between different joint orientations in ultimate capacity calculations and when assigning stiffness peroperties within structural analyses.

5. For connectors of the split-ring, shear-plate, toothed -plate and punched metal-plate types, distinctions will probably need to be made in design calculations between different joint orientations.

6. There is need for an improved understanding of the effect of including 'unclear' timber in the area of a joint. This would lead not only to better assignments of characteristic resistances but also to better specification of code clauses dealing with workmanship.

7. The scope of test programmes for various types of mechanical joints should reflect the requirements implicit in conclusions 3 to 5 above.

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CIB-W18/18-7-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

INFLUENCE OF NUMBER OF ROWS OF FASTENERS OR CONNECTORS UPON THE ULTIMATE CAPACITY OF AXIALLY LOADED TIMBER JOINTS

by

I Smith
TRADA
United Kingdom
and
G Steck
University of Karlsruhe
Federal Republic of Germany

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

1. INTRODUCTION

Through various investigations, (1, 2, 3, 4, 5, 6,7), it is known that the ultimate load capacity of an axially loaded timber joint with n rows of fasteners or connector units is less than n times the capacity of a similar joint with a single row of fasteners or connector units. Here a line of fasteners perpendicular to the axes of jointed members is termed a row. A line of fasteners parallel to the axes of jointed members is termed a column. Attempts have been made to explain the loss in joint efficiency by analogy to stress concentrations that occur in lap joints with continuous contact surfaces, (3, 8, 9, 10, 11). Proposed theoretical models consider fasteners and jointed members to behave as a number of interconnected springs with either linear or non-linear load-displacement characteristics, (3, 8, 9, 11). Fairly good agreement has been found between theory and experiment for the influence of number of rows of fasteners upon load-displacement relationships for various types of mechanical joints at small displacements, (12). Unfortunately the same is not always true for ultimate capacity predictions, (11, 12). Available theories tend to overestimate the strengths of joints with multiple rows of fasteners, in situations where failure can be as a result of brittle fracture, e.g. shear-plate connectored joints.

A number of current 'structural timber design codes', (e.g. 13, 14, 15) make explicit allowance for the influence of the number of rows of fasteners upon the design strengths of axially loaded joints. Codes such as the CIB Code, (13), and BS 5268: Part 2, (14), contain relatively simple rules which are functions of the type of fastener and the number of rows, Figures 1 to 8. Other Codes, for example CSA 086, (15), contain relatively complex rules which are also functions of the cross-sectional areas of the jointed members. According to Wilkinson, (12), the CSA 086 rules were derived from the work of Lantos, (9). Despite their existence various Code rules are believed to be nominal rather than rational allowances for observed effects.

In response to discussions prompted at the last CIB-W18 meeting by Gunter Steck's paper, (16), this paper examines the correctness of existing design code rules for the influence of number of rows of fasteners upon the ultimate capacity of axially loaded timber joints.

2. TEST DATA AND COMPARISON WITH EXISTING CODE RULES

The following notation is adopted:

$$\beta = \frac{P}{pnm} \qquad (1)$$

where β = observed reduction factor,
P = observed ultimate capacity of a joint with nm fasteners or connector units,
p = observed ultimate capacity of a joint with one fastener or connector unit,
n = number of rows of fasteners or connector units,
m = number of columns of fasteners or connector units.

Relatively little relevant data has been identified in the literature. Despite this, some comparisons are possible; Figures 1 to 8 show β values plotted against n for various types of joints. Individual points represent mean observations. No distinction has been made between data for compressive and tensile loadings. Also Figures 1 to 8 show relevant design rules specified in the CIB code, (13) and BS 5268: Part 2, (14).

2.1 NAILED JOINTS

In Figures 1 to 4 experimental results are compared with theoretical predictions using Lantos' theory, (9). Even though the theory is for fasteners with a linear-elastic response, the predictions are reasonably good. The sensitivity of β to any variations in joint member cross-sections is indicated in the same figures by the curves for

$A_m = \frac{1}{2}A_s = A_{min}^{CIB}$ or $A_m = A_s = A_{min}^{CIB}$, where:

A_m = cross-sectional area of main member,

A_s = cross-sectional area of side member, and

A_{min}^{CIB} = minimum cross-section permitted by CIB Code, (13),

$= 7 \times 5 \times d \times (m + 1)$

$(E_m = E_s = 10^4\ N/mm^2$, nail spacing = 20d)

The experimental data is for fairly unusual joints with side member stiffnesses (E_s A_s) either 2.0 or 2.67 times that of the main member (E_m A_m). For 'normal joints' E_s A_s is in the order of 0.5 to 1.0 times E_m A_m. The effect of the number of rows of nails on the effective strength capacity per nail would be expected to be less than that observed in the tests by Potter and Nozynski. This is why the proposed design rules in Figures 1 to 4 lie above the experimental observations. Also some account is taken in the proposal of the expected reduction in variabilty of strength with any increases in the number of nails per joint, (18).

2.2 DOWEL JOINTS AND BOLTED JOINTS

A comparison between Lantos' theory and the current CIB Code design rule for dowel joints is given in Figure 5. This suggests that the CIB Code rule is conservative for joints with more than four rows of dowels. The validity of the Lantos theory will in part depend upon whether dowel joints with multiple rows of fasteners have a ductile or a brittle failure mode.

In Figure 6 experimental data for bolted joints is compared with existing design rules. It is likely that the apparent lack of a trend relating β and n, results from a change in failure mode (ductile to brittle), between joints with one bolt and joints with more than one bolt in a row.

2.3 SPLIT-RING JOINTS AND SHEAR-PLATE JOINTS

Figures 7 and 8 show a comparison of available experimental data with existing design rules for split-ring joints and for shear-plate joints respectively. There appears to be a significant influence of any increase in the number of rows of connectors for both types of connector. Substantial losses in strength per connector for $n = 2$ are thought likely to result from changes in failure mode relative to joints with $n = 1$.

3. PROPOSED DESIGN RULES FOR AXIALLY LOADED JOINTS

$$P_d = \alpha p_d \; nm \qquad (2)$$

where:

P_d = ultimate limit state design capacity of a joint with nm fasteners or connector units,

p_d = ultimate limit state design capacity of a joint with one fastener or connector unit, and

α = reduction factor.

3.1 NAILED JOINTS

When $n \geqslant 5$:

$$\alpha = \frac{90 - n}{85} \qquad \text{for } m > 4, \text{ and}$$

$$\alpha = \frac{38 - n}{33} \qquad \text{for } m \leqslant 4. \qquad (3)$$

When $n < 5$: $\alpha = 1.0$.

3.2 DOWEL JOINTS AND BOLTED JOINTS

In the absence of any reliable basis for change it is suggested that the existing CIB Code design rule be retained:

$$\alpha = 1.0 \quad \text{for } n \leqslant 4, \text{ and}$$
$$\alpha = \frac{4}{n} + \frac{2}{3n}(n - 4) \quad \text{for } n \geqslant 4. \tag{4}$$

3.3 SPLIT - RING JOINTS AND SHEAR - PLATE JOINTS

It is proposed that the design rule from BS 5268: Part 2, be adopted:

$$\alpha = 1.0 - \frac{3(n - 1)}{100} \quad \text{for } n < 10, \text{ and} \tag{5}$$
$$\alpha = 0.70 \quad \text{for } n \geqslant 10.$$

4. PROPOSED REVISIONS TO CLAUSES OF CIB CODE

The Appendix gives the proposed revisions to clauses of the CIB Code for structural timberwork.

5. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

It is concluded that there is a need to include new design rules for ultimate limit state calculations on axially loaded joints with nails, split-rings or shear-plates to account for the effect of the number of rows of fasteners or connector units.

It is important to recognise that the rules proposed have been arrived at on the basis of very limited test evidence. There is a need to collect more data before definite conclusions can be reached and before all primary influences upon fastener group action can be established. For joints with multiple fasteners such as nails or bolts, it is possible that existing theories, (3, 8, 9, and 11), provide reliable estimates for ultimate strengths of joints with ductile failures. New theories appear to be necessary for ultimate capacities of joints with brittle failures.

Topics not addressed here but which are worthy of discussion include:

1. The need to account for the number of fasteners or connector units in ultimate limit states design calculations for joints with arbitrary combinations of thrust, moment and shear forces.

2. The need to account for the number of fasteners or connector units in serviceability limit states design calculations for joints.

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APPENDIX - Proposed revisions to clauses of CIB - Structural timber design code, (13)

Clause 6.1.1.1, third paragraph:

For more than five nails in line the effective number of nails n_{ef} is:

$$n_{ef} = n\left(\frac{38 - n}{33}\right) \quad \text{for } m \leqslant 4 \qquad (6.1.1.1b)$$

$$n_{ef} = n\left(\frac{90 - n}{85}\right) \quad \text{for } m > 4 \qquad (6.1.1.1c)$$

where n is the number of nails in line in the direction of loading,

m is the number of nails in line transverse to the direction of loading.

Clause 6.1.4, insert after second paragraph:

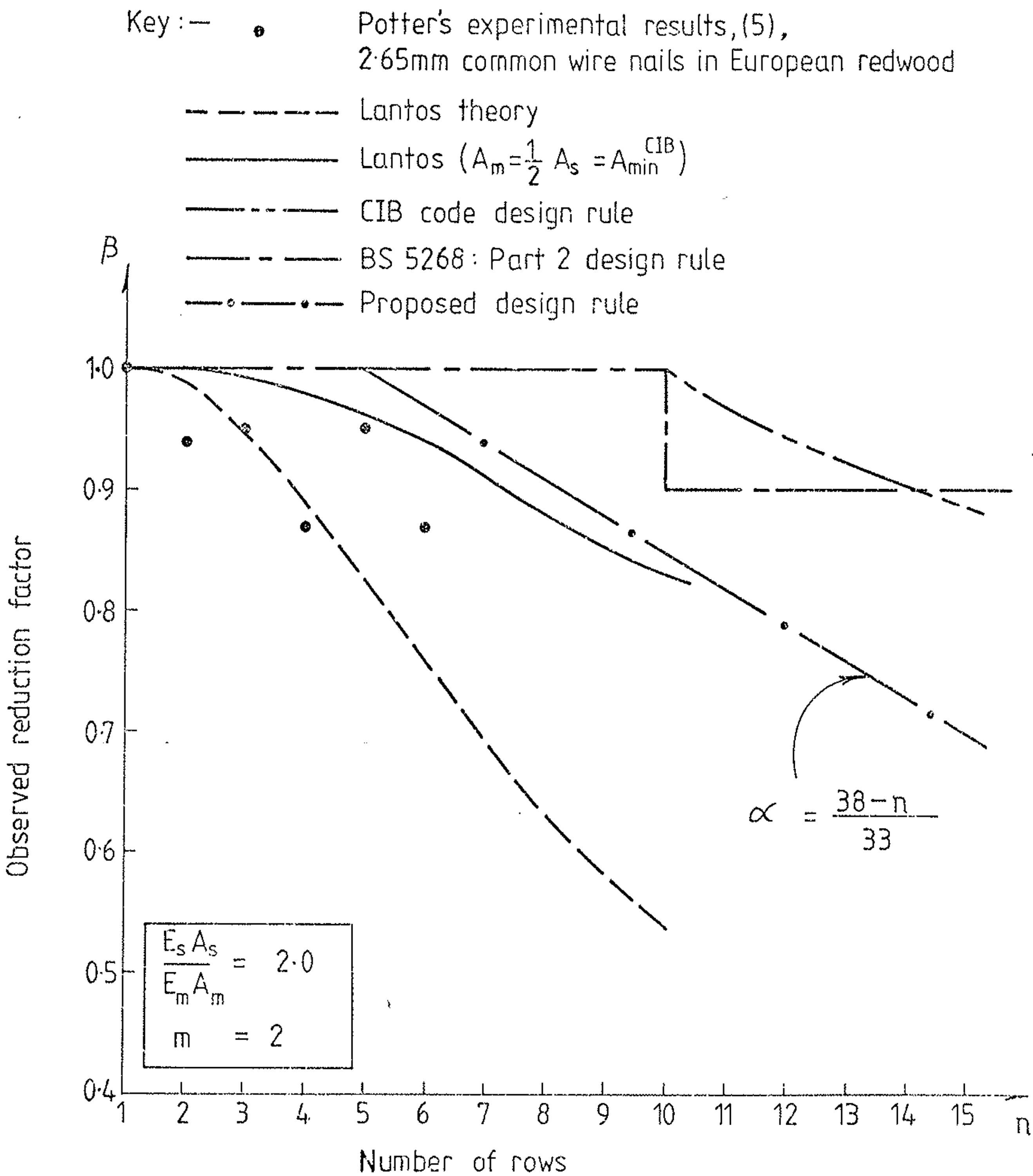
Unless there is contradictory test evidence, it should be assumed that for laterally loaded joints with more than one connector in line the effective number of connectors n_{ef} is:

$$n_{ef} = n\left(1.0 - \frac{3(n-1)}{100}\right) \quad \text{for } n < 10 \qquad (6.1.4a)$$

$$n_{ef} = 0.70\ n \qquad \text{for } n \geqslant 10 \qquad (6.1.4b)$$

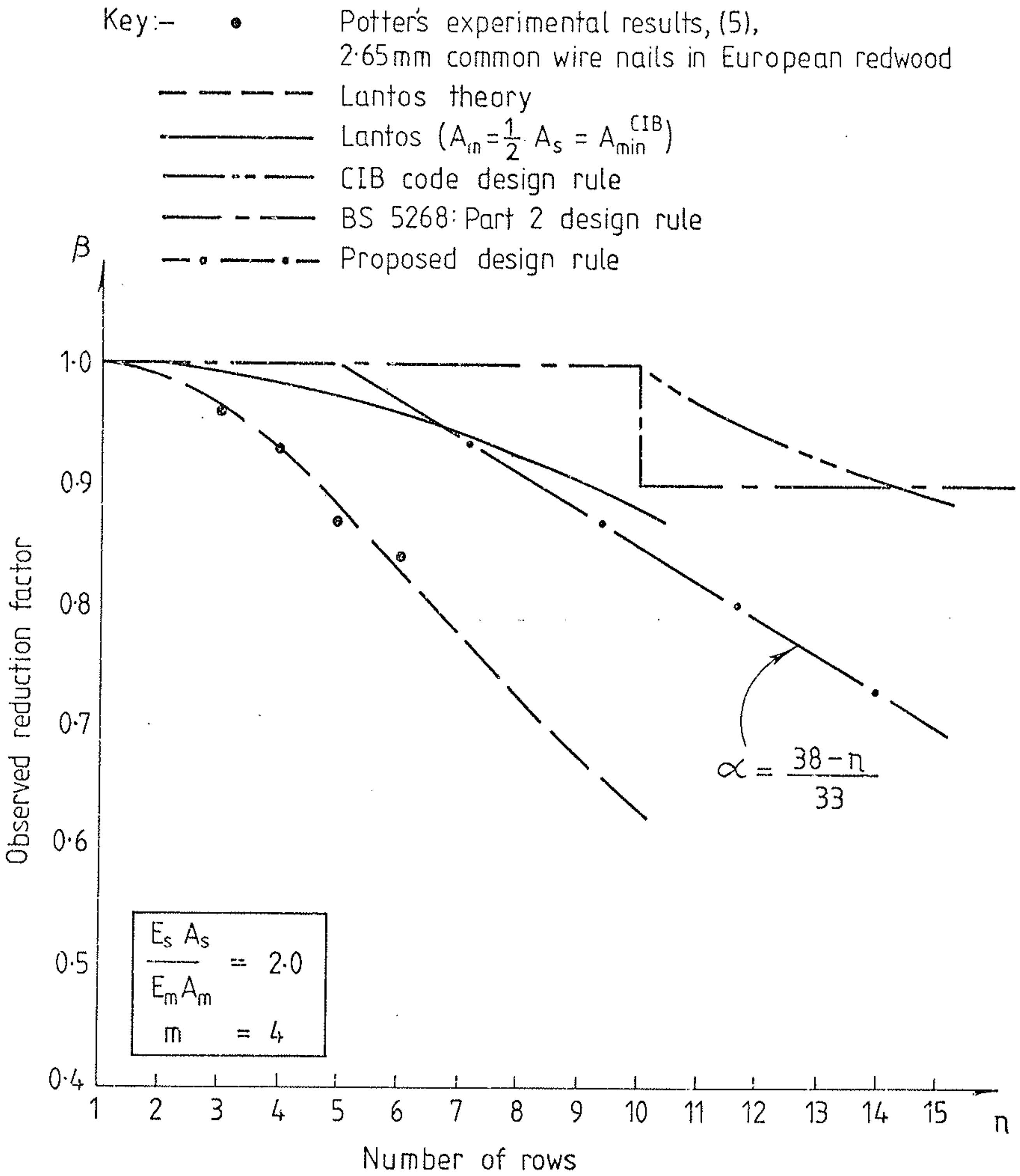
where n is the number of connectors in line in the direction of loading.

Fig. 1
(Nails)



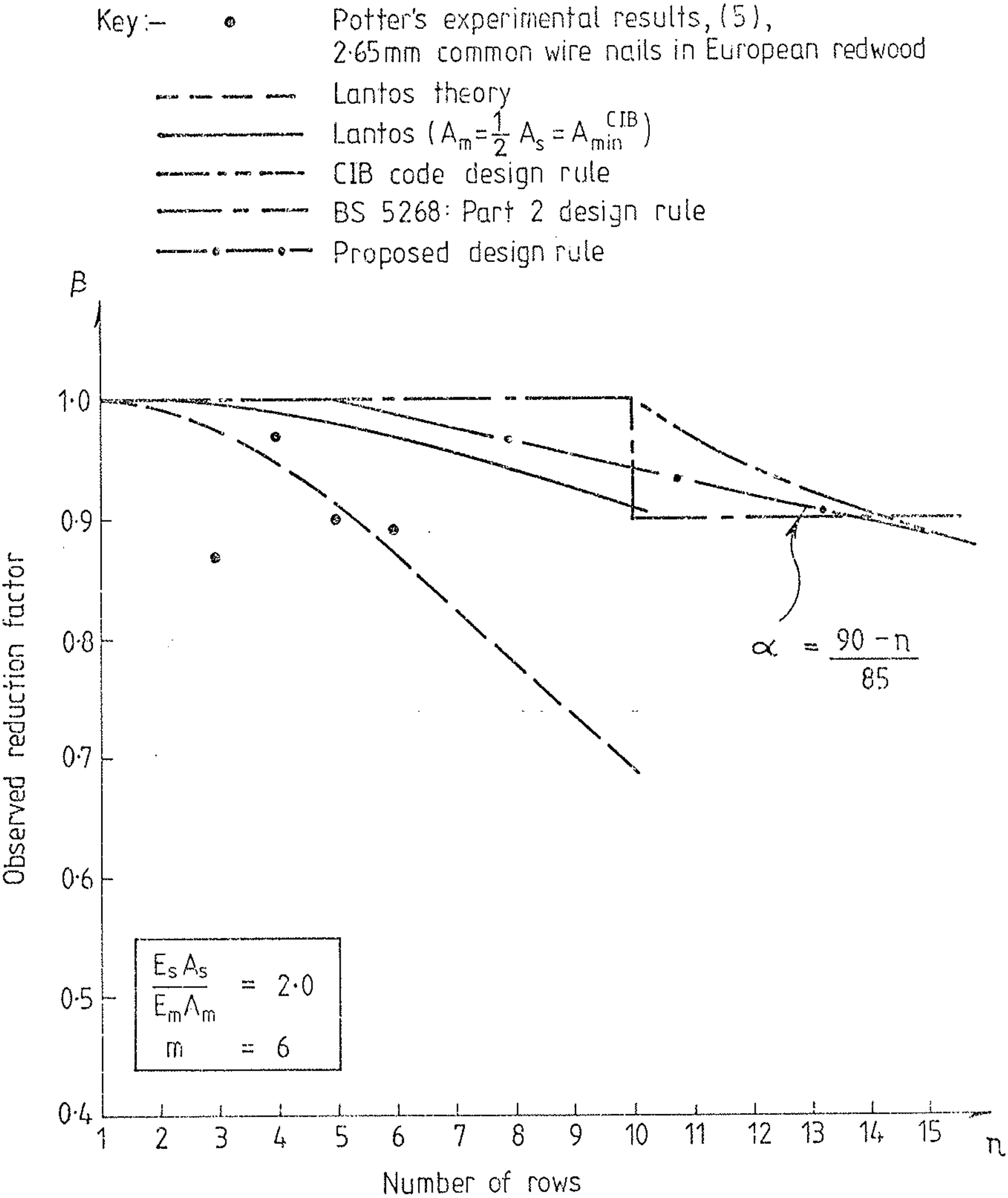
Observed reduction factors, β values, for nailed joints with loading parallel to grain

Fig. 2
(Nails)



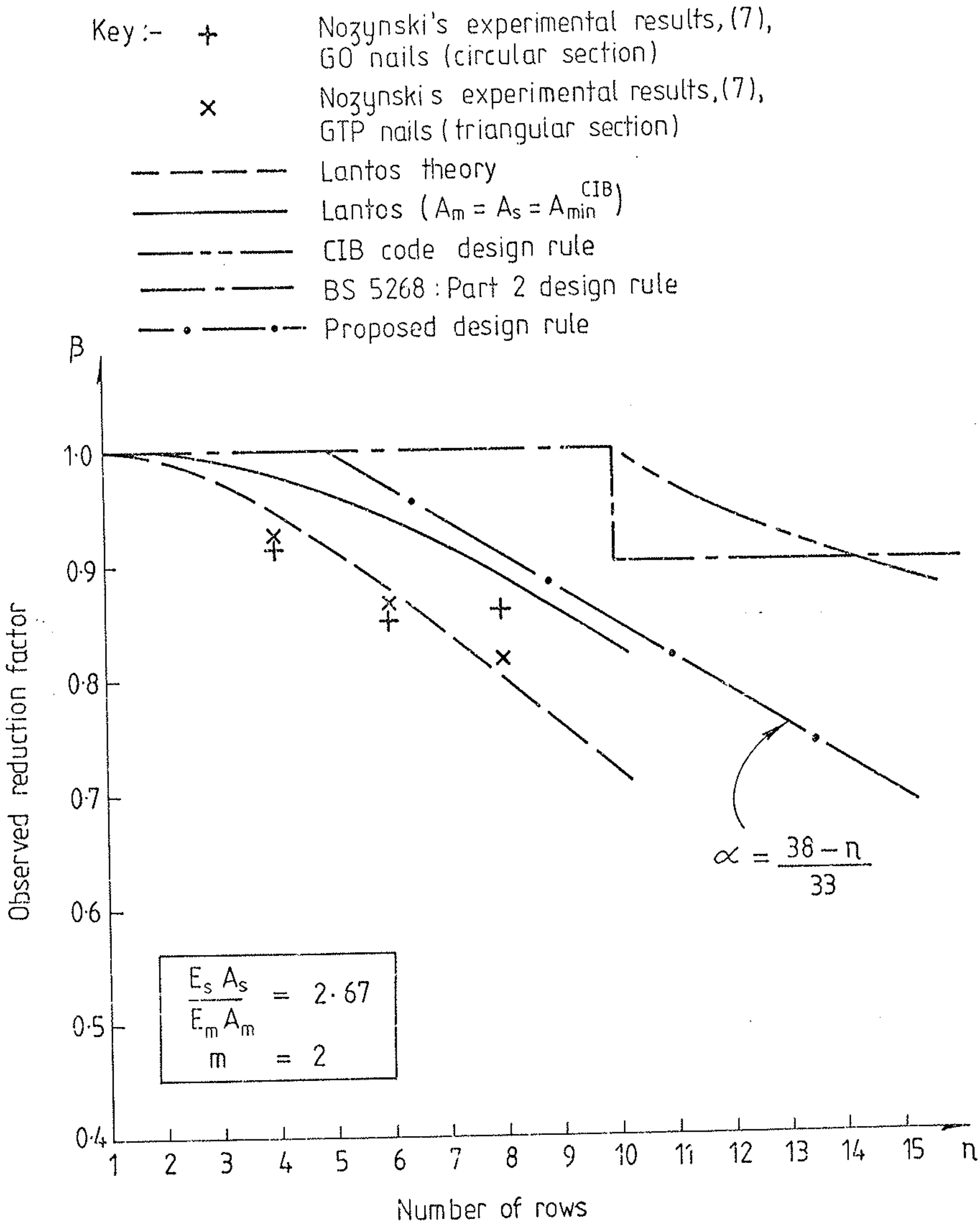
Observed reduction factors, β values, for nailed joints with loading parallel to grain

Fig. 3
(Nails)



Observed reduction factors, β values, for nailed joints with loading parallel to grain

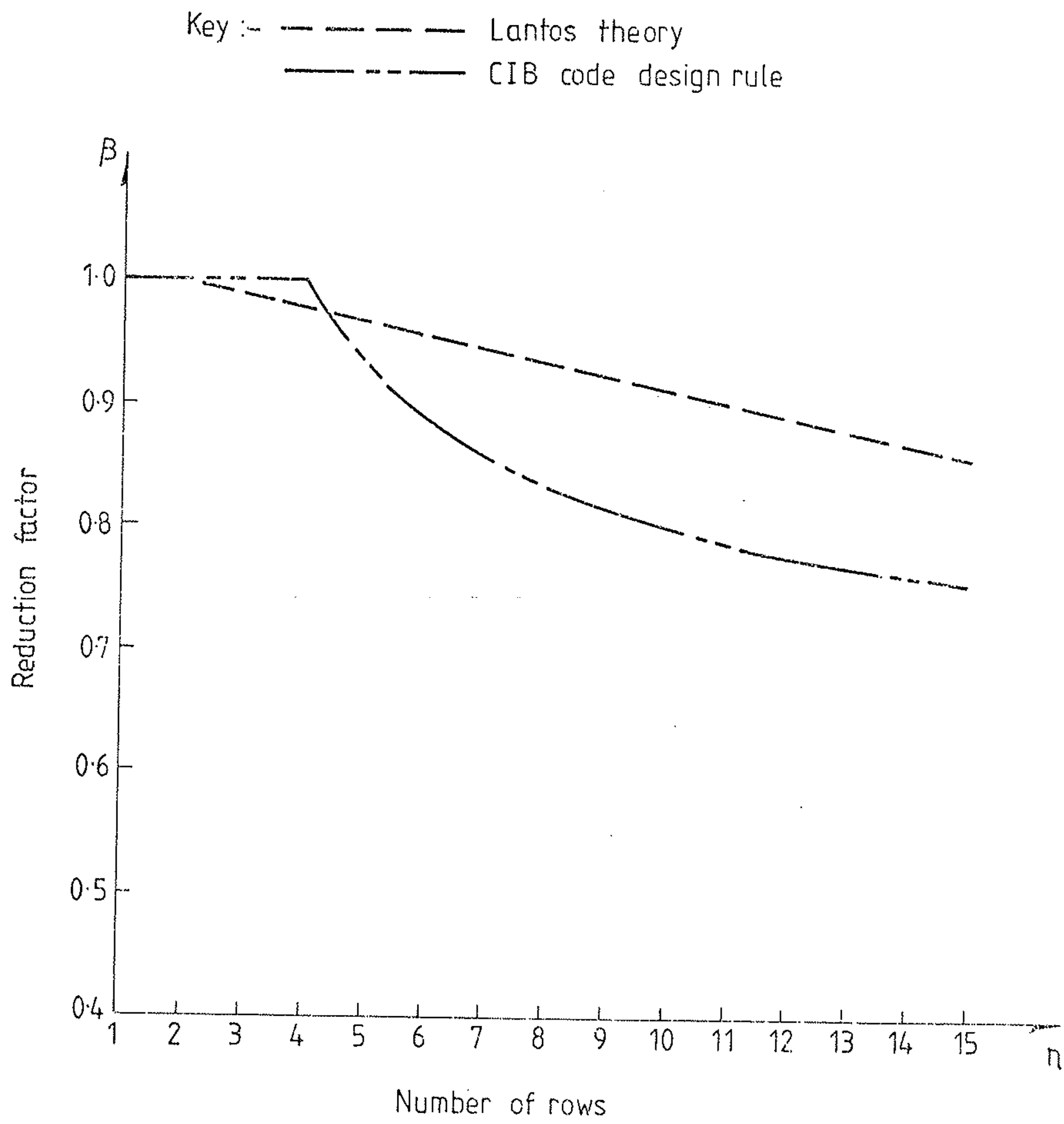
Fig. 4
(Nails)



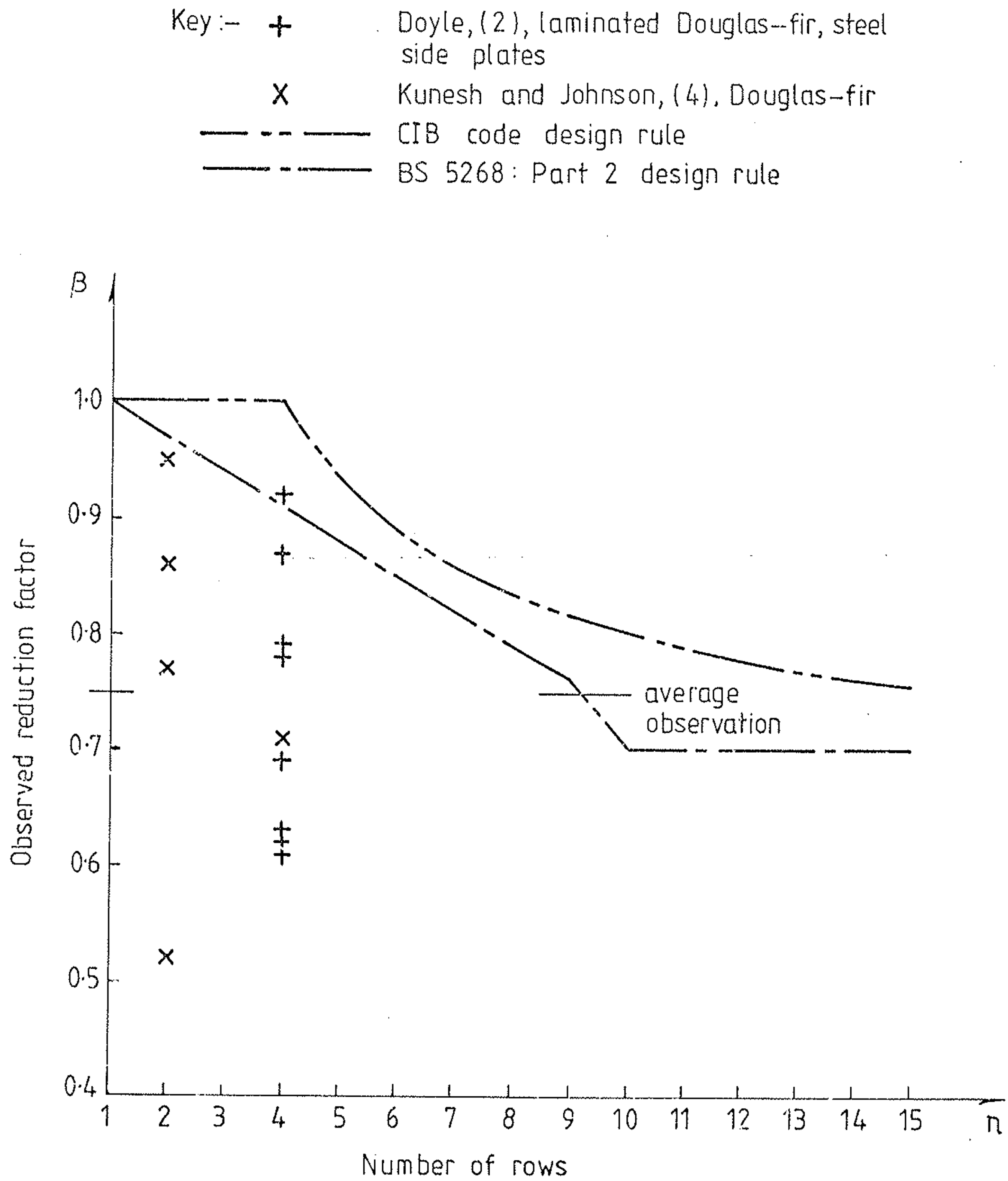
Observed reduction factors, β values, for nailed joints with loading parallel to grain

Fig. 5
(Dowels)

Note :- This figure corresponds to Fig. 3 in Steck's 1984 CIB paper, (16)

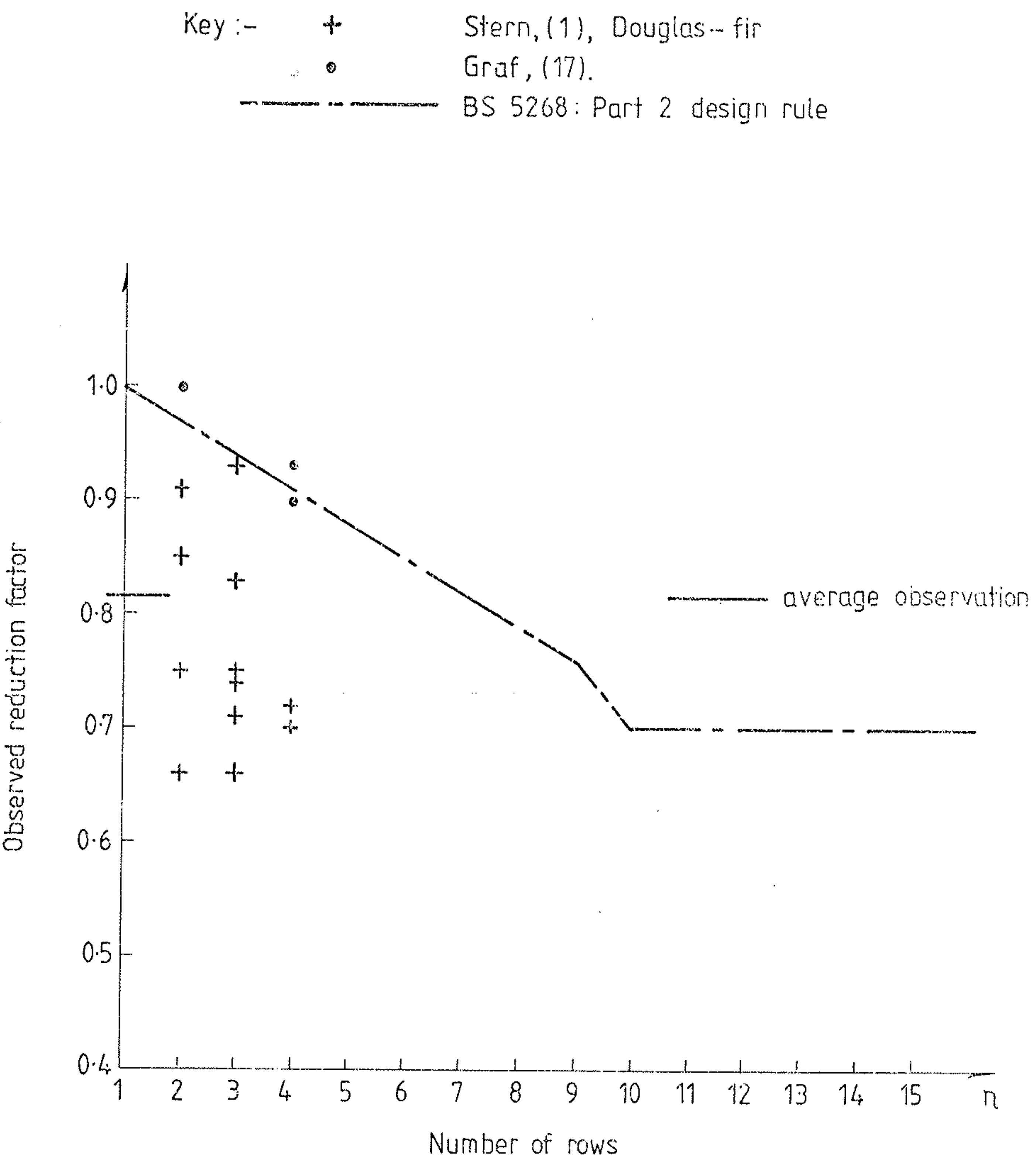


Reduction factors for dowel joints with loading parallel to grain



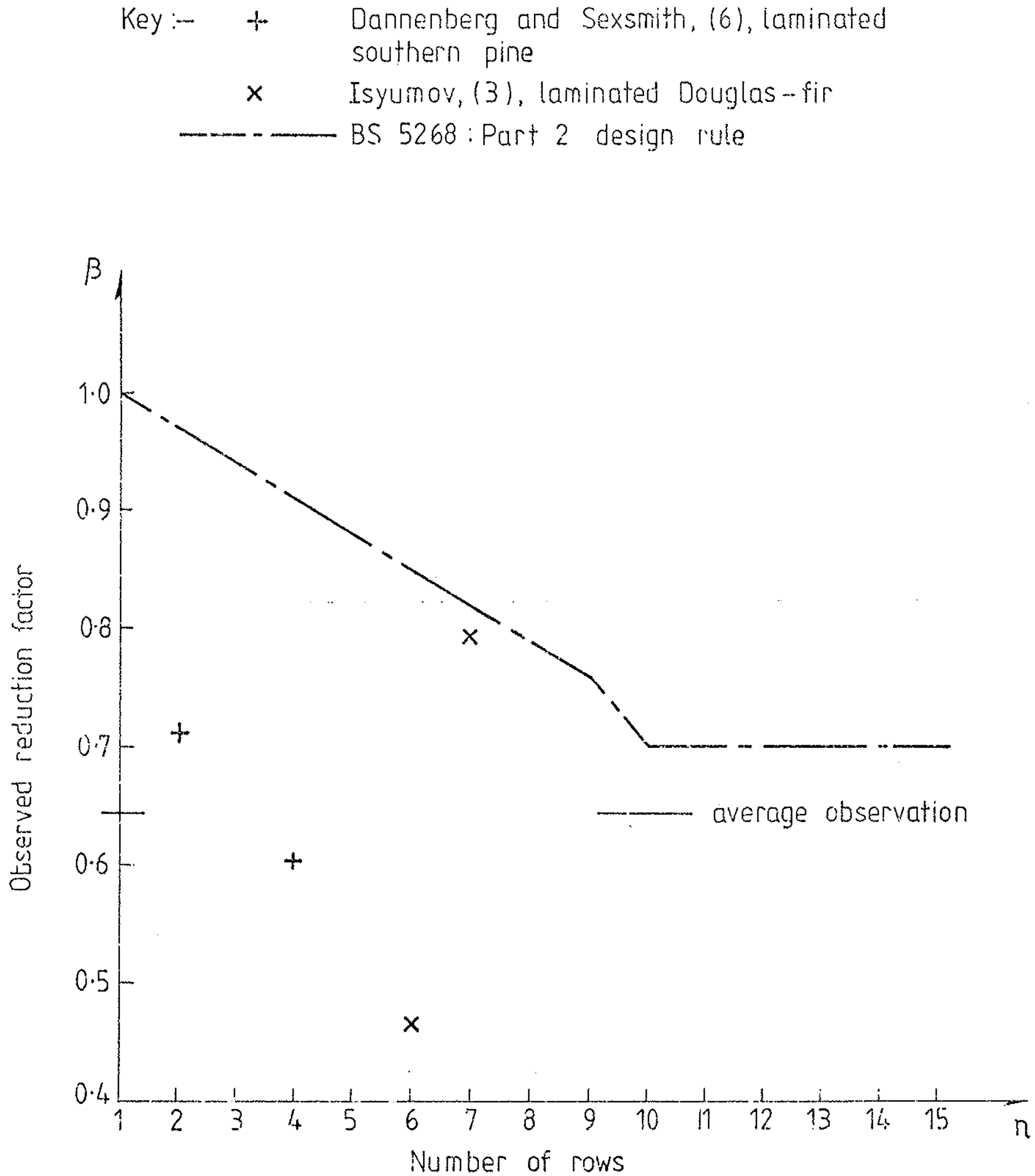
Observed reduction factors, β values, for bolted joints with loading parallel to grain

Fig. 7
(Split-rings)



Observed reduction factors, β values, for split-ring connectored joints with loading parallel to grain

Fig. 8
(Shear-plates)



Observed reduction factors, β values, for shear-plate connectored joints with steel side plates and loading parallel to grain

CIB-W18/18-7-4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

A DETAILED TESTING METHOD FOR NAILPLATE JOINTS

by

J Kangas
Technical Research Centre of Finland
Finland

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

INTRODUCTION

Different nailplates have several similar characteristics. The differences in design values may be due to test arrangements and particularly to the shapes and dimensions of the nailplates used in the tests.

In Finland, several different types of nailplates are used, where the number of test series in the original tests has often been very small. To revise the outdated nailplate approvals several new tests were needed during 1984. Several new nailplates were sent for tests at the same time. On the basis of earlier tests the Technical Research Centre of Finland drafted a detailed method that is in agreement with the final recommendation TT-1A from 1982 of the Joint Committee RILEM/CIB-3TT, Testing methods for joints with mechanical fasteners in load-bearing timber structures ANNEX A Punched metal plate fasteners. It presents, for instance, the choice of the nailplate dimensions and the sampling of timber used in the different tests and the dimensions and assembling of the specimens.

In winter 1980 - 1981 VTT made an extensive nailplate investigation for a private customer following the testing rule M.O.A.T. No. 16:1979 June 1979 of UEA_{tc} (European union of agrement): Rule for the Assessment of Punched Metal Plate Timber Fasteners. Some experts have suggested that it should be published. The difficulties associated with the choice of nailplate sizes, for instance, have also often emerged in the discussions. It has therefore been considered justifiable to present this meeting the detailed nailplate testing method now used in Finland and the method for determining the design values of nailplates.

1 THE AIM OF THE METHOD

The strength and deformation characteristics of joints made with nailplates intended for load-bearing timber structures are studied in order to determine the strength and deformation parameters of the design values.

2 SCOPE OF APPLICATION

The method is applied separately to each nailplate type.

3 REFERENCES

The loading procedure, measurements and calculations follow principally the standard ISO 6891 from 1983, Timber structures - Joints made with mechanical fasteners - General principles for the determination of strength and deformation characteristics.

The principles for specimen selection follow mainly the final recommendation TT-1A. The deviations are adjustments to the general text of the code concerning specimens and nailplate sizes or allowances for the number of specimens. The tensile strength of the plate material is determined according to DIN 50114 (Flachprobe 20 x 80).

4 DEFINITIONS AND SYMBOLS

The definitions are according to point A1 in TT-1A. The symbols are according to point 4 in ISO 6891. Other symbols in the test method:

- ρ_{ou} is density of wood when weight is determined from ovendried wood and volume from wood in moisture content u, kg/m^3
- τ anchorage strength (lateral resistance of the plate projections), N/mm^2
- p_c compression strength of nailplate, N/mm
- p_t tensile strength of nailplate, N/mm
- s shear strength of plate or capacity of nailplate joint in shear loading, N/mm.

5 SAMPLING OF NAILPLATES

The test plates are taken from the manufacturer's production. Before and after the test plates are made, specimens, in which there are punched nails and appr. 30 cm unpunched band, are taken from every band for determination of the material values. These are delivered with the nailplates to VTT. At least one plate size is prepared in the presence of VTT's representative. In case of imported plates the official supervisor from the country in question takes care of this control.

6 TEST METHOD

6.1 Principle

The test method clarifies in controlled circumstances and using specimens suitable for each case the anchorage, tensile, compression and shear strengths of timber joints assembled using different types of nailplates. The number of specimens is the smallest possible for a covering and reliable study of the characteristics.

6.2 Apparatuses

The loading and measurement equipment is according to standard ISO 6891 clause 7. They can be either manual or automatic devices connected to a minicomputer, which during loading output the load-slip curve and the desired strength and stiffness values.

6.3 Preparation and preliminary treatment of specimens

6.3.1 Timber

Attempts are made to acquire the timber in rather large parcels and to preselect it thus that it is suitable for the tests.

The timber is acquired air-dry and planed to 45 x 147 mm. The boards are given numbers 1, 2, 3... and cut to 1 m long pieces, which are marked with the corresponding board numbers and also with a, b, c... The pieces are weighed and inspected. The annual ring width shall in the approved specimens be 1.5 - 3.5 mm. They may not have knots in the joint area, compression wood nor uneven growth. The sawn and inspected timber are selected as evenly as possible to test series, whereat only one piece of the same board comes to one series.

The objective density (ρ_{ou}) of timber for the different test types is

anchorage strength tests (τ) 360 ± 15 kg/m^3
shear strength tests (s) 375 - 400 kg/m^3
compressive strength tests (p_c) about 400 - 410 kg/m^3
tensile strength tests (p_t) 410 - 450 kg/m^3

The timber members are conditioned and stored at a temperature of +20^0 ±2 ^{0}C and in a constant humidity of 0.80 + 0.05 RH.

6.3.2 Test series and plate sizes

Anchorage strength ($\tau_{\alpha\beta}$) between plate and wood

τ_α, when $\alpha = 0^0$, 30^0, 60^0, 90^0 and $\beta = 0$.

The size of fastener is selected thus that the anchorage length l_e in the direction of applied force is the largest with which anchorage failure occurs. With angles of 0^0 and 90^0 the width is about 70 - 120 mm. The limiting factor is the strength of the timber members (F_{max} < 70 kN). With angles of 30^0 and 60^0 the width of timber is B = 100 mm (80 mm). The shape of the plate is selected thus that the excentricity of the load is small, for instance corresponding to plate sizes 130 x 160 mm and 160 x 130 mm.

$\tau_{\alpha\beta}$ (T-joint) with angles

a) $\alpha = 0^0$, $\beta = 45^0$ and 90^0
b) $\alpha = \beta = 0^0$

The width of nailplate is about 50 mm in the case a) and the length of nailplate is about 80 mm in the case b).

In the main test series of τ_0 there are 10 specimens with an angle of $\alpha = 0^0$ and in the other test series 5 specimens.

Tensile strength (p_α) of nailplate with angles of $\alpha = 0^0$, (15^0), 30^0, 60^0, 90^0.

The width of plate is about 70 - 100 mm (F_{max} < 70 kN).
The length of the plate in load direction shall be long enough for plate failure. The same plate size can be used with angles $\alpha = 0^0$, 15^0 and 30^0 and correspondingly with angles $\alpha = 60^0$ and 90^0.
With each angle there are 3 specimens.

Compression strength of plate (buckling) ($p_{c\alpha}$) with angles $\alpha = 0^0$, 90^0.

With angle $\alpha = 0^0$ the plate size is the same as in the main series τ_0.
With angle $\alpha = 90^0$ the plate size is the same as in series p_{90}.
With each angle there are 3 specimens.

Shear strenght s_α, when $\alpha = n \times 15^0$, n = 0.1 - 11.

Plate size about 120 x 160 mm is used with all angles. In addition, when $\alpha = 0^0$ a narrower plate size is used (about 100 x 160 mm) and when $\alpha = 30^0$, 45^0, 60^0 and 150^0 plate size about 75 x 200 mm. Plate size about 160 x 120 mm is also used, if necessary, when $\alpha = 45^0$ and 150^0.

There are 5 specimens in the main test series with $\alpha = 0^0$ and 3 in the other test series.

6.3.3 Specimens

The specimens are shown in figure 1 - 4. They are put together by means of nailed strips of wood. The nailplates are fastened with small nails and pressed into timber using servohydraulic equipment. The compression force is determined so that a compression pressure of 6 MPa can be obtained on the nailplate area. The piston velocity is about 30 mm/s. The plates can also be embedded using the truss manufacturer's roller. The strips of wood facilitating assembly, visible in the figure, are removed immediately after embedding. During pressing the moisture of the timber members corresponds to equilibrium moisture content when air humidity is about 0.80. After assembly the specimens are conditioned at least 2 weeks at a temperature of $+20^{0} \pm 2\ ^{0}C$ and in a humidity of 0.65 ±0.05 RH before loading.

6.4 Material tests

6.4.1 Timber

The density (ρ_{ou}) and moisture (u) of wood is determined from the ø 50 mm drilled sample taken from a representative point of the loaded specimen.

6.4.2 Plate material

The following determinations are made from unpunched specimens:

The thickness of plate zinc coated and without zinc coating (t and t_{Fe}).

The ultimate elongation A_{50}, yield stress R_{el} and tensile strength R_m of the nailplate material are determined in accordance with the standard DIN 50114 (Flachprobe 20 x 80) by the tension bars made from the specimens.

The ductility of the fasteners at the nail root position is determined by following the testing rule M.O.A.T. No. 16:1979 June 1979 of UEA_{tc}. At least three nails per plate from different parts of the plate and one plate per each used band are tested.

6.5 Loading procedure and results

The method of loading of the specimens and the presentation of the results follow in the main the ISO 6891 standard. In accordance with the recommendation 3TT-1A the greatest load $F_{7.5}$ until the slip limit v = 7.5 mm is recorded.

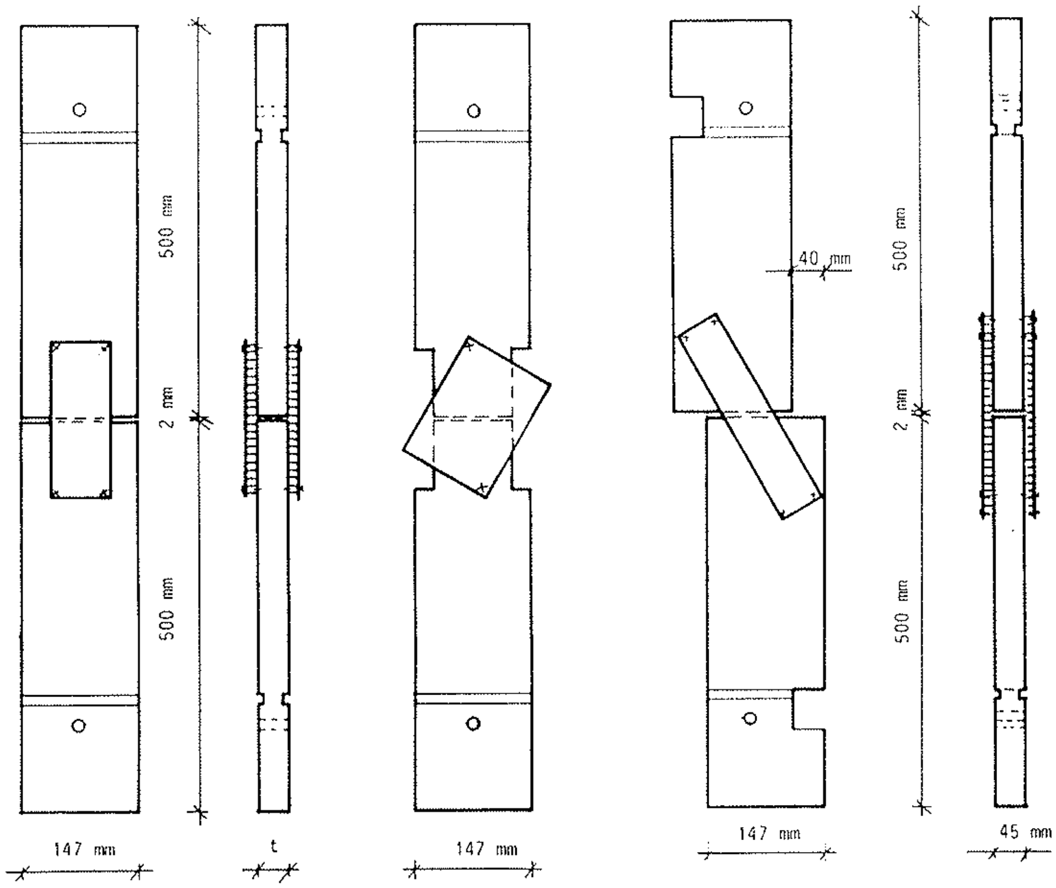


Figure 1. Tension tests specimens
a) τ_0, τ_{90}, p_o, (p_{15}), p_{90}
b) τ_{30} and τ_{60}
c) p_{30} and p_{60}.

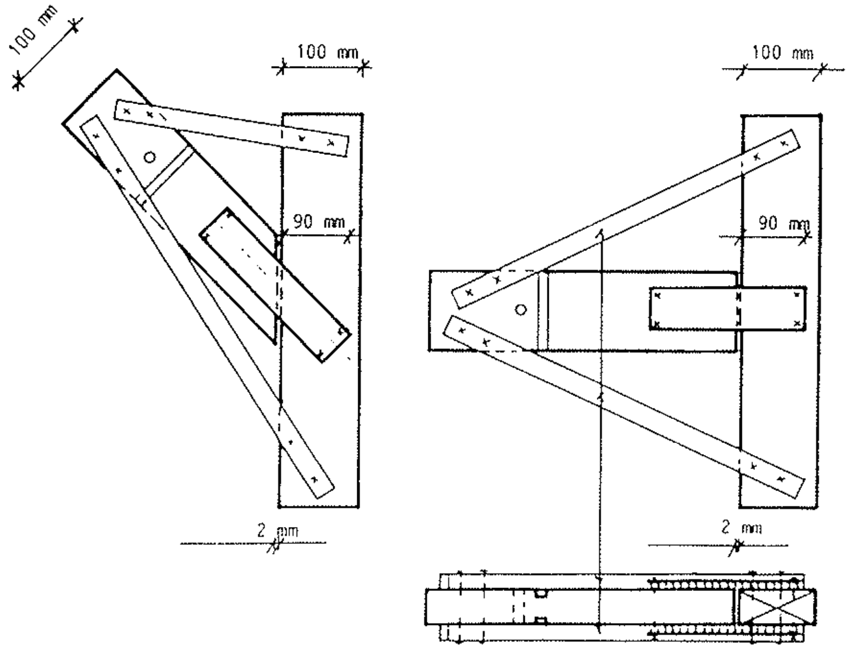


Figure 2. T-joint sepecimens with strips of wood for facilitating assembling a) $\tau_{0.90}$ and $\tau_{90.90}$ b) $\tau_{0.45}$

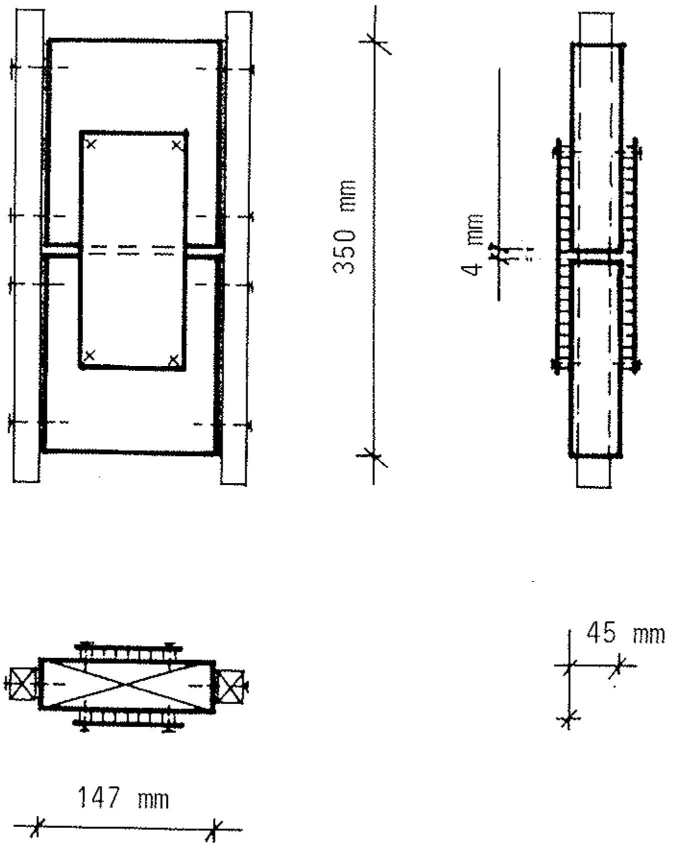


Figure 3. Compression test specimen with strips of wood for facilitating assembling.

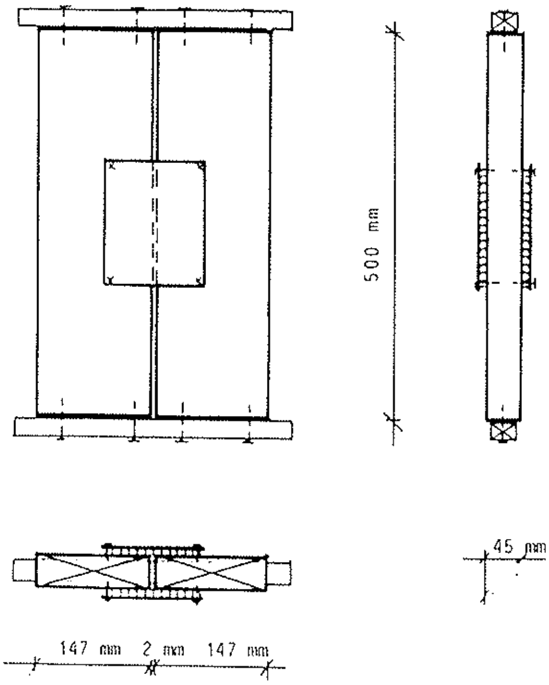


Figure 4. Shear test specimen with strips of wood for facilitating assembling.

CIB-W18/18-7-5

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PRINCIPLES FOR DESIGN VALUES OF NAILPLATES IN FINLAND

by

J Kangas
Technical Research Centre of Finland
Finland

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

18.7-5

1 AIM AND CONTENT

These instructions contain the methods and principles which the Technical Research Centre of Finland (VTT) follows when drafting on the basis of test results its recommendations for the allowed values of design quantities in timber joints assembled with each nailplate type.

2 EXPERIMENTAL BASIS

The method is based on tests made according to the instructions drafted by VTT's Laboratory of Structural Engineering or tests of the same quality and extent made by some other acceptable research institute.

3 CALCULATORY AMENDMENTS

3.1 Strength of plate

When needed, the strength of the plate is adjusted to correspond to the design thickness and design strength of the plate in the specimens which have failed due to metal failure or in which the material values otherwise have affected the result. This is done by multiplying the tensile strength p_t (mean of the series), compression strength p_c and shear strength s of the plate obtained as test result with the coefficient

$$k(t,R) = \frac{t_d \cdot R_d}{t_{Fe} \cdot R_m} \text{, where}$$

t_d is design thickness of plate (mm)
t_{Fe} thickness of plate without zinc-coating (mm)
R_d design strength of plate material (N/mm^2)
R_m tensile strength measured from plate sample (N/mm^2).

3.2 Lateral resistance of the plate projections

The compression strength of wood (density) affects the anchorage strength τ between the nailplate and timber. Therefore efforts are made to get the compression strength of the wood in the specimens close to the objective value, which in Scandinavia has been selected to 35 MN/m^2. The deviation of the compression strength during the test from the objective value can approximately be considered in the following way. We calculate the compression strength (MN/m^2) of wood during the test from the equation, given in CIB - RILEM Timber Standard No. 07

$$f_c = 0.095\ \rho_{ou}\ (2 - \frac{u}{0.15}), \text{ where}$$

ρ_{ou} is the density of wood when weight was determined from oven-dried wood and the volume of wood in the moisture content u kg/m^3

u the moisture content of the test sepcimen.

The coefficient for anchorage strength τ is then

$$k_\tau = (\frac{35}{f_c})^{1/2}.$$

4 SAFETY FACTORS

The safety factors used are in the different cases (the different cases are checked and the determining - which gives the smallest value - is selected).

4.1 Anchorage strength

I 3.0 calculated from the mean of the test results (corresponding variation coefficient 6.7 % with a series of 5 specimens in case II).

II 2.5 when calculated from characteristic strength, if variation coefficient is larger than above.

The safety factors are applied to the test results, which are checked according to point 3.2.

4.2 Plate tensile and compression strength

$k(v) \times 1.2 \times 1.55/k(t,R)$ from the mean of the series

$k(v)$ is deviation factor $1 - 2.5 \frac{v}{100}$, where v is the variation coefficient (%) of the series' test results

$k(t,R)$ coefficient calculated according to point 3.1.

4.3 Plate shear strength

I $\quad$ 3.0 a) failure of lateral resistance
b) when slip limit 7.5 mm is exceeded

II $\quad 1.2 \times 1.2 \times 1.55/k(t,R) = 2.23/k(t,R)$
a) in metal failure when $v_{max} \leq 7.5$ mm
b) is checked at slip limit 7.5 mm, if case Ib is fulfilled

III $\quad 1.2 \times 1.55/k(t,R) = 1.86/k(t,R)$ the strength value is checked to an extrapolated value according to the load-slip curve's initial curvature, if case I or IIa is fulfilled.

IV $\quad 1.2/k(t,R)$ to the point in the load-slip curve where slip exceeds the value 1.5 mm (figure 1).

5 ALLOWED VALUES

The single values are also graphically smoothed out on the safe side. The allowed values are presented in table and/or equations with different values of angles α and β, and also in sets of graphical curves (figure 2). The allowed anchorage stresses are given separately for shear force.

6 TIME OF VALIDITY

The approval is normally valid for five years.

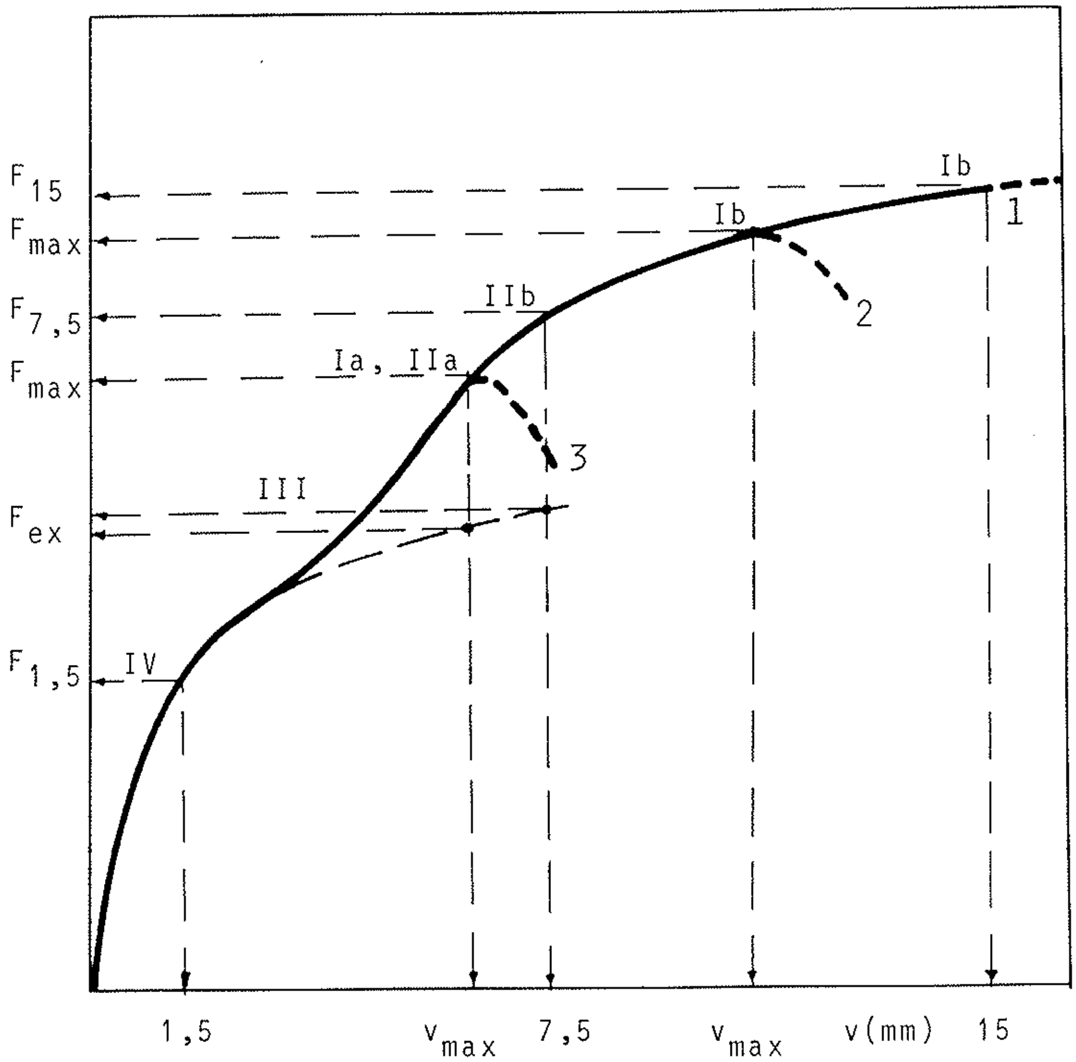


Figure 1. The diagram of different limit states (point 4.3) for nail-plate joints under shear load. Three schematic load-slip curves (1.2 and 3) have been drawn one upon another.

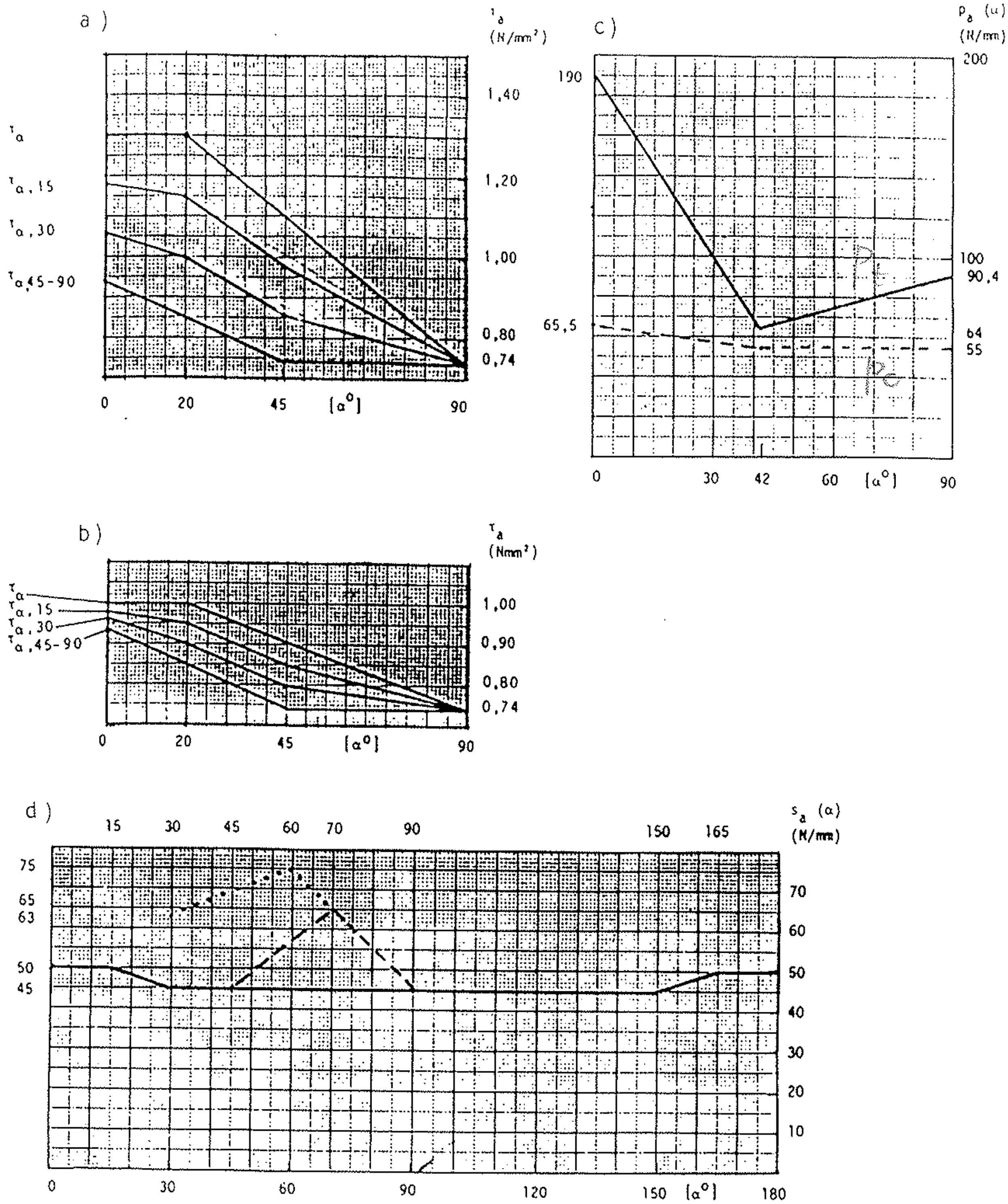


Figure 2. Examples of the allowed strength values of nailplate joints
a) anchorage, $\tau_{\alpha\beta}$-values
b) $\tau_{\alpha\beta}$-values for joints under shear load
c) tension, p_t-values (upper curve)
compression, p_c-values (lower curve)
d) shear, s-values, dotted line is for long nailplates.

CIB-W18/18-7-6

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE STRENGTH OF NAILPLATES

by

N I Bovim and E Aasheim
Norwegian Institute of Wood Technology
Norway

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

The Strength of Nail Plates

Introduction

The strength of nail plates with respect to failure in the plate material have been examined by NTI as background for a Nordic proposal.[1)] This proposal has been presented by B.Norén in CIB/W18/paper 14-7-1.

The present paper gives a summary of test results and proposed design method for design against plate failure.

Today, the design against plate failure is based on:

1) strength of plate with respect to forces parallel to the joint between connected timber members. This is called the shear strength of the plate.

2) strength with respect to forces perpendicular to the joint. This is called the plate strength of the plate.

The existing rules give no interaction formula for combination of "shear" and "plate"-forces. Consider Fig. 1. The example demonstrates clearly that existing rules does not take care of the orthotrop behavior of nail plates. An interaction formula based on "plate" and "shear"-components would be still worse and in reality meaningless!

In many cases the rules are extremely unsafe (up to 200 percent for design values given in the norwegian approval of the tested plate). This is the main reason for the work presented here.

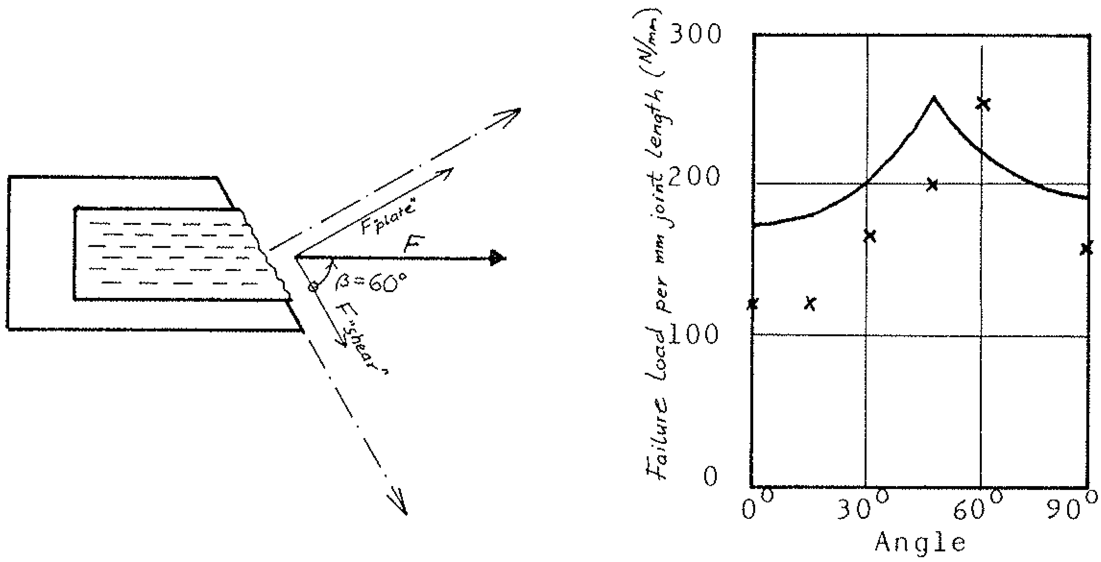


Figure 1. Example of nail plate joint. Existing design rules are drawn as a curve. Test results and common sense tell us that maximum strength is achieved for $\beta=60^{\circ}$!

Definitions and symbols

Joint direction x is parallel to the gap between the connected members. y-direction is perpendicular to x and positive out from the studied member.

Plate main directions are a and b. In general a is the direction having the highest tensile strength of the plate. b-direction is perpendicular to a. For both axis positive direction is pointing out from the studied member and part of the plate.

α is the angle of x-direction to the a-direction. $0 \leq \alpha \leq 90^{\circ}$. Limitation of α is a consequence of the definitions above.

α_{aF} is angle of a-direction to direction of the force F. $0 \leq \alpha_{aF} \leq 360^{\circ}$

a is length of nail plate in a-direction

b is width of nail plate in b-direction

f is length of the joint covered by nail plate, measured along the gap between connected members.

a_n is effective length of nail plate covering gap between connected members. $a_n = f \cos \alpha$

b_n is effective width of nail plate covering gap between connected members. $b_n = f \sin \alpha$

$$f \text{ is minimum of } \begin{cases} a/\cos\alpha & \text{for } \alpha \neq 90^{\circ} \\ b/\sin\alpha & \text{for } \alpha \neq 0^{\circ} \end{cases}$$

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1) Members of the Nordic group have been M.Johansen, Denmark, B.Norén, Sweden, T.Poutanen, Finland, N.I.Bovim, Norway and E.Aasheim, Norway for the last period of time.

Proposed design method

Design conditions for the strength of the plate are based on design values in the principal directions a and b. In total 6 different design values have to be determined:

p_{at} - plate tensile strength in a-direction
p_{ac} - plate compression (buckling) strength in a-direction
s_a - plate shear strength in a-direction

p_{bt} - plate tensile strength in b-direction
p_{bc} - plate compression (buckling) strength in b-direction
s_b - plate shear strength in b-direction

Recommended test specimens for determination of design values are shown in Fig. 2. All tests should be carried out without friction. The plate size has to be chosen so that the intended failure mode is reached.

Consider Fig. 3. The force F has to be transmitted by the plate. F is acting under an angle α_{aF} to the a-axis. F could be replaced by the components F_a and F_b in a- and b-direction.

The strength of the plate should be verified by the design condition:

$$\left(\frac{F_a}{N_{aD}}\right)^2 + \left(\frac{F_b}{N_{bD}}\right)^2 = 1 \qquad (1)$$

N_{aD} and N_{bD} denote design values in a- and b-direction.

N_{aD} is taken as the greatest (favourable) value of

$$N_{aD} = \text{max. of} \begin{cases} p_{at} \cdot b_n \\ s_a \cdot a_n \end{cases} \quad \text{when } F_a \text{ is positive} \quad (\text{i.e. } \cos\alpha_{aF} \geq 0) \qquad (2)$$

else

$$N_{aD} = \text{max. of} \begin{cases} p_{ac} \cdot b_n \\ s_a \cdot a_n \end{cases} \quad \text{when } F_a \text{ is negative} \quad (\text{i.e. } \cos\alpha_{aF} < 0) \qquad (3)$$

N_{bD} is taken as the greatest (favourable) value of

$$N_{bD} = \text{max. of} \begin{cases} p_{bt} \cdot a_n \\ s_b \cdot b_n \end{cases} \quad \text{when } F_b \text{ is positive} \quad (\text{i.e. } \sin\alpha_{aF} \geq 0) \qquad (4)$$

else

$$N_{bD} = \text{max. of} \begin{cases} p_{bc} \cdot a_n \\ s_b \cdot b_n \end{cases} \quad \text{when } F_b \text{ is negative} \quad (\text{i.e. } \sin\alpha_{aF} < 0) \qquad (5)$$

It is seen from Fig. 3 that

$$F_a = F \cdot \cos \alpha_{aF} \quad \text{and} \quad F_b = F \cdot \sin \alpha_{aF} \tag{6}$$

From (1) and (6) the following design formulae is derived

$$F = \frac{1}{\sqrt{\left(\frac{\cos\alpha_{aF}}{N_{aD}}\right)^2 + \left(\frac{\sin\alpha_{aF}}{N_{bD}}\right)^2}} \tag{7}$$

The design formulas (1) or (7) make it possible to check the strength of nail plates for all possible combinations of α and α_{aF} on the basis of the 6 design values for a spesific nail plate.

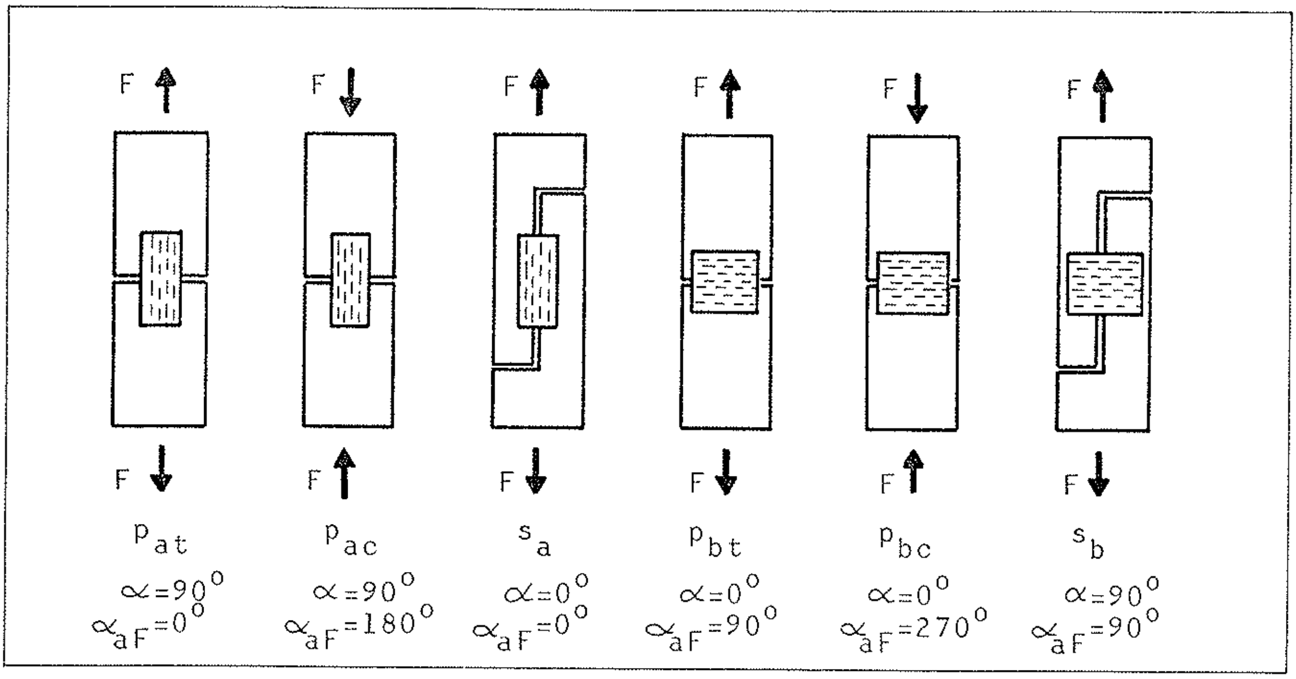


Figure 2. Test specimens used for determination of design values

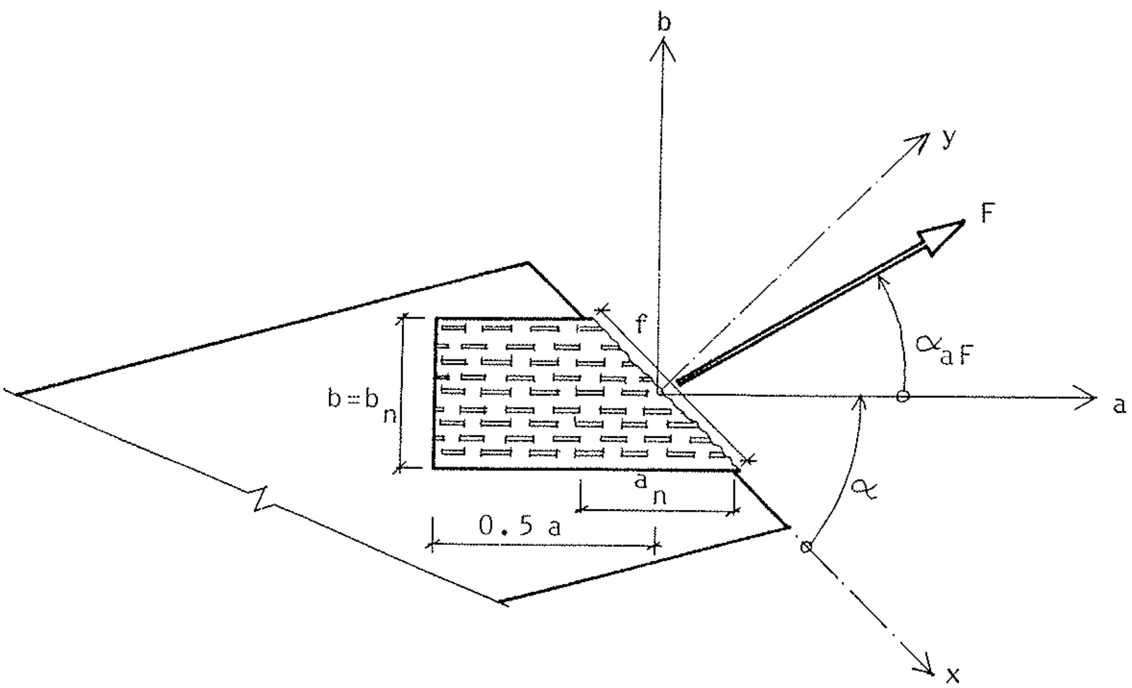


Figure 3. Illustration of geometry and angles in a nail plated joint

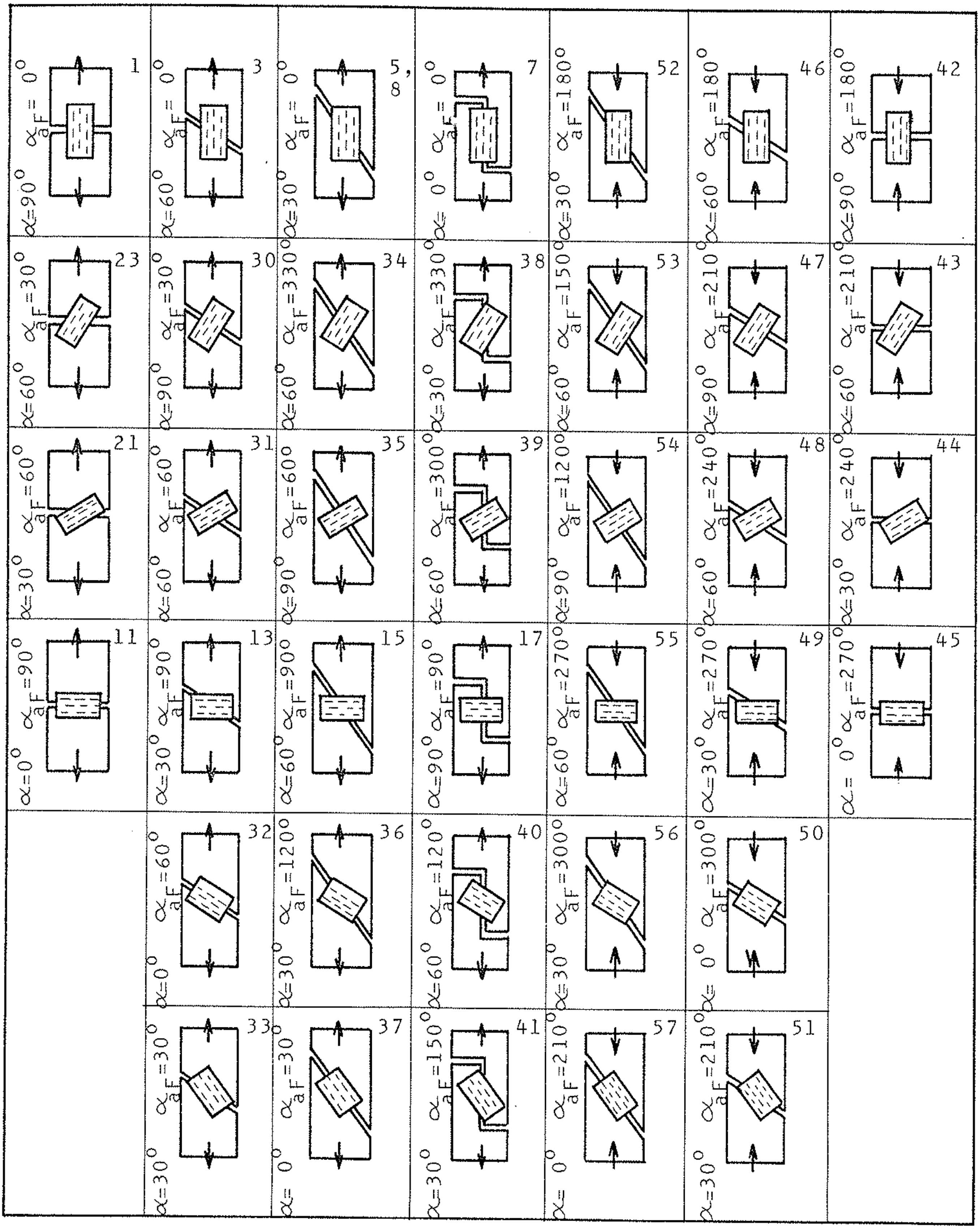


Figure 4. Survey of test specimens. Specimen number in upper right corner is referred to in diagrams with test results. Load directions are marked with arrows.

Test results

Tests were carried out for all combinations of angles α and α_{aF} with maximum intervals of 30°, as shown in Fig. 4. Because testing of shear and plate strength of nail plates ordinary result in small values for the Coefficient of variation (3 to 5 percent), only two test specimen of each type were tested. Test results shown are mean values. Difference in failure load for each pair of specimen gives an indication of the deviation of test results. In percent of mean values for each pair, the average difference for all series was 5.4 percent.

In total 96 specimens were tested. For all these tests the wellknown Gang Nail 18 ga. were used, in 2 different sizes

Size I - 72 x 196 mm
Size II - 107 x 117 mm

Strength in a-direction

Results from testing of strength in a-direction ($\alpha_{aF}=0^{0}$) for various angles α (joint/plate-angle) are shown in Fig. 5a). Failure loads F are divided to the length of the joint covered by plate, f. When $\alpha_{aF}=0^{0}$ the plate is under tension or shear.

Tension failures are marked with S. Failure mode is a steel-tensile failure of the bars in a-direction, see Fig. 6. It is seen from Fig. 5 and 6. that this failure mode does occur for $30^{0} \leq \alpha = 90^{0}$. Steel-tensile failure will only occur if the anchorage area is strong enough to withstand the force until steel-failure.

Test results marked with A is from tests with plate size II where anchorage failure took place. Design against anchorage failure will avoid such low values. The excentricity moment in the anchorage area is for these tests of great importance. Stresses resulting from excentricity are 50% higher than stresses from the transverse load F alone. The resulting anchorage stress calculated by elastic theory is 100 percent higher than the anchorage capacity determined from tests without excentricity. Thus, excentricity should not be neglected in design against anchorage failure.

Shear failures are seen as deformation of the plate in the ineffective area between the connected members, see Fig. 7. Shear failure is

Legend: The number given refer to test type number, see Fig. 4.
A - anchorage failure, B - buckling, D - deformation of zone near joint,
S - steel tensile failure.
I - plate size 72 x 196 mm, II - plate size 107 x 117 mm.

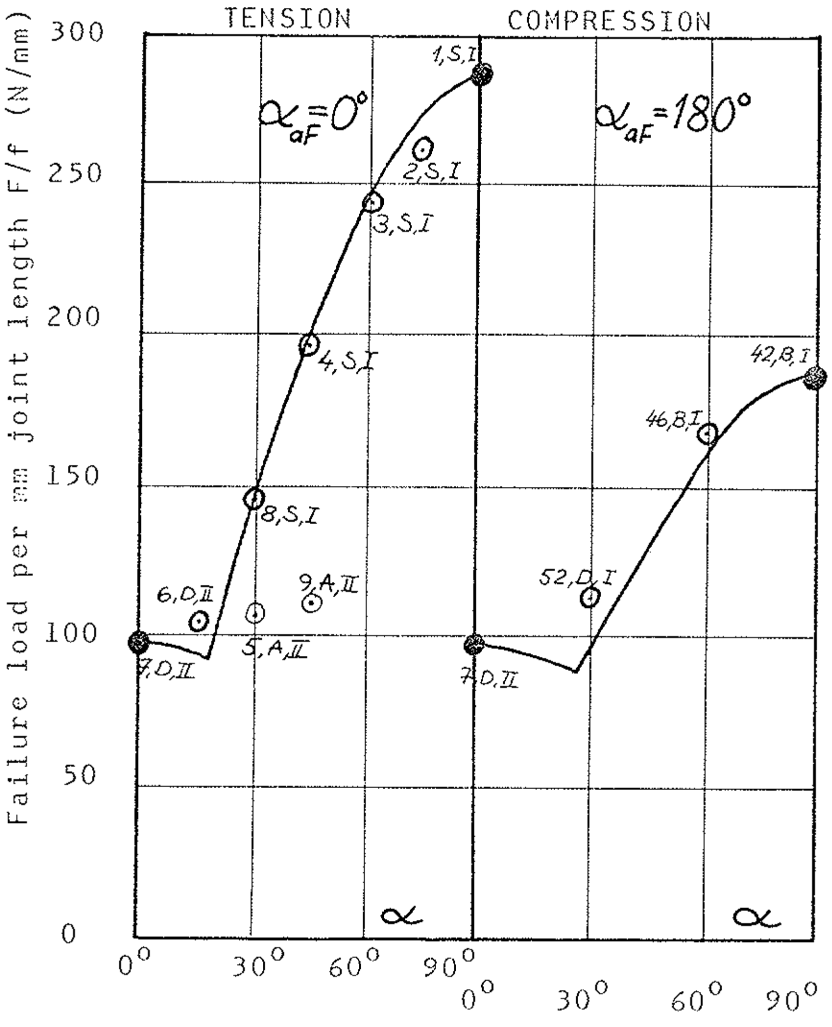


Figure 5a) Results from test of plate-strength in a-direction.

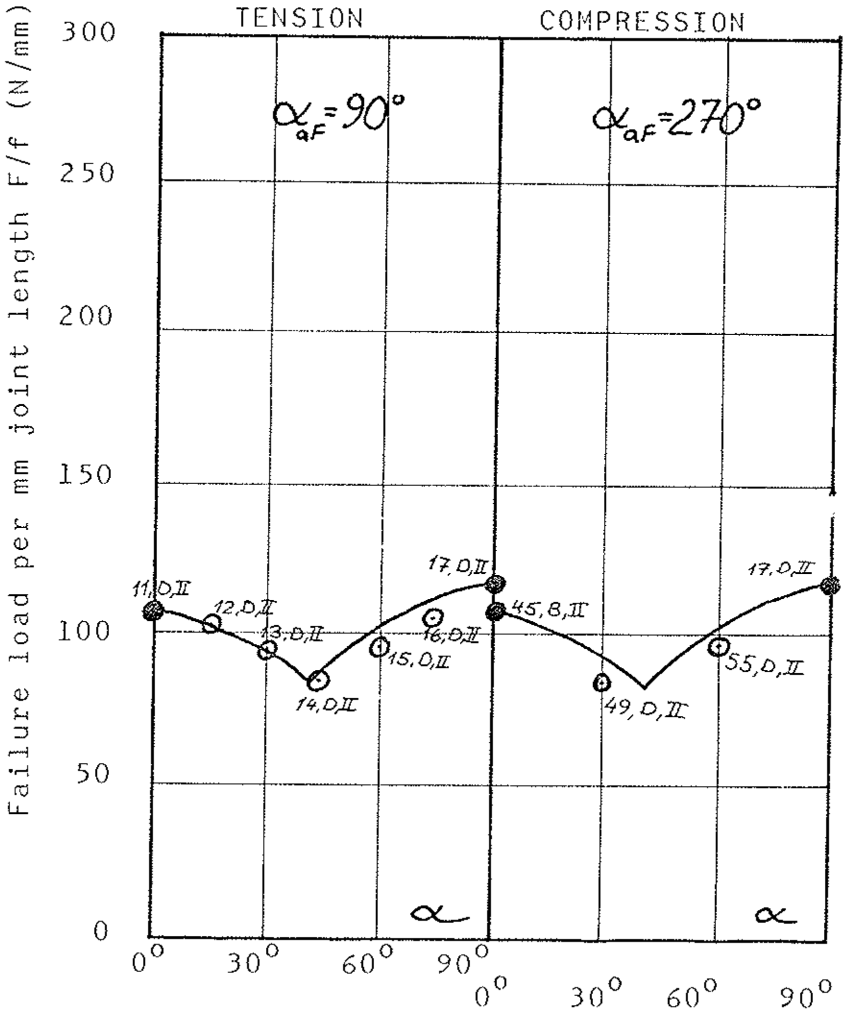


Figure 5b) Results from test of plate-strength in b-direction.

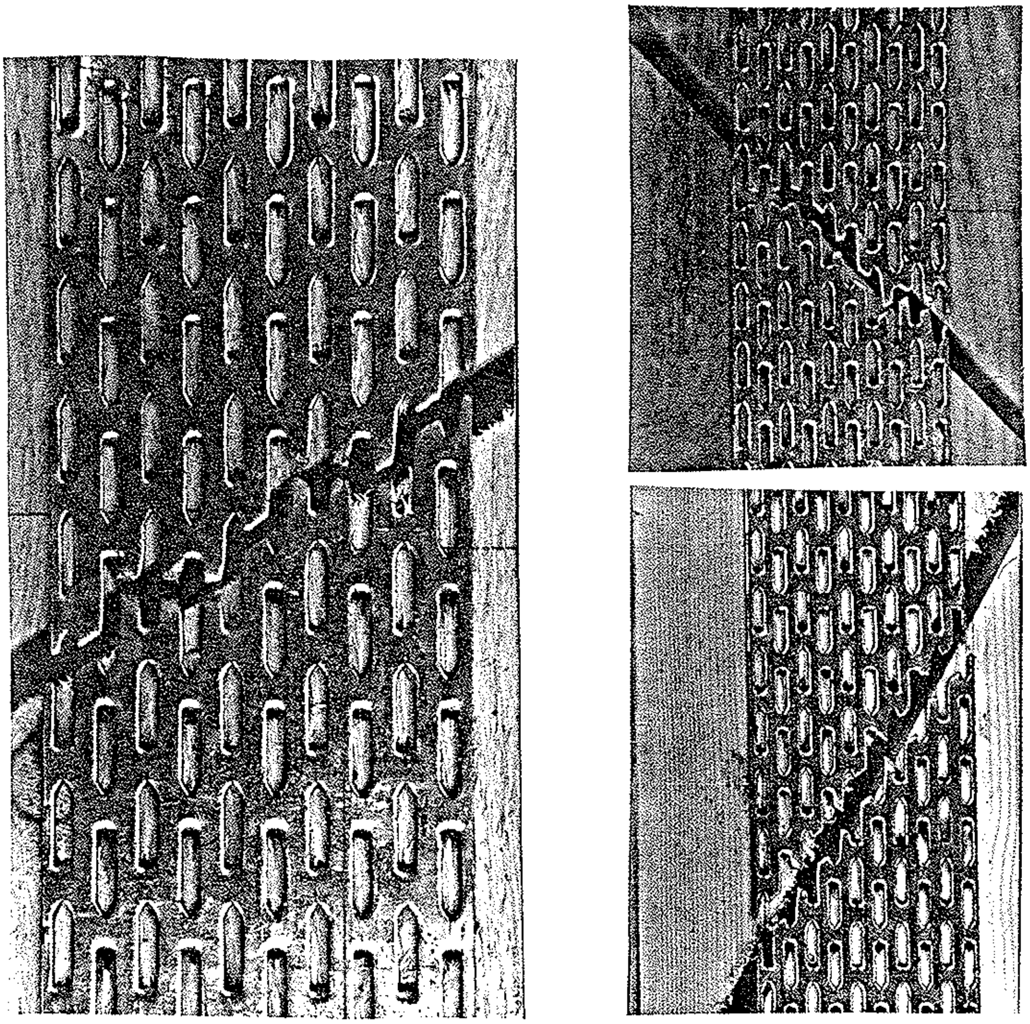


Figure 6. Steel-tensile failure in a-direction
The failure mode and failure load is almost the same for angles $30^{\circ} \leq \alpha \leq 90^{\circ}$. Load in a-direction.

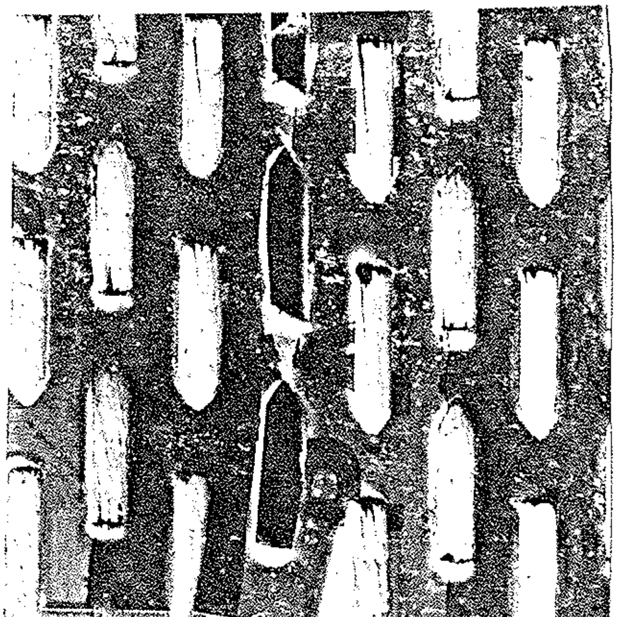


Figure 7. Shear failure in a-direction. The plate is deformated in the zone over the joint between connected members.

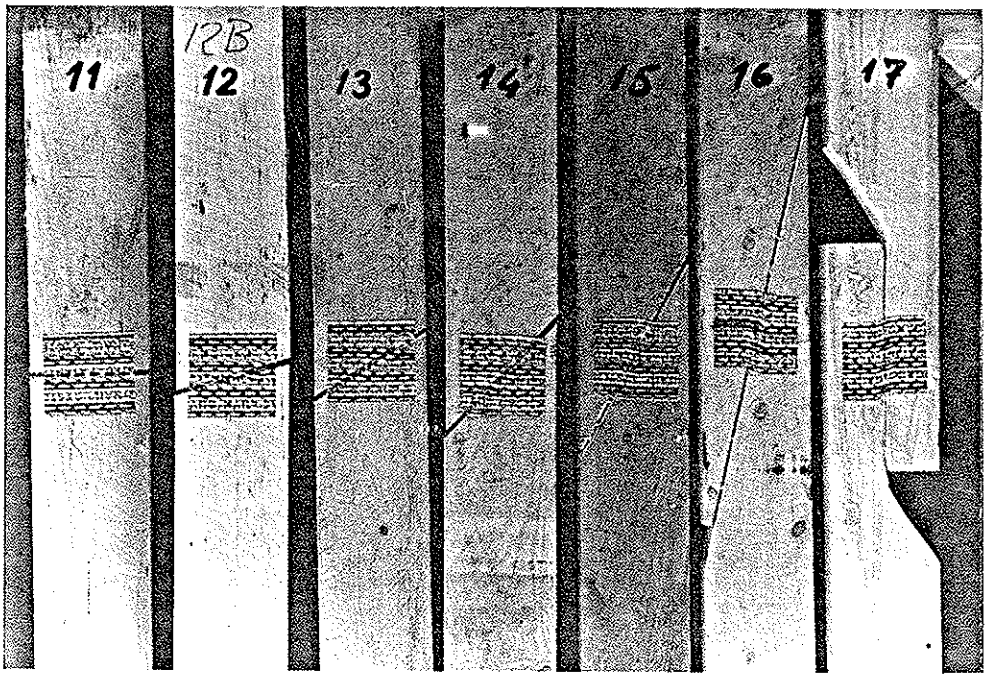


Figure 8. Failure modes from testing of plate strength in b-direction. For all tests failure loads are almost equal and failure involves the same mechanism of yielding hinges.

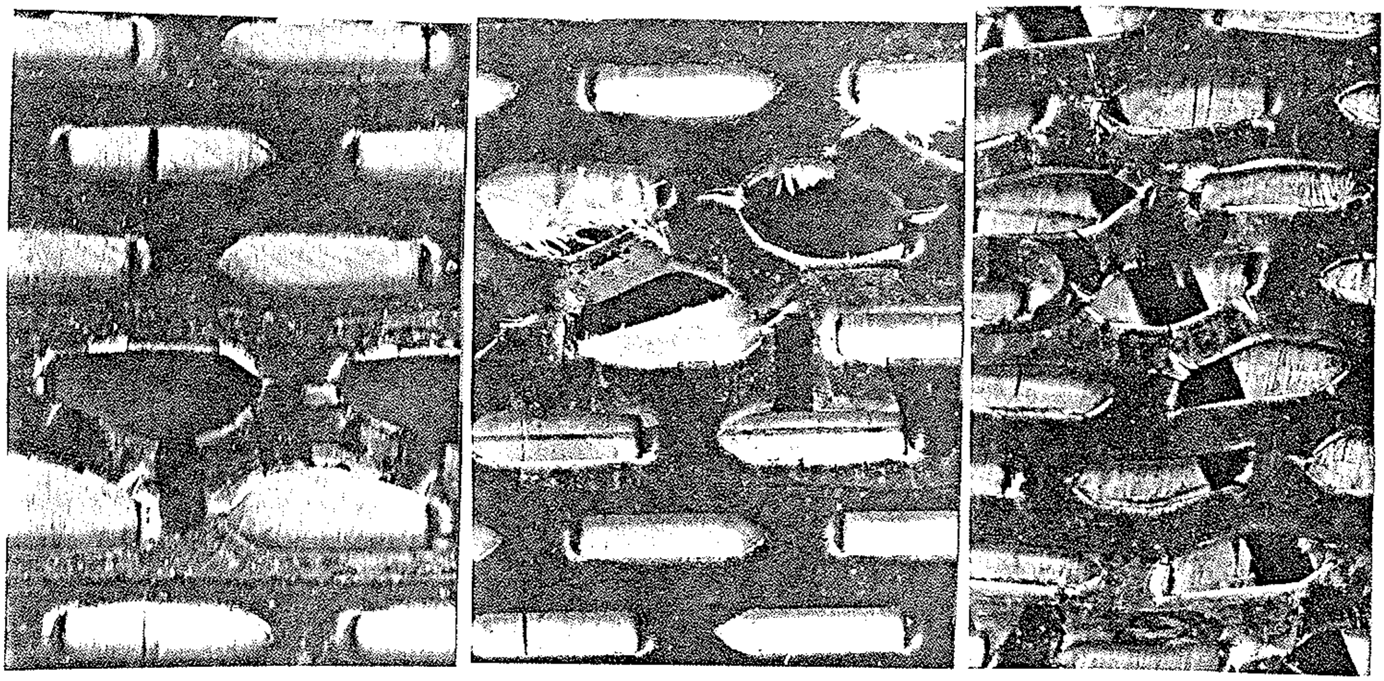


Figure 9. Details of mechanism of failure for various joint/plate-angle α.

marked with D in the diagrams.

In Fig.5 also the calculated strength based on the design method have been drawn. The black dots are design values from tests, used for calculation of the curves.

Also results from compression tests have been included. In compression tests the gap between timber members were 4 mm. Failure load was recorded before closure of gap.

Strength in b-direction

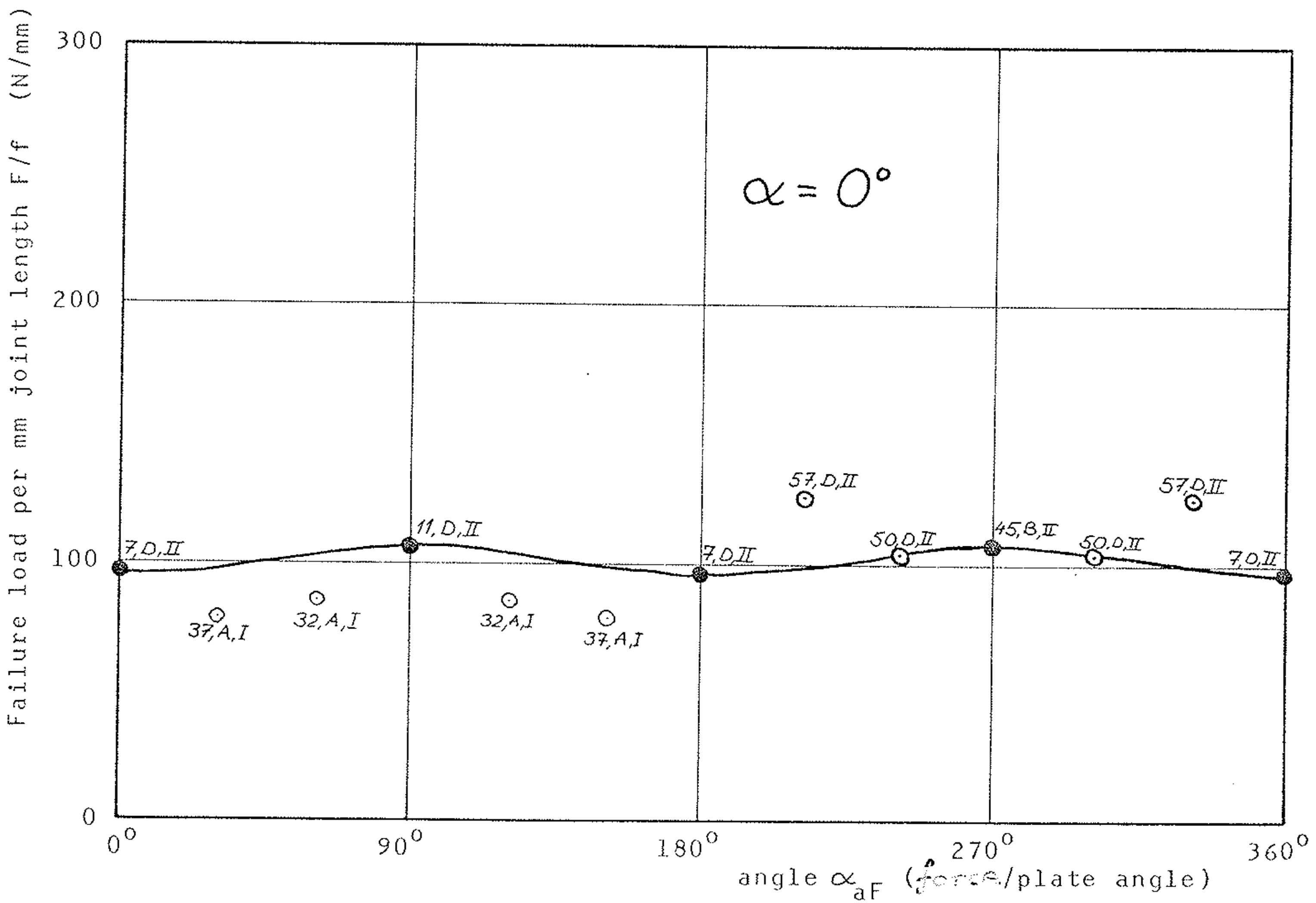
Results from testing of plate strength in b-direction are given in Fig. 5b). Failure modes are shown in Fig.8 and 9. It could be seen that the mechanism of failure and the failure load is almost equal for all values of angle α. Joint length f is longer when angle α is lying between 0^{o} and 90^{o} resulting in a lower fraction F/f. That is the only reason of the curved form of results and theory.

Interaction of forces in a- and b-direction.

Whenever α_{aF} differs from 0, 90, 180 and 270^{o} there will be forces in both a- and b-direction. The test results show a distinct interaction effect on the failure load. Therefore the second order interaction formula (1) and (7) has been drawn through the test results, see Fig. 9, 10, 11 and 12. Each figure covers forces acting in all possible directions α_{aF} to the plate located at one specific joint/plate angle α.

Note that only the black dots (design values) have been used for determination of the design curves.

A few values are weaker than the design curve because of anchorage failure. Design against anchorage failure would avoid this in practice.



Legend:

-Number refer to test type, see Fig.4.

A - anchorage failure

B - buckling of plate

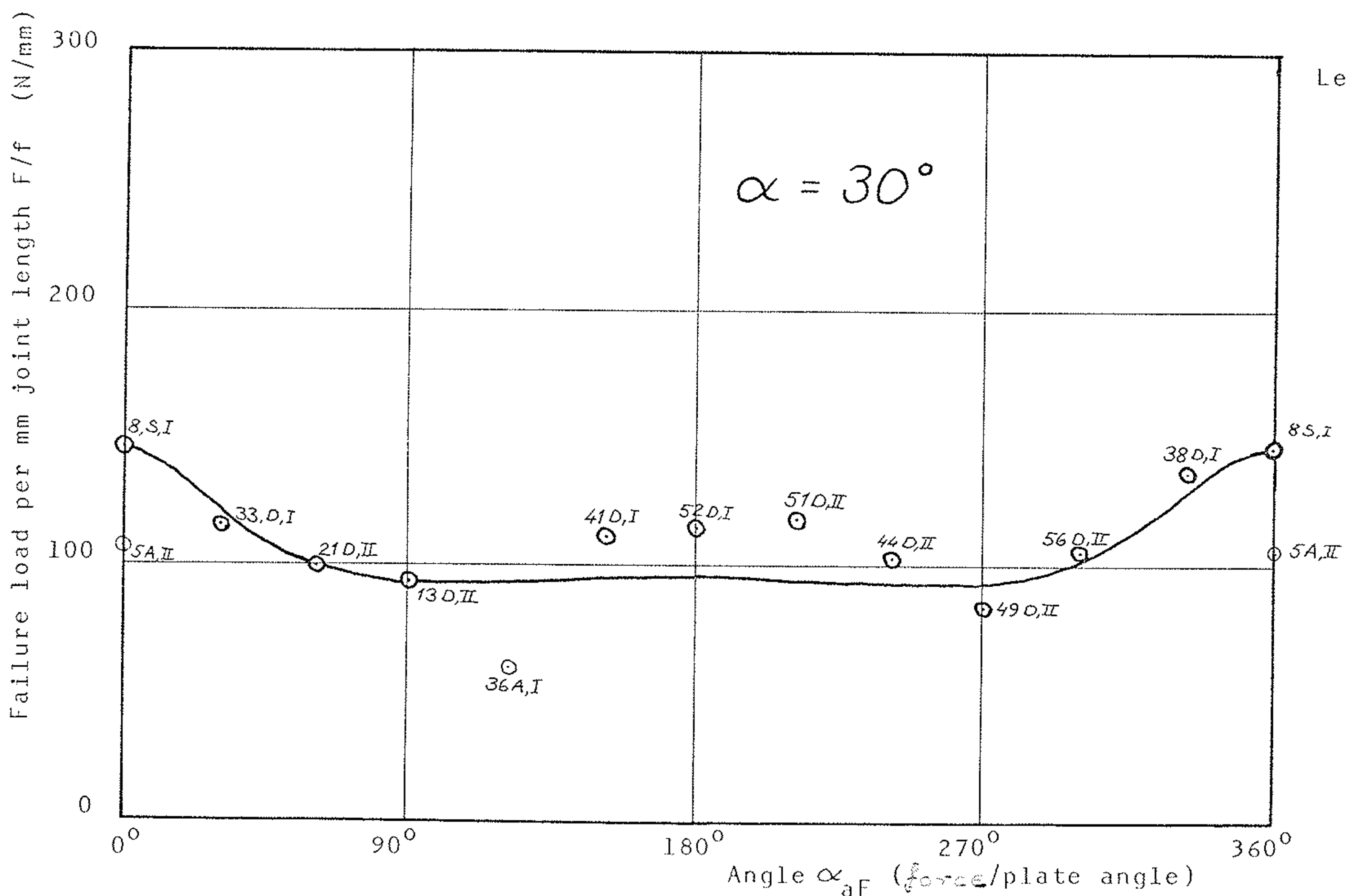
D - deformation of plate near joint.

S - steel tensile failure of plate.

I - plate size 72 x 196

II - plate size 107 x 117

Figure 9. Test results for $\alpha = 0^{\circ}$ and various angles of α_{aF}. Results below design curve were due to anchorage failure.
Black dots are design values used for calculation of design curve. from eq. (7).



Legend, see Fig. 9.

Figure 10. Test results for $\alpha = 30^0$ and various angles of α_{aF}.
Test series number 36 failed in anchorage area.
The design curve is based on design values marked with black dots in Fig. 9 and 12, and eq. (7).

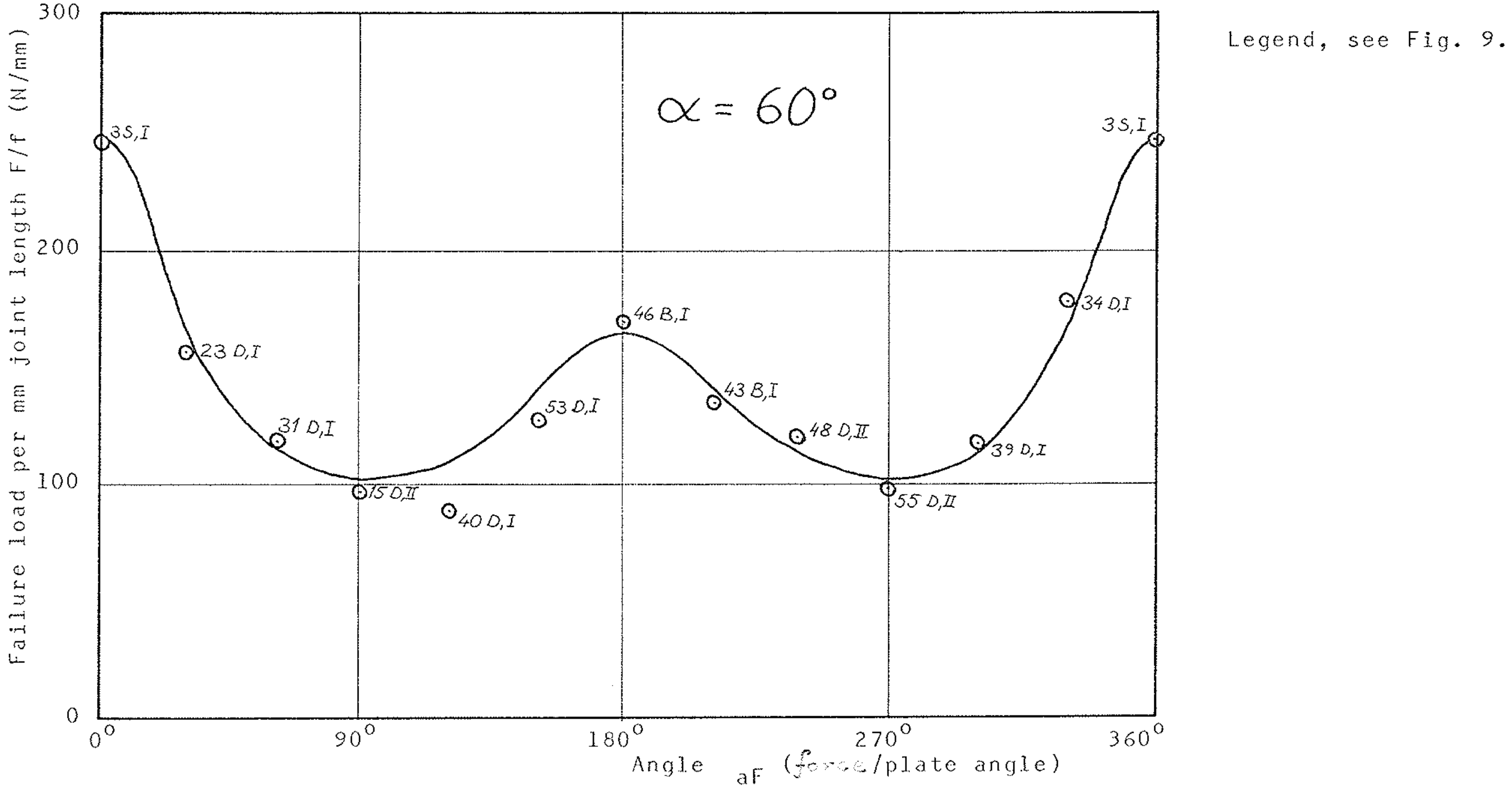
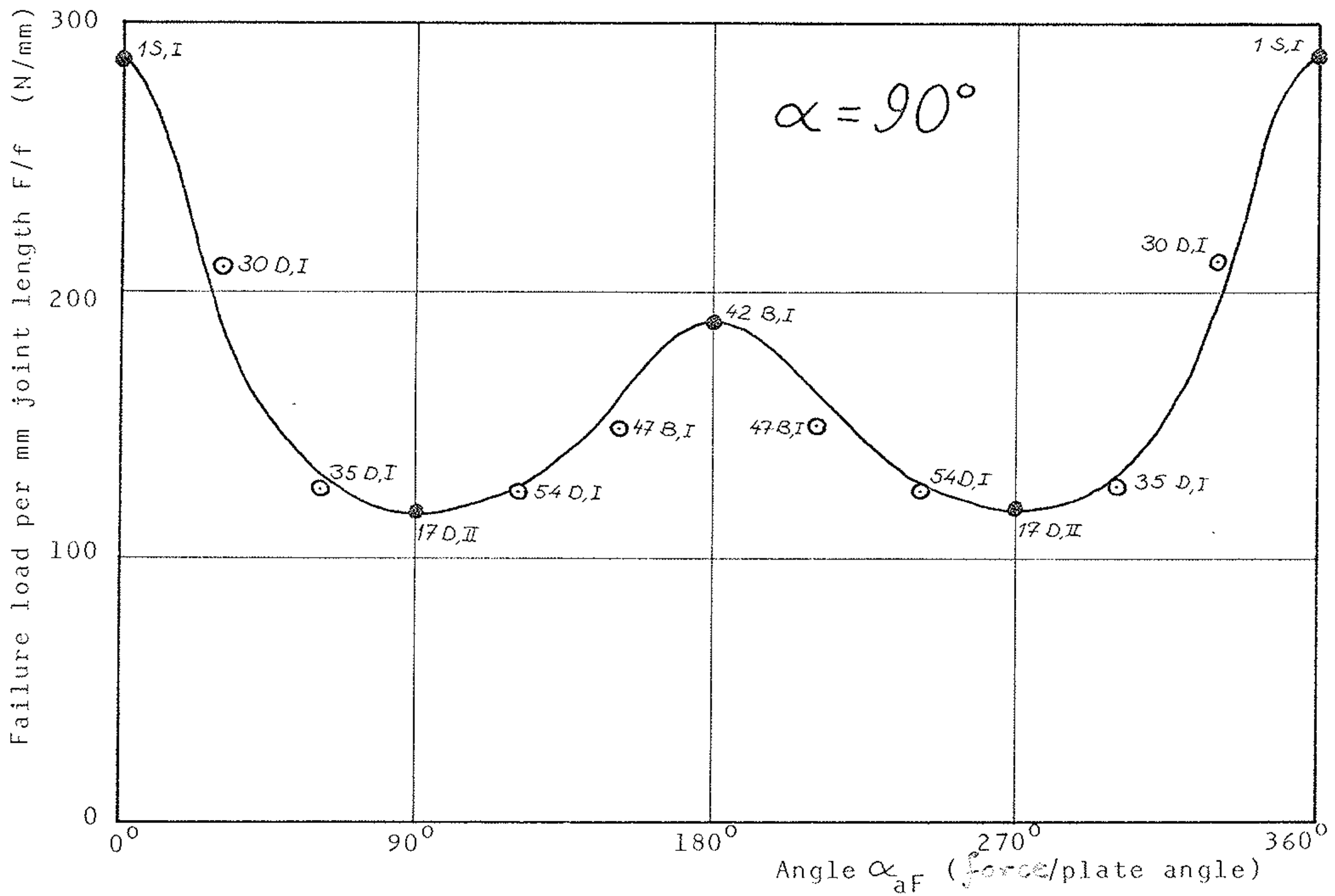


Figure 11. Test results for $=60^0$ and various angles of $_{aF}$.
The design curve is based on design values marked with black dots in Fig. 9 and 12, and eq. (7).



Legend, see Fig. 9

Figure 12. Test results for $\alpha = 90^0$ and various angles of α_{aF}.
Black dots are design values used for calculation of design curve, from eq. (7).

Conclusion

The presented design method gives a good prediction of the plate strength of nail plates. The existing Nordic rules are partly unsafe and do not reflect the orthotrop behaviour of nail plates.

The amount of testing is considerably reduced - down to the 6 design values, see Fig. 2. Tests with skew plates often give confusing results - the need of these tests is eliminated.

CIB-W18/18-9-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PREDICTION OF CREEP DEFORMATIONS OF JOINTS

by

J Kuipers

Stevin Laboratory

Netherlands

MEETING EIGHTEEN

BEIT OREN

ISRAEL

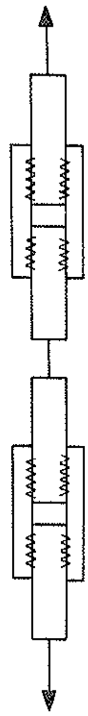
JUNE 1985

Prediction of creep deformations of joints.

1. Introduction.

In 1970 an investigation was started about the long-duration strength and the creep properties of joints with a special type of fastener. Four such joints remained loaded until 1983 at a load level of 0,30, i.e. a load of 30% of the mean short-duration strength.
Six different creep formulas were applied and compared with the creep measurements. Only one of them seems to predict the behaviour of the joints reasonably well, using the measurements of one half year.

2. Considered part of the investigation.



Two double-joint specimens with four joints were loaded in a string to a tensile force of 30% of the short-duration strength (mean value of 25 joints was 38,91 kN; coeff. of var. 0,10). The string was loaded with dead load, connected with a spring system to the cranerail in the not-conditioned hall of the Stevin Laboratory (temp. about 20 C; Rel. Hum. 0,45 to 0,75. The slip in the joints was measured manually during 13 years.
Other load levels were also incorporated in the investigation, namely 0,22; (0,30); 0,40; 0,60; 0,70 and 0,80.
The results were mentioned in [1].

3. Damage due to long duration of loading.

Five specimens loaded to a level of 0,22 during 2640 hours, another five loaded to a level of 0,40 during 4776 hours and four specimens loaded to a level of 0,30 during 13 years, were unloaded after these periods and then tested in a short-duration load. The results are given in table 1.

In these formulas a, b, c and d are constants, the values of which have been determined by fitting procedure. This has been done after several periods: 0,5 year; 1; 2; 5; 10 and 13 years.
Each set of constants was then used to predict the further development of the creep curve and the results compared with the real measurements.

The tables 1 to 6 and the graphs show that the predictive properties of the different formulas are not the same. If for instance a creep period of a half year is taken to predict the deformation after 13 years these predictions must be compared with the measured value of 1,125 mm, or, may be better, with the result of the fitted curves through the measurements during 13 years.

formula	prediction based on measurements during		real measurement
	½ year	13 years	
1	1,433	1,145	
2	0,988	1,107	
3	1,025	1,133	1,125
4	1,146	1,123	
5	0,615	1,057	
6	5,103	1,153	

From these results it was concluded that

- for this case — it is: this joint, at this load level in these circumstances — the formula 4 of these six gives the best results when predicting the deformations after 13 years on the basis of measurements during one half year;
- the simpler formula 3 is a good second-best, although it under-estimates the reality a little bit. It is of interest that in this case the only two constants a and b do not vary much when the fitting is based on longer periods of time than half a year (see table 3 for formula 3 and table 4 for formula 4).

Creep measurements of joints (Menig) loaded at different load levels.

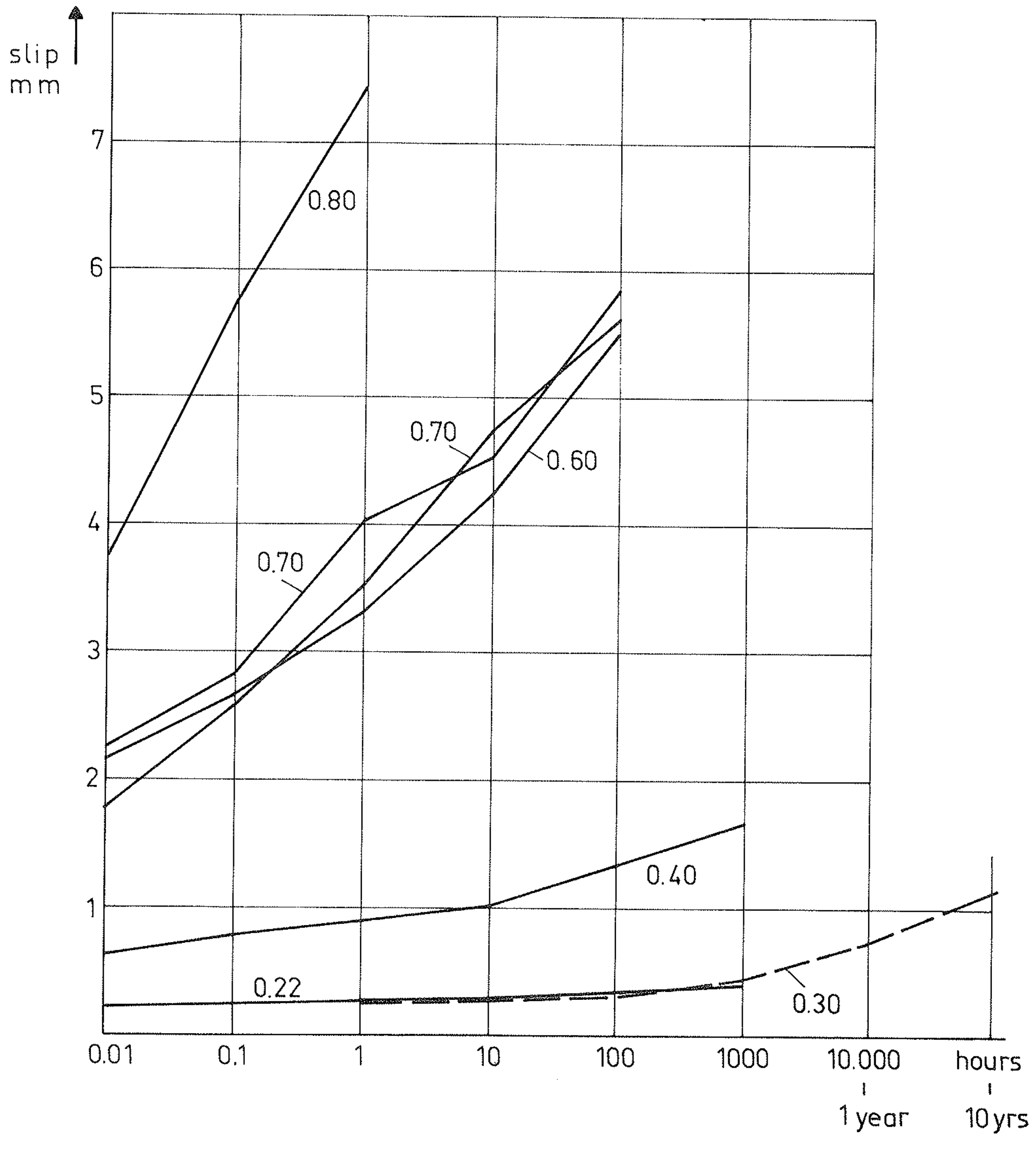
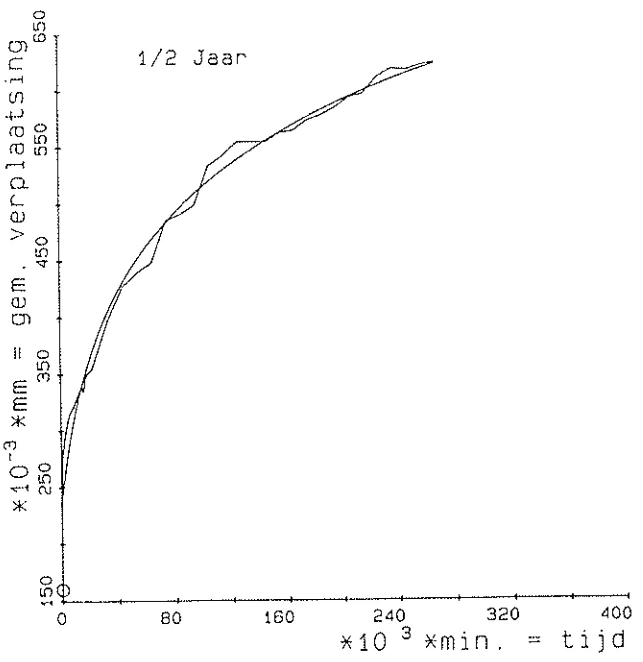


Table 2 - formula 2: $\varepsilon_t = a + b \ln(t + c)$

based on measurements of ... years	constants				fitted values (left) and predicted values (right) of slip after ... years							
	a	b	c	d	½	1	2	5	10	13	30	50
½	-0,8405	0,1168	9991		0,622	0,698	0,779	0,886	0,968	0,998	1,095	1,155
1	-0,9175	0,1233	11780		0,628	0,707	0,792	0,905	0,991	1,023	1,126	1,189
2	-1,067	0,1355	16050		0,633	0,720	0,813	0,936	1,031	1,066	1,179	1,248
5	-1,128	0,1409	16430		0,640	0,730	0,826	0,955	1,053	1,090	1,207	1,279
10	-1,157	0,1434	16430		0,642	0,734	0,832	0,963	1,063	1,100	1,220	1,293
13	-1,182	0,1454	16430		0,642	0,735	0,835	0,967	1,069	1,107	1,228	1,302
test results	-	-	-		0,624	0,715	0,826	0,940	1,076	1,125	-	-

Proefstukid. et = a + b*ln(t+c)



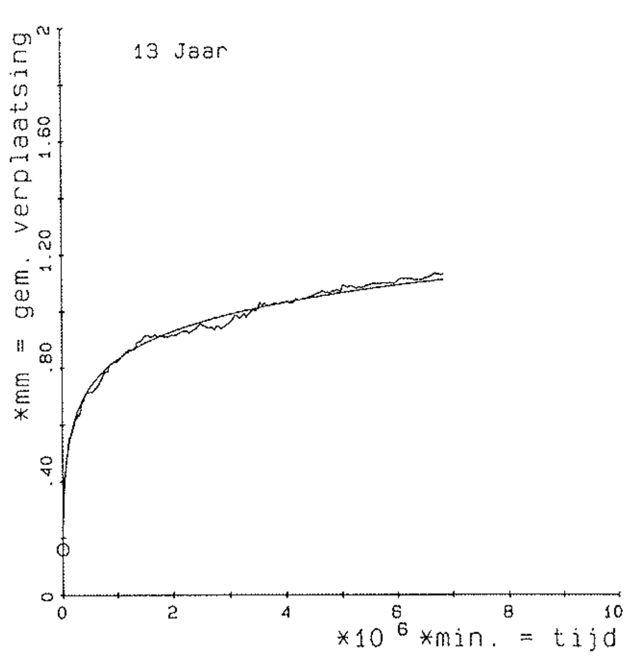
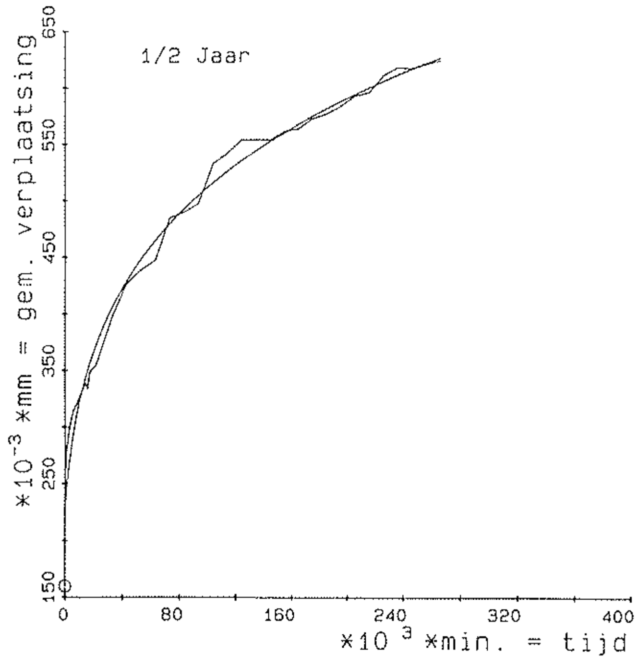


Table 4 - formula 4: $\varepsilon_t = a + b\,\ell n(t + d) + c\{\ell n(t + d)\}^2$

based on measure-ments of ... years	constants				fitted values (left) and predicted values (right) of slip after ... years							
	a	b	c	d	½	1	2	5	10	13	30	50
½	0,5481	-0,1155	0,00975	1702	0,627	0,717	0,820	0,970	1,096	1,146	1,312	1,421
1	0,5649	-0,1189	0,009914	1690	0,627	0,717	0,821	0,973	1,100	1,149	1,318	1,427
2	0,5918	-0,1243	0,01018	1692	0,628	0,719	0,824	0,978	1,107	1,157	1,328	1,440
5	0,1811	-0,0590	0,00759	5275	0,631	0,721	0,822	0,967	1,087	1,133	1,288	1,389
10	0,08599	-0,04237	0,006871	5640	0,631	0,719	0,819	0,960	1,077	1,121	1,271	1,368
13	0,1329	-0,04970	0,007154	5272	0,630	0,719	0,818	0,961	1,078	1,123	1,274	1,372
test results	-	-	-	-	0,624	0,715	0,826	0,940	1,076	1,125	-	-

Proefstukid. et = a+b*ln(t+d)+c*[ln(t+d)]**2



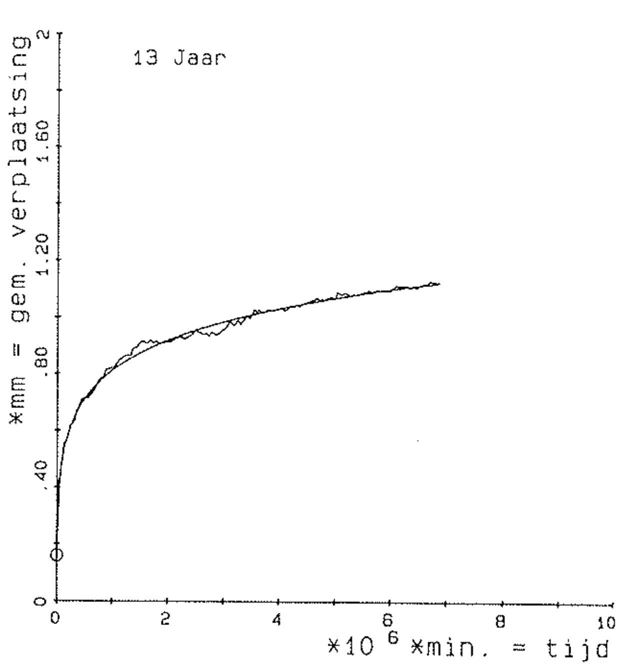


Table 6 - formula 6: $\varepsilon_t = a + b\,(1 - e^{-ct}) + dt$

based on measure-ments of ... years	constants				fitted values (left) and predicted values (right) of slip after ... years							
	a	b	c x 10^{-6}	d x 10^{-6}	½	1	2	5	10	13	30	50
½	0,2444	0,2093	33,67	0,6805	0,632	0,811	1,169	2,242	4,030	5,103	11,184	18,337
1	0,2495	0,2475	24,67	0,4646	0,618	0,741	0,985	1,718	2,939	3,672	7,823	12,707
2	0,2578	0,2931	16,78	0,2886	0,623	0,703	0,854	1,309	2,068	2,523	5,102	8,135
5	0,2880	0,4257	5,715	0,1017	0,645	0,746	0,820	0,981	1,248	1,409	2,317	3,386
10	0,2986	0,4854	4,17	6,060	0,637	0,761	0,842	0,943	1,103	1,198	1,740	2,377
13	0,3042	0,5099	3,615	4,961	0,629	0,764	0,855	0,944	1,075	1,153	1,596	2,118
test results					0,623	0,715	0,826	0,940	1,081	1,125	-	-

CIB-W18/18-10-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SUBMISSION TO THE CIB-W18 COMMITTEE ON THE DESIGN OF PLY WEB BEAMS BY CONSIDERATION OF THE TYPE OF STRESS IN THE FLANGES

by

J A Baird
The Swedish Finnish Timber Council
United Kingdom

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

SUBMISSION TO THE CIB/W18 COMMITTEE ON THE DESIGN OF PLY WEB BEAMS BY CONSIDERATION OF THE TYPE OF STRESS IN THE FLANGES

BY Jack A Baird
C.Eng.

May 1985

INTRODUCTION

The current CIB/W18 draft details a method of designing ply web beams. Considering the 'tension flange', the method calls for the applied stress at the extreme fibre to be limited to the permissible 'pure bending' value and the applied stress at mid-depth of the flange to be limited to the permissible 'pure tension' value.

The method may work in practice. However, it is not feasible to consider that the stress in the flange can change in half the depth of the flange from pure tension to pure bending. Particularly because of the large differences likely to be given in future codes between permissible tension and bending values I believe that a more logical method should be considered for CIB/W18 and suggest the following.

Here is an outline which should be adequate to explain the principle:-

1. Assuming that web splices are adequate, allocate bending moment to the flanges about the XX axis and the web/s in proportion to their stiffness about the XX axis. (If web splices are not adequate, allocate all bending moment to the flanges.)

2. Calculate the extreme fibre stress on the flanges by dividing the bending moment allocated to the flanges by the modulus of the flanges about the XX axis of the beam.

   Now look at the plot of this in Figure 1, shown for an I beam.

3. Express the stress distribution as shown in Figure 2. Considering this figure one can see that, in the 'tension flange', the stress in the extreme fibre is mainly 'pure tension' but contains an element of bending which varies over the depth of the flange. The 'compression flange' can be considered in a similar way.

4. As far as the tension flange is concerned the extreme fibre stress is checked as:

$$\frac{\text{Applied Bending Stress}}{\text{Permissible Bending Stress}} + \frac{\text{Applied Tension Stress}}{\text{Permissible Tension Stress}} \leq 1.0$$

$$\frac{\sigma_{m,a,\parallel}}{\sigma_{m,adm,\parallel}} + \frac{\sigma_{t,a,\parallel}}{\sigma_{t,adm,\parallel}} \leq 1.0$$

5. The compression flange is checked in a similar way.

Because of the differing permissible bending and tension stresses allocated in the U.K. Code of Practice, considering the extreme fibre stress as part tension, part bending, rather than entirely tension, allows a useful increase in the moment capacity of the beam.

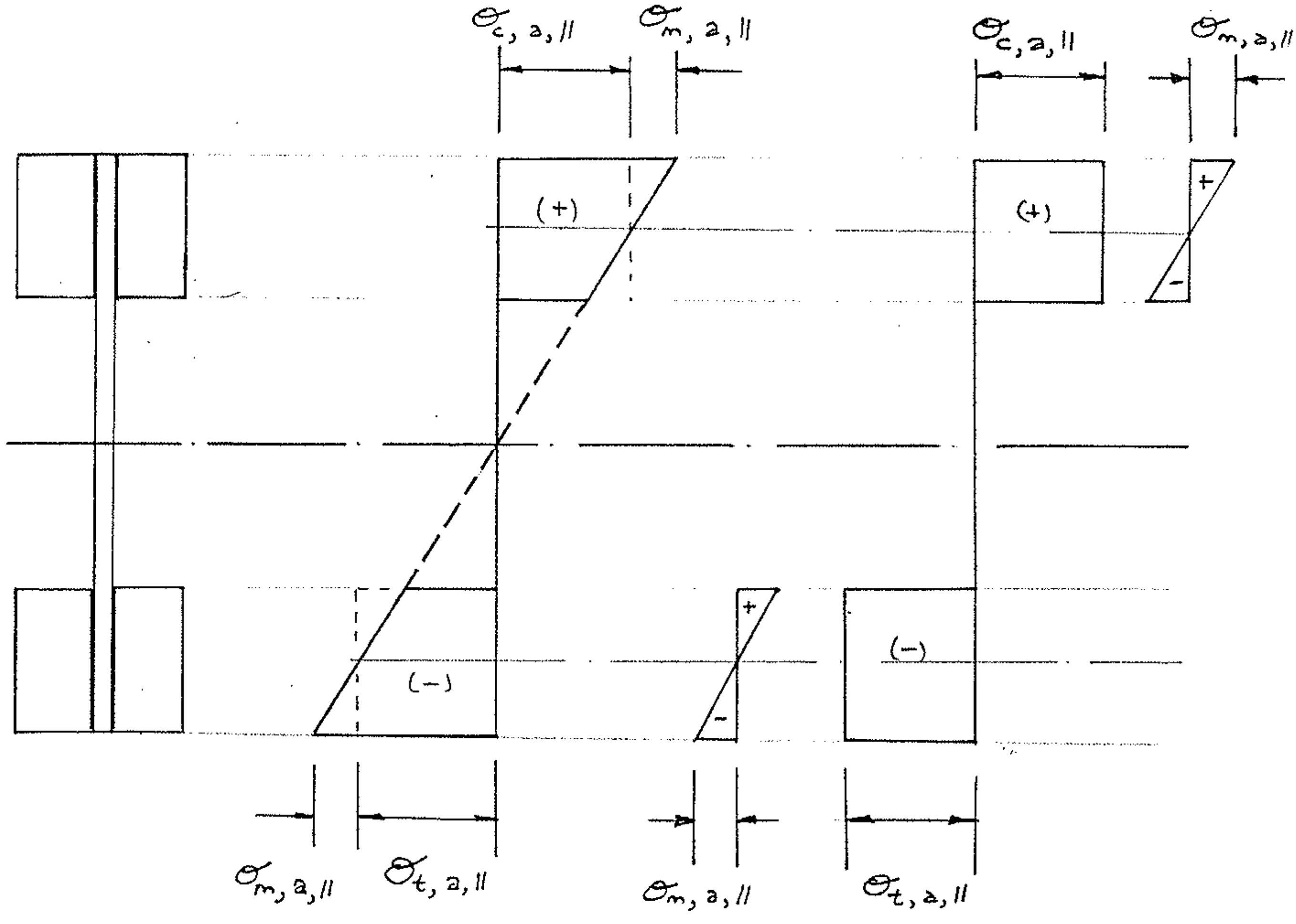


Figure 1. // Figure 2.

JAB
MAY 1985

CIB-W18/18-10-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

LONGITUDINAL SHEAR DESIGN OF GLUED LAMINATED BEAMS

by

R O Foschi
University of British Columbia
Canada

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

CIB-W18 Meeting, June 1985, Haifa, Israel

LONGITUDINAL SHEAR DESIGN OF GLUED LAMINATED BEAMS

(Comments on the Canadian Recommendations CSA-086)

by

Ricardo O. Foschi

Department of Civil Engineering
University of British Columbia
Vancouver, B.C., Canada

This short commentary refers to the design recommendations for longitudinal shear in glued-laminated beams, as they are currently implemented in the Canadian Code CSA-086.

The design criteria are based on research conducted during the 1970's at the (then) Western Forest Products Laboratory in Vancouver (now Forintek Canada Corp.). This research was based on the premise that longitudinal shear failures are of a brittle nature and that, therefore, the statistical theory of strength of brittle materials could be used to interpret test data and to formulate design procedures.

The fundamentals of brittle fracture theory are discussed by V.V. Bolotin in his book "Statistical Methods in Structural Mechanics" (1). The application of the theory to shear in timber has been shown in two publications in the Canadian Journal of Civil Engineering (2,3). The first of these discussed the theory and the second focusses on the derivation of a design procedure, which is essentially identical to the one adopted for CSA-086. A reprint of the paper in Ref. (3) accompanies this commentary.

The Theory

The concept of brittle fracture can also be described as a criterion for the failure of the weakest link in a chain. The probability of failure of such a chain was studied by Weibull and is controlled by the so-called Weibull probability distributions. Assume that we have a volume V of brittle material under a stress distribution S(x,y,z). The probability of failure p_f of such as volume is given by

$$p_f = 1.0 - e^{-\int_V \left(\frac{S-S_o}{m}\right)^k dV} \tag{1}$$

where m, S_o and k are distribution parameters (material properties) to be determined from tests. In particular, S_o is a threshold or minimum strength (there is zero probability of finding a strength lower than S_o). As shown in Eq. (1), the probability distribution is known as a 3-parameter Weibull. In the particular (and conservative) case of $S_o = 0.0$, the problem becomes controlled by only a 2-parameter Weibull (m and k).

According to Eq. (1), what is then required is the integration of the stresses S(x,y,z) over the volume V. Eq. (1) can be used in another manner. Consider two volumes, V_1 and V_2, the first with stresses S_1 and the second with stresses S_2. Then, if they have the same probability of failure, it follows that

$$\int_{V_1} {S_1}^k \, dV = \int_{V_2} {S_2}^k \, dV \tag{2}$$

Let V_2 now be a unit volume (say, 1 m3) under a uniform shear stress S*. Then, any other volume V under stresses S will have the same probability of failure as the unit volume under uniform shear if

$$\int_V S^k \, dV = S^{*k} \tag{3}$$

Now, letting the stresses S be normalized with respect to a nominal (or representative) stress S_N, we can write

$$S_N^k \int_V \theta^k(x,y,z) \, dV = S^{*k} \tag{4}$$

where $\theta(x,y,z)$ is the ratio between the actual stress S and the nominal S_N. Since the integral in Eq. (4) can always be written as

$$\int_V \theta^k(x,y,z) \, dV = I \, V \tag{5}$$

we can finally express the nominal stress at failure as follows:

$$S_N = \beta \frac{S^*}{V^{1/k}} \tag{6}$$

where β is a coefficient given by

$$\beta = \left(\frac{1}{I}\right)^{1/k} \tag{7}$$

Eq. (6) represents the design equation. If the strength of the unit volume S* is known at different probability levels, Eq. (6) permits the calculation of the strength S_N at the same probability levels. Thus, knowing the median we can compute the median, knowing the 5th-percentile we can compute the 5th-percentile. The strength is inversely proportional to the volume V, thus, the larger the volume the smaller the strength.

The coefficient β is a function of the type of loading applied. This coefficient can be computed from a stress analysis of the beam (say, a finite element analysis), or whenever the stress distribution is known. Since the coefficient k is generally of the order of 5.0 (approximately 1.2/coefficient of variation of strength), the integration of stresses will be dominated by those areas of stress concentration. Thus, the effect of high concentration may be offset by the reduced volume over which the stresses have high values. For this reason, for example, the theory predicts higher strength for a uniformly loaded beam than for a beam with a concentrated load at midspan.

The Canadian Code CSA-086 (1980) has chosen to use the notation K1 for the coefficient β of Ref. (3). The tables of K1 as they appear in CSA-086 (1980) are enclosed with this commentary.

The definition of the nominal stress used for checking in CSA-086 is as follows:

$$S_N \approx 1.5 \frac{W}{A} \tag{8}$$

where W is the total load applied to the beam (i.e., sum of all concentrated loads plus all distributed loads), and A is the beam cross-section. Thus, this avoids the need to compute the shear force, a most convenient choice in the case of moving loads. In this situation, the force W is the sum of all the wheel loads plus all the distributed loads, even if all wheel loads cannot be on the span at the same time.

Combining Eqs. (6) and (8), it is possible to arrive at a direct design equation (since Eq. (6) would appear to require a trial and error procedure in its use). As the result of the combination, CSA-086-1980 (Clause 5.3.4.1) requires that the beam cross-section, in order to meet shear requirements, satisfy the following relationship:

$$A = BH \geqslant \left(\frac{1.5\ W}{\beta\ S^{*}}\right)^{\frac{k}{k-1}} L^{\frac{1}{k-1}} \tag{9}$$

where L is the beam span.

The values of m and k and the distribution for the strength S* can be obtained from tests, not necessarily of beams. One of the advantages of the theory is that it allows a comparison of different test specimens: thus, beam tests can be combined with torque tubes or (in North America) ASTM standard specimens. The fit of the distribution parameters to test data is shown in Ref. (2).

The theory has been confirmed experimentally, as shown in (2). Furthermore, it has been applied to obtain the wood failure loads in "glulam rivet connections", when this failure is controlled by shear around the cluster of nails. Again, these predictions have been verified experimentally (4).

In conclusion, Weibull's brittle fracture approach appears to be a very useful tool to characterize the size effect implicit in shear strength, and has been implemented with success for several years in the Canadian Code CSA-086.

(1) Bolotin, V.V. "Statistical Methods in Structural Mechanics", Holden-Day Inc., San Francisco, 1969, p. 51.

(2) Foschi, R.O. and Barrett, J.D. "Longitudinal Shear Strength of Douglas Fir", Canadian Journal of Civil Engineering, Vol. 3, No. 2, 1976, pp. 198-206.

(3) Foschi, R.O. and Barrett, J.D. "Longitudinal Shear in Wood Beams: A Design Methods", Canadian Journal of Civil Engineering, Vol. 4, No. 3, 1977, pp. 363-370.

(4) Foschi, R.O. and Longworth, J. "Analysis and Design of Griplam Nailed Connections", Journal of the Structural Division, ASCE, Vol. 101, No. ST12, Dec. 1975, pp. 2537-2555.

Table 17a
Factor K_1 for Single Span Beams*†

Number of Equal and Equally Spaced Concentrated Loads	Ratio e/L	r			
		0.0	0.5	2.0	10.0 and Over
1	—	3.69	3.15	2.55	2.11
	‡	3.69	3.34	2.92	2.46
2	0.1	For Uniformly Distributed Loading Only: K_1 = 3.69	3.23	2.68	2.25
	0.2		3.29	2.81	2.40
	0.3		3.35	2.94	2.56
	0.4		3.38	3.04	2.71
	0.5		3.41	3.12	2.84
	0.6		3.47	3.22	2.99
	0.7		3.58	3.40	3.21
	0.8		3.83	3.73	3.60
	0.9		4.34	4.53	4.53
	‡		3.37	3.01	2.67
3	0.1		3.28	2.80	2.40
	0.2		3.38	3.04	2.71
	0.3		3.46	3.22	2.98
	0.4		3.75	3.70	3.59
	‡		3.41	3.12	2.84
4	0.1		3.33	2.91	2.54
	0.2		3.45	3.21	2.97
	0.3		3.99	4.25	4.41
	‡		3.45	3.21	2.97
5	0.05		3.27	2.78	2.37
	0.10		3.37	3.01	2.68
	0.15		3.44	3.19	2.96
	0.20		3.64	3.57	3.48
	0.25		4.03	4.39	4.73
	‡		3.48	3.28	3.08
6	0.05		3.30	2.84	2.44
	0.10		3.39	3.10	2.81
	0.15		3.55	3.40	3.26
	0.20		3.94	4.21	4.46
	‡		3.51	3.34	3.16

$$r = \frac{\text{Total of concentrated loads}}{\text{Total of uniform loads}}$$

e = distance between two consecutive concentrated loads

**Intermediate values of K_1 can be linearly interpolated.*

†*Values shown correspond to the worst position of concentrated load unless noted ‡.*

‡*Indicated values for concentrated loads equally spaced and symmetrically located on the simple span.*

Table 17b
Factor K_1 for Distributed Loads

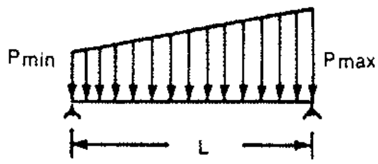
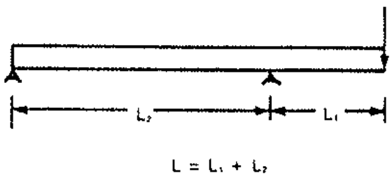
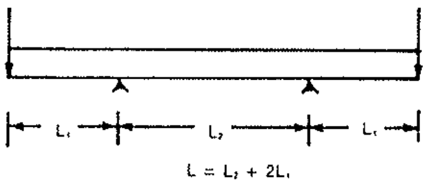
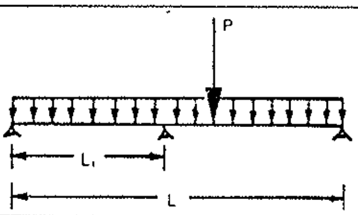
Type of Loading	P_{min}/P_{max}					
	0.0	0.2	0.4	0.6	0.8	1.0
	3.40	3.55	3.63	3.67	3.69	3.69

Table 17c
Factor K_1 for Cantilevered Beams

Beam Type and Loading	L_1/L_2	r			
		0.0	0.5	2.0	10.0 and Over
	0.05 0.10 0.20 0.30	3.91 4.13 4.55 4.88	5.64 5.19 4.36 3.83	4.06 3.07 2.53 2.31	2.73 2.08 1.75 1.62
	0.05 0.10 0.20 0.30	4.13 4.58 5.50 6.40	6.19 6.72 6.90 6.31	7.13 5.42 4.49 4.10	4.86 3.72 3.17 2.97

$$r = \frac{\text{Total of concentrated loads}}{\text{Total of Uniform loads}}$$

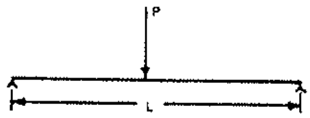
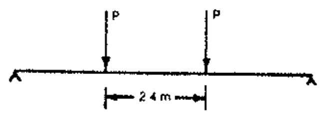
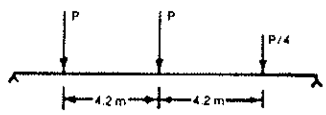
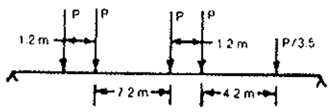
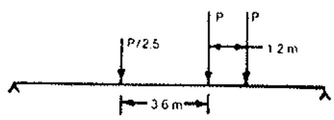
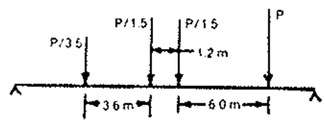
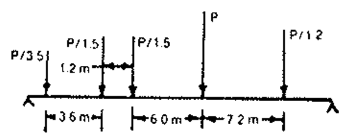
Table 17d
Factor K_1 for 2-Span Continuous Beams

Loading Case*	L_1/L	r			
		0.0	0.5	2.0	10.0 and Over
	0.2 0.3 0.4 0.5	4.09 5.10 6.09 6.66	3.04 3.48 3.96 4.42	2.35 2.57 2.82 3.07	2.01 2.15 2.32 2.50

$$r = \frac{\text{Concentrated load}}{\text{Total of Uniform loads}}$$

**Values shown correspond to the worst position for the concentrated load.*

Table 17f
Factor K_1 for Moving Loads

Type of Concentrated Loads	Span L (m)	r = 0.5	r = 1.5	r = 3.0	r = 10.0 and Over
	10	3.29	2.63	2.37	2.01
	20	3.29	2.63	2.37	2.01
	30	3.29	2.63	2.37	2.01
	40	3.29	2.63	2.37	2.01
	10	3.57	2.97	2.72	2.37
	20	3.42	2.80	2.53	2.18
	30	3.38	2.73	2.47	2.11
	40	3.35	2.71	2.44	2.09
	10	3.99	3.56	3.35	3.04
	20	3.67	3.11	2.86	2.52
	30	3.58	2.99	2.73	2.38
	40	3.52	2.91	2.65	2.29
	10	4.67	4.63	4.60	4.52
	20	3.99	3.56	3.35	3.05
	30	3.75	3.23	2.97	2.65
	40	3.65	3.08	2.82	2.47
	10	3.66	3.09	2.84	2.49
	20	3.54	2.95	2.68	2.33
	30	3.46	2.84	2.57	2.22
	40	3.42	2.79	2.52	2.16
	10	4.42	4.22	4.09	3.90
	20	3.86	3.37	3.14	2.81
	30	3.70	3.14	2.90	2.56
	40	3.60	3.01	2.76	2.41
	10	4.86	5.00	5.08	5.18
	20	4.29	4.01	3.87	3.65
	30	4.01	3.58	3.37	3.08
	40	3.84	3.33	3.10	2.77

$$r = \frac{\text{Total of all concentrated loads}}{\text{Total of all uniformly distributed loads}}$$

CIB-W18/18-12-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

REPORT ON EUROPEAN GLULAM CONTROL AND PRODUCTION STANDARD

by

H Riberholt
Technical University of Denmark
Denmark

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

Report on European Glulam Control and Production Standard.

From an initiative taken by F.E.M.I.B., Sous Commission GLULAM a process has started which eventually will end up with a simplification of the the production and the control of glulam.

The following representatives of European control bodies have attended two meetings, one in November 1984 and one in March 1985.

Franz Solar Österreich. Holzforschungsinstitut	A
E. Sauvage (chairman) CTIB, Brussel	B
Christian H. Burgbacher Studiengemeinschaft Holz-Leimbau	BRD
A. Epple Forschungs- und Materialprüfungsanstalt	BRD
H. Riberholt Techn. University of Denmark	DK, SF, N, S
A. Demange CTB, Paris	F
Gérard Sagot Syndicat National des Constructeurs.	F
G.N. Ruysch TNO, Delft	NL

It is the intention of the group to set up:

Bilateral or multilateral control agreements so the control body of the export country surveies that the glulam is manufactured according to the standards and rules of the import country.

A common production standard and common rules for approval and control of glulam factories.

This intention has been endorsed by 8 representatives, see the enclosure.

Bilateral control agreements have already for some years been established between some countries, and will thus be extended further.

The work with the common production standard have started, and it will comprise only the production of soft wood glulam. The requirements will be formulated considering European conditions mainly.

It has been agreed that the production standard will go along the same line as the present national standards. The main object will be to set up requirements based on the common experience and knowledge in the European countries.

Concerning the rules for approval and control it has been realized that there should be a set of rules partly for the factory and the production equipment, partly for the control of the product, i.e. the glulam.

CIB-W18/18-14-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIMPLIFIED CALCULATION METHOD FOR W-TRUSSES (PART 2)

by

B Källsner
Swedish Institute for Wood Technology Research
Sweden

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

Introduction

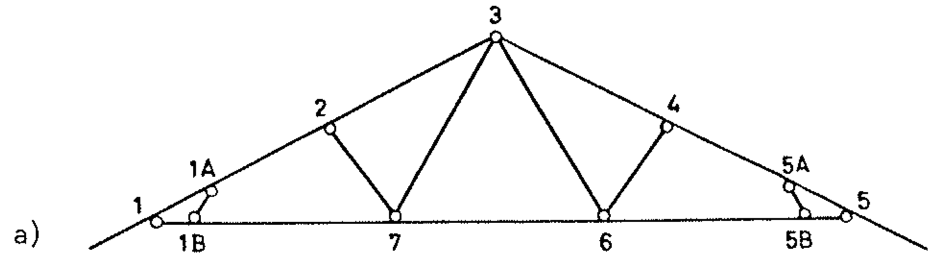
At the CIB-W18 meeting in Rapperswil 1984 the author presented a paper /1/ entitled "Simplified calculation method for W-trusses". It was stated in this paper that a more complete report dealing with W- and WW-trusses would be published later. This complete report is now under preparation.

The calculation method presented in /1/ dealt with the general case of a heel joint subjected to an eccentric support reaction. At the CIB-W18 meeting some participants suggested that the proposed analytical method was too complex for design application. The author's response to the suggestion was that it was possible to develop a simplified procedure based on the results of the study. This is demonstrated in this paper. The calculation method is simplified by neglecting some moment contributions which have only a small influence on the total bending moment distribution in the truss, and which are thus not significant.

The calculation method is presented for two different cases, a W-truss without support eccentricity at the heels and a W-truss with support eccentricity at the heels.

Assumptions

In the case of a W-truss with support eccentricity at the heels the calculation method is based on the static model presented in Figure 1a. In the case of a W-truss without support eccentricity at the heels, the fictitious bars 1A-1B and 5A-5B are omitted and the static model shown in Figure 1b is obtained. The upper and lower chords are assumed to be continuous beams and pin-connected at the heels. The fictitious bars in the heel joints and the diagonals are assumed to be pin-connected to the chords. As far as the chords are concerned, the system lines are placed at the centre of gravity of the timber. In the diagonals, the system lines have been placed so that no eccentricities occur at their connection to the upper and lower chords. In the same way it is assumed that the hinges between the fictitious bars and the chords are placed on the system lines of the chords.



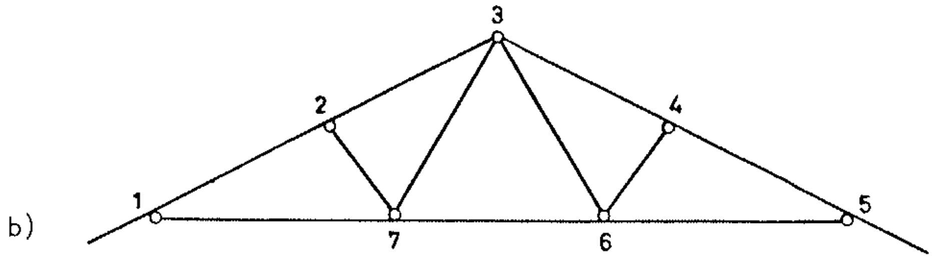


Figure 1. Static model for a W-truss.
a) with support eccentricity at the heels
b) without support eccentricity at the heels.

In the case of support eccentricity at a heel, the angle between the fictitious bar and the upper chord shall not be less than 14 degrees, and the point of intersection of the fictitious bar with the system line of the upper chord shall be 50 mm from the end of the wedge along the upper chord, see Figure 2.

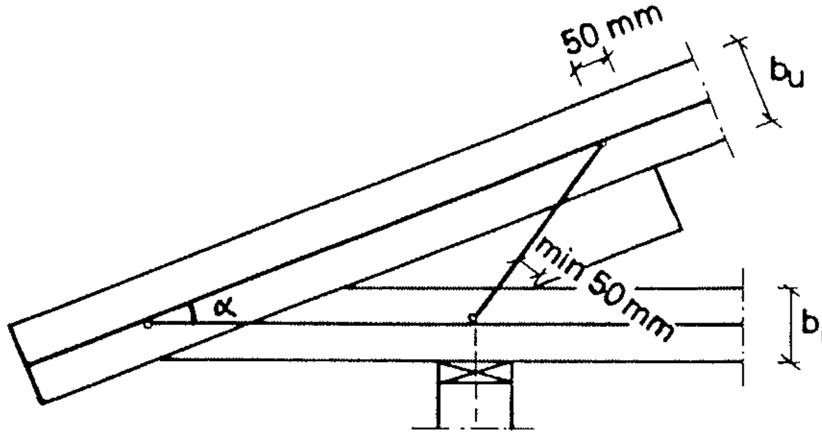


Figure 2. Example of a heel joint with a wedge. Assumed static model.

Finally, the fictitious bar shall be placed at least 50 mm from the point of intersection of the underside of the wedge and the upper side of the lower chord.

The roof truss is assumed to be symmetrical with respect to the orientation of the chords and the diagonals but not with regard to the supports. The notations used in the paper are defined in Figure 3.

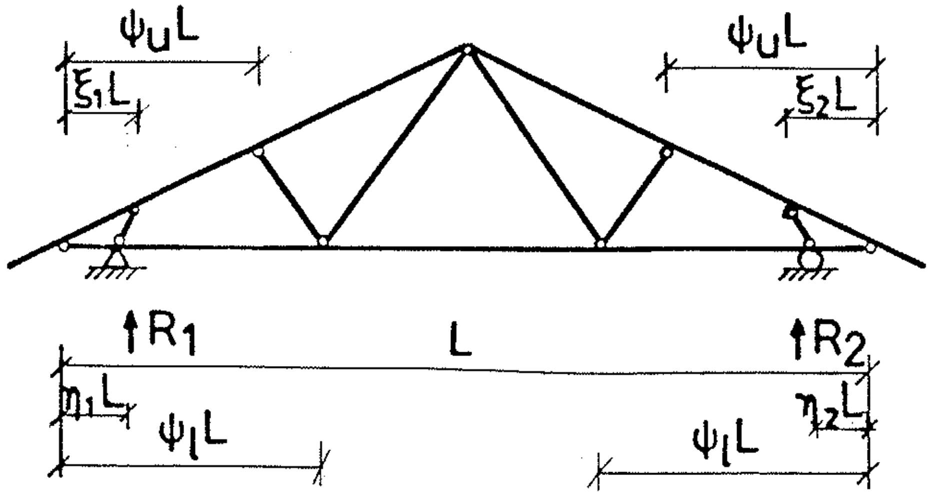


Figure 3. Notations introduced for geometry and support reactions.

The uniformly distributed loads acting on the roof truss are divided into three parts, as shown in Figure 4. These are q_{us}, the symmetrical upper chord load, q_{ua}, the asymmetrical upper chord load, and q , the symmetrical lower chord load.

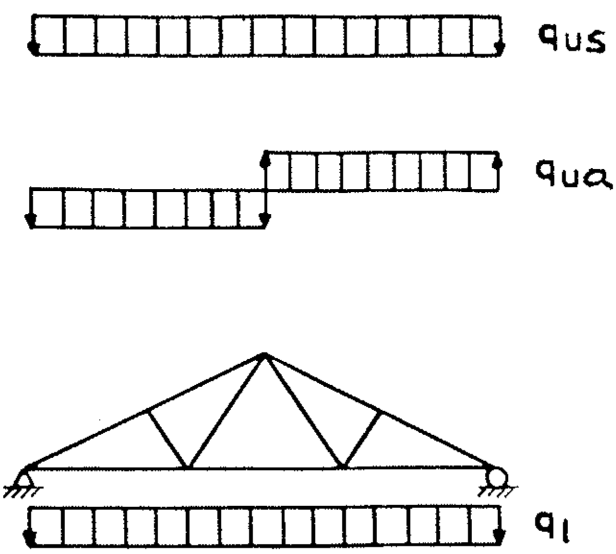


Figure 4. Notations introduced for loads.

The unit-load method was used to calculate the displacements of the joints and the resulting bending moments. A detailed description of the application of this analysis method is included in the report under preparation and the reader is recommended to refer to this report.

W-truss without support eccentricity at the heels

In the case where the support eccentricity is either very small or non existent and no special arrangements have been made to increase the stiffness of the heel joints it is reasonable to assume that there is no transfer of bending moments between the upper and the lower chords. By using the static model shown in Figure 1b the bending moment distribution of a W-truss easily can be calculated in two steps.

1. Calculate the bending moment distribution caused by the external loads q_{us}, q_{ua} and q_{ℓ} under the assumption that the chords are simply supported on rigid supports at nodes 1-7.

2. Consider the influence of axial deformation in the chords and the diagonals caused by the axial forces. The corresponding displacements of the nodes will generate the additional bending moments:

$$\Delta M_2 = (0.55\ q_{us} + 0.21\ q_{ua} + 0.52\ q_{\ell}) \frac{b_u^2}{\sin^2\alpha}$$

$$\Delta M_4 = (0.55\ q_{us} - 0.21\ q_{ua} + 0.52\ q_{\ell}) \frac{b_u^2}{\sin^2\alpha}$$

$$\Delta M_6 = (0.30\ q_{us} - 0.19\ q_{ua} + 0.29\ q_{\ell}) \frac{b_{\ell}^2}{\sin^2\alpha}$$

$$\Delta M_7 = (0.30\ q_{us} + 0.19\ q_{ua} + 0.29\ q_{\ell}) \frac{b_{\ell}^2}{\sin^2\alpha}$$

where b_u = the depth of the upper chord (Figure 2).
b_{ℓ} = the depth of the lower chord (" ").
α = the slope of the roof (" ").

In the derivation of the coefficients it was assumed that the cross-sectional properties of the upper and lower chords did not differ considerably from each other. Also, the cross-sectional properties of the diagonals were assumed to be small relative to those of the chords.

W-truss with support eccentricity at the heels

In the case of support eccentricity at the heels, the solution with a wedge can be used. An example of such an approach is shown in Figure 2. Even when the eccentricities are moderate, the bending moments caused by the support eccentricity can be rather high.

For the calculation of the bending moments the static model of Figure 1a can be used. A simplification can be made by neglecting the forces transmitted to the fictitious bars at the heels when the bending moments caused by uniformly distributed loads are calculated. With this simplification it is only necessary to consider the forces transmitted to the fictitious bars originating from the support locations at the heels. Another simplification can be made by neglecting the influence of the axial deformations caused by the support reactions at the heels when the truss is regarded as simply supported at the nodes 1 and 5.

The bending moment distribution for this type of W-truss can thus be determined in three steps.

1. Calculate the bending moment distribution caused by q_{us}, q_{ua}, q_ℓ in the same way as for the W-truss without support eccentricity at the heels.

2. Consider the influence of the axial deformations in the chords and the diagonals in the same way as for the W-truss without support eccentricity at the heels.

3. Determine as to how the support reactions R_1 and R_2 are distributed to the upper and lower chords via the fictitious bars at

the left and the right heel joints. The vertical components of the forces acting on the upper and lower chords denoted by X_{ui} and $X_{\ell i}$ (Figure 5) can be approximated as follows:

$$X_{\ell i} = \frac{R_i}{1 + \frac{E_u\, I_u}{E_\ell\, I_\ell}\,\frac{\Psi_\ell}{\Psi_u}\cos\alpha\,\frac{\left(1 - \frac{\eta_i}{\Psi_\ell}\right)^2}{\left(1 - \frac{\xi_i}{\Psi_u}\right)^2}}$$

$$X_{ui} = R_i - X_{\ell i}$$

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

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

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

Calculate the additional bending moments in the upper and lower chords by assuming that the chords are simply supported on rigid supports at nodes 1-7.

Influence of joint slip on moment distribution

The influence of joint slip on the moment distribution can easily be included in the calculation method by assuming that these reach specific fixed values. The additional bending moments can be calculated by the following equations:

$$\Delta M_2 = c_2 \frac{E_u\, I_u}{L^2 \sin\alpha}$$

$$\Delta M_4 = c_4 \frac{E_u\, I_u}{L^2 \sin\alpha}$$

$$\Delta M_6 = c_6 \frac{E_\ell\, I_\ell}{L^2 \sin\alpha}$$

$$\Delta M_7 = c_7 \frac{E_\ell\, I_\ell}{L^2 \sin\alpha}$$

The above equations were developed by considering the slips as axial deformations in the members. The values of the coefficients "c", which have the dimension of length, depend on the type of connector used.

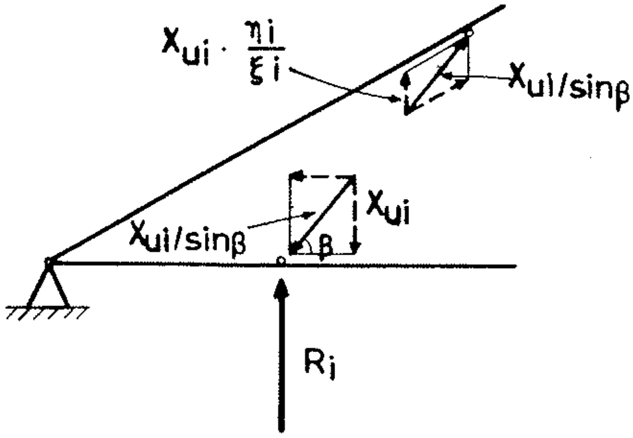


Figure 5. Assumed forces acting in the fictitious bar.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

MODEL FOR TRUSSED RAFTER DESIGN

by

T Poutanen
Tampere
Finland

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

MODEL FOR TRUSSED RAFTER DESIGN

by

Tuomo Poutanen

CONTENTS

1. SUMMARY

The paper describes a design model for trussed rafter with emphasis on stress analysis. The model is based on a two-dimensional nonlinear frame program. Special attention is paid to joint modeling. Joint eccentricities are calculated assuming that all forces are transferred through nail plates, which means no contact and friction between timber beams. The model is used in everyday trussed rafter design. Some design experience is described.

2. INTRODUCTION

The nordic M-group (Aasheim, Källsner, Poutanen, Riberholt) has under the leadership of Riberholt worked out paper /1/. The aim of the group has been to define guidelines both in sophisticated and simple design.

In Finland trussed rafters are very different from those used elsewhere due to several factors e.g.

- heavy snow load
- wide spacing of trusses (900 or 1200 mm)
- thick insulation (350...450 mm)
- large use of attic trusses (frames) with composite beams

The author has worked paper /1/ further and developed a design method and computer program which differ from /1/ mainly in 4 points:

a) All connections are assumed to be semi-rigid (moment and force) and in extreme cases include pin jointed and fully stiff assumptions

b) Eccentricites are calculated assuming that forces are transferred through centroids of anchorage area (not through center lines of timber beams)

c) Buckling is taken into consideration using geometric nonlinear (physically linear) analysis

d) Rotation between nail plate and timber is calculated and its effect on deflection and strength is considered.

Otherwise design principles are the same as in /1/, though many limitations can be omitted because the model seems to be suitable to analyze and design any kind of 2-dimensional structure made of nail plates and straight timber beams.

3. NOTATION

The same notation as in /1/ and other general notations are used. The following notation is further explained:

Trussed rafter is a load bearing structure which is loaded, supported and deflected in two dimensions and made of nail plates and timber beams. The structural model of trussed rafter may be truss, frame or beam or a composition of these.

Timber beam is a straight timber member having constant cross section with one or two end cuts: in an extreme case the timber beam is triangular (wedge).

Connection is the part of trussed rafter where the nail plate is pressed into the timber beam. One nail plate and one timber beam form one connection.

Joint is a composition of several connections with the same nail plate and several timber beams. If several nail plates are near each other, they form several joints, which may overlap each other.

Node is a point which is utilized in (two dimensional) frame analysis. It can move in 0, 1, 2 or 3 dimensions, each connection having 3 or 5 nodes. Most nodes are located in connections but there may also be nodes elsewhere in the structure e.g. in supports and point loads.

Member is a straight line utilized in frame analysis connecting two nodes. Members may be "real", having correspondence in real geometry and having the same static properties as in real geometry, on fictitious, having calculative static properties.

Spring is an element utilized in frame analysis connecting two nodes (elastically) to each other.

4. ASSUMPTIONS

4.1 General

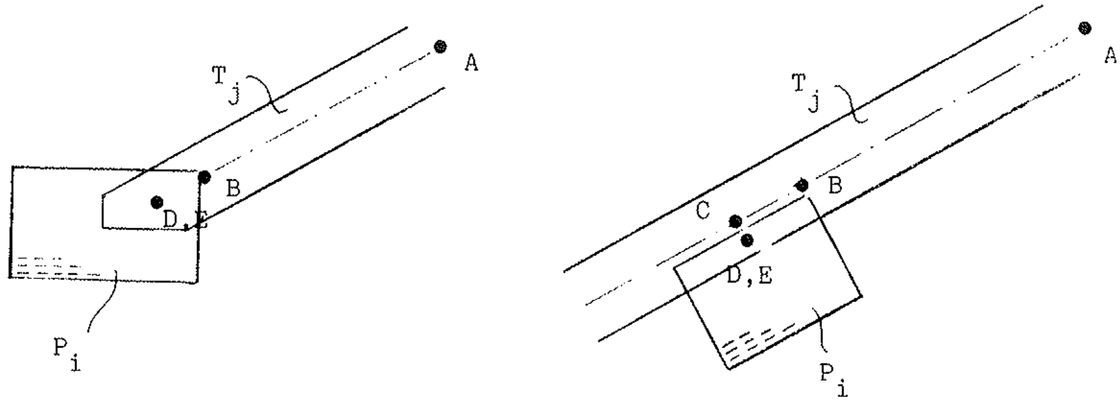
A trussed rafter has two kinds of physical elements: timber beams and nail plates. For calculation purposes a structural model is created. It can be defined by members, nodes and springs. In addition, physical reality and its structural model have boundary elements: supports and loads.

4.2 Connection model

A connection has one nail plate and one timber beam which may end in the connection or pass through it. According to this, a timber beam may have two structural patterns in the connection, shown in figure 1, where plate P_i is pressed into timber beams T_j.

Nodes A-E are located in the following way:

- A is a node which is located on the center line of a timber beam, its position on the center line being defined by load, support, timber end or other connection.
- B is located on the center line of a timber beam and on the boundary line of a connection, which is a line perpendicular to the center line touching upon the anchorage area of a nail plate.
- C is a node used only if the timber beam passes through a connection. It is on the center line of a timber beam, and on a line perpendicular to the center line passing through point (node) D.
- D is in the center of an anchorage area formed by plate i and timber beam j, D is located on the timber and moves with it.
- E has the same position as D but is located on the plate and moves with it.



a) Timber beam ends in connection.

b) Timber beam passes through connection.

Fig. 1.
Connection model, connection between nail plate Pi and timber beam Tj.

Nodes D and E are connected to each other by (elastic) springs. Spring value k_δ between deflection and force is calculated:

$$k_\delta = 2A\, k_\tau \tag{1}$$

where

k_τ = stiffness of nail plate (N/mm^3), got from nail plate tests.
A = anchorage area of nail plate and timber

Spring value k_φ between rotation and moment is calculated:

$$k_\varphi = 2I\, k_m \tag{2}$$

where

I = polar moment of inertia of anchorage area A
k_m = stiffness of nail plate (N/mm^3), got from nail plate tests. It may be justified (if better information is not available) to use value

$km = k_{\tau}/1{,}5$, but in most connections the km value has no practical effect on the results. If km is reduced, rotation is increased but stresses remain unchanged. There are nevertheless a few connections where km has a large influence.

Nodes A, B, C, D, E are nodes which are used in two dimensional frame analysis. Between nodes there are members which have the following stiffness and strength values:

	stiffness value	strength value
A - B	calculated from cross section (bxh) and elasticity (E)	calculated from timber code
B - C,D	as A - B	∞ (failure never occurs in this member)
C - D	∞	∞

One nail plate connects several timber beams and there are several patterns of nodes in each joint. It is assumed that the nail plate moves as a solid element. This is achieved by connecting all nodes F to each other by members with structural stiffness ∞.

If there is support or point load in the joint, an extra node or nodes may be needed. Then members B - C,D are divided into two or more short members.

It is assumed that timber failure never occurs under a nail plate. This assumption is made for practical reasons. Point loads and supports are often located on a joint which creates large stress peaks. It is assumed that these stress peaks are not real and they can be ignored.

4.3 Contact

In connections of trussed rafters gaps between timber beams must be allowed due to production inaccuracies.

Present Finnish tolerance standards allow timber beams to move (appr.) 0...2 mm and rotate 0...10 mrad until full contact is made between connecting timber beams.

If contact is somehow utilized, assumptions about size and location of contact force and also stiffness of contact connection must be made.

Many combinations of assumptions can be made, and different assumptions may lead to complicated calculations.

In this model the following assumption has been made: compressed connection behaves in the same way as tension connection. This means that stresses are calculated assuming no contact. This assumption is justified for several reasons:

- Calculation is very simple e.g. it is not necessary to distinguish between tension and compression connections before calculation.
- The assumption is on the safe side as far as deflection is concerned and calculation results correspond relatively well to the case of large gap.
- The assumption is a relatively good compromise between the different locations of contact force. It has turned out that eccentricities calculated in this way are reasonable.
- The assumption leads to a simple tolerance requirement: "timber beams must have good contact under the nail plate in compressed connection".

There are however some joints with large eccentric contact force, especially heel and apex joints, which need special attention. Our present design methods are liberal in utilizing contact, which may be large, 60...90 % out of the total compressed force. No attention is paid to the location of compressed force and the location of the force which is transferred through the plate. This design method is correct only if the contact force between timber beams and the compressed force of the plate are close to each other. This means that the nail plate must cover practically all the area where contact between timber beams may occur. The model necessitates the use of large nail plates because if too small nail plates are used it very often leads to large eccentricities and stresses.

It must be emphasized that an assumption of no contact is made only in stress analysis. Present nail plates cannot transfer all compressed forces and contact must be utilized in plate design. This procedure may not lead to any essential error if there is good contact under the nail plate and a large gap elsewhere or connection is formed in such a way that essential contact cannot occur outside the nail plate. It must also be required that the nail plate must not be located essentially outside the contact area of timber beams in compressed connections.

4.4 Friction

Calculation of friction requires knowledge of the compression force between timber beams. So friction calculation creates the same problems as contact calculation. Besides, friction has features which make it even more complicated.

Friction becomes essential in joints with large shear force. Connection types behave very differently in shear connection depending on plate and force direction and plate type, e.g.

a) A plate may act like a large number of cords compressing timber beams to each other. In this case essential plate deformation is not necessary. The ultimate shear load may be very high.

b) A plate may deform plastically, usually creating contact and friction between timber members. Deflection and ultimate load are usually large.

c) A plate may lose strength through buckling. This may happen suddenly in the elastic loading phase.

d) A plate may have a small anchorage area compared to shear strength; the plate has large deflection and rotation compared to timber and this dominates the connection. The failure is anchorage failure with small anchorage strength compared to strength measured from a tension test.

Our present nail plate design procedure does not differentiate between cases a-d. It is difficult to make shear tests as cases vary because of different plate sizes and plate directions; a small change in plate size or direction may create a large change in connection behavior (ultimate load and/or deflection). Friction brings a great advantage to connection design because ultimate strength increases essentially, even up to double.

Friction has also disadvantages:

- Friction means compression force between timber beams, which closes the gap which may occur in the connection. This leads to internal stresses in timber. It may be estimated that as much as 20 % of the timber strength may be lost because of this reason.
- If shear force includes a large amount of friction it is not possible to define stiffness of connection (or else calculation of spring value for force and moment becomes very complicated). Connection stiffness is essential when eccentricity is calculated.
- It is found that plate anchorage strength is essentially smaller in shear than tension connection. The reason for this is rotation between plate and timber. Calculation of rotation is essentially simpler if it is assumed that there is no friction (or contact) between connecting timber beams.

When friction is removed from plate shear strength, the design shear values may be decreased to half. In most cases the design value for shear will be decreased only by c. 20 %. When shear forces are completely transferred via steel plate it leads to only a small deviation of strength. It is also possible to use a smaller safety factor for steel than composite steel and friction. All this means that leaving friction out of shear connection will lead to no practical reduction in design shear values. However, there will be an essential reduction in anchorage strength in shear connection compared to tension connection. This reduction happens also if friction is utilized.

4.5 Plate behavior

When nail plate behavior in trussed rafter is investigated the following considerations become relevant:

- linearity - nonlinearity
- elasticity - plasticity
- buckling

Buckling does not create any problem. If a plate buckles it forms a clear design state.

Plasticity is more difficult. Connections usually do not lose strength when plate plasticity limit is reached but may bear even a double load. The reason for this is contact and friction of timber beams.

Plate plasticity has thus great advantage but also disadvantages:
- Plasticity leads to contact, friction and increased deflection
- It is very difficult to calculate connection stiffness and strength. This is important in eccentricity calculation
- It is less justified to assume that timber never fails under the nail plate.

In this design model problems of plate plasticity are ignored; therefore this design model is correct only if design values are calculated from plastic limit.

If plate plasticity is not allowed the plate is elastic and linear. At any rate connection between plate and timber is nonlinear with no clear nonlinearity limit. Nonlinearity depends on stress level.

This design model is based entirely on linear behavior of connections. The calculation is satisfactory if spring values (k_T , k_m) are calculated from the same load level as that from which the calculation is made.

5. DESIGN PROCEDURE

The design consists of stress analysis (which is described in chapter 4) as well as timber and joint design.

The timber design can be done according to timber code, yet it seems useful to include size effect in the design to cut large eccentricity moment peaks.

Timber and joint design are in a development and calibration phase; therefore this topic is not discussed further.

Usually trussed rafter design is carried out by doing analysis work before connection design and before knowing exact timber cross sections and grades. In this respect this design method is different. The exact geometry of timber and joints must be known before analysis because different joints may lead to completely different eccentricites and stresses.

6. COMPARISON WITH TEST RESULTS

It is very difficult to make trussed rafter tests because timber has a large elasticity and strength deviation. If strength is used as reference the number of tests must be large enough to make statistical analysis. It would be much cheaper to find out stresses.

A test method has been developed in which stresses can be determined relatively quickly, cheaply and reliably.

Everything is based on the fact that normal forces can be analyzed reliably with a simple analysis method.

Testing is carried out in the following way:

- A trussed rafter is loaded and strain is measured in the upper and lower cross section edges of the timber beam. These measurements are made in as many cross sections as possible.
- After testing, the trussed rafter is analyzed to find out the normal force in each measured cross section.
- The modulus of elasticity is calculated from normal force and mean strain.
- Cross section moments are calculated in each cross section from modulus of elasticity and upper and lower strain.

When this method is used it is found that deviation in (measured and calculated) moment is small and it can be estimated that total error may be below 15 %.

One tested truss is shown in figure 2 and figure 3 shows the moment distribution calculated from the model. The measured moments are marked with a dot. As a comparison, calculation has been made with some other models /2/. Moment values are shown in figures 4...7. Many other calculation models have been tested. The following conclusions could be drawn:

- Frame analysis must be used if deflection is large.
- Assumption of stiff connection gives better results than pinned connection.
- Connection eccentricities have an essential influence on stresses.

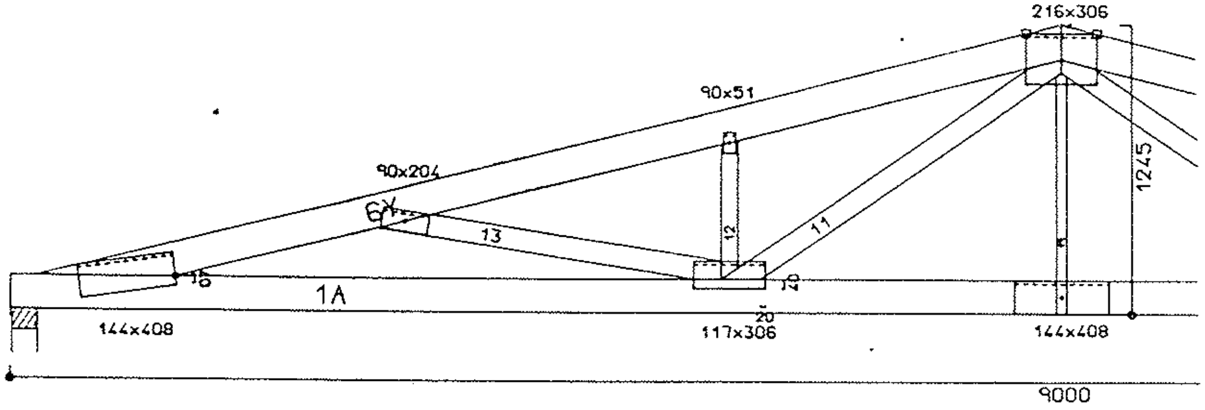


Fig. 2. Test truss.

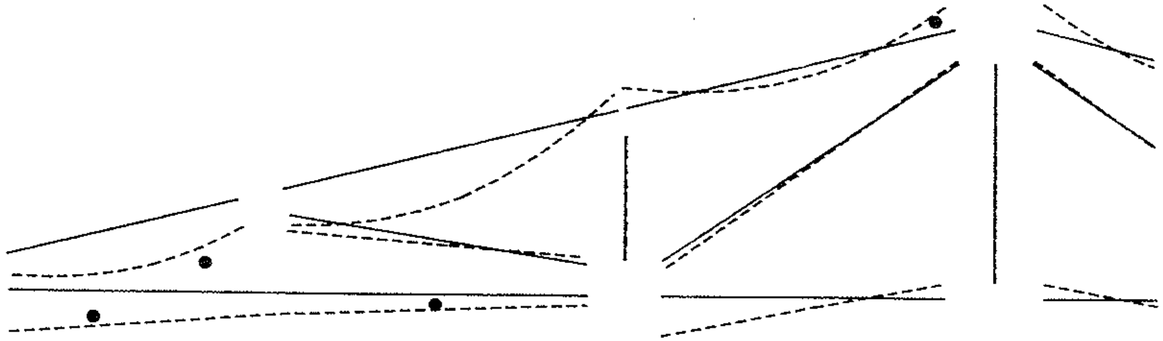


Fig. 3. Moment curve of test truss.

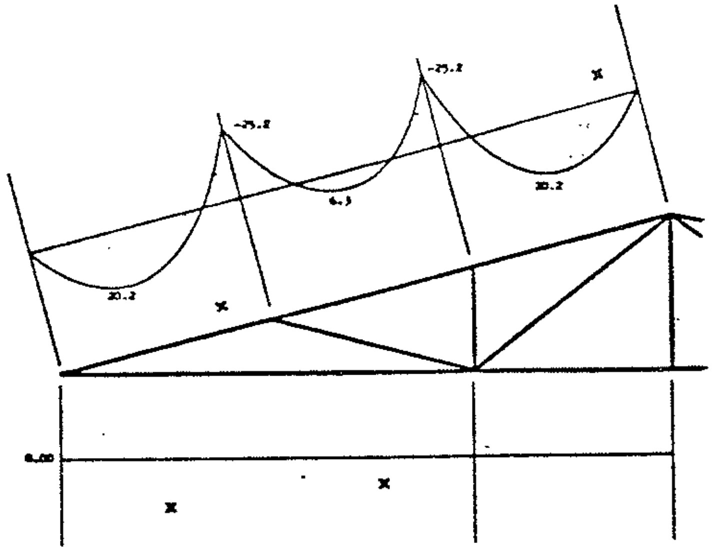


Fig. 4. Pinned connections, continuous beam analysis, eccentricities excluded.

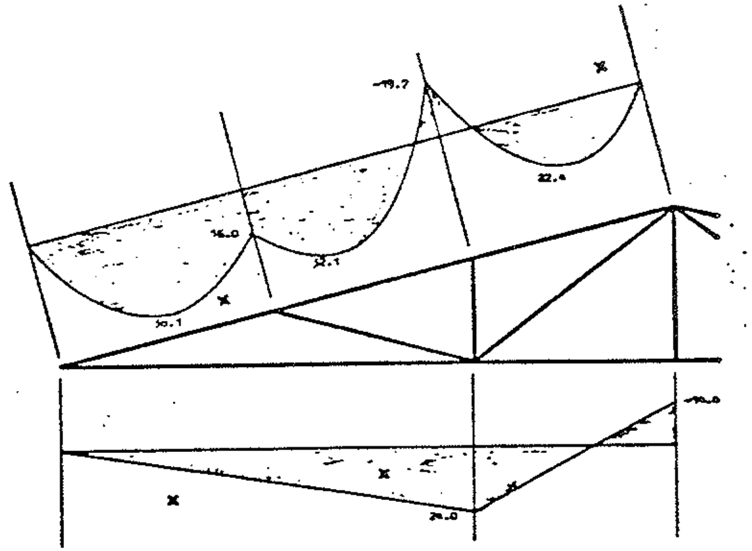


Fig. 5. Pinned connections, frame analysis, eccentricities excluded.

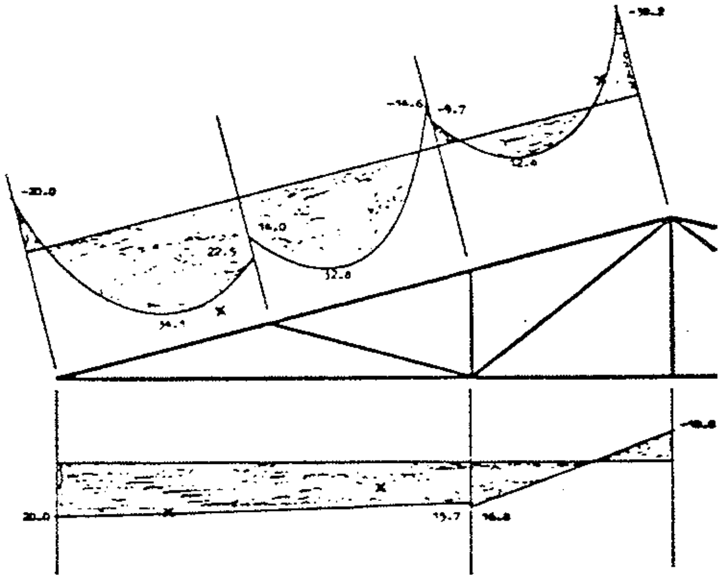


Fig. 6. Stiff connections, frame analysis, eccentricities excluded.

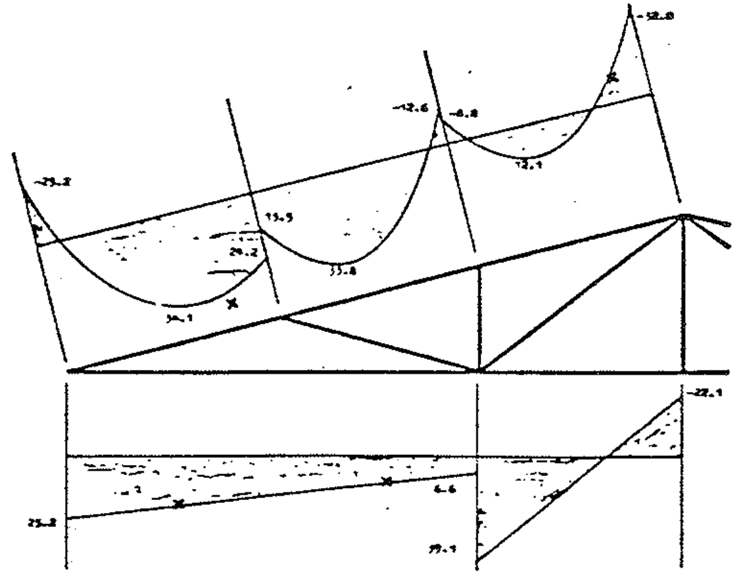


Fig. 7. Stiff connections, frame analysis, eccentricities included appr.

7. DESIGN EXPERIENCE

7.1 General

The model has been used since autumn 1984, and there is experience of c. 1500 designs.

The model requires a large computer and the design procedure is much more laborious than many other design methods because the exact geometry must be known before analysis.

The model has however proved useful mainly for two reasons:

- It gives economical structures. These can be achieved by connection eccentricity, which can be utilized to get balanced moment distributions.
- The model does not make any practical restrictions to connection or rafter configuration. This has made it possible to introduce completely new rafter types.

7.2 Eccentricity

According to the model, connection eccentricity is very important in trussed rafter design. Eccentricity moment may be far larger than e.g. maximum moment calculated from continuous beam theory. As a result eccentricity may completely dominate moment calculation.

Eccentricity calculation of normal K and E joints has earlier been done in the following way: Eccentricity has been assumed to be either half a chord in depth or eccentricity has been calculated from diagonal and vertical center lines. In both cases the eccentricity moment is shared by chord beams only because diagonals and verticals are assumed to have pinned connections.

The model is different in this respect. Calculation made with the model shows the following things:

a) If it is assumed that eccentricity is half a chord in depth, difference from the model may be large (though, usually on the safe side).

b) Eccentricity cannot be calculated from the center lines of members. If center lines pass through the same point, meaning no eccentricity in center line calculation, the model may show large eccentricity. If center lines of diagonals and verticals are changed but the nail plates with anchorage areas remain unchanged, there is no practical change in chord moments (but an essential change in diagonal and vertical eccentricity moments).

c) If timber members (and center lines) remain unchanged but the nail plate is changed there may be an essential change in eccentricity. In practice this change is not large because plate changes must be made below strength limits. Anyhow it is possible to use larger plates than necessary for connection strength to avoid large timber stresses. Though plate costs are increased, total costs may be decreased, due to timber or labor saving.

d) Diagonals, verticals and wedges (members with composite beams) may have large eccentricity moments which may have the same size as maximum moments on chords.

The model shows that the strength capacity of diagonals and verticals can be utilized to carry eccentricity moments, which means small eccentricity moments on chords and economical designs.

7.3 New rafter models

There are three major features in the model which make it possible to analyze new rafter types: nonlinear analysis of buckling, careful calculation of connection deflection and transferring moment via nail plate. Figures 8-11 show four examples.

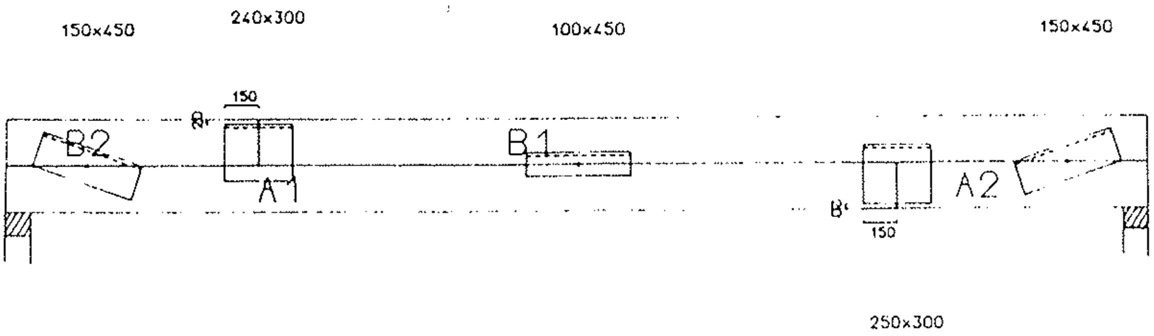


Fig. 8.
Composite beam for heavy loads, span 4...6 m.

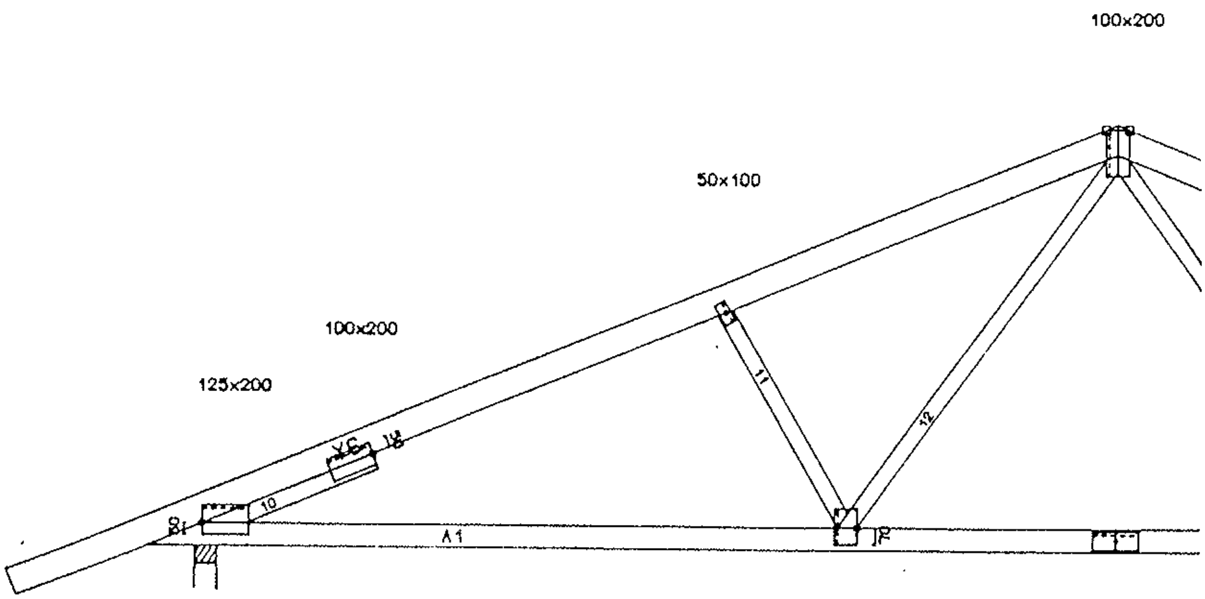


Fig. 9.
W-trussed rafter with large support eccentricity; large stresses can be eliminated with long wedge.

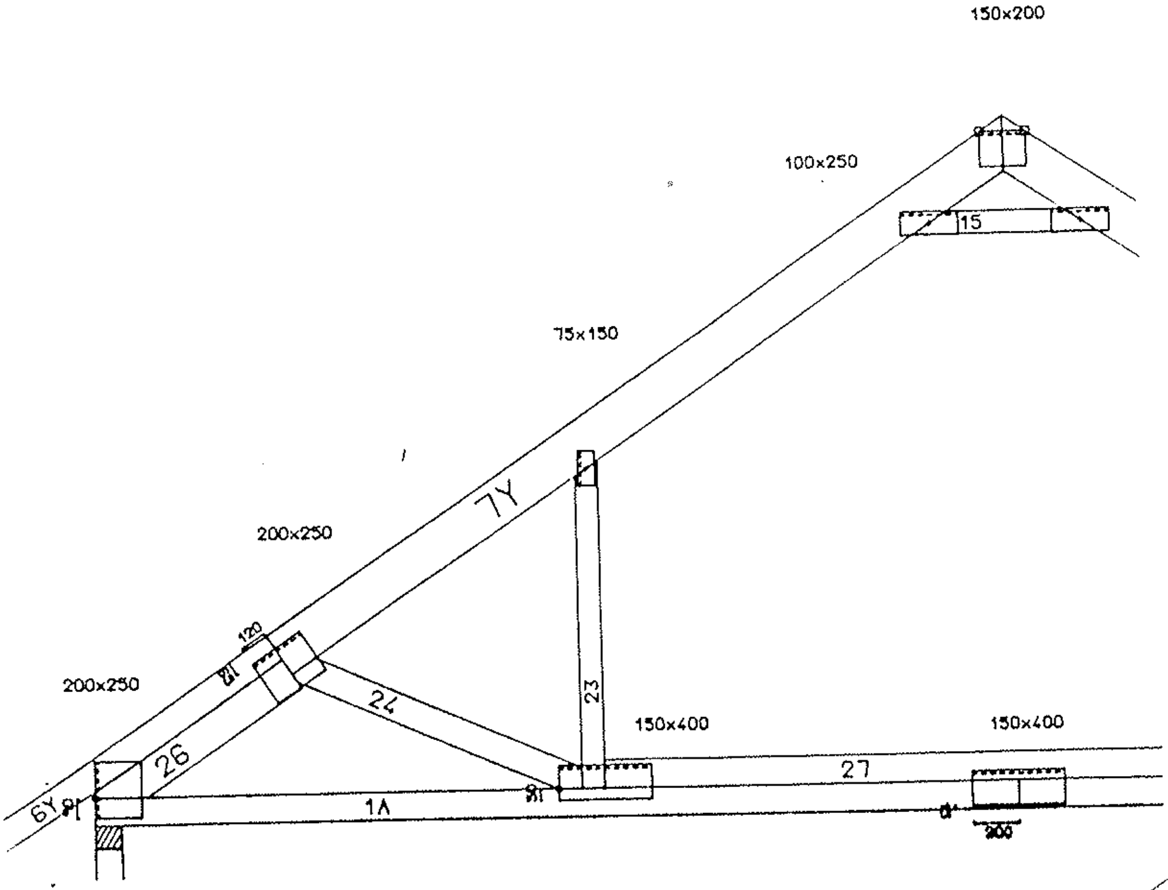


Fig. 10.

Attic frame with no internal bearing wall.

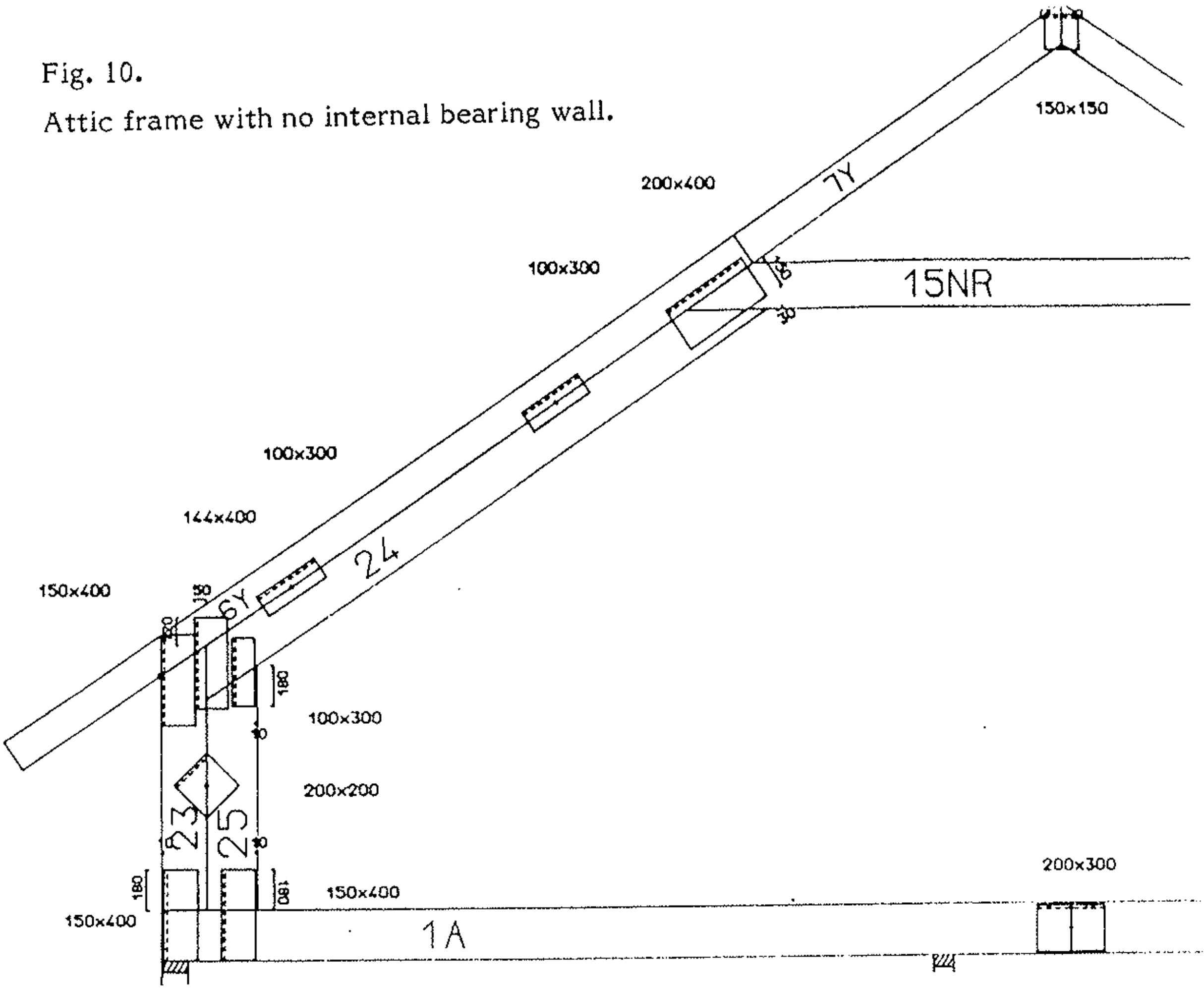


Fig. 11.

Attic frame with good space economy and large connection moments.

8. FURTHER DEVELOPMENTS

As stated before, connection and timber design is still in a development and calibration phase. In timber design there are two problems:

a) Eccentricity dominates moment calculation which means high stress peaks. The present timber code does not allow reduction of these stress peaks. The author believes that these stress peaks should be cut off by size (or "stress wideness") factor.

b) The model is based on physically linear analysis. Yet it is known that timber has nonlinear behavior in compression. It would seem that physically nonlinear analysis should be used. (Connections are also physically nonlinear, though it is not possible to consider this so far due to insufficient knowledge of connection behavior and its great complexity).

Connection design has also two major problems:

a) Present nail plates cannot transfer all compression forces which occur in connections. As a result, contact must be utilized, which means that assumptions about size, location and stiffness of contact must be made. In stress analysis a clear assumption is made about these factors. However, in plate design other assumptions must be made. A good assumption might be the following: Contact force is max. 60...80 % of the total compressed force, it is located in the center of the contact area perpendicular to it and contact stiffness is ignored.

   Another assumption is the following:

   - It is assumed that connecting timber beams have a constant gap (0.3...0.5 mm) in the unloaded stage
   - A check is made to see if timber beams get into each other in the loaded stage and if so a contact force is created. The size and location of this force is calculated from linear surface stiffness, which depends upon timber grade and grain direction.

   This assumption leads to an iterative process but it can be included in the nonlinear analysis cycles and it does not essentially increase the calculation time. Some tests of this calculation method have been made and it seems to work well. It also seems that friction can be considered by this means.

b) Present nail plate tests are carried out in a way which is not completely suitable for the model. The following changes should be introduced:

- Design values should be defined from linear state. Range after plate plasticity should not be utilized.
- Tests should be carried out without contact and friction. A large enough gap (c. 4 mm) should be used in shear tests or friction should be removed by other means.
- Tests to find out rotation stiffness should be added.

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CIB-W18/18-15-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

FULL-SCALE STRUCTURES IN GLUED LAMINATED TIMBER, DYNAMIC TESTS: THEORETICAL AND EXPERIMENTAL STUDIES

by

A Ceccotti and A Vignoli
University of Florence
Italy

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

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A. CECCOTTI, A. VIGNOLI
Department of Civil Engineering, University of Florence,
Via di S. Marta, 3; I 50139 Florence, ITALY.

FULL - SCALE STRUCTURES IN GLUED LAMINATED TIMBER, DYNAMIC TESTS: THEORETICAL AND EXPERIMENTAL STUDIES.

Abstract

This papers presents the results obtained by application of a particular methodology of dynamic tests being carried out on full-scale assembled timber structure, analysing in particular the theoretical approach and the importance of results obtainable from experiments.

1. Introduction

1.1 Factory control

In Italy quality control on glued-laminated timber is usually conducted by cecking single structural elements while they are being manufactured in factories. This involves first of all the quality control of machinery and staff training, followed by the checking of glueings in the finger joints (bending tests) and between one lamina and another (shear tests) according to statistics.

1.2. Assembly Field control

Checks in the finished structure are left to the discretion of the "Direttore dei lavori" (Chief Resident Engineer),firstly, and the "collaudatore"(Testing Engineer), secondly. They have the right to carry out tests that they consider suitable both before assembly as well as after assembly in order to check the quality of the materials themselves and to check the structural diagram provided by the designer.

Without entering upon the various problematics that appear at this stage (cfr. (1)), from now on we would like to concentrate on the particular aspects of loading tests in structures after assembly, proposing an agile dynamic variant to them.

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2. General

2.1 Dynamic tests on structures (free vibrations)

In the field of structures already assembled, be they in steel, reinforced concrete or masonry (2, 3) "dynamic" testing methods have already started making themselves known.

Just to give a broad outline, these methods consist of exciting the structure's free vibrations, detecting the relative frequencies by means of accelerometers or other instruments capable of detecting vibrations more generically, and comparing results to those predicted according to a mathematic model previously prepared.

2.2 Application field

These tests are not only appicable to structures which have to support dynamic actions during their life span, but also to every type of structure as they enable one to obtain experimental indications useful on the hand in finding a more precise theoretical model of the structure, and on the other in finding principal "elastic" parameters of the material. All this is done through general information, that is not limited to a few sections and which is obtained through tests that are not expensive owing to loading means and times even if, on the contrary, they require sophisticated experimental survey equipment.

2.3 The case of wooden structures

The present paper's writers think it is not unsuitable to start experimenting this testing methodology on wooden structures, especially if one considers that this method requires excitations lasting only for few seconds; it enables one to derive indications that are not affected by time as in the case of static loading.

Moreover, the static loading due to their own nature, in the case of assembled structures of a certain dimension and structural complexity, can't be easily codified and lead to results that are anyway affected by creep phenomena in the wood, which are not negligible right from the tests beginning unlike other building materials such as steel and concrete, for example.

2.4 Other dynamic tests

Other dynamic methods like the resonance or ultrasonic tests which have already come into use and which can be used for the stress-grading of sawn pieces of wood are unsuitable for large structures already being assembled whether due to the very nature of the test (in the case of the resonance test, it can be carried out easily in a laboratory on specimens of reduced dimensions but it can become very problematic on assembled structures (1)) or due to the power of the necessary equipment which in the case of ultrasounds should be much higher than that used at present for stress-grading.

The sonic method is easier to use (4) but it enables one to obtain usefull indications of single structural elements but not of the whole structure taken as a whole.

2.5 Subject of the present paper

The present work illustrates an example of application of the proposed methodology just at it was used during testing carried out on a structure in glued-laminated timber with a span of 40 m.

3. <u>Description of the structure that was the subject of research</u>

The structure (fig. 1) comprises a series of 16 three-hinged arches in glued-laminated timber of common pruce, placed at 5 m from one to the other with a span of 40 m (central "aisle"). On the sides are two 12 m train sheds in continuity with the arches and supported by columns also in glue-laminated timber (side "aisles"). The cross sections of the arches is rectangular with a base of 14, 5 cm and depth of 70 to 120 cm.

The arches are hinged at the springer on slender columns in reinforced concrete which discharge on foundation plinths connected on the arches plane, by means of a steel tie bar, and longitudinally, by means of a reinforced concrete girder.

The windbracing is formed by purlins in solid wood and tie bars in steel bolted at the arches. The secondary structure is formed by purlins in solid wood nailed at the arches. The roofing is in corrugated asbestos-cement sheets.

4. Experimental surveys and instrumentations

The strutcure has been submitted to numerous tests to ceck the behaviour both in the static as well as the dynamic fields. For the static tests refer to (5). During tests the average moisture content of the timber measured with electrical methods was about 12%; before tests the metal wind-brace had been strained.

4.1 Testing methods

Free vibrations were carried out both in a vertical direction on arch plane, as well as in an approximately horizontal direction parallel to the longitudinal axis of the construction (fig. 2a).

The first were obtained by hanging a weight of 900 daN and roughly cutting a connection specially prepared. The test was repeated both at the crown and at a quarter of the span (fig. 2b).

The oscillations in the longitudinal direction (that is orthogonal on the arches plane) has been primed by means of the contem-poraneous release of three cables anchored at 50 m from the arch and strained by means of mechanical jack until they transmit to the quarters and crown of the 2nd arch three actions of about 1000 daN each (fig. 2c).

4.2 Experimental measurements

During oscillations accelerations have been measured in five points, of which three are localized on the arch and two at the top of each column of the side aisles, and displacement of one of the two reinforced concrete columns, that support the springer hinges of the arches.

Some of the most significant results are summed up in figures 3 and 4:

in the vertical dynamic tests with excitations at 1/4 of the span, the frequencies detected in several measuring points have the same value, the "mode" of vibration excited a frequency of about 2,4 Hz;

in the horizontal dynamic tests, the frequencies detected in several measuring points don't have the same value, the structure oscillated but not according to a particular "mode".

5. Numerical modeling

5.1 Structural shape

For the numerical study of the structure, a model was built to the finite elements of the whole structure including 16 arches connected by purlins and windbracing cross, with the two side aisles and reinforced concrete columns which carry the springer hinges of the arches.

5.2 Elastic parameters

As reference values for the wood's elastic parameters, higher values than those usually found in static stress are adopted, namely: for the dynamic modulus Ed = 16500 N/mm2, and for the corresponding tangential modulus Gd = 1/15 Ed (6).
As the average value of the wood's density mass we took ρ = 500 kg/m3.

5.3 Automatic calculation code

The calculation of the quantities necessary for a comparison with experimental results was developed by using the automatic SAP IV code which enables both static as well as dynamic calculation in a linear field.

5.4 Modal analysis of the whole structure

In the dynamic case, the modal dynamic analysis, up to the 12th mode, of the whole shed has been carried out. On examining the relative deformed shapes (in fig. 5 are shown those relative to the first 6 modes), it could be deducted that the various modal shapes of entire shed can always be traced back to the "joining" of the modal shapes of a double windbraced arch. Note amongst other things that the shed can be considered as a group of many double wind-braced arches homogenously distributed in alignment and which are merely connected to the secondary purlins.

5.5 Choice of referring model (partial)

It was observed moreover, in the case of dynamic vertical tests, that, anyway, certainly in order to excite one of the modal shapes involving the entire shed it would take a distribution of impulses and a quantity of energy much greater than those transmitted acting with a single impulse at one point in only one arch, (impulse amongst other things certainly quickly damped in a longitudinal direction because of the high capacity of dissipating energy of the connections nailed of the purlins).
On the basis of these considerations, we thought it not incorrect to limit ourselves to the simpler theoretic model (and with easier computational manipulation) than the double wind-braced arch.

5.6 Influence of the connections

At this point, we tried to quantify the influence on the model of the various limit conditions of rigidity, within which certain connections between structural elements could find themselves, in terms of modal shapes and relative frequencies.

5.7. Single arch connections

We examined the cases in which the junction element between the arch and train sheds of the side aisles was restrained (case a) or hinged at the ends (case b); the support column of the train shed has always been considered hinged at the ends.

5.8 Connections between one arch and another

The system of wind-brace purlins was taken as hinged (case c) as well as restrained at the ends (case d). Moreover, we considered both the case of the presence of the purlins of the wind-brace system (case e) on their own as well as the presence of secondary purlins (using an oversize section equivalent for the wind-brace purlins: case f).
We did not consider the presence of the corrugated asbestos-cement sheets both due to assembly modalities (through rubberized bearings) as well as due to their negligible relative rigidity as compared to other elements: the beams and purlins, depending on the direction considered (the direction of the greatest rigidity being placed along the pitch of the roof).

6. <u>Numeric results</u>

6.1 Connection rigidity

Finally, we had maximum deviations in the values of frequency in the order of 1%, depending on the various combinations examined of the aforementioned cases.
One can therefore reach the conclusion that for a structure such as one being examined, with a static scheme which is already basically clear and considerably stiffened by wind-braces the rigidity of certain connections has little influence.

6.2 The chosen case: the first modal shapes

As reference for the comparison with experimental results we chose the case b) + c) + e) of which the first six modal shapes and relative frequencies are shown in fig. 6a).

6.3 Comparison with the whole model

It was then observed that there is, as further confirmation of the correctness of the choice of the partial structural arrangement, a substantial coincidence not only in certain modal shapes but also in the relative values of the corresponding frequencies themselves: the shape of the 5th vibrating mode of the entire shed is similar to that of the 3rd vibrating mode of the double wind-brace arch with a frequency of 2,43 Hz as opposed to 2.39 Hz (Δ = 1,7%).

7. Comparison with experimental results

7.1 A vibration mode was excited

On examining the experimental results, it turns out that while in the case of horizontal and vertical at the crown stress there is no real vibrating "mode" of the structure itself, in the case of vertical stress at the quarter of the span, there is an evident 3rd "mode" of vibrating if one consider that because of the load's assymetries (applied only to an arch) one must find a phase shift in the way of vibrating of the two arches.

7.2 Comparison between theoretical and experimental frequencies

The corresponding experimental frequency turned out to be 2,40 Hz as opposed to a calculated value of 2,39 Hz. Even within the limits of experimental survey approximations and theoretical scheme, this would appear to confirm after all the choice of the value of dynamic modulus of the timber Ed = 16500N/mm2

8. Observations on the method's accuracy

8.1 Variation of the referring frequency with the varying of the size of masses and the mechanical characteristics of materials.

For the aforementioned static scheme of reference, we tasted the variability, in terms of frequency, of the modal shapes primed with dynamic tests, doubly assymetric (cfr. par. 7.1) with the varying of the timber's density mass (evidently not directly measurable on elements already assembled) and the timber's dynamic modulus.

The diagram of fig. 7 synthetically shows the results of this survey.

8.2 Method's accuracy

It can be observed that in order to determine exactly the timber's dynamic modulus (apart from the obvious consideration that a correct valuation of the dead-loads during testing is very important), as the dead-loads increase in proportion to the structure's mass, an error in the valuation of the timber's density mass loses importance. (*)

Finally, considering the density mass of the timber included between 450 and 550 Kg/m3 (cfr. fig. 7), corresponding to the value of experimental frequency (2,40 Hz), it can be observed that Ed can oscillate between 14000 and 19000 N/mm2; therefore Ed = 16500 ± 15%N/mm2. This accuracy when determining Ed is good in order to valuate the resistance characteristics required by Test-Engineer (7).

Fig. 8 synoptically shows all the frequency values relative to the first 6 modes of vibrating for the various values of Ed and ρ studied. Fig. 6 b) and c) also shows the modal shapes relative to the cases Ed = 11000N/mm2 ρ = 500 Kg/m3 and Ed = 22000N/mm2 ρ = 500Kg/m3.
Note that for Ed = 11000N/mm2 the modal shape similar to the experimental one is no longer the 3rd but becomes the 4th.

8.3 Observations on Ed value

Finally, it must be observed that the value of elasticity modulus obtained in this way is an average value which gives an "overall" idea of timber. According to the writers, since it is a real structure, this is a much more significant indication than that supplied by the knowledge, which might be more precise but is only local, of a specifc value of the modulus (traction, compression or other).

9. Conclusions

Also for timber structures, the dynamic testing methods with the excitation of free oscllations, already applied successfully to structures in other materials, seem worthy of being refined through further theoretical-experimental studies, due to their positive contribution when checking structures already assembled in terms of results validity and in terms of reduction of test means.

(*) In the case being examined, the incidence of loads carried was roughly equal to the weight of the timber (purlin included), therefore an error of 10% on the density mass of the timber would lead to an error, in the total of the masses, of the half, that is just 5%.

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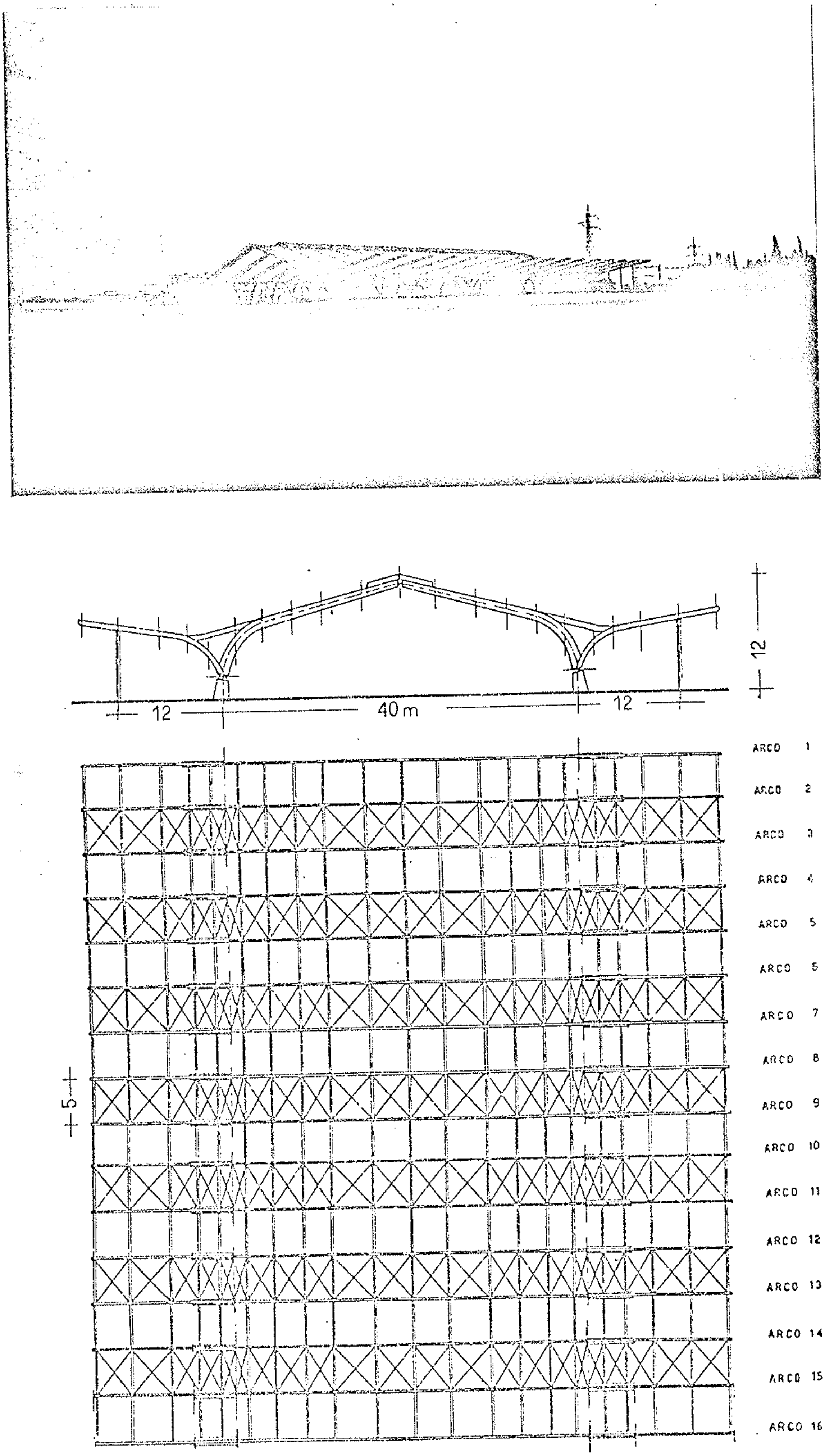
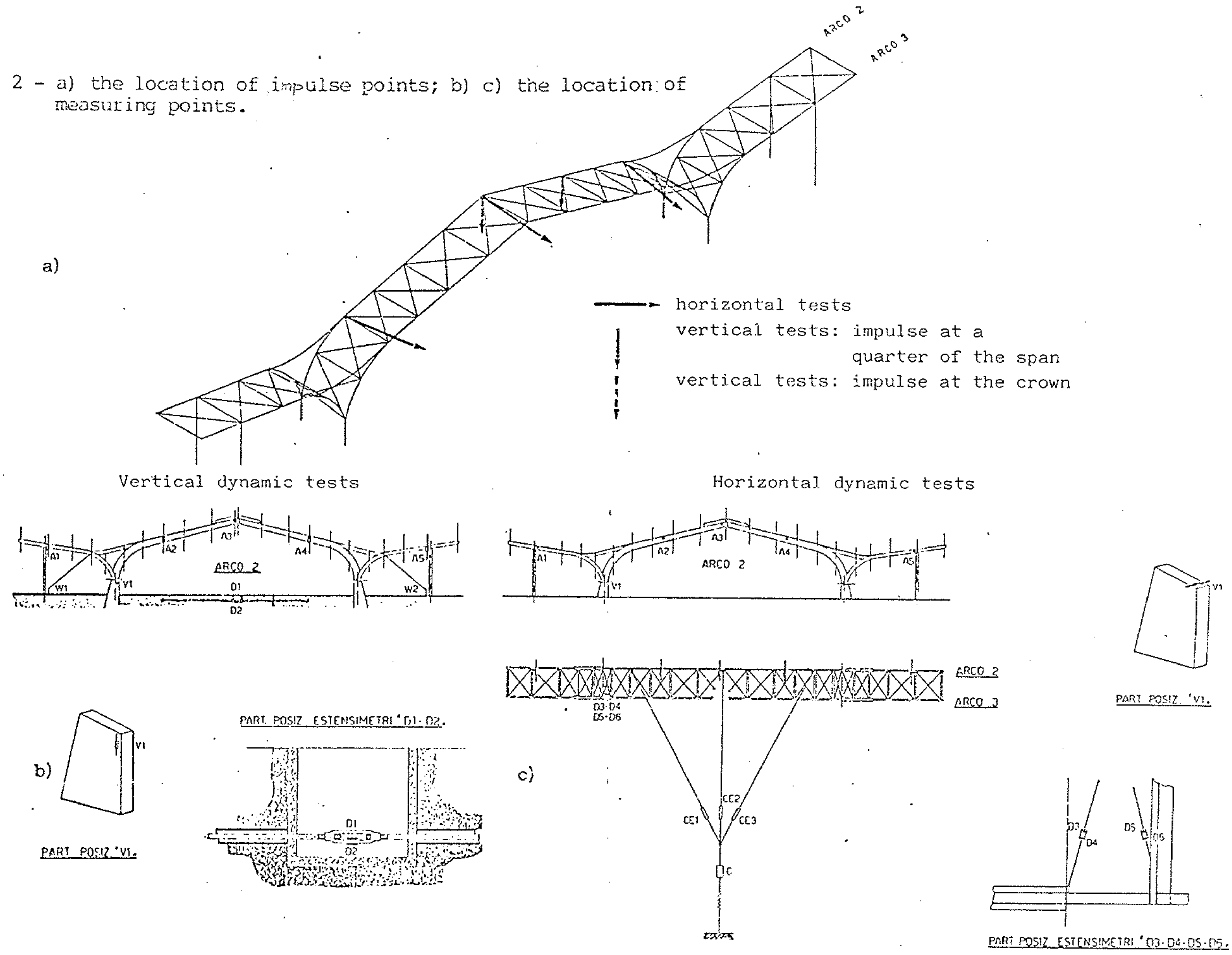


Fig. 1 - The tested structure.

Fig. 2 - a) the location of impulse points; b) c) the location of measuring points.



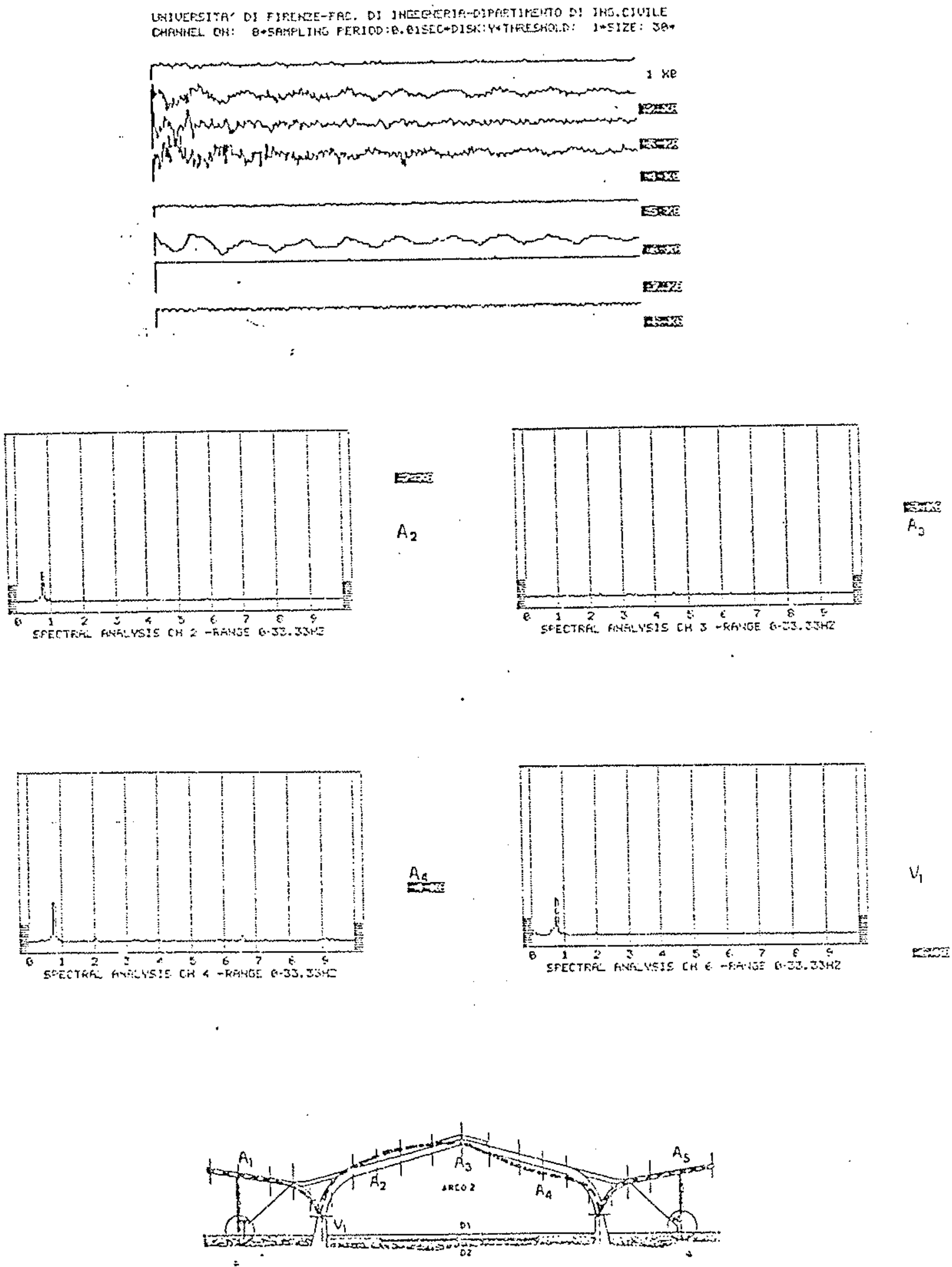


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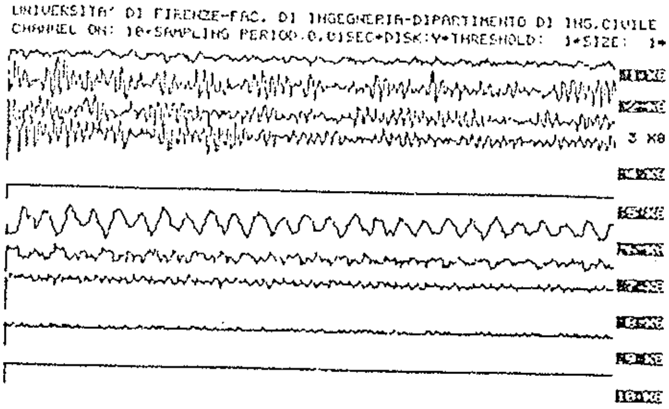
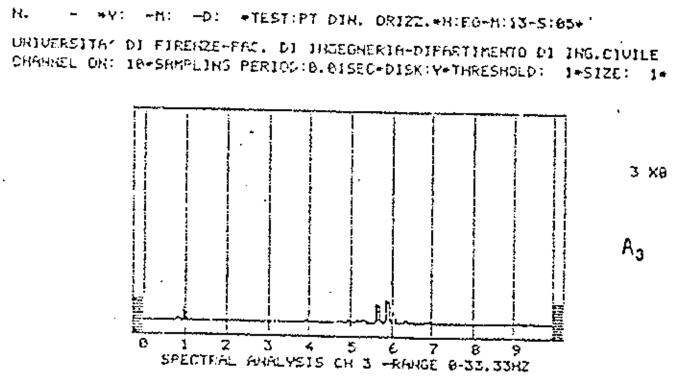
Fig. 3 - Vertical tests: impulse of a quarter of the span. The frequencies detected in several measuring points have the same value: the excited "mode" of vibration has a frequency of about 2,4 Hz; the modal shape is similar to the 3rd theoretical mode.





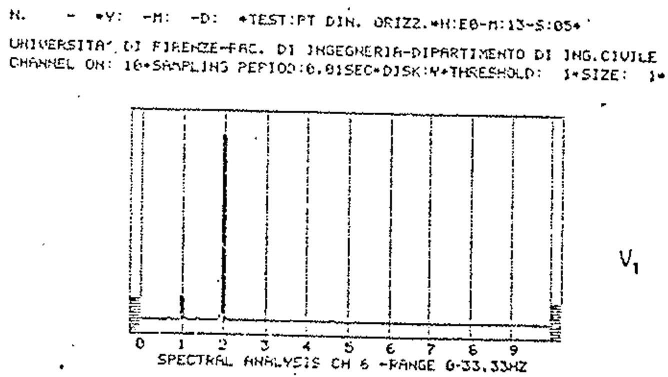


Fig. 4 - Horizontal tests. The frequencies detected in several measuring points don't have the same value: the structure was osciliated but not according to a particular mode. The highest values of frequencies, in the middle diagram, are probably caused by local vibrations.

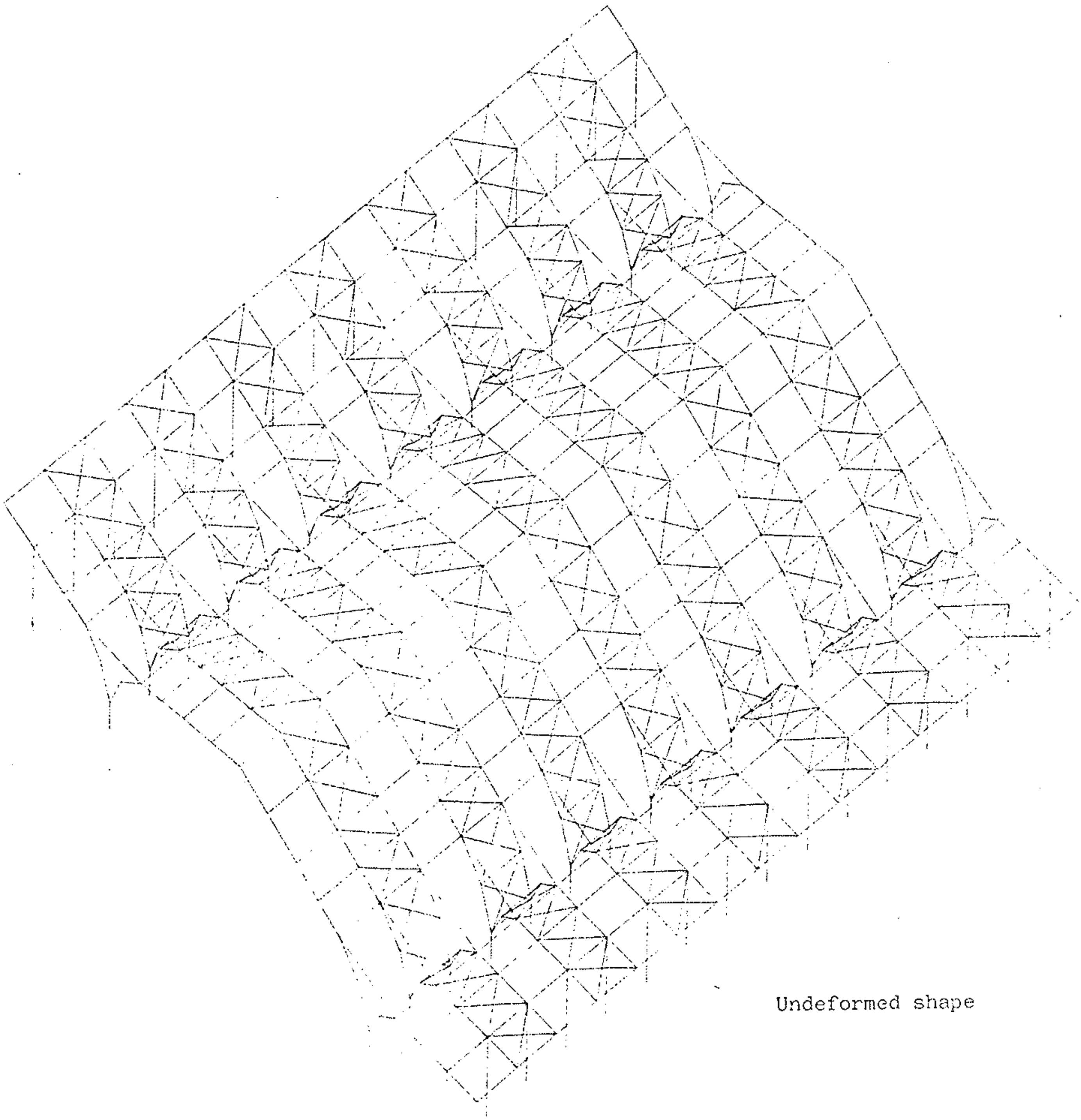
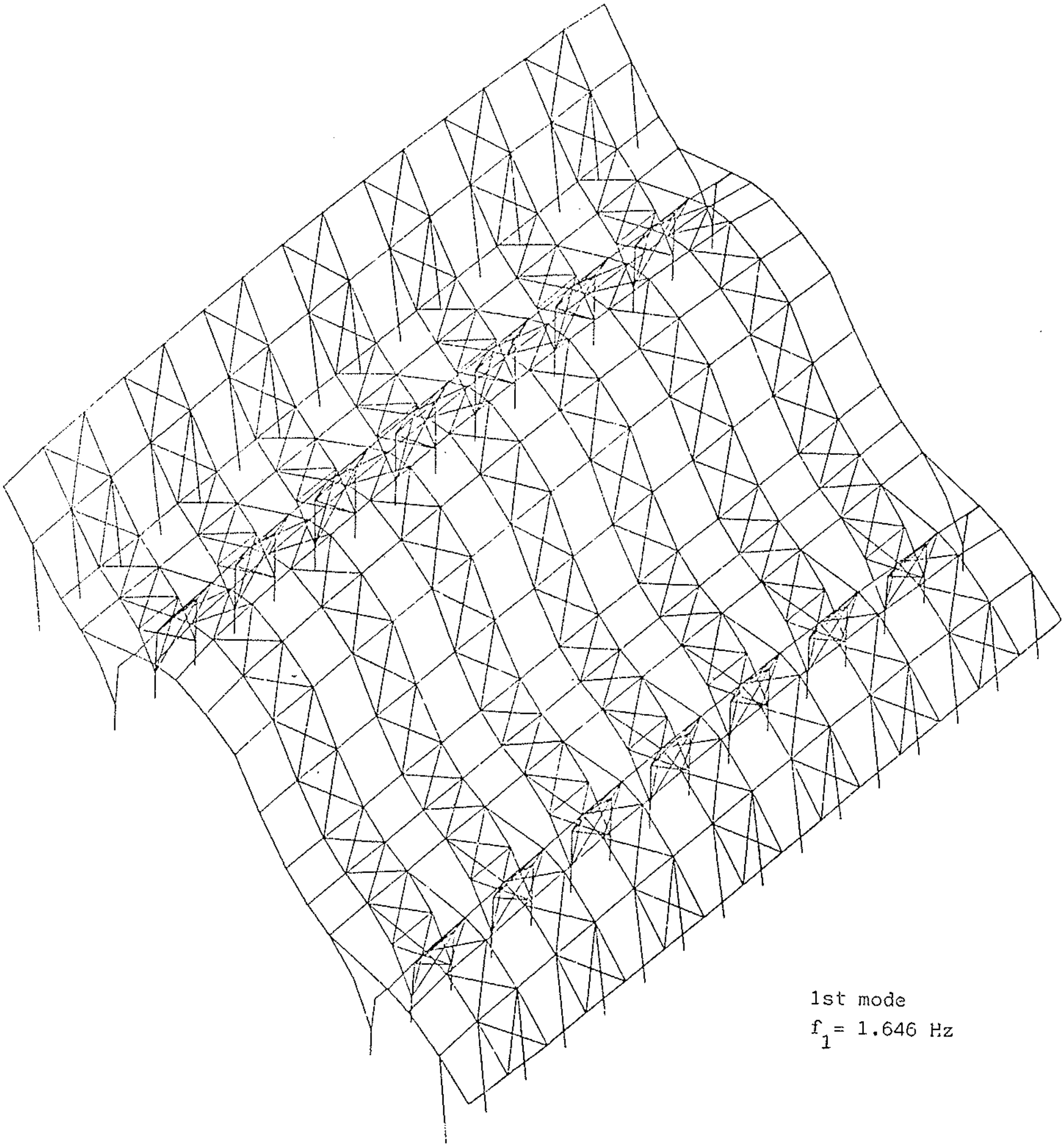


Fig. 5 - Whole structure: the first six modal shapes and their frequencies.



1st mode
$f_1 = 1.646$ Hz

Fig.5 (cont.)

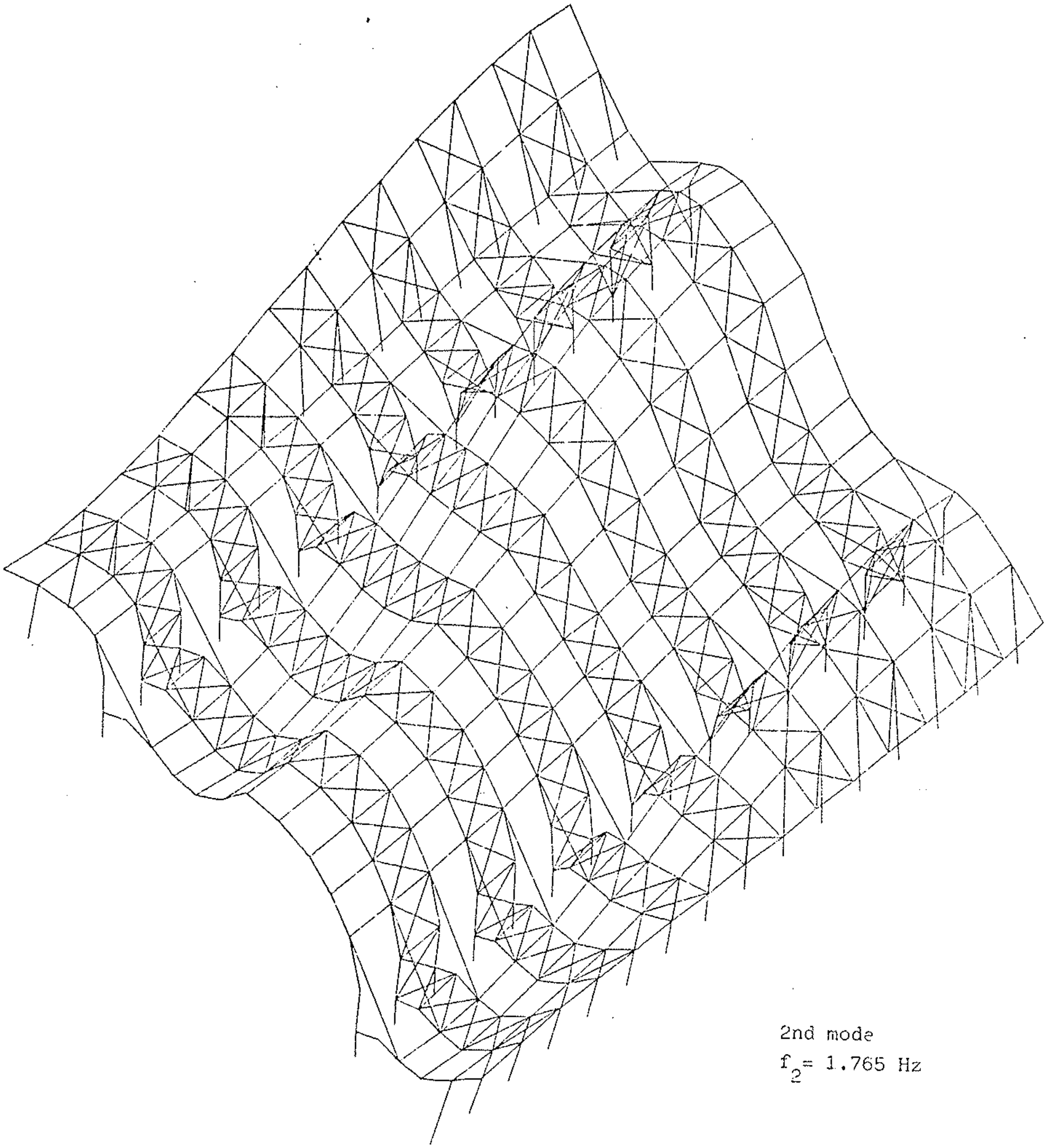


Fig.5 (cont.)

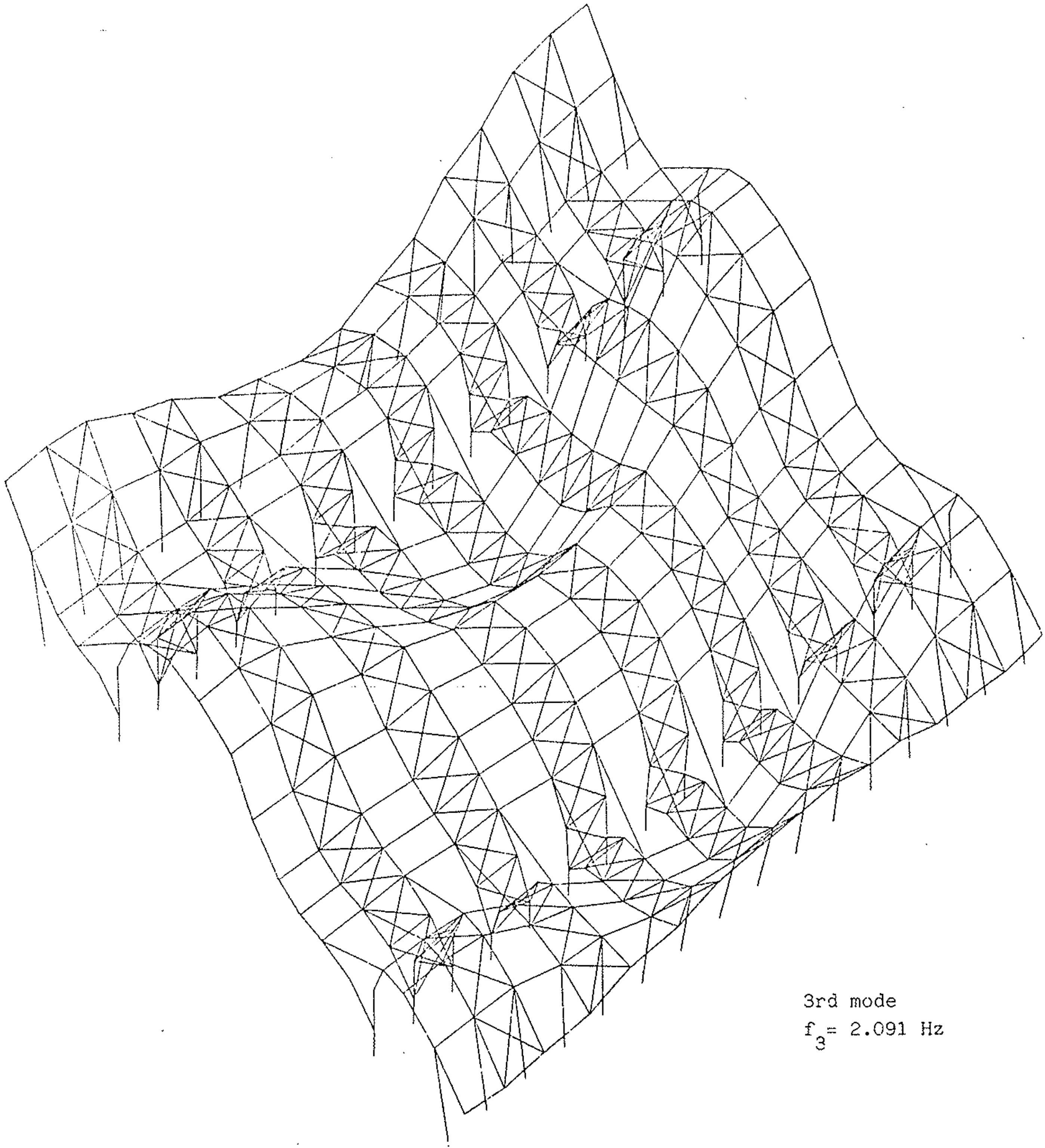


Fig.5 (cont.)

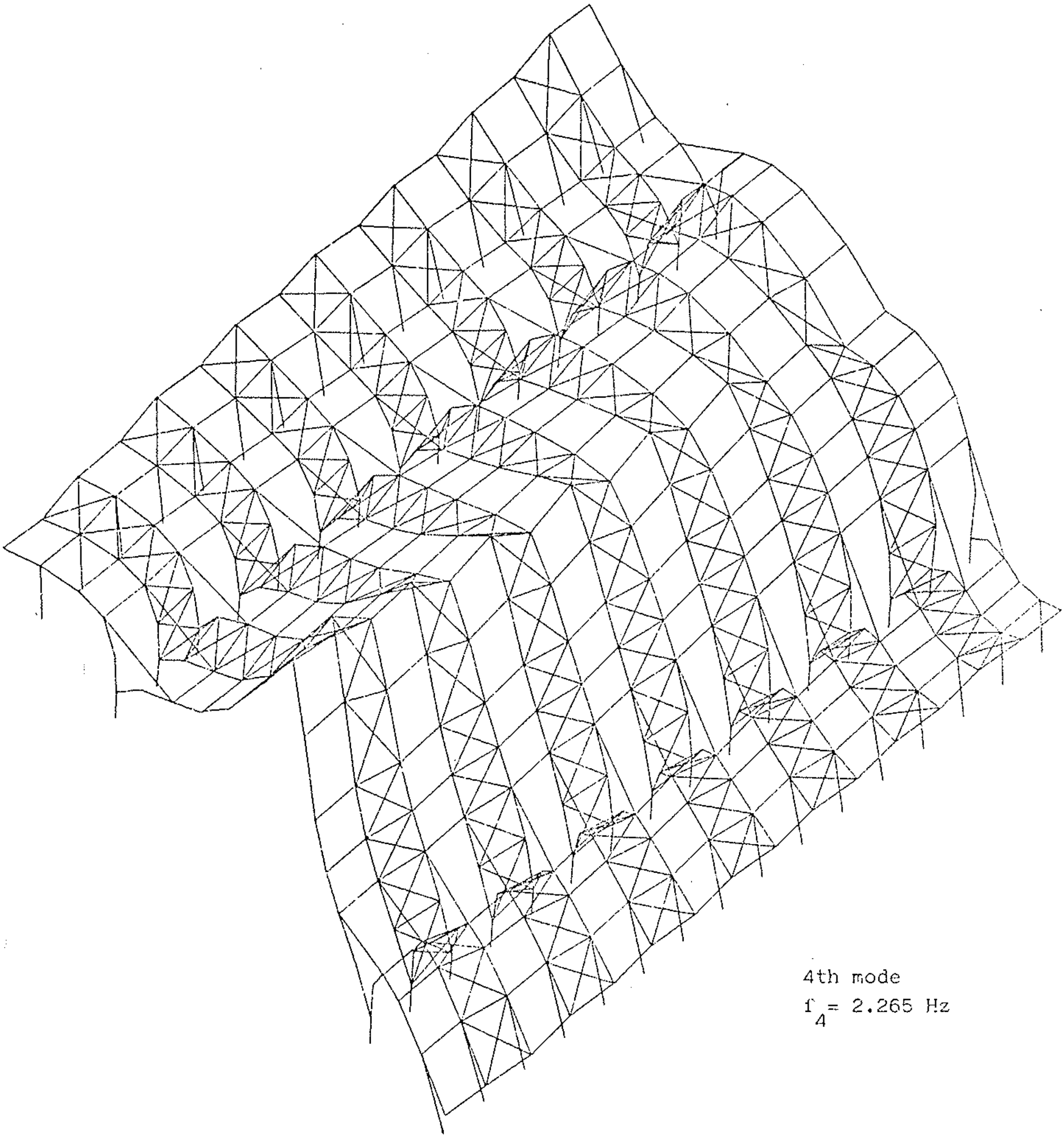
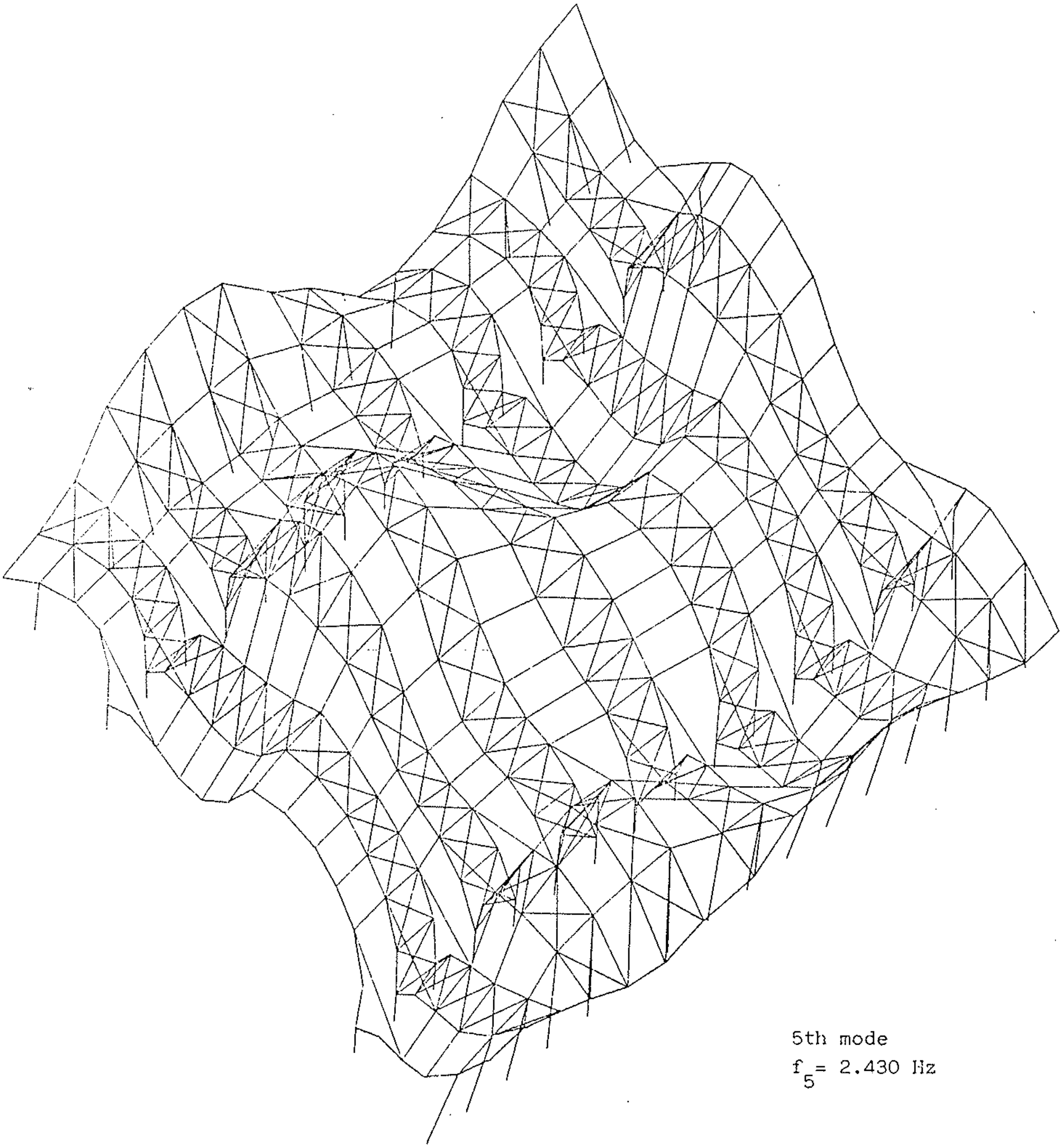


Fig.5 (cont.)



5th mode
$f_5 = 2.430$ Hz

Fig.5 (cont.)

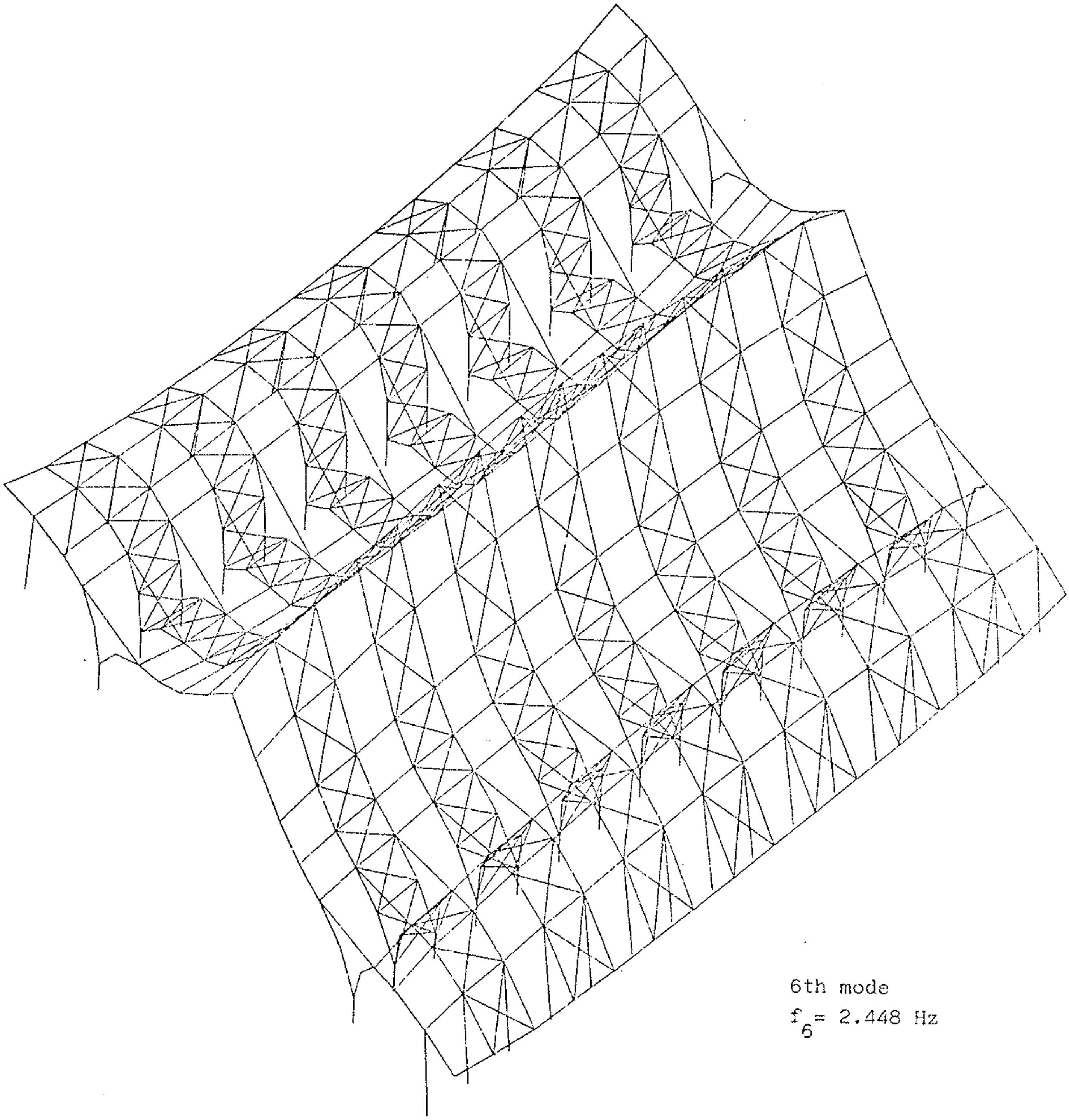


Fig.5 (cont.)

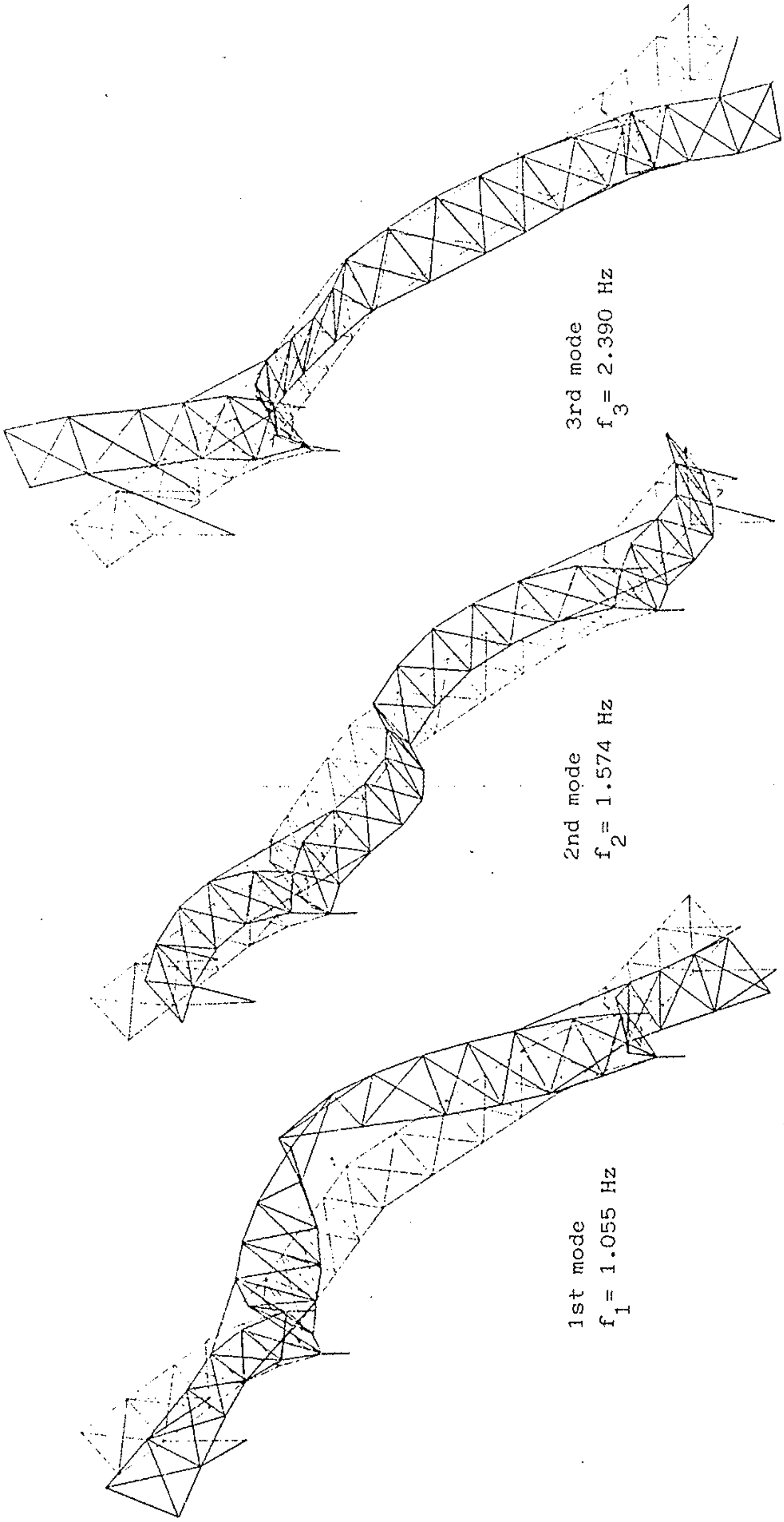


Fig.6a

Fig. 6 - Double braced arch: the first six modal shapes and their frequencies:

a) Ed = 16500 N/mm2 ρ = 500 Kg/m3
b) Ed = 11000 N/mm2 ρ = 500 Kg/m3
c) Ed = 22000 N/mm2 ρ = 500 Kg/m3

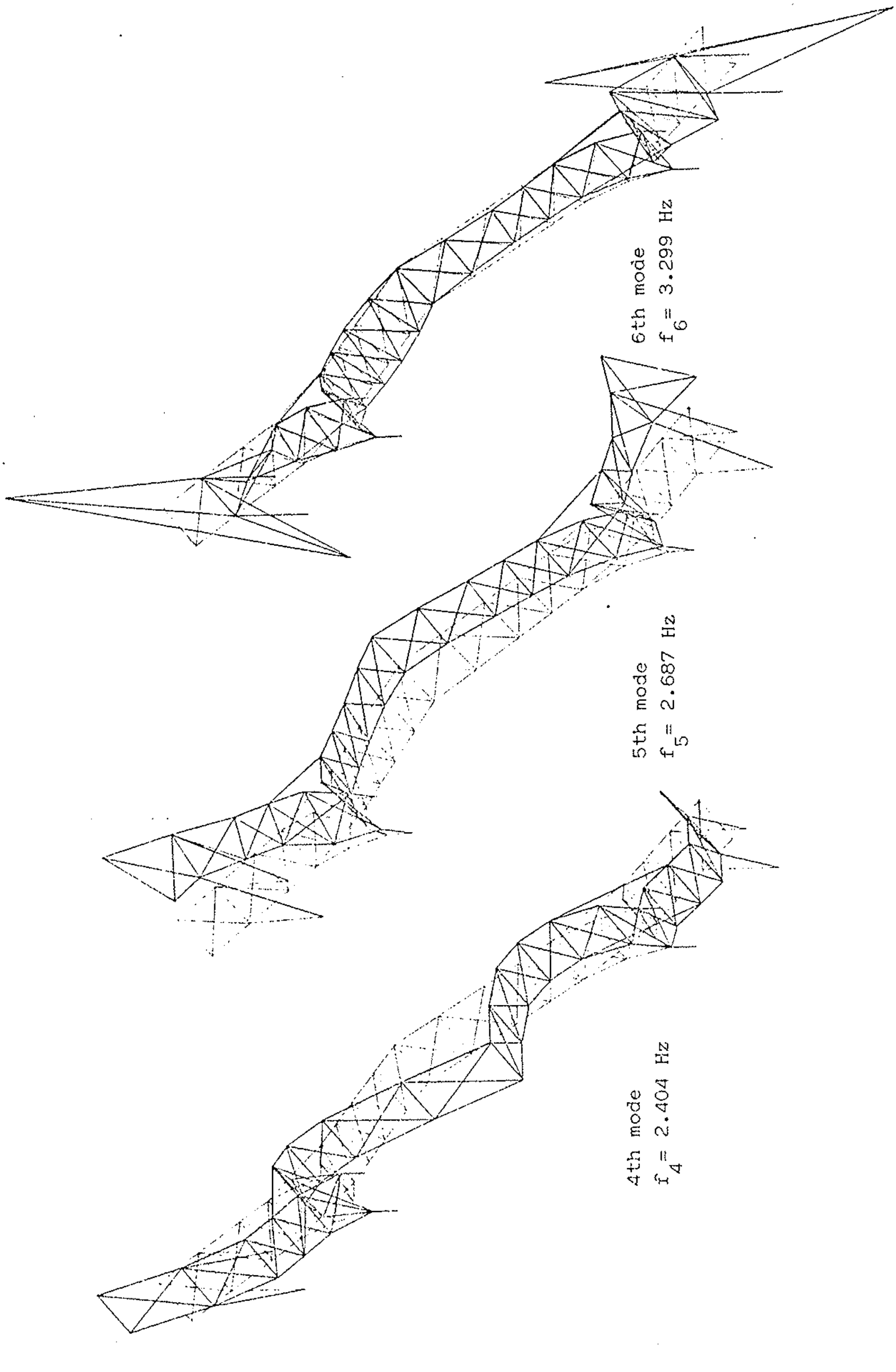


Fig.6a(cont.)

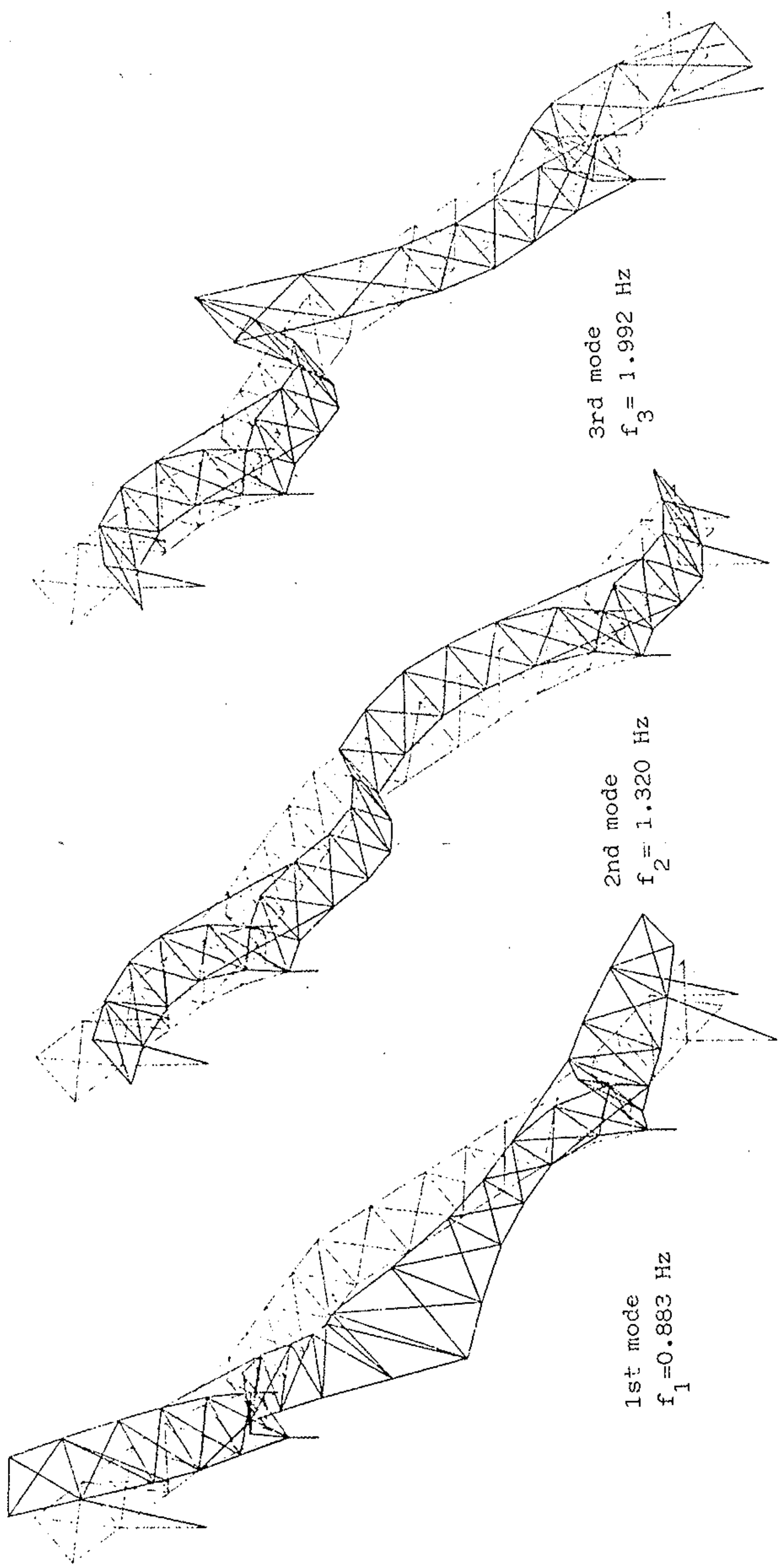


Fig.6b

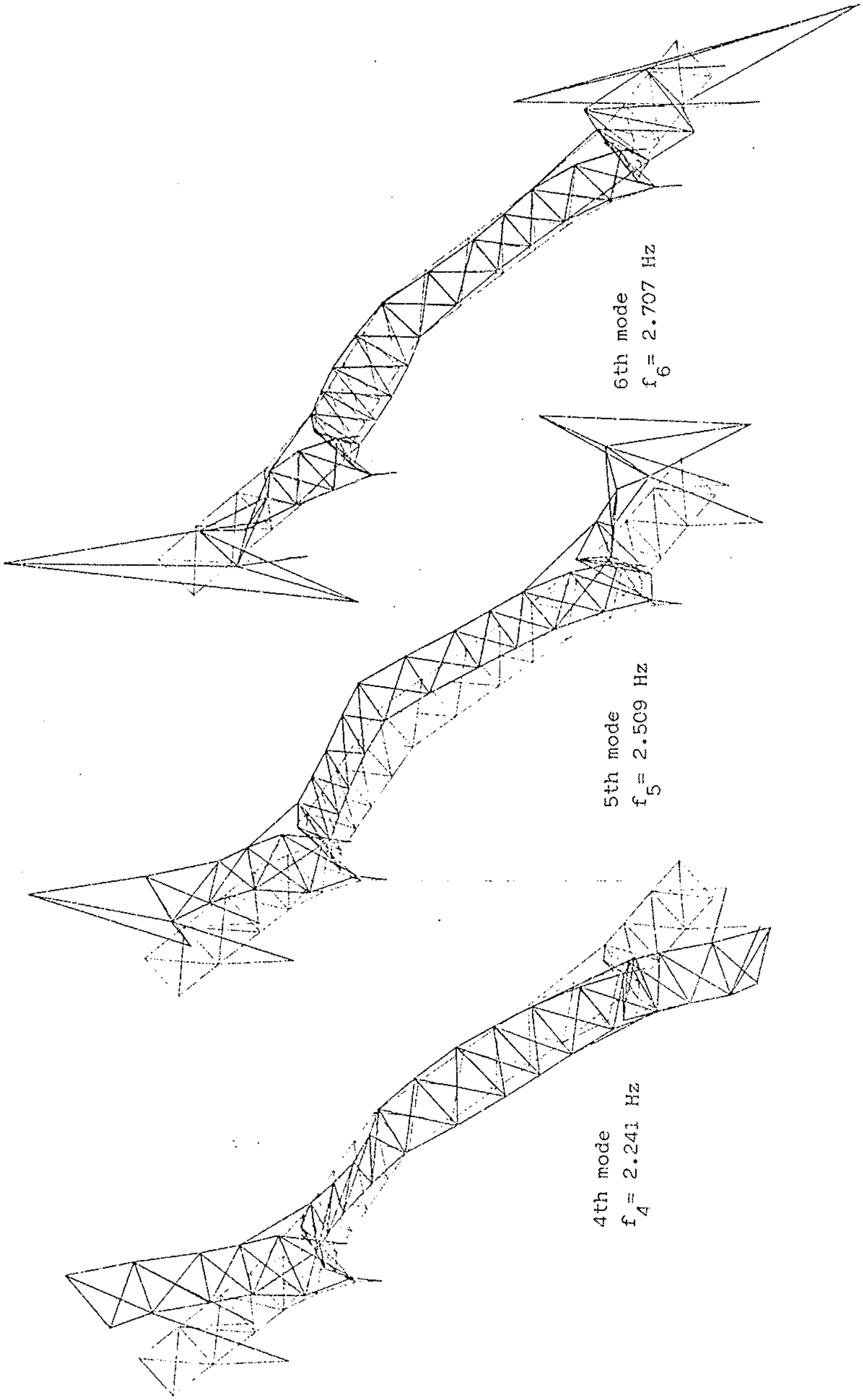


Fig.6b(cont.)

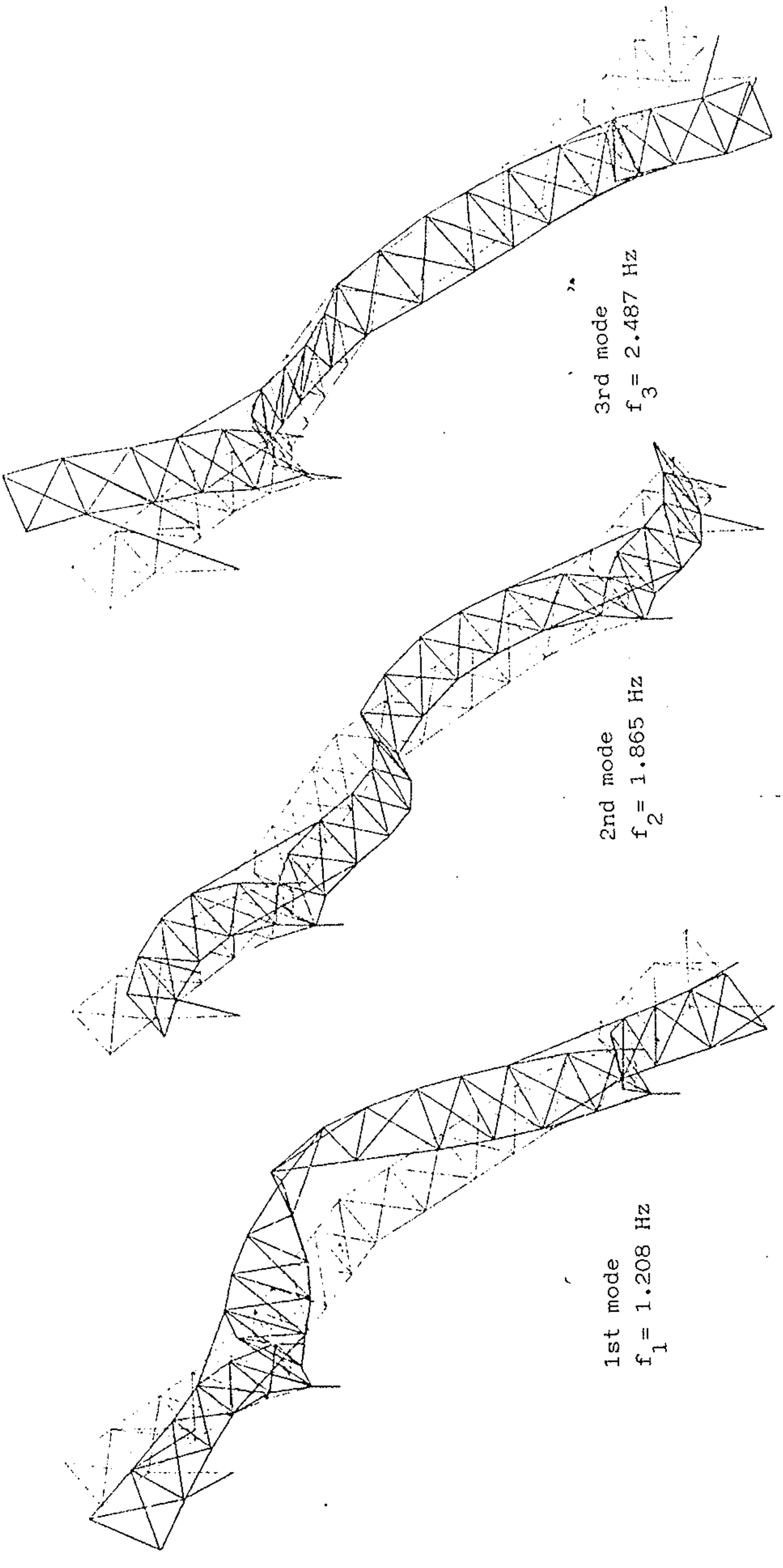


Fig.6c

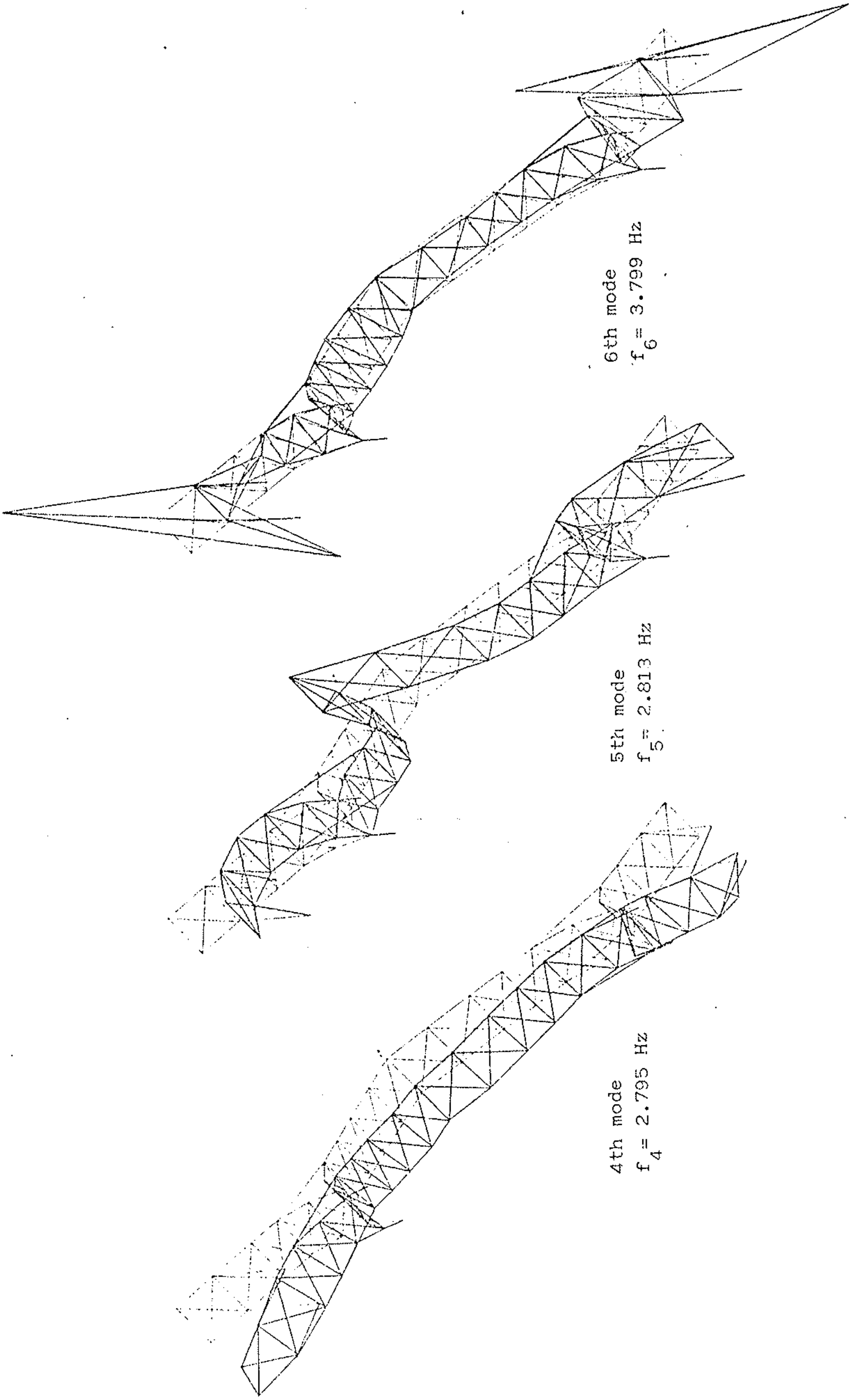


Fig.6c(cont.)

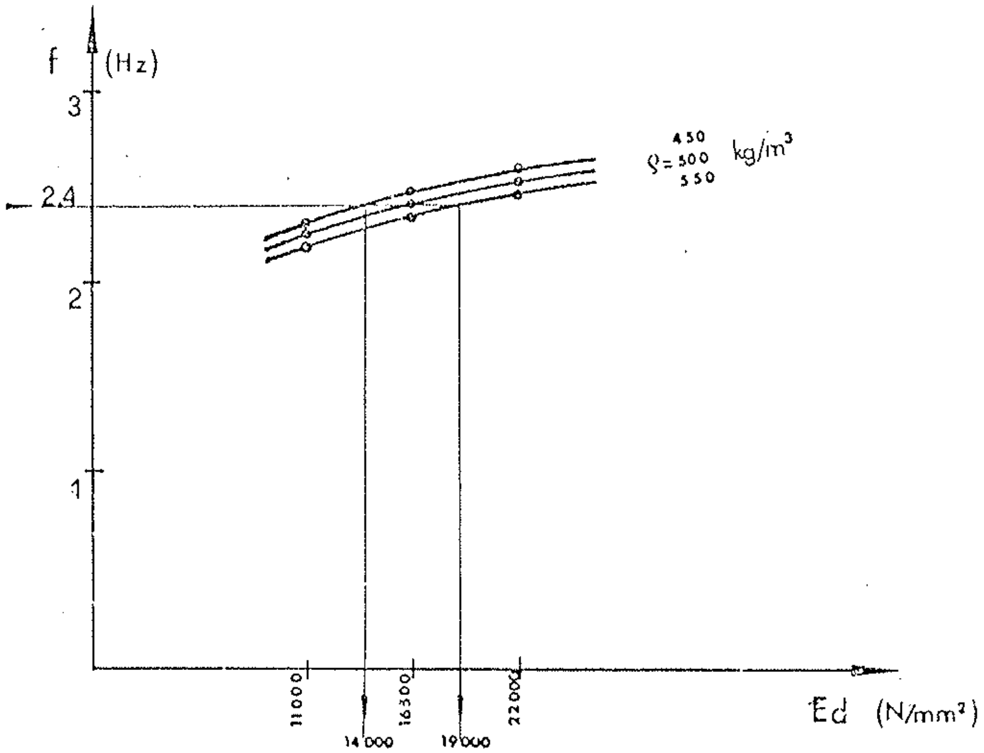


Fig. 7 - The frequencies of the modal shapes like the experimental shape, versus the dynamic elastic modulus (Ed) and the density mass of timber (ρ).

	Ed = 11000 N/mm² kg/m²			Ed = 16500 N/mm² kg/m³			Ed = 22000 N/mm² kg/m³		
	ρ=450	ρ=500	ρ=550	ρ=450	ρ=500	ρ=550	ρ=450	ρ=500	ρ=550
1st	0.903	0.883	0.863	1.088	1.055	1.040	1.236	1.208	1.181
2nd	1.350	1.320	1.291	1.653	1.574	1.581	1.908	1.865	1.824
3rd	2.308	1.992	1.948	2.448	2.390	2.346	2.542	2.487	2.436
4th	2.290	2.241	2.195	2.495	2.404	2.384	2.857	2.795	2.736
5th	2.563	2.509	2.457	2.760	2.687	2.643	2.379	2.813	2.751
6th	2.761	2.707	2.656	3.367	3.299	3.239	3.874	3.799	3.727

(frequencies in Hz)

fig. 8 - The frequencies of the first six vibrating modes versus Ed and ρ. The values of modal shapes similar to the experimental shape, are underlined.

CIB-W18/18-15-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STABILIZING BRACINGS

by

H Brüninghof
Bergische Universität - Gesamthochschule Wuppertal
Federal Republic of Germany

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

Bracing

Adequate bracing should be provided to avoid lateral instability of individual members and the collapse of the whole structure due to external loading such as wind.

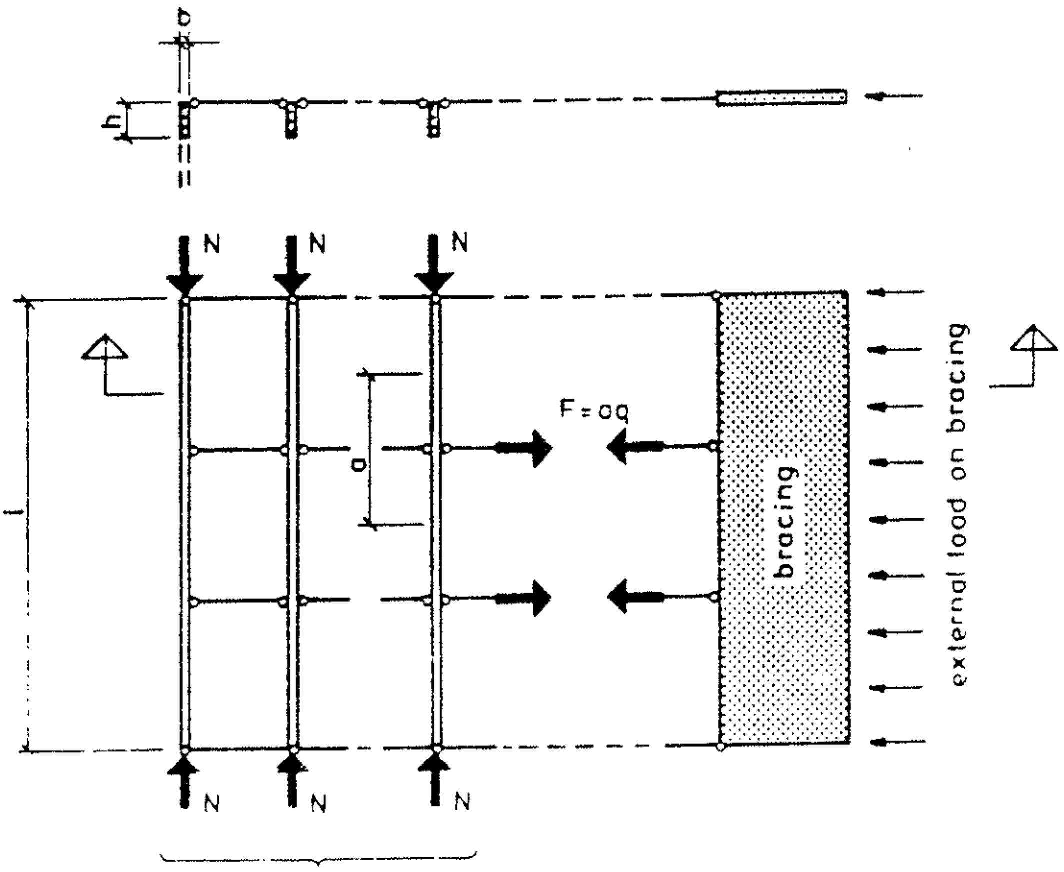


Fig. 1: Bracing System

With n equal members (e.g. beams or chords in a truss) of rectangular cross-section, the bracing should in addition to external loads (e.g. wind) be designed for a uniform load q per unit length:

$$q = \frac{n \cdot N}{l \cdot k_{br,c} \cdot k_{br,m}}$$

where

$$k_{br,c} = \frac{12}{l} \cdot (k_n k_l u_o + u)$$

N is the axial force in a member.

Where the member is a beam with maximum moment M and depth h, N should be taken as 1.5 M/h.
Where the member is a truss N is the maximum compressive force.

$$k_l = \min. \begin{cases} 1 \\ \sqrt{15/l} & (l \text{ is span in m}) \end{cases}$$

$$k_n = 0.5 \cdot \left(1 + \frac{1}{n}\right)$$

u_0 is the initial deviation from straightness at midspan.

u is the deflection of the bracing caused by the sum of q and external loads calculated with $E = E_{0,k} f_{m,d}/f_{m,k}$.
The effect of slip in joints should be taken into account.

$$k_{br,m} = \frac{\pi}{8} \cdot \left(1 + 1.5\pi \cdot \left(1 - 0.63 \frac{b}{h}\right) \cdot \left(\frac{b}{h}\right)^2 \cdot \frac{E_{0,k}}{f_{m,k}} \cdot \frac{G_{mean}}{E_{0,mean}}\right.$$

4/c

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAMPLING OF TIMBER IN STRUCTUREL SIZES

by

P Glos
Universität München
Federal Republic of Germany

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

SAMPLING TIMBER IN STRUCTURAL SIZES

P. Glos, Institut für Holzforschung, Universität München

1. SCOPE

The mechanical properties of structural timber strongly depend upon natural growth characteristics and manufacturing practices. Under otherwise same conditions (species, grade, size, moisture content etc.) timber may exhibit a substantial variation in properties. From this follows that test results are strongly influenced by the respective sampling procedure and that the method of choosing a sample is an all-important factor in determining what use can be made of it. If a generalization is to be made from the results of an empirical investigation to a specific population, it is a prerequisite that the test material sampled is representative for that population.

In the field of timber engineering various questions can arise, that presume different test material selection procedures. Among these are e.g.:

- Evaluation of characteristic mechanical properties related to a population defined in space and time.
  This requires sampling of specimens that are representative for the defined population.

- Evaluation of the effect of specific treatments on characteristic mechanical properties.
  This may require so-called matched subsamples, that are selected from a given sample so that they have the same distribution of mechanical properties.

- Evaluation of mechanical properties of timber structures and of joints

made with mechanical fasteners.
In general this requires selecting timber whose growth characteristics vary only in predetermined narrow limits.

In the following, sampling or selection procedures are described for the aforementioned points under discussion.

These sampling procedures are generally applicable. They do not depend upon the planned sample size nor on whether all elements of the sample or, as in the so-called In-Grade Testing Programs, only a part of the sample will be destructively tested.

Decisions on sample size and number of elements tested destructively are governed by the financial scope of the program, the chosen degree of precision with which the properties are to be estimated and the chosen test and data analysis procedures. Some relations between sample size and the precision of the statistical inference are shown as a general information in section 2.4.

2. SAMPLING OF TIMBER FOR THE EVALUATION OF CHARACTERISTIC PROPERTIES

2.1 CONDITIONS FOR VALID SAMPLING

2.1.1 Population

It is imperative that first of all the population of interest, termed the t a r g e t population, to which we would like our conclusions to be applicable, is defined as clearly as possible in terms of geographical area, time span, and manufacturing practices. In practice it is most likely that sampling from the target population is not possible, e.g. because there is no access to the production of coming years. Hence, the population from which the sample is taken, termed the s a m p l e population, may be different from the target population.

There exist no statistical methods to convert inferences on the sample population into inferences on the target population. The further the sample population is removed from the target population, the more the desired conclusions will have to be based on perhaps unwarranted "other considerations". This aspect should be borne in mind throughout the whole sampling, testing, and data analysis process.

Care must be taken that a consideration for convenience does not unknowingly lead to a sampling of the wrong population, such as timber *now* being produced instead of timber expected to be produced within the next 10 to 20 years.

In the following the term "valid sampling" and the discussion of sampling methods are restricted to sampling from the sample population, which itself must be defined as clearly as possible.

2.1.2 Sampling procedure

In order to make valid nontrivial generalizations from a sample about characteristics of the population from which it comes, the sample must have been obtained by a sampling scheme which insures that the distribution of values in the sample represents the distribution of values in the population, i.e. that the sample is a so-called *representative* sample. This requires that the sampling scheme involves some type of *random* selection.

In practice, it is not always easy to obtain a random sample from a given population. Unconscious selections and biases tend to enter.

It cannot be overemphasized that the randomness of a sample is inherent in the sampling scheme employed to obtain the sample and is not an intrinsic property of the sample itself. Experience teaches that it is not safe to select a sample haphazardly, without any conscious plan. Nor does it seem to be possible to consciously draw a sample at random. In order to avoid bias, e.g. bias through personal judgment of appearance in either direction when selecting the material, it is mandatory that the selecting procedure is defined in advance and that sampling follows this predetermined plan strictly.

Some possible schemes for formally drawing a sample at random from a particular population are given in section 2.2.

2.2 METHODS OF SAMPLING

2.2.1 Sampling with equal probabilities (simple random sampling)

The most widely recommended type of random selection is simple or unrestricted random sampling. This type of sampling is defined by the requirement that e a c h individual in the population (commonly the individual piece of timber) has an e q u a l c h a n c e of being the f i r s t member of the sample; after the first member is selected, each of the remaining individuals in the population has an equal chance of being the second member of the sample; and so forth.

Where a population consists of a finite set of N elements, each element is allocated a number from 1 to N. In order to obtain a sample of size n, n numbers are depicted randomly from the numbers from 1 to N (e.g. by using a random number generator or random number tables) and the pertaining elements are selected for the sample.

For typical timber research tasks with large populations involved, this method is generally uneconomic and hardly feasible from a practical point of view.

2.2.2 Sampling with unequal probabilities (stratified cluster sampling)

If there is an indication of any potential systematic sources of variation within the population, that is, when the population can be subdivided into diverse groups (strata) and the probability of an individual element of the population belonging to one of these groups is known, this information should be considered in selecting the sample. By doing this appropriately, it is ensured that the distribution of potential sources of variation in the sample coincides with the distribution in the population.

In a multistage procedure, the successive stages being e.g. regions, mills (size, type), pieces within mills; or logging areas, tree sizes (age, yield class), positions of specimen in tree, the sample population is subdivided into potential groups, or strata. In order to prevent bias, it is imperative

. that the number of individual pieces selected from each stratum at
  each stage corresponds to the respective relative volume of timber ap-

pertaining to that stratum (volume of timber available or volume of timber being milled at present or in future), and

. that within each stratum, the pieces are selected at random.

It can be shown that stratified cluster sampling, as more information is utilized, in general yields more consistent results than simple random sampling, especially when the sample size is small.

Test results at hand indicate that regional differences in timber properties, if observed at all, can generally be explained by density differences. This raises the question whether the required regional and possibly temporal stratification could be reduced if appropriate, i.e. sufficient closely correlated concomitant variables, such as density, were identified. If so, sampling could be restricted to a very limited area if it were ensured that these concomitant variables are equally distributed both in the sample and in the population. However, there are objections to this procedure: There remains the uncertainty of other potential regional factors, as e.g. specific processing effects.

2.2.3 Sequential sampling

Sequential sampling, in its original meaning, is a stepwise sampling and data analysis procedure. It is applicable to both simple random sampling and stratified cluster sampling.

The first step is to select a sample of size n_1. Based on the test results from that sample a decision is made whether to continue with sampling. If so, the second step is to select a second sample of size n_2. Based on the results from the joint sample ($n_1 + n_2$) a decision is made whether to continue with sampling; and so forth.

O n t h e a v e r a g e the number of pieces to be sampled and tested can be reduced by applying this method. However, the practical application may be complicated.

Sometimes selection of packages, that is serial selection of individual pieces of timber may be appropriate and sometimes this method is also called sequential sampling. In the case of serial sampling the principles in 2.2.1 and 2.2.2 apply to the composite sampling units (packages), they do not apply, however, to the serially sampled individual pieces of timber.

2.3 REPORT

In order to make interpretation of test results and conclusions feasible, it is highly recommended to include the following in the report:

- the definition of the target population
- the definition of the sample population
- the sampling method chosen, and
- the steps taken or planned to bridge the gap, if any, between the target and the sample population.

2.4 SIZE OF SAMPLE

Apart from the given financial scope of the program the required sample size depends upon the objective of the research, on the chosen precision with which the properties are to be estimated, as well as on the variation of the test material and the chosen data analysis procedure.

For general information, Figs. 1 and 2 show, based on a population that follows a Weibull distribution, how the precision of the estimated property depends upon the size of the sample and the chosen statistical model. Fig. 1 shows the influence of the chosen confidence level, Fig. 2 the influence of the assumed statistical distribution. The true population 5 percent exclusion limit is 24.3 MPa; the graphs indicate the spread of the estimated values of the exclusion limit, based on the estimation from 100 independent samples.

It can be seen from these Figs. that independent of the chosen confidence level and of the assumed statistical distribution the estimated value may vastly differ from the true population value. The larger the sample size, however, the more likely the estimated exclusion limit will be close to the population value. It is, therefore, desirable to select a sample size as large as possible commensurate with the cost of sampling and testing.

The Figs. indicate that a sample size of n = 200 may be adequate in many cases.

Fig. 3 shows how many elements out of a sample of size n = 200 on the average must be tested destructively, when the M = 10...40 weakest elements are to be broken and when the testing plan makes use of a non-destructively tested concomitant variable, which exhibits a correlation coefficient of ρ with the investigated property.

3. SAMPLING OF TIMBER MATCHED FOR STRENGTH FOR EXPERIMENTAL EVALUATION OF SPECIFIC EFFECTS SUCH AS DURATION OF LOAD, MOISTURE CONTENT, SIZE ETC.

For a description of problems involved and possible procedures that may be employed reference is made to

- Gerhards, 1976
- Madsen and Barrett, 1976
- Pierce, 1980

If the results are to be representative for a given population, the total test material must first be selected according to one of the methods outlined in section 2. This material is then subdivided into matched groups.

The chosen sampling method shall be completely reported.

4. SAMPLING OF TIMBER FOR JOINT TESTS

One of the two sampling procedures as specified in the International Standard ISO xxxx ' Timber Structures. Testing of joints made with mechanical fasteners. Requirements to the wood' shall be applied.
The chosen sampling method shall be reported.

5. SAMPLING OF TIMBER FOR STRUCTURES TO BE TESTED AS PROTOTYPES

For a description of problems involved and possible procedures that may be employed reference is made to

. Norén, 1983.

6. REFERENCES

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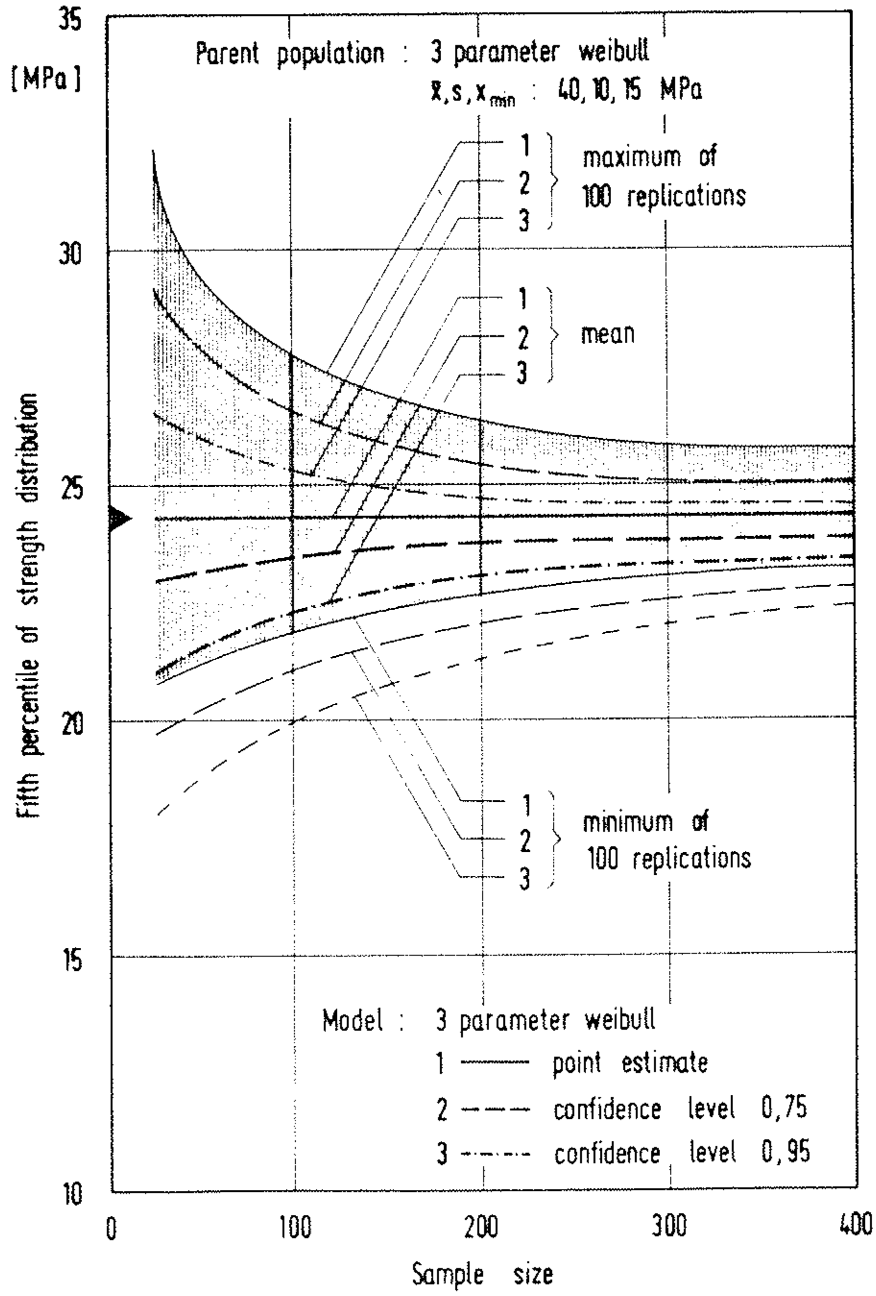

Fig. 1: Estimate of 5 percent exclusion limit as dependent upon sample size and confidence level.
Simulation results. Population: 3 parameter Weibull

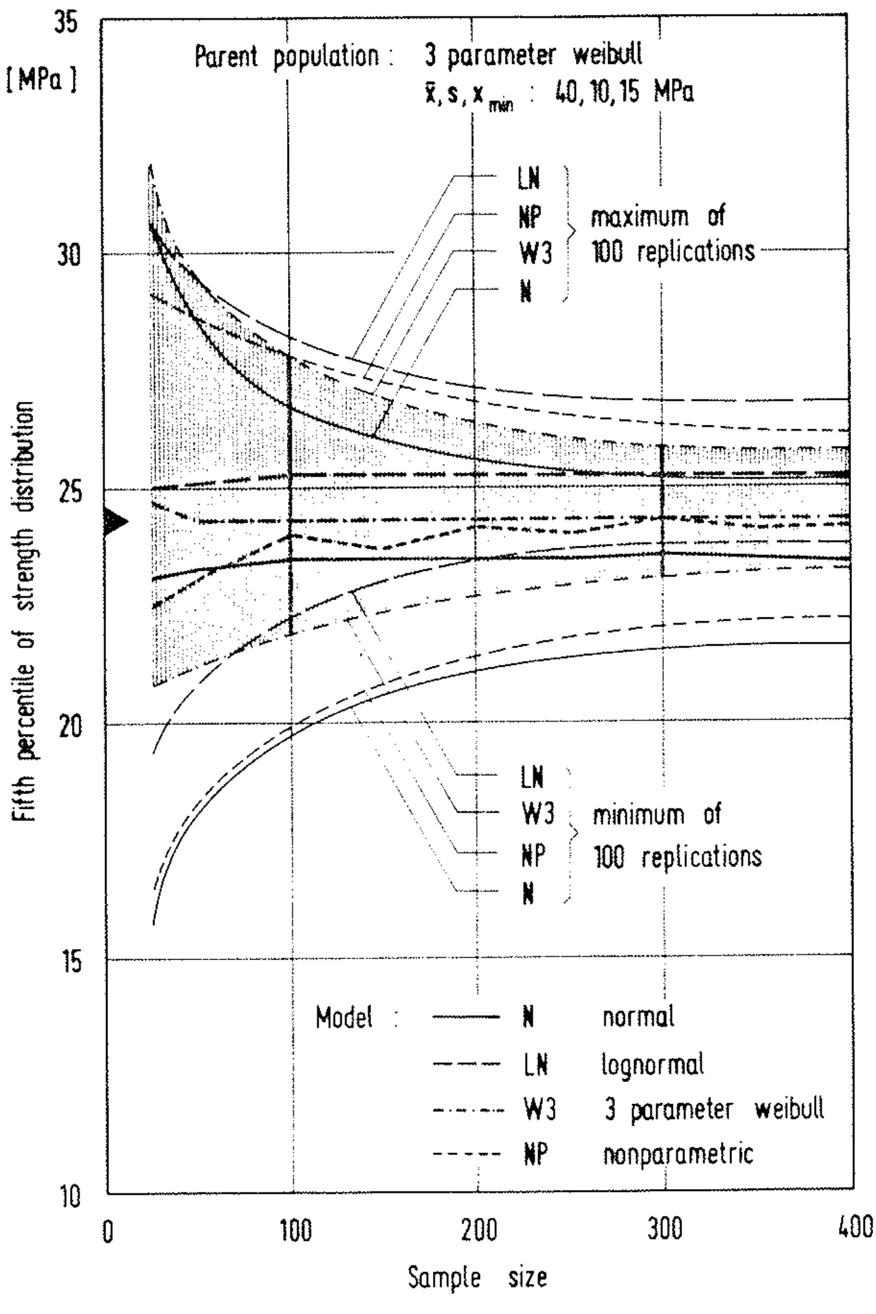

Fig. 2: Point estimate of 5 percent exclusion limit as dependent upon sample size and statistical model used. Simulation results. Minimum, maximum value and mean of 100 replications. Population: 3 parameter Weibull

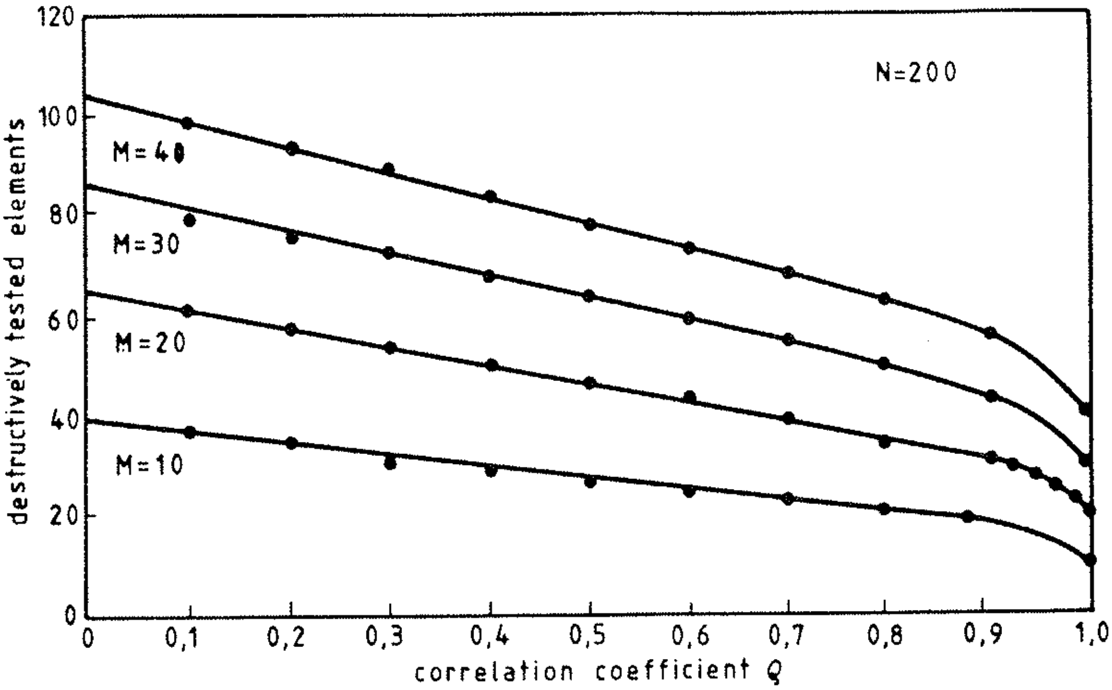

Fig. 3: Relationship between expected number of destroyed elements and degree of correlation with a concomitant variable, ensuring that the lowest M out of N elements will be tested.
From Warren, 1975.

CIB-W18/18-102-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ANTISEISMIC RULES FOR TIMBER STRUCTURES: AN ITALIAN PROPOSAL

by

G Augusti and A Ceccotti
University of Florence
Italy

MEETING EIGHTEEN
BEIT OREN
ISRAEL
JUNE 1985

ANTISEISMIC RULES FOR TIMBER STRUCTURES: AN ITALIAN PROPOSAL

Giuliano AUGUSTI* and Ario CECCOTTI**

*Prof.ing.,Ph.D.(Cantab.)

**Dr.ing.

Dept. of Civil Engineering, Universita' di Firenze,
Via S.Marta 3; I-50139 Florence Italy.

ABSTRACT

Most modern Codes of practice for antiseismic constructions accept the limit state approach, but allow to take advantage of structural ductility in order to reduce the effects of earthquake actions. A recent draft proposal for a substantial revision of the current Italian Rules follows the same trends.

Structural timber however, be it massive or glue-laminated, exhibits a comparatively fragile behaviour, mainly because of its natural defects: the necessary ductility can only be obtained by means of appropriate structural joints.

This paper illustrates the "Timber Structures" Chapter in the proposed new Italian Rules, as an example of how provisions for structural timber can be incorporated in modern antiseismic rules.

Paper presented at the Engineering
Seminar during the Symposium on
Forest Products Research International-
Achievements and the Future
Pretoria, South-Africa 22/26 april 1985

1. INTRODUCTION

In the law that has ruled "Constructions in Seismic Areas" of Italy for the last decade [Roma 1974], four structural systems are formally allowed, namely:

a) reinforced or prestressed concrete and/or steel frames,
b) load-carrying panel structures,
c) masonry,
d) timber structures.

Nevertheless, in the "official" Instructions so far issued by the Ministry of Public Works for application of the said law, the indications regarding timber structures have been limited to a Clause of few lines, of which the latest version [Roma 1984] is reported in Appendix A.1 of this paper.

This has generated lots of confusion about the admissibility of timber in earthquake-resistant constructions (in particular of glue-laminated timber, which is not explicitly mentioned in the Law nor in the relevant Clause of the Instructions) and has certainly been a great hindrance to the development of timber buildings in the "official" seismic zones, which have been greatly extended between 1981-83 and at present cover about 80% of the national territory.

Therefore, it is evident that if the present situation did not change, all timber construction in Italy could be practically blocked, notwithstanding the growing interest in this material, confirmed by some recent examples of very beautiful and seismically adequate structures (see e.g. AUGUSTI <u>et al.</u> 1984b).

Fortunately, an "ad hoc" Task Group was recently set up by the (Italian) National Research Council (C.N.R.) in order to draft a new set of Rules for antiseismic constructions in Italy, intended to replace the currently valid Ministerial Instructions, that in many technical circles are considered partially obsolete. The writers, who think that timber may be a suitable and convenient construction material in seismic areas, have taken the occasion and prepared a Chapter on Timber Structures, which has been included among the proposed rules. The draft has been completed a few months ago and, in accord with previous practice of other analogous C.N.R. proposals, has now been published [Roma 1985] in order that all interested research and professional people may test the proposed rules, point out possible mistakes, suggest modifications and improvements, before submitting a final text to the Minister of Public Works for their adoption as officially valid Ministerial Instructions.

The proposed rules, following a generalized international trend, are biased towards limit state design procedures rather than the more traditional allowable stress methods. As well known, this trend poses very special problems for timber structures, because limit state design requires exploitation of the structural ductility and capacity to dissipate energy, but the knowledge about these properties is still appropriate response parameter (e.g. horizontal displacement) in the "actual" elastic-plastic behaviour and the value x_s of the same parameter at the limit of elasticity. This ratio can be shown (Fig.4) to be equal to the ratio between the forces, F_{el} and F_u respectively, that correspond to x_u in the elastic-plastic behaviour and in an ideal, indefinitely elastic behaviour.

3. ANTISEISMIC STRUCTURAL DESIGN

3.1 Seismic actions

Because of the randomness of the seismic input, most modern Codes prescribe one or more "design spectra" (usually, acceleration spectra) for the design of earthquake-resistant structures. The "intensity" (i.e. the values of the ordinates of the spectrum) is calibrated versus the earthquakes "expected" in the area, while its "shape" is related to the general geological characteristics of the region and the geotechnical nature of the foundation soil. By way of example, the spectra included in the repeatedly quoted draft proposal for new Italian Rules [Roma 1985] are shown in Fig.5.

One of the ways in which some Codes (and the new Italian draft) take account of ductility, is to reduce the prescribed actions by dividing the design spectrum by an appropriate "structure factor" $K \geq 1$: the choice of the values of this factor for each construction type is clearly one of the most delicate and difficult points in the writing of a Code. In fact, by the very nature of the simplifications and approximations introduced in such an approach, no choice can be fully and always satisfactory.

In the current Italian by-law [Roma 1984], that follows the "admissible stress" approach, the structural ductility factor is implicitly assumed equal to 5 for all structures, irrespective of type and material; no differentiation with the foundation soil is introduced in the seismic loading. The dotted line in Fig.5 is the spectrum for the high-seismicity zone prescribed by the current by-law, multiplied by 1.5 in order to allow comparisons with the spectra of the new draft, written from the "limit state" viewpoint: remember that the latter spectra must be divided by the relevant structure factor K, but note that when K = 1 the load requirements of the new rules are much more stringent than the current ones.

3.2 Combination of actions

Another very important point in earthquake-resistant structural design is the choice of the other actions that are assumed present when the quake occurs. The relevant provisions of the Italian draft are reported in Appendix A.2 b). It can be seen that two short-duration loads must never be considered as acting together; thus, if the structure is calculated for another comparatively heavy but short-duration load (such as may be snow loading), it may result also safer with respect to seismic actions, always of short duration.

Thus, the final text of the draft Chapter, although much longer than the single paragraph of the current by-law (cf. Appendix A.1), is still much less detailed than the analogous provisions of other Codes. For instance, only three types of timber constructions have been listed, which is a very rough classification. However, there are at least two significant improvements: first, no height limitation is set; second, glue-laminated constructions, whose admissibility had given rise to doubts in the recent past, are explicitly accepted.

Moreover, the strength under short duration loads (such as seismic actions) can be taken 1.5 times that under long duration loads.

As for ductility, the draft states that none can be expected from timber as a structural material but that, in principle, the ductility of timber structures can be assured by appropriate structural joints. Consequently, the writers had proposed an articulated set of values for the divisor "structure coefficient" K, derived from the New Zealand Seismic Code [Wellington 1976], in which ductility is accounted for by applying to the actions of a multiplying factor ≤ 1. However, the same prudential attitude led the Task Group finally to agree on two conservative values, namely

K = 2 for structures braced by panels (i.e. "sheathed"), and

K = 1 (i.e. no ductility) for framed and truss-braced structures.

By comparison, note that in the draft typical values of K for reinforced concrete frames vary between 2.5 and 4, and for steel structures between 2 and 6. Thus, the horizontal actions assumed on a timber structure are usually much larger than those considered for an analogous reinforced concrete or steel structure (typically, 2 to 4 times higher) and, as already noted with reference to Fig.5, than the actions prescribed by the current rules.

Some numerical calculations, developed by the writers as examples of application of the proposed rules and not reported here for brevity, have shown that timber structures well designed to resist vertical and other environmental loads, may be well adequate for the seismic loading required by the draft, when the rule of load combination and/or the increment of strength under short-duration actions are taken into account. This was e.g. the case of the structure quoted in AUGUSTI <u>et al.</u> 1984b.

8. REFERENCES

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8.2 Laws, by-laws and drafts

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Wellington 1976: Code of practice for general structural design and design loading. Standard Ass. of New Zealand, N.Z.S.4203.

Roma 1984: Ministerial Decree 19.6.1984: Technical rules for constructions in seismic zones [*]. Gazzetta Ufficiale, n.208 (30.7.1984).

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Ψ_{Ei}: simultaneity coefficients to be applied to the masses in the determination of the forces of inertia.......... The values of the coefficients Ψ_{Ei}, relevant to the floor loads on buildings, are reported in Section II.3.4.

c) From Section II.3.4 Coefficients of combination

The combination coefficients Ψ_i to be used in local checks as quoted in Sec.I.4.2, are shown in the following Table:

1. Buildings... not open to the public		Ψ_i	= 0.30
2. Public premises ... (shops, restaurants, ...)			0.30
3. Public premises with possible large crowds (meeting places, theaters, churches,.....)			1.00
	....		
5. Balconies and staircases:	a) residential buildings		0.50
	b) public premises		1.00
	....		
8. Archives and libraries			1
9. Snow in sites H > 1000 m			0.30
10. Snow in sites H < 1000 m			0
11. Wind			0
12. Temperature			0

The simulteneity coefficients Ψ_{Ei} quoted in sec.I.4.2 for the calculation of the inertia forces, are evaluated as follows.
In general, $\Psi_{Ei} = \phi\, \Psi_i$
For multi-storey buildings with independent uses, $\phi = 1$ at the top floor and $\phi = 0.5$ at all other floors.
For multi-storey buildings with correlated uses, $\phi = 1$ at the top floor, $\phi = 0.8$ at the floor with correlated use, $\phi = 0.3$ at all other floors.
For loads of type 8) in the Table above, $\phi = 1$.

d) Text of Chapter III.4 BUILDINGS WITH TIMBER STRUCTURE

III.4.1 Structural types

1. All wooden structural elements which must resist seismic actions shall fullfill the provisions of Part I and II of the present Instructions, besides the following rules.
   Usually, these structures belong to one of the following types:
   a) framed structures;
   b) structures with truss bracings;
   c) structures with panel bracings.

2. Timber structures shall fullfill the following requirements:
   - details of the junctions between timber structure and foundations and of the structural elements with each other shall be studied in such a way to avert the possibility of slips and connection failures;

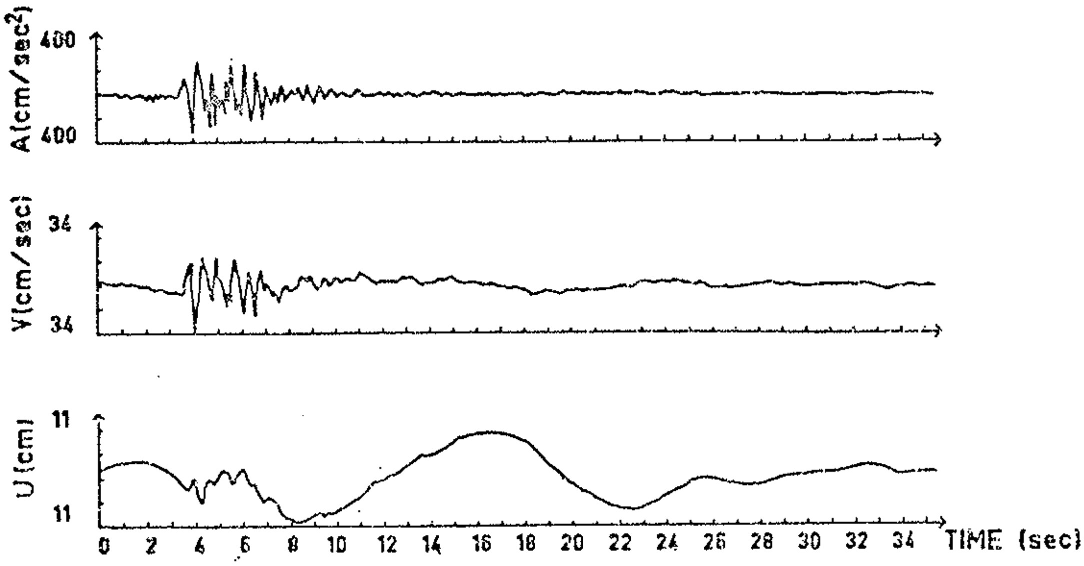


FIG. 1) RECORDS OF E-W COMPONENT OF MAY 1976 FRIULI EARTHQUAKE AT TOLMEZZO

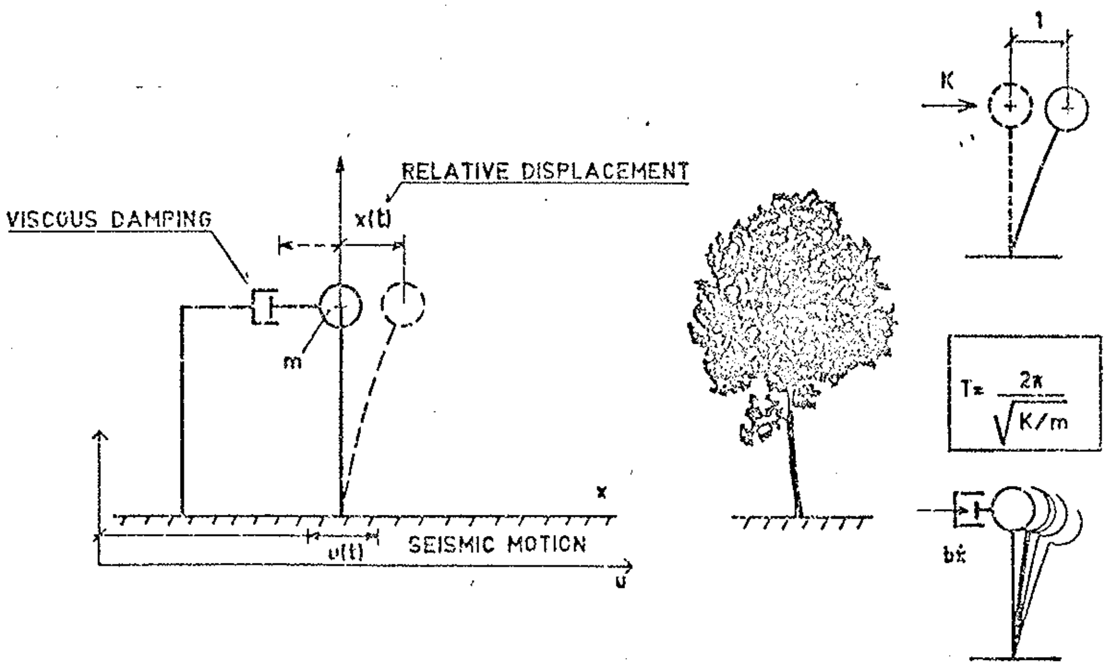


FIG. 2) ELEMENTARY OSCILLATOR AND SEISMIC ACTION (DIAGRAMMATIC)

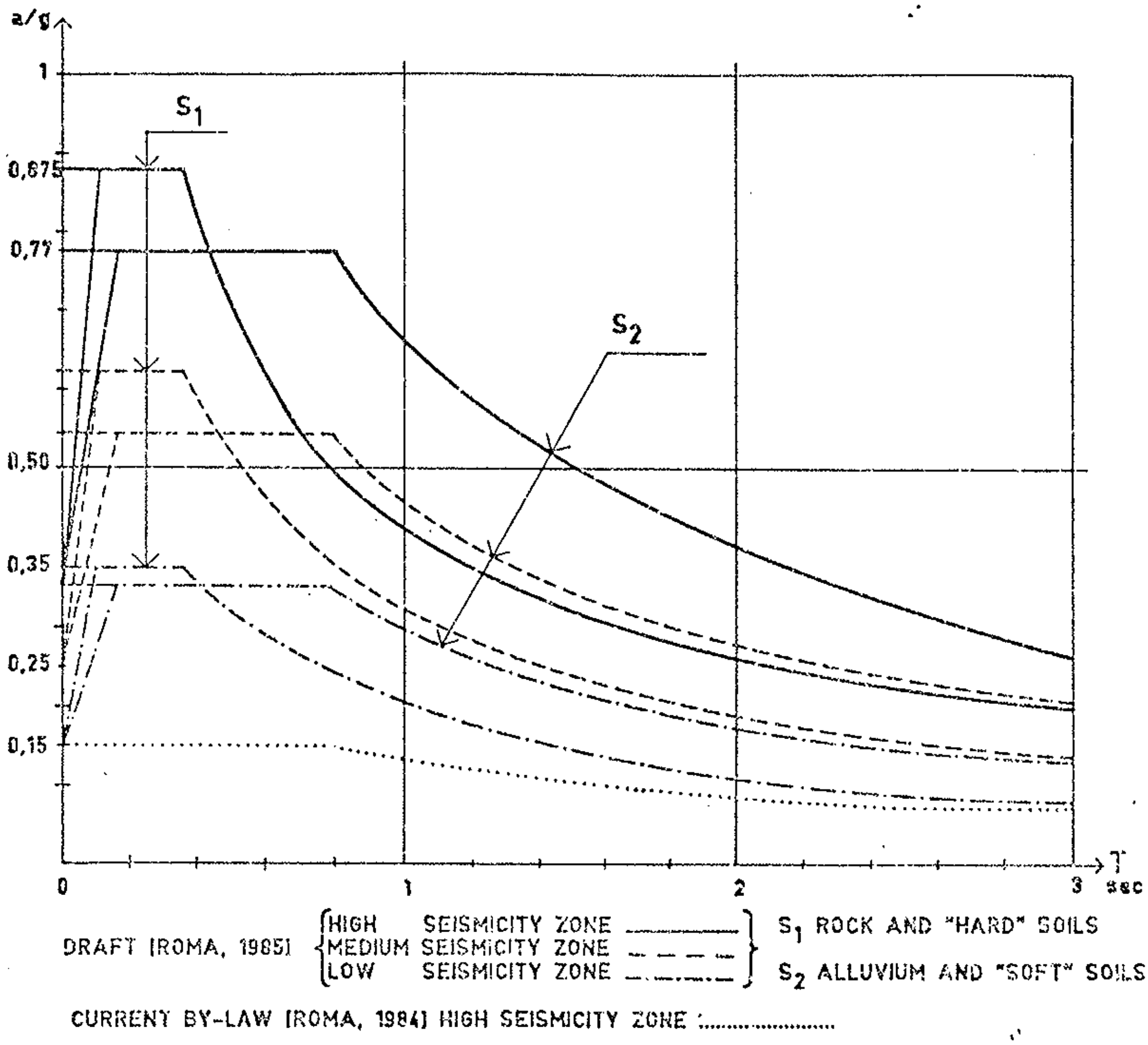


FIG. 5) ACCELERATION DESIGN SPECTRA OF ITALIAN SEISMIC RULES

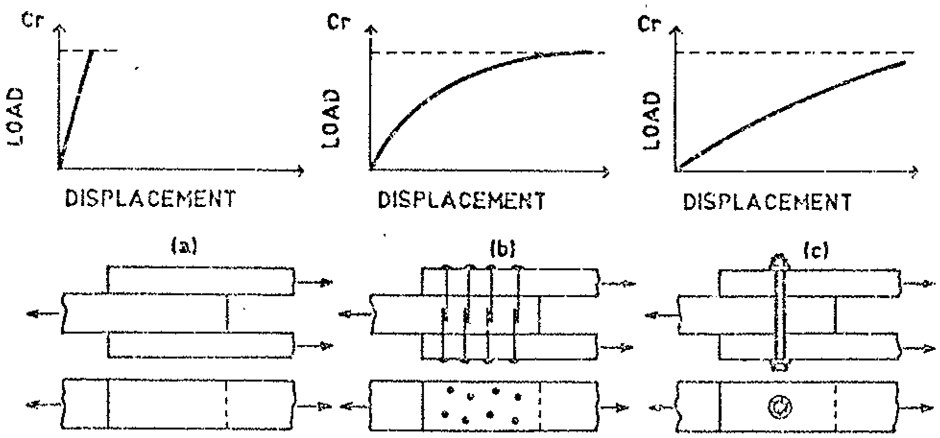


FIG. 6) LOAD DISPLACEMENT DIAGRAMS FOR a) GLUED JOINT; b) NAILED JOINT c) BOLTED JOINT; AFTER GIORDANO 1983.

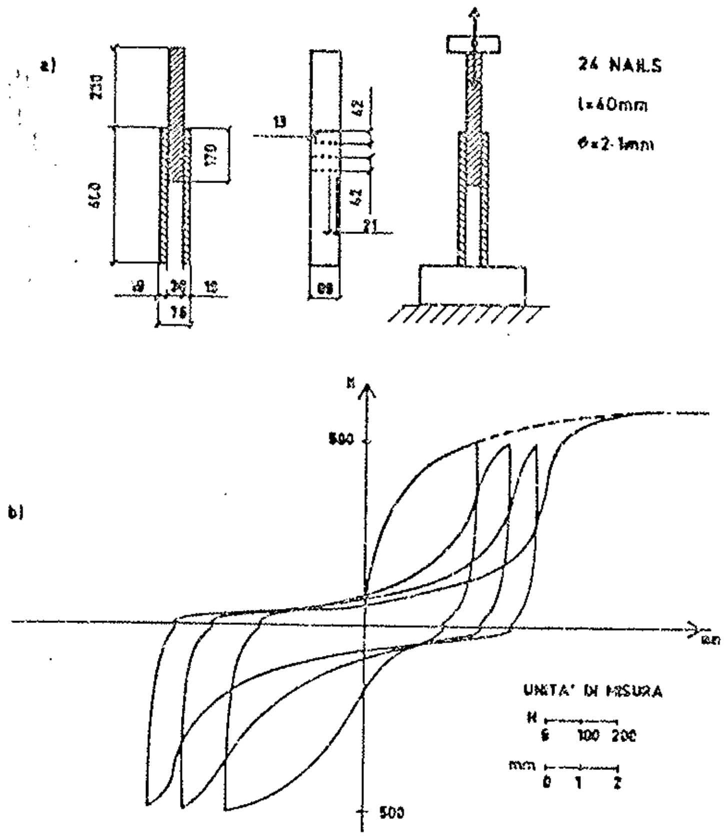


FIG. 7) TESTSON NAILED JOINTS UNDER CYCLIC LOAD: a) DIAGRAM OF TEST SET-UP; b) TYPICAL EXPERIMENTAL PLOT; AFTER DI IORIO, 1984.

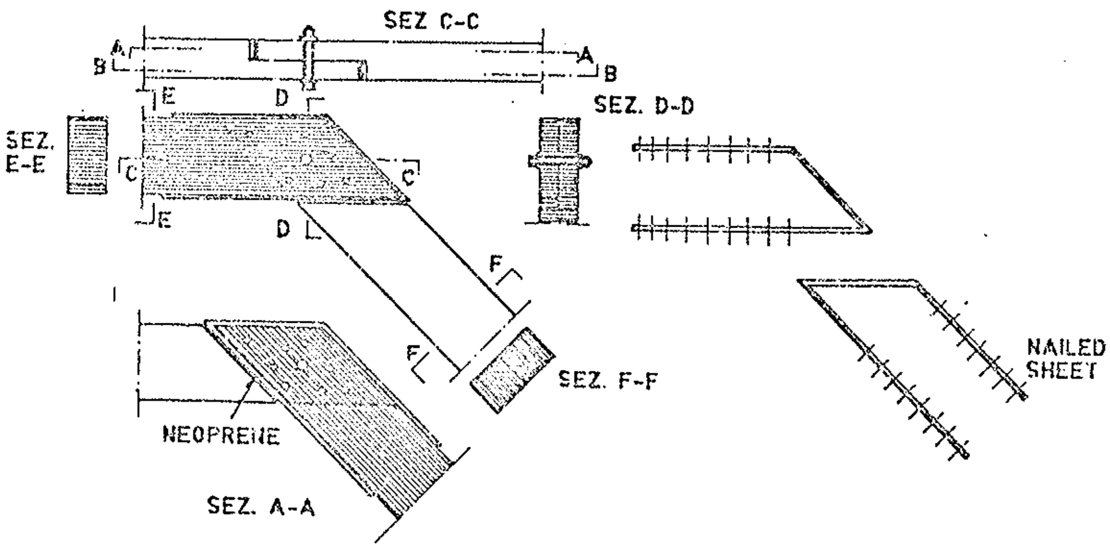


FIG. 8) EXAMPLE OF BOLTED-NAILED JOINT WITH DISSIPATIVE CAPACITY

NOTES AND PAPERS

OF THE RILEM-MEETING

BEIT OREN

ISRAEL

JUNE 1985

RILEM 57-TSB

Beit Oren, Haifa, Israel. 3 June 1985

Present:

J Kuipers, Chairman, The Netherlands	P Leirtun, Norway
E Aasheim, Norway	B Madsen, Canada
D Brown, United States of America	R Marsh, United Kingdom
J Brundin, Sweden	U Meierhofer, Switzerland
H Burgess, United Kingdom	B Norén, Sweden
A Ceccotti, Italy	T Ramstad, Norway
P Colclough, Ireland	I Smith, United Kingdom
B Edlund, Sweden	E G Stern, United States of America
J Ehlbeck, Federal Republic of Germany	C Stieda, Canada
P Glos, Federal Republic of Germany	J Sunley, United Kingdom
M Johansen, Denmark	B Thunell, Sweden
J Kangas, Finland	J Tory, United Kingdom
B Kälsner, Sweden	R Verhorst, Belgium
U Korin, Israel	A Vignoli, Italy

Member by correspondence: W.Nozyński Poland.

1 The RILEM group was welcomed to Israel by Dr Korin on behalf of the Building Research Station, Technion.

2 Nailed joints

2.1 Comments had been received on the draft recommendation 3TT-1B for the testing of nails. The chairman drew attention to his own remarks on these comments and invited discussion.

2.2 It was agreed that figures to illustrate the configuration of timber-to-board material joints should be included in the test standard.

2.3 Norén, Tory and Meierhofer spoke in favour of using fewer nails in the specimens to provide data more appropriate for analysis. They pointed out that the test included a special joint configuration with possible grouping effects and would not necessarily be appropriate for different types of nails. Norén said that although Sweden accepted the principles of the standard it would probably not be used in its present form.

2.4 Ehlbeck said that the standard was for testing joints, not nails, and variability would be less with more nails in the joint. Kuipers also supported the draft recommendation and pointed out that the test ensured the same numbers of nail shear planes for single and double shear tests.

2.5 It was concluded that the no change should be made to the standard as regards the numbers of nails in the test joints.

2.6 It was agreed that clause B.4.1 should be amended to require that the nail head should not protrude above the surface of the test specimen.

2.7 The symbols for nail spacings (clause B.6.1.1.4) are to be replaced by text.

2.8 It will be made clear in the text that laminated timber is permissible for the manufacture of specimens but when used the laminations should be matched for density.

2.9 Kuipers and Stern are to draft a clause to cover the withdrawal strength of nails in end grain. The figure illustrating the withdrawal test specimen is to show a longer specimen containing more nails and may also include a nail in the end grain.

2.10 Smith suggested that the tests of material properties, ie embedding strength and nail bending, were more suited to another test standard. After some discussion it was agreed that no reference should be made to the testing of embedding strength but that the nail bending test should be retained.

2.11 Norén, on behalf of the committee, thanked Professor Kuipers for his efforts in preparing and carrying through the work on this test standard.

2.12 The test standard for nails will now be submitted for publication as a RILEM test standard.

3 Embedding strength

3.1 The paper 'A Method for Determining the Embedding Characteristics of Wood and Wood-based Materials' was introduced by Smith who said that TRADA had carried out 2000 tests in developing the test method. He explained that when testing board materials in their manufactured thicknesses there were likely to be problems with nail bending which could be adversely reflected in the application of the results.

3.2 Meierhofer suggested that with such thin specimens splitting of the timber would be a problem and the proportion of summer to spring wood could also be quite significant. He also asked how test results could be interpreted and related to design loads.

3.3 Tory said that the test method was not too relevant to the work of RILEM; it was a research tool related to a theoretical model. However Norén supported the idea of a standard embedding strength test to compare different types and shapes of nail. Ehlbeck disagreed, saying that basic and practical research should not be mixed.

3.4 The discussion was concluded with no proposal for a standard test method for embedding strength.

4 Wood-based board materials

4.1 It was agreed that the American Plywood Association should provide a definition of wood-based board materials for inclusion in the scope of the draft standard and that the testing of plywood should be specifically excluded.

4.2 There was criticism of the panel shear test specimen that had been taken from the plywood standard. Ehlbeck suggested that the test should be removed from the standard. Smith said that in tests carried out by TRADA they had not had a true shear failure. Tory stated that it was a difficult test to carry out with very cumbersome apparatus; he described an alternative 'T' shaped specimen but admitted that few had been tested. Brown supported the test, saying that APA had satisfactorily tested many particle boards.

4.3 Smith was also critical of the test for panel shear modulus of rigidity. He said that shear stress distribution was not uniform across the specimen and therefore the shear modulus measured was unreliable.

4.4 It was agreed that a test for panel shear strength was required and since there was more experience of the ASTM method than of any other, it should be retained.

4.5 It was agreed that the provision for selecting small specimens for quality control checks (clause 8.2) should be removed from the standard.

4.6 Clause 8.5.3 relating to creep in bending is to be deleted.

4.7 Smith and Tory told the meeting that they had both had difficulties in achieving acceptable failures in the tension testing of thick high quality plywoods. However there were no proposals for a change to the standard.

5 Timber structures

5.1 Norén proposed that the third draft of the test standard for timber structures should be published as a tentative recommendation. He agreed that significant differences had been introduced since the second draft but explained that it provided a general test programme which could be modified.

5.2 There was a short discussion on the sampling of materials for the test structures but no changes were agreed.

6 Conclusion

6.1 The chairman summarised the state of progress of the testing standards being considered by the committee. The final recommendation on nails would now be submitted to RILEM for publication; the tentative recommendation on staples was awaiting publication; the tentative recommendations on wood-based board materials and timber structures would also be submitted to RILEM. An editorial group comprising Kuipers, Ehlbeck, Norén and Tory is to deal with comments received on these standards. Should major difficulties arise they will be referred to CIB-W18.

6.2 The chairman brought the meeting to a close and told members that now that the objectives of the committee had been achieved he would inform RILEM of their progress and committee 57-TSB would cease to function.

RESULTS OF RESEARCH ON WOODEN BEAMS SUBJECTED TO LONG TIME LOADING*)

by Vladyslaw Nozyński

Centralny Ośrodek Badawczo-Rozwojowy Przemysłu Stolarki Budowlanej, Wołomin, Poland

Research on solid timber beams and on glulam beams was carried out from 1971 to 1980.

All beams were

50 x 100 x 2400 mm

and tested in bending as in fig. 1. Deflections were measured at mid span. Several series of beams were tested.

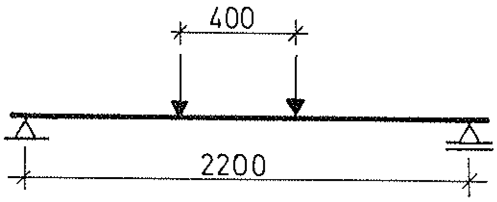
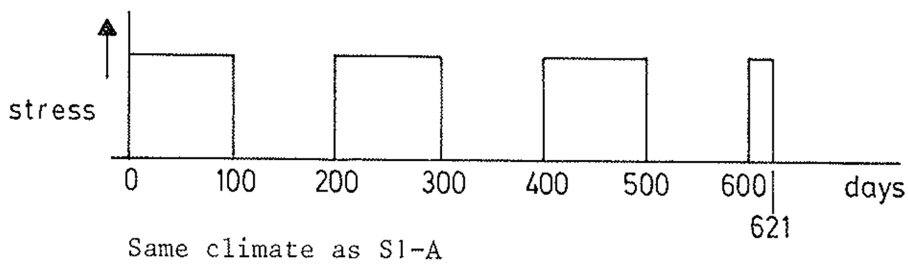


figure 1

Series S1-A Solid timber, tested for 588 days in a not heated, close room, loaded at 3 levels.

	stresses			
	10	14	20	N/mm²
repetitions	3	3	3	

Series S1-B Solid timber; 3 cycles of loading and unloading.



Same climate as S1-A

Series S2 Beams loaded for 1091 days in a tent for protection against rain and sun.

	stresses		
	13	16	N/mm²
repetitions	6	6	

*) Late-received contribution

Series S3 — Solid timber and glulam (10 layers; glued with resorcinol) loaded in the open air during 1170 days.

	solid		glulam	
stresses	13	16	13	16
repetitions	3	3	3	3

Results

Series S1-A

duration of loading days	deflections in mm stress level		
	10	14	20
	11.60	14.50	17.00
1	13.40	16.66	20.00
9	15.30	19.06	23.10
15	16.43	20.73	25.60
20	16.46	21.33	27.40
40	17.20	21.96	28.70
59	17.26	22.40	30.30
80	16.93	22.80	31.40
104	17.83	23.60	33.60
151	18.30	24.40	35.10
202	19.00	25.30	37.70
248	20.20	26.50	39.50
297	17.40	24.30	42.50
364	19.16	25.50	41.80
412	21.86	26.20	44.00
455	19.23	26.80	44.60
512	20.10	27.50	-
588	22.90	29.00	-

Series S2

duration of loading days	deflections in mm stress level	
	13	16
	9.25	9.52
1	10.70	11.10
12	13.18	13.30
45	16.40	16.06
108	17.93	17.41
168	18.18	17.80
200	17.85	17.55
240	18.15	17.75
300	18.16	18.10
360	18.96	18.75
420	19.55	19.28
520	20.28	20.63
600	20.50	20.30
690	20.88	20.71
750	20.86	20.85
840	22.78	22.75
990	23.60	24.13
1091	23.56	24.23

Series S1-B

duration of loading	10	14	20	10	14	20	10	14	20	10	14	20
	after cycle 1			after cycle 2			after cycle 3			after cycle 4		
	11.63	15.90	19.60	18.60	17.40	29.10	19.00	19.10	29.50	20.30	19.60	31.60
1	11.90	17.00	21.90	19.20	17.90	29.70	20.30	20.90	31.20	21.10	20.40	32.40
20	–	–	–	19.50	19.20	30.60	–	–	–	21.70	21.60	33.20
50	15.13	21.93	29.90	–	–	–	22.00	23.50	34.60	–	–	–
52	–	–	–	20.30	19.60	31.80	–	–	–	–	–	–
98	17.70	25.30	33.10	–	–	–	–	–	–	–	–	–
107	–	–	–	21.60	21.00	34.10	–	–	–	–	–	–
111	–	–	–	–	–	–	22.50	24.00	35.50	–	–	–
recovery after												
	7.03	9.80	16.80	11.70	7.53	15.30	12.80	9.03	16.50	–	–	–
1	6.46	9.20	15.90	11.20	7.33	14.70	12.40	8.43	15.60	–	–	–
50	4.10	5.80	12.10	–	–	–	11.50	7.40	14.00	–	–	–
52	–	–	–	9.46	4.73	11.90	–	–	–	–	–	–
107	–	–	–	8.76	3.66	10.10	–	–	–	–	–	–

Series S3

duration of loading	solid 13	solid 16	glulam 13	glulam 16
	9.07	11.33	11.50	15.45
1	9.90	12.40	11.75	15.90
8	12.46	14.53	14.60	19.65
37	14.76	16.90	16.50	23.80
52	15.49	17.66	16.35	24.60
97	16.13	18.40	18.75	28.05
157	16.50	18.46	19.40	28.50
197	16.60	19.23	20.75	29.39
290	16.76	19.10	21.35	30.40
350	17.23	19.70	21.20	30.45
510	18.87	23.33	21.10	32.10
750	19.70	23.13	22.60	32.75
990	20.53	23.93	24.10	33.55
1170	20.53	24.16	24.90	33.70

Analysis of results

The aim of the research was to study the behaviour of beams under long duration of loading and to find methods for the prediction of deflections. Therefore the deflections were expressed as increasing percentages of the elastic deflections (table 5)

Table 5

Series	Time in days					
	1	30	180	365	730	1095
S1-A - I	116	145	160	165	-	-
S1-A - II	115	149	171	176	-	-
S1-A - III	118	165	214	246	-	-
S2 - I	116	162	195	204	226	255
S2 - II	117	156	184	197	219	255
S3 - I	110	163	182	190	217	226
S3 - II	109	149	168	174	204	212
S3 - III	102	143	174	184	197	217
S3 - IV	103	154	188	197	212	218

From this table it can be seen that after 1 day the deflections increase with about 15%, after 1 month with about 50%, after one year with about 80%, after two years with about 100% and after 3 years with more than about 130%.

A creep formula was proposed:

$$U_t = (1 + ct^{1+k})\, U_o$$

From this creep formula it follows that

$$\frac{U_t - U_o}{U_o} = ct^{1+k}, \text{ or } \frac{U_t - U_o}{U_o t} = ct^k.$$

In figure 2 plots of $\frac{U_t - U_o}{U_o t}$ versus t have been given, from which the values of k and c were calculated and given in table 6.

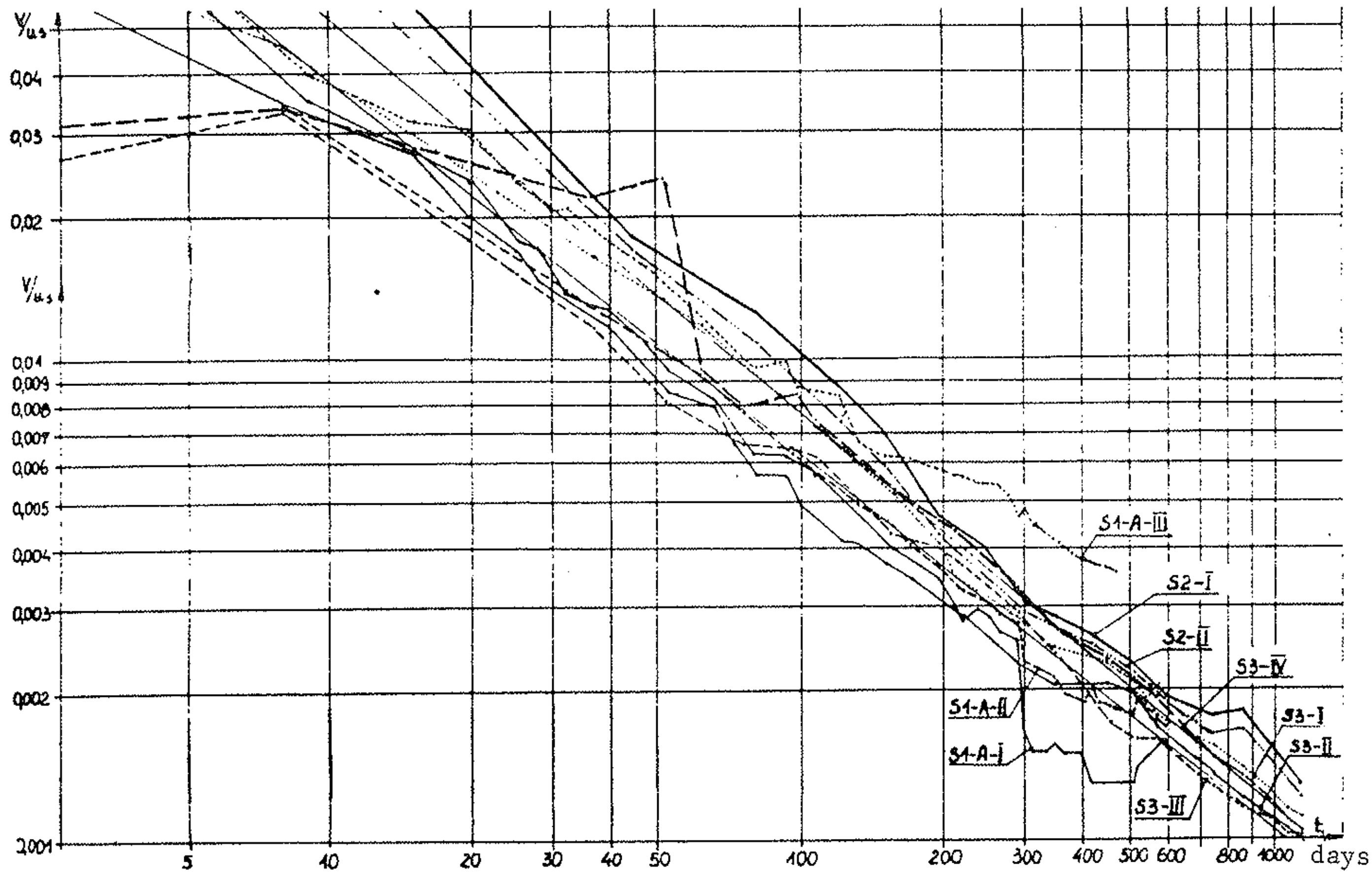


figure 2 Dependence of fast deflections to time of active load (in relation to elasticity deflections).

Series	k	c
SI-A - I	-0,785	0,18
II	-0,771	0,20
III	-0,684	0,21
S2 - I	-0,732	0,24
II	-0,713	0,18
S3 - I	-0,797	0,30
II	-0,745	0,18
III	-0,786	0,25
IV	-0,829	0,36

Table 6

From this it was concluded to a proposal for k- and c-values below:

- in conditions of clim. class 1: c = 0,20; k = -0,750
- in conditions of clim. class 2: c = 0,20; k = -0,720
- in conditions of clim. class 3: c = 0,30; k = -0,800

(clim. classes as in CIB-W18 code).

Conclusions:

1. Deflections of beams increase in time, but growth is smalling in time.
2. Propose to define the deflections of beams after time of activity of load by formula given above.
3. Suitable is to subject beams in natural scale research under long time load. The beams with 6,00 m span.