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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB-W18

10. Sitzung

MEETING TEN

VANCOUVER, CANADA

AUGUST 1978

CONTENTS

1 LIST OF DELEGATES

BELGIUM

A R Egerup	Automated Building Components, Bruxelles

CANADA

J D Barrett	Western Forest Products Laboratory, Vancouver
T A Eldridge	Lamco Structural Products Ltd, Quebec
E Fowler	Eastern Forest Products Laboratory, Ottawa
C R Henderson	Weldwood of Canada Ltd, Vancouver
B Madsen	University of British Columbia, Vancouver
D R Meeks	Macmillan Bloedel, Vancouver
P Nielsen	University of British Columbia, Vancouver
J Stevenson	Macmillan Bloedel Research Ltd, Vancouver
H Vokey	Canadian Wood Council, Vancouver
C R Wilson	Council of Forest Industries of British Columbia, Vancouver

DENMARK

H J Larsen	Aalborg University Centre, Aalborg

FRANCE

P E H Crubile	Centre Technique du Bois, Paris

FEDERAL REPUBLIC OF GERMANY

J Ehlbeck	University of Karlsruhe, Karlsruhe
M Kufner	Institut für Holzforschung, München

JAPAN

T Nakai	Forestry and Forest Products Research Institute

NETHERLANDS

J Kuipers	Technical University of Delft, Delft
H Ploos van Amstel	Technical University of Delft, Delft

NORWAY

P Aune	University of Trondheim, Trondheim
R Birkeland	Norsk Treteknisk Institutt, Oslo
N I Bovim	Norsk Treteknisk Institutt, Oslo

SWEDEN

B Edlund	Chalmers University of Technology, Göteborg
B Kalsner	Swedish Forest Products Research Laboratory, Stockholm
B Norén	Swedish Forest Products Research Laboratory, Stockholm
B Thunell	Swedish Forest Products Research Laboratory, Stockholm

UNITED KINGDOM

L G Booth	Imperial College of Science and Technology, London
(1) J G Sunley	Timber Research and Development Association, High Wycombe
(2) J R Tory	Building Research Establishment, Princes Risborough

UNITED STATES OF AMERICA

B Bohannen	Forest Products Laboratory, Madison
D H Brown	American Plywood Association, Tacoma
R A Eckert	Weyerhaeuser Company, Tacoma
R G Pearson	North Carolina State University, Raleigh
E G Stern	Virginia Polytechnic Institute, Blacksburg

VENEZUELA

J C Centeno	Universidad de los Andes, Mérida

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

2 CHAIRMAN'S INTRODUCTION

Delegates to the meeting were welcomed by the Chairman MR SUNLEY, co-ordinator of CIB-W18. He explained that this short half-day meeting of the Commission, held during the course of the IUFRO S 5.02 Wood Engineering Group meeting had been especially conveened to introduce the work of W18 to those in North America who had not yet attended the regular meetings of the Commission. He explained that W18 functioned as an unofficial club and would welcome active participation from anywhere in the world.

MR SUNLEY continued by introducing paper CIB-W18/10-105-1, 'The Work of CIB Working Commission W18 - Timber Structures'. He detailed the links that existed between W18 and other international organisations including IUFRO, RILEM and ISO. He said that the work of W18 could be judged on its merit and was increasingly being sought by international bodies such as ECE and EEC who needed to formulate international timber codes. The work of W18 was also recognised by the recently formed ISO technical committee TC165 - Timber Structures that had asked W18 to draft an international timber code together with supporting standards.

3 CIB TIMBER CODE

PROFESSOR LARSEN introduced 'The CIB Timber Code (Second Draft)' (CIB-W18/9-100-1). He told the meeting that this paper had previously been discussed by the Commission at Perth (meeting nine, June 1978) but it was being presented again to familiarise the North American delegates with the contents. At the last meeting several amendments had been made to the Code and a third draft would shortly be ready for circulation. Professor Larsen explained that there were several omissions from the code, most noticeably Chapter 3 which would deal with safety. As it stood at present the Code could be used for either the deterministic or the limit state design method; all that was required were suitable safety or partial factors. The safety principles given in CEB Bulletin 116 could also be adopted after modifications to suit timber.

PROFESSOR LARSEN also drew attention to the environmental and load duration classes that had been adopted for the Code.

MR SUNLEY told delegates that considerable progress was being made in Europe towards international harmonisation, both for timber and between different construction materials.

Speaking of limit state design, DR BARRETT said that he considered it essential that there should be a consistent approach to design by those responsible for different materials since they could manipulate only one side of the design equation. He did not think it possible for timber design to function satisfactorily on the ratios of live to dead load that had been adopted in some countries for steel and concrete.

Since CEB Bulletin 116 could accept different partial factors for load transfer from components to a system, timber could adopt different load factors, suggested DR BOOTH.

PROFESSOR MADSEN suggested that the very short term and the instantaneous load classifications should be combined.

It was pointed out by DR NOREN that the classification systems for load duration and environmental conditions had to be suitable for all wood-based materials and not just for solid timber.

DR BARRETT proposed that tables 2.3.a and 2.3.b should also be related to recurrence intervals for wind and earthquake loadings.

PROFESSOR LARSEN agreed with MR PEARSON and PROFESSOR MADSEN that a confidence limit should be included in the definition of characteristic values.

There was a general discussion on the tentative proposal to introduce a strength class system for structural timber. It was agreed that if a satisfactory system could be formulated this would be a useful move towards the simplification of design procedures.

PROFESSOR LARSEN said that changes in the Code would be introduced for notched beams. He accepted a suggestion from DR BARRETT to look into the method used in the Australian code. Professor Larsen also said that annexes in the next draft would cover jointed beams and columns, spaced columns and lattice columns. In answer to a question from PROFESSOR MADSEN, he said that the Code should be applicable to all timber structures but it would not deal with constructional methods.

4 RILEM 3-TT

DR KUIPERS outlined the scope of the Union of Testing Research Laboratories for Materials and Structures (RILEM) and particularly committee 3-TT which was concerned exclusively with timber and timber-based materials. He explained that close links were maintained with CIB-W18 and agreed recommendations on the testing of joints and structural plywood had been published in the RILEM Journal "Materials and Structures". Dr Kuipers said that test methods for metal plate fasteners and structural sized timber would soon be published and methods for nailed joints were in the course of preparation. Test methods for fibre boards, particle boards and prototype structures were also to be considered by 3-TT/W18. Dr Kuipers concluded his verbal account of the work of RILEM by emphasising that they were concerned only with test methods; sampling and the evaluation of test results was the responsibility of CIB-W18.

MR SUNLEY told the North American delegates that unfortunately the American Society for Testing and Materials (ASTM) was not represented on the 3-TT/W18 committee. Naturally however the existing ASTM test methods were taken into account and many of the proposed plywood tests had evolved from ASTM procedures.

DR KUIPERS told delegates that committee 3-TT invited comments on their published test procedures from any source.

There followed a discussion on the various test procedures for structural sized timber with particular emphasis on the methods adopted by Professor Madsen who uses very short duration loading and random defect location.

MR BOHANNAN told delegates that the Madison Forest Products Laboratory was looking closely at Professor Madsen's test work and, together with Washington State University would investigate how to correlate the results from his procedures with those that were currently accepted.

PROFESSOR THUNELL suggested that the correlations between different test methods could vary with species because of the frequency and spacing of defects.

DR BOOTH told PROFESSOR MADSEN that he felt the code committees in the United Kingdom would have considerable difficulty in accepting results from his test programme. He said that in Europe the policy was to find the lower percentiles for the material and to predict from them the lower percentiles for the components.

What Professor Madsen was doing, said Dr Booth, was to find the fifth percentiles for beams under particular loading conditions with little possibility for extrapolation to basic material properties.

5 ROOF BRACING

MR BRYANT introduced "Design of Roof Bracing - The State of the Art in South Africa" (CIB-W18/10-14-1). He explained that provision for the bracing of trussed rafter roofs was being made in the South African code although they still had difficulties in adequately transmitting the bracing loads to the wallplates and also with the buckling modes of rafters.

PROFESSOR LARSEN pointed out that only dead loads appeared to influence the bracing requirements in Australia and South Africa and the stiffness of members did not.

MR EGERUP supported Professor Larsen's observations. He considered that side and torsional stiffness should be taken into account and that the complete roof should be considered. He suggested that if thicker members were used the initial problem might be solved with less bracing.

MR SUNLEY said that there appeared to be a lack of theory and experimentation on the bracing of trussed rafter roofs. In the United Kingdom guidance on bracing was given but this was based largely on rules of thumb.

6 PLYWOOD

"Buckling Strength of Plywood" (CIB-W18/10-4-1) was introduced by DR PLOOS VAN AMSTEL who told delegates that the paper reported the results of extensive tests on the buckling of plywood panels. The tests had shown that for two widths and for a range of length:width ratios with free and clamped edges plywood had complied with the usual buckling theories.

PROFESSOR LARSEN said that he was pleased to see that both buckling and post buckling loads were given in this report thus recognising buckling as a serviceability limit state and not an ultimate limit state.

7 NEXT MEETING

The chairman thanked PROFESSOR MADSEN for the facilities that had been made available for the meeting of the Commission and he reminded delegates that the next meeting would be held in Vienna, 27-30 March 1979.

Topics for discussion will include:-

1 CIB Timber Code (third draft)
2 Glulam
3 Sampling of plywood and evaluation of test results
4 Trussed Rafters
4 Fibre and Particle boards

For those not on the regular mailing list of CIB-W18, details of the next meeting are available on request from:

J R Tory, Princes Risborough Laboratory, Princes Risborough, Buckinghamshire HP17 9PX, United Kingdom.

8 PAPERS PRESENTED AT THE MEETING

CIB-W18/10-4-1	Buckling Strength of Plywood - Results of Tests and Recommendations for Calculations - J Dekker, J Kuipers and H Ploos van Amstel.
CIB-W18/10-14-1	Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon.
CIB-W18/9-100-1	The CIB Timber Code (Second Draft)
CIB-W18/10-105-1	The Work of CIB-W18 - Timber Structures - J G Sunley

9 CURRENT LIST OF CIB-W18 TECHNICAL PAPERS

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

Example: CIB-W18/4-102-5

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

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

Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden, Feb/March 1977
8 Bruxelles, Belgium, October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978

Subjects are denoted by the following numerical classification:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing

9 Duration of Load

10 Timber Beams

11 Environmental Conditions

12 Laminated Members

13 Particle and Fibre Building Boards

14 Trussed Rafters

15 Structural Stability

100 CIB Timber Code

101 Loading Codes

102 Structural Design Codes

103 International Standards Organisation

104 Joint Committee on Structural Safety

105 CIB Programme, Policy and Meetings

106 International Union of Forestry Research Organisations

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

LIMIT STATE DESIGN

1-1-1 Paper 5 Limit State Design - H J Larsen

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

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

1-1-4 Paper 8 Working stresses report to British Standards Institution Committee BLCP/17/2

6-1-1 On the application of the uncertainty theoretical methods for the definition of the fundamental concepts of structural safety - K Skov and O Ditlevsen

TIMBER COLUMNS

SYMBOLS

PLYWOOD

STRESS GRADING

STRESSES FOR SOLID TIMBER

6-6-1 Standard Methods of Test for the Determination of some Physical and Mechanical Properties of Timber in Structural Sizes (third draft) - W T Curry

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

9-6-1 Classification of Structural Timber - H J Larsen

9-6-2 Code Rules for Tension Perpendicular to the Grain - H J Larsen

9-6-3 Tension at an Angle to the Grain - K Möhler

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

TIMBER JOINTS AND FASTENERS

1-7-1 Paper 12 Mechanical fasteners and fastenings in timber structures - E G Stern

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

4-7-2 Paper 9 Test Methods for Wood Fasteners - K Möhler

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

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

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

6-7-1 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures (seventh draft) - RILEM, 3TT Committee

6-7-2 Proposals for Testing Joints with Integral Nail Plates - K Möhler

6-7-3 Rules for Evaluation of Values of Strength and Deformation from Test Results - Mechanical Timber Joints - M Johansen, J Kuipers, B Norén

6-7-4 Comments to Rules for Testing Timber Joints and Derivation of Characteristic Values for Rigidity and Strength - B Norén

7-7-1 Testing of Integral Nail Plates as Timber Joints - K Möhler

7-7-2 Long Duration Tests on Timber Joints - J Kuipers

7-7-3 Tests with Mechanically Jointed Beams with a Varying Spacing of Fasteners - K Möhler

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

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

9-7-2 Staples - K Möhler

LOAD SHARING

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

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

4-8-2 Paper 13 Load Sharing - B Norén

DURATION OF LOAD

3-9-1 Paper 7 Definitions of Long Term Loading for the Code of Practice - B Norén

4-9-1 Paper 14 Long Term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen

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

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

6-9-2 X Long Term Loading - K Möhler

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

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

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

TIMBER BEAMS

4-10-1 Paper 6 The Design of Simple Beams - H J Burgess

4-10-2 Paper 7 Calculation of Timber Beams Subjected to Bending and Normal Force - H J Larsen

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

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

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

ENVIRONMENTAL CONDITIONS

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

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

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

LAMINATED MEMBERS

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

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

8-12-2 Instructions for the Reinforcement of Apertures in Glulam Beams - H Kolb and P Frech

8-12-3 Glulam Standard Part 1: Glued Timber Structures; Requirements for Timber

9-12-1 Experiments to Provide for Elevated Forces at the Supports of Wooden Beams with particular regard to Shearing Stresses and Long-term Loadings - F Wassipaul and R Lackner

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

PARTICLE AND FIBRE BUILDING BOARDS

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

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

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

TRUSSED RAFTERS

4-9-1 Paper 14 Long Term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen

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

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

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

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

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

CIB TIMBER CODE

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

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

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

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

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

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

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

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

LOADING CODES

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

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

STRUCTURAL DESIGN CODES

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

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

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

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

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

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

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

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

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

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

8-102-1 Polish Standard PN-73/B-3150: Timber Structures; Statistical Calculations and Designing

8-102-2 The Russian Timber Code: Summary of Contents

9-102-1 Svensk Byggnorm 1975 (2nd Edition); Chapter 27: Timber Construction

INTERNATIONAL STANDARDS ORGANISATION

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

4-103-1 Paper 1 A Proposal for Undertaking the Preparation of an International Standard on Timber Structures - International Standards Organisation

5-103-1 Comments on the Report of the Consultation with Member Bodies concerning ISO/TS/P129 - Timber Structures - Dansk Ingeniorforening

7-103-1 ISO Technical Committees and Membership of ISO/TC 165

8-103-1 Draft Resolutions of ISO/TC 165

JOINT COMMITTEE ON STRUCTURAL SAFETY

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

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

CIB PROGRAMME, POLICY AND MEETINGS

1-105-1 Paper 1 A note on international organisations active in the field of utilisation of timber - P Sonnemans

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

10-105-1 The Work of CIB-W18 Timber Structures - J G Sunley

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

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

CIB-W18/10-4-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

BUCKLING STRENGTH OF PLYWOOD - RESULTS OF TESTS AND RECOMMENDATIONS TO CALCULATIONS

by

J Dekker, J Kuipers and H Ploos van Amstel
Stevin Laboratory, Delft
Netherlands

MEETING TEN

VANCOUVER, CANADA

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Rapport 4 - 78 - 2 Onderzoek tc - 36

Buckling strength of plywood

Results of tests and recommendations for calculations,
by ir J. Dekker, ir. J. Kuipers and ir. H. Ploos van Amstel.

januar 1978

Buckling strength of plywood

Results of tests and recommendations for calculations, by ir. J. Dekker, ir. J. Kuipers and ir. H. Ploos van Amstel.

1. Summary

Tests have been carried out on 100 specimens of Canadian Oregonpine-plywood to verify that a reasonable good agreement exists between the buckling theories and the real behaviour of plywood. From load-deflection curves values for a critical buckling strength can be determined, which are in good agreement with theoretical values in the case of simply-supported edges. A clamped boundary condition could not be realised in such a way that the theoretical values were approximated. For design purposes this condition should not be presumed.
Attention has been paid to combinations of normal and shear stresses on the basis of theoretical considerations. This leads to proposals for the scope of design- and calculation-recommendations, which were not worked out in detail here.

2. The investigation

2.1. Object

The object of the investigation is to verify that plywood follows the usual buckling theories and that the use of the different mechanical properties as laid down in design guide lines lead to sufficient accuracy, or in any case to a sufficient degree of safety, in the prediction of the buckling strength.
A further goal of the investigation is to deduce safe design rules for the calculation of structures and structural parts, where plywood is loaded in compression and/or shear in its plane.

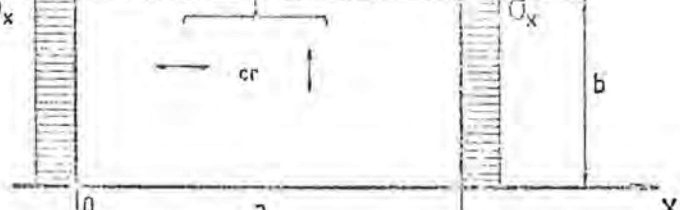


Fig. 1.

2.2. Theory

Plywood has been assumed to behave like an orthotropic linear elastic material.

On the basis of the differential equation of the rectangular orthotropic plate together with an assumed plane after buckling an equilibrium state can be found if such a plate is loaded in its plane, having certain boundary conditions along its sides.

If for a rectangular plate, simply supported along its four sides, the curved plane after buckling is

$$w = A_{mn} \sin \frac{m\pi}{a} x \sin \frac{n\pi}{b} y,$$

the smallest value of the stress σ_x in fig. 1, necessary to accomplish this equilibrium state is found to be

$$\sigma_{xcr} = \frac{4\pi^2}{b^2 t} \left(N_x \frac{m^2 b^2}{4a^2} + N_{xy} \frac{n^2}{2} + N_y \frac{n^4 a^2}{4m^2 b^2}\right),$$

where $N_x = \frac{E_x t^3}{12(1 - \nu_x \nu_y)}$; $N_y = \frac{E_y t^3}{12(1 - \nu_x \nu_y)}$; $N_{xy} = \frac{1}{2} GI_w + \frac{1}{2} (\nu_x N_y + \nu_y N_x)$

and where m and n are the number of half-waves in X- and Y-direction respectively.

If $\frac{a}{b} = \alpha$; $\alpha_\nu = \alpha \sqrt[4]{\frac{N_y}{N_x}}$; $\eta = \frac{N_{xy}}{\sqrt{N_x N_y}}$

the formula for the critical stress becomes

$$\sigma_{xcr} = \frac{4\pi^2}{b^2 t} \sqrt{N_x N_y} \left(\frac{m^2}{4\alpha_\nu^2} + \frac{1}{2} \eta n^2 + \frac{n^4 \alpha_\nu^2}{4m^2} \right) = K \frac{4\pi^2}{b^2 t} \sqrt{N_x N_y}$$

where K = "buckling factor".

Generally in the not-loaded Y-direction of the plate only one half-wave will be developed; in that case n = 1 and

$$K = \frac{m^2}{4\alpha_\nu^2} + \frac{\eta}{2} + \frac{\alpha_\nu^2}{4m^2}$$

Values of K have been given in fig. 2.

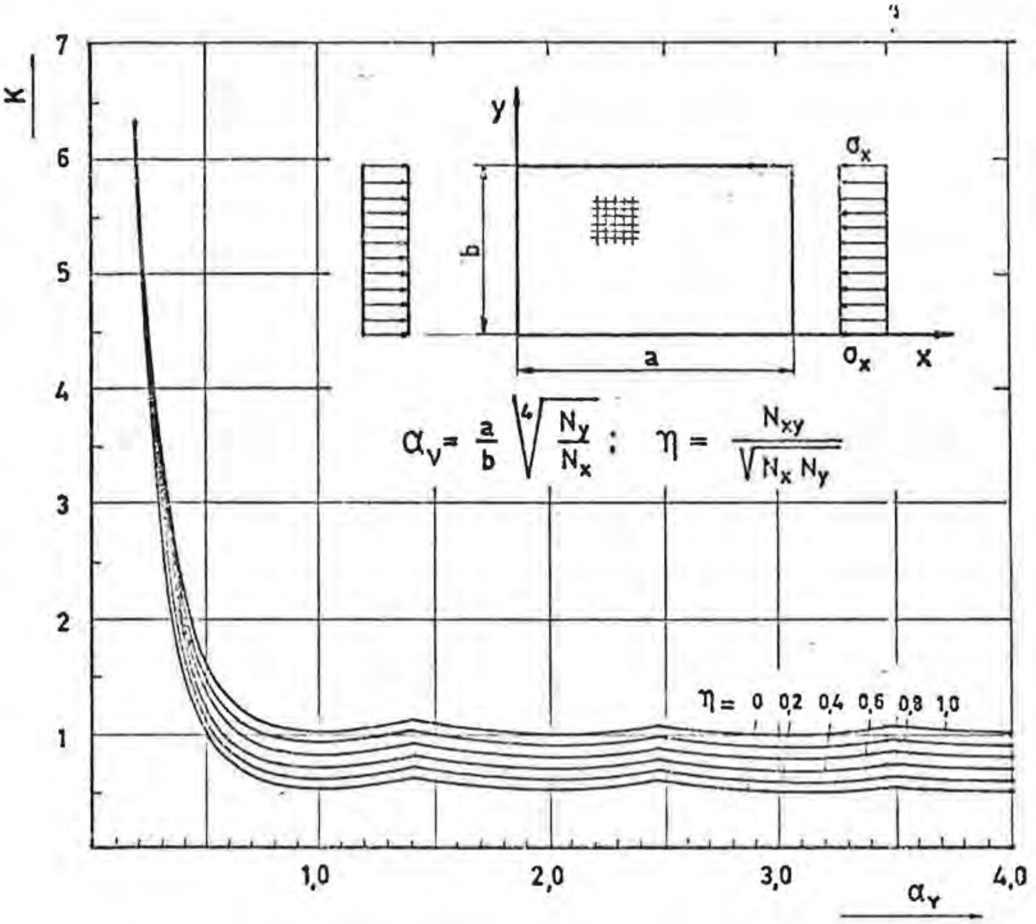


Fig. 2. Values of K for plates with all sides simply supported [3].[1)]

For rectangular plates with clamped edges in X-direction Lekhnitskii has given a solution from which follows a critical stress (n = 1)

$$\sigma_{cr} = \frac{4\pi^2}{b^2 t}\sqrt{N_x N_y}\left(\frac{m^2}{4\alpha_\nu^2} + \frac{2}{3}n + \frac{4}{3}\frac{\alpha_\nu^2}{m^2}\right) = K\frac{4\pi^2}{b^2 t}\sqrt{N_x N_y}$$

For this case values of

$$K = \frac{m^2}{4\alpha_\nu^2} + \frac{2n}{3} + \frac{4\alpha_\nu^2}{3m^2}$$

have been given in fig. 3.

2.3. Testprogram

To control the validity of the theories for plywood, specimens of two thicknesses (8 resp. 13 mm) and of different dimensions have been tested. Data of the test program is been given in table 1.

1) Numbers in [..] refer to lit. on page 44.

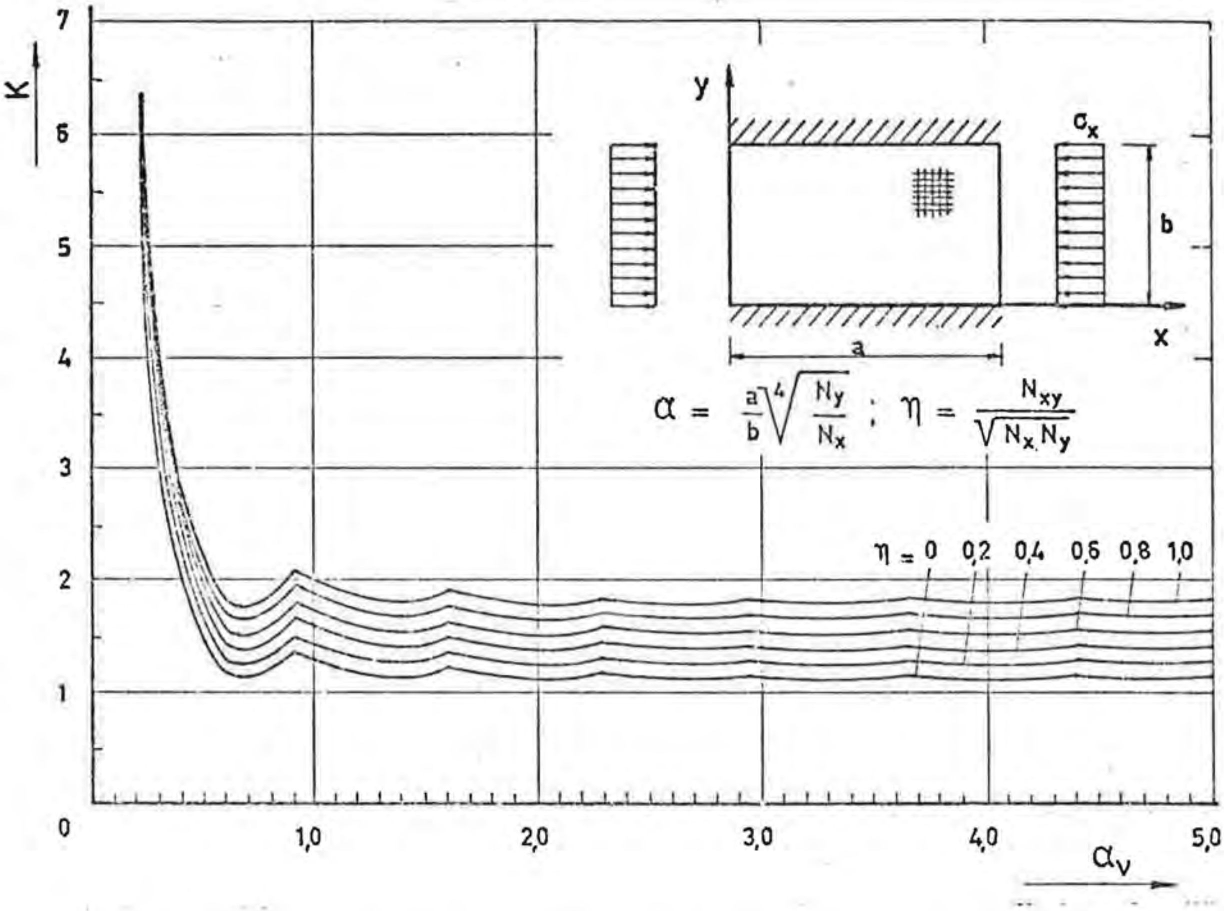


Fig. 3. Values of K for plates with sides //x-axis clamped in; the sides //y-axis are simply supported. [3]

Table 1. Testprogram

b mm		α = a/b (see fig. 1)									number of variables
400	tests // grain → $\sigma_{//}$	½	1	1½	2	2½	3	3½	4	4½	9
	tests ⊥ grain → $\sigma_{\perp}$	"	"	"	"	"	"	"	"	"	6
600	tests // grain → $\sigma_{//}$	"	"	"	"	"	"	-	-	-	6
	tests ⊥ grain → $\sigma_{\perp}$	"	"	"	"	-	-	-	-	-	4
											25

Each test specimen is made in two thicknesses - 8 and 13 mm - and furthermore two kinds of supports were used: simply supported along all sides and clamped sides in X-direction combined with simply supported sides in Y-direction. In this way 25 x 2 x 2 = 100 buckling tests were done.

In all cases Canadian Oregon Pine-plywood, of the quality Select Sheating, Ext. 1 was used. The test specimens were stored in the unconditioned laboratoryhall for several months; the moisture content at test was 8 to 10%.

From each test specimen the thickness t, the length a and the width b were measured, furthermore the stiffness properties

$$N_x = \frac{E_x t^3}{12(1 - \nu_x \nu_y)} \; ; \; N_y = \frac{E_y t^3}{12(1 - \nu_x \nu_y)} \quad \text{and}$$

$$N_{xy} = \frac{1}{2}\, GI_w + \frac{1}{2}(\nu_x N_y + \nu_y N_x) \approx \frac{1}{6}\, Gt^3$$

were determined.

With these quantities the values of the governing factors

$$\alpha_\nu = \alpha \sqrt[4]{\frac{N_y}{N_x}} \quad \text{and}\; \eta = \frac{N_{xy}}{\sqrt{N_x N_y}} \quad \text{for all single panels could}$$

be calculated, as well as the critical stresses.

2.4. Test set-up

All bending tests to determine values of N_x and N_y were carried out by direct loading of the individual panels and measuring the deflections (see foto's).

N_{xy} was found by a four-point loading system according to Nadaï (fig. 4), from which can be calculated

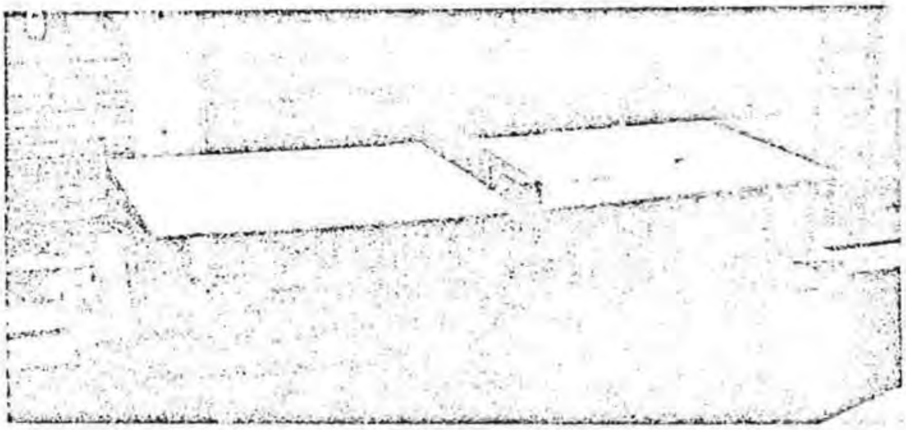
$$G = \frac{3 l_1 l_2 F}{w t^3} \quad \text{, and hence} \quad N_{xy} = \frac{1}{6}\, Gt^3.^{1)}$$

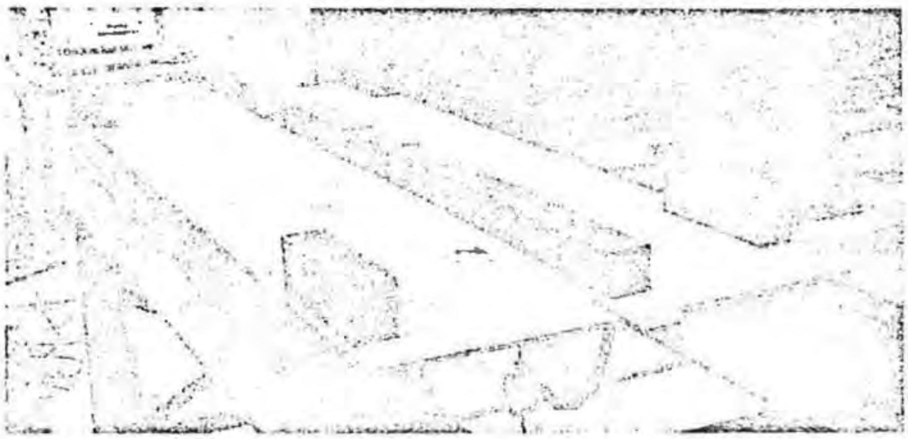
Especially in these tests only small loads and deformations were used to reach sufficient accuracy.

In order to avoid too great differences between l_1 and l_2, N_{xy} could only be determined for the square test panels in the program. For the other test panels N_{xy} had to be determined for representative parts

1) Larsen, H.J. draws attention to the fact that this test set-up is not the most suitable for G inplane of plies in a layered system, with mixed veneers.

of the whole plywood panel where the test specimens were made from.





Bending tests for the determination of N_x and N_y

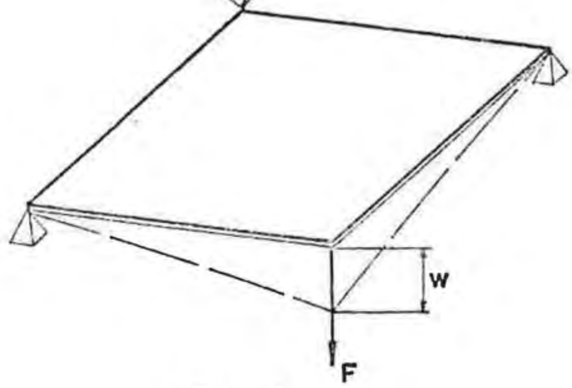
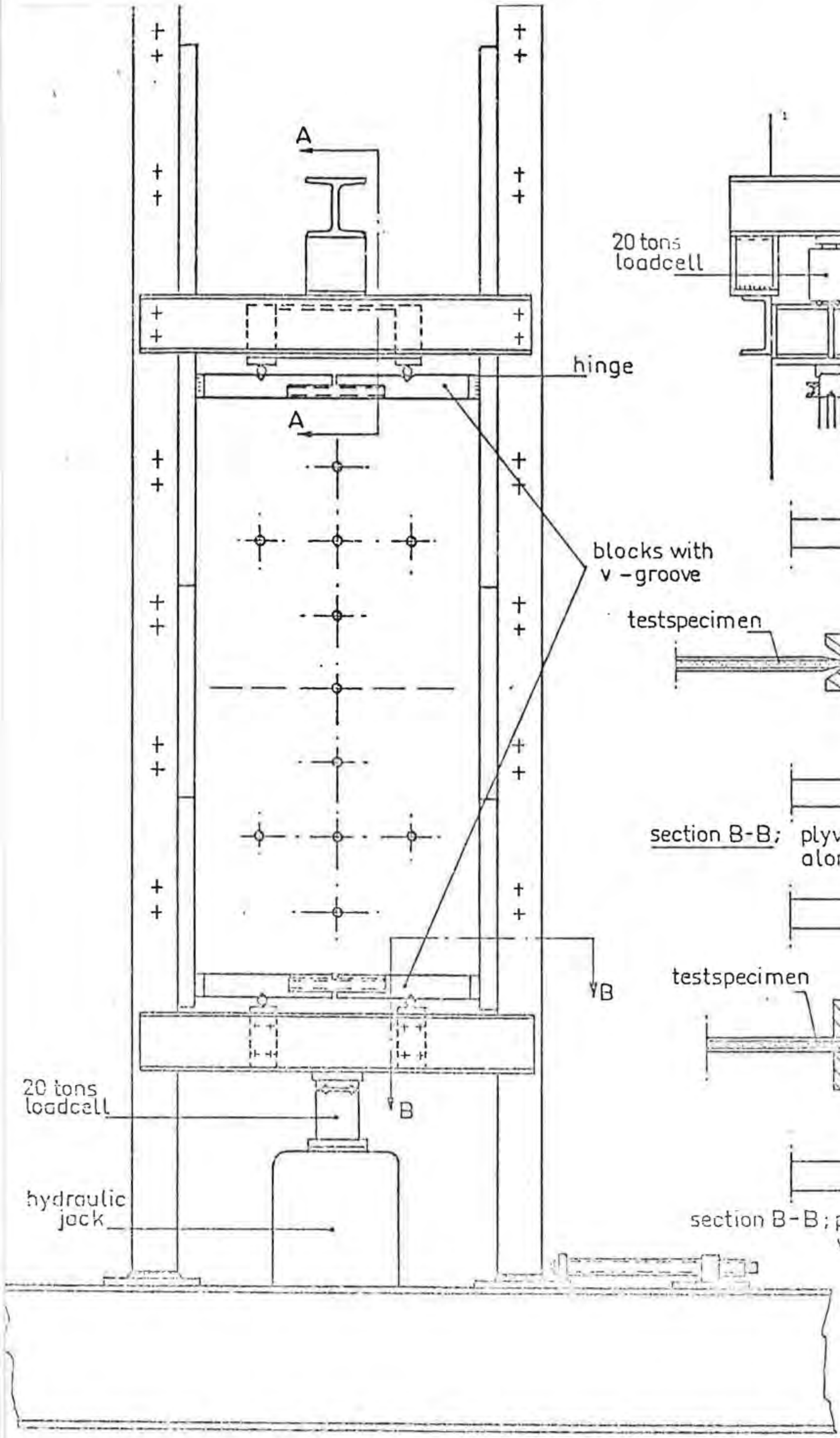
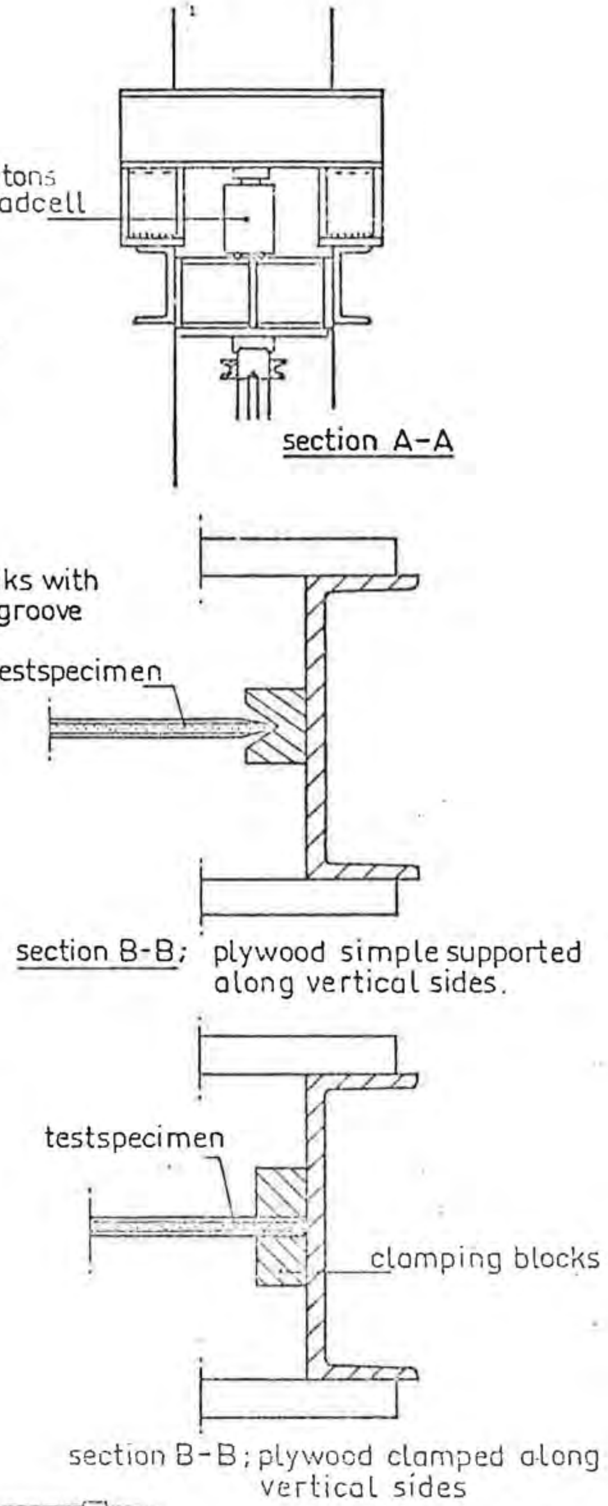


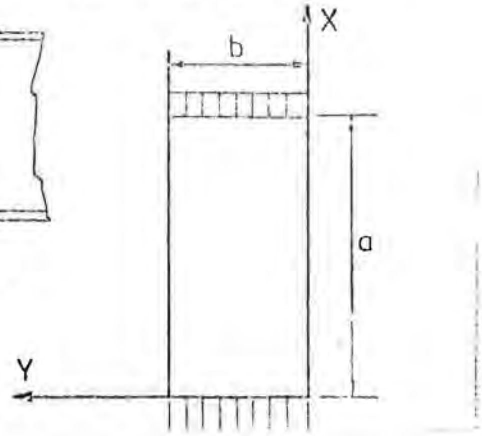
Fig. 4.

The buckling tests were carried out in a special designed frame. The set-up and some details are given in fig. 5. The load was given by a





load calls top & bottom to detect load losses along the sides



hydraulic jack, via a balanced beam to two blocks with a V-groove, where the panel rested in. The upper side had the same construction. At the direct loaded underside and at the upper side the load respectively the reaction force was measured, a difference being possible due to friction along the sides of the panel.

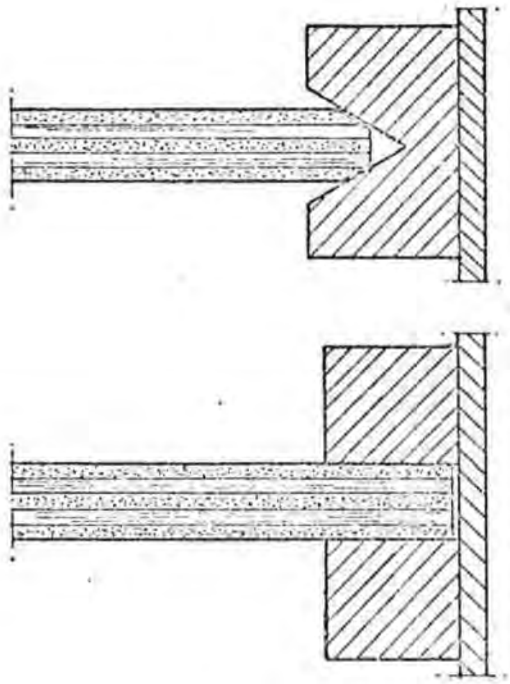
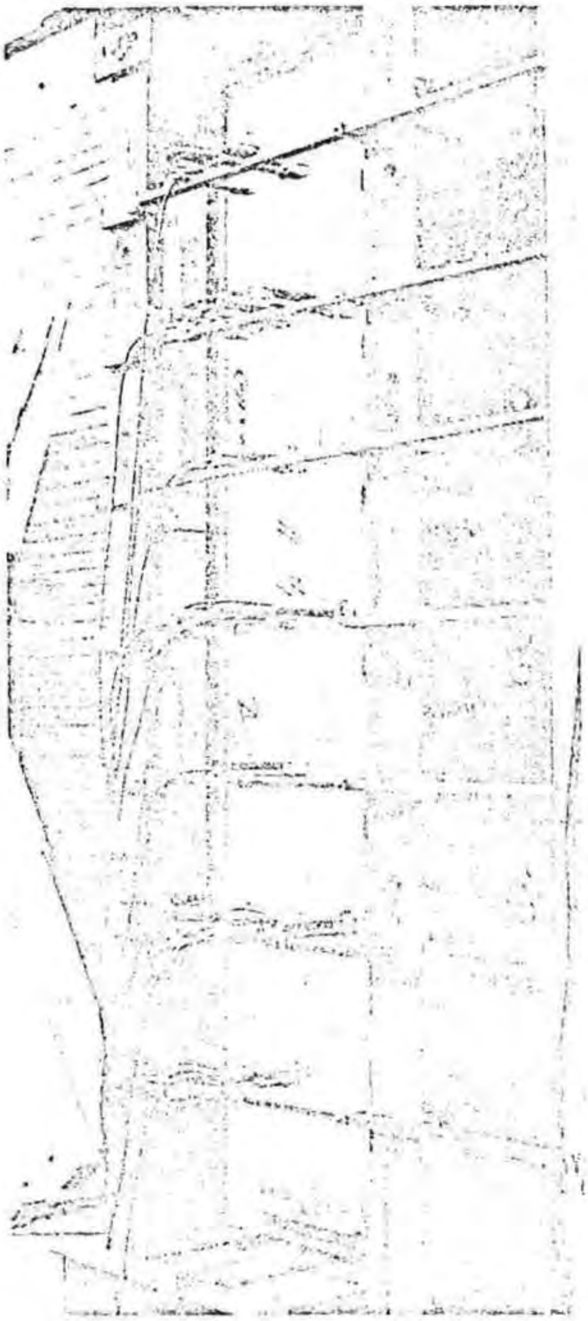
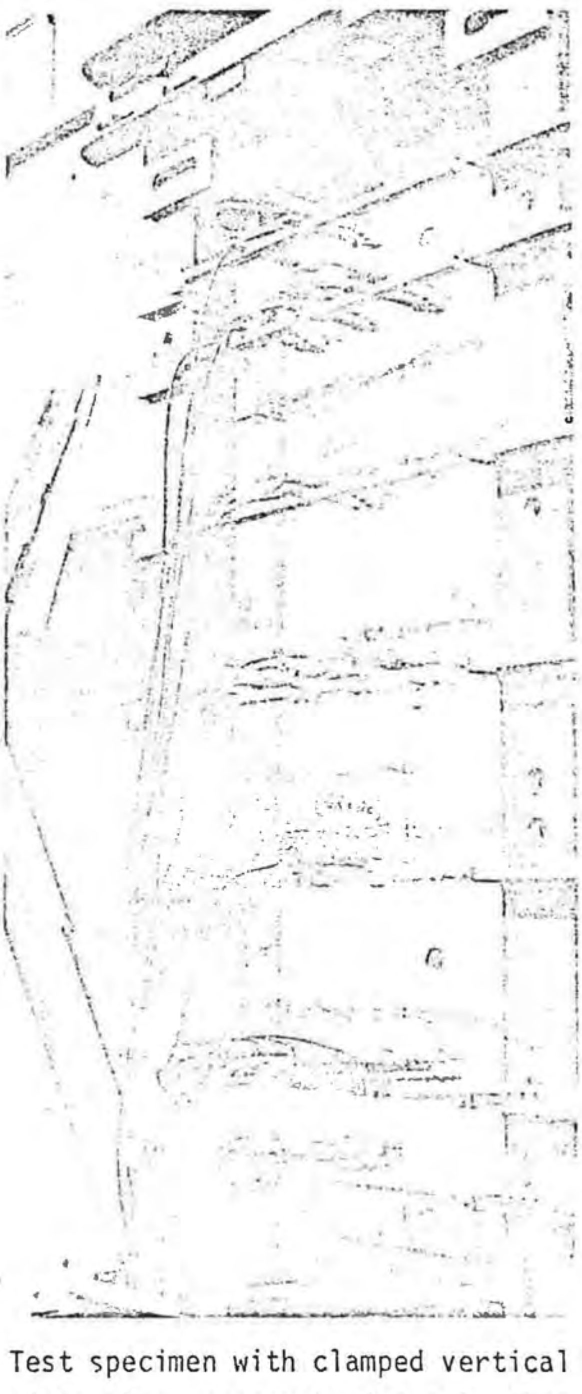


Fig. 6.

For the simple supported panels the sides in X-direction had a V-groove, where the somewhat tapered panel-sides were held. The panels to be tested with clamped sides in X-direction were held between steel clamping blocks, which were tightened with screw-clamps to make the clamping effective.
Depending on the dimensions of the panels the deflections normal to the plane were measured at several places by inductive displacement transducers (see photo's).

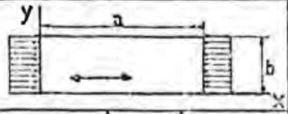


Simply supported test specimen, loaded parallel to direction of face grain. $\alpha = 4$; 3 halfwaves.



Test specimen with clamped vertical sides Face veneer horizontal, that is to the direction of the load. $\alpha = 3$; 4 halfwaves.

Table 2 Stiffness properties; α_ν- and η- values. Simply supported panels loaded // grain of face veneers.

			α								
			½	1	1½	2	2½	3	3½	4	4½
400	8	t(mm)	8.15	8.10	7.83	8.15	7.95	7.98	8.03	8.00	8.15
		N_x	170	315	424	697	579	593	524	780	834
		N_y	79	43	94	89	94	90	98	96	95
		N_{xy}	68	77	63	77	78	77	63	77	77
		α_ν	0.41	0.61	1.03	1.19	1.59	1.87	2.30	2.37	2.61
		η	0.59	0.66	0.32	0.31	0.33	0.33	0.28	0.28	0.27
	13	t(mm)	12.1	12.0	12.0	11.9	11.9	11.9	12.2	11.9	12.0
		N_x	911	915	1705	1846	1838	1818	1819	1925	2095
		N_y	580	479	528	439	492	362	522	404	451
		N_{xy}	206	228	229	206	206	206	229	206	206
		α_ν	0.45	0.85	1.12	1.39	1.80	2.00	2.56	2.71	3.07
		η	0.28	0.34	0.24	0.23	0.23	0.25	0.24	0.23	0.21
600	8	t(mm)	7.45	8.48	7.56	7.55	7.58	7.51			
		N_x	434	455	535	581	584	538			
		N_y	39	34	40	39	32	37			
		N_{xy}	73	71	54	54	73	73			
		α_ν	0.28	0.52	0.79	1.02	1.21	1.53			
		η	0.56	0.57	0.37	0.36	0.53	0.52			
	13	t(mm)	12.3	12.2	12.2	12.2	12.2	12.2			
		N_x	1176	1698	1613	1472	1838	1937			
		N_y	371	491	472	536	434	432			
		N_{xy}	245	248	272	272	245	245			
		α_ν	0.38	0.73	1.10	1.55	1.74	2.06			
		η	0.37	0.27	0.31	0.31	0.27	0.27			

Note: 1) N_x and N_y is determined for each specimen separately; N_{xy} sometimes was determined for the plywood panel from which the specimen were taken.

2) N_x, N_y and N_{xy} in kNmm^2/mm

3) The values of N_x and N_y show great differences, especial if α is small. This can partly be ascribed to the fact that the E-values were measured in a 3-points bending test, where the load had a relative great length compared to the span (resp. 50 mm and 160 mm).

Table 3 Stiffness properties; α_ν- and η-values. Panels with clamped sides in X-direction loaded // grain of face veneers

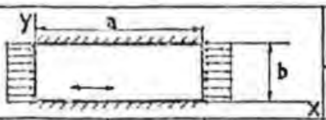
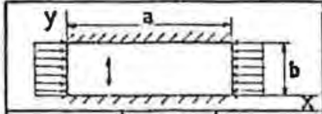
			α								
			½	1	1½	2	2½	3	3½	4	4½
400	8	t(mm)	7.95	7.85	8.21	7.98	8.47	7.83	7.95	7.85	8.05
		N_x	241	501	436	449	496	437	487	748	549
		N_y	91	62	81	72	76	67	64	71	78
		N_{xy}	95	59	95	57	55	57	93	94	93
		α_ν	0.39	0.59	0.98	1.27	1.56	1.87	2.10	2.22	2.76
		η	0.61	0.33	0.51	0.32	0.28	0.33	0.53	0.41	0.45
	13	t(mm)	12.2	12.0	12.2	12.7	12.0	12.6	12.3	12.0	12.2
		N_x	765	1069	2448	1891	1420	1875	2261	2555	2155
		N_y	603	467	660	729	620	640	591	377	572
		N_{xy}	289	184	289	233	180	238	306	206	306
		α_ν	0.47	0.81	1.08	1.58	2.03	2.29	2.50	2.48	3.23
		η	0.43	0.26	0.23	0.20	0.19	0.22	0.26	0.21	0.28
600	8	t(mm)	7.58	7.85	8.44	7.85	7.86	7.90			
		N_x	269	502	461	177	474	568			
		N_y	73	67	79	82	82	95			
		N_{xy}	85	91	55	94	85	91			
		α_ν	0.26	0.60	0.96	1.65	1.61	1.91			
		η	0.61	0.50	0.29	0.78	0.43	0.39			
	13	t(mm)	12.0	12.0	12.0	12.0	12.1	12.0			
		N_x	645	1435	1072	1698	1409	1810			
		N_y	640	474	601	384	620	480			
		N_{xy}	183	177	183	206	183	177			
		α_ν	0.50	0.75	1.29	1.38	2.0	2.15			
		η	0.28	0.21	0.23	0.26	0.20	0.19			

Table 4 Stiffness properties; α_ν- and η-values. Simply supported panels loaded ⊥ grain of face veneers.

			α								
			½	1	1½	2	2½	3	3½	4	4½
400	8	t(mm)	7.58	7.63	7.75	7.73	7.80	7.98			
		N_x	73	102	92	95	107	117			
		N_y	349	382	348	328	328	375			
		N_{xy}	53	63	63	63	63	63			
		α_ν	0.73	1.39	2.09	2.71	3.30	4.01			
		η	0.33	0.32	0.35	0.36	0.34	0.30			
	13	t(mm)	12.1	12.0	12.2	12.3	12.3	12.4			
		N_x	354	523	530	498	557	559			
		N_y	1567	1207	1464	1356	1276	950			
		N_{xy}	229	259	229	229	229	229			
		α_ν	0.72	1.23	1.93	2.56	3.08	3.43			
		η	0.31	0.33	0.26	0.28	0.27	0.31			
600	8	t(mm)	7.43	8.49	7.54	7.53					
		N_x	53	40	41	43					
		N_y	524	551	494	575					
		N_{xy}	54	74	54	54					
		α_ν	0.89	1.92	2.78	3.3					
		η	0.32	0.50	0.38	0.34					
	13	t(mm)	12.4	12.4	12.4	12.3					
		N_x	429	418	579	515					
		N_y	1247	1524	1569	1497					
		N_{xy}	272	242	272	272					
		α_ν	0.65	1.38	1.92	2.91					
		η	0.37	0.30	0.29	0.31					

Table 5 Stiffness properties; α_ν- and η-values. Panels with clamped sides in X-direction loaded $\perp$ grain of face veneers.

			α								
			½	1	1½	2	2½	3	3½	4	4½
400	8	t(mm)	7.95	7.85	7.95	7.90		7.83			
		N_x	63	102	98	82		116			
		N_y	429	461	423	351		368			
		N_{xy}	91	71	91	57		57			
		α_ν	0.81	1.46	2.16	3.63		4.00			
		η	0.55	0.33	0.45	0.34		0.28			
	13	t(mm)	12.6	12.0	12.3	12.8		12.6			
		N_x	298	388	649	866		831			
		N_y	1422	1489	2420	1349		1702			
		N_{xy}	238	150	323	238		238			
		α_ν	0.74	1.40	5.60	2.23		3.59			
		η	0.37	0.20	0.26	0.22		0.20			
600	8	t(mm)	7.75	7.85	7.80	7.73					
		N_x	75	61	83	119					
		N_y	312	493	401	425					
		N_{xy}	55	94	55	55					
		α_ν	0.71	1.68	2.22	2.75					
		η	0.36	0.54	0.30	0.24					
	13	t(mm)	12.0	12.1	12.0	12.0					
		N_x	372	470	698	650					
		N_y	1060	2950	928	999					
		N_{xy}	183	206	183	183					
		α_ν	0.64	1.58	1.61	2.23					
		η	0.29	0.17	0.23	0.23					

3. Test results

3.1. Stiffness properties

Tables 2 to 5 give values of N_x, N_y and N_{xy} for the test specimens. From all these panels some mean properties were as given in table 6.

Table 6. Mean values of $N_{//}^{1)}$, $N_{\perp}^{1)}$ and N_{xy} (in kNmm^2/mm)

plywood t	$N_{//}$			$N_{\perp}$			N_{xy}		
	mean	stand. dev.	coeff var.	mean	stand. dev.	coeff. var.	mean	stand. dev.	coeff. var.
8 mm	464	137	0.30	76	24	0.32	71	15	0.21
13 mm	1578	490	0.31	523	121	0.23	226	44	0.19

1) In this table $N_{//}$ and $N_{\perp}$ are used. In tables 2 and 3 is $N_{//} \equiv N_x$ and $N_{\perp} \equiv N_y$. In table 4 and 5 is $N_{//} \equiv N_y$ and $N_{\perp} \equiv N_x$.

The mean ratio $\frac{N_{//}}{N_{\perp}}$ is 6.1 for the 8 mm and 3.0 for the 13 mm plywood; this means that $\sqrt[4]{\frac{N_{//}}{N_{\perp}}} = 1.57$ and 1.33 respectively. The mean values of $\frac{N_{xy}}{\sqrt{N_{//}N_{\perp}}} = 0.67$ resp. 0.26. In the investigation the values of α and η were calculated for each panel individually and used to predict the buckling strength; these values of α_{ν} and η are given also in tables 2 to 5.

3.2. Buckling tests

3.2.1. General remarks

During the first series of tests it was noticed that long test panels, showing more than one buckling field, buckled first at the direct-loaded side. A second buckling field occurred more to the reaction-side of the panel, at a higher load at the hydraulic jack. (cf. fig. 5), etc. After this had happened during the first series the second load cell at the reaction-side was placed.
At the following tests a varying difference between the loads at under and upper side was measured, due to the friction along the supported sides in X-direction. This difference has a tendency to grow with increasing length of the specimens, although the variance is too high to make this tendency significant. For individual test specimens the load at the upper side ranged from $F_{up} = 0.96\ F_{low}$ to $F_{up} = 0.45\ F_{low}$.

These differences occurred at failure; before reaching this stage they were generally smaller.

It could be shown that it is most probably not too far from reality to assume that the normal stress σ_x in the panel varies linearly from σ_{low} to σ_{up}. It was therefore decided that "the" stress on the panel could be found as the mean value of σ_{low} and σ_{up}.

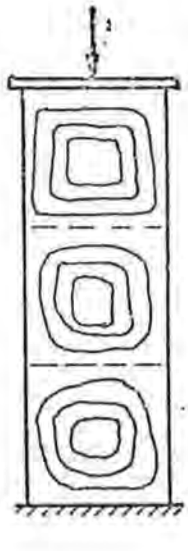


Fig. 7.

Furthermore the critical stresses for the individual bucklingfields were determined and the mean value thereof was given as the buckling stress for the panel.

3.2.2. Determination of the critical stresses

The critical stresses themselves have been determined from the measurements of the deformations. Examples of the relation between the stress and the displacements normal to the plane have been given in fig. 8.

It can clearly be seen that right from the beginning of the loading procedure the originally existing excentricities[1)] lead to increasing deformations, which after some time follow a nearly straight line.

It is also clear that during the tests the deformation was gradually growing; there was no sign of a sudden occurrence of buckling as was found in the investigation described in [3]. This different behaviour could possibly be the effect of other boundary conditions. In [3] the "simple support" was made as shown in fig. 9, where half-round laths were glued to the plywood. In the investigation described here such a simple support was made as shown in fig. 5 and 6.

In most cases the failing load or ultimate load is much higher than the critical load. It must be added however, that this "post-buckling behaviour" was much stronger with longer plates than with shorter ones, and that this effect disappeared with the very short plates, where instead the compressive strength was the governing property.

1) Initial excentricities were measured from ½ to 9% from the shortest side of the plywood panel.

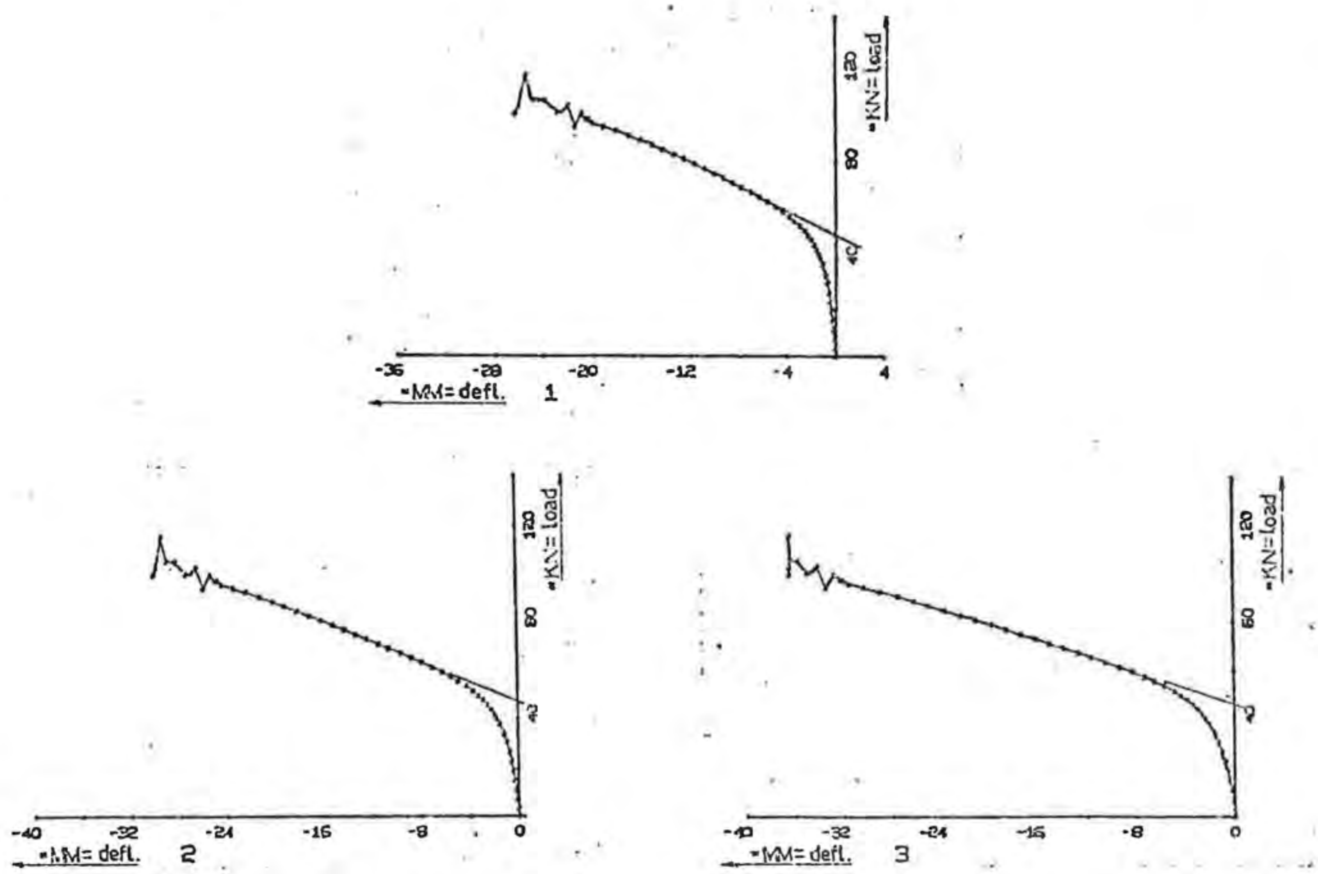


Fig. 8.

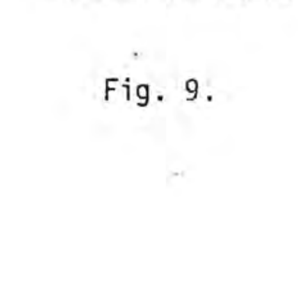


Fig. 9.

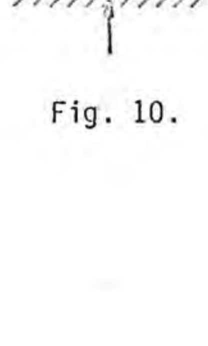


Fig. 10.

In most cases the highest load caused a failure in the plywood plate like in fig. 10.
In tables 7 to 10 the values of σ_{cr} and σ_{ult} are listed.

4. Discussion of the test results

In the tables 7 to 10 the first two lines for a certain type of panel are headed "TGH". The TGH[1] gives among other design information E and G-values for plywood.

For 8 mm plywood it gives $E_{//} = 8000\ N/mm^2$

$E_{\perp} = 500\ N/mm^2$

$G = 750\ N/mm^2$.

With face grain in X-direction:

$$\alpha_{\nu 1} = \alpha \sqrt[4]{\frac{E_{\perp}}{E_{//}}} = \alpha \sqrt[4]{\frac{500}{8000}} = 0.5\alpha.$$

$$\eta = \frac{1/6\ Gt^3.12(1 - \nu_x \nu_y)}{t^3 \sqrt{E_{//}E_{\perp}}} = \frac{2.750}{\sqrt{4000000}} = 0.75$$

With face grain in Y-direction:

$$\alpha_{\nu 2} = \alpha \sqrt[4]{\frac{E_{//}}{E_{\perp}}} = \alpha \sqrt[4]{\frac{8000}{500}} = 2\alpha.$$

For 13 mm plywood $E_{//} = 8000\ N/mm^2$

$E_{\perp} = 2500\ N/mm^2$

$G = 750\ N/mm^2$

Now $\alpha_{\nu 1} = \alpha \sqrt[4]{\frac{2500}{8000}} = 0.75\ \alpha$

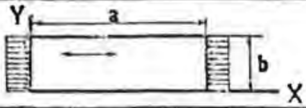
$$\eta = \frac{2 \times 0.75}{\sqrt{20.10^6}} = 0.36$$

$$\alpha_{\nu 2} = \alpha \sqrt[4]{\frac{8000}{2500}} = 1.34\ \alpha$$

---

1) "TGH" = "Tabellen en Grafieken voor Houtconstructies"
= Tables and Graphs for timber structures; [7].

Table 7 Calculated values of σ_{cr} (in N/mm²) and test results.
Simply supported panels loaded // grain of face veneers.



					α: ½	1	1½	2	2½	3	3½	4	4½
	8	cal-cu-la-ted	TGH	α_ν	0.25	0.5	0.75	1.00	1.25	1.50	1.75	2.00	2.25
				σ_{cr}	11.6	3.79	2.44	2.23	2.36	2.44	2.25	2.23	2.25
			meas-urements	α_ν	0.41	0.61	1.03	1.19	1.59	1.87	2.30	2.37	2.61
				σ_{cr}	6.37	3.95	4.15	5.21	5.20	4.76	4.58	5.63	5.59
			test	σ_{cr}	13.37	8.21	4.58	7.15	4.81	6.96	3.88	5.11	3.94
				σ_{ult}	15.1	12.3	11.3	18.0	11.2	16.1	11.3	12.8	10.3
0	13	calc.	TGH	α_ν	0.38	0.75	1.13	1.50	1.88	2.25	2.63	3.00	3.38
				σ_{cr}	30.8	11.8	10.6	11.8	10.6	10.8	10.7	10.6	10.7
			meas-urements	α_ν	0.45	0.85	1.12	1.39	1.80	2.00	2.56	2.71	3.07
				σ_{cr}	21.4	9.50	12.3	13.4	12.2	10.6	12.6	11.5	12.2
		test		σ_{cr}		15.4	11.6	14.6	14.9	10.4	11.6	10.5	12.2
				σ_{ult}	24.1	17.6	19.7	18.6	21.1	18.0	18.6	18.4	18.8
	8	calc.	TGH	α_ν	0.25	0.5	0.75	1.00	1.25	1.50			
				σ_{cr}	5.16	1.69	1.12	1.03	1.08	1.12			
			meas-urements	α_ν	0.28	0.52	0.79	1.02	1.21	1.53			
00				σ_{cr}	7.12	2.08	1.43	1.31	1.45	1.66			
		test		σ_{cr}	10.1	3.37	2.15	0.96	1.04	1.40			
				σ_{ult}	14.7	5.43	6.96	6.24	5.46	5.25			
	13	calc.	TGH	α_ν	0.38	0.75	1.13	1.50	1.88	2.25			
				σ_{cr}	14.0	5.25	4.69	5.25	4.63	4.69			
			meas-urements	α_ν	0.38	0.73	1.10	1.55	1.74	2.06			
				σ_{cr}	13.2	5.77	5.15	5.34	5.11	4.94			
		test		σ_{cr}	13.7	6.53	6.29	5.83	4.76	3.95			
				σ_{ult}	13.8	9.96	13.5	13.1	10.3	10.6			

Table 8 Calculated values of σ_{cr} and test results. Simply supported panels loaded $\perp$ grain of face veneers.

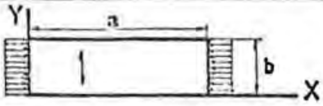
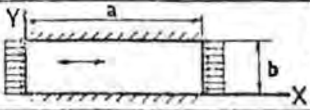
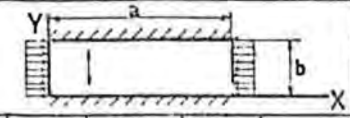
					α								
					½	1	1½	2	2½	3	3½	4	4½
400	8	calc.	TGH	α_ν	1	2	3	4	5	6			
				σ_{cr}	2.23	2.23	2.23	2.23	2.23	2.23			
			meas	α_ν	0.73	1.39	2.09	2.71	3.30	4.01			
				σ_{cr}	3.98	4.98	3.87	3.84	3.86	4.21			
		test		σ_{cr}	6.53	5.96	3.91	4.62	3.53	4.19			
				σ_{ult}	8.40	9.54	9.13	9.88	9.74	10.4			
	13	calc.	TGH	α_ν	0.67	1.34	2.01	2.68	3.35	4.02			
				σ_{cr}	13.1	11.8	10.6	10.6	10.6	10.4			
			meas	α_ν	0.72	1.23	1.93	2.56	3.08	3.43			
				σ_{cr}	11.6	11.4	11.2	10.8	10.8	9.9			
		test		σ_{cr}	14.4	14.9	13.0	12.7	11.8	11.4			
				σ_{ult}	17.4	18.7	15.6	17.9	15.6	16.5			
600	8	calc.	TGH	α_ν	1	2	3	4					
				σ_{cr}	1.03	1.03	1.03	1.03					
			meas	α_ν	0.89	1.92	2.78	3.3					
				σ_{cr}	1.43	1.59	1.33	1.4					
		test		σ_{cr}		1.21	0.90	1.11					
				σ_{ult}	4.83	4.22	5.30	4.65					
	13	calc.	TGH	α_ν	0.67	1.34	2.01	2.68					
				σ_{cr}	5.80	5.25	4.63	4.69					
			meas	α_ν	0.65	1.38	1.92	2.91					
				σ_{cr}	5.74	4.71	4.55	4.73					
		test		σ_{cr}	6.93	5.31	5.14	5.34					
				σ_{ult}	9.30	9.0	10.8	10.9					

Table 9 Calculated values of σ_{cr} and test results. Panels with clamped sides in X-direction, loaded // grain of face veneers.

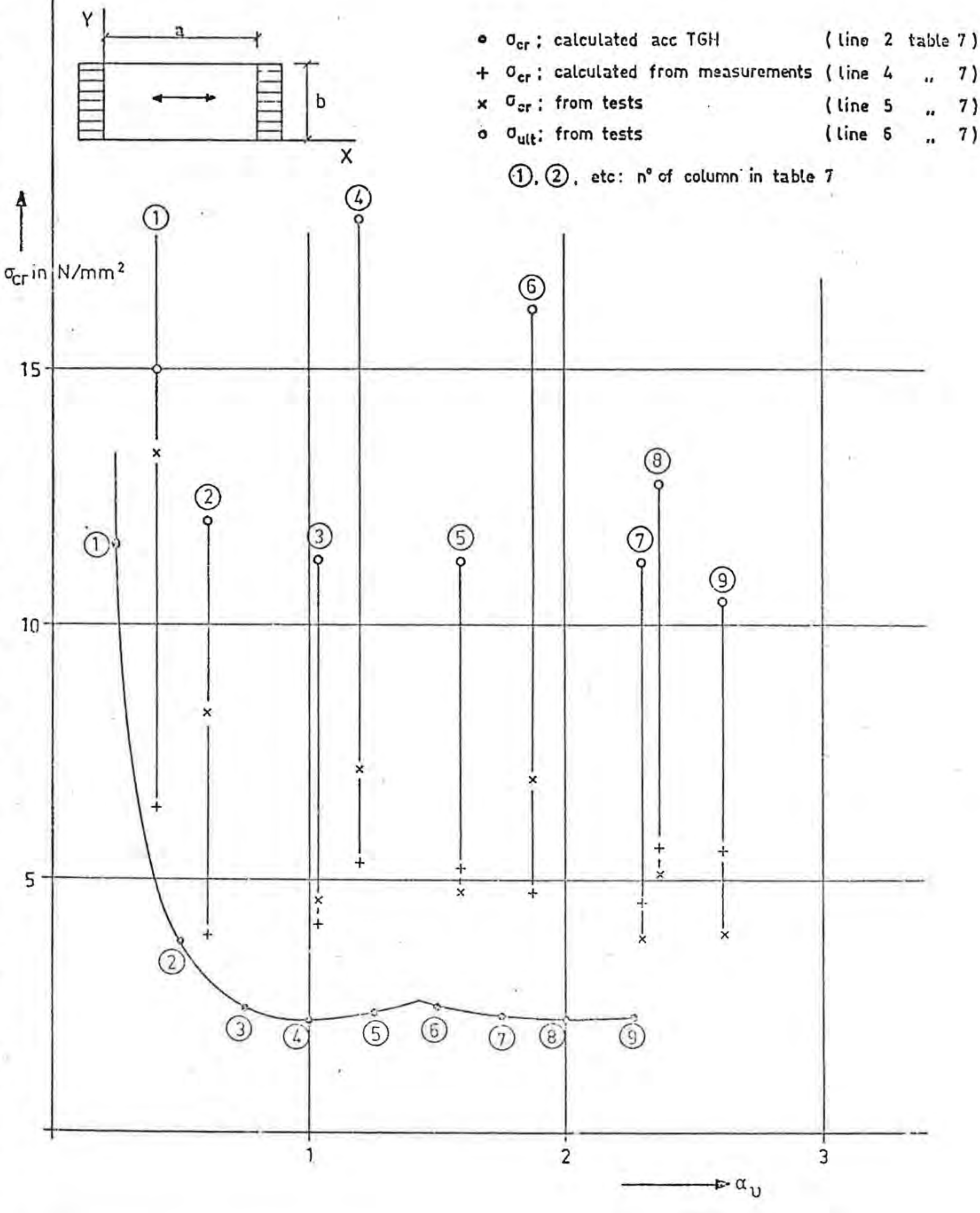


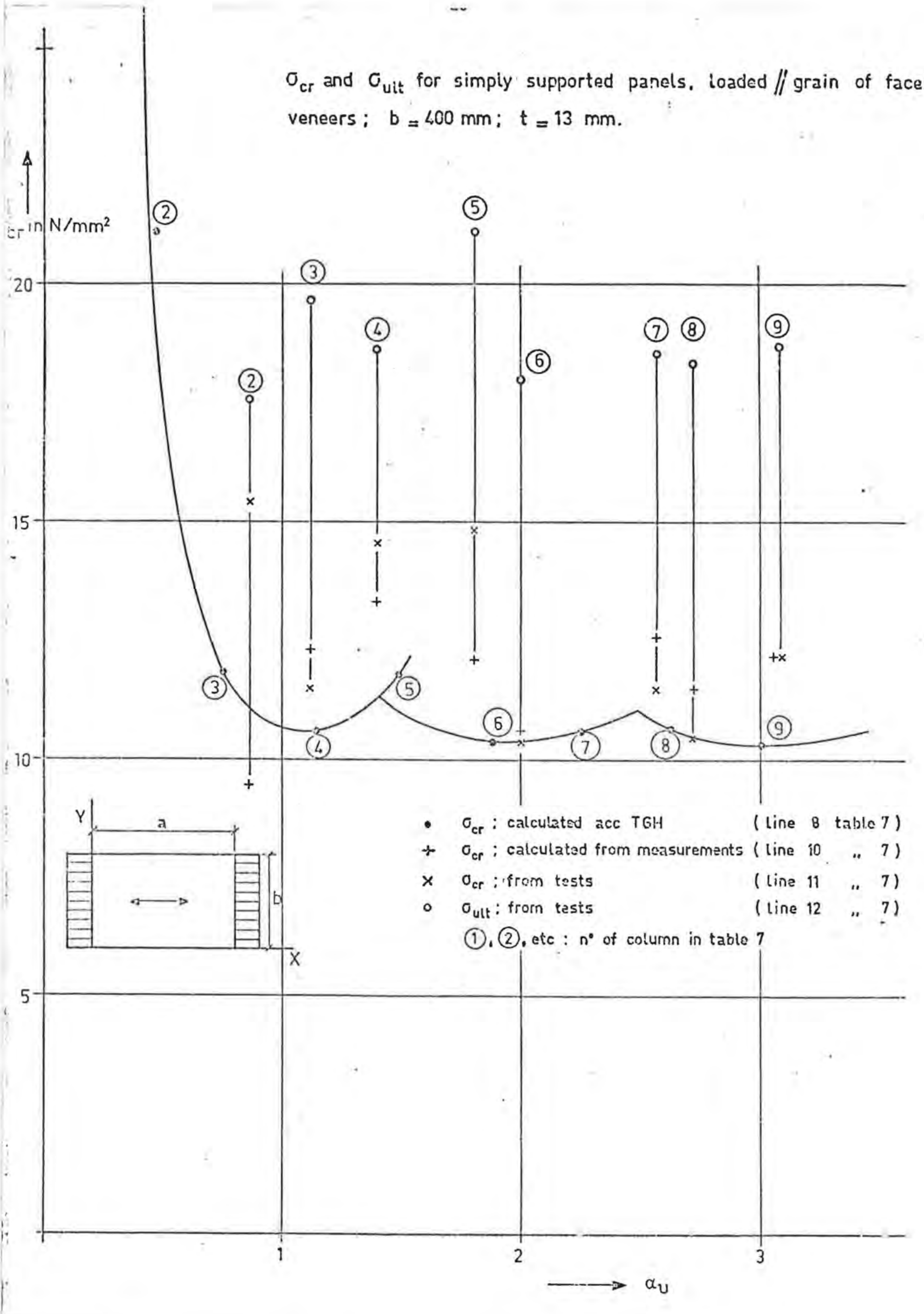
					α								
					½	1	1½	2	2½	3	3½	4	4½
400	8	cal-cu-lated	TGH	α_ν	0.25	0.5	0.75	1.00	1.25	1.50	1.75	2.00	2.25
				σ_{cr}	12.0	4.85	4.48	4.85	4.39	4.48	4.46	4.37	4.48
			meas	α_ν	0.39	0.59	0.98	1.27	1.56	1.87	2.10	2.22	2.76
				σ_{cr}	10.5	7.80	9.58	7.54	7.94	7.42	8.28	10.6	9.22
		test		σ_{cr}	10.1	7.39	4.98	8.07	5.28	4.79	4.75	5.81	5.59
				σ_{ult}	12.8	13.9	14.7	10.2	11.5	11.2	14.2	15.9	13.8
	13	cal-cu-lated	TGH	α_ν	0.38	0.75	1.13	1.50	1.88	2.25	2.63	3.00	3.38
				σ_{cr}	34.1	22.1	22.3	22.1	21.6	22.3	21.5	21.8	21.5
			meas	α_ν	0.47	0.81	1.08	1.58	2.03	2.29	2.50	2.48	3.23
				σ_{cr}	23.4	22.8	35.9	31.1	24.7	28.9	30.9	26.3	30.2
		test		σ_{cr}		14.4	7.52	13.9	10.0	13.7	15.1	13.9	13.9
				σ_{ult}	25.7	21.4	26.3	22.5	20.5	21.1	26.1	28.5	25.3
600	8	cal-cu-lated	TGH	α_ν	0.25	0.5	0.75	1.00	1.25	1.50	1.75	2.00	2.25
				σ_{cr}	3.88	1.68	1.57	1.68	1.48	1.52			
			meas	α_ν	0.26	0.60	0.96	1.65	1.61	1.91			
				σ_{cr}	4.91	3.83	3.91	2.93	4.24	4.58			
		test		σ_{cr}	4.76	3.27	2.79	3.14	2.67	3.20			
				σ_{ult}	6.68	10.4	8.50	9.10	10.6	8.32			
	13	cal-cu-lated	TGH	α_ν	0.38	0.75	1.13	1.50	1.88	2.25			
				σ_{cr}	14.2	8.54	8.66	8.54	8.30	8.54			
			meas	α_ν	0.50	0.75	1.29	1.38	2.0	2.15			
				σ_{cr}	8.93	10.1	9.59	9.80	10.9	11.1			
		test		σ_{cr}	8.86	5.42	6.35	5.82	5.56	6.68			
				σ_{ult}	11.7	13.3	11.6	15.4	12.3	13.9			

Table 10 Calculated values of σ_{cr} and test results. Panels with clamped sides in X-direction, loaded $\perp$ grain of face veneers.

					α					
					½	1	1½	2	2½	3
400	8	calc.	TGH	α_ν	1	2	3	4	5	6
				σ_{cr}	4.85	4.37	4.48	4.37	4.39	4.48
			meas	α_ν	0.81	1.46	2.16	3.63		4.00
				σ_{cr}	8.25	9.56	9.29	7.37		8.68
		test		σ_{cr}	6.60	6.53	5.45	3.7		7.60
				σ_{ult}	8.74	11.1	11.4	6.58		11.9
	13	calc.	TGH	α_ν	0.67	1.34	2.01	2.68	3.35	4.02
				σ_{cr}	21.5	21.5	21.5	21.5	21.5	21.5
			meas	α_ν	0.74	1.40	5.60	2.23		3.59
				σ_{cr}	18.3	20.3	33.5	27.8		30.5
		test		σ_{cr}	12.03	8.58	11.6	11.8		11.0
				σ_{ult}	13.4	13.8	19.4	16.3		14.8
600	8	calc.	TGH	α_ν	1	2	3	4		
				σ_{cr}	1.68	1.47	1.57	1.58		
			meas	α_ν	0.71	1.68	2.22	2.75		
				σ_{cr}	3.03	3.51	3.53	4.20		
		test		σ_{cr}	2.32	2.12	2.35	2.49		
				σ_{ult}	5.68	5.69	7.32	7.35		
	13	calc.	TGH	α_ν	0.67	1.34	2.01	2.68		
				σ_{cr}	8.27	8.27	8.27	8.27		
			meas	α_ν	0.64	1.58	1.61	2.23		
				σ_{cr}	7.75	14.4	10.3	9.86		
		test		σ_{cr}	6.86	7.40	5.26	5.23		
				σ_{ult}	9.30	9.79	10.1	9.58		

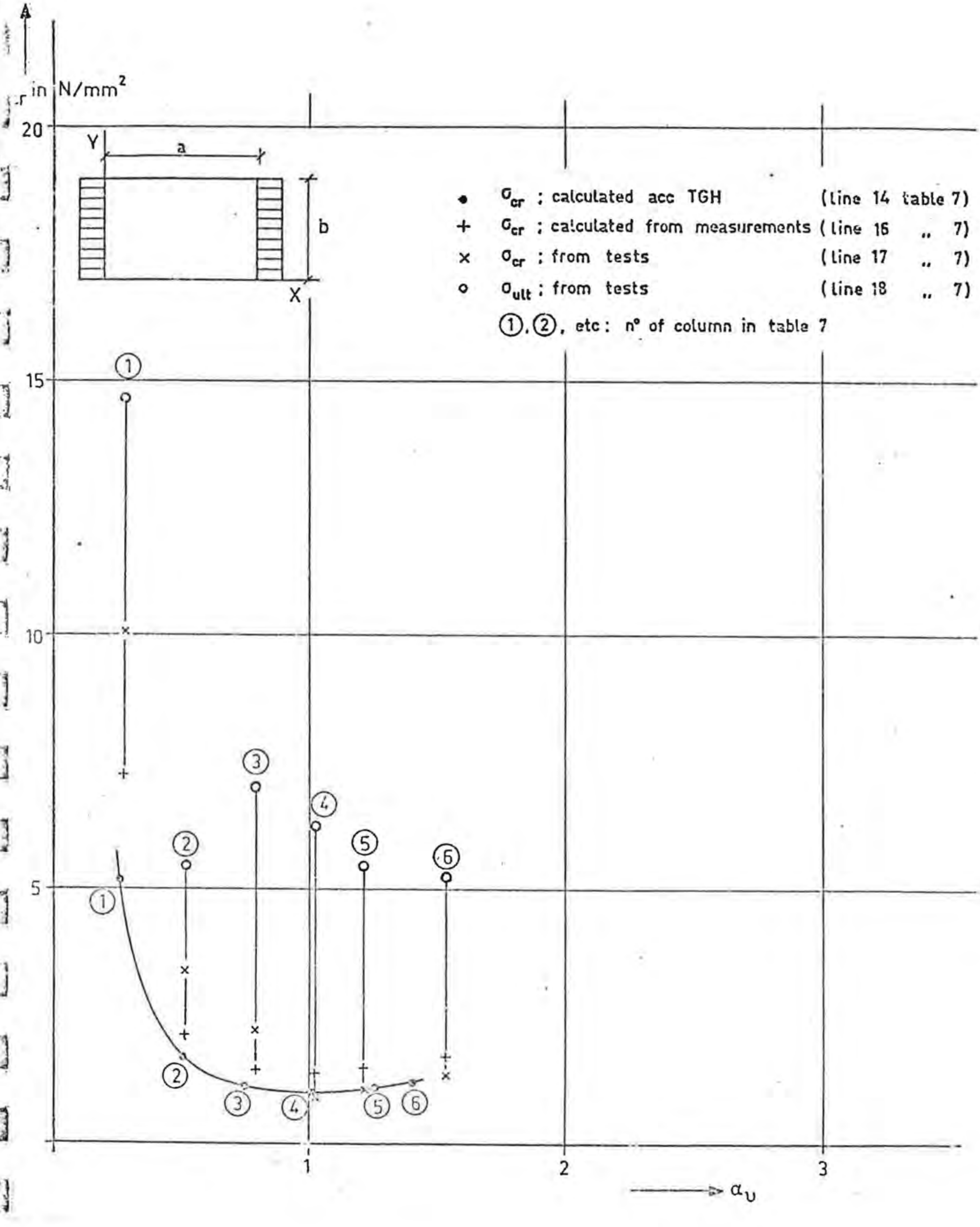
σ_{cr} and σ_{ult} for simply supported panels, loaded // grain of face veneers ; $b = 400$ mm ; $t = 8$ mm.



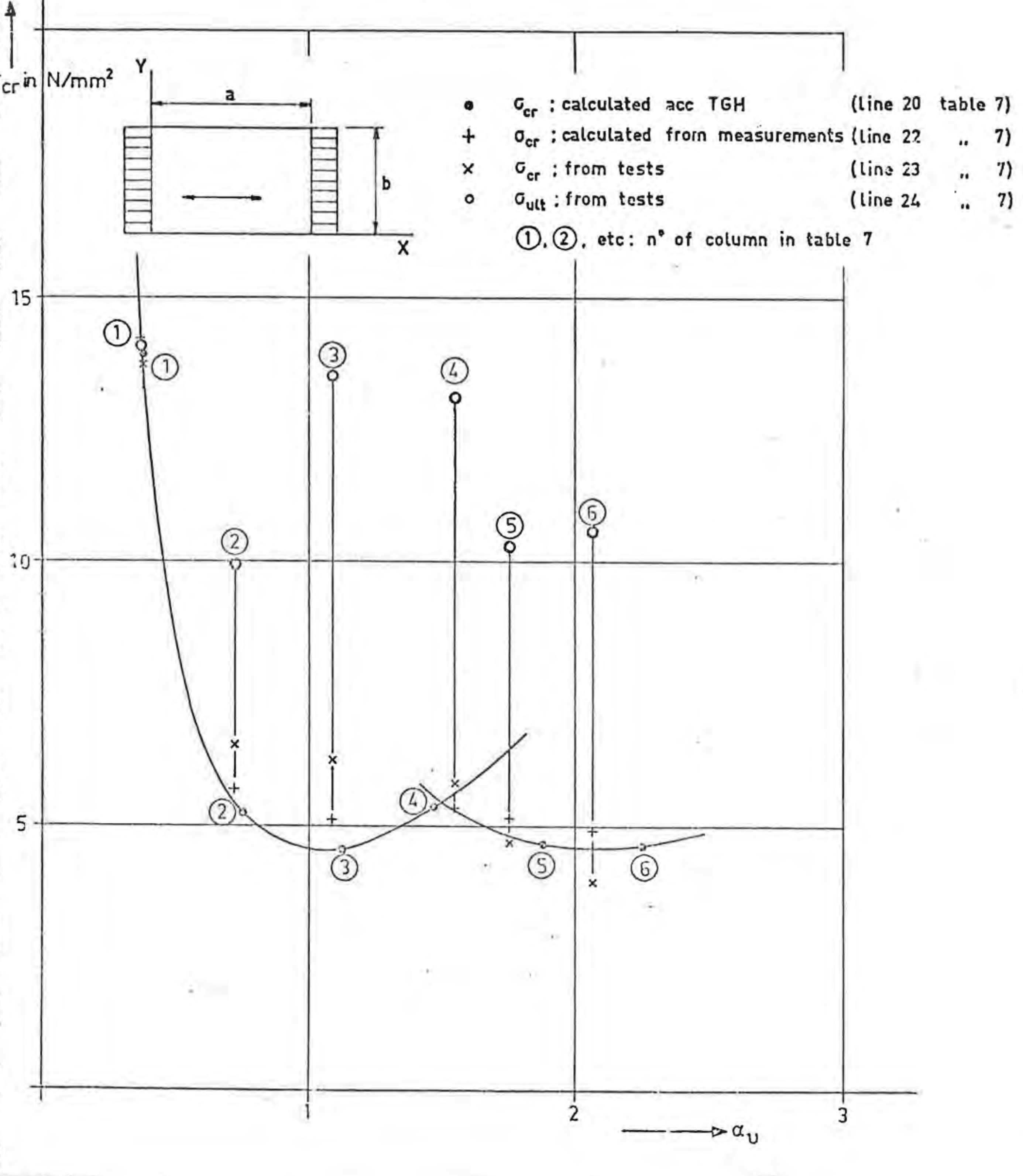


σcr and σult for simply supported panels, loaded // grain of face veneers ; b = 400 mm ; t = 13 mm.
σcr in N/mm²
20
15
10
5
1
2
3
αu
Y
a
b
X
• σcr : calculated acc TGH (line 8 table 7)
+ σcr : calculated from measurements (line 10 " 7)
× σcr : from tests (line 11 " 7)
o σult : from tests (line 12 " 7)
①, ②, etc : n° of column in table 7

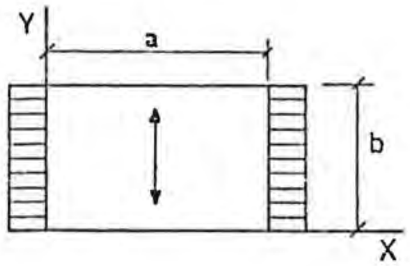
σ_{cr} and σ_{ult} for simply supported panels, loaded // grain of face veneers,
b = 600 mm, t = 8 mm.



σ_{cr} and σ_{ult} for simply supported panels, loaded // grain of face veneers ;
b = 600 mm, t = 13 mm

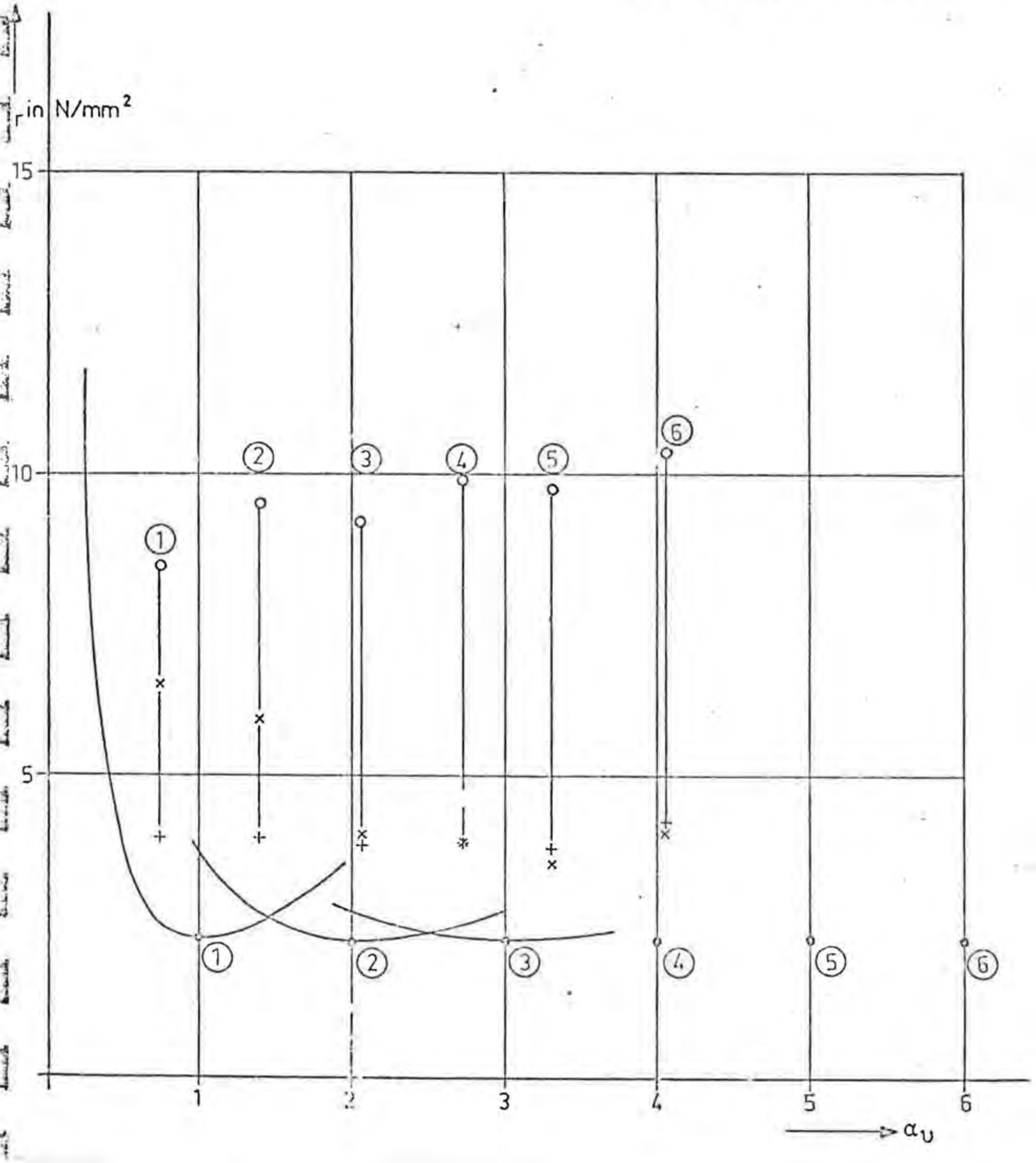


σ_{cr} and σ_{ult} for simply supported panels, loaded ⊥ grain of face veneers; b = 400 mm t = 8 mm

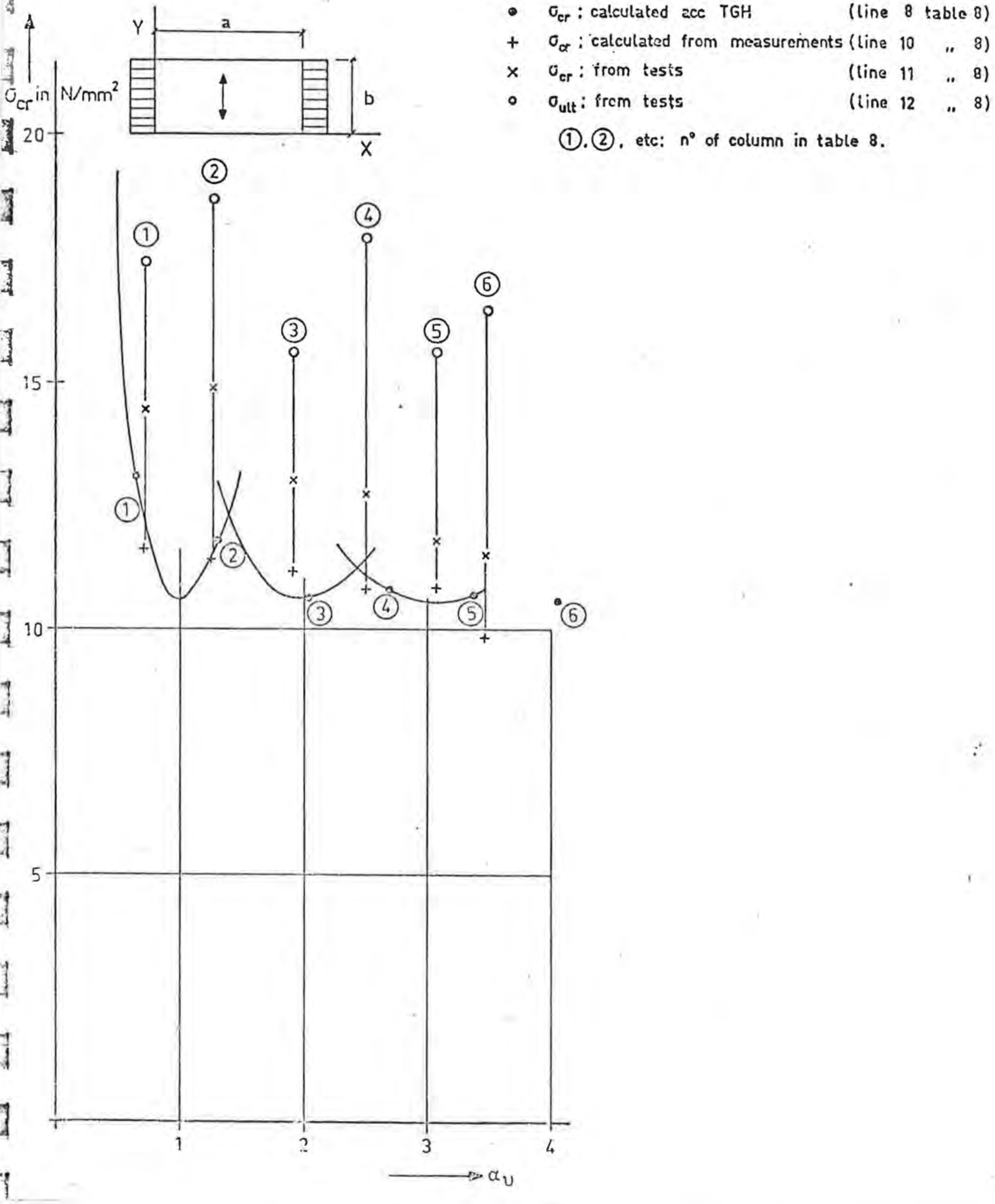


- • σ_{cr} : calculated acc TGH (line 2 table 8)
- + σ_{cr} : calculated from measurements (line 4 .. 8)
- × σ_{cr} : from tests (line 5 .. 8)
- o σ_{ult} : from tests (line 6 .. 8)

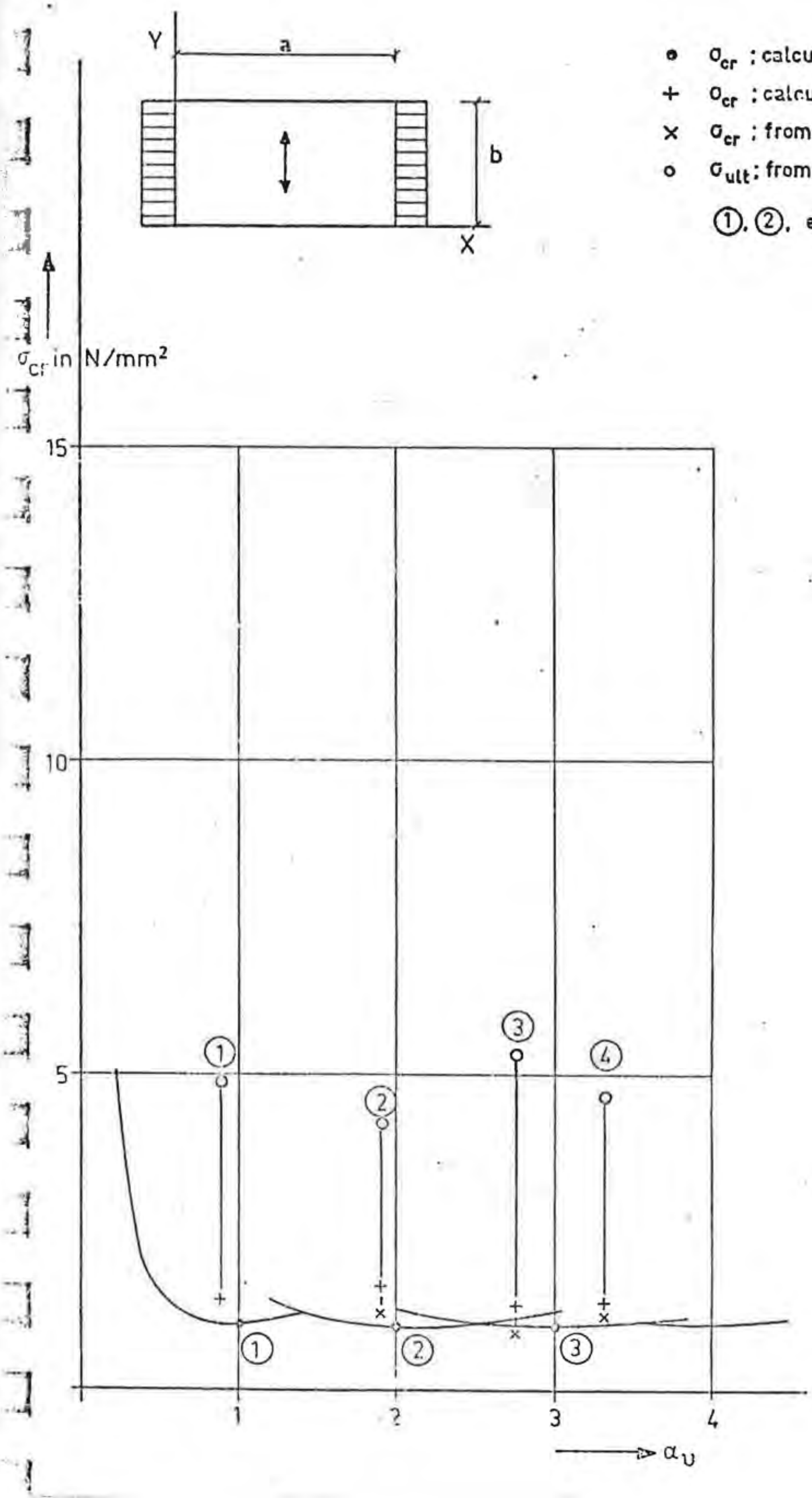
①, ②, etc: n° of column in table 8.



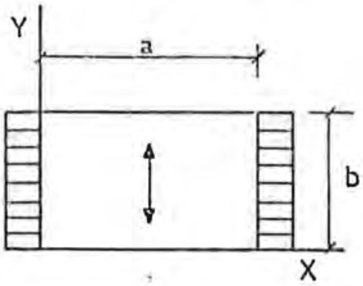
σ_{cr} and σ_{ult} for simply supported panels, loaded ⊥ grain of face veneers; $b = 400$ mm, $t = 13$ mm



σ_{cr} and σ_{ult} for simply supported panels, loaded ⊥ grain of face veneers; $b = 600$ mm ; $t = 8$ mm

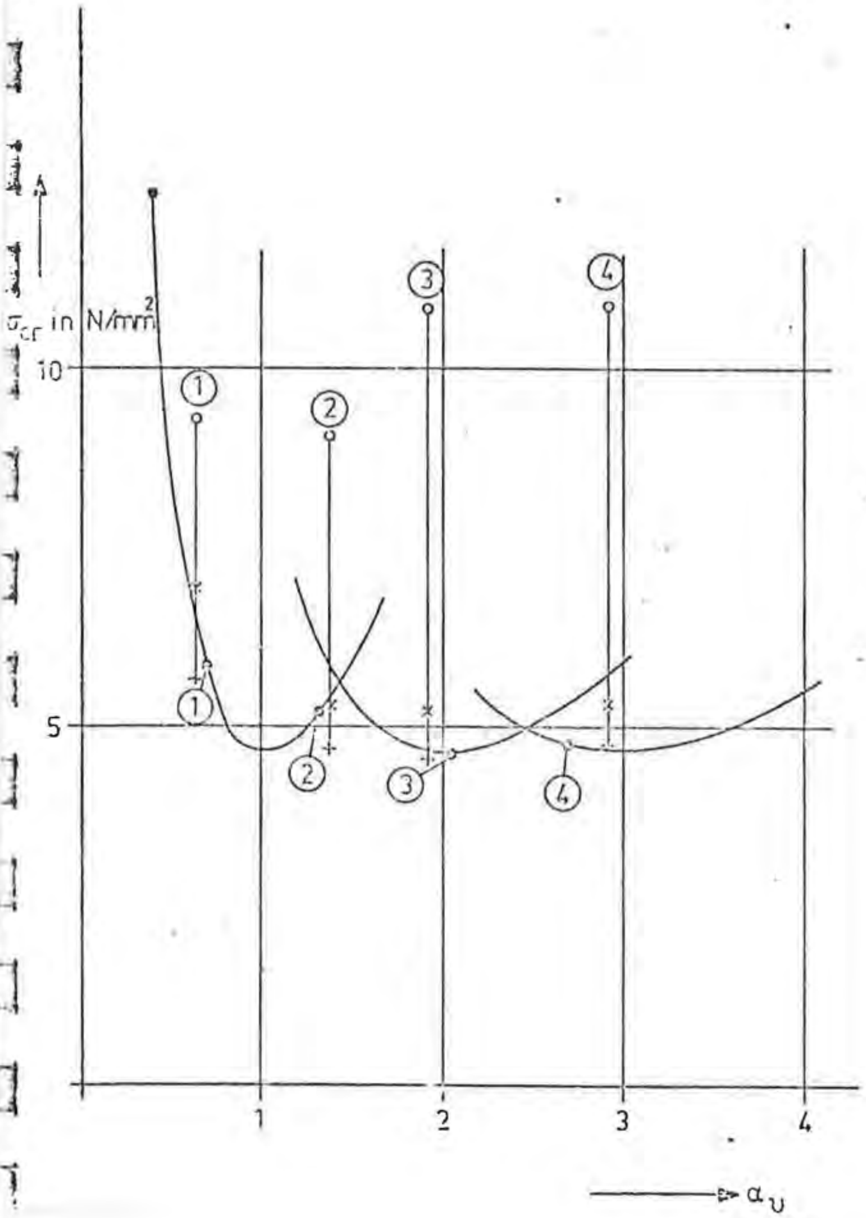


σ_{cr} and σ_{ult} for simply supported panels, loaded $\perp$ grain of face veneers ; $b = 600$ mm, $t = 13$ mm



- σ_{cr} ; calculated from TGH (line 20 table 8)
- $+$ σ_{cr} ; calculated from measurements (line 22 .. 8)
- $\times$ σ_{cr} ; from tests (line 23 .. 8)
- o σ_{ult} ; from tests (line 24 .. 8)

①, ②, etc: n° of column in table 8.



With these values of E, G, α_ν, and η the practical design values of σ_{cr} were calculated. These should be a safe approximation of the real values of σ_{cr}.

The 3^{rd} and 4^{th} line in the tables give calculated values of σ_{cr}, according to measurements; in this case values of σ_{cr} are based upon the measured values of N_x, N_y and N_{xy} of each panel seperately. If the theory is valid for plywood and if in our test set up the boundary conditions are not too bad, these calculated values of σ_{cr} should be in good accordance with the real test values. These real values are listed in the 5^{th} and 6^{th} lines of the table; here σ_{cr} is the critical value determined from the deflections as shown in fig. 8; σ_{ult} is the mean stress reached at the maximum load (post-buckling-strength).

For the simply supported panels the figures 11 to 18 give also the different values. From these figures it becomes clear that the design values of the TGH lead to conservative values of σ_{cr}; they are on the safe side. The mean value of the ratio $\frac{\sigma_{cr;test}}{\sigma_{cr;TGH}}$ is 1.48, with a standard deviation of 0.58; the smallest ratio in the tests was 0.84, the greatest 3.21. In fig. 19 values of $\sigma_{cr;\ test}$ and $\sigma_{cr;TGH}$ are plotted; corresponding values must be expected to lie above the straight 45^o-line, only three results show lower values.

The ratio $\frac{\sigma_{cr;test}}{\sigma_{cr;\ calc.\ meas.}}$ can be used to control if the theory holds for the plywood panels. The calculated values of σ_{cr} are in this case based on the measured quantities N_x, N_y and N_{xy}. The mean value of the ratio $\frac{\sigma_{cr;test}}{\sigma_{cr;calc.\ meas.}}$ is 1.13, with a standard deviation of 0.32; the smallest ratio was 0.68, the largest 2.10. In fig. 20 values of $\sigma_{cr;test}$ and $\sigma_{cr;calc.meas.}$ are plotted; corresponding values should lie on the straight 45^o-line.

Deviations of the expected and the test values may be ascribed to friction in the supports along the sides as described before.

For the plates with clamped edges it appears that the test results are essentially lower than the calculated values. Also from the deformations it becomes clear that the supposed cosinus line between the clamped edges does not occur but that much more a sinus-line is reached. This means that even the rather

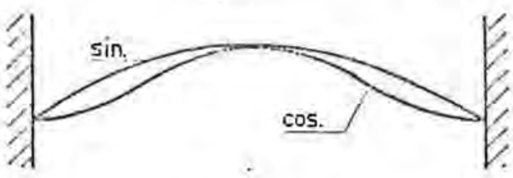


fig. 21.

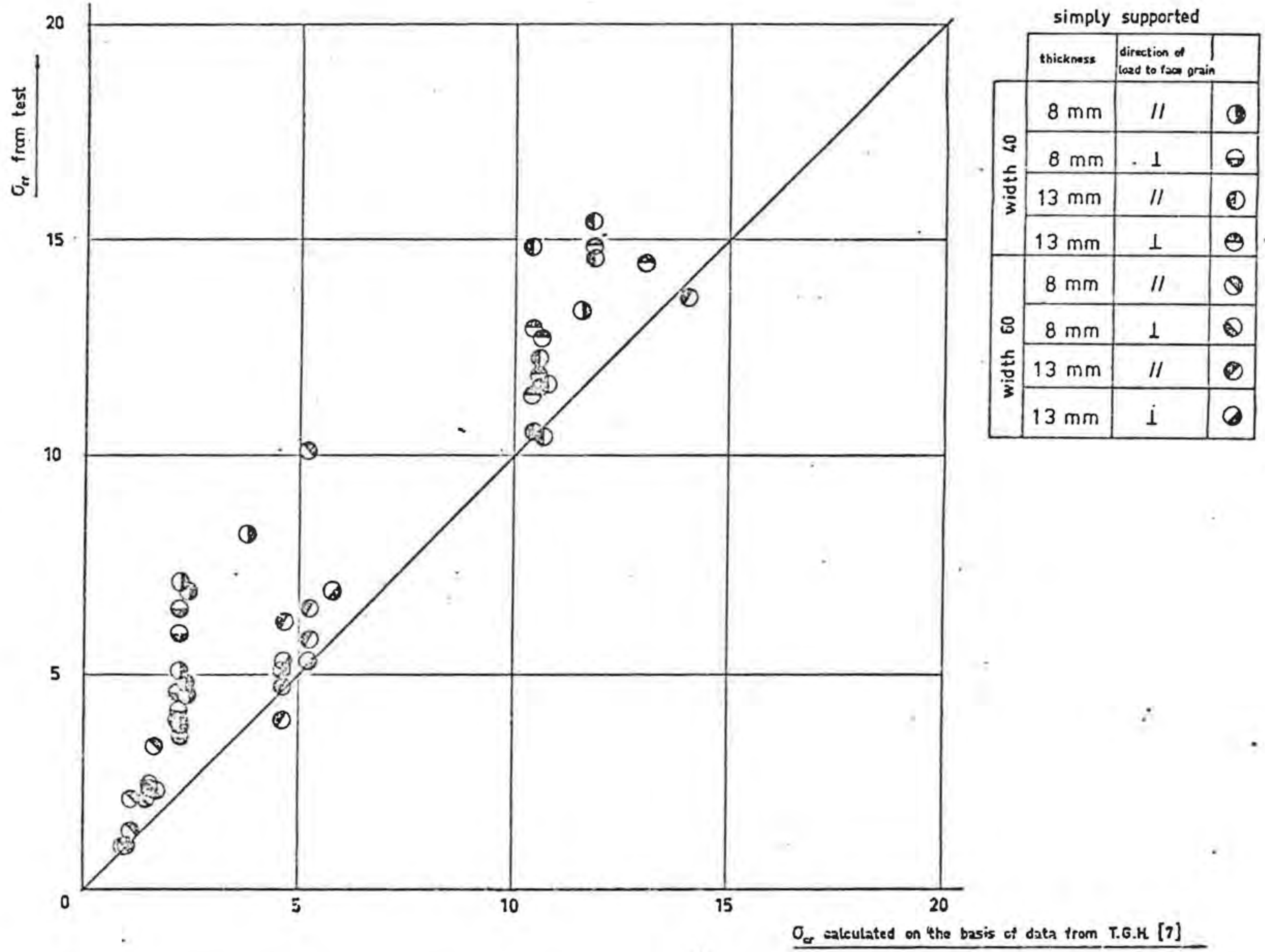

σcr from test
σcr calculated on the basis of data from T.G.H. [7]
0
5
10
15
20
simply supported
thickness
direction of load to face grain
width 40
8 mm //
8 mm ⊥
13 mm //
13 mm ⊥
width 60
8 mm //
8 mm ⊥
13 mm //
13 mm ⊥

fig. 19

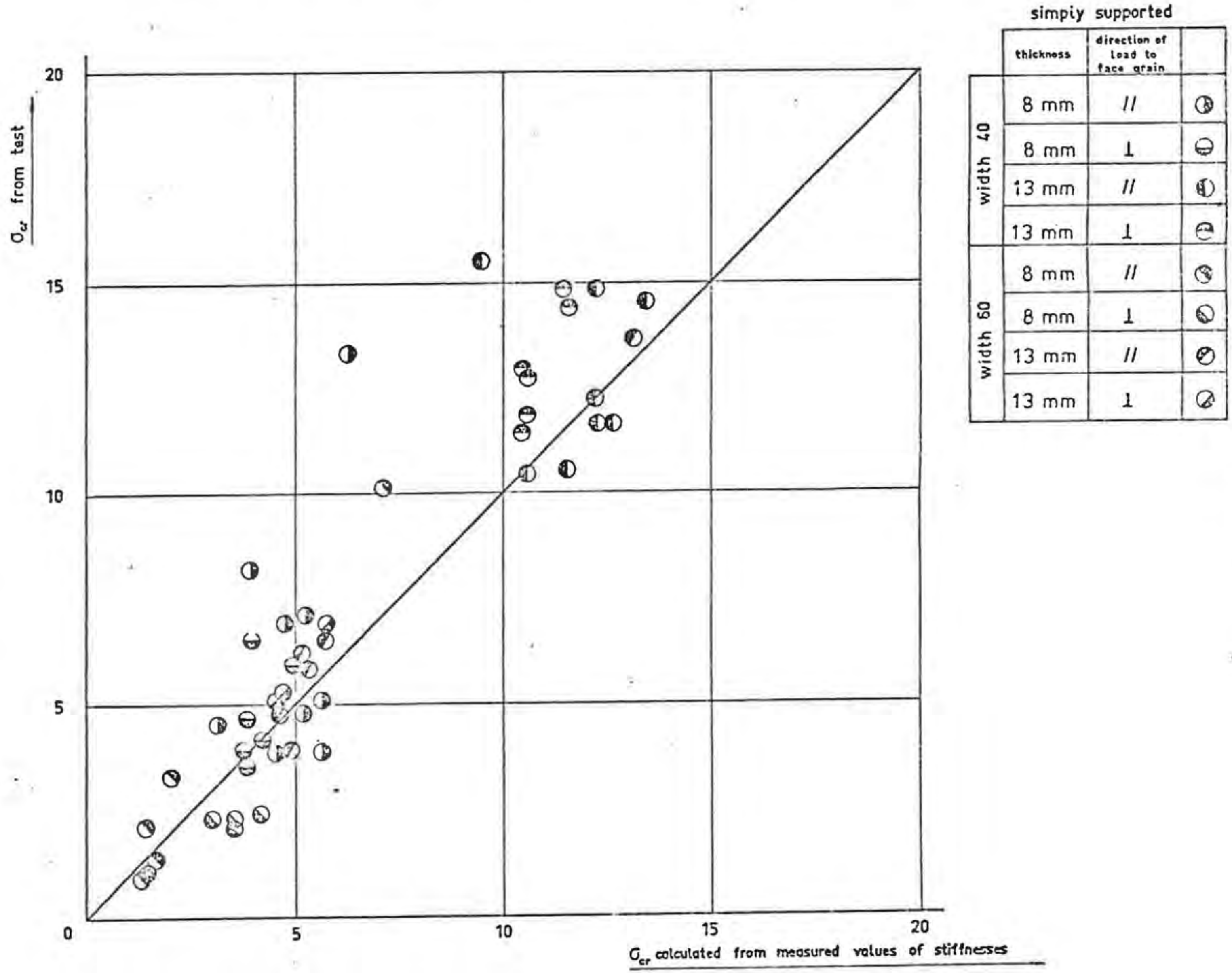

simply supported
thickness
direction of load to face grain
width 40
8 mm //
8 mm ⊥
13 mm //
13 mm ⊥
width 60
8 mm //
8 mm ⊥
13 mm //
13 mm ⊥
σ_{cr} from test
σ_{cr} calculated from measured values of stiffnesses
0
5
10
15
20

fig. 20

heavy clamping pressure along the steel clamping blocks is not enough to realise this theoretical situation. It must be doubted therefore if in practice such clamped edges can be effected. In most cases the critical stresses as well as the ultimate stresses are somewhat higher for the clamped panels than for the simply supported ones.

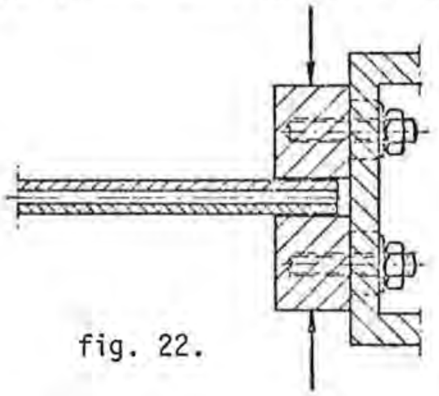


fig. 22.

5. Conclusions

There is a good conformity between the test results σ_{cr} and the theoretical values calculated on the basis of measured dimensions and properties of the panels. This holds for the simply supported panels.
Deviations must partly be ascribed to friction along the edges of the panels. The panels with clamped sides reached not the expected values, most probable due to insufficient clamping. It must therefore be doubted if in practice effective clamping can be realised.
The values of σ_{cr}, calculated on the basis of design values in the TGH are much on the safe side.
Safety is furthermore increased by the fact that the post-buckling-strength is generally much higher than σ_{cr}. Only in very short panels this effect disappeared; in those cases the compressive strength of the plywood determined the strength.

6. Recommendations for design and calculations

Assuming on the basis of the foregoing that the theories developed by several authors are a good tool for the prediction of the behaviour of plywood, the theory has been elaborated for more complex situations. These theoretical results lead to some design rules after simplifications have been made.

6.1. Theoretical values for different loading conditions

In the following it is assumed that a plywood plate can be loaded with normal and with shear stresses along the sides. The theory is limited to combinations of normal stresses which are linear distributed and shear stresses with are uniform along the sides. Both cases will be dealt with seperately and afterwards combinations will be studied.

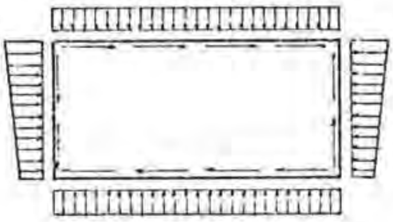


fig. 23.

6.1.1. Normal stresses.

For the orthotropic material it can be shown that the critical stress σ_{xcr} may be calculated following

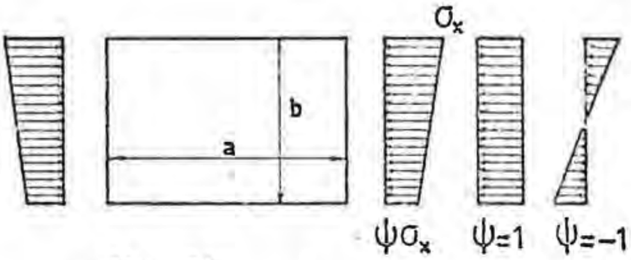


fig. 24

$$\sigma_{xcr} = K\frac{4\pi^2}{b^2t}\sqrt{N_xN_y}.$$

where (cf fig. 24.):

σ_x = greatest value of the compressive stress

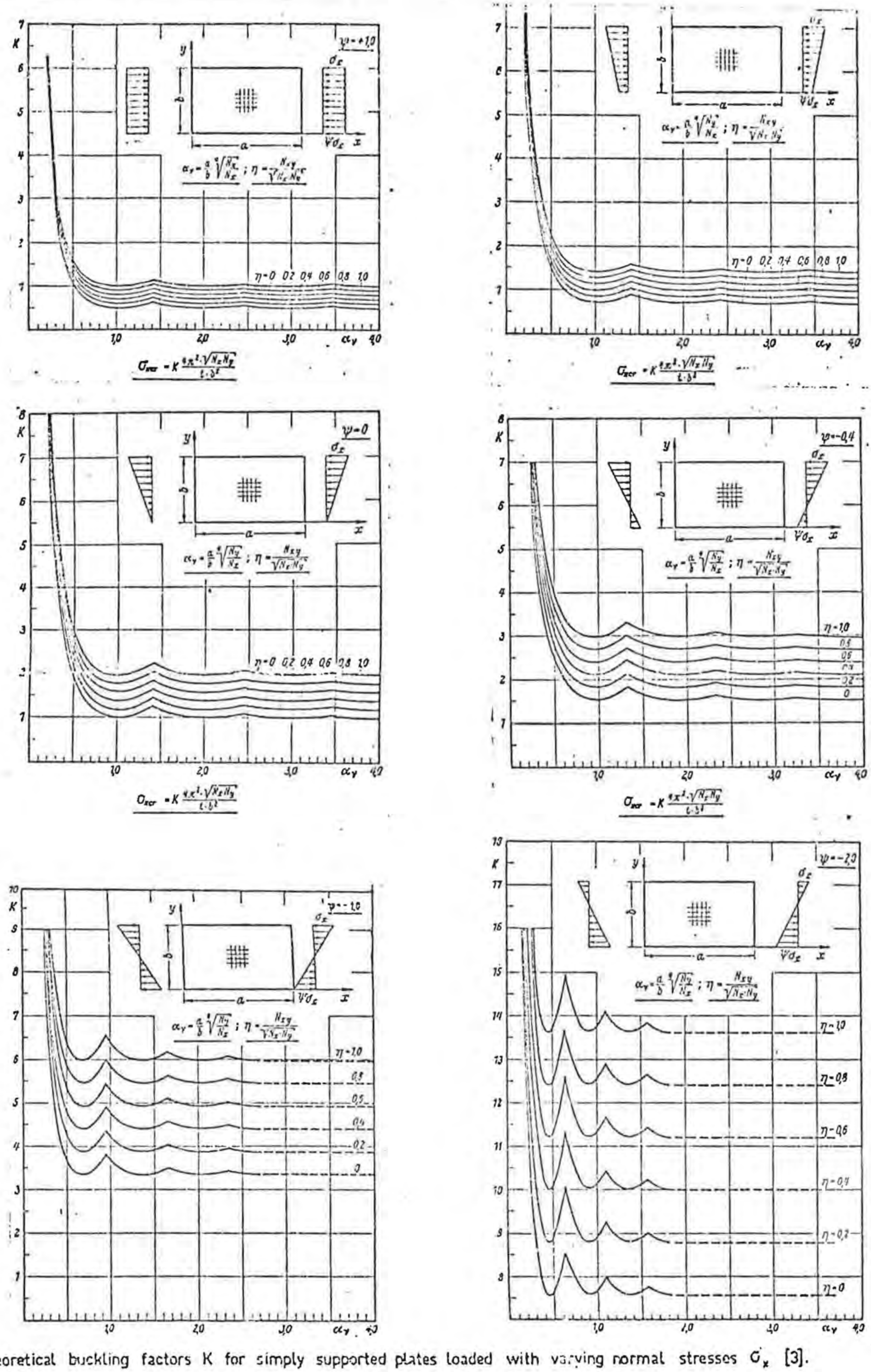
$\Psi.\sigma_x$ = the other normal stress

σ_{xcr} and $\Psi\sigma_{xcr}$: the critical values of σ_x and $\Psi\sigma_x$

N_x and N_y : plate stiffnesses as defined before

K = buckling factor, the values of which depends on $\alpha_\nu = \alpha\sqrt[4]{\frac{N_y}{N_x}}$ as well as on ψ. and on $\eta = \frac{N_{xy}}{\sqrt{N_xN_y}}$

Guirlande curves for different values of ψ and dependent on α have been given in fig. 25. It appears that ψ and η both have great influence. For constant values of ψ and η the effect of α_ν is less important if $\alpha_\nu > 1$; at least for design purposes it seems to be justified to neglect this effect. This means that then the effect of $\alpha = \frac{a}{b}$ and of the number of halfwaves don't play a role anymore. This leads to a graph as given in fig. 26, where the minimum values of the curves of fig. 25 are adapted as sufficient accurate values for K.



Theoretical buckling factors K for simply supported plates loaded with varying normal stresses σ_x [3].

Fig. 25

If, like in [3] $\nu_x = \nu_y = 0$, then

$$N_x = \frac{E_x t^3}{12}, \; N_y = \frac{E_y t^3}{12}, \; N_{xy} = \frac{E_{xy} t^3}{6}, \text{ and } \eta = \frac{2E_{xy}}{\sqrt{E_x E_y}},$$

with which $\sigma_{cr} = K. \frac{\pi^2 t^2}{3b^2} \sqrt{E_x E_y}$.

Further simplification can be reached if for a certain material or a group of materials the values of η deviates not too much from a mean value. In that case K could be given, e.q. for the most frequent compression load ($\Psi = 1$) and for bending ($\Psi = -1$); together with some realistic values of E_x and E_y real simple design formulas can be found.

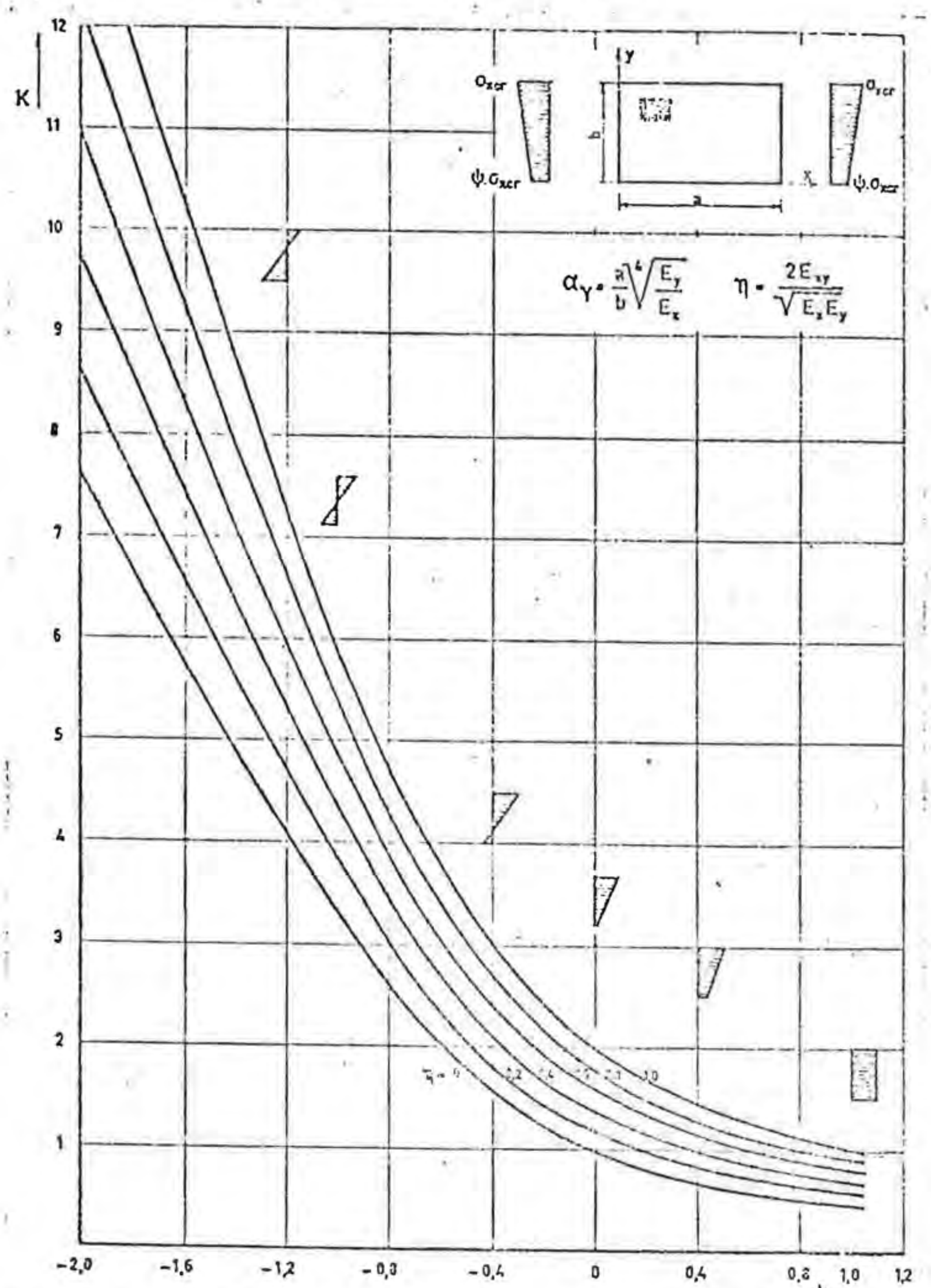


fig. 26.

Practical approximation of buckling factor K for simply supported plates, loaded with linear distributed normal stresses σ_x.

6.1.2. Shear Stresses.

The relative simple case of constant shear stress τ along the sides is considered here. According to [3] it can be proved that

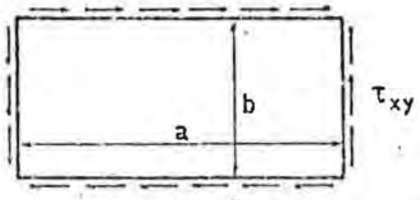


fig. 27.

$$\tau_{cr} = K \frac{4\pi^2}{b^2 t} \sqrt[4]{N_x N_y^3}$$

With $\nu_x = \nu_y = 0$ this becomes $\tau_{cr} = K. \frac{\pi^2 t^2}{3b^2} \sqrt[4]{E_x E_y^3}$

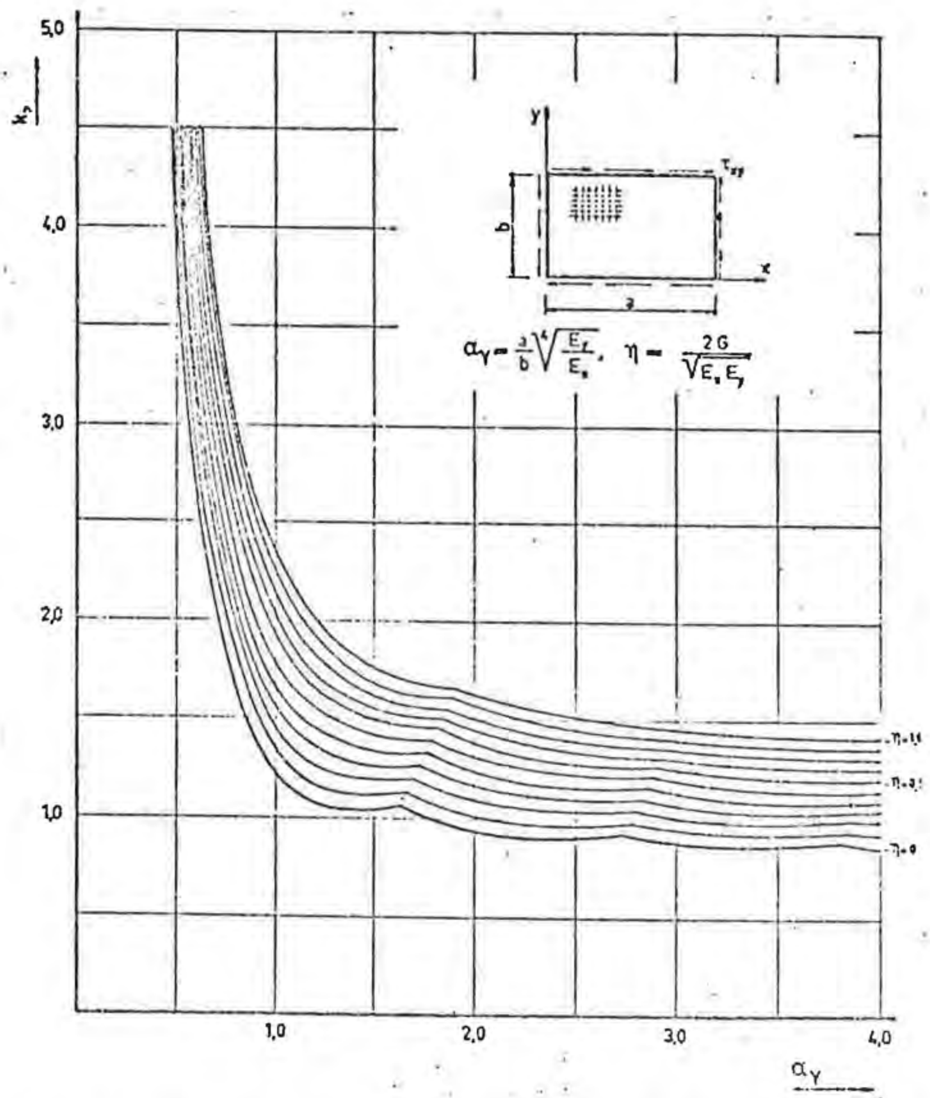
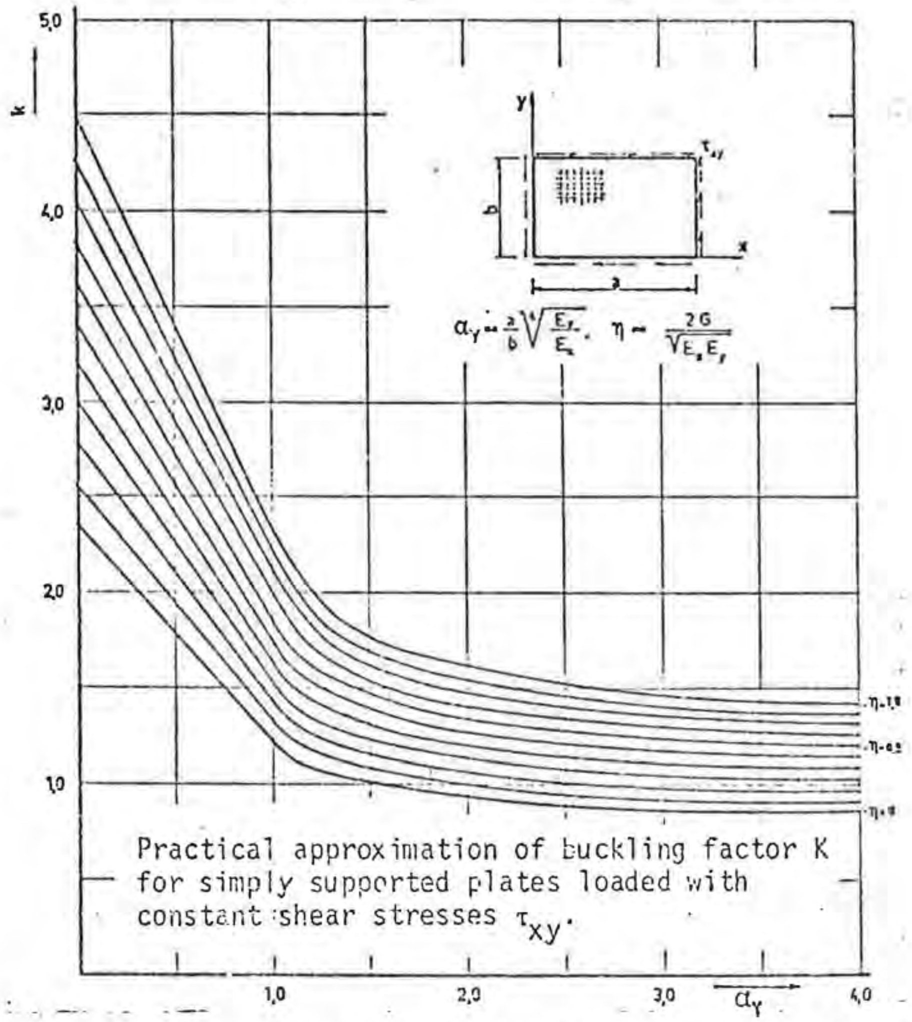


fig. 28. Buckling factor K for simply supported plates loaded with constant shear stresses τ_{xy}. [3]

Values of K can be read of fig. 28, which graph for practical purposes can be simplified to fig. 29, where th "wrinkles" of the curves — depending on the number of half-waves — are neglected and where for practical reason a tangent line is used for α_ν<1. instead of the asymptotic curves.



Practical approximation of buckling factor K for simply supported plates loaded with constant shear stresses τ_{xy}.

fig. 29

The variability in the thickness and mechanical properties of plywood cannot lead to an accurate value of α_ν. The original fig. 28 gives way to very different values of K with small changes of α_ν if α <1; this effect has been avoided with the use of the tangent lines instead of the curves.

6.1.3. Combinations of bending- or normal stresses with shear.

If the symbols σ_{xcr} and τ_{cr} remain used for the plate with normal and

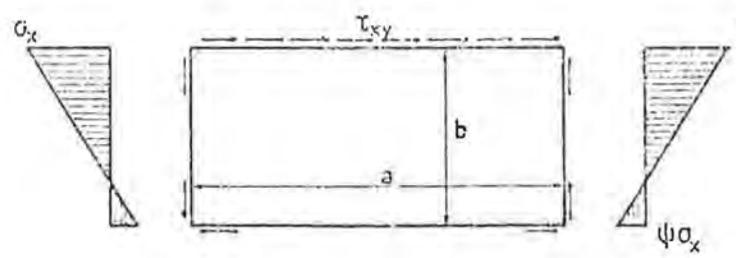


fig. 30

shearstress only, and if σ^{1}_{crx} and τ^{1}_{cr} will be used for normal and shear stresses in combination, according to [3] graphs were drafted where the relationship between $\frac{\sigma^{1}_{crx}}{\sigma_{crx}}$ and $\frac{\tau^{1}_{cr}}{\tau_{cr}}$ is given both for the case where $\Psi = 1$ and $\Psi = -1$. (cf fig. 31.)

The curves in the fig. 31[a] follow more or less a part of a parabola, in the second (of fig. 31[b]) the circle is a better approximation. For reasons of simpleness it is proposed to use a safe circular boundary

$$\left(\frac{\sigma^{1}_{crx}}{\sigma_{crx}}\right)^2 + \left(\frac{\tau^{1}_{cr}}{\tau_{cr}}\right)^2 = 0{,}85 \ (= 0{,}92^2)$$

This circle has been given too in fig. 31.

6.1.4. Combination of normal stresses in two direction (uniform distributed)

If again σ^{1}_{crx} and σ^{1}_{cry} are used as the symbols for the combined actions and σ_{crx} resp σ_{cry} the critical values if there are only stresses in the X- or in the Y- direction, it can be shown that the following relationship holds (see appendix):

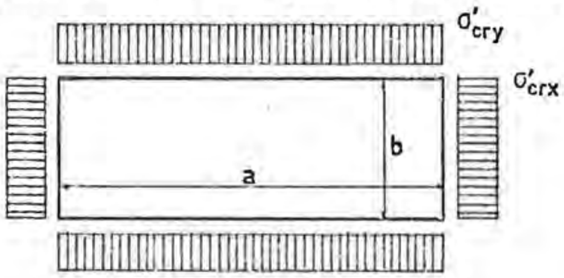


fig. 32

$$\frac{\sigma^{1}_{crx}}{\sigma_{crx}} + \frac{\sigma^{1}_{cry}}{\sigma_{cry}} = 1$$

which is a straight line in fig. 33.

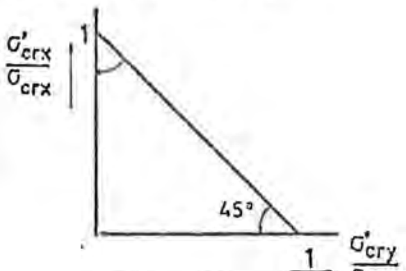


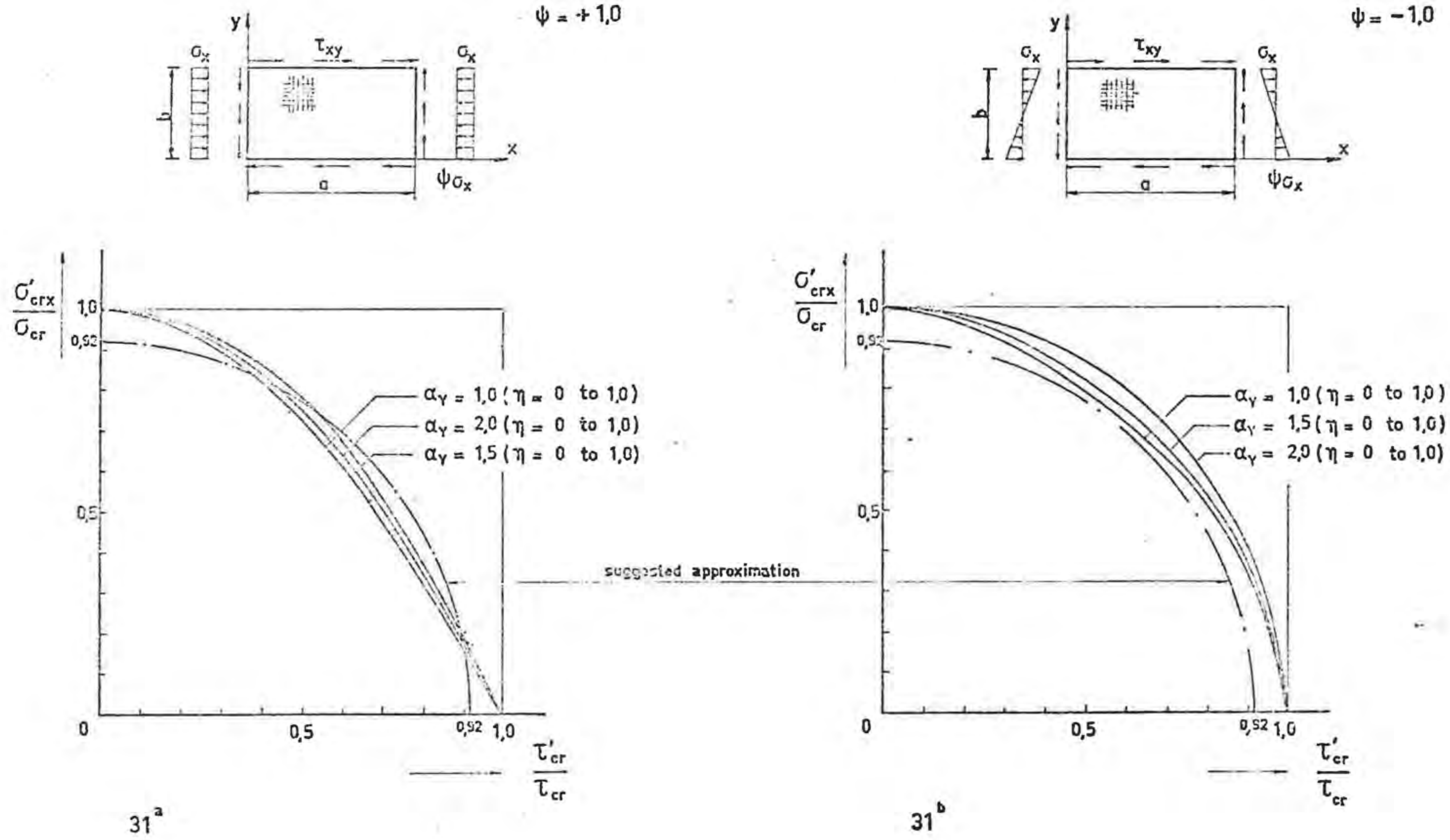
fig. 33.

6.1.5. Combination of two normal and shear stresses.

Based upon the relationships in the foregoing it seems not too hazardous to extend the boundaries to a three-dimensional system as fig. 34 , which could be described by

$$\left(\frac{\sigma^{1}_{crx}}{\sigma_{crx}} + \frac{\sigma^{1}_{cry}}{\sigma_{cry}}\right)^2 + \left(\frac{\tau^{1}_{cr}}{\tau_{cr}}\right)^2 = 0{,}85$$

fig.31. Approximation of calculated boundary for stress combinations by a circle.



This three-dimensional figure is only an extrapolation of the three boundaries in the three main planes; there is no verification available as yet.

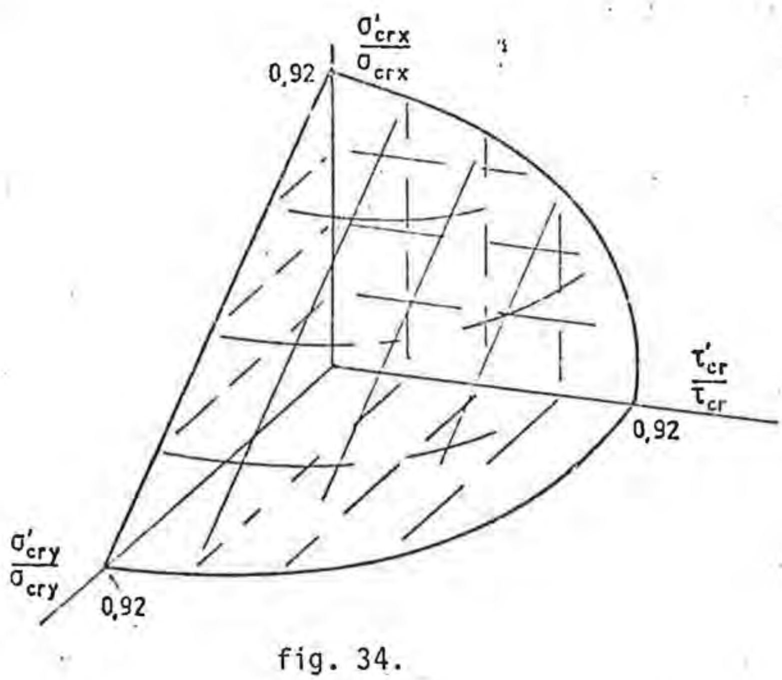


fig. 34.

7. Aspects of safety

Assuming that the design values of the TGH will remain unchanged, design calculations will be based thereupon in the near future. Safe design values for buckling must therefore also be based upon the TGH.
In tables 7 and 8 values have been given for σ_{cr};TGH and σ_{cr},test. For the simply supported panels - the clamped panels are not considered here any more - the mean value of the ratio $\frac{\sigma_{cr;test}}{\sigma_{cr;TGH}}$ was 1,48, with a standard deviation of 0.58.
In table 11 the values are given in certain groups, from which a coëfficient of variation can be calculated, according to $v_{mean} = \sqrt{\frac{\Sigma[(n_i-1)v^2]}{\Sigma(n_i-1)}}$, from which $v_{mean} = 0,19$.

Table 11 Values of $\frac{\sigma_{cr;test}}{\sigma_{cr;TGH}}$; simply supported panels

b	t	// or ⊥	ratio	st.dev.	var. coeff.	n
400	8	//	1,99	0,25	0,12	9
	13	//	1,16	0,15	0,13	8
	8	⊥	2,16	0,54	0,25	6
	13	⊥	1,18	0,07	0,06	6
600	8	//	1,49	0,53	0,35	6
	13	//	1,09	0,18	0,17	6
	8	⊥	1,04	0,15	0,15	3
	13	⊥	1,08	0,03	0,08	4

This latter estimation is a better one than the earlier calculated one while here the systematic errors for the different groups are eliminated. Using this coëfficient of variation an allowable value of a buckling stress can be calculated according to the theory in [6], from which

$\bar{\sigma}_{cr} = \dfrac{\sigma_{cr;TGH}}{2,1}$, where not yet a reduction for long time loading is introduced. With respect to the test values there is a mean safety of $\approx 3,1$

$$\left(\bar{\sigma}_{cr} = \frac{1,48.\sigma_{cr;TGH}}{2,1x1,48} = \frac{\sigma_{cr;test}}{3,11}\right)$$

It is assumed furthermore that a good approximation of the behaviour of the plywood plate under increasing loads, after creep and eventual other effects have taken place, may be given by the — . — line in fig. 35.

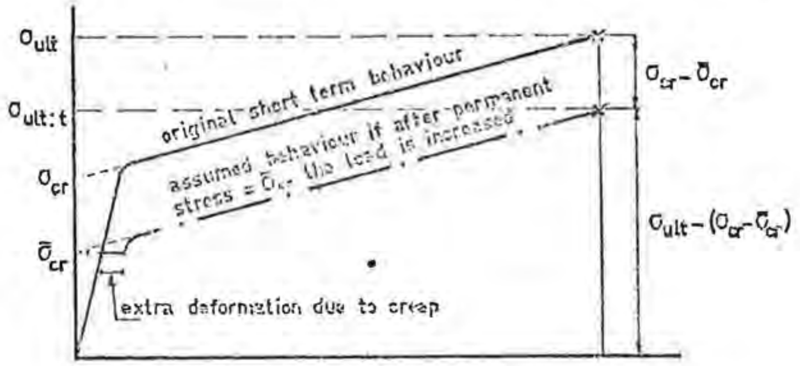


fig. 35.

From the figure it is clear that it is assumed that after the creep period at full σ_{cr}, an overload causes deformations on the basis of the original plate stiffness. Failure is supposed to occur at the same deformation as in the short duration test. As $\sigma_{ult} \approx 2\tau_{cr}$ failure after constant preload to $\bar{\sigma}_{cr}$ occurs at a load of

$$\sigma_{ult} - (\sigma_{cr} - \bar{\sigma}_{cr}) = 2\sigma_{cr} - (\sigma_{cr} - 0,32\sigma_{cr})$$
$$= 1,32\sigma_{cr}, \text{ this is at } \frac{1,32\ \sigma_{cr}}{0,32\ \sigma_{cr}} \approx 4 \times \bar{\sigma}_{cr}.$$

On the basis of the foregoing a set of limitations to the stresses can be proposed:

1) the calculated stresses in a plate, resulting from the loads on the structure may not exceed the allowable normal stress $\bar{\sigma}_{compr}$ and the allowable shear stress $\bar{\tau}$.
2) the calculated stresses may not exceed allowable critical stresses $\bar{\sigma}_{cr}$, $\bar{\tau}_{cr}$ or certain combinations thereoff.

In the case of stress combinations the calculated stresses σ_x, τ_y and τ

must fulfill the equation:

$$\left(\frac{\sigma_x}{\bar{\sigma}_{xcr}} + \frac{\sigma_y}{\bar{\sigma}_{ycr}}\right)^2 + \left(\frac{\tau}{\bar{\tau}_{cr}}\right)^2 \leq 0{,}85$$

For practical purposes the following equation can be used:

$$\left(\frac{\sigma_x}{\bar{\sigma}_{xcr}} + \frac{\sigma_y}{\bar{\sigma}_{ycr}}\right)^2 + \left(\frac{\tau}{\bar{\tau}_{cr}}\right)^2 \leq 1$$

In this case we have to multiply the formulae for $\bar{\sigma}_{xcr}$, $\bar{\sigma}_{ycr}$ and $\bar{\tau}_{cr}$ by a factor 0,92.

Based on the foregoing the calculation control of plywood as a structural material in load-bearing structures can be effected. For practical purposes sets of calculated values for different plywoods and for certain loading conditions can be given. Experience and further research may give way to lower safety factors in the future.

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Appendix 1

1. Stability of a rectangular orthotropic plate compressed in two directions

A rectangular plate with thickness t and the principal directions parallel to the sides is compressed by uniformly distributed stresses σ_x and σ_y. The problem of stability of such a plate has been solved for the case of four supported sides (see Lekhnitskii: "Anisotropic plates").
The differential equation will have the form:

$$N_x \frac{\partial^4 w}{\partial x^4} + 2N_{xy} \frac{\partial^4 w}{\partial x^2 \partial y^2} + N_y \frac{\partial^4 w}{\partial y^4} + t\sigma_x \frac{\partial^2 w}{\partial x^2} + t\sigma_y \frac{\partial^2 w}{\partial y^2} = 0 \qquad (1)$$

For a solution in the form of:

$$w = \sum_m \sum_n A_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} \qquad (2)$$

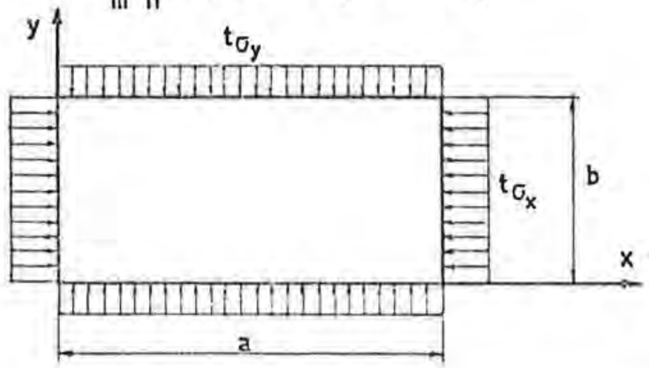


Fig. 1 Rectangular plate compressed in two directions.
We get the relation:

$$t\sigma_x \left(\frac{m}{a}\right)^2 + t\sigma_y \left(\frac{n}{b}\right)^2 = \pi^2 \left[N_x \left(\frac{m}{a}\right)^4 + 2N_{xy} \left(\frac{mn}{ab}\right)^2 + N_y \left(\frac{n}{b}\right)^4\right]$$

For the case that σ_x en σ_y vary but maintain a constant ratio:

$$\sigma_x = \zeta \text{ and } \sigma_y = \phi\,\zeta,$$

a critical value of ζ can be found as

$$\zeta_{cr} = \frac{\pi^2\sqrt{N_x N_y}}{tb^2} \cdot \frac{\sqrt{\frac{N_x}{N_y}}\left(\frac{m}{\alpha}\right)^2 + \frac{2N_{xy}}{\sqrt{N_x N_y}}\, n^2 + \sqrt{\frac{N_y}{N_x}}\left(\frac{\alpha}{m}\right)^2 n^4}{1 + \phi\left(\frac{\alpha}{m}\right)^2 n^2}$$

$$= \frac{4\pi^2\sqrt{N_x N_y}}{tb^2} \cdot \frac{\left(\frac{m}{2w\alpha}\right)^2 + \frac{1}{2}\eta n^2 + \left(\frac{w\alpha}{2m}\right)^2 n^4}{1 + \phi\left(\frac{\alpha}{m}\right)^2 n^2} \qquad (3)$$

where:

N_x, N_y	: bending stiffness in resp. x- and y-direction
N_{xy}	: torsional stiffness
t	: thickness of the plate
α	: ratio $\frac{a}{b}$
w	: $\sqrt[4]{\frac{N_y}{N_x}}$
η	: $\frac{N_{xy}}{\sqrt{N_x N_y}}$
m, n	: number of waves in resp. x- and y-direction.

From Lekhnitskii's formula 3 we can go further as follows:
the critical value for ζ will be minimum when $n = 1$. Equation (3) reduces to:

$$\zeta_{cr} = \frac{4\pi^2\sqrt{N_x N_y}}{tb^2} \cdot \frac{\left(\frac{m}{2w\alpha}\right)^2 + \frac{1}{2}\eta + \left(\frac{w\alpha}{2m}\right)^2}{1 + \phi\left(\frac{\alpha}{m}\right)^2} = \frac{\pi^2\sqrt{N_x N_y}}{tb^2} \cdot K \qquad (4)$$

For compression in the x-direction only: $\zeta_{cr}\,|\,\phi = 0 = \sigma_{xcr}$; for compression in the y direction only: $\phi\zeta_{cr}\,|\,\phi = \infty = \sigma_{ycr}$
For other values of ϕ the combination:

$$\sigma'_{xcr} = \zeta_{cr} \text{ and } \sigma'_{ycr} = \phi\,\zeta_{cr}$$

becomes critical.

In general we find the minimum value for ς_{cr} by taking the minimum value of the second part of equation (4):

$$K = \frac{\left(\frac{m}{2w\alpha}\right)^2 + \frac{1}{2}\eta + \left(\frac{w\alpha}{2m}\right)^2}{1 + \phi\left(\frac{\alpha}{m}\right)^2} \tag{5}$$

In figure 2 the minimum values of K are plotted for $w = 1$ and different values of η (= 0; 0,2; 0,4; 0,6; 0,8; 1,0) and ϕ (=0; 0,2; 1,0; 10,0)

From equation 4 and for m = 1 we obtain the following relations:

$$\sigma'_{xcr} = \frac{\sigma_{xcr}}{1 + \phi\alpha^2} \; ; \; \sigma'_{ycr} = \frac{\phi\sigma_{xcr}}{1 + \phi\alpha^2} \text{ and } \sigma'_{ycr} = \frac{\sigma_{xcr}}{\alpha^2}$$

so that:

$$\frac{\sigma'_{xcr}}{\sigma_{xcr}} + \frac{\sigma'_{ycr}}{\sigma_{ycr}} = \frac{1}{1 + \phi\alpha^2} + \frac{\phi\alpha^2}{1 + \phi\alpha^2} = 1 \tag{6}$$

From fig. 2 it can be seen that for different values of ϕ and a particular value of α the minimum value of K will be reached for different values of m. In that case equation (6) doesn't hold anymore.
However, from fig. 3 it can be seen that the differences with the straight line are rather small and that relation (6) is safe. These graphs are given for:

w = 0,5; 1,0; 1,5; 2,0

η = 0,4;

ϕ varies from 0 to 100.

However we want to use the real minimum value for K (see fig.2) so we have to check if relation (6) is also operative in that case.
From $\frac{dK}{d\alpha} = 0$ (see equation 5) we obtain α where K is a minimum:

$$\alpha^2 = \phi \frac{m^2}{w^2} \quad \frac{1}{w^2 - 2\eta\phi} \quad [\, 1 \pm \sqrt{1 + \frac{w^2}{\phi^2}(w^2 - 2\eta\phi)}\,]$$

Only for $w^2 - 2\eta\phi > 0$ we get real values for α and real minima for K . If $w^2 - 2\eta\phi \leq 0$ the curve for K has no real minimum but an asymptote.

voor $\phi = 0$ $K_{min} = 1$

In case $w^2 - 2\eta\phi > 0$ and for $\eta = 1$ we find: $\alpha^2 = \dfrac{m^2}{w^2 - 2\phi}$ so that:

$K_{min} = (1 - \frac{\phi}{w^2})$;

for $\eta = 0$ we find in case $\phi >> w^2$: $\alpha^2 = \frac{2m^2}{w^4} \phi$ so $K_{min} = \frac{w^2}{4\phi}$.

In case $\phi << w^2$ we find: $\alpha^2 = \frac{m^2}{w^2}$ and $K_{min} = \dfrac{0,5}{1 + \frac{\phi}{w^2}}$

Relation (6) becomes now respectively:

$$\eta = 1: \qquad \frac{4(1 - \frac{\phi}{w^2})}{1} + \frac{4\,\phi\,(1 - \frac{\phi}{w^2})}{w^2} = 1 + \frac{3\phi}{w^2} - \frac{4\phi^2}{w^4}$$

The condition was $w^2 - 2\phi > 0$ or $\phi < \frac{1}{2}w^2$. For $\phi = 0$ relation (5) becomes 1 and for $\phi \approx \frac{1}{2}w^2$ relation (5) is less than 1,5.

$$\eta = 0 \;\text{ and }\; \phi >> w^2: \quad \frac{\frac{w^2}{\phi}}{2} + \frac{w^2}{w^2} = \frac{w^2}{2\phi} + 1 \approx 1$$

$$\eta = 0 \;\text{ and }\; \phi << w^2: \quad \frac{\frac{2}{1 + \frac{\phi}{w^2}}}{2} + \frac{\phi \cdot \frac{2}{1 + \frac{\phi}{w^2}}}{w^2} = \frac{w^2 + 2\phi}{w^2 + \phi} \approx 1$$

As mentioned before the curve K will have an asymptote when $w^2 - 2\eta\phi \leq 0$. This asymptote is $K = \frac{w^2}{\phi}$

Relation (6) becomes now in case $\eta = 1$ $(\phi \geq \frac{1}{2}w^2)$:

$$\frac{\frac{w^2}{\phi}}{4} + \frac{w^2}{w^2} = 1 \quad \text{to} \quad 1{,}125$$

and in case $\eta > 0$ $(\phi \to \infty)$ then relation (6) becomes equal to 1.

Both theoretically and based on the minimum values of K relation (6) appears to be a rather good and safe relation to approximate the critical stability situation of a plate compressed in two directions. Relations (6) varies between the values 1 and 1,5.

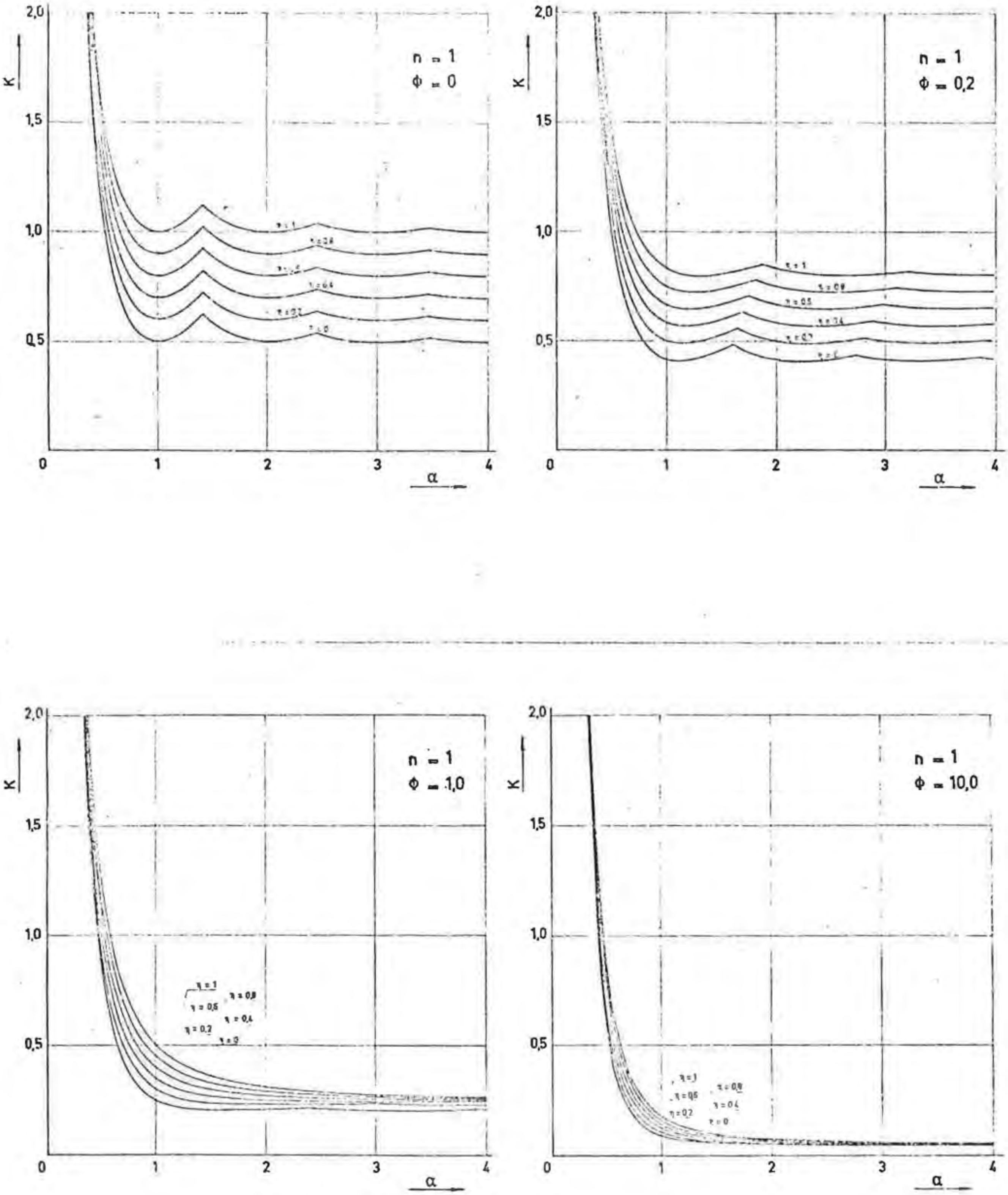


fig. 2
Guirlande curves for K for $w = \frac{E_y}{E_x} = 1$ and different values of η and Φ .

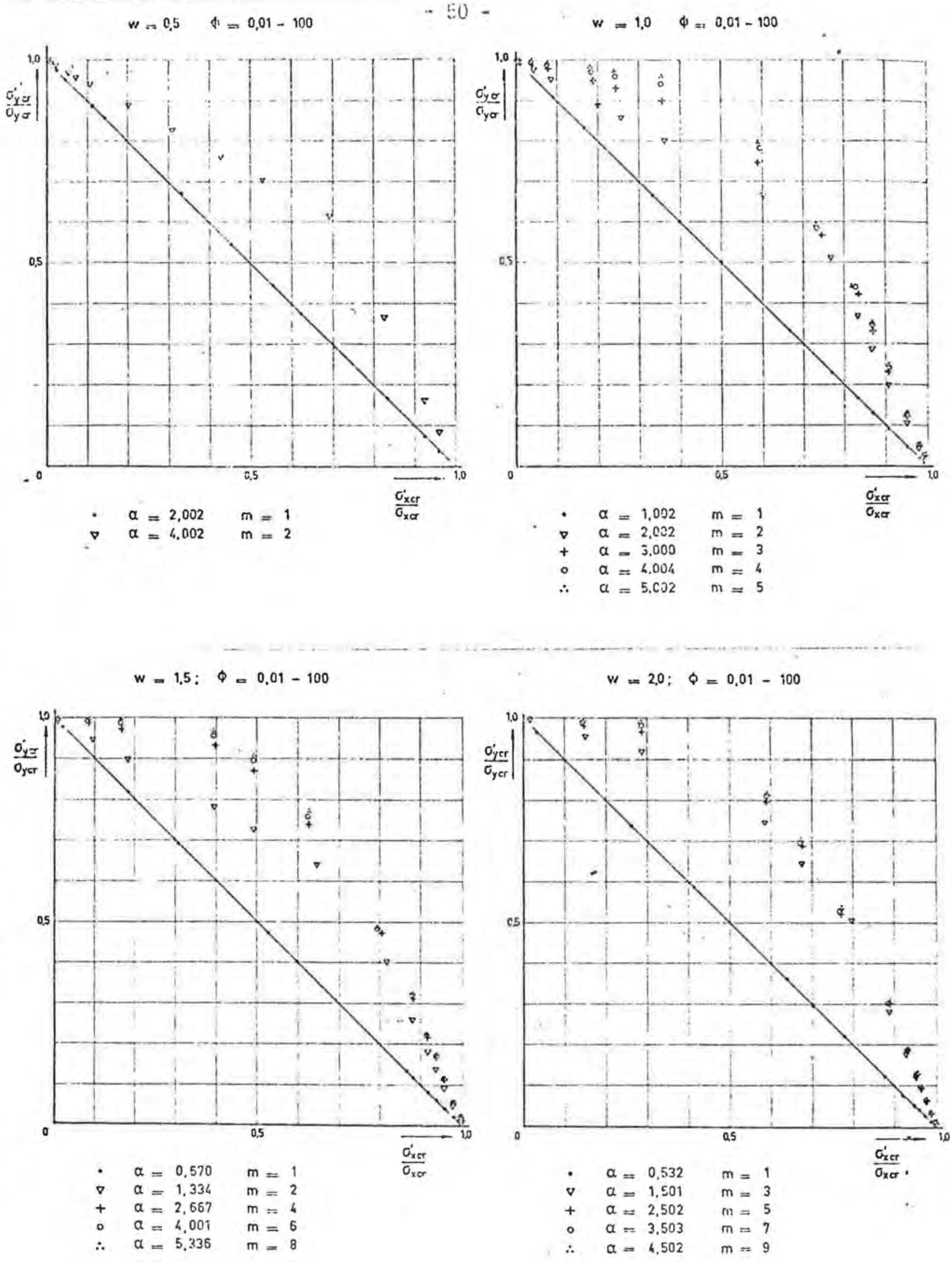


fig. 3

Relation $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ for $\beta = 0$ and different values of α, w and Φ.

Appendix 2

2. Stability of a rectangular orthotropic plate uniformly compressed in one direction and loaded by linear distributed forces in the other direction

The problem of stability of an orthotropic plate loaded by linear distributed forces in one direction has been solved among others by Lekhnitskii. The combination with uniformly compression forces in the other direction (fig. 4) can be solved as follows.

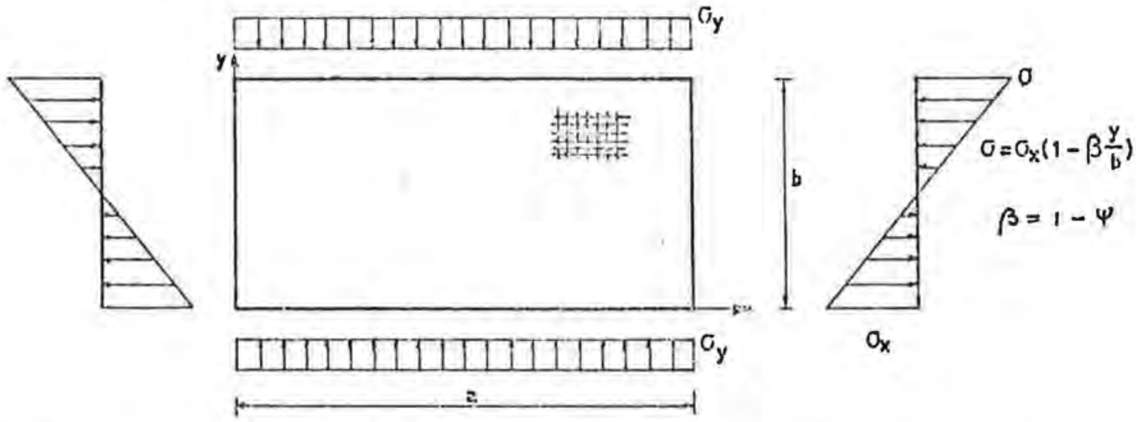


Fig. 4 . Rectangular plate uniformly compressed in one direction and loaded by linear distributed forces in the other direction.

The potential energy of the external loads is:

$$U = \int_o^a \int_o^b \frac{t\sigma_x}{2} (1 - \beta \frac{y}{b})(\frac{dw}{dx})^2 \, dxdy + \frac{t\sigma_y}{2} \int_o^a \int_o^b (\frac{\partial w}{\partial y})^2 \, dxdy$$

The strain energy is:

$$I = \frac{1}{2} \int_o^a \int_o^b \{N_x (\frac{\partial^2 w}{\partial x^2})^2 + 2 N_{xy} (\frac{\partial^2 w}{\partial x \partial y})^2 + N_y (\frac{\partial^2 w}{\partial y^2})^2 \} \, dxdy$$

The solution for the deflection that satisfies the boundary conditions is:

$$w = \sum_m \sum_n A_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}$$

When the number of waves in y-direction is fixed to, n = 2, then it may be expected that a satisfactory accuracy will be reached. In that case the deflection is

$$w = \sum_m (A_{m1} \sin \frac{\pi y}{b} + A_{m2} \sin \frac{2\pi y}{b}) \sin \frac{m\pi x}{a}$$

This leads to:

$$\frac{\delta w}{\delta x} = (A_{m1} \sin \frac{\pi y}{b} + A_{m2} \sin \frac{2\pi y}{b}) \, . \, \frac{m\pi}{a} \cos \frac{m\pi x}{a}$$

$$\frac{\delta^2 w}{\delta x^2} = -(Am_1 \sin \frac{\pi y}{b} + A_{m2} \sin \frac{2\pi y}{b}) \, . \, (\frac{m\pi}{a})^2 \sin \frac{m\pi x}{a}$$

$$\frac{\delta^2 w}{\delta x \delta y} = \frac{m\pi^2}{ab} (A_{m1} \cos \frac{\pi y}{b} + 2A_{m2} \cos \frac{2\pi y}{b}) \cos \frac{m\pi x}{a}$$

$$\frac{\delta^2 w}{\delta y^2} = \frac{\pi^2}{b^2} (A_{m1} \sin \frac{\pi y}{b} + 2A_{m2} \sin \frac{\pi y}{b}) \sin \frac{m\pi x}{a}$$

Substitution in the equation for the potential energie gives:

$$U = \frac{t\sigma_x}{2} \int_o^a \int_o^b (1-\beta \frac{y}{b})(A_{m1} \sin \frac{\pi y}{b} + A_{m2} \sin \frac{2\pi y}{b})^2 (\frac{m\pi}{a} \cos \frac{m\pi x}{a})^2 \, dxdy +$$

$$\frac{t\sigma_y}{2} \int_o^a \int_o^b \{\frac{\pi}{b} (A_{m1} \cos \frac{\pi y}{b} + 2A_{m2} \cos \frac{2\pi y}{b}) \sin \frac{m\pi x}{a}\}^2 \, dxdy$$

$$= \frac{t\sigma_x}{2}\,[\tfrac{1}{2}bA_{m1}^2 + \tfrac{1}{2}bA_{m2}^2 - \frac{\beta}{b}\,(A_{m1}^2 \,.\, \tfrac{1}{4}b^2 - A_{m1}A_{m2}\,\frac{16b^2}{9\pi^2} + A_{m2}^2 . \tfrac{1}{4}b^2)]\,\frac{m^2\pi^2}{a^2}\,.\,\tfrac{1}{2}a$$

$$+ \frac{t\sigma_y}{2}\,.\,\frac{\pi^2}{b^2}\,[A_{m1}^2\,.\,\tfrac{1}{2}b + A_{m2}^2\,.\,\tfrac{1}{2}b]\,.\,\tfrac{1}{2}a$$

$$= \frac{t\sigma_x}{2}\,.\,\frac{ab}{4}\,.\,\frac{m^2\pi^2}{a^2}\,[A_{m1}^2\,(1-0{,}5\beta) + \frac{32\beta}{9\pi^2}\,A_{m1}A_{m2} + A_{m2}^2\,(1-0{,}\ 5\beta)] +$$

$$\frac{t\sigma_y}{2}\,.\,\frac{\pi^2 ab}{4b^2}\,[A_{m1}^2 + A_{m2}^2]$$

$$= \frac{t\sigma_x}{8}\,.\,\frac{m^2\pi^2}{\alpha}\,[A_{m1}^2\,(1-0{,}5\beta) + \frac{32}{9\pi^2}\,A_{m1}A_{m2} + A_{m2}^2\,(1-0{,}5\beta)] + \frac{t\sigma_y}{8}\,\pi^2\alpha[A_{m1}^2+A_{m2}^2]$$

Substitution in the equation of the strain energy:

$$I = \tfrac{1}{2}\int_o^a\int_o^b \{N_x(A_{m1}\sin\frac{\pi y}{b} + A_{m2}\sin\frac{2\pi y}{b})^2\,(\frac{m^2\pi^2}{a^2}\sin\frac{m\pi x}{a})^2 +$$

$$+ 2N_{xy}\,(A_{m1}\cos\frac{\pi y}{b} + 2A_{m2}\cos\frac{2\pi y}{b})^2\,(\frac{m\pi^2}{ab}\cos\frac{m\pi x}{a})^2 +$$

$$+ N_y(A_{m1}\sin\frac{\pi y}{b} + 4Am_2\sin\frac{2\pi y}{b})^2\,(\frac{\pi^2}{b^2}\sin\frac{m\pi x}{a})^2\}\,dxdy$$

$$I = \tfrac{1}{2}\,\{N_x[A_{m1}^2\,.\,\tfrac{1}{2}b + A_{m2}^2\,.\,\tfrac{1}{2}b]\,\frac{m^4\pi^4}{a^4}\,.\,\tfrac{1}{2}a + 2N_{xy}[A_{m1}^2\,.\,\tfrac{1}{2}b + 4A_{m2}^2\,.\,\tfrac{1}{2}b]\,\frac{m^2\pi^4}{a^2b^2}\,.\,\tfrac{1}{2}a$$

$$+ N_y[A_{m1}^2\,.\,\tfrac{1}{2}b + 16A_{m2}^2\,.\,\tfrac{1}{2}b]\,\frac{\pi^4}{b^4}\,.\,\tfrac{1}{2}a\}$$

$$= \frac{ab}{8}\,\{N_x[A_{m1}^2 + A_{m2}^2]\,\frac{m^4\pi^4}{a^4} + 2N_{xy}[A_{m1}^2 + 4A_{m2}^2]\frac{m^2\pi^4}{a^2b^2} + N_y[A_{m1}^2 + 16A_{m2}^2]\,\frac{\pi^4}{b^4}$$

$$= \frac{m^2\pi^4}{8\ ab}\,\sqrt{N_xN_y}\,[A_{m1}^2\{\sqrt{\frac{N_x}{N_y}}\,(\frac{m}{\alpha})^2 + \frac{2N_{xy}}{\sqrt{N_xN_y}} + \sqrt{\frac{N_y}{N_x}}\,(\frac{\alpha}{m})^2\} +$$

$$+ A_{m2}^2\{\sqrt{\frac{N_x}{N_y}}\,(\frac{m}{\alpha})^2 + \frac{8N_{xy}}{\sqrt{N_xN_y}} + 16\,\sqrt{\frac{N_y}{N_x}}\,(\frac{\alpha}{m})^2\}$$

$$\doteq \frac{m^2\pi^4}{8\ ab}\,\sqrt{N_xN_y}\,[A_{m1}^2\,a_{m1} + A_{m2}^2\,a_{m2}]$$

Now is

$$I - U = \frac{\pi^4 m^2}{8\ ab} \sqrt{N_x N_y}\ [A_{m1}^2\ \{a_{m1} - \frac{t\sigma_x\ .\ b^2}{\pi^2 \sqrt{N_x N_y}} (1 - 0{,}5\ \beta) - \frac{t\sigma_y\ a^2}{m^2 \pi^2 \sqrt{N_x N_{y_2}}}\} +$$

$$- 2A_{m1}A_{m2}\ .\ \frac{16\ t\sigma_x\ \beta\ b^2}{9\pi^4\ \sqrt{N_x N_y}} + A_{m2}^2\ \{a_{m2} - \frac{t\sigma_x\ b^2}{\pi^2 \sqrt{N_x N_y}} (1 - 0{,}5\ \beta) - \frac{t\sigma_y\ a^2}{m^2 \pi^2 \sqrt{N_x N_y}}\}]$$

Differentiation to A_{m1} and A_{m2} gives:

$$A_{m1}\ \{a_{m1} - \lambda_1\ (1 - 0{,}5\ \beta) - \lambda_2\} - A_{m2}\ \frac{16\ \lambda_1 \beta}{9\pi^2} = 0$$

$$-A_{m1}\ \frac{16\ \lambda_1 \beta}{9\pi^2} + A_{m2}\ \{a_{m2} - \lambda_1\ (1 - 0{,}5\ \beta) - \lambda_2\} = 0$$

in which

$$\lambda_1 = \frac{t\sigma_x\ b^2}{\pi^2 \sqrt{N_x N_y}} \quad \text{and} \quad \lambda_2 = \frac{t\sigma_y\ a^2}{m^2 \pi^2 \sqrt{N_x N_y}}$$

The solution of these two equation can be found by putting the determinant equal to zero. Thus:

$$\{\ a_{m1} - \lambda_1\ (1 - 0{,}5\ \beta) - \lambda_2\}\{a_{m2} - \lambda_1 (1 - 0{,}5\ \beta) - \lambda_2\} - (\ \frac{16\ \lambda_1 \beta}{9\pi^2}\ )^2 = 0$$

For the case that σ_x and σ_y vary but maintain a constant ratio:

$$\sigma_x = \xi \quad \text{and} \quad \sigma_y = \phi . \xi$$

then

$$\lambda_2 = \lambda_1 .\ \phi .\ \frac{\alpha^2}{m^2}$$

The critical value of ξ is now:

$$\xi_{cr} = \frac{4\pi^2\sqrt{N_x N_y}}{t\, b^2} \; \frac{(a_{m1}+a_{m2})(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})+\sqrt{(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})^2(a_{m1}-a_{m2})^2+a_{m1}a_{m2}(\frac{32\beta}{9\pi^2})^2}}{8\{(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})^2-(\frac{16\beta}{9\pi^2})^2\}}$$

The buckling factor K_p becomes

$$K = \frac{(a_{m1}+a_{m2})(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})+\sqrt{(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})^2(a_{m1}-a_{m2})^2+a_{m1}a_{m2}(\frac{32\beta}{9\pi^2})^2}}{8\{(1-0{,}5\beta+\phi\frac{\alpha^2}{m^2})^2-(\frac{16\beta}{9\pi^2})^2\}}$$

In the figures 9 up to 16 guirlande curves for K are given for different values of β, ϕ and w (in figure omega).

In figures 5, 6, 7 and 8 combinations of $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ are given for different values of ϕ, w, α and β. It can be seen that for m = 1 relation (6) $\frac{\sigma'_{xcr}}{\sigma_{xcr}} + \frac{\sigma'_{ycr}}{\sigma_{ycr}} = 1$ is a good approximation. For other values of m relation (6) is safe.

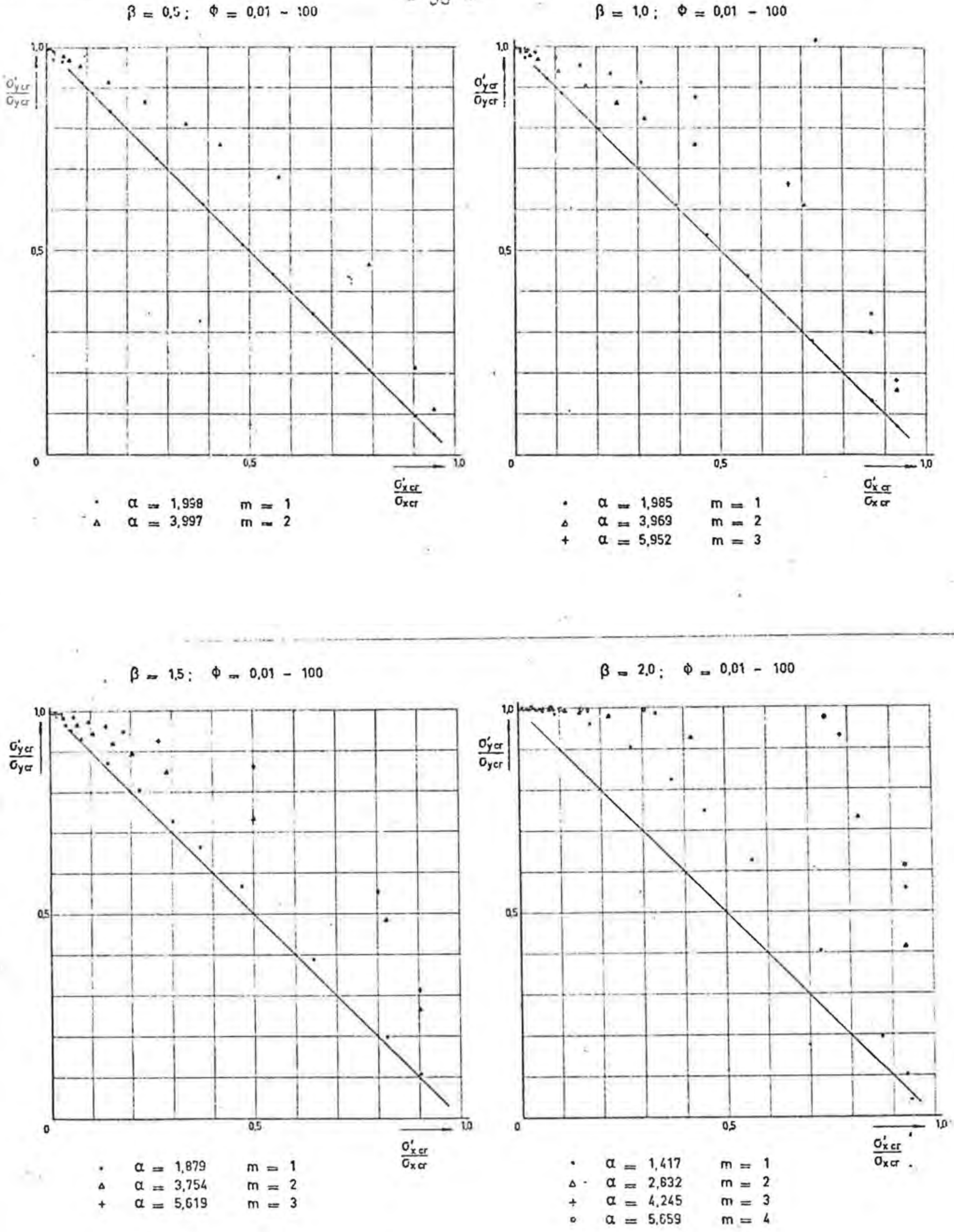


fig. 5
Relation $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ for w = 0,50 and different values of α, β and Φ.

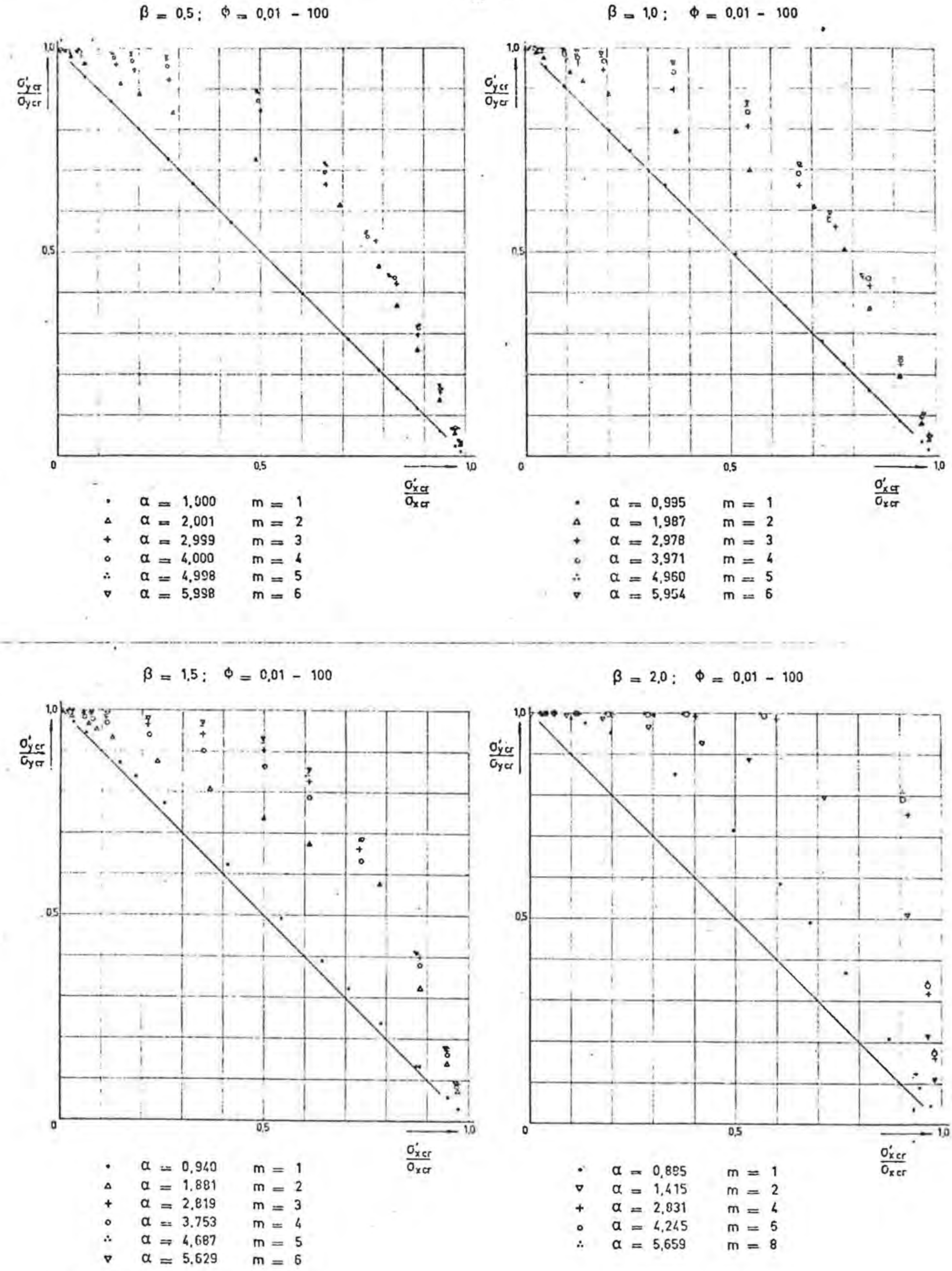


fig. 6

Relation $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ for w = 1,00 and different values of α,β and Φ.

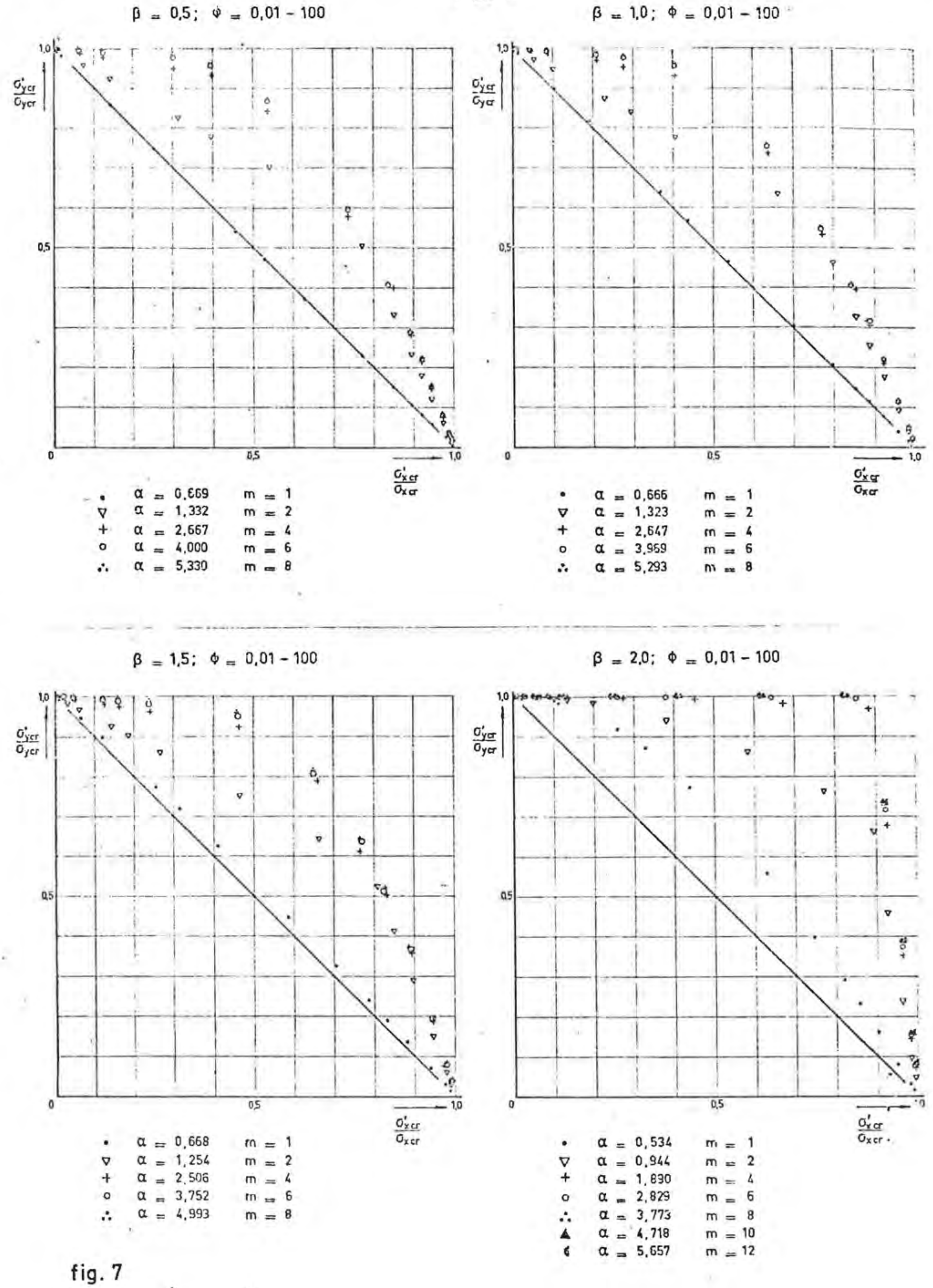
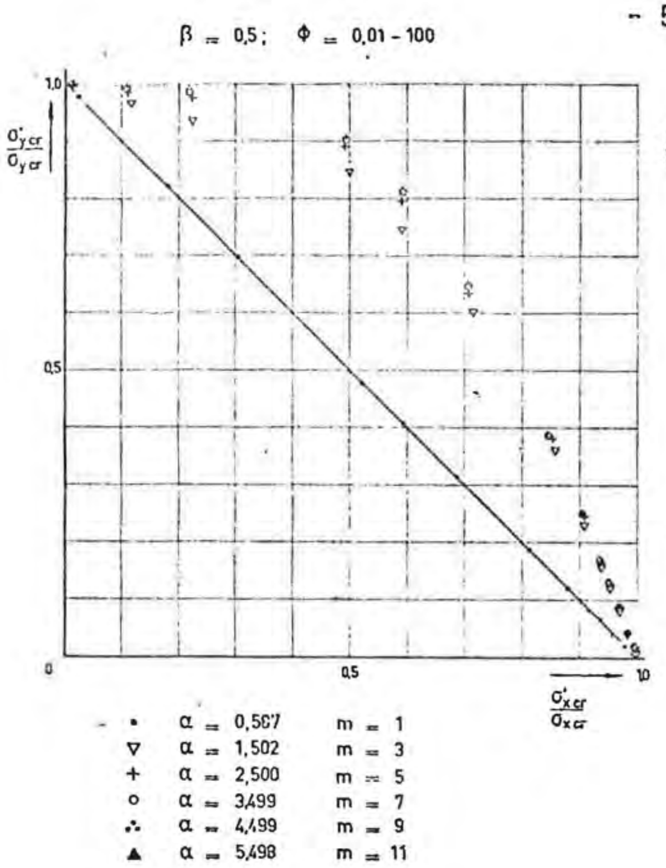
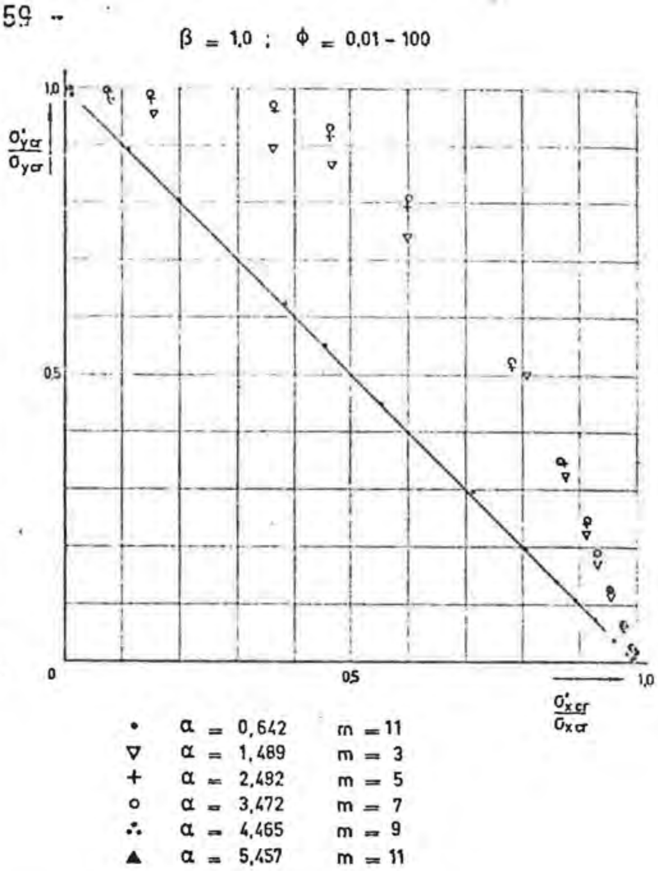
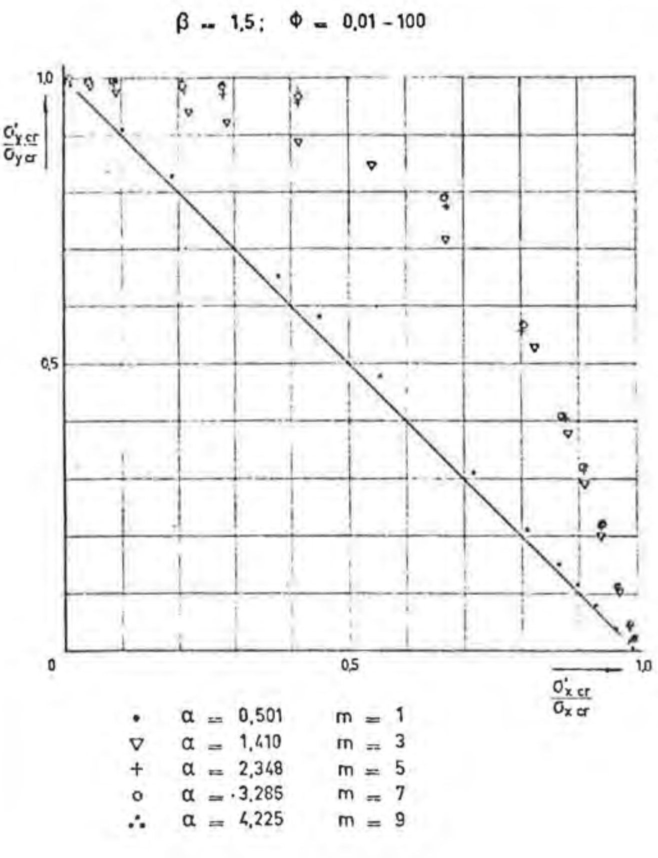


fig. 7

Relation $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ for w = 1,50 and different values of α, β and Φ.







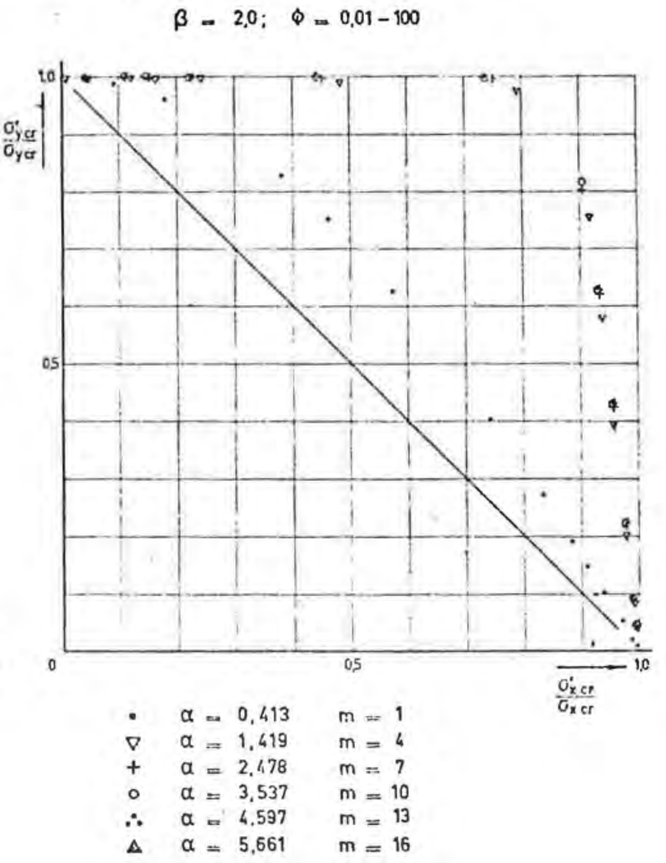
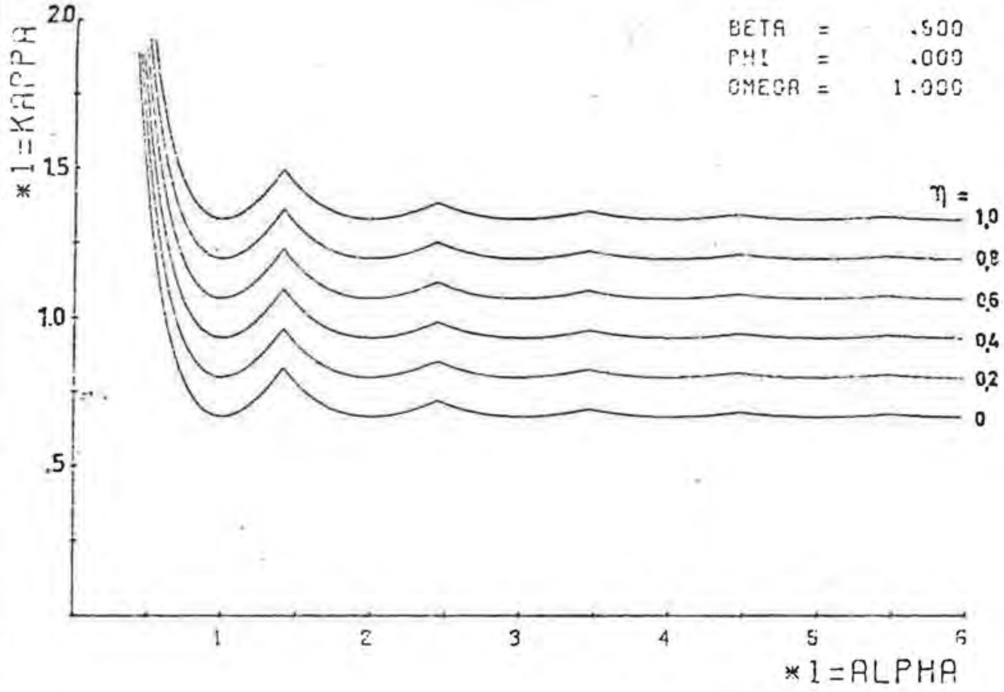


fig. 8
Relation $\frac{\sigma'_{xcr}}{\sigma_{xcr}}$ and $\frac{\sigma'_{ycr}}{\sigma_{ycr}}$ for w = 2,00 and different values of α, β and Φ.



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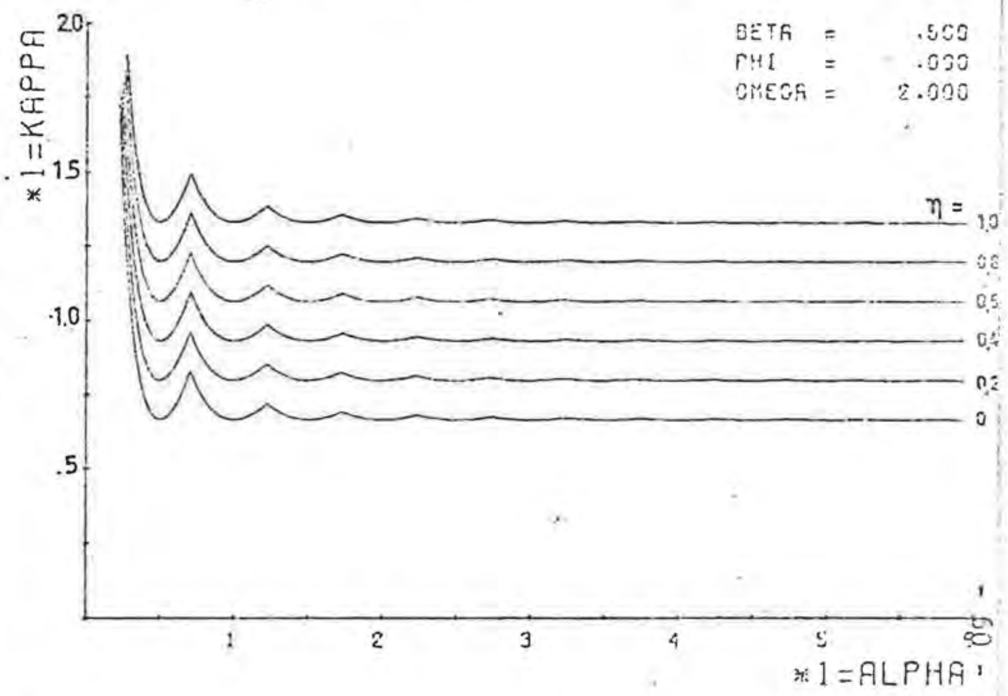

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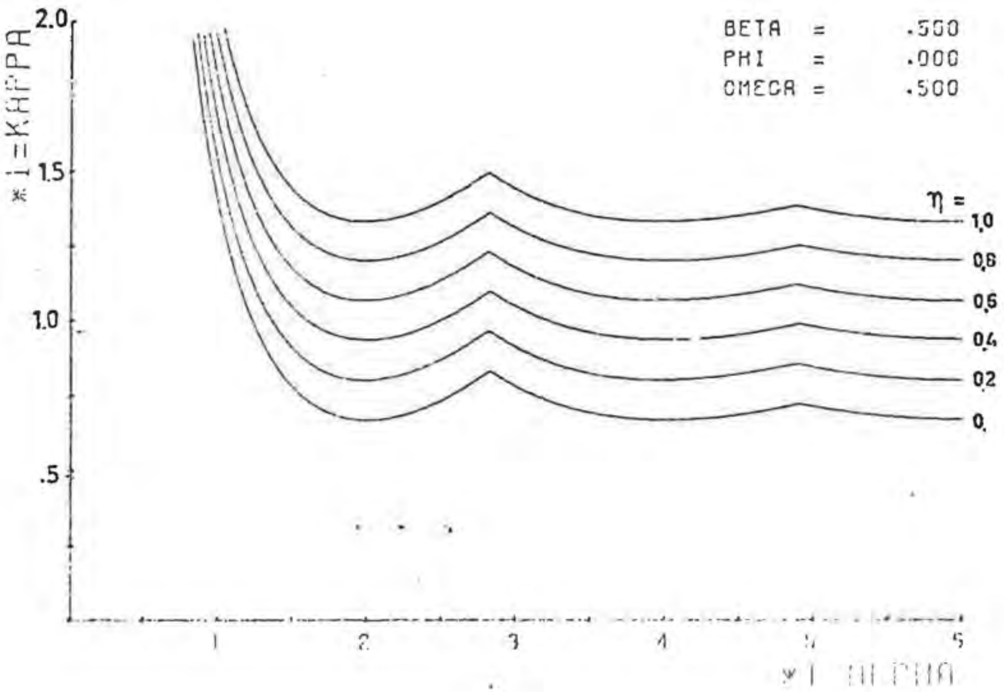

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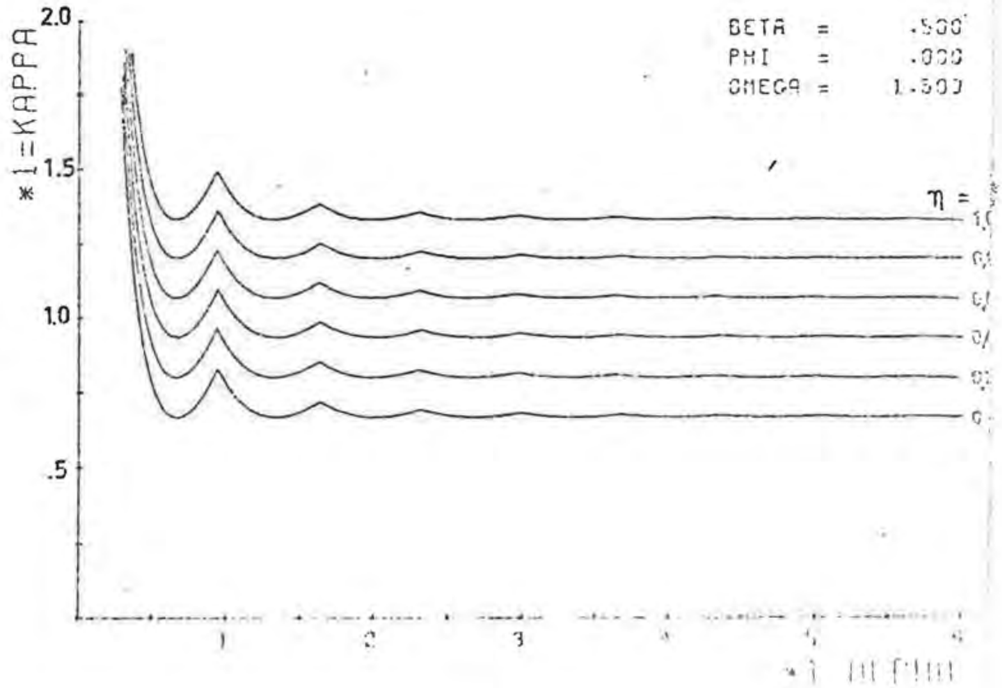

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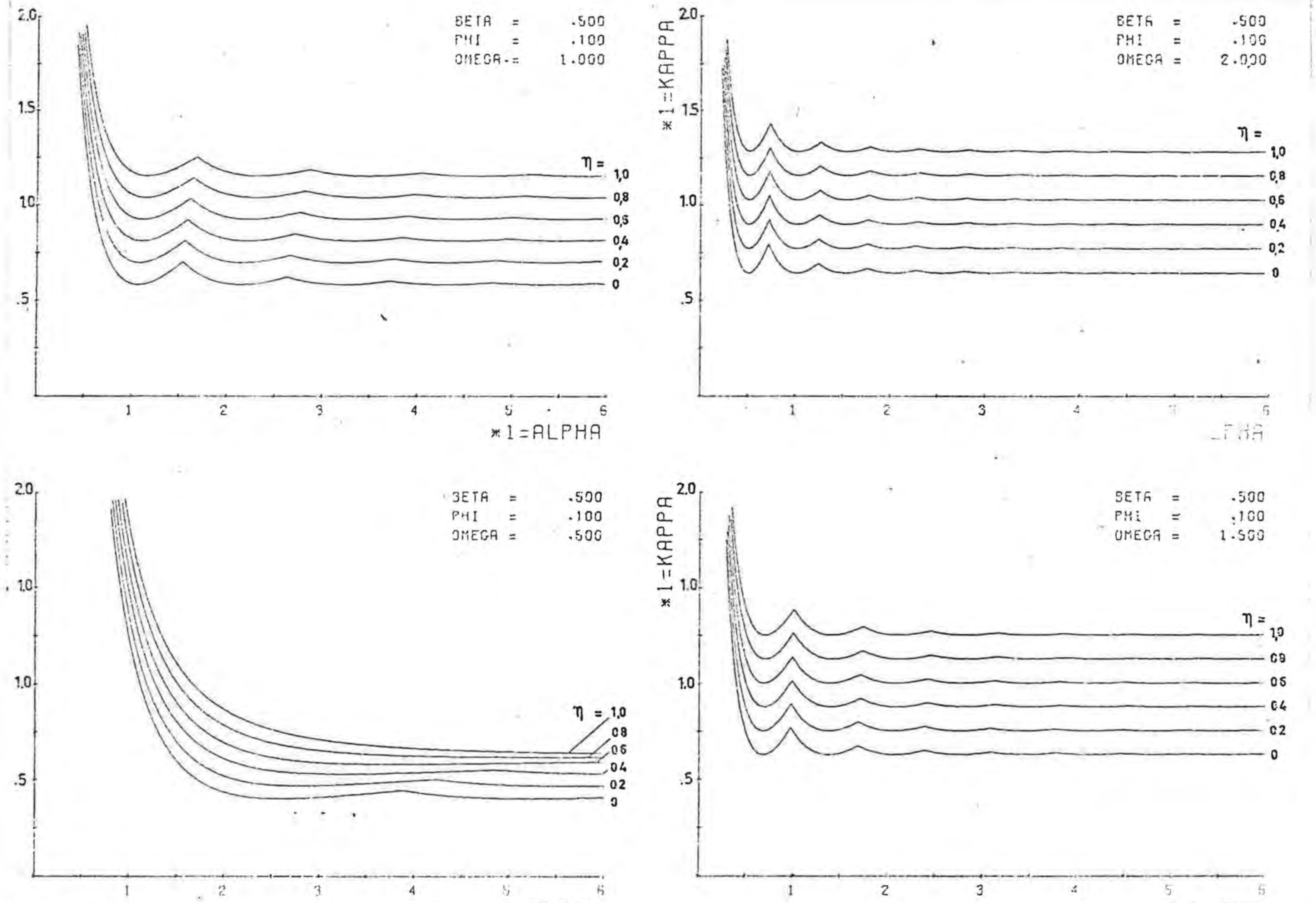
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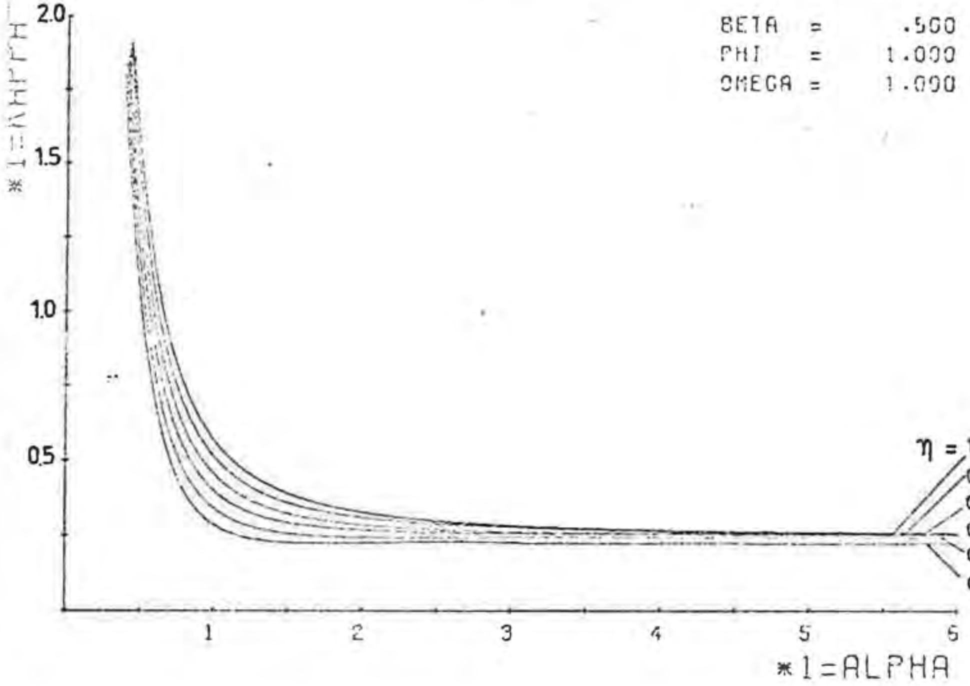
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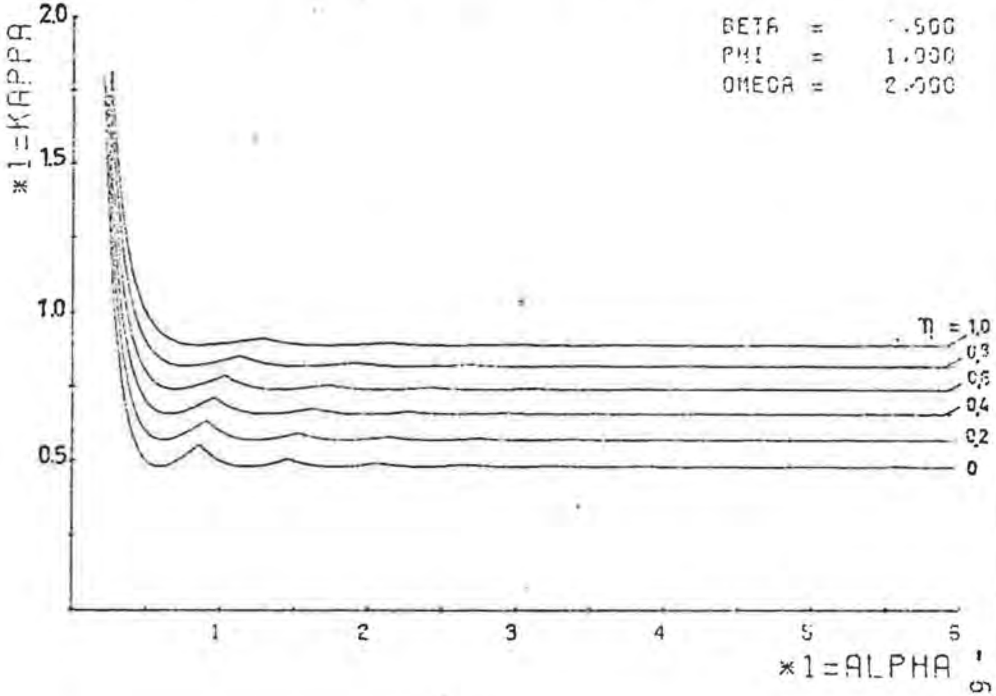
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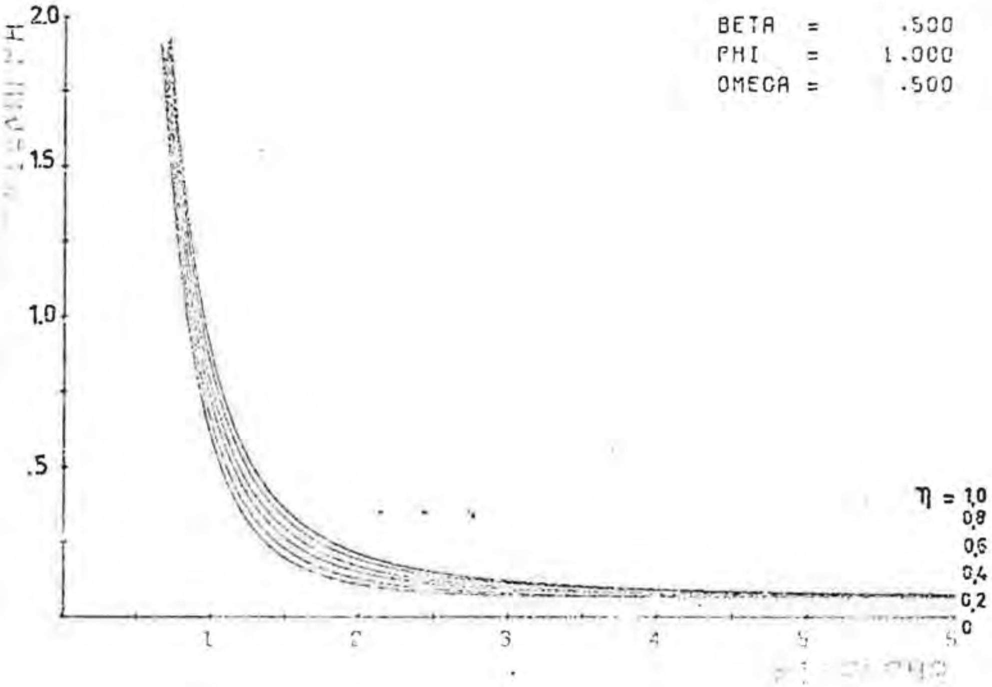
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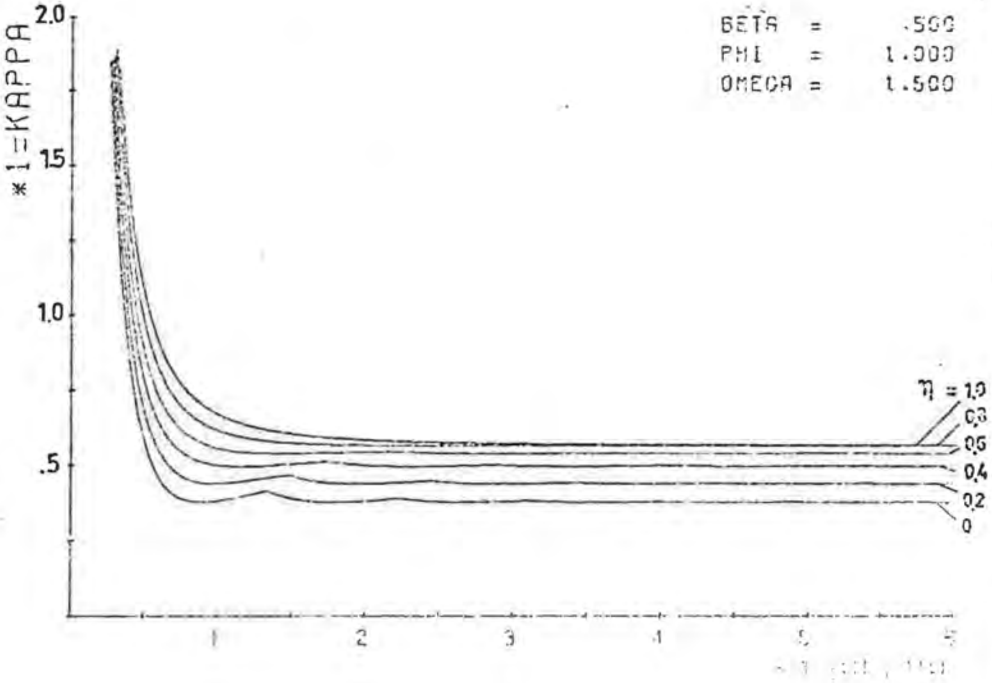
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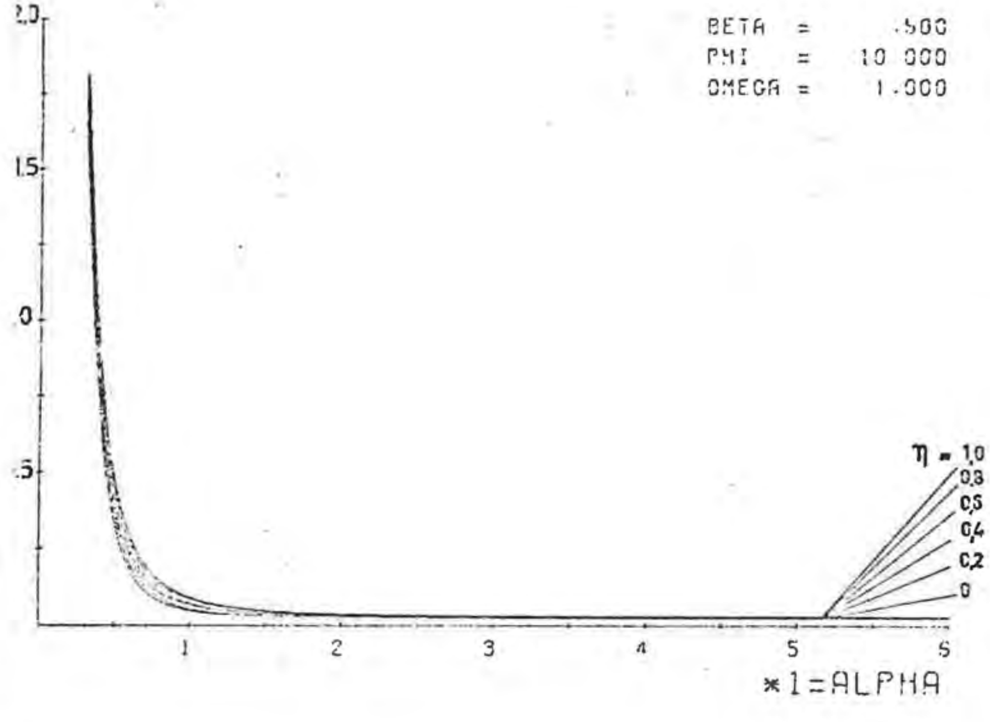
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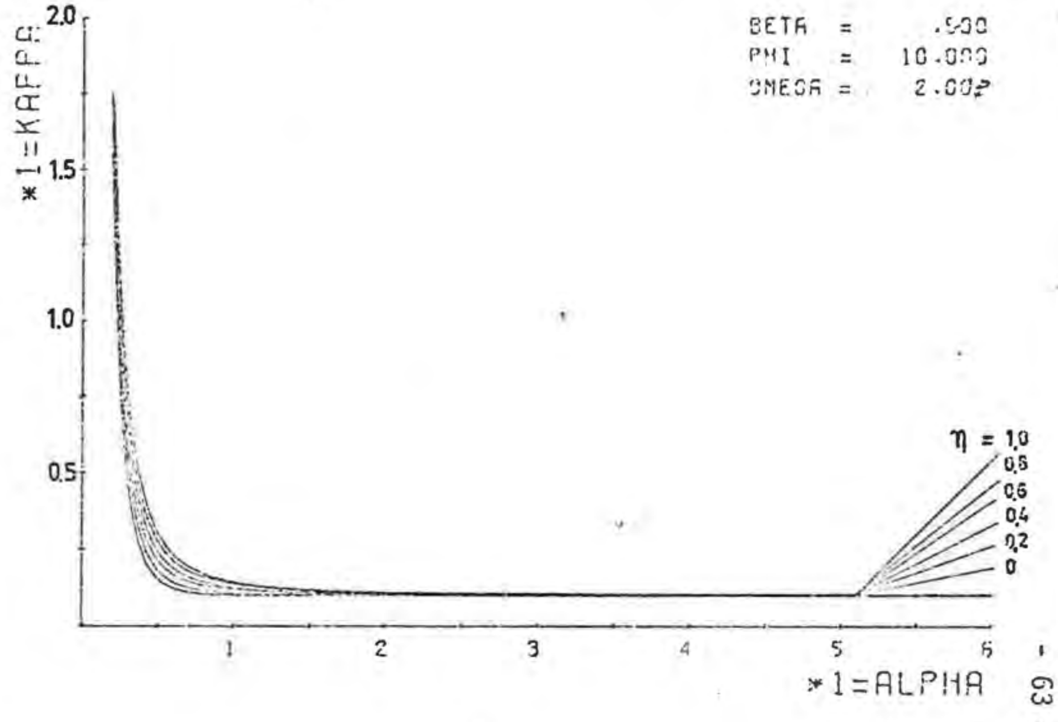
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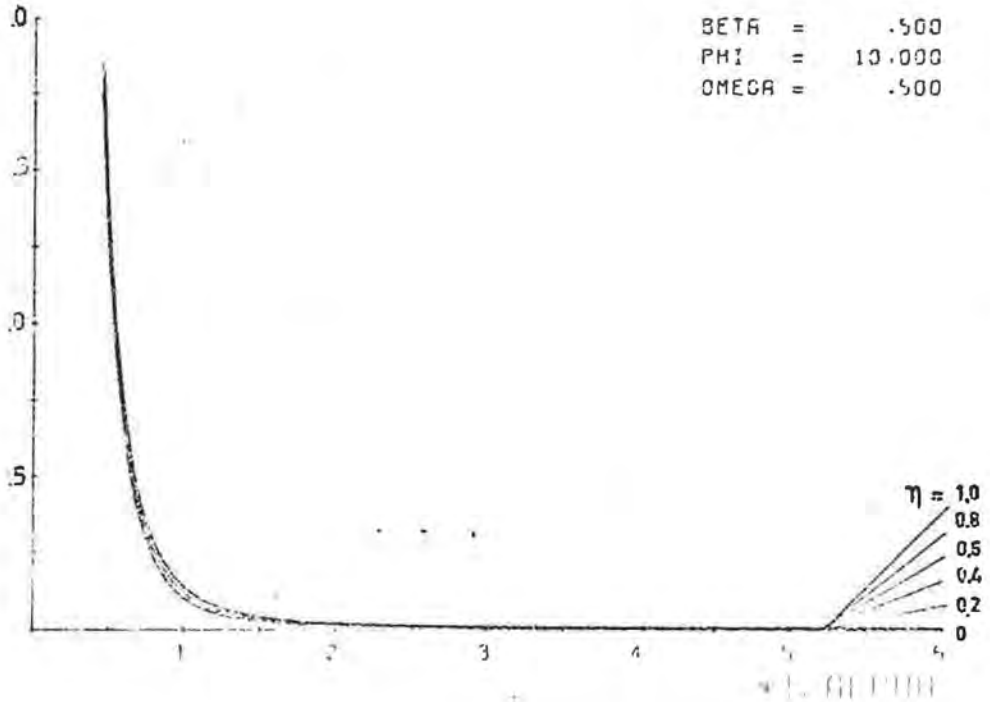
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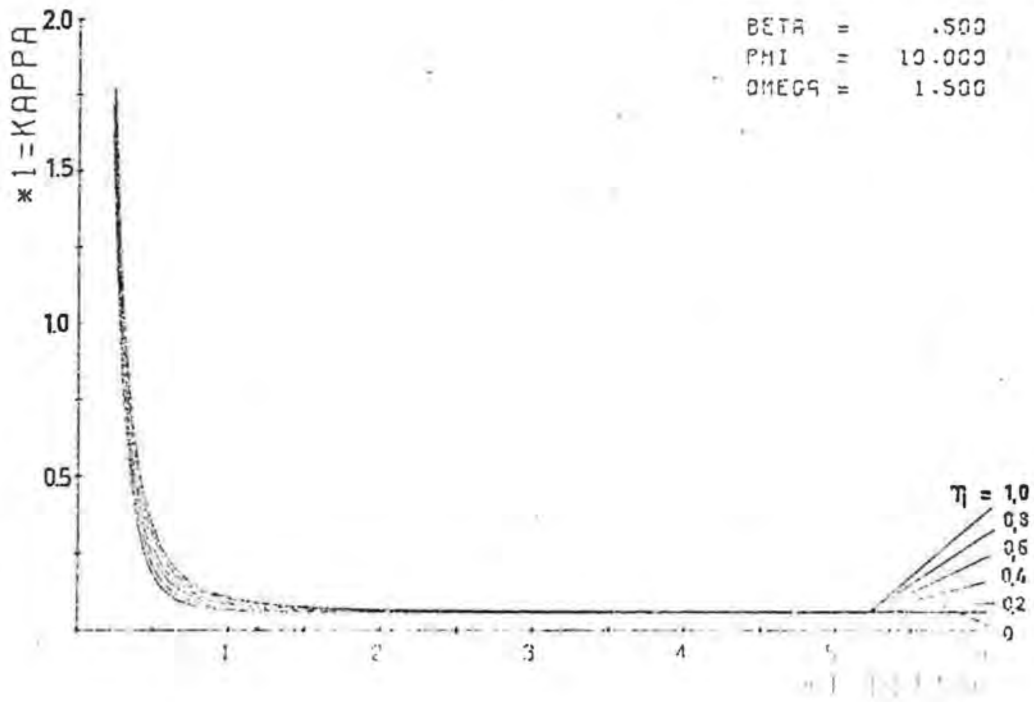
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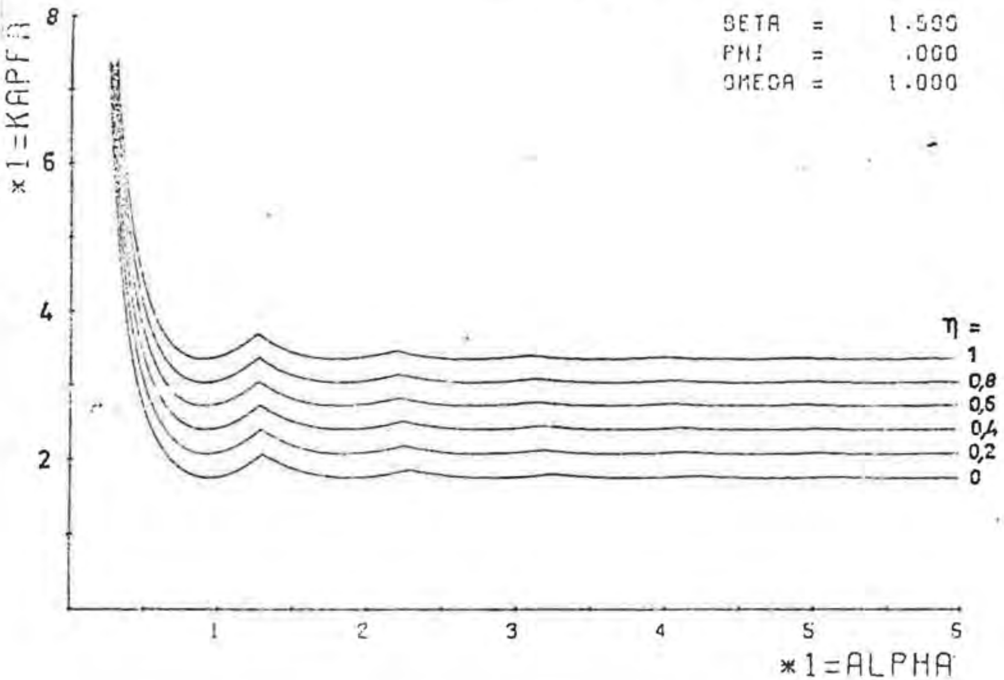

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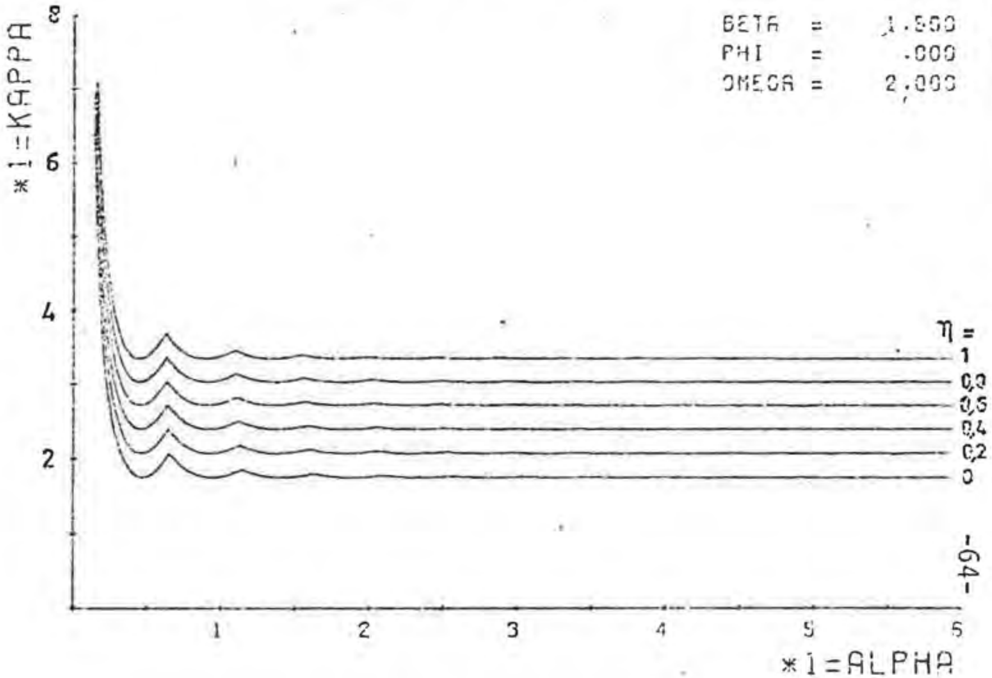

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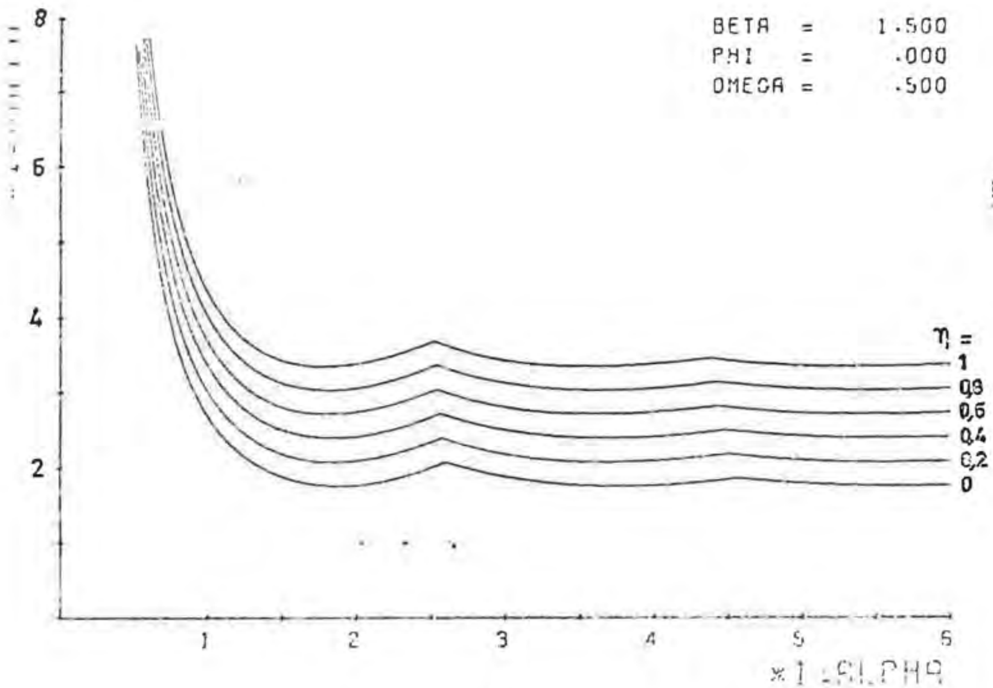

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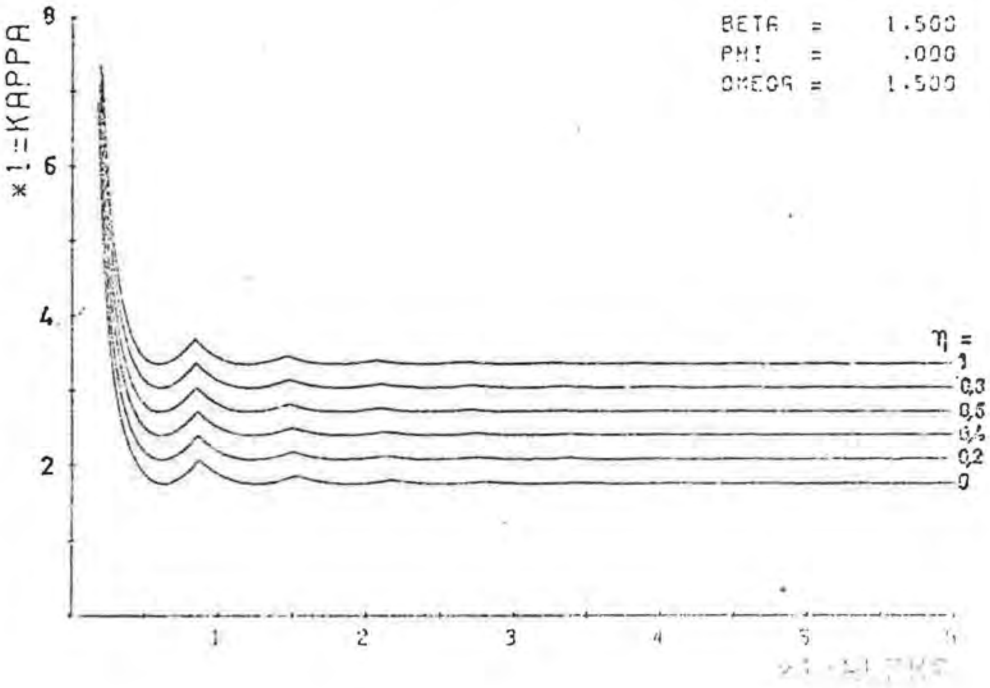

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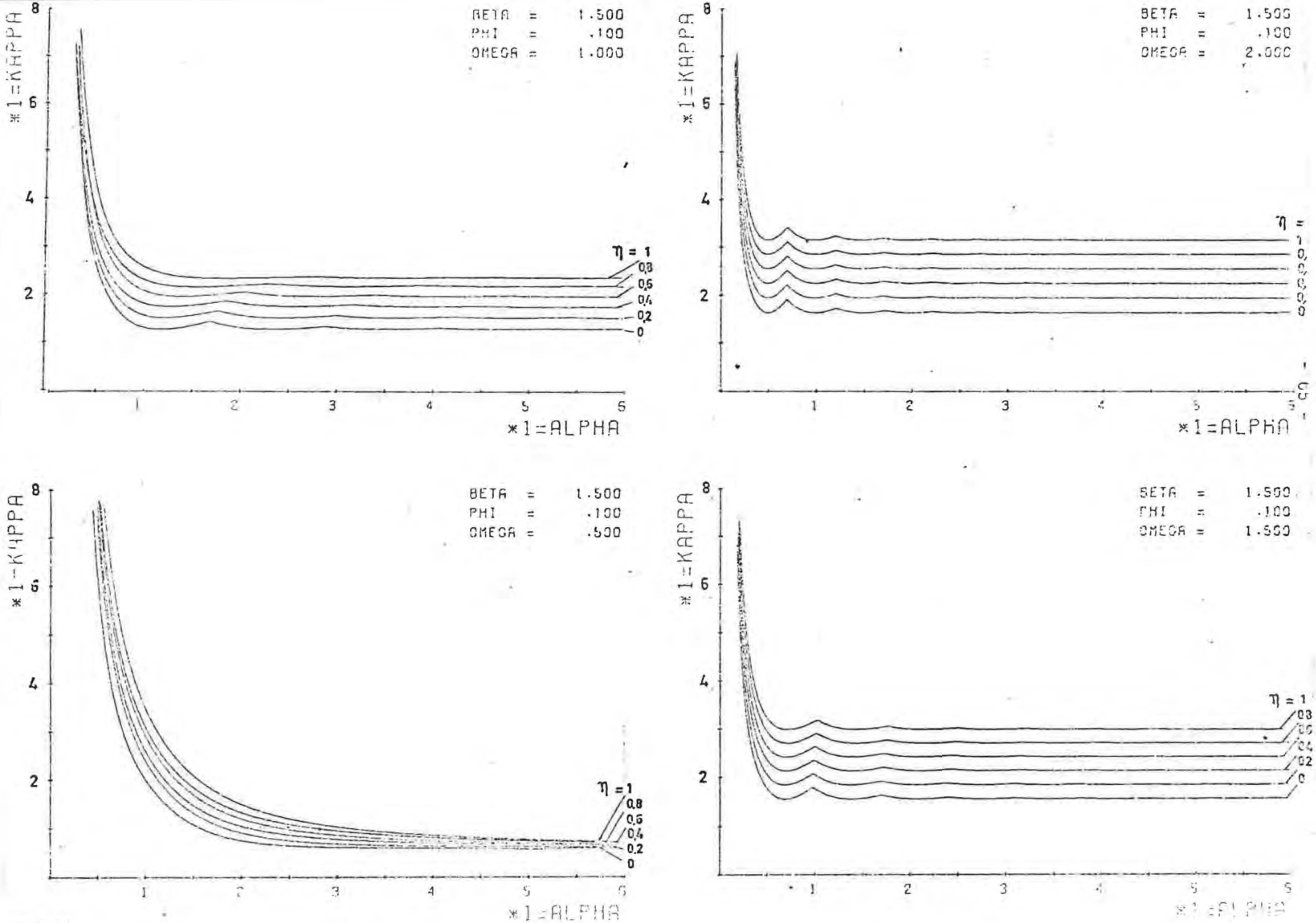
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fig.14

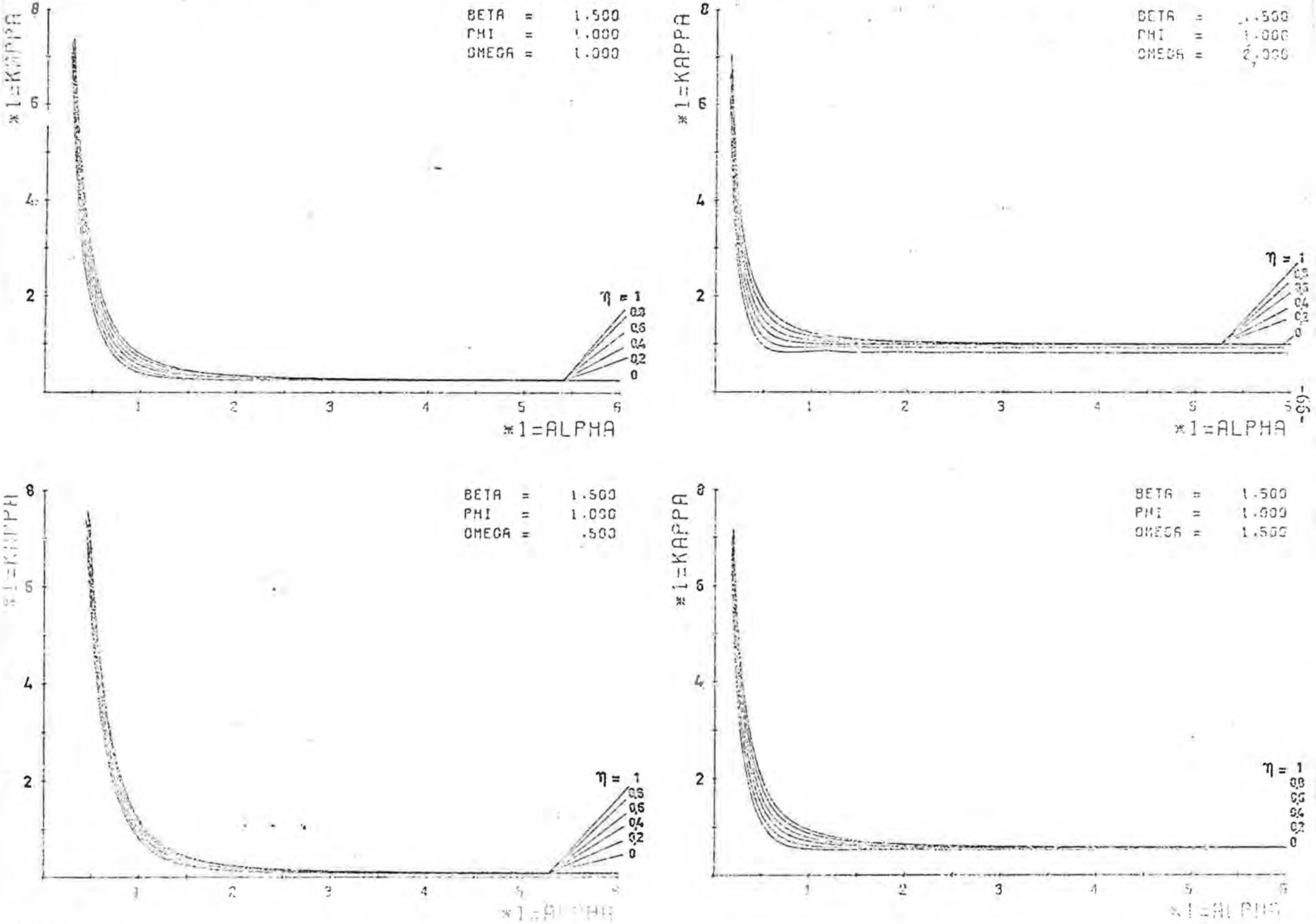
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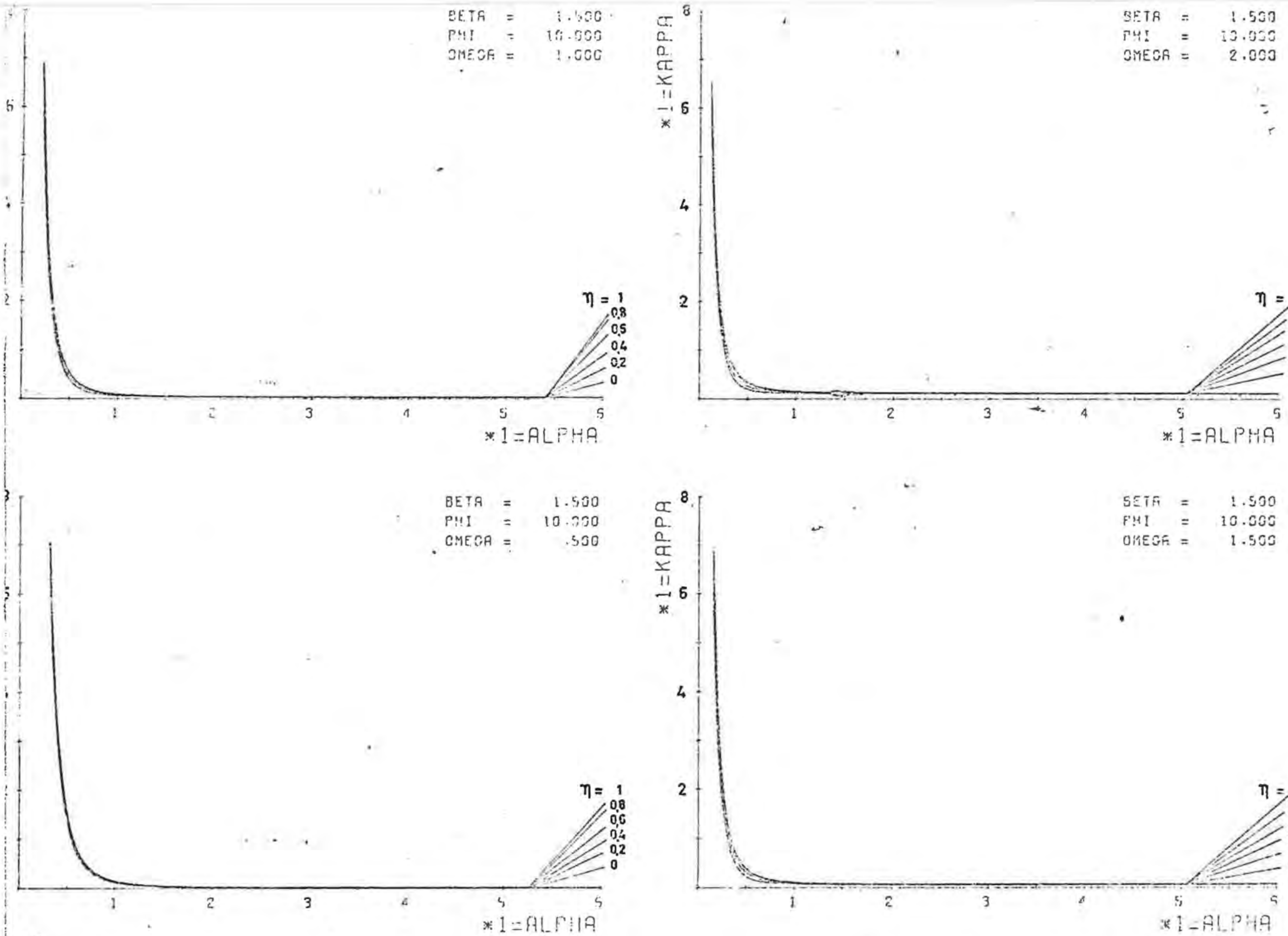
fig 15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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF ROOF BRACING - THE STATE OF THE ART IN SOUTH AFRICA

by

P A V Bryant and J A Simon
CSIR Pretoria
South Africa

MEETING TEN
VANCOUVER, CANADA
AUGUST 1978

DESIGN OF ROOF BRACING – THE STATE OF THE ART IN SOUTH AFRICA

by

P.A.V. Bryant and J.A. Simon*

The NTRI has, over a period of a number of years, been involved in the design of roof bracing. Although this was initially done purely on an empirical basis,[1] work carried out at the NTRI[2] and in Australia[3] has shown that suitable analytical methods can be developed.

As a result of this work, a clause on the design of bracing has been included in the recently published draft South African Code of Practice for the Structural Use of Timber. This clause has been based on the following formula used in the Australian Code :

$$F_L = \frac{0,1\ F_A}{N + 1}$$

where F_L = force on each lateral restraint

F_A = average axial force in strut due to dead load only

N = number of restraints along strut.

In a trussed rafter roof the cumulative lateral force from a number of adjacent rafters in compression is transferred to the wallplate by means of the braced bay, which should, according to the South African code, be designed to resist the cumulative forces from a maximum of <u>five</u> rafters.

* National Timber Research Institute (NTRI), CSIR, Pretoria.
Paper presented at the meeting of Subject Group S5.02 of IUFRO Division 5 held in Vancouver, B.C., September 1978.

The interval between braced bays is set using the empirically developed formula :

interval between bays (m) = 16,5 - 0,3 x span in metres

Using this approach, a sub-committee of the Truss Plate Association of South Africa is currently designing standard bracing details for use on all domestic roofs constructed from prefabricated timber trusses. One of the problem areas has been the design of the connection between the bracing and the wallplate, since the simple nailed detail used at present is inadequate. Connections which are both effective and simple to install are currently being investigated.

As a result of this work on the design of bracing, and as a result of a series of buckling failures experienced with tiled roofs, the influence of the spacing and stiffness of lateral bracing on the strength of braced compression members is currently under review. In most codes, the rafters supporting tiling battens are assumed to be continuously supported along their length, although in many cases the connections between the battens and the rafters do not have a sufficiently high stiffness to validate these design assumptions. The designation of a suitable minimum slenderness ratio to be used for the design of battened rafters may be a rational solution to this problem.

REFERENCES

1. BRYANT, P.A.V. Notes on the construction and erection of timber truss roofs. Hout 123, NTRI, CSIR, Pretoria, 1976
2. SIMON, J.A. Lateral bracing of timber struts. Paper presented to CIB-W.18 meeting, Stockholm, March 1977.
3. LEICESTER, R.H. Design of bracing systems for roof trusses. IUFRO, Delft, Holland, 1975.

CIB-W18/9-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE CIB TIMBER CODE
(Second Draft)

MEETING TEN
VANCOUVER, CANADA
AUGUST 1978

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

WORKING GROUP W18
TIMBER STRUCTURES

CIB TIMBER CODE

Second draft
May 1978

FOREWORD

A first draft of the CIB Timber Code was discussed at CIB-W18 meetings in June 1976 (document CIB-W18/6-100-2), February 1977 (document CIB-W18/6-100-2; Joints) and October 1977 (document CIB-W18/8-100-1; List of Contents).

Based on comments received on these documents a final draft was prepared of the List of Contents and presented to ISO/TC 165 for comment. A new preliminary draft for the Timber Code was prepared and discussed by a Code Drafting Sub-Committee consisting of CIB members G. Booth, W. T. Curry, J. Kuipers, H. J. Larsen, K. Möhler, J. G. Sunley, J. R. Tory and also T. A. Eldridge, F. Keenan and W. R. Meyer representing the Canadian Standards Association and Dutch TNO.

This final version of the code has been prepared by G. Booth, H. J. Larsen and J. R. Tory.

The draft contains only rules for the design of timber structures and recommendations which define their validity. It does not contain rules common to the construction of other structures or safety criteria, and reference is made to Comité Euro-International du Béton, Volume 1, Common Rules for Different Type of Construction and Material. When a final version of this document is produced Chapter 3, Basic Design Rules, will be included.

This draft standard is equally applicable to either deterministic or partial factor methods of design provided material properties are given as characteristic values and suitable safety factors for strength and stiffness parameters are introduced to the design calculations.

Related Documents
The draft code makes reference to other documents at a preliminary stage which have been submitted to ISO/TC 165 for comment. These are:

Timber Structures - Joints - Determination of Strength and Deformation Characteristics of Mechanical Fasteners - prepared by CIB-W18 & RILEM 3TT.

Timber Structures - Plywood - Determination of some Physical and Mechanical Properties.

Timber Structures - Timber in Structural Sizes - Determination of some Physical and Mechanical Properties.

Other documents relating to the sampling of test specimens and the analysis of test data to produce characteristic values will be prepared by CIB-W18.

LIST OF CONTENTS

1. INTRODUCTION

1.1 Scope

Thr primary purpose of this code is to provide an agreed background for the international bodies and national committees responsible for formulating timber codes, to ensure a reasonable and consistent quality of timber structures.

The code relates primarily to the structural use of timber and is intended for use in the design, execution and appraisal of structural elements made from timber or wood products and of structures substantially composed of such elements.

It is based on the principles of structural mechanics, engineering design, and experimental data, interpreted statistically as far as possible.

Deviations from the requirements of this code and the use of materials and methods of design or construction of wood structures not covered by this code are permitted when the validity is substantiated by analytical and engineering principles or reliable test data, or both, which demonstrate that the safety of the resulting structure for the purpose intended is equivalent to the safety demanded in this code.

1.2 Conditions for the validity of this document

Safety and serviceability are not simply functions of design, but depend also on the care and skill of all personnel involved in the construction process, and on the proper use and maintenance of the structure. Essential requirements are therefore that

- projects are carried out by qualified engineers,
- the construction is carried out by personnel having both the required skill and experience,
- the required supervision is always available,
- the structure, by design or the use of suitable materials or by impregnation, is protected against attack by fungi, insects, shipworm, gribble, etc.,
- the intended life of the structure is ensured by correct maintenance,
- the actual conditions of use of the structure during its life do not depart significantly from those specified during the design stage.

1.3 Units

The units used are generally in accordance with the »International System of Units, SI» and »Rules for the Use of the International System of Units» established by ISO and prepared by ISO/TC98/SC2.

Exceptions are the units for time, temperature and plane angle. In accordance with common and well established practice the Celsius scale is used rather than the Kelvin scale for thermodynamic temperature; degrees are used rather than radians as the non-dimensional units of plane angle; and hours, days, weeks, months and years are accepted as derived units of time.

The following basic units and derived units are used for structural timber design calculations:

Table 1.3 Units for structural timber design

Physical quantity	Unit	Abbreviation (and derivation)
Length	Metre	m
Mass	Kilogram	kg
Temperature	Degree Celsius	°C
Time	Second	s
Plane angle	Degree	° $(1° = \frac{180}{\pi})$
Force	Newton	N (1 N = 1 kgm/s^2)
Stress, pressure Elastic moduli	Pascal	Pa (1 Pa = 1 N/m^2, 1 MPa = 1 N/mm^2)

Only multiples of $10^{\pm 3}$; e.g. MN, kN, N are used.

1.4 Notations

The notations used are in accordance with International Standard ISO 3898.

In addition the notations given in document CIB-W18-1 »Symbols for Use in Structural Timber Design» are used.

The following general terms and symbols are used. Symbols which are not explained here are defined when used.

(will be prepared at completion of the work)

1.5 Definitions

(will be prepared at completion of the work)

2. BASIC ASSUMPTIONS

2.1 Characteristic values

The characteristic strength and stiffness values given in this code for timber and wood-based materials are defined as lower 5-percentile values (i.e. 95% of all possible test results exceed the characteristic value) directly applicable to a very short-term load condition (3 to 5 mins.) at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02.

The characteristic bending strength values for solid timber are related also to a section depth of 200 mm.

For some elastic properties the mean values are also given in this code and are defined at the same temperature and humidity conditions as the characteristic values.

The specific gravity for a species or species group is defined as the lower 5-percentile value with mass measured at moisture content $\omega = 0$ and volume measured at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02.

2.2 Climate classes

Structures dependent on moisture content shall be assigned to one of the climate classes given below:

: The examples given below each climate class definition are particularly appropriate to European conditions.

Climate class 0
The climate class is characterized by a temperature of 23 ± 3°C and an average annual relative humidity not exceeding 0.40.

: This climate class corresponds to conditions in permanently heated buildings without artificial air-moistening.

Climate class 1
The climate class is characterized by a temperature of 23 ± 3°C and a relative humidity of the surrounding air never exceeding 0.80 and only exceptionally, and then only for short periods (less than a week), exceeding 0.60.

: The following structures can be included in this class:
: - structures in outer walls in permanently heated buildings where the structures are protected by a well-ventilated tight cladding.

Climate class 2
The climate class is characterized by a temperature of 23 ± 3°C and a relative humidity of the surrounding air only exceptionally, and then only for short periods (less than a week), exceeding 0.80.

: The following structures can be included in this class:
: - structures in not permanently heated, but ventilated, buildings in which no activities particularly likely to give rise to moisture take place, for example, holiday houses, unheated garages and warehouses, together with service space,
: - ventilated roof structures and other structures protected against the weather.

Climate class 3
All other climatic conditions.

: The following structures are included in this class:
: - concrete forms and unprotected scaffolding,
: - marine works.

2.3 Load-duration classes

For strength and stiffness calculations actions are to be assigned to one of the load-duration classes given in table 2.3a.

The load-duration classes are characterized by the effect of a constant load acting for a certain period of time. For variable action the appropriate class is determined on the basis of an estimate of the interaction between the typical variation of the load with time and the rheological properties of the materials or structures.

: In table 2.3b are given examples of loads in the different classes for permanent buildings (i.e. a life time of 50-100
: years).

Table 2.3a Load-duration classes

load - duration class	duration
permanent	$> 10^5$ h (> 10 years)
normal	10^3 - 10^5 h (6 weeks - 10 years)
short-term	10 - 10^3 h (10 h - 6 weeks)
very short-term	< 10 h
instantaneous	< 3 seconds

Table 2.3b Examples of action classifications

permanent	dead load earth and water pressure, loads in some warehouses and storage tanks
normal	floor loads loads in warehouses loads on grandstands and some scaffolds frequent value of snow load in some countries
short-term	load on most scaffolds characteristic value of snow load in some countries frequent value of wind load temperature actions
very short-term	imposed load from persons on roofs not intended for traffic characteristic wind load mooring forces (ships)
instantaneous	wind gusts impact earthquake

REQUIREMENTS FOR MATERIALS

4.0 General

Strength and stiffness properties shall be determined by tests for all actions to which the material may be subjected in the structure.

It must be shown that the form stability, environmental behaviour etc. are satisfactory for the purposes of construction and eventual end-use.

4.1 Solid structural timber

4.1.0 General

Structural timber, i.e. timber where the strength and stiffness are of importance, shall be graded in accordance with rules ensuring that the strength, stiffness and other properties of the timber are satisfactory.

The strength grading rules may be based on a visual assessment of the timber, on the non-destructive measurement of one or more properties or on a combination of the two methods.

Strength and stiffness parameters shall be determined by standardized short-term tests in accordance with ISO/TC 165: Timber structures - Timber in structural sizes - Determination of some physical and mechanical properties.

4.1.1 Standard strength classes

In this code the following standard strength classes are used for solid timber: SC15, SC19, SC24 and SC30.

A given visual grade can be referred to one of the standard strength classes if the characteristic bending strength, f_m (5-percentile), and the mean modulus of elasticity in bending, $E_{0,mean}$, are not less than the values given in table 4.1.1. For machine stress-rated timber it should further be shown that the characteristic tensile strength, $f_{t,0}$, is not less than given in the table.

Table 4.1.1 Standard strength classes. Minimum characteristic values in MPa and UN/ECE grades which comply

standard strength class	SC15	SC19	SC24	SC30
bending f_m	15	19	24	30
bending $E_{0,mean}$	6000	7200	8500	10000
tension $f_{t,0}$	6	9	16	20
UN/ECE-grade		S6	S8	S10

: It is emphasized that the introduction of standard strength classes does not prevent the introduction of other grades
: with for example higher values for $E_{0,mean}/f_m$ and $f_{t,0}/f_m$.
: European redwood/whitewood graded according to UN/ECE Recommended standard for stress grading of coniferous
: sawn timber (UN/ECE TIM/WP.3/AC3/B-Annex I) can be assumed to meet the demands of standard grades as given
: in table 4.1.1.
: Annex 4 contains a survey of which national grades can be assumed to satisfy the requirements of the different
: standard grades. (Under preparation).

The specification of structural timber by strength classes (sections 4.1.1, 4.2.1 and tables 5.1.0a and 5.1.0b) has been included as a background for discussion and as an illustration of a simpler method for the engineer than the one based on species and grades which is generally used.

4.2 Finger jointed structural timber

4.2.0 General

The manufacture of finger jointed structural timber should be subject to external control which does not require less of the production than stated in UN/ECE Recommended standard for finger jointing in structural coniferous sawn timber (UN/ECE TIM/WP.3/AC3/8-Annex II).

Strength and stiffness parameters shall be determined according to section 4.1.0 coupled with the rules in the above-mentioned UN/ECE Recommended standard.

4.2.1 Standard structural classes

Finger jointed structural timber can be referred to one of the standard strength classes stated in 4.1.1 if the characteristic values are not less than given in table 4.1.1.

: European redwood/whitewood finger jointed according to the UN/ECE Recommended standard category A can be
: assumed to satisfy the requirements of SC24, and category B the requirements of SC19.

4.3 Glued laminated timber

4.3.0 General

The manufacture of glued laminated timber (glulam) should be subject to external control which does not require less of the production than stated in (CIB-glulam standard under preparation).

In principle, strength and stiffness parameters shall be determined as given in section 4.1.0, combined with recognized methods for determining the strength and stiffness of the glulam from the properties of the laminae.

4.3.1 Standard glulam strength classes

In this code the following standard glulam strength classes are used: SCL30, SCL38, SCL47.

Glulam made from the same wood species in the entire cross-section may be referred to a standard glulam strength class if the characteristic bending strength, f_m, and its mean modulus of elasticity in bending, $E_{0,mean}$, are not less than the values given in table 4.3.1. In other cases it is furthermore required that the characteristic tensile strength is not less than given in the table.

Table 4.3.1 Standard glulam strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

		standard glulam strength class		
		SCL30	SCL38	SCL47
bending	f_m	30	37.5	47
bending	$E_{0,mean}$	10000	12000	12000
tension	$f_{t,0}$	20	25	30

: Glulam made from finger jointed timber corresponding to SC30 in the extreme eighths of the cross-section on
: either side, however at least two lamellas on either side, and to SC24 in the rest of the cross-section can be con-
: sidered to correspond to SCL38. A corresponding combination of SC24 and SC19 can be assumed to correspond
: to SCL 30.
: CIB-W18 will produce an annex to this code indicating how the requirements of these standard glulam strength
: classes may be met by existing national practices.

4.4 Wood-based sheet materials

The manufacture of plywood, particle board and fibre board for load-bearing structures shall be subject to an approved control arrangement.

The specification of glued laminated members by strength classes (sections 4.3.1 and table 5.2.0) has been included as a background for discussion and as an illustration of a simpler method for the engineer than the one based on species and grades which is generally used.

Testing must be carried out in accordance with the following standards:

For plywood: ISO/TC 165: Timber structures. Plywood. Determination of some physical and mechanical properties.

For particle board and fibre board:

4.5 Glue

Only glue giving joints of such strength that the integrity of the glue-line is maintained throughout the life of the structure, is allowed for timber structures.

4.6 Mechanical fasteners

Refer to chapter 6.

4.7 Steel parts

Nails, screws and other steel parts should as a minimum be protected against corrosion according to Table 4.7. The requirements for protection against corrosion may be relaxed for heavy steel parts where surface corrosion will not significantly reduce the load-carrying capacity.

Table 4.7 Minimum protection against corrosion

climate class	steel parts except nails and screws	nails, screws and bolts
0	none	none
1 2	galvanizing with a min. thickness of 20 μm	none
3	hot galvanizing with a minimum thickness of 70 μm	

: The consideration for the finish of the structures may call for stricter rules for corrosion protection, especially
: in climate class 2. Attention is drawn to the fact that certain woods, e.g. oak, and some treatments, e.g. fire re-
: tardant, may have a corroding effect on unprotected steel.

5. DESIGN OF BASIC MEMBERS

5.1 Solid timber members

5.1.0 Characteristic values

Characteristic values for the standard strength classes defined in section 4.1.1 are given in table 5.1.0 a. For the load-duration classes and climate classes defined in sections 2.2 and 2.3 the factors in table 5.1.0 b should be applied.

Table 5.1.0 a Characteristic values and mean elastic moduli, in MPa

		SC15	SC19	SC24	SC30
characteristic values (for strength calculations)					
bending	f_m	15	19	24	30
tension parallel to grain	$f_{t,0}$	6	9	16	20
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	14	18	23	28
compression perpendicular to grain	$f_{c,90}$	6	7	7	7
shear*	f_v	2.5	3	3	3
modulus of elasticity	E_0	4200	5400	6900	8000
mean values (for deformation calculations)					
modulus of elasticity, parallel	$E_{0,mean}$	6000	7200	8500	10000
modulus of elasticity, perpendicular	$E_{90,mean}$	250	300	350	400
shear modulus	G_{mean}	500	600	700	800

* In rolling shear the shear strength may be put equal to $f_v/2$

Table 5.1.0 b Factors to the basic values

values for	strength calculations		deformation calculations		
climate classes	0, 1 and 2	3	0 and 1	2	3
permanent	0.6 (0.4)	0.5 (0.35)	0.7	0.6	0.4
normal	0.6 (0.4)	0.5 (0.35)	1	0.8	0.7
short-term	0.7 (0.6)	0.6 (0.5)	1	0.8	0.7
very short-term	0.85 (0.8)	0.7 (0.65)	1	0.8	0.7
instantaneous	1.0 (1.0)	0.85 (0.85)	-	-	-

Where a load case is composed of loads belonging to different load-duration classes the values corresponding to the shortest load may be used.

Values in parantheses apply to tension perpendicular to grain.

5.1.1 Straight beams, columns and tension members

5.1.1.0 General

This section applies to prismatic or cylindrical as well as slightly conical members, e.g. timber logs and poles.

The effective span of flexural members shall be taken as the distance between the centres of areas of bearing. With members extending further than is necessary over bearings the span may be measured between centres of bearings of a length which would be adequate according to this code; attention should be paid to the eccentricity of the load where advantage is taken of this provision.

See footnote on page 4.1.

The effective cross-section and geometrical properties of a structural member shall be calculated from the minimum cross-section acceptable for the given nominal size or from the actual cross-section. Nominal dimensions may be used in calculations when the actual dimensions at a moisture content of 0.20 are not less than the nominal dimensions reduced by 1 mm for dimensions of 100 mm or less; 2 mm for dimensions between 100 mm and 200 mm and 1 per cent for larger dimensions.

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary by nails and screws with a diameter of 6 mm or less.

5.1.1.1 Tension

The stresses shall satisfy the following conditions:

$$\sigma_t \leq k_{size,0} f_{t,0} \qquad (5.1.1.1\ a)$$

for tension parallel to the grain direction, and

$$\sigma_t \leq k_{size,90} f_{t,90} \qquad (5.1.1.1\ b)$$

for tension perpendicular to the grain, and

$$k_{size,90} = \begin{cases} 1 & \text{for} \quad V \leq 0.02\ m^3 \\ \left(\frac{0.02}{V}\right)^{0.2} & \text{for} \quad V \geq 0.02\ m^3 \end{cases} \qquad (5.1.1.1\ c)$$

for a volume of V uniformly load in tension perpendicular to the grain. Other examples of $k_{size,90}$ are given in section 5.2.2.

: Recommendations on the size factor $k_{size,0}$ will be produced.
: A recommendation on the method of calculation for tension strength at an angle to the grain is also in the course
: of preparation.

5.1.1.2 Compression without column effect

For compression at the angle θ to the grain the stresses should satisfy the following condition:

$$\sigma_c \leq f_{c,0} - (f_{c,0} - f_{c,90})\sin\alpha \qquad (5.1.1.2\ a)$$

cf. fig. 5.1.1.2 a.

: This condition only ensures that the compressive stress directly under the load is acceptable, but not that an ele-
: ment in compression can carry the load in question. Refer to section 5.1.1.9.

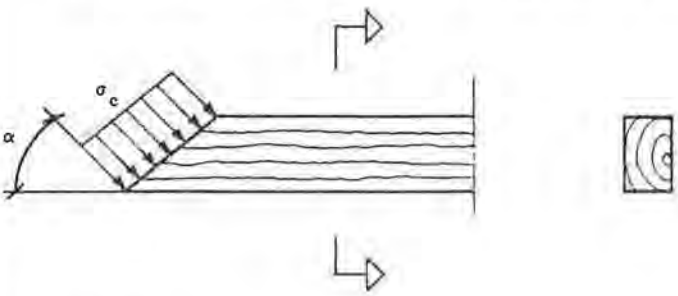
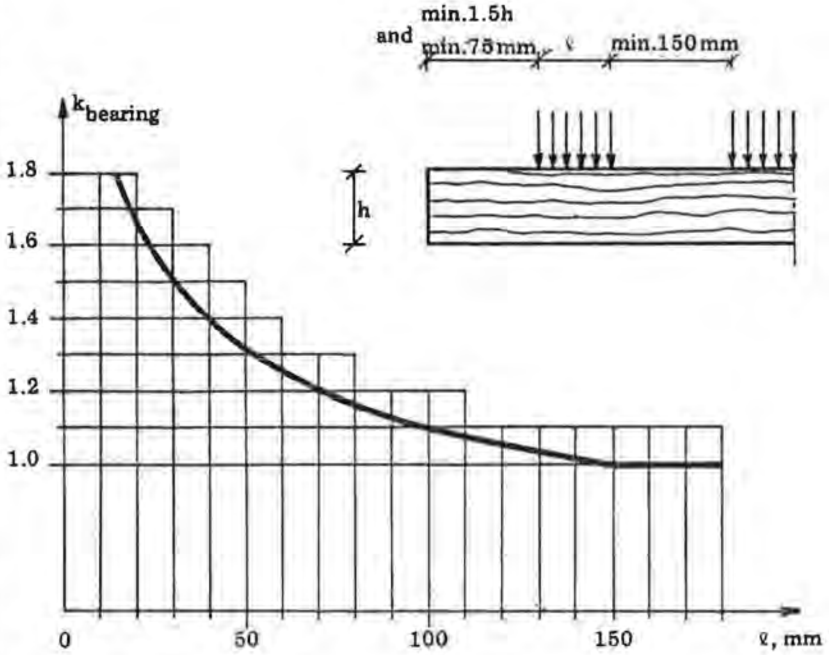


Fig. 5.1.1.2 a

For bearings on the side grain ($\alpha = 90°$) formula (5.1.1.2 a) may be replaced by

$$\sigma_c \leq k_{bearing} f_{c,90} \qquad (5.1.1.2\ b)$$

For bearings located at least 75 mm and 1.5 h from the end and 150 mm from other loads $k_{bearing}$ may be taken from fig. 5.1.1.2 b. In other cases $k_{bearing} = 1$.



$k_{bearing} = \sqrt[4]{150/\ell}$

$1 < k_{bearing} < 1.8$

Fig. 5.1.1.2 b

Where the deformations resulting from compression perpendicular to the grain are significant to the function of a structure, an estimate of the deformations shall be made.

: The strain perpendicular to the grain can be estimated as $\sigma_c/(k_{bearing}E_{90,mean})$.

5.1.1.3 Bending

For pure bending the following condition shall be satisfied:

$$\sigma_m \leqslant k_{depth} k_{inst} f_m \qquad (5.1.1.3\ a)$$

where k_{depth} is a factor ($\leqslant 1$) taking into account the reduced strength of deep sections:

$$k_{depth} = \begin{cases} 1 & \text{for } h \leqslant 200 \text{ mm} \\ \left(\frac{200}{h}\right)^{1/9} & \text{for } h \geqslant 200 \text{ mm} \end{cases} \qquad (5.1.1.3\ b)$$

where k_{inst} is a factor ($\leqslant 1$) taking into account the reduced strength due to failure by lateral instability (lateral buckling). k_{inst} is determined so that the total bending stresses, taking into account the influence from initial curvature, eccentricities and the deformations developed, do not exceed f_m.

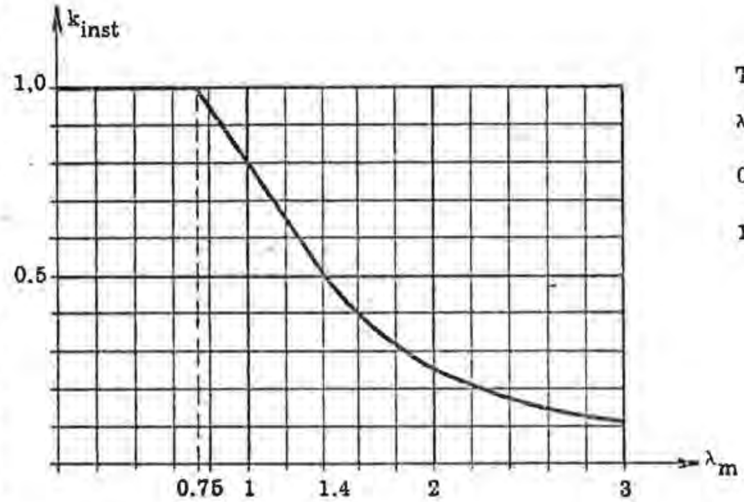
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

$$\lambda_m = \sqrt{f_m/\sigma_{m,crit}} \leqslant 0.75 \qquad (5.1.1.3\ c)$$

In (5.1.1.3 c) λ_m is the slenderness ratio for bending, and $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability.

k_{inst} may also be put equal to 1 for beams where lateral displacement of the compression side is prevented throughout its length and where torsion is prevented at the supports.

k_{inst} may be determined from fig. 5.1.1.3 if the lateral deviation from straightness measured at midspan is less than $\ell/200$.



The curve corresponds to

$\lambda_m < 0.75$; $k_{inst} = 1$

$0.75 < \lambda_m < 1.4$; $k_{inst} = 1.56 - 0.75\,\lambda_m$

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m$

Fig. 5.1.1.3

For a beam with rectangular cross-section k_{inst} may be determined from fig. 5.1.1.3 dependent on the slenderness ratio λ_m determined from

$$\lambda_m = \sqrt{\frac{\ell_c h}{b^2}\,\frac{f_m}{E_0}\,\sqrt{\frac{E_{0,mean}}{G_{mean}}}} \qquad (5.1.1.3\ d)$$

where ℓ_e is the effective length of the beam. For a number of structures and load combinations ℓ_e is given in table 5.1.1.3 in relation to the free beam length ℓ.

The free length is determined as follows:

a) When lateral support to prevent rotation is provided and no other support to prevent rotation or lateral displacement is provided throughout the length of a beam, the unsupported length shall be the distance between such points of bearing, or the length of a cantilever.

b) When beams are provided with lateral support to prevent both rotation and lateral displacement at intermediate points as well as at the ends, the unsupported length may be the distance between such points of intermediate lateral support. If lateral displacement is not prevented at points of intermediate support, the unsupported length shall be as defined in a).

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

Type of beam and load	ℓ_e/ℓ
Simply supported, uniform load or equal end moment	0.35
Simply supported, concentrated load at centre	0.30
Cantilever, uniform load	0.20
Cantilever, concentrated end load	0.30
Cantilever, end moment	0.35

The values apply to loads acting in the gravity axis. For downwards acting loads ℓ_c is increased by 0.75 h for loads on the top side and reduced by 0.25 h for loads on the bottom side.

5.1.1.4 Shear

The shear stresses shall satisfy the following condition

$$\tau \leqslant f_v \qquad (5.1.1.4\ a)$$

For beams with bearing in the bottom side and load on the top, loads placed nearer than the beam depth from the support may be disregarded in calculation of the shear force. Loads placed more than the beam depth but less than twice the beam depth from the support may be reduced to one-half their true value in calculation of the shear force.

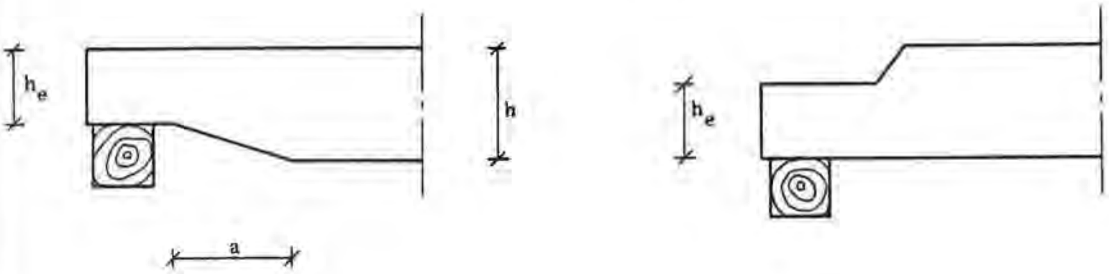


Fig. 5.1.1.4

For beams notched at the ends, see fig. 5.1.1.4, the shear stresses should be calculated on the effective depth h_e, and for notches in the bottom the condition (5.1.1.4 a) should, for $a < 3(h - h_e)$, be replaced by

$$\tau \leqslant \frac{h}{6h - 5h_e} f_v \qquad (5.1.1.4\ b)$$

Notches with $h_e < 0.5$ h are not allowed.

: Notches or abrupt changes of section that will produce tension perpendicular to grain stresses at the notch should
: be avoided. Stress concentrations produced are likely to cause splitting at the notch at low tension values and no
: satisfactory means are available for determining this tension stress. A gradual change of section, perhaps by well
: rounded corners, will reduce these stress concentrations.
: Further recommendations on notching are being considered.

5.1.1.5 Torsion

The torsional stresses, τ_{tor}, calculated according to the theory of elasticity shall satisfy the following condition

$$\tau_{tor} \leqslant f_v \qquad (5.1.1.5)$$

5.1.1.6 Combined stresses

General

: Recommendations on combined stresses are being prepared.

Tension and bending

Only the case with tension in the grain direction is considered.

The stresses should satisfy the following condition

$$\frac{\sigma_t}{f_{t,0}} + \frac{\sigma_m}{f_m} \leqslant 1 \qquad (5.1.1.6\ a)$$

and in the parts of the cross-section, if any, where $\sigma_t + \sigma_m \leqslant 0$, furthermore

$$|\sigma_m| - \sigma_t \leq f_m \qquad (5.1.1.6\ b)$$

Compression and bending without column effect

Only the case with compression in the grain direction is considered.

The stresses in the parts of the cross-section, where $\sigma_m + \sigma_c \leq 0$ should satisfy the following condition

$$\frac{|\sigma_c|}{f_{c,0}} + \frac{|\sigma_m|}{f_m} \leq 1 \qquad (5.1.1.6\ c)$$

and in the parts of the cross-section, if any, where $\sigma_c + \sigma_m \geq 0$

$$\sigma_m + \sigma_c \leq f_m \qquad (5.1.1.6\ d)$$

: **The condition only ensures that the stresses directly under the load are acceptable, but not that e.g. a laterally loaded**
: **column can carry the load in question. Reference is made to section 5.1.1.7.**

Torsion and shear

The stress τ from shear and τ_{tor} from torsion calculated as stated in section 5.1.1.4 and section 5.1.1.5 shall satisfy the following condition

$$\frac{\tau^2}{f_v} + \tau_{tor} \leq f_v \qquad (5.1.1.6\ e)$$

5.1.1.7 Compression and bending with column effect (columns)

For columns it must be verified that the conditions in section 5.1.1.6 for compression and bending are satisfied, when apart from bending stresses from lateral load, if any, the bending stresses from initial curvature and stresses caused by the deflections are taken into consideration.

These conditions can be assumed satisfied if the stresses meet the following demand:

$$\frac{|\sigma_c|}{k_{col} f_{c,0}} + \frac{|\sigma_m|}{f_m} \frac{1}{1 - \frac{k_{col}}{k_E} \frac{|\sigma_c|}{f_{c,0}}} \leq 1 \qquad (5.1.1.7\ a)$$

σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_{col} and k_E are factors depending on the slenderness ratio λ, the material parameters and the initial curvature. The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

$$e = \eta r_{core} \lambda \qquad (5.1.1.7\ b)$$

where r_{core} is the core radius.

$$k_E = \frac{\sigma_E}{f_{c,0}} = \frac{\pi^2 E_0}{f_{c,0} \lambda^2} \qquad (5.1.1.7\ c)$$

where E_0 is the characteristic value of modulus of elasticity

$$k_{col} = 0.5[(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E) - \sqrt{(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E)^2 - 4k_E}\,] \qquad (5.1.1.7\ d)$$

σ_E is the Euler stress.

: Fig. 5.1.1.7 gives k_{col} and k_{col}/k_E for columns with $e < \ell_c/300$, i.e. $\eta \sim 0.005$, $f_{c,0}/f_m = 0.96$ is assumed.

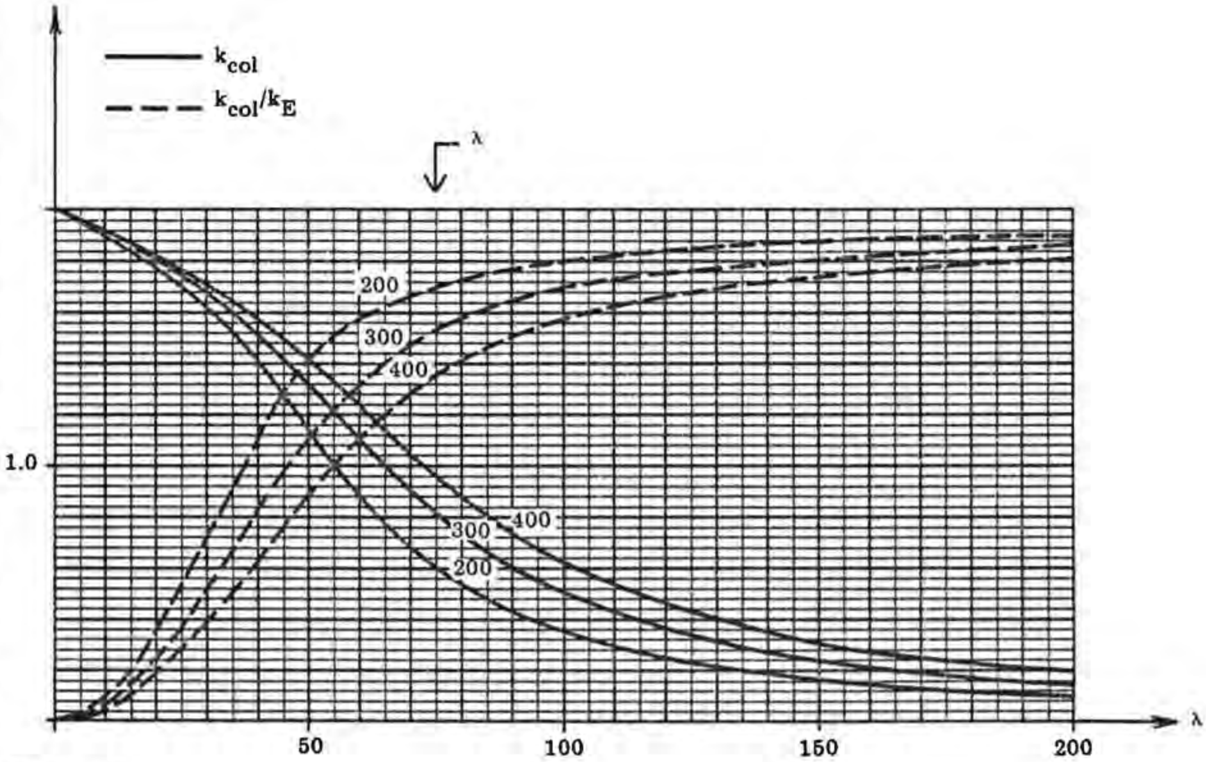


Fig. 5.1.1.7

: The condition (5.1.1.7 a) is on the safe side in cases where the tension side is decisive, cf. (5.1.1.6 d).

For the purpose of calculating the slenderness ratio of compression members, the values of the length ℓ_c should be calculated for the worst conditions of loading to which a compression member is subjected, paying regard to the induced moments at the ends or along the length of the compression member and to slip in the connections. The length should be judged to be the distance between two adjacent points of zero bending moment, these being two points between which the deflected member would be in single curvature.

For straight members with mechanical fasteners the values of ℓ_c can be taken from table 5.1.1.7. The actual length of the member is denoted ℓ.

Table 5.1.1.7 Relative effective length of compression members

Condition of end restraint	ℓ_c/ℓ
Restrained at both ends in position and direction	0.7
Restrained at both ends in position and one end in direction	0.85
Restrained at both ends in position but not in direction	1.00
Restrained at one end in position and direction and at the other end partially restrained in direction but not in position	1.50
Restrained at one end in position and direction, but not restrained in either position or direction at the other end	2.00

The slenderness ratio should not exceed 170, or for secondary members, 200.

5.1.2 Cambered beams

Relevant parts of section 5.2.2 may be applied.

5.2 Glued laminated members

5.2.0 Characteristic strength and stiffness values

Characteristic values for the standard glulam strength classes defined in section 4.2.1 are given in table 5.2.0. For the load duration classes and climate classes defined in sections 2.2 and 2.3 the factors in table 5.1.0 b should be applied.

Table 5.2.0 Characteristic values and mean elastic moduli, in MPa

		SCL30	SCL38	SCL47
Characteristic values (for strength calculations)				
bending	f_m	30	38	47
tension parallel to grain	$f_{t,0}$	20	25	30
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	28	36	45
compression perpendicular to grain	$f_{c,90}$	7	7	7
shear*	f_v	3	3	3
modulus of elasticity	E_0	8000	9600	9600
Mean values (for deformation calculation)				
modulus of elasticity parallel to grain	$E_{0,mean}$	10000	12000	12000
modulus of elasticity perpendicular to grain	$E_{90,mean}$	400	500	500
shear modulus	G_{mean}	800	1000	1000

* In rolling shear the shear strength may be put equal to $f_v/2$

5.2.1 Straight beams and columns

Section 5.1.1 for solid timber applies except that

$$k_{depth} = \begin{cases} 1 & \text{for } h \leq 300 \text{ mm} \\ (\frac{300}{h})^{1/9} & \text{for } h > 300 \text{ mm} \end{cases} \qquad (5.2.1.3)$$

cf. formulas (5.1.1.3 a) and (5.1.1.3 b).

5.2.2 Cambered beams

This section applies to double tapered curved beams with rectangular cross-section (fig. 5.2.2 a) and double tapered beams with flat soffit and rectangular cross-section (fig. 5.2.2 b). In the latter case $h/r_m = 0$, cf. below.

The influence of the cross-sectional variation shall be taken into account. Especially it shall be ensured that the tensile stresses satisfy the condition 5.1.1.1 b, i.e.

$$\sigma_t \leq k_{size,90} f_{t,90} \qquad (5.2.2\text{ a})$$

with

$$k_{size,90} = \begin{cases} \frac{0.5}{V^{0.2}} & \text{for uniformly distributed load} \\ \frac{0.35}{V^{0.2}} & \text{for other loading} \end{cases} \qquad (5.2.2\text{ b})$$

See footnote on page 4.2.

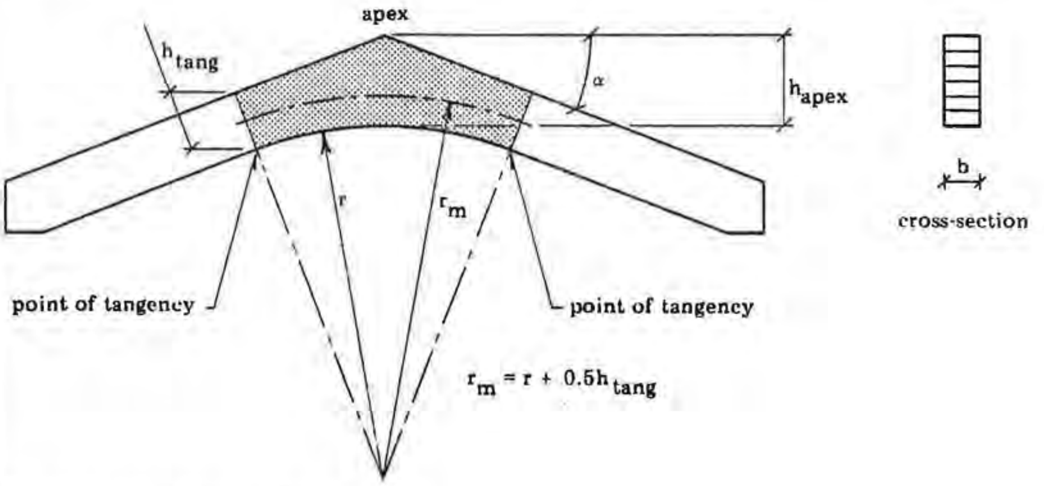


Fig. 5.2.2 a Double tapered curved beam

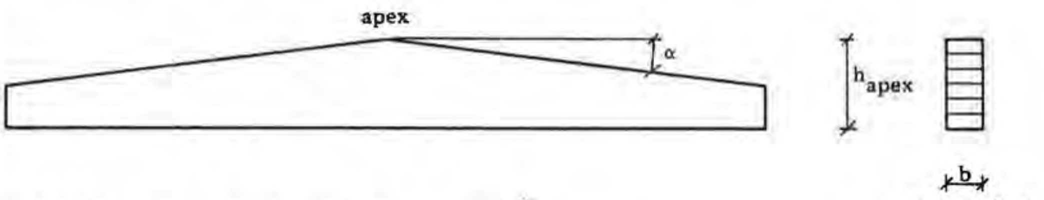


Fig. 5.2.2 b Double tapered beam with flat soffit

For double tapered curved beams V is the beam volume between the points of tangency (corresponding to the shaded area in fig. 5.2.2 a). V shall, however, not be taken as less than $V = 0.6\ bh^2_{apex}$.

For double tapered beams with flat soffit $V = 0.6\ bh^2_{apex}$.

: The following method may be used for calculating the maximum stresses in beams with rectangular cross-section.

: The radial tensile stresses perpendicular to the grain are at a maximum near the mid-depth of the apex, and the maximum value can be calculated as

$$\sigma_t = k_t \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2\ c)$$

: where M_{apex} is the bending moment at the apex-section and k_t is given in fig. 5.2.2 c for $E_{0,mean}/E_{90,mean} = 15$ and $E_{0,mean}/E_{90,mean} = 30$.

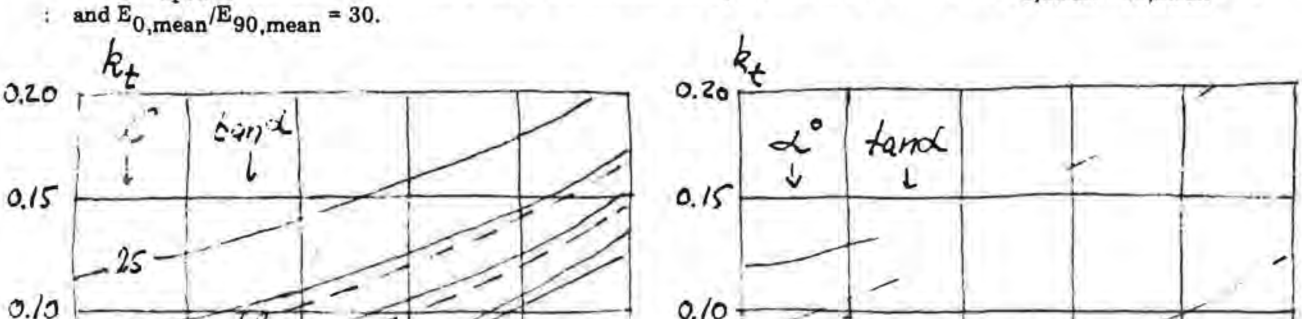


Fig. 5.2.2 c

: The maximum bending stress in the apex cross-section occurs at the lower face and can be calculated as

$$\sigma_m = k_m \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2\ d)$$

: where k_m is given in fig. 5.2.2 d for $E_{0,mean}/E_{90,mean} = 15$ and $E_{0,mean}/E_{90,mean} = 30$.

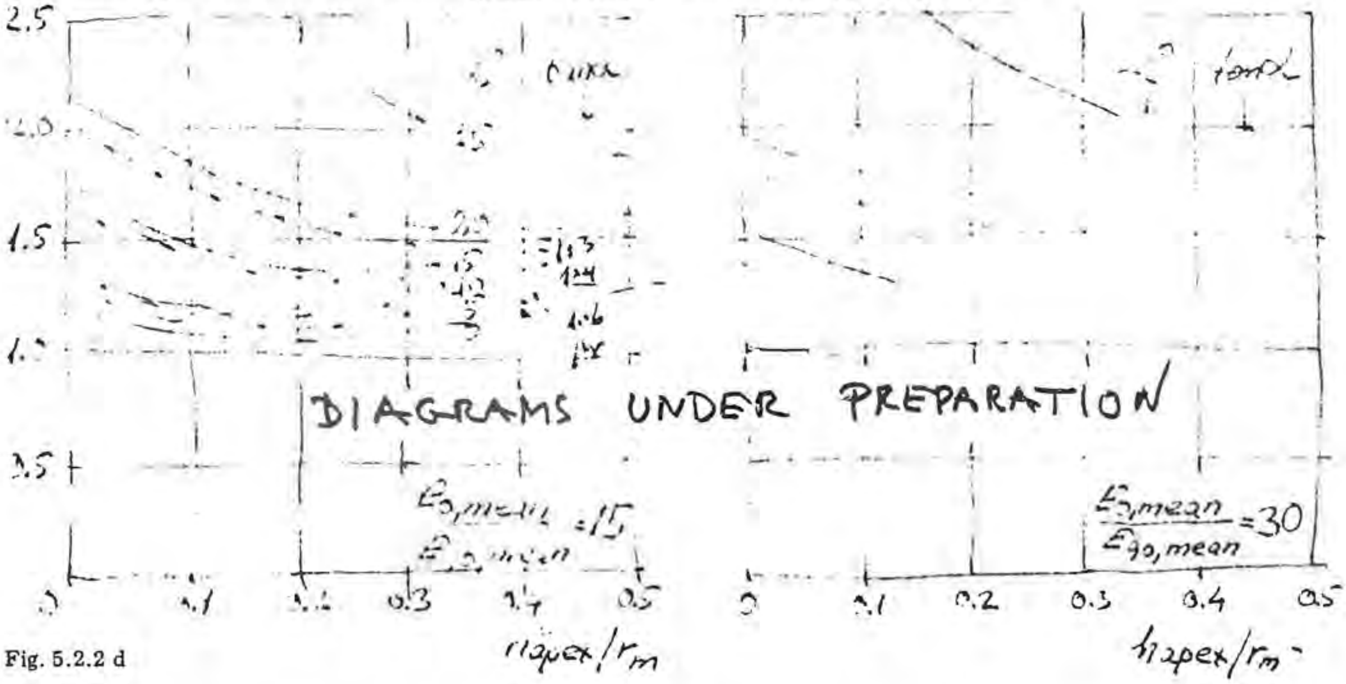


Fig. 5.2.2 d

: The bending stresses between the supports and the points of tangency are calculated as usual.

In deflection calculations contributions from shear force deformations shall be taken into account.

5.2.3 Curved beams

This section applies to curves beams with constant, rectangular cross-section, see fig. 5.2.3 a.

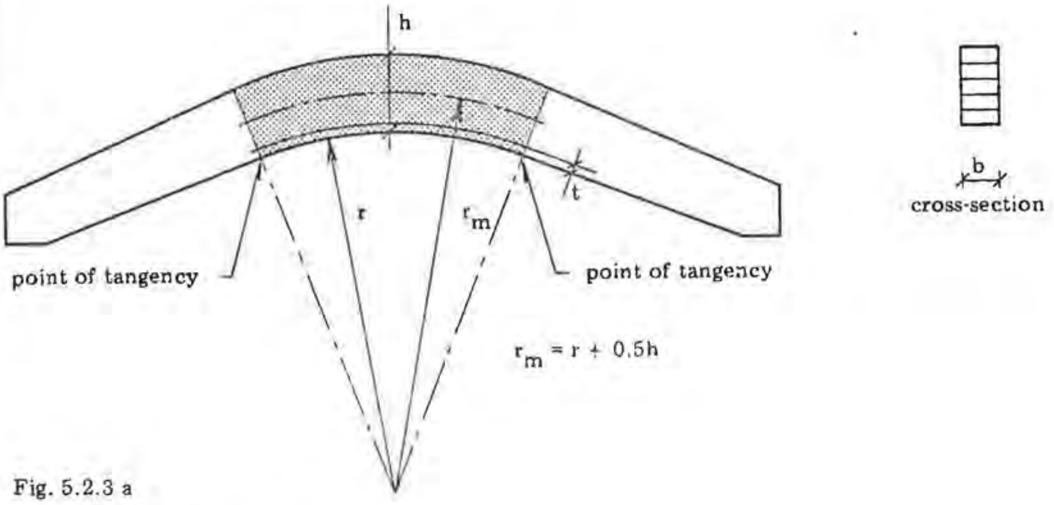


Fig. 5.2.3 a

Reduction of strength

The ratio between the radius of curvature, r, and the laminae thickness, t, should be greater than 125. For $r/t < 240$ the reduction of the strength in bending, tension and compression parallel to the grain due to the bending of the laminae should be taken into account.

: This can be done by multiplying f_m, $f_{c,0}$ and $f_{t,0}$ by the factor k_{curv}, where

$$k_{curv} = 0.76 + 0.001 \frac{r}{t} \qquad (5.2.3\ a)$$

Distribution of bending stresses

In heavily curved beams (i.e. the ratio between minimum mean-radius of curvature, r_m, and depth, h, less than 15) the influence of the curvature on the distribution of axial stresses from bending moments shall be taken into consideration.

The bending stresses in the innermost fibre can be calculated as

$$\sigma_{mi} = k_i \frac{6M}{bh^2} \qquad (5.2.3\ b)$$

while the stresses in the outermost fibre can be calculated by the usual expression

$$\sigma_{mo} = \frac{6M}{bh^2} \qquad (5.2.3\ c)$$

The modification factor k_i is given in fig. 5.2.3 b.

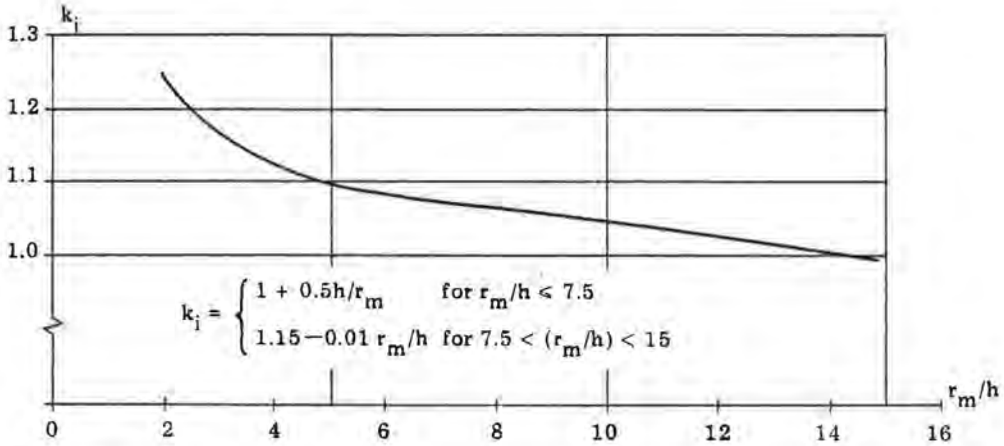


Fig. 5.2.3 b

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

$$\sigma_t \leq k_{size,90} f_{t,90} \qquad (5.2.3\ d)$$

where

$$k_{size,90} = \begin{cases} \dfrac{0.4}{V^{0.2}} & \text{for uniformly distributed load} \\ \dfrac{0.3}{V^{0.2}} & \text{for other loading} \end{cases} \qquad (5.2.3\ e)$$

V is the volume of the curved part of the beam (corresponding to the shaded area in fig. 5.2.3 a).

The tensile stresses perpendicular to the grain in the curved part may be calculated as

$$\sigma_t = \frac{1.5\,M}{r_m bh} \qquad (5.2.3\ f)$$

6. MECHANICAL FASTENERS

6.0 General

6.0.1 General requirements

When the joint is non-symmetric or where the load is eccentric consideration should be given to these factors by the determination of the load-carrying capacities of the fasteners as well as by the design of the members.

In a joint where several identical fasteners are used symmetrically it can be assumed that each fastener is loaded equally, except where a large number of fasteners are used. However, the load-carrying capacity of a multiple-fastener joint will be less than the sum of the individual fastener capacities.

The entire load on a joint should normally be carried by one type of fastener. In some cases, however, two types of fastener may be used provided they have similar stiffness characteristics.

: Glud and mechanical fasteners have very different stiffness properties and thus they can never be assumed to act
: in unison.

The arrangement of timber joints and the size of the fasteners, mutual distances and distance to end or edge of the timber should be chosen so that the expected strengths can be obtained without splitting or damaging the timber.

6.0.2 Determination of characteristic load-carrying capacities

The characteristic load-carrying capacities are determined from tests carried out in conformity with ISO/TC 165: Timber structures - Joints - Determination of strength and deformation characteristics of mechanical fasteners.

For a number of fasteners characteristic load-carrying capacities are given in section 6.1 - 6.4.

Where nothing else is stated the load-carrying capacities for the load duration classes defined in sections 2.2 and 2.3 are found by multiplication by the factors given in table 6.0.2.

Table 6.0.2 Factors for load-duration and climate

materials to be jointed	load-duration class	climate class 0 and 1	2	3
wood,	permanent and normal	0.6	0.6	0.5
plywood or	short-term	0.7	0.7	0.6
steel to wood	very short-term	0.85	0.85	0.7
	instantaneous	1.0	1.0	0.85
particle board	permanent and normal	0.4	0.3	-
or fibre board	short-term	0.6	0.45	-
to wood	very short-term	0.8	0.6	-
	instantaneous	1.0	0.75	-

The stated load-carrying capacities apply to static loads and may be less for some cases of fluctuating or dynamic loads especially when the stresses alternate between compression and tension.

: Attension is drawn to the fact that certain fasteners, e.g. nails, bolts without connectors and bolts with split ring
: or shear-plate connectors, have only inferior strength and will reveal great slip when exposed to heavy stresses with
: frequently alternating directions or vibrating load.

6.1 Joints with mechanical fasteners

6.1.1 Nails and staples

6.1.1.1 Laterally loaded nails

Timber-to-timber joints

In joints where the timber dimensions, the mutual distances between nails, and the distances between nails and end or edge are sufficient to prevent splitting, the characteristic load-carrying capacity per shear plane can be determined, in N, by:

$$F = k_{nail} d^{1.7} \qquad (6.1.1.1\ a)$$

where d (in mm) is the diameter for round nails and the side length for square nails. The factor, k_{nail}, which is dependent on among other things nail type and yield moment of the nails, wood species and grade (especially the density) must be determined by tests.

A joint should contain at least 2 nails. If there are only one or two nails the values according to formula (6.1.1.1 a) are multiplied by 0.5.

Nails in end grain should normally not be considered capable of transmitting force.

For round nails with a characteristic tensile strength of at least 40(20 — d)MPa the following equation can be used for Nordic softwood and other woods with corresponding properties

$$k_{nail} = 200\sqrt{\rho} \qquad (6.1.1.1\ b)$$

where ρ is the specific gravity defined in section 2.1.

For structural timber at least corresponding to SC19 $\rho = 0.36$ and thus $k_{nail} = 120$ can be assumed.

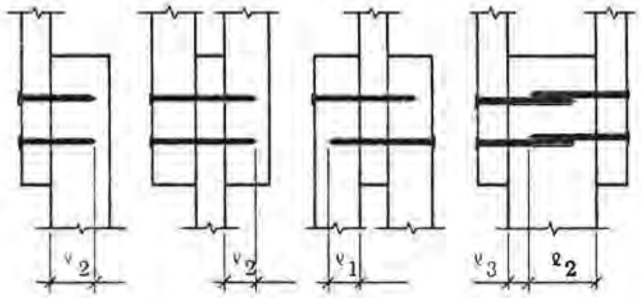


Fig. 6.1.1.1 a

The values assume that the nails are driven in perpendicular to the grain, that the thinnest member has a thickness of not less than 7d, and that the penetration depths (including the point) satisfy the following conditions (cf. fig. 6.1.1.1 a):

Nails in double shear	
(driven in alternating from either side)	$\ell_1 > 8d$
Other cases	
smooth nails	$\ell_2 > 12d$
annular and spirally grooved nails	$\ell_2 > 8d$

For smaller thicknesses and lengths the load-carrying capacity is reduced in proportion to the length. For smooth nails it is required that the nail length in any timber member is at least 5d and that the penetration length ℓ_2, is at least 8d. For annular grooved nails the penetration length should at least be 4d.

If ℓ_3 is greater than 3d (cf. fig. 6.1.1.1 a) nails from the two sides are allowed to overlap in the middle member.

Minimum distances are given in fig. 6.1.1.1 b. The nails should be staggered in the best possible way, for example as shown in fig. 6.1.1.1 b, one nail thickness in relation to the system lines.

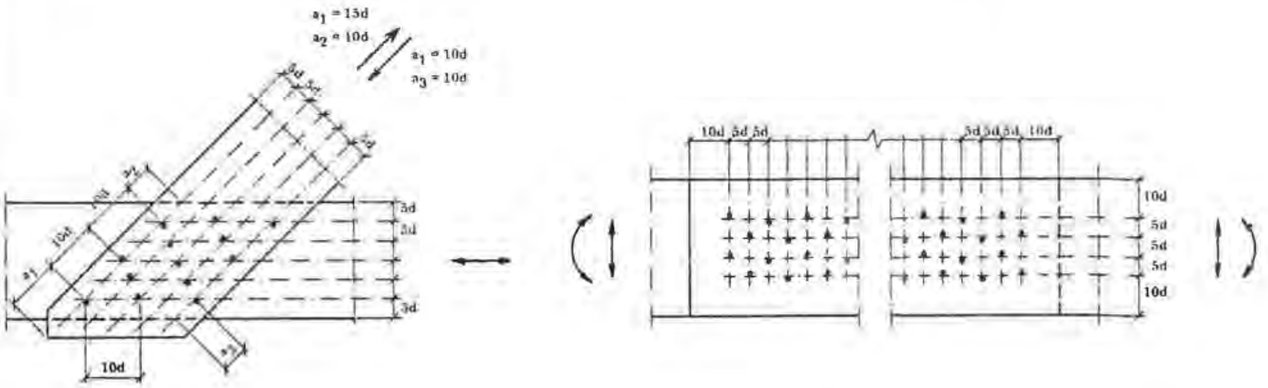


Figure 6.1.1.1 b

Steel-to-timber joints
The recommendations for timber apply, but the load-carrying capacities may be multiplied by 1.25.

Board materials-to-timber joints
What is stated for timber applies, but a board with thickness t can be assumed to correspond to a softwood timber member of quality SC19 with the thickness -

- 2.5 t for plywood of birch, beech, and similar hardwood
- 1.5 t for plywood of fir, pine, and similar softwood
- 2.0 t for plywood with plies of alternating hardwood and fir or pine (combi-plywood)
- 1.0 t for structural particle board and semihard structural fibre board
- 3.0 t for hard or oil-tempered structural fibre board.

This assumes the use of ordinary nails with heads which have a diameter of about 2.5 d.

For smaller heads the load-carrying capacity is reduced. For pins and oval headed nails, for example, the load-carrying capacity in particle boards and fibre boards is reduced by half.

6.1.1.2 Axially loaded nails
The characteristic withdrawal resistance of nails in N *for all climate classes* for nailing perpendicular to the grain and for slant nailing as shown in fig. 6.1.1.2 a - b is calculated as the smallest of the values according to formula (6.1.1.2 a) corresponding to withdrawal of the nail in the member receiving the point, and formula (6.1.1.2 b - c) corresponding to the head being pulled through.

The length of the point is denoted ℓ_p.

$$F = \min \begin{cases} f_{axial} d(\ell - \ell_p) & & (6.1.1.2\ a) \\ f_{axial} dh + f_{head} d^2 & \text{for smooth nails} & (6.1.1.2\ b) \\ f_{head} d^2 & \text{for annular and spirally grooved nails} & (6.1.1.2\ c) \end{cases}$$

: The parameters f_{axial} and f_{head}, depend, among other things, on type of nail, timber species and grade (especial-
: ly) density, must be determined by tests.

For spirally or annular grooved nails only the grooved part is considered capable of transmitting force.

By slant nailing ℓ and h are measured as shown in fig. 6.1.1.2 b and the load-carrying capacity is calculated as if the force were parallel to the nail. Unless otherwise ensured, e.g. by pre-boring, $\alpha = 45°$ is assumed.

Nails in end grain should not be considered capable of transmitting force.

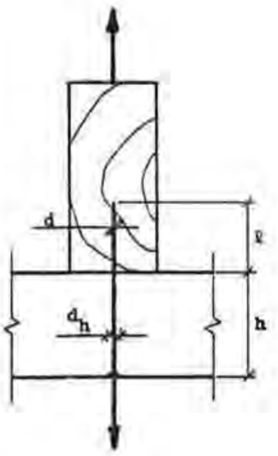


Fig. 6.1.1.2 a

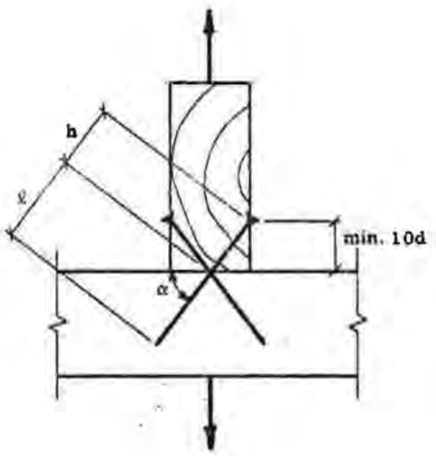


Fig. 6.1.1.2 b

: **The distances for laterally loaded nails should be complied with and the distance to loaded edge by slant nailing should be at least 10d, cf. fig. 6.1.1.2 b.**

: **Normally the values of f given in table 6.1.1.2 can be assumed. For structural timber at least corresponding to SC19 a characteristic density of $\rho \sim 0.36$ is assumed.**

: Table 6.1.1.2

	f_{axial} in MPa		f_{head} in MPa	
	general	SC19	general	SC19
ordinary nails, round	$12.5\,\rho^2$	1.6	$45.0\,\rho^2$	58
ordinary nails, square	$15\ \ \rho^2$	1.9		
spirally grooved nails 1)	---	to be determined by tests		---
annular grooved nails 1)	---	to be determined by tests		---

6.1.1.3 Staples

: Recommendations for staples will be provided.

6.1.2 Bolts and dowels

The characteristic load-carrying capacity in N per shear plane for bolts and dowels with a yield strength f_y of at least 240 MPa (corresponding to ISO grade 4.6) is the smallest value found by the formulas (6.1.2 a) - (6.1.2 e).

$$F = \min \begin{cases} 18\rho(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\ a) \\ 35\rho\, k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ b) \\ 70\rho\, k_1 t_1 d & (6.1.2\ c) \\ 42\sqrt{\rho}\, d^2 + 12\rho\, k_1 t_1 d & (6.1.2\ d) \\ 75d^2 \sqrt{\rho}\, \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.2\ e) \end{cases}$$

where

- t is timber thickness in mm
- d is the diameter in mm
- k is a factor, obtained from table 6.1.2, taking into consideration the influence of the angle between force and grain direction.

In three-member joints subscript 1 denotes side member and subscript 2 denotes middle member. In two-member joints the subscripts are chosen so that $k_1 h_1 \leqslant k_2 h_2$.

Table 6.1.2 Factor $k(k_1, k_2)$ in calculation of the load-carrying capacity of bolts, dowels and screws

Angle between force and grain direction	Diameter d(mm)		
	6	12	24
0°	1	1	1
30°	1	0.88	0.82
45°	1	0.76	0.70
60°	1	0.70	0.58
90°	1	0.64	0.52

: For structural timber at least corresponding to SC19 (i.e. ρ = 0.36) the following is found by inserting into (6.1.2 a) - (6.1.2 e):

$$F = \min \begin{cases} 6.5(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\ f) \\ 12.5 k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ g) \\ 25 k_1 t_1 d & (6.1.2\ h) \\ 25d^2 + 4.5 k_1 t_1 d & (6.1.2\ i) \\ 45d^2 \sqrt{(k_1 + k_2)/2}\ \sqrt{f_y/240} & (6.1.2\ j) \end{cases}$$

Minimum distances are given in fig. 6.1.2. (The distance from bolt or dowel to loaded end can be reduced to a minimum of 4d provided the load is reduced correspondingly). If the load-carrying capacity is assumed to be higher than corresponding to formula (6.1.2 e) with f_y = 240 MPa the distance in the grain direction should be increased correspondingly.

The stated distances to loaded edge are not always adequate to prevent splitting when the bolts are fully loaded. When the force acts at an angle to the grain it should therefore be shown that the force can be sustained without splitting.

: Where a detailed analysis is not carried out, this can be verified by showing that $V < 2f_v b_e t/3$, where V is the shear force produced by the bolt or dowel, t is the thickness in mm of the member, and b_e is the distance in mm from loaded edge to the farthest point of the bolt.

In multiple shear the load-carrying capacity can be found by considering the structure as a number of three-member joints.

Where the side members are steel plates the loads calculated from the above formulas may be used with t_1 equal to t_2 equal to the thickness of the wood member, and multiplied by 1.25.

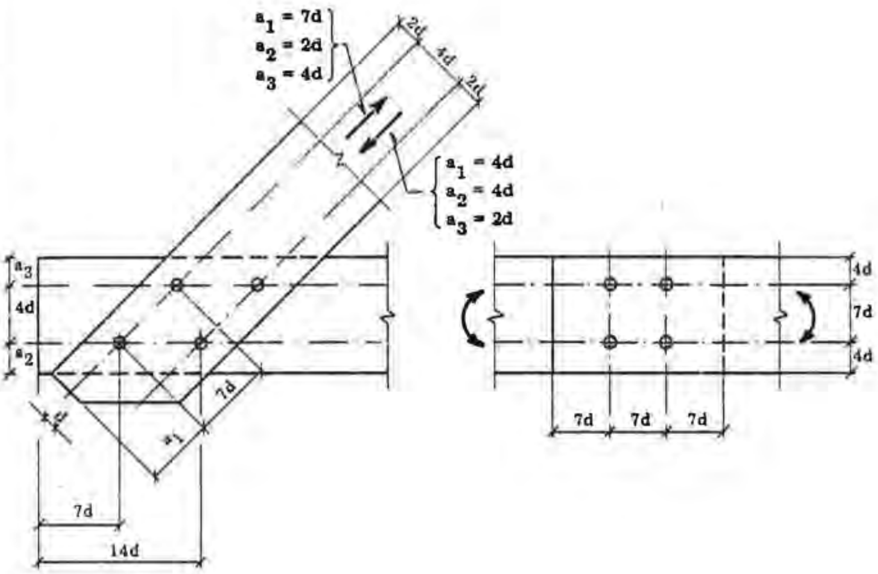


Fig. 6.1.2

Where the middle member is a steel plate formula (6.1.2 b) is omitted and the values of the formulas (6.1.2 d) and (6.1.2 e) can be multiplied by 1.4.

6.1.3 Wood and lag screws

6.1.3.1 Laterally loaded screws

Timber to timber

The characteristic load-carrying capacity in N of screws with a yield strength f_y of at least 240 MPa screwed at right angles to the grain is the smallest of the values from the formulas (6.1.3.1 a) - (6.1.3.1 c)

$$F = \min \begin{cases} 70\rho k_1 td & (6.1.3.1\ a) \\ 42\sqrt{\rho}\, d^2 + 12\rho k_1 hd & (6.1.3.1\ b) \\ 75d^2 \sqrt{\rho}\, \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.3.1\ c) \end{cases}$$

where

- t is the thickness in mm of the timber,
- d is the diameter in mm of the screw, measured on the smooth shank,
- k_1,k_2 are factors, obtained from table 6.1.2, taking into consideration the influence of the angle between force and grain direction in the member under the screw head (k_1) and the member receiving the point (k_2).

: For structural timber at least corresponding to SC19

$$F = \min \begin{cases} 25k_1 td & (6.1.3.1\ d) \\ 25d^2 + 4.5k_1 td & (6.1.3.1\ e) \\ 45d^2 \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.3.1\ f) \end{cases}$$

: is found by inserting $\rho = 0.36$ into (6.1.3.1 a) - (6.1.3.1 c).

In these expressions it is assumed that:

- screws are screwed into pre-bored holes, see section 8.3.
- the minimum distances between screws and between screws and end or edge do not exceed those given for bolts (refer to section 6.1.2),

- the length of the smooth shank is greater than or equal to the thickness of the member under the screw head,
- the penetration depth of the screw, i.e. the length in the member receiving the point, is at least 8d,
- the recommendations of section 8.3 are complied with for lag screw holes.

If the penetration depth is less than 8d the load-carrying capacity is reduced proportionally. However, the penetration depth should be at least 5d.

Screws in end grain should not normally be considered capable of transmitting force.

Steel to timber

The characteristic load-carrying capacity in N is (cf. formula (6.1.3.1 c))

$$1.4 \cdot 75d^2 \sqrt{\rho} \sqrt{(1 + k_2)/2} \sqrt{f_y/240} \qquad (6.1.3.1\ g)$$

and furthermore, what is stated for timber-to-timber joints applies.

6.1.3.2 Withdrawal loads of screws

The characteristic withdrawal strength in N of screws screwed at right angles to the grain is

$$F = (f_0 + fd)(\ell_t - d) \qquad (6.1.3.2\ a)$$

where

- d is the diameter in mm measured on the smooth shank,
- ℓ_t is the threaded length in mm in the member receiving the screw,
- f_0 and f are parameters dependent on among other things the shape of the screw and timber species and grade.

: For screws according to ISO 0000 the following can be assumed for structural timber at least corresponding to SC19

$$F = (30 + 12.5)(\ell_t - d) \qquad (6.1.3.2\ b)$$

It is assumed that the minimum distances and penetration lengths given for laterally loaded screws are complied with and that the strength of the screw is adequate.

6.1.4 Connectors

The characteristic load-carrying capacity $F_{bolt+conn}$ of a fastener comprising bolt (or screw) and connector may be determined as stated in section 6.0.2. The contribution from the bolt (or screw) may be calculated as stated in 6.1. The characteristic load for the connector F_{conn} is then determined from:

$$F_{conn} = F_{bolt+conn} - F_{bolt} \qquad (6.1.4)$$

If a connector is to be used together with several bolt diameters the investigation should comprise at least maximum and minimum bolt diameter.

: For characteristic load-carrying capacities of different types of connector, type approvals, etc. are referred to.

The rules given in section 6.1.2 for bolts should be complied with and the minimum distances between connectors should be sufficient to prevent splitting or shearing of the timber under the maximum permissible load.

When a load is applied at an angle to the grain direction it should be shown that the load can be sustained without causing splitting or shearing of the timber.

: Where a detailed analysis is not carried out this can be proved by showing that $V \leq 2f_v t b_e/3$, where t is the thickness of the member and b_e is the distance from loaded edge to farthest edge of the connectors, cf. fig. 6.1.4.

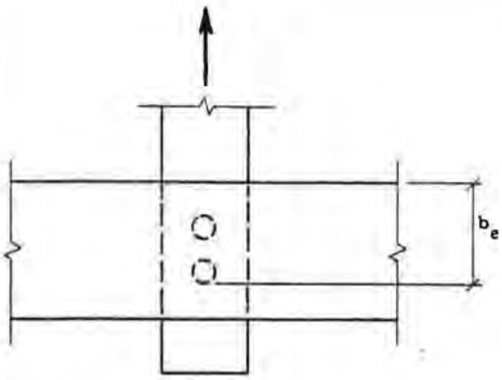


Fig. 6.1.4

6.1.5 Nail plates

: Recommendations for nail plates will be provided.

6.2 Glued joints

For continuous glued joints connecting unjointed laminae (e.g. between laminae, and between flanges and webs in beams or columns) the glued joint may be assumed to have the same strength as the weakest of the jointed materials for the action in question.

For other glued joints consideration should be given to the reduction in strength caused by an uneven distribution of the stresses over the glued area, including concentration of stresses at edges etc.

: For lap joints or gusset joints a characteristic shear strength of $(1.5 - 0.75 \sin\alpha)$MPa, where α is the angle between
: the force and grain direction, may be assumed for structural timber at least corresponding to SC19. The force per
: section, however, should not be assumed greater than $(75 - 37.5 \sin\alpha)$kN corresponding to an area of $0.05\ m^2$.

7. DESIGN OF COMPONENTS AND SPECIAL STRUCTURES

7.1 Glued components

7.1.1 Thin-webbed beams

The stresses in thin-webbed beams may be calculated under the assumption of a linear variation of strain over the depth. In principle the stresses must satisfy the conditions given in section 5.

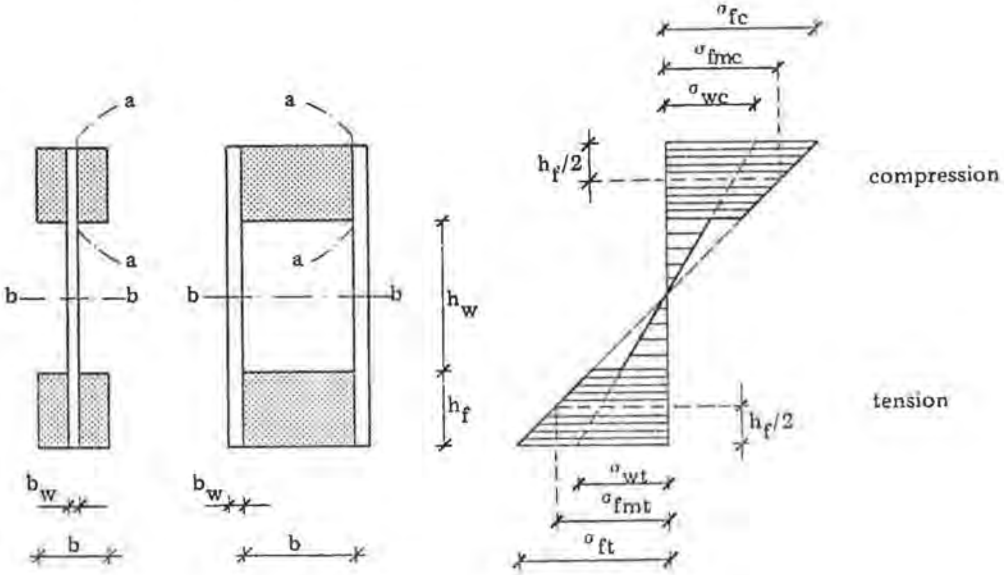


Fig. 7.1.1 a

For a beam with flanges of timber or glulam the stresses in the flanges should satisfy the following conditions, cf. fig. 7.1.1 a:

$$|\sigma_{fc}| \leqslant f_m \qquad (7.1.1\ a)$$

$$|\sigma_{fmc}| \leqslant k_{col} f_{c,0} \qquad (7.1.1\ b)$$

$$\sigma_{fmt} \leqslant f_{t,0} \qquad (7.1.1\ c)$$

$$\sigma_{ft} \leqslant f_m \qquad (7.1.1\ d)$$

k_{col} is determined according to section (5.1.1.9) with $\lambda = \sqrt{12}\, \ell_c/b$, where ℓ_c is the distance between the sections where lateral deflection of the compression flange is prevented, and b is given in fig. 7.1.1 a. If a special investigation into lateral instability of the beam is made as a whole $k_{col} = 1$ may be assumed.
For box beams an investigation of the lateral instability may be omitted if (7.1.1 a) is replaced by

$$|\sigma_{fc}| \leqslant k_{inst} f_m \qquad (7.1.1\ e)$$

where k_{inst} is determined according to section 5.1.1.3. This is on the safe side.
The web stresses σ_{wc} and σ_{wt} should be limited according to the materials used.

: If the web is made of plywood or other sheet materials the following conditions should be satisfied:

$$|\sigma_{wc}| < f_{wc} \qquad (7.1.1\ f)$$

$$\sigma_{wt} < f_{wt} \qquad (7.1.1\ g)$$

: where f_{wc} is the compression strength and f_{wt} the tensile strength.

The shear stresses may be assumed uniformly distributed over the width of the sections a-a and b-b shown in fig. 7.1.1 a.

It must be shown that the webs do not buckle.

If the webs are made from structural plywood, structural particle board or fibre board and the free depth, h_w, of the webs is less than $2h_{max}$, where h_{max} is given in table 7.1.1 and the shear force V satisfies the following conditions:

$$V \leqslant \begin{cases} f_v b_w (h_w + h_f) & \text{for} \quad h_w \leqslant h_{max} \\ f_v b_w h_{max} (1 + \frac{h_f}{h_w}) & \text{for} \quad h_{max} \leqslant h_w \leqslant 2h_{max} \end{cases} \qquad (7.1.1\ h)$$

a buckling investigation is not necessary.

It is assumed that the web is stiffened at the supports and under concentrated loads. The stiffeners should be fastened to the web and tightly fit between the top and bottom flanges. The cross-section of the stiffeners are chosen so that the whole force can be transferred between flange and stiffener.

Table 7.1.1.2

Web	h_{max}
Plywood with $\varphi < 0.5$	$(20 + 50\,\varphi)b_w$
Plywood with $\varphi \geqslant 0.5$	$45\ b_w$
Particle board or fibre board with $\varphi \approx 0.5$	$35\ b_w$

φ is the ratio between the bending stiffness of a strip with the width 1 cut perpendicularly to the beam axis and the bending stiffness of a corresponding strip cut parallelly to the longitudinal direction of the beam

In cases where a special investigation must be carried out it can be done according to the linear elastic theory for perfect plates simply supported along flanges and web stiffeners.

: For the case shown in fig. 7.1.1.b these assumptions lead to the following condition:

$$\frac{\sigma}{\sigma_{crit}} + \left(\frac{\tau}{\tau_{crit}}\right)^2 < 1 \qquad (7.1.1\ i)$$

: where σ_{crit} is the critical stress if only the axial stresses were acting and τ_{crit} the critical stress if only the shear stresses were acting.

: σ_{crit} can be determined as

$$\sigma_{crit} = k_{buck} \frac{\pi^2 \sqrt{(EI)_x (EI)_y}}{ta^2} \qquad (7.1.1\ j)$$

: where k_{buck} for a number of cases is given in fig. 7.1.1 c, and τ_{crit} can be determined as

$$\tau_{crit} = k_{buck} \frac{\pi^2 \sqrt[4]{(EI)_x^3 (EI)_y}}{ta^2} \qquad (7.1.1\ k)$$

: where k_{buck} for pure shear is given in fig. 7.1.1d.

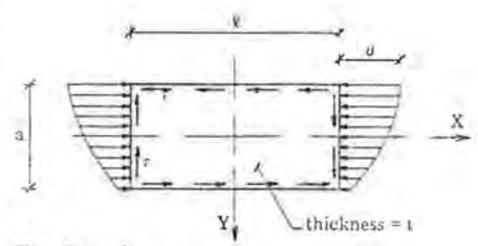


Fig. 7.1.1 b

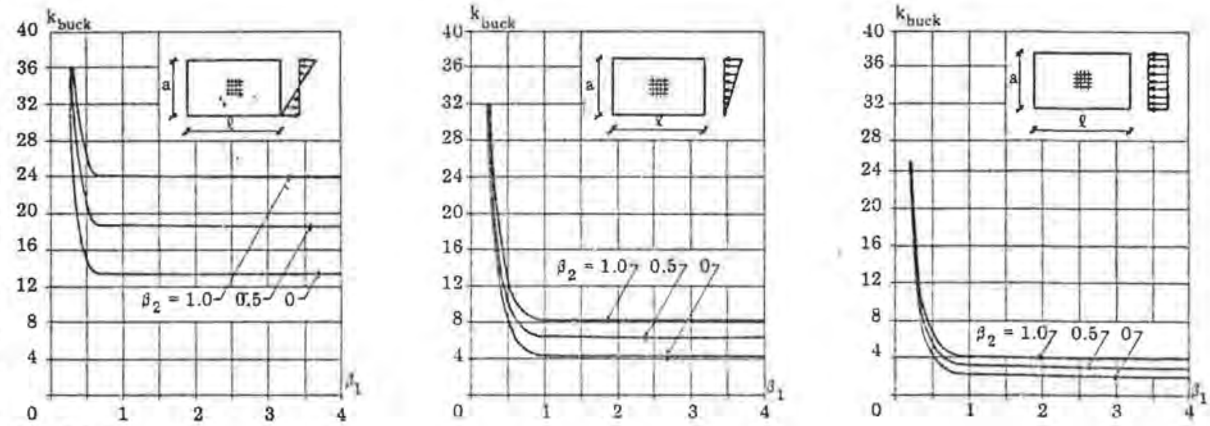


Fig. 7.1.1 c

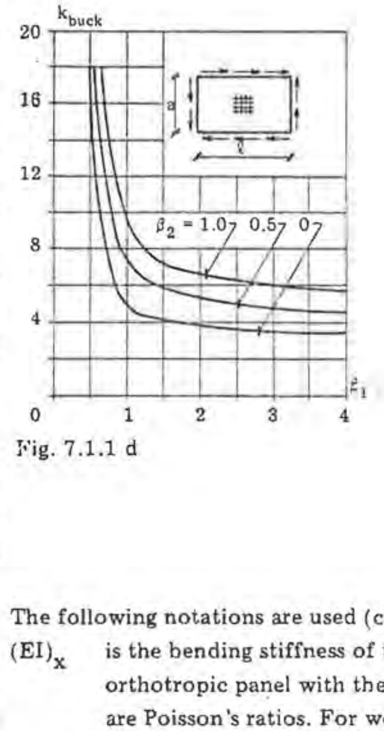


Fig. 7.1.1 d

The following notations are used (cf. fig. 7.1.1 b):

$(EI)_x$ is the bending stiffness of the panel per unit length in bending about the X-axis. For a homogeneous orthotropic panel with the main directions X and Y, $(EI)_x = \frac{1}{12} Et^3/(1 - \nu_{xy}\nu_{yx})$, where ν_{xy} and ν_{yx} are Poisson's ratios. For wood-based panels $\nu_{xy}\nu_{yx} \approx 0$ can be assumed.

$(EI)_y$ as $(EI)_x$, but in bending about the Y-axis.

$(GI)_v$ is the torsional stiffness per unit length of the panel. For a homogeneous orthotropic panel, $(GI)_v = Gt^3/3 + [\nu_{xy}(EI)_x + \nu_{yx}(EI)_y] \approx Gt^3/3$.

β_1 $= \frac{\ell}{a}\sqrt[4]{(EI)_x/(EI)_y}$. For an isotropic panel, $\beta_1 = \ell/a$.

β_2 $= 0.5\,(GI)_v/\sqrt{(EI)_x(EI)_y}$. For an isotropic panel, $\beta_2 = 2G/E$.

In calculations of deflection the contributions from the shearing stresses in the webs should be taken into account.

7.1.2 Thin-flanged beams (stiffened plates)

The stresses may be calculated under the assumption of a linear variation of strain over the depth and the stresses must in principle satisfy the conditions given in section 5.

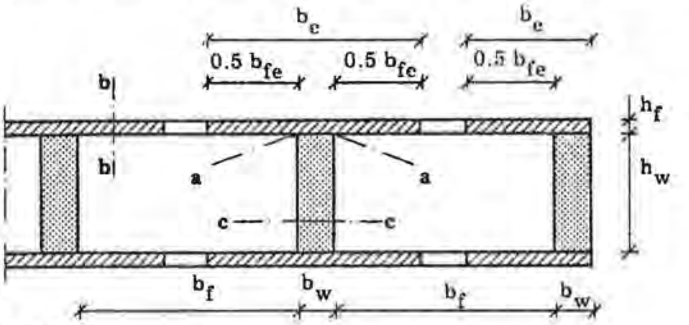


Fig. 7.1.2

The influence of the stresses not being uniformly distributed over the flange width should be taken into consideration. Unless otherwise proved the calculations can be based on an effective flange width, b_e, cf. fig. 7.1.2 , where

$$b_e = b_{fe} + b_w \tag{7.1.2 a}$$

or

$$b_e = 0.5\, b_{fe} + b_w \tag{7.1.2 b}$$

respectively.

The free effective width, b_{fe} ($\leq b_f$), is given in table 7.1.2.

Unless an investigation into the buckling instability of the compression flange is made, b_{fe} should not be put higher than b_{max}, also given in table 7.1.2.

Table 7.1.2

Flange	b_{fe}/ℓ	b_{max}
Plywood with fibre direction in extreme plies		
parallel to the web	0.1	$25\, h_f$
perpendicular to the web	0.1	$20\, h_f$
Particle board or fibre board w. random fibre orientation	0.2	$30\, h_f$

ℓ is the span, however, for continuous beams ℓ is the distance between the points with zero moment

The buckling investigation of the compression flange can be made according to section 7.1.1.

The buckling instability of webs made of plywood or other sheet materials should be investigated in accordance with section 7.1.1, unless $h_w \leq 0.5\, h_{max}$ where h_{max} is given in table 7.1.1.

The shear stresses may be assumed uniformly distributed over the width of the sections a-a, b-b and c-c shown in fig. 7.1.2.

7.1.3 I- and box columns, spaced columns, lattice columns

To I- and box columns the relevant parts of section 5.1.1.9, 7.1.1 and 7.1.2 apply.

What is stated for solid columns (section 5.1.1.7) applies to spaced columns and lattice columns, but furthermore, the deformation due to shear and bending in packs, battens, shafts and flanges and to the extension of the lattice should be taken into consideration.

: Design methods for spaced columns are given in appendix 7B and for lattice columns in appendix 7C.

7.2 Mechanically jointed components

If the cross-section of a structural member is composed of several parts connected by mechanical fasteners consideration must be given to the influence of the slip occurring in the fasteners.

In addition the recommendations of sections 5 and 7.1 apply.

Calculations may be carried out according to the theory of elasticity. For slip modulus the values given in table 7.2 may be used.

Table 7.2

Fastener	Slip modulus (N/mm)
Round nails with $d < 6$ mm☆	$0.017\ E_0 d$
Round nails with $d > 6$ mm☆	$0.1\ E_0$
Bolts with pressed-in connectors	$1.3\ E_0$

E_0 is the modulus of elasticity of the timber in N/mm^2. d is the diameter in mm for round nails or the side length for square nails.

☆ For square nails 15% higher values are allowed.

: For beams a design method for a number of cross-sections is given in appendix 7A and for columns in appendix
: 7A-B-C.

7.3 Trusses

Trusses may be analysed as frame structures where the influence of initial curvature of the elements, eccentricities, deformations of elements and slip and rotation in the joints are taken into consideration in the determination of the resultant stresses.

As an alternative a simplified calculation after the following guidelines is permitted: The axial forces are calculated assuming hinges in all nodal points, and the moments in continuous members, if any, are assumed to lie between 80% and 100% of the simple moments (corresponding to hinges in both ends) dependent upon the degree of end-fixing and the support conditions. For non-continuous members the moments are assumed equal to the simple moments. The free column length is assumed between 85% and 100% of the theoretical nodal point distance dependent upon continuity and degree of restraint.

Appendices 7A, 7B, 7C have been presented as CIB Timber Standards 02, 03, 04 and are included in document CIB-W18/6-100-2.

8. CONSTRUCTION

8.0 General

The recommendations given in this chapter are necessary conditions for the applicability of the design rules elsewhere in this code.

Timber structures shall be so constructed that they conform with the principles and practical considerations of the design.

Materials for the structures shall be applied, used or fixed so as to adequately perform the functions for which they are designed.

Workmanship in fabrication, preparation and installation of materials shall conform in all respects to accepted good practice.

8.1 Materials

Timber and wood-based components of structural elements should not be unnecessarily exposed to climatic conditions more severe than those to be encountered in the finished structure. In particular they should not be subject to prolonged exposure to the weather or to conditions conducive to fungal or insect attack.

Timber which is damaged locally, crushed or otherwise misused should not be used for structural work.

Before construction timber should be seasoned as near as practicable to the moisture content appropriate to its climatic condition in the completed structure.

The limitations on bow in most national stress grading rules are inadequate for the selection of material for columns and beams where lateral instability may occur. Particular attention should therefore be paid to the straightness of columns; e.g. limiting bow to about 1/300 of the length. It may also be necessary to introduce more stringent limits on other particular members, e.g. twist for torsional members.

8.2 Machining

The size, shape and finish of all timber and other materials shall conform with the detailed design drawings and specifications for the structure.

The cutting of timber after preservative treatment should be avoided. However, when it is unavoidable, and exposure of untreated timber results, a liberal application of preservative should be made to the cut surfaces.

8.3 Joints

Fasteners should be placed in conformity with the drawings. The minimum distances given in section 6.1.1 - 6.1.4 should be complied with.

Wane, splits, knots or other defects are not allowed in joints to such a degree that the load-carrying capacity of the joints is reduced.

Unless otherwise specified nails should be driven in at right angles to the grain and to such depth that the surfaces of the nail heads are flush with the timber surface.

Unless otherwise stated slant nailing should be carried out in conformity with fig. 6.1.1.2 a.

Bolt holes may have a diameter not more than 2 mm larger than the bolt. Washers with a length and thickness of at least 3d and 0.3d, respectively (d is the bolt diameter), should be used under the head and nut.

Bolts should be tightened so that the members fit closely, and they should be tightened up if necessary when the timber has reached its equilibrium moisture content.

At least 2 dowels should be used in a joint. The minimum dowel diameter is 8 mm. Turned dowels should be used and the pre-bored holes in timber members should have a diameter which is 0.2 - 0.5 mm less than the dowel diameter while the pre-bored holes in steel plates should have the same diameter as the dowel. The dowels should be at least 2d longer than the total thickness of the joint.

Through the centre of each connector a bolt or screw for which the above rules are valid should be placed. Connectors should fit tightly in the grooves.

When using toothed plates the teeth should be completely pressed into the timber. In smaller and lighter structures the bolt may be used for impressing provided it has at least 16 mm diameter. The washer should then have at least the same side length as the connector and the thickness should at least be 0.1 times the side length. It should be carefully checked that the bolt has not been damaged in tightening.

: Impressing should normally be carried out with special press tools or special clamping bolts with washers large and
: stiff enough to protect the timber from damage.

Lag coach screw holws shall be pre-drilled and treated as follows:

a. The lead hole for the shank shall have the same diameter as the shank and the same depth as the length of the unthreaded shank.

b. The lead hole for the threaded portion shall have a diameter determined by the characteristic density of the species or species group and by the length and diameter of the screw.
   : Recommendations on lead hole diameters will be provided.

c. Soap, or other non-corrosive lubricant (e.g. not ordinary petroleum) may be used to facilitate insertion of the screw.

d. Screws are to be inserted by turning with a suitable wrench, not by driving with a hammer.

8.4 Assembly

Assembly should be in such a way that unintentional stresses do not occur. Members which are warped, split or badly fitting at the joints should be replaced.

8.5 Transportation and erection

The over-stressing of members during storage, transportation and erection should be avoided. If the structure is acted upon or supported otherwise than in the finished building it must be proved that this is permissible and it must be taken into consideration that such action micht have a dynamic effect. In the case of e.g. framed arches, portal frames, etc., special care should be taken to avoid distortion in hoisting from the horizontal to the vertical position.

CIB-W18/10-105-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE WORK OF CIB-W18 TIMBER STRUCTURES

by

J G Sunley
Timber Research and Development Association
United Kingdom

MEETING TEN
VANCOUVER, CANADA
AUGUST 1978

THE WORK OF CIB (INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION) WORKING COMMISSION W.18 - TIMBER STRUCTURES

by

MR. J.G. SUNLEY, MSc.,
Director, Timber Research and Development Association, United Kingdom

CIB W.18 Timber Structures was formed early in 1959 as a Study Group on Timber Structures under the leadership of Mr. Levin of the Timber Research and Development Association, United Kingdom. Generally, the Commission operated as a small corresponding group and did not have any regular meetings. In 1961 and 1965, CIB, through W.18, supported two international conferences on timber engineering in the United Kingdom, the first at Southampton and the second in London. From 1969 until 1973 the Commission was inactive and was re-formed in 1973 with myself as Co-Ordinator.

CIB W.18 was re-formed in 1973 with the following terms of reference - "To study and highlight the major differences between the relevant National Design Codes and Standards and suggest ways in which the future development of these Codes and Standards might take place in order to minimise or eliminate these differences".

It was considered that with these terms of reference, CIB W.18 work would not overlap with that of the other organisations concerned with timber engineering research and design.

Some of the other organisations involved are:-

1. RILEM (Reunion International des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions), dealing with test methods.

2. IUFRO (International Union of Forest Research Organisations), Wood Engineering Group, dealing with timber engineering research.

3. Standards organisations dealing with the actual codification (ISO, CEN, and SMEA).

CIB W.18 is ideally constituted and has emerged at the right time as a leading forum in the world on timber structures to provide maximum assistance for harmonisation of codes and standards. This has been recognised by ISO, ECE and EEC, who have asked CIB W.18 to provide them with drafts and assist them with the harmonisation of building regulations and structural codes.

The work of CIB W.18 is implemented in three main ways:

1. As an independent group of timber engineering experts who publish their own recommendations through CIB so the rest of the world is aware of their views and of recommended ways of dealing with timber in structural codes.
2. By ensuring that the recommendations of CIB W.18 are made available to the appropriate organisations which wish to use them (e.g. ISO, ECE, EEC, etc). Also to ensure that the recommendations are correctly used, members are encouraged to take part in the activities of these organisations.
3. Individual members are able to report back to their national organisations and encourage the adoption of CIB agreements and recommendations.

PUBLICATIONS

The proceedings of the first nine meetings of CIB W.18 have been published and are available to other organisations. These cover:-

1. March 1973 - meeting in United Kingdom
2. October 1973 - meeting in Denmark
3. June 1974 - meeting in Holland
4. February 1975 - meeting in France
5. October 1975 - meeting in West Germany
6. June 1976 - meeting in Denmark
7. March 1977 - meeting in Sweden
8. October 1977 - meeting in Belgium
9. June 1978 - meeting in Scotland

A large number of documents have been presented at various meetings and a number of additional publications issued. All these are listed in Appendix I.

RELATIONS WITH OTHER ORGANISATIONS

(a) IUFRO Wood Engineering Group. The Wood Engineering Group of IUFRO is a forum for discussion of all aspects of research on the structural utilisation of timber and wood-based products. Therefore the work of this group is complement to the work of CIB W.18 which makes recommendations to encourage research on subjects on which information is lacking.

To enable the best possible liaison between the two groups there was a joint meeting in June 1976 in Denmark. Discussions at the IUFRO meeting in Denmark covered the following subjects:-

1. Research programme in different Institutes
2. Load and load sharing effects

Continued/

3. Floor design
4. Trussed rafter design
5. Design of bracing
6. Probabilistic design
7. Strength properties of materials
8. Effects of moisture and long term loading on strength
9. Modulus of elasticity and rigidity
10. Fracture mechanics, shear and load duration
11. Visual and mechanical stress grading
12. Non-destructive testing
13. Effectiveness of grading
14. Review of the use of particle and fibreboard for structural purposes

(b) RILEM. There was a small committee in RILEM which dealt with timber testing, called 3-TT, Chairman, Professor H.R.W. Kühne, Switzerland. This group had become largely inactive but has been reconstituted as a joint RILEM/CIB W.18 committee chaired by Dr. J. Kuipers, Netherlands, which meets at the same time as the CIB W.18 meetings and all the test proposals are discussed as a joint exercise. Subjects currently under discussion include testing methods for wood-based sheet materials, joints and structural timber.

(c) ISO (International Standards Organisation). Although there are numerous ISO committees with which it is desirable that the CIB W.18 should liaise, the main links are with the following:-

(i) ISO TC.55 which deals with solid timber and covers such items as terminology, defects, testing small clear specimens, timber sizes, grading, etc.

(ii) TC.98 'Bases for the design of structure'. This committee is concerned with the general principles of structural engineering design common to all materials and the work of W.18 will use the information generated by TC.98 and link with it.

(iii) TC.165. This is a relatively new ISO committee, the secretariat is held by Denmark. The committee has been formed to deal specifically with timber design codes and link with a number of ISO TC's covering different timber-based materials such as solid timber, plywood, particleboard, fibreboard, etc.

(d) Joint Committee on Structural Safety (JCSS). Initiative came from a CEB/CECM/FIP/CIB/IABS/Joint Committee on structural Safety with a strong lead from CEB (Comite Euro-International du Beton) trying to draft a series of codes covering all materials. Volume I will contain information general to all materials. Other volumes will deal with specific materials, e.g. Volume II concrete and Volume VI timber. A first draft of Volume I is now complete and is being discussed by various organisations and CIB W.18 has discussed this at its recent meetings. CIB W.18 has a member on the editing committee on Volume I and it has been agreed that CIB W.18 will supply Volume VI on timber to link with Volume I and the other material volumes. When Volume I has been agreed it is expected that it will be sent to ISO TC.98 for development as an ISO standard. Similarly, it is expected that Volume VI on timber will go to TC.165 to be developed as a timber code.

(e) EEC (European Economic Community) and ECE (Economic Commission for Europe) are involved in the harmonisation of structural codes mainly through their desire to obtain harmonisation of building regulations in various countries. Both these organisations have asked CIB W.18 to be their main source of information as far as the design of timber structures is concerned.

As will be seen from Appendix I, a wide range of subjects have been discussed within CIB W.18, some of which are:-

1. At the first meeting at Princes Risborough in 1973, a summary was given of existing national building regulations and design codes for timber structures in the various countries represented at this meeting. These covered Brazil, Denmark, France, Holland, Norway, Sweden, Switzerland, West Germany, USA, United Kingdom, with information being available from Australia and South Africa.

2. Stress grading. CIB W.18 has co-operated with ECE in Geneva in producing stress grading rules for European timber. In addition, it has recommended characteristic design stresses for the timber grades recommended by ECE.

3. Timber joints. Discussions have been held and agreement reached on methods of testing joints and on means of assessing nailplates for trussed rafters.

4. Probabilistic and limit state design. Proposed methods of code layout on limit state design principles have been considered and reference made to codes or draft codes in Sweden, Denmark, Norway, Canada and United Kingdom.

5. Layout of codes. Different methods of laying out design codes in a uniform way have been discussed but so far no agreement has been reached.

6. An international code for timber structures has been drafted.

Continued/

7. Timber columns. A method of designing solid timber columns has been agreed. Discussions are now taking place on the design of spaced columns.

8. Plywood. Discussions have taken place on methods of test and the way of presenting data on plywood in design codes. It is hoped to obtain full agreement on methods of test at the next meeting.

9. Symbols and notations. Agreement has been reached on symbols and notations to be used within CIB W.18 and these generally agree with ISO recommendations. They have been published as a CIB W.18 document.

10. Load sharing. Factors used in different countries have been considered. It is hoped to discuss theoretical methods of predicting such interaction at the next meeting.

11. Long term loading. Information available from North America has been studied and it has been agreed to accept current methods in use there. CIB W.18 has recommended that more research is needed in this area and is co-operating with IUFRO in developing the necessary programmes.

12. Design of timber beams. A paper has been presented and discussed on the theory of beam design.

13. Climatic groups for structural design. A report recommending specified climatic groups has been presented and discussed and agreement reached on three climatic classes.

14. Comparison of existing codes in EEC countries. A report has been presented on the subject and used as background information for the work of the CIB W.18.

15. Methods of testing structural sizes. Methods have been presented, discussed and agreed. This agreement permits the exchange of research information.

16. Glued laminated structures. Grading rules, design stresses and methods of design of glued structures have been discussed and suitable methods for some of these agreed.

17. A start has been made on a Code for trussed rafters.

APPENDIX 1

Current list of CIB-W18 Technical Papers

SECRETARY'S NOTE

To avoid unnecessary duplication the list of technical papers that formed Appendix 1 of this document has not been reproduced. An indexed list of papers may be found earlier in these proceedings.