INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB-W18

Meeting Thirteen

OTANIEMI, FINLAND

JUNE 1980

CONTENTS

1 LIST OF DELEGATES

CANADA

J D Barrett	Forintek Canada Corporation, Vancouver
T A Eldridge	T A Eldridge and Associates, Ontario
C K Stieda	Council of Forest Industries of British Columbia, Vancouver

DENMARK

A Egerup	Technical University of Denmark, Copenhagen
M Johansen	Danish Building Research Institute, Horsholm
H J Larsen	Aalborg University Centre, Aalborg
H Riberholt	Technical University of Denmark, Copenhagen

FINLAND

P Kallioniemi	Ministry of the Interior, Helsinki
J Kangans	Technical Research Centre of Finland, Otaniemi
E K Leppävuori	Technical Research Centre of Finland, Otaniemi
E Pennala	Helsinki University of Technology, Otaniemi
T Poutanen	Helsinki University of Technology, Otaniemi
U Saarelainen	Technical Research Centre of Finland, Otaniemi

FEDERAL REPUBLIC OF GERMANY

H Brüninghoff	Ingenieurbüro für Bauwesen Baustatistik, Ilm
J Ehlbeck	Universität Karlsruhe, Karlsruhe
K Möhler	Universität Karlsruhe, Karlsruhe

FRANCE

P Crubilé	Centre Technique du Bois, Paris
M Escudié-Calvignac	Centre Technique du Bois, Paris

NETHERLANDS

J Kuipers	Steven Laboratory, Delft

NORWAY

E Aasheim	Norsk Treteknisk Institutt, Oslo
N Bovim	Norsk Treteknisk Institutt, Oslo

POLAND

W Marosz	Union of Building Joinery, Warsaw
Z Mielczarek	Technical University of Czczecin, Czczecin
W Nozynski	Centralny Osrodek Badawczo, Laskowa

SWEDEN

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

SWITZERLAND

E Gehri	Eidgenössische Technische Hochschule, Zurich
U A Meierhofer	Federal Laboratory for Materials Testing, Dübendorf

UNION OF SOVIET SOCIALIST REPUBLICS

A Zamarev	Gosstroy USSR, Moscow

UNITED KINGDOM

L G Booth	Imperial College, London
H J Burgess	Timber Research and Development Association, High Wycombe
W W Chan	Consulting Engineer, Wembley
W T Curry	Building Research Establishment, Princes Risborough
(1) J G Sunley	Timber Research and Development Association, High Wycombe
(2) J R Tory	Building Research Establishment, Princes Risborough

UNITED STATES OF AMERICA

V Ellebracht	American Plywood Association, Tacoma
G Stern	Virginia State Polytechnic Institute, Blacksburg

INTERNATIONAL STANDARDS ORGANISATION

Mrs A Sørensen	Secretariat ISO/TC 165 Timber Structures, Copenhagen, Denmark

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

2 CHAIRMAN'S INTRODUCTION

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

MR SUNLEY told delegates that the United Kingdom Building Research Establishment had written to him to explain that after holding the secretariat for CIB-W18 for more than seven years they now wished to relinquish those duties within the next twelve months. Delegates were asked to consider what alternative secretarial arrangements could be made and were notified that in future some form of payment might be necessary for proceedings and papers for meetings.

3 CO-OPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: MRS SØRENSEN circulated paper CIB-W18/13-103-1 as a report of activity within ISO/TC 165. PROFESSOR LARSEN said that there had been no action by TC 139 on the plywood testing standard. No date has been fixed for the next meeting of TC 165.

RILEM: DR KUIPERS reported that the RILEM 3-TT/CIB-W18 group had now completed their work on the structural timber testing standard and the joints testing standard. Both had been submitted to ISO/TC 165. The first annex to the joint testing standard, on nail plates, had also been completed and annexes on nails and on staples were being prepared. A new RILEM committee TBS 57 has been formed with two sub-groups to deal with board testing and structural testing. Lee from the United Kingdom is to provide a draft standard for the testing of board materials which will follow closely the methods adopted for plywood. Two papers had been received on structural testing and these would enable a draft standard to be prepared.

CEI-Bois/FEMIB: MR SUNLEY told delegates that it was hoped that the final draft of the CIB Code would be completed at this meeting for presentation to ISO/TC 165. It was also hoped that the same document could form the basis for the EEC Eurocode 5 if agreement could be reached with CEI-Bois. He expected that it would be possible to produce Eurocode 5 from the CIB Code without major technical changes. If agreement could not be reached between W18 and CEI-Bois it would be a problem for the EEC on how to proceed with the production of their timber code.

PROFESSOR LARSEN said that it would be unacceptable for a small drafting group from CEI-Bois/CIB-W18 to make significant changes to the body of the CIB Code.

DR KUIPERS pointed out that although CEI-Bois had been invited to comment on the content of the CIB Code there had so far been no response from them.

4 TRUSSED RAFTER SUB-GROUP

DR EGERUP reported that the trussed rafter sub-group had had a useful meeting in their efforts to provide a section on truss design for the CIB Code. The Nordic countries tended to favour frame analysis as the basic design method but for some designs this could be grossly conservative. The sub-group had agreed that for about ninety per cent of cases the pin-joint method with moment co-efficients was most suitable. The frame analysis method was to be preferred only when all parameters could be adequately defined.

MR RIBERHOLT suggested the need for a comprehensive analysis of the large number of truss tests carried out in different countries to identify the significant parameters for frame analysis. The model, he said, became impracticable if attempts were made to include all effects.

PROFESSOR LARSEN agreed with Mr Riberholt that a complex analytical method calibrated against test results was required to produce the moment coefficients for the simpler pin-jointed model.

Papers CIB-W18/13-14-1 'Truss Design Method for CIB Timber Code' CIB-W18/13-14-2 'Trussed Rafters, Static Models' and CIB-W18/13-14-3 'Comparison of 3 Truss Models, Designed by Different Assumptions for Slip and E-modulus', were circulated from the sub-group. The minutes of the Trussed Rafter Group meeting are included in these proceedings (page 12).

5 PLYWOOD SUB-GROUP

DR NOREN and DR BOOTH reported that the plywood sub-group had begun to make progress with the difficult problems of sampling which they had been working on for more than two years. The sub-group had agreed that the Princes Risborough Laboratory and Dr Booth should investigate methods of predicting the properties of plywood from species and lay-up data and should calibrate their model against test results. Dr Barrett, Dr Stieda and Mr Warren are to continue to pursue the concepts of compliance testing set out in paper CIB-W18/13-17-1 'On Testing Whether a Prescribed Exclusion Limit is Attained'. The sub-group are of the opinion that if a satisfactory predicative method can be established for plywood it will considerably simplify the problem of adequate sampling.

6 STANDARDISATION WORK IN POLAND

DR MAROSZ briefly introduced paper CIB-W18/13-102-1 'Programme of Standardisation Work Involving Timber Structures and Wood-based Products in Poland'. He told the meeting that as the programme of work progressed they would report progress and submit draft standards to CIB-W18 for information.

7 STRENGTH CLASSES

Paper CIB-W18/13-6-1 'Strength Classes for the CIB Code' was introduced by MR TORY who explained the UK, Australian and CIB strength class systems, pointing out that all introduced some penalties. He suggested that there should be more classes to cover hardwoods and a wider range of softwoods and that the interval between classes should be smaller to reduce the possible penalties for those wishing to adopt a strength class system. He also explained probable difficulties with secondary properties and suggested that until more characteristic stress values were defined by national committees it was difficult to predict the probable success of any system.

PROFESSOR LARSEN accepted the need for more classes and was satisfied that a geometric series of bending strength with a common ratio of 1.12 either side of 15 N/mm^2 would provide a workable system. He also accepted the idea for 5 per cent relaxation for secondary properties as proposed for the UK BS 5268.

MR ELDRIDGE questioned the use of bending strength as the property for defining class intervals but it was generally agreed that it should be retained as the primary property.

DR NOREN did not agree with the introduction of a modified R20 series of classes. For exporting countries he saw considerable disadvantages in so many classes.

He was supported by MR SUNLEY who said that in his opinion the United Kingdom would probably accept a system with a common ratio of 1.26 centred on 15 N/mm^2.

PROFESSOR LARSEN agreed to draft a revised table of strength classes and to circulate it for comment. He agreed to reconsider the profile of properties for the classes taking into account a wide range of species and grades.

8 FIRE

MR TORY told delegates that he had drafted paper CIB-W18/13-100-2 'CIB Structural Timber Design Code: Chapter 9: Performance in Fire' as instructed at Bordeaux, from British Standard BS 5268:Part 4.1.

PROFESSOR MÖHLER said that DIN 4102 Part 4 gave particular member sizes for specific fire resistance. In Germany they would be unable to use the idea of charring rates.

Units of mm/min are to be included in Table 9.1. The title of Figure 9.2 is to be changed to: 'Sections and joints with metal fasteners'
Clause 9.2.4.3 is to be deleted.

PROFESSOR LARSEN is to redraft the paper taking account of the shorter wording in the Dutch code and condensing section 9.2.

9 JOINTS

DR MAROSZ introduced paper CIB-W18/13-7-1 'Polish Standard BN-80/7159-04; Parts 00-01-02-03-04-05'. He explained that it was a draft standard, circulated for comment. He drew attention to the adoption of the RILEM general test procedure.

DR KUIPERS said that this paper would be most useful in drafting further annexes to the RILEM joint testing standard.

DR MÖHLER expressed interest in equation 4/14/ asking for the basis for the fifty year strain. Dr Marosz was unable to provide an immediate answer.

Paper CIB-W18/13-7-2 'Investigation of the Effect of Number of Nails in a Joint on its Load Carrying Ability' was presented by PROFESSOR NOZINSKI who said that although triangular nails were used in some of the tests they were only manufactured in small quantities. In the paper the 'd' for the triangular nails was the 'height' and not the side length of the triangular section. Both circular and triangular nails were manufactured from similar steels.

PROFESSOR STERN said that the manufacture of triangular nails would be more difficult and expensive than circular nails and he did not consider the apparent 10 per cent improvement in performance significant. He also expected more problems with splitting.

PROFESSOR LARSEN suggested that triangular nails were theoretically superior and this paper tended to confirm that, particularly when load per unit area was considered. He asked Professor Nozinski to extend his work to joints with larger numbers of nails.

DR NOREN introduced paper CIB-W18/13-7-4 'Design of Joints with Nail Plates - Calculation of Slip', and paper CIB-W18/13-7-8 'Comments on Paper CIB-W18/12-7-3'. He explained that paper 12-7-3 that had been presented at Bordeaux had not been fully discussed and 13-7-4 was an extension of the design method outlined there.

It was pointed out by MR BOVIM that if this design method was adopted it would be unnecessary to carry out the full range of tests given in the RILEM annex on nail plates.

DR EGERUP said that these Nordic proposals for joint design were very much different and more complex than current European practices. He stressed the need for practical verification before the method was included in the CIB Code.

PROFESSOR LARSEN asked whether slip should be calculated about the centre of gravity or the centre of rotation of the plate; there could be considerable differences between the two answers. He also suggested that paper 12-7-3 should be rewritten as an annex to the CIB Code. DR NOREN agreed to do this.

Paper CIB-W18/13-7-5 'Design of Joints with Nail Plates - The Heel Joint' was introduced by MR KALLSNER.

MR JOHANSEN told delegates that within a few months he expected to publish a report which would relate his test results to the design method given in papers 12-7-3, 13-7-4 and 13-7-5.

DR KUIPERS briefly introduced paper CIB-W18 13-7-7 'Test on Bolted Joints' explaining that it was a recent translation of a report on tests carried out in 1962. He drew particular attention to the comparison of test results and theory (page 28 and graphs 4 and 5).

MR BURGESS said that the Timber Research and Development Association were conducting a survey of published literature on bolted joints and they expected to publish a report on this in the future.

MR BURGESS presented paper CIB-W18/13-7-6 'Nail Deflection Data for Design' pointing out that the variety of units used in the paper complied with the published papers referred to.

PROFESSOR LARSEN questioned the value of giving slip modulii in the CIB Codes since they were obviously so imprecise and so seldom used.

DR BOOTH, MR GEHRI, MR RIBERHOLT and MR MEIERHOFER were among the majority favouring some guidance in the Code on joint deflection.

Papers CIB-W18/13-7-3 'International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber' and CIB-W18/13-7-9 'Strength of Finger Joints' were circulated for information.

10 COMMENTS ON CIB STRUCTURAL TIMBER DESIGN CODE

The comments included in papers CIB-W18/13-100-3a, CIB-W18/13-100-3b, CIB-W18/13-100-3c, and CIB-W18/13-100-4 were considered, and the following changes to the CIB Structural Timber Design Code were agreed:

Contents: Some of the Polish proposals should be incorporated in annexes when the subjects have been more fully discussed.

Title of Chapter 1 to be changed to 'General Provisions'

Chapter 9 'Performance in Fire' is to be included.

Section 1.4: Other symbols are required for fire and trussed rafter design and for partial factors.

Section 1.5: No proposals have been received for this section.

Section 2.1: The tolerences on temperature and relative humidity are to conform with ISO 554 ie $20 \pm 2^{o}C$ 0.65 ± 0.05 RH. The term 'relative density' is to replace 'specific gravity'.

Section 2.2: The comments below the climate classes are to be deleted. The wording of the definitions of climate classes is to be reconsidered.

Tables 2.3a: and 2.3b: Replace 'permanent' by 'long'
Replace 'long-term' (introduced by CIB-W18/13-100-1) by 'medium'.

Chapter 3: Remove references to accidental loads.

Clause 4.2.0: Accept wording proposed in 13-100-3c.

Clause 4.3.1: Glulam strength classes are to be deleted from the Code.

Section 4.4: Reference to be changed to the RILEM document. Add a note that standards on the interpretation of the test data are in the course of preparation.

Formula 5.1.1.1c: Replace '$\geq$' by '$>$'

Below Figure 5.1.2: Replace '$\alpha \leq 20^{o}$' by '$\alpha \leq 15^{o}$'

Tables 5.1.0a: and 5.2.0 Delete '(for strength calculations)' and '(for deformation calculations)'

Section 5.2: MR GEHRI and PROFESSOR LARSEN are to redraft this section along the lines of the Swiss Code.

Formula 5.2.3e:	To be changed.
Section 6.0:	Reword text for combinations of different types of fastener.
Clause 6.1.1.1:	For more than 10 nails the effective number of nails, $n_{eff} = 10 + \frac{2}{3}(n-10)$.
Table 6.1.2:	Replace by MR BOVIM'S formula (13-100-3a). The wording above this table for more than four bolts is to conform to the wording of clause 6.1.1.1 for more than 10 nails.
Chapter 7, page 2:	Replace third line by: 'It must be shown that the webs do not buckle, or that the forces can otherwise be resisted by the diagonal tension strength of the web. Delete fibre board from line 4.
Table 7.1.1:	Delete third line.
Section 7.3:	DR EGERUP is to reword this section.
Annexes	DR BOOTH reported that he and DR FOSCHI were still working on an annex for the design of thin webbed beams but they lacked test results to verify their model for birch, birch-faced and combi plywoods. Dr Booth did not think it would be possible to draft a sufficiently general annex to cover all design cases but offered to draft a warning note to cover certain thin plate constructions. It was agreed that the wording for clause 6.1.5 Nail Plates should be more general than proposed by MR BOVIM in paper CIB-W18/13-100-4. The following changes to that paper were agreed:

6.1.5.1;	first paragraph:	Delete last sentence.
	second paragraph:	Delete second sentence. Change reference from 12-7-3 to an Annex in course of preparation.
6.1.5.2;	first paragraph:	Provide more general wording. Delete indented paragraphs.
6.1.5.2A:		Delete F_p and M_p
6.1.5.2B:		Delete F_p and M_{rc}

11 SHEAR STRENGTH OF BEAMS

DR BARRETT introduced papers CIB-W18/13-6-2 'Consideration of Size Effects in Longitudinal Shear Strength for Uncracked Beams' and CIB-W18/13-6-3 'Consideration of Shear Strength on End-cracked Beams'. He agreed that there were very few problems with the shear strength of beams but the methods adopted by some codes to assess shear strength were theoretically incorrect.
Dr Barrett said that test methods to evaluate K_{IIc} were required although it might be possible to relate this parameter more precisely to standard shear strengths than had been done in paper 13-6-3.

MR ELDRIDGE pointed out that adoption of these proposals would introduce complication to design procedures since superposition theory could not be applied.

12 OTHER BUSINESS

It was suggested that since manufacturing standards should not be incorporated as part of the CIB Structural Timber Design Code the publication of Glulam Standard Part 3:Glued Timber Structures:Performance (4th draft) CIB-W18/13-12-1, was the responsibility of the FEMIB/GLULAM organisations. The co-ordination between design and manufacturing standards is to be the subject of discussion between CIB-W18 and CEI-Bois.

Papers CIB-W18/13-14-3 'Comparison of 3 Truss Models Designed by Different Assumptions for Slip and E-modulus' and CIB-W18/13-15-1 'Timber and Wood-based Products Structures. Panels for Roof Coverings. Methods of Testing and Strength Assessment Criteria. Polish Standard BN-78/7159-03' were referred to the truss rafter sub-group and the W18/RILEM sub-group respectively.

MR SUNLEY closed the meeting and thanked the Finnish delegates and the State Institute for Technical Research for their hospitality and for the facilities that had been made available for the meeting. On behalf of all the delegates, Mr Sunley congratulated MR SAARELAINEN for the efficient organisation of accommodation, excursion and administration arrangements.

13 NEXT MEETING

The next meeting of CIB-W18 will take place during the week 11-15 May 1981 in Warsaw, Poland. The programme for the week will be:

Monday 11 May: RILEM 3-TT (members only; Chairman Dr Kuipers)

Tuesday 12 May: Plywood sub-group (Chairman, Dr Noren)
Truss rafter sub-group (Chairman, Dr Egerup)

Wednesday 13 May: CIB-W18
Thursday 14 May: CIB-W18
Friday 15 May: CIB-W18

Items for discussion will include:

1 Final draft of the CIB Code
2 Review of annexes required for the code, eg truss rafter design and design of nail-plate joints.
3 Code requirements for supporting standards on testing, sampling and analysis.

The meeting-after-next is tentatively scheduled for either Göteburg, Sweden or Oslo, Norway in May 1982.

CIB-W18 TRUSSED RAFTER GROUP
Meeting at Otaniemi, Finland, 3 June 1980

Present:	A. Egerup (Chairman)	Denmark	Technical University of Denmark
	E. Aasheim	Norway	Norsk Treteknisk Institutt
	J.D. Barrett	Canada	Forintek Canada Corp.
	H.J. Burgess	U.K.	Timber Research and Development Association
	W.W.L. Chan	U.K.	FIDOR
	P. Crubilé	France	Centre Technique du Bois
	W.T. Curry	U.K.	Princes Risborough Laboratory
	B. Edlund	Sweden	Chalmers University of Technology
	V. Ellebracht	Germany	American Plywood Association
	M. Escudié-Calvignac	France	Centre Technique du Bois
	B. Källsner	Sweden	Swedish Forest Product Research Laboratory
	J. Kangas	Finland	Technical Research Centre of Finland
	J. Kuipers	Netherlands	Technical University Delft
	W. Marosz	Poland	Union of Building Joinery Industry (Zjednoizenie Pnemystu Stolarki Budowlanej)
	K. Möhler	Germany	Universität Karlsruhe
	B. Norén	Sweden	Swedish Forest Products Research Laboratory
	W. Nożynski	Poland	Centralny Ośrodek Badawczo-Rozwojowy PSB
	T. Poutanen	Finland	Tampere University of Technology
	H. Riberholt	Denmark	Technical University of Denmark
	U. Saarelainen	Finland	Technical Research Centre of Finland
	J.R. Tory	U.K.	Princes Risborough Laboratory

COUNTRY POSITIONS

United Kingdom

Burgess said the U.K. trussed rafter code, Part 3 of CP112, had been revised as part of the new code BS 5268 which will replace CP112. However agreement had not been reached on the design method and it was proposed to engage a consultant to draw up recommendations for design. One system owner's computer program was known to be based on pin-jointed analysis with members subsequently treated as continuous at nodes, and it was understood that other systems followed similar methods. In the particular system known, no reduction was applied to the moment coefficients but an advantage was gained by taking a very low slenderness ratio at the intermediate node and using a combined stress ratio of 1.0 instead of 0.9. Curry added that the limiting spans currently used in the U.K. had been established by prototype testing.

Norway

Aasheim said there were five system owners in Norway, four using a common frame-analysis program and the fifth using his own program also based on frame analysis. There was no special trussed rafter code and design followed the code for timber structures in general. No big test programme had been conducted as in the U.K., so frame analysis had been chosen for design and this was based on a complex model.

Designs of special form were peformed 'manually' (not using a computer) but the use of computers was being extended to these, again using frame analysis. There had been no failures in service, and a recent increase in design snow load had been countered by a change in the timber design code.

Denmark

Riberholt said the six or seven systems in his country used pinned-joint analysis with adjusted moment coefficients but the method was not liked by some approving authorities. Special designs such as attic trusses were performed by frame analysis. The industry wanted a simple design method but this would sometimes be unrealistic. He felt the CIB-W18 group should put forward a simple method together with a more sophisticated one. The competition between different systems was partly based on differences in design methods.

Egerup added that short and long term tests had been made, and also tests on joints, and the influence of member stiffness variations had been investigated. The results of seven years' work had been collated in a draft report by T. Feldborg and M. Johansen, 'Wood Trussed Rafter Design', SBI report draft, May 1980.

Canada

Barrett reported that designs in Canada were derived from pin-point analysis and that span tables were available. Proposals had been made for cantilever trusses. A constant moment coefficient was applied rather than the TPI type varying with roof slope.

Finland

Poutanen said there were thirty fabricators in Finland. There was no strict standard for design and there were no span tables. Design was mainly on the pin-jointed assumption, with some frame analysis. For joint design, a code similar to the Swedish one was applied.

Producers were required to follow a strict quality control procedure which led to 50-100 tests being made each year. He thought there would be a move to frame analysis over the next two or three years. Joint design was a problem, calling for moment stiffnesses; some tests that had been done showed a large variation between plates.

Truss plates were not allowed in timber with a moisture content greater than 20 percent, but this requirement was concerned with anchorage strength and not corrosion. Research was in progress on the effect of moisture content on anchorage strength.

The timber used in trussed rafters was mostly T30 grade. Multiple plates were commonly used, resulting from the high snow load and a 1.2 m spacing.

Kangas reported that a load factor of 3 was generally achieved in tests, only 5 percent falling below 2.5.

Sweden

Norén reported that Swedish practice was similar to that in Denmark, using pin-joint analysis with adjusted moment coefficients and also frame analysis. Prototype testing was required for the validation of any special assumptions proposed for use in calculations, but it was too early to say if the procedure was confirmed by experience. There was a collaboration between the Nordic countries on design models and calculation methods. The approval procedure took account of the model chosen and if thought necessary a limited number of tests were made and the results interpolated. It had been found necessary to alter some earlier approvals after finding that tests were not reproducible because of sampling difficulties.

Källsner stated he found a tendency for a lower factor of safety with lower pitches, and there were some problems with heel joint failure. Egerup added that a greater combined stress was allowed for higher roof slopes.

Germany

Möller said the design rules in Germany were derived from tests and were printed in a document he produced at the meeting. The moisture content had to be not greater than 20-22 percent when the trusses were pressed. There were no fixed rules for design, which followed the same method as for nailed trusses, using pinned-joint analysis with the moment coefficients given by the TPI. A supervision scheme was applied, involving visits twice a year to the factories concerned. Double plates were sometimes used for large spans, and rules derived from tests were applied to the plate width in these cases.

In connection with the revision of DIN 1052 some recommendations for trussed rafters were being introduced but these would be no alteration to the design procedure unless this was influenced by CIB proposals.

Poland

Marosz said only about a thousand trussed rafters a year were used because Polish buildings were mostly built with flat concrete roofs. The design standard 'Static calculations and design' had been presented at the Stockholm meeting. Pinned-joint analysis was applied, followed by a form of limit state design.

France

Crubilé said the French code made no reference to trussed rafters but provisional texts had been issued as Cahier 90 of 1972 and Cahier 111 replacing it in 1978. Member forces were found from pinned-joint analysis and perimeter members were designed as continuous beams. Tests had been made to determine moment coefficients and work was in hand to improve these values. Cahier 111 provided for cantilever trusses a manner in which agreed with test results, and there was a demand for similar work on attic trusses.

It was proposed to introduce a prototype test method calling for a load factor of 2.5 for a single truss falling to 2.2 for three trusses, compared with a value of 3 expected from calculated designs.

Nine configurations had been tested to provide calibration on the conservative side for the design method. From the tests it appeared that the deformation limitation in the calculations gave satisfactory results in service.

DESIGN METHOD FOR CIB CODE

In presenting his paper 'Trussed rafters, static models', Riberholt said the Nordic countries aimed to establish common rules for design. He suggested the frame analysis method (method 2 of the paper) was applicable widely as the systems using it covered most of Europe. There was a need to discover how a simplified model might be used, for example in choosing an appropriate effective length for a column, to obtain results the same as given by a sophisticated model. The models in the paper were based on judgement and not on test. The two on the last page were for moment-stiff joints and gave similar results; the support position was vital. In summary, Riberholt felt that method 2 should be developed, while non-computer calculations could be made for most common truss types.

Egerup thought frame analysis should be applied carefully because it tended to give conservative results which could be regarded as a lower-bound solution. Chan said the distance between points of contraflexure was sensitive to the type of cladding. He felt it was difficult to be precise about distances in the last diagram of the paper because of the plate positioning tolerance. Poutanen described a truss type common in Finland which contained a large heel-joint eccentricity leading to a need for frame analysis to secure economy.

A long discussion ensued, and Riberholt finally asked how a simple method could be decided upon for the CIB code. Norén suggested starting with guidelines for method 2 together with the pinned-joint method. He said the type of calculation in his Bordeaux paper corresponded to method 2. The method was under development and he was hoping to receive comments from CIB-W18 members. Chan thought the real economic difference between methods was not very great because of the steps in member depth using commercial timber sizes.

Egerup said his impression from the meeting was that pinned-joint analysis should be put forward for the standard configurations composing the great bulk of trussed rafters, and that frame analysis was acceptable for the more special cases. However he thought the exact method of application should be set down for pinned-joint analysis, together with guidelines for frame analysis which could lead to more formal rules later.

DOCUMENTS

The following papers were put before the meeting:

H. Riberholt - Trussed rafters, static models. CIB-W18 paper, Helsinki, June 1980.

A.R. Egerup - Design of metal plate connected wood trusses. CIB-W18 paper 11-14-1, Vienna, March 1979.

A.R. Egerup - A simple design method for standard trusses. CIB-W18 paper 12-14-1, Bordeaux, October 1979.

A.R. Egerup - Truss design method for CIB timber code. CIB-W18 paper 13-14-1, Helsinki, June 1980.

References was also made to the following CIB papers:

B. Norén - Design of joints with nail plates. CIB-W18 paper 12-7-3, Bordeaux, October 1979. Corrected version in bound volume. Further corrections in manuscript paper numbered BN 1980-06-04, distributed for Helsinki meeting.

B. Norén - Design of joints with nail plates - calculation of slip. CIB-W18 paper 13-7-4, Helsinki, June 1980.

B. Källsner - Design of joints with nail plates - the heel joint. CIB-W18 paper 13-7-5, Helsinki, June 1980.

15 PAPERS PRESENTED AT THE MEETING

CIB-W18/13-6-1	Strength Classes for the CIB Code - J R Tory
CIB-W18/13-6-2	Consideration of Size Effects in Longitudinal Shear Strength for Uncracked Beams - R O Foschi and J D Barrett
CIB-W18/13-6-3	Consideration of Shear Strength on End-cracked Beams - J D Barrett and R O Foschi
CIB-W18/13-7-1	Polish Standard BN-80/7159-04:Parts 00-01-02-03-04-05. 'Structures from Wood and Wood-based Materials. Methods of Test and Strength Criteria for Joints with Mechanical Fasteners'
CIB-W18/13-7-2	Investigation of the Effect of Number of Nails in a Joint on its Load-carrying Capacity - W Nożyński
CIB-W18/13-7-3	International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber - B Norén
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CIB-W18/13-7-5	Design of Joints with Nail Plates - The Heel Joint - B Källsner
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CIB-W18/13-7-9	Strength of Finger Joints - H J Larsen
CIB-W18/13-12-1	Glulam Standard Part 3. Glued Timber Structures: Performance (4th draft)
CIB-W18/13-14-1	Truss Design Method for CIB Timber Code - A R Egerup
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CIB-W18/13-15-1	Timber and Wood-based Products Structures. Panels for Roof Coverings. Methods of Testing and Strength Assessment Criteria. Polish Standard BN-78/7159-03
CIB-W18/13-17-1	On Testing Whether a Prescribed Exclusion Limit is Attained - W G Warren
CIB-W18/13-100-1	Agreed Changes to CIB Structural Timber Design Code
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CIB-W18/13-103-1	Report from ISO TC/165

CURRENT LIST OF CIB-W18 TECHNICAL PAPERS 80.06.16

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

Example: CIB-W18/4-102-5

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

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

Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden, Feb/March 1977
8 Bruxelles, Belgium, October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979
13 Otaniemi, Finland; June 1980

Subjects are denoted by the following numerical classification:

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

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

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

LIMIT STATE DESIGN

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

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

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

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

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

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

TIMBER COLUMNS

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

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

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

4-2-2 Paper 4 Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns - B Johansson

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

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

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

6-2-1 Lattice Columns - H J Larsen

6-2-2 A Mathematical Basis for Design Aids for Timber Columns - H J Burgess

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

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

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

SYMBOLS

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

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

1 Symbols for Use in Structural Timber Design

PLYWOOD

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

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

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

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

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

STRESS GRADING

STRESSES FOR SOLID TIMBER

TIMBER JOINTS AND FASTENERS

1-7-1 Paper 12 Mechanical Fasteners and Fastenings in Timber Structures - E J Stern

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

7-7-2 Long Duration of 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

11-7-1 A draft Proposal for an International Standard: ISO Document ISO/TC 165N 38E

12-7-1 Load-carrying Capacity and Deformation Characteristics of Nailed Joints - J Ehlbeck

12-7-2 Design of Bolted Joints - H J Larsen

12-7-3 Design of Joints with Nail Plates - B Norén

13-7-1 Polish Standard BN-80/7159-04:Parts 00-01-02-03-04-05. 'Structures from Wood and Wood-based Materials. Methods of Test and Strength Criteria for Joints with Mechanical Fasteners'.

13-7-2 Investigation of the Effect of Number of Nails in a Joint on its load-carrying Capacity - W Nożyński.

13-7-3 International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber - B Norén.

13-7-4 Design of Joints with Nail Plates - Calculation of Slip - B Norén.

13-7-5 Design of Joints with Nail Plates - The Heel Joint - B Källsner.

13-7-6 Nail Deflection Data for Design - H J Burgess.

13-7-7 Test on Bolted Joints - P Vermeyden.

13-7-8 Comments to paper CIB-W18/12-7-3 'Design of Joints with Nail-Plates' - B Noren.

13-7-9 Strength of Finger Joints - H J Larsen.

13-100-4 CIB Structrual Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim.

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

4-8-2 Paper 13 Load Sharing - B Norén

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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 Long Term Loading - K Möhler

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

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

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

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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 Beams Notched at the Ends - K Möhler

11-10-1 Tapered Timber Beams - H Riberholt

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

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

ENVIRONMENTAL CONDITIONS

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

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

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

LAMINATED MEMBERS

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

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

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

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

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

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

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

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

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

13-12-1 Glulam Standard Part 3: Glued Timber Structures; Performance (4th draft)

PARTICLE AND FIBRE BUILDING BOARDS

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

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

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

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

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

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

TRUSSED RAFTERS

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

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

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

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

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

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

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

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

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

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

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

FIRE

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

13-100-2 CIB Structural Timber Design Code. Chapter 9. Performance in Fire.

CIB TIMBER CODE

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

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

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

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

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

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

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

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

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

11-100-2 Comments Received on the CIB Code

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

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

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

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

13-100-1 Agreed Changes to CIB Structural Timber Design Code

13-100-2 CIB Structural Timber Design Code. Chapter 9: Performance in Fire

13-100-3a Comments on CIB Structural Timber Design Code:

13-100-3b Comments on CIB Structural Timber Design Code - W R A Meyer

13-100-3c Comments on CIB Structural Timber Design Code - British Standards Institutio

13-100-4 CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim

LOADING CODES

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

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

STRUCTURAL DESIGN CODES

1-102-1 Paper 2 Survey of Status of Building Codes, Specifications etc, in USA - E G Stern

1-102-2 Paper 3 Australian Codes for Use of Timber In Structures - R H Leicester

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

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

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

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

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

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

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

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

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

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

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

11-102-1 Eurocodes - H J Larsen

13-102-1 Programme of Standardisation Work Involving Timber Structures and Wood-Based Products in Poland

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WORKING COMMISSION W18 - TIMBER STRUCTURES

Strength Classes for the CIB Code

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JUNE 1980

STRENGTH CLASSES FOR THE CIB CODE

J R Tory: Princes Risborough Laboratory; United Kingdom - May 1980

The strength classes given in the fourth draft of the CIB Structural Timber Design Code were accepted by CIB-W18 as tentative proposals until they could be substantiated or superseded by alternatives based on structural sized tests and an agreed method of stress derivation. There has been little progress in either of these directions and CIB-W18 must now make more positive decisions on strength classes before the Code is submitted to ISO/TC 165.

There are at present three strength class systems that could form the basis for the CIB Code strength classes. They originate from:

1 A proposal by Larsen; at present given in the fourth draft of the Code.
2 Australia[8].
3 The United Kingdom[4].

Estimates of the characteristic bending stresses associated with each of these systems are given in Table 1. It should be emphasised that the characteristic stresses associated with the UK code are crude estimates since moisture content, section depth and other factors in the UK code are not identical to those in the CIB or Australian codes.

Larsen's original proposal[3] for four strength classes T15, T18, T24 and T30 was later modified to five; SC15, SC19, SC24, SC30 and SC38. The characteristic bending stresses for the system were linked to tentative stresses for the ECE grades[1]. As time passed these stresses have tended to be viewed by some as final values, even though a planned verification programme to agreed standards for sampling, testing and analysis of results has yet to be undertaken. The system has also been criticised at meetings of CIB-W18 for not adequately covering a sufficiently wide range of species. In the United Kingdom for example the class characteristic bending stresses need to cover the range 5 to 50 N/mm^2. For Australia the range should be about 6 to 80 N/mm^2.

The Australian system more adequately caters for a full range of species/grades. Characteristic stresses estimated from their design stresses by Larsen[3] suggest that the system is reasonably compatible with that given in the present CIB Code, both being approximate geometric series with common ratio 1.259.

The maximum possible penalties inherent in these systems, for a species/grade which just fails to qualify for a class, is therefore approaching 26 per cent. It has been pointed out at earlier meetings that this possible penalty is unacceptably high.

The system included in the draft UK Code of Practice BS 5268:Part 2[4,2] was initially based on a geometric series with a common ratio of 1.4, centred on the grade stresses for GS and SS European redwood/whitewood. However, the original series was manipulated to particularly fit the species and visual grades that were most important to the UK. The large ratio between adjacent classes was chosen partly to limit the number of classes, partly in recognition of the coarseness of visual grading, and also in anticipation of the continuing success of machine stress grading which enables penalties to be minimised by the setting of machine limits at the class boundaries. The UK experience in deriving a class system suggests that departure from a rigid mathematical series is almost inevitable at the national level to utilise to the full the most important commercial species and grades. Practical visual grading rules, which are the predominant means of specifying structural timber, cannot be written to produce specific stresses: some discretion is therefore bound to be exercised by national code committees in defining classes for their domestic use.

The CIB approach to strength classes could ideally be one which ignores all current design stress values since none can be translated into characteristic values which comply with internationally accepted standards for the derivation of characteristic stresses: such standards do not exist. Also, if strength classes are established on the basis of present grade stresses, when these grades are tested in accordance with agreed procedures it may be found that the classes are biased against them. Consider for example the ECE grades: if an agreed test programme on ECE 8 redwood/whitewood produces a stress of, say 23 N/mm^2; or both ECE 8 and ECE 10 produce stresses between 24 and 30 N/mm^2, how acceptable would the class system in the present code draft be within Europe?

Any system of strength classes will inevitably introduce some measure of conservatism into the utilisation of structural timber. By adopting a system which contains a large number of classes for a given range the inefficiencies may be minimised and therefore the system becomes more readily acceptable to more countries. Within a large number of classes, from which each country would select a lesser number to suit its present needs, it might be possible to identify a more limited range that machine grading and visual grading revisions should eventually aim for.

The maximum possible penalties inherent in a class system based on a geometric series are a function of the common ratio between successive classes. To establish a system an acceptable maximum percentage penalty must therefore be decided, as well as a starting point for the series. Larsen and the Australians adopted for their systems a common ratio of $\sqrt[10]{10}$ (= 1.259) which was presumably taken from the R10 series of preferred numbers[5,9] even though the system stresses did not follow the preferred series. The use of preferred numbers in timber engineering has been suggested and explained by Booth[6]. They are introduced here as an arbitrary series as well suited to strength classes as any other geometric series. The terms of the R5, R10, R20 and R40 series which would be relevant to bending strength are given in Table 2.

It could justifiably be argued that the 5.9 per cent difference between successive terms in the R40 series is unnecessarily small for a strength class system, considering the inaccuracies and errors in grading. But some designers and commercial interests on code committees might well consider that even this value represents a significant penalty.

The R20 series, which would require a more manageable 25 terms to cover the full range of timber strength, increases the maximum penalty to 12.2 per cent. This figure, although not large considering the inaccuracies in the stresses, would perhaps deter some from adopting a standard system. Table 2 indicates how estimated characteristic stresses from the ECE document , the draft BS 5268[2] and the Australian code[3] might fit into classes based on the R20 series. The penalties are also given.

Since the R10 series has the same common ratio as the system in the fourth draft of the CIB Code it would probably be unacceptable as an initial class system because, as explained earlier, the inherent penalty of about 26 per cent is considered too high. However, either the R10 or the R5 series could provide the more limited range of stresses for machine grading and for longer term target class stresses.

A more difficult problem than establishing strength class boundaries simply on the basis of a single primary property is that of assigning values to other properties for the classes. In the fourth draft of the CIB Code the other property values associated with bending strength are particularly dependent on experience with European redwood/whitewood; they are therefore unlikely to receive worldwide acceptance. Leicester and Keating have identified this problem and have calculated regressions for the small clear mean strengths of several properties against mean bending strengths for different regions of the world[8], but their approach is also unlikely to have general validity or acceptance. The approach adopted for the revised UK code is to have a tolerance on the secondary class values. Species/grades which exceed 95 per cent of the class value are permitted in the class. This relaxation is applicable to modulus of elasticity, shear and compression stresses.

It may be of interest to note that in the machine stress grading research at PRL the relations between the characteristic values of modulus of elasticity bending, tension and compression strengths are being examined. These relations have been determined for redwood/whitewood[11,12] and are currently being explored for Canadian spruce-pine-fir. If such relations can be established they may considerably simplify the development of a strength class system, but obviously much more research has yet to be done.

It is felt that the definition of other properties for the strength classes requires further international consultation and a more comprehensive survey of current values; perhaps through ISO/TC 165. The first priority must be the basic framework for strength classes: that is, the number of classes, the common ratio between classes, the fixed datum within the series and the choice of the primary property.

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Table 1 CIB (fourth draft), Australian and United Kingdom Strength Classes. Estimated characteristic bending stresses.

(i) CIB (fourth draft)

Strength class	SC15	SC19	SC24	SC30	SC38
f_m (N/mm^2)	15	19	24	30	38

(ii) Australian*

Stress grade	F2	F3	F4	F5	F7	F8	F11	F14	F17	F22	F27	F34
f_m (N/mm^2)	6	7.5	9.5	12	15.5	19	24.5	31	38	49	61	77

(iii) United Kingdom+

Strength class	C1	C2	C3	C4	C5	C6	C7	C8
f_m (N/mm^2)	5.6	8.7	11.4	16.4	22.4	31.4	38.1	52.9

*See reference 3

+See references 2,4. Characteristic stress estimated as 2.24 grade stress.

Table 2 Series of Preferred Numbers and Penalties for ECE, UK and Australian Characteristic Bending Stresses Fitted to the R20 Series

Series and Common Ratios				ECE		UK		Australia	
R5 (1.585)	R10 (1.259)	R20 (1.122)	R40 (1.059)	Stress (N/mm^2)	% Penalty	Stress (N/mm^2)	% Penalty	Stress (N/mm^2)	% Penalty
	5.0	5.0	5.0						
			5.3						
		5.6	5.6			5.6	0.0	6.0	6.7
			6.0						
6.3	6.3	6.3	6.3						
			6.7						
		7.1	7.1					7.5	5.3
			7.5						
	8.0	8.0	8.0			8.7	8.0		
			8.5						
		9.0	9.0					9.5	5.3
			9.5						
10.0	10.0	10.0	10.0						
			10.6						
		11.2	11.2			11.4	1.8	12.0	6.7
			11.8						
	12.5	12.5	12.5						
			13.2						
		14.0	14.0	15.0	6.7			15.5	9.7
			15.0						
16.0	16.0	16.0	16.0			16.4	2.4		
			17.0						
		18.0	18.0	18.0	0.0			19.0	5.3
			19.0						
	20.0	20.0	20.0						
			21.2						
		22.4	22.4	24.0	6.7	22.4	0.0	24.5	8.6
			23.6						
25.0	25.0	25.0	25.0						
			26.5						
		28.0	28.0	30.0	6.7	31.4	10.8	31.0	9.7
			30.0						
	31.5	31.5	31.5						
			33.5						
		35.5	35.5			38.1	6.8	38	6.6
			37.5						
40.0	40.0	40.0	40.0						
			42.5						
		45.0	45.0					49	8.2
			47.5						
	50.0	50.0	50.0			52.9	5.5		
			53.0						
		56.0	56.0					61	8.2
			60.0						
63.0	63.0	63.0	63.0						
			67.0						
		71.0	71.0					77	7.8
			75.0						
	80.0	80.0	80.0						

CIB-W18/13-6-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CONSIDERATION OF SIZE EFFECTS IN LONGITUDINAL SHEAR STRENGTH FOR UNCRACKED BEAMS

by

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JUNE 1980

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CONSIDERATION OF SIZE EFFECTS IN LONGITUDINAL SHEAR STRENGTH FOR UNCRACKED BEAMS

Two recent publications in the Canadian Journal of Civil Engineering [1,2] report on research conducted at the Western Forest Products Laboratory on size effects in the longidutinal shear strength of uncracked beams. Results from this research form the basis for new design recommendations in the Canadian Code CSA-086 (1980).

In the context of the CIB Code, Clause 5.1.1.4 specifies that the shear stresses, τ , calculated according to the theory of elasticity, shall satisfy the condition

$$\tau \leqq f_v \tag{1}$$

where f_v is the characteristic value given in Table 5.1.0.a (with the appropriate modification factors) and τ is the maximum shear stress

$$\tau = 1.5 \frac{Q_M}{A} \tag{2}$$

where Q_M is the maximum shear force and A the beam cross-section.

The characteristic value f_v changes with the volume of the beam and the loading condition. It can be written as follows:

$$f_v = \beta f_{v,o} \left(\frac{V_o}{A\ L}\right)^{1/k} \frac{Q_M}{Q_o} \tag{3}$$

where $f_{v,o}$ = characteristic shear strength value for the volume V_o , uniformly stressed and arbitrarily chosen (0.02 m^3 if chosen the same as for tension perpendicular-to-the-grain);

Q_o = total load applied to the beam;

k = size effect coefficient;

β = factor dependent on the loading condition and the coefficient k;

A L = total beam volume, A being the cross-section and L the beam length.

The coefficient k must be obtained by tests. For Douglas-fir [1,2] , k = 5.53; for Hem-fir [3] , k = 5.94. It may be assumed, for convenience, that k = 6.0 represents a value sufficiently accurate and approximately species-independent.

The loading condition factor β is obtained by integration of the shear stress distribution over the volume of the beam, and the background for this integration is given in [1,2] . The general formulae for β is:

$$\beta = \left\{ F(k) \sum_{i=1}^{N} \left[\frac{L_i}{L} \int_{-1}^{1} \left(S_c \frac{Q_c(\xi)}{Q_o} + S_d \frac{Q_d(\xi)}{Q_o} \right)^k d\xi \right] \right\}^{-1/k} \tag{4}$$

where F(k) is the integral

$$F(k) = \frac{1}{4} \int_{-1}^{1} (1 - \xi^2)^k \, d\xi \tag{5}$$

and the remaining parameters in Eq.(4) are defined as follows:

N = is the number of segments along the beam length such that, within each segment, the shear force diagram is continuous and does not include a zero shear force;

L_i = the length of the particular segment;

Q_c = the shear force produced by concentrated (live) loads, given as a function of ξ , a non-dimensional integration variable which takes the values -1 and 1, respectively, at the beginning and the end of the segment of length L_i ;

Q_d = the shear force produced by distributed (uniform dead)loads, given as a function of ξ ;

S_c = factor (smaller than 1) that accounts for the influence of the supports upon the magnitude of the shear stresses near the beam mid-depth;

S_d = factor analogous to S_c but corresponding to distributed (dead) loads.

The factors S_c and S_d can be represented by the following exponential relationships:

$$
\begin{aligned}
S_c &= 1.0 - e^{-2.592\,(x/d)} \\
S_d &= 1.0 - e^{-3.410\,(x/d)}
\end{aligned}
\qquad (6)
$$

where d is the depth of the beam and x is the distance to the nearest support.

The shear forces in Eq.(4) must be taken in absolute value.

Eq.(4) can be integrated numerically for any value of k and any loading condition (including any combination of live/dead load ratios). Numerical integration is imperative for non-integer k, as in the case of k = 5.53 for Douglas-fir. Values of the integral F(k) are as follows:

Values of F(k)

k	F(k)
5.00	0.184704
5.53	0.176752
6.00	0.170496
7.00	0.159130
8.00	0.149769

Values of β for different loading conditions are shown in [2]. Table 1 shows the values of β obtained for distributed load, and for a single concentrated load at different positions along the span of a single span beam. There is a position that corresponds to the lowest load carrying capacity for the beam, and the coefficients β listed in [2] correspond to this "worst" position. The CIB Code could include the general equation for the calculation of β, and different national codes could take different approaches to the presentation of these calculations in their recommendations. Ideally, a designer should be able to estimate β for the particular loading condition that faces him. This can always be done by resorting to the basic equation, but a simplified approach may be desirable. In Canada, the approach followed is to publish tables of β values corresponding to common loading situations (as shown in [2]), and to give an approximate integration procedure to estimate β for other cases. This approximate procedure leads to conservative estimations. Another approach that could be followed is the following:

- Take the factors S_c and $S_d = 1$ (i.e., neglect the favorable effect of supports and adopt a conservative assumption);
- Compensate for this conservatism by increasing the factor k and thus reduce the size effect;

- In increasing the value of k, choose an integer value to facilitate the integration;
- Introduce modification factors (as needed) to minimize the differences with the basic formula for a range of loading conditions and volumes.

Approximate approach calibrated to Douglas-fir

Consider Figure 1, showing a segment of the beam where the shear force diagram is linear, with Q_L and Q_R the shear force values at the left and at the right end of the segment, respectively. For an integer k, the integration in Eq.(4) simplifies to the following:

$$I_i = \frac{2}{k+1} \frac{\left(\frac{Q_R}{Q_o}\right)^{k+1} - \left(\frac{Q_L}{Q_o}\right)^{k+1}}{\frac{Q_R}{Q_o} - \frac{Q_L}{Q_o}} \tag{7}$$

which, for the case of $Q_R = Q_L$ becomes:

$$I_i = 2 \left(\frac{Q_R}{Q_o}\right)^k \tag{8}$$

Eqs.(7) and (8) are easily computed for each segment and Eq.(4) becomes the approximate equation for β :

$$\beta \approx \beta^* = \left\{ F(k) \sum_{i=1}^{N} \frac{L_i}{L} I_i \right\}^{-1/k} \tag{9}$$

Furthermore, in order to minimize differences between the "exact" and the approximate approaches, let us introduce a factor α :

$$\beta \simeq \alpha \beta^* \tag{10}$$

Using k = 6, Table 2 shows the values of β^* corresponding to different loading conditions taken for the calibration. The ratio r shown is the ratio between

$$\beta \,/\, (\text{ Volume })^{1/5.53} \quad \text{and} \quad \alpha \beta^* \,/\, (\text{ Volume })^{1/6}$$

as these quotients affect directly the characteristic value f_v . The ratio r is, therefore,

$$r = \alpha \frac{\beta^*}{\beta} (\text{ Volume })^{0.014165} \tag{11}$$

and Table 2 shows this ratio for a small volume of 10,000 in^3 (0.164 m^3) and for a large volume of 500,000 in^3 (8.194 m^3). The factor α can be obtained to minimize

$$S = \sum_{j=1}^{M} (1.0 - r_j)^2 \tag{12}$$

since, ideally, all the ratios should be as close to 1.0 as possible. For the conditions considered in Table 2, $\alpha = 0.91095$. Thus, the approximate equation for β becomes:

$$\beta \simeq \beta^* = 0.91095 \left(0.170496 \sum_{i=1}^{N} (L_i \,/\, L)\, I_i \right)^{-1/6} \tag{13}$$

$$\beta \simeq \beta^* = 1.223 \left\{ \sum_{i=1}^{N} \frac{L_i}{L} I_i \right\}^{-1/6}$$

which can be verified to lead to errors of $\pm$ 7% for the calibration conditions considered in Table 2.

This calibration example has been presented as a possible avenue for national Codes to make simplifications to the basic Equation (4). It would allow the designer to compute β for the particular situation, rather than taking the conservative assumption of location at the worst position (as done in CSA-086, of necessity to reduce the number of Tables shown).

REFERENCES

1. Foschi, R.O. and Barrett, J.D. 1976. Longitudinal Shear Strength of Douglas fir. Canadian J. of Civil Engineering, 3, (2), pp. 198-208.

2. Foschi, R.O. and Barrett, J.D. 1977. Longitudinal Shear in Wood Beams: A Design Method. Canadian J. of Civil Engineering, 4, (3), pp. 363-370.

3. Barrett, J.D. 1976. Size Effects in Shear and Tension Perpendicular to Grain Strength of Timber. (in Wood Engineering Seminar 1975, A. Tayelor ed., Western Forest Products Lab. VP-X-162), pp.41. Data on Hem-fir obtained by S. P. Fox (unpublished).

TABLE 1

Values of β ($k = 5.53$)

Single Concentrated Load

Live Load to Dead Load (UDL) ratio	Load Location: At one beam depth from support	At worst location (x/L=0.2)	At beam midspan	Moving across span
0.0	3.69	3.69	3.69	3.69
0.5	4.03	3.15	3.34	2.60
1.0	3.71	2.83	3.14	2.25
1.5	3.51	2.66	3.01	2.08
2.0	3.38	2.55	2.92	1.98
3.0	3.23	2.43	2.80	1.87
5.0	3.09	2.31	2.68	1.77
10.0	2.98	2.22	2.58	1.68
infinite	2.85	2.11	2.46	1.59

Note: $L/d = 20$

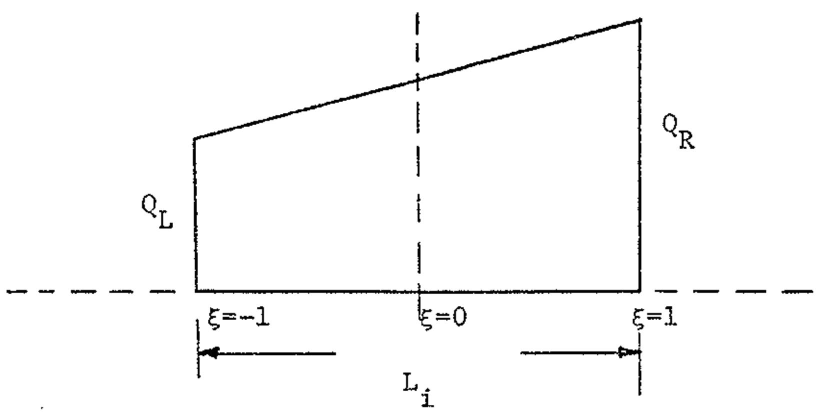
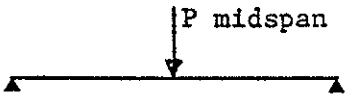
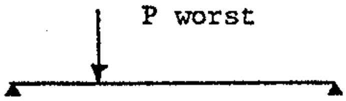
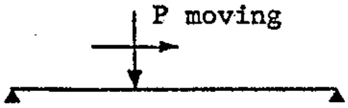
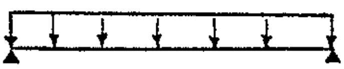
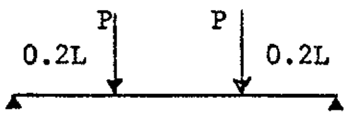


FIGURE 1

TABLE 2

Calibration of Simplified Approach to β

(Ratios r)

Load Case Considered	β	β*	Volume 10,000 in^3	Volume 500,000 in^3
	2.46	2.39	1.107 α	1.170 α
	2.11	1.93	1.042 α	1.102 α
	1.59	1.49	1.068 α	1.129 α
UDL: 	3.69	3.31	1.022 α	1.080 α
	2.73	2.61	1.089 α	1.151 α

CIB-W18/13-6-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CONSIDERATION OF SHEAR STRENGTH ON END-CRACKED BEAMS

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JUNE 1980

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CONSIDERATION OF SHEAR STRENGTH ON END-CRACKED BEAMS

Figure 1 shows a case of a beam with an end-crack (split) of length a. The reaction at the support where the crack is present is R, and the cross-sectional dimensions of the beam are, respectively, b and d.

The stress intensity factor K_{II} at the tip of the crack for such a configuration has been studied by Barrett and Foschi [1] and Murphy [2], and can be expressed as follows:

$$K_{II} = 2.8 \frac{R\,a}{b\,d^{3/2}} + 0.75 \frac{R}{b\,d^{1/2}} \qquad (1)$$

According to the hypothesis of Linear Elastic Fracture Mechanics, the end-crack will propagate in a shear mode when the stress intensity factor K_{II} equals a critical value K_{IIc}. If the shear stress τ at the support is defined as

$$\tau = 1.5 \frac{R}{b\,d} \qquad (2)$$

Eq.(1) can be re-written in terms of τ :

$$\tau = \frac{K_{II}}{d^{1/2}\ (\ 1.867\ a/d + 0.500\ )} \qquad (3)$$

Therefore, the design condition must be

$$\tau \leq k_{crack} \; K_{IIc} \tag{4}$$

where the factor k_{crack} is given by

$$k_{crack} = \frac{1}{d^{1/2} \; (1.867 \; a/d + 0.500)} \tag{5}$$

and K_{IIc} is the characteristic value for the critical stress intensity factor. This characteristic value must be shown, along with other characteristic values, on Table 5.1.0.a . Unless determined by tests for different species, a conservative estimate is

$$K_{IIc} \approx 2.0 \; f_{ASTM} \tag{6}$$

where f_{ASTM} is the shear strength determined with the ASTM standard block.

A beam with an end-crack must also be verified for shear strength using the formulation for un-cracked beams, as the crack may be located over the support corresponding to the beam end least stressed in shear.

REFERENCES

1. Barrett, J.D., and Foschi, R.O. 1977. Mode II Stress-Intensity Factors for Cracked Wood Beams. Engineering Fracture Mechanics, 9 pp. 371-378.

2. Murphy, J.F. 1979. Strength of Wood Beams with End Splits. USDA, Forest Products Laboratory, Madison, WI. For. Ser. Res. Paper FPL 347, 12pp.

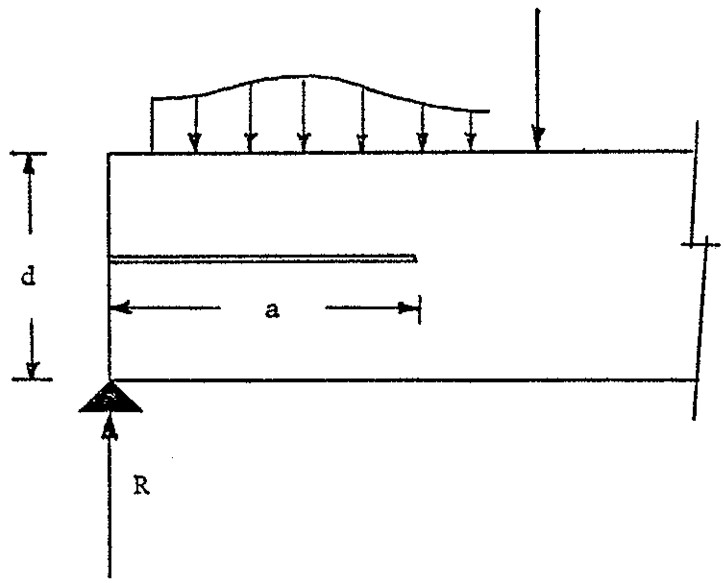

FIGURE 1

CIB-W18/13-7-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

POLISH STANDARD BN-80/7159-04:PARTS 00-01-02-03-04-05
"STRUCTURES FROM WOOD AND WOOD-BASED MATERIALS.
METHODS OF TEST AND STRENGTH CRITERIA FOR
JOINTS WITH MECHANICAL FASTENERS"

OTANIEMI, FINLAND
JUNE 1980

Proposal.

Building members from wood and wood-based materials	INDUSTRY-WIDE STANDARD Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. General provisions.	BN-80 7459-04 Sheet /part/ 00 Supersedes Catalogue group VII 39

I N T R O D U C T I O N

This standard is composed of a number of sheets /parts/. Fundamental tests, common for all types of mechanical connectors used in structures from wood and wood - based materials are given on sheet 01. Every successive sheet is dedicated to the tests on determined mechanical connectors, together with additional tests, special for this type of connectors. Every sheet constitutes a component of a sheet standard, being an independent standard by itself. This standard shall be completed with new sheets as the need arises, it means, when new connectors for such structures are introduced.

Actually the following sheets are prepared: 01, 02, 03, 04, 05.

Prepared by the Building Joinery Research and Development Centre having been approved by the Director of Building Joinery Union as an obligatory one since ....

1. Scope. The standard provides for methods of tests and strength assessment criteria for joints with mechanical fasteners used in load-bearing structures from wood and wood-based materials under static loads. This standard does not concern methods of test and strength assessment criteria for mechanical fasteners under dynamic loads.

2. Field of application. This standard shall be used for:

a/ determining of load-carrying ability of joints with mechanical fasteners in structures from wood and wood-based materials,

b/ determining of design load-carrying ability of mechanical fasteners in structures from wood and wood-based materials,

c/ determining of modulus of flexibility for joints with mechanical fasteners in structures from wood and wood-based materials,

d/ quality inspection of joints being rejected during acceptance.

3. Themes included in the standard are given on sheets as follows:

Sheet 00 - Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. General provisions.

Sheet 01 - Structures from wood and wood-based materials. Methods of tests and strenght assessment criteria for joints with mechanical fasteners. Testing.

Sheet 02 - Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. Nailed joints.

Sheet 03 - Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. Screw joints.

Sheet 04 - Structured from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. Bolted and screwed joints.

Sheet 05 - Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. Joints with toothed rings.

The successive sheets shall be prepared for joints with new mechanical fasteners.

ADDITIONAL INFORMATIONS

1. This standard has been prepared by - Building Joinery Research and Development Centre in Wołomin, Poland.

2. Authors of draft

- Prof. Eng.	- MICHNIEWICZ Wincenty
- Ass.Prof.Eng.	- DZIARNOWSKI Zbigniew
- Eng.	- SKALMOWSKA Danuta

Consultant: Ass.Prof.Eng. - NOŻYNSKI Władysław

Proposal.

Building members from wood and wood-based materials	INDUSTRY-WIDE STANDARD Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical fasteners. Testing.	BN-80 7159-04 PART-01 Supersedes Catalogue grou VII 39

1. INTRODUCTION

1.1. Scope. This standard provides for testing procedure and strength assessment criteria.

1.2. Definitions

1.2.1. Structures from wood and wood-based materials: building structures from timber, composite building structures from timber and wood-based materials, building structures from wood-based materials.

1.2.2. Joints with mechanical fasteners - part of structure composed of members joined together with mechanical connectors.

1.2.3. Mechanical fastener - fastener joining members of structure together, transmitting forces from one member to another.

1.2.4. Sample - a joint to be tested.

1.2.5. Bilogarithmic net - a net having ordinates and abscissae in logarithmic scale.

Prepared by Building Joinery Research and Development Centre having been approved by the Director of Building Joinery Union .... as an obligatory one since ....

1.3. Designations. In this standard the following symbols and designations are used:

A - area of connector, mm^2,

F - load /force/ , N,

F_c - characteristic load carrying ability of joints, N,

F_n - failure load of a sample /joint/ ,N,

F_o - design load-carrying ability of joints,N,

i - index corresponding to the order of samples or results,

k - symbol corresponding to the number of samples and result

k_o - flexibility of joint, mm/N,

M - modulus of flexibility of joint, N/mm,

m - number of shearing surfaces per joint, directivity facto of straight-line,

m_o - coefficient,

n - number of connectors per sample,

s - symbol of mean value, symbol of flexibility,

t - time , days,

T_c - characteristic load carrying ability of connector per 1 incision, N,

T_n - load carrying ability of connectors per 1 incision,N,

T_o - design load carrying ability per 1 incision, N,

u_d - matching strain , mm,

u_k - slip between members of sample, mm,

$u_{k_{50}}$ - deformation of joint /slip between its members/ after 50 years of loading, mm,

u_o - initial strain of sample ,mm,

u_s - elastic strain of joint ,mm,

u_t - slip of members of sample after t time,mm,

v_t - deformation rate of joint after t time, mm/day, z - theoretical deformation rate v_t after 1 day from loading mm/day,

γ_m - partial safety factor.

2. TESTS

2.1. Types of tests

2.1.1. Determination of load carrying ability of joints

2.1.2. Determination of slip of jointed members

2.1.3. Determination of the effect of loading time

2.1.4. Determination of the effect of connectors number

2.1.5. Additional tests

2.2. Specifications for samples /joints/

2.2.1. Materials of the sample /joint/. Materials of jointed and joining members shall comply with the requirements of standard specifications and with special requirements determined in corresponding sheets of this standard. In some special cases deviation from these requirements is allowed.

2.2.2. Joint. The form and dimensions of a joint shall comply with the requirements of the respective sheet of this standard, with corresponding standard specifications and with technical documentation. In some special cases deviation from these requirements is allowed.

2.3. Test requirements.

2.3.1. Samples. The samples shall be of rectangular form having the edges straight-lined, being perpendicular to each other. Other requirements are as in clause 2.

2.3.2. Minimum numbers of samples. For all the tests specified in 2.1.1. and 2.1.2. the number of samples k is of 12, for the tests as in 2.1.3. , 2.1.4. and 2.1.5. $k_{min} = 6$.
In the test as in clause 2d of BN-80/5059-04.00 , k depends on method of acceptance. The number of samples to be tested shall be at least 3.

2.3.3. Preparation of samples. Before starting the test every sample shall be:

a/ seasoned at least for 6 days in a climate as in 2.3.5.,

b/ marked , not to be mistaken with another ones,

c/ described, specifying:

- form and dimensions of its jointed and joining members,
- type of materials of sample members and their defects,
- water uptake of materials of sample members,
- other characteristics influencing the positioning of sample members,

d/ position of sample members shall be determined, using e.g. an ruler.

2.3.4. Determination of water uptake in materials of the sample
Determination shall be carried out using devices having an accuracy at least of 1%.

2.3.5. Climate. The tests shall be carried out in a closed room having a temperature of 20 ± 2°C and relative humidity of 60 ± 5%.

In special cases other climate is allowed.

2.3.6. <u>Static arrangement</u>. Static arrangement shall comply with the respective sheet of this standard.

2.3.7. <u>Equipment</u>. The machines and loading devices as well as gauges and ancillary equipment shall comply with requirements of PN-73/B-06281. Equipment having an accuracy of loading of 1% and an accuracy of measurement of slip of 0,01 mm is allowed.

2.3.8. <u>Determination of density of materials of joints</u> - according with the respective standard specifications.

2.4. <u>Determination of load carrying ability of joints</u>

2.4.1. <u>Static arrangement</u> - acc. 2.3.6.

2.4.2. <u>Method of test</u>. A joint shall be submitted to a load increasing from zero until failure value and failure load /force/ is recorded.

2.4.3. <u>Test procedure</u>. A sample prepared as in 2.3.3. shall be placed in testing machine as loaded as in 2.4.2.

As a moment of failure is considered a recess of the indicator of testing machine or its stopping the determined value when the slip between members of sample exceeds 15 mm. After the test is over, water uptake of samples shall be measured as in 2.3.4. as well as their density acc. 2.3.8.

2.4.4. <u>Test results</u>. The following values shall be calculated:

a/ load carrying ability per 1 incision from the formula:

$$T_{n_i} = \frac{F_{n_i}}{m \cdot n} \quad /N/ \qquad /1/$$

or

$$T_{n_i} = \frac{F_{n_i}}{m \cdot A} \quad /P_a/ \qquad /2/$$

where:

F_{n_i} - load carrying ability of i-sample, /N/,

m - number of shearing surfaces according with the respective sheet of this standard,

A - area of a connector in the sample, according with the respective standard /sheet of this standard/ ,mm^2,

n - number of connectors in the sample.

b/ mean load carrying ability of a serie of joints from the formula:

$$F_{n_s} = \frac{\sum_{i=1}^{i=k} F_{n_i}}{k} \quad /N/ \qquad /3/$$

c/ mean load carrying ability of a connector per 1 incision from the formula:

$$T_{n_s} = \frac{\sum_{i=1}^{i=k} T_{n_i}}{k} \quad /N, Pa/ \qquad /4/$$

2.5. Determination of slip between jointed members

2.5.1. Static arrangement - acc. 2.3.6.

2.5.2. Method of test. The sample shall be submitted to an initial loading, unloaded and then loaded again until failure, as on static arrangement on fig. 1. During the test the slip between sample members u_{k_i} /fig. 2/ and failure load are recorded / F_{n_i} /.

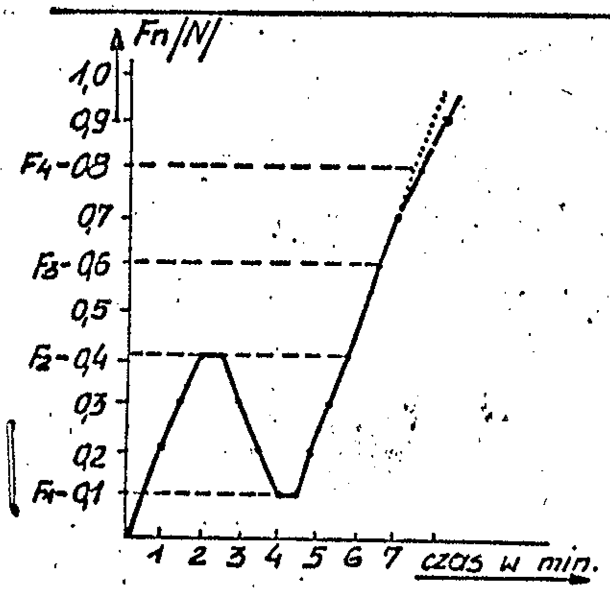

fig. 1
Loading diagram

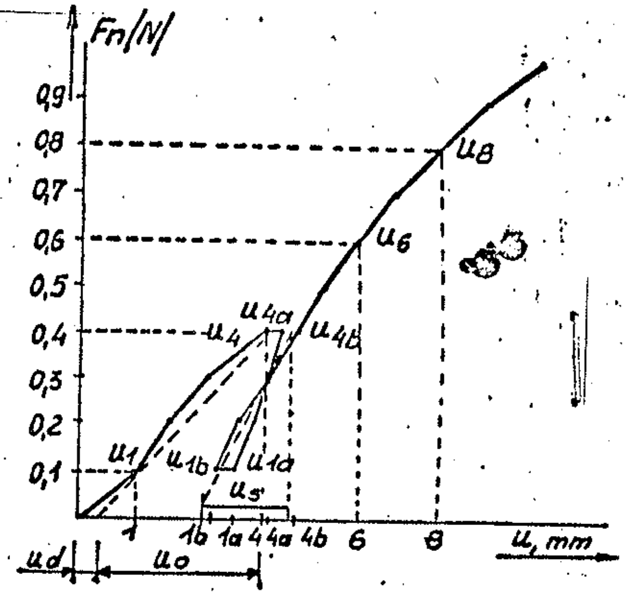

fig. 2
Slip diagram

2.5.3. <u>Test procedure</u> . The sample prepared as in 2.3.3. shall be placed in testing machine according with the respective static arrangement. After the dial gauges for slip measurement are installed, the sample shall be loaded with $F_1 = 0,1\ F_{ns}$ and the respective readings of u_1 taken. Then the load shall be increased until $F_2 = 4F_1$ gradually /4 times F_1/ and the respective readings of u_4 slip are taken. The load shall be increased to F_1 in 30 sec After F_2 load is reached, the sample shall be left under load for 30 sec and the readings of u_{4a} taken, and then the sample shall b[e] unloaded until F_1 value is reached. The unloading time shall be of 30 sec for each F_1 value. After this value is reached, the slips u_{1a} shall be read and the sample left under this load for 30 sec. After u_{1b} slip values are read, the sample shall be loaded gradually F_1 by F_1 until $F_2 = 4F_1$, and then until $F_3 = \cancel{8}F_1$ and $F_4 = 8F_1$,
6

always t-aking the respective readings of u_{4b}, u_6 and u_8 slips. Later the sample shall be loaded until failure. Every load increment in F_1 shall be reached in 30 sec.

2.5.4. Test results. The following values shall be calculated for each joint:

a/ initial strain ,u_o, as the minimum value from the formulae:

$$u_o = \frac{4/ u_4 - u_1/}{3} \quad /mm/ \qquad /5/$$

$$u_o = u_4 \quad /mm/ \qquad /6/$$

b/ matching strain ,u_d, from the formula:

$$u_d = u_4 - u_o \quad /mm/ \qquad /7/$$

c/ elastic strain , u_s , from the formula:

$$u_s = \frac{4}{3} \; / \frac{u_{4a} + u_{4b}}{2} - \frac{u_{1a} + u_{1b}}{2} / \quad /mm/ \qquad /8/$$

d/ u_{F_3} strain from the formula:

$$u_{F_3} = u_o + u_6 - u_{4b} \quad /mm/ \qquad /9/$$

e/ u_{F_4} strain from the formula:

$$u_{F_4} = u_o + u_8 - u_{4b} \quad /mm/ \qquad /10/$$

f/ flexibility of joint from the formula:

$$k_o = \frac{u_o}{4F_1} \quad /mm/N/ \qquad /11/$$

g/ load carrying ability of joint, F_n,

and for a serie o joints tested

h/ mean values of u_{os}, u_{ds}, u_{ss}, $u_{F_{4s}}$, $u_{F_{3s}}$, k_{os}, F_{ns}

2.6. Determination of the effect of loading time

2.6.1. Static arrangement - acc. 2.3.6.

2.6.2. Method of test. The sample shall be submitted to a load F_2 according with 2.5. for a determined period of time, during which the slips between members of the sample are measured.

2.6.3. Climate. The test shall be carried out in a closed room, having the climate as in 2.3.5.

2.6.4. Measurements. During the test the following values shall be recorded:

- slips between the members of the sample during loading,mm,
- time to failure, if it occurs, days,
- temperature and humidity of air in the testing station, °C,%

2.6.5. Measurement of slips. The slips shall be recorded continouosly.

Gradual readings of slip values are allowed in a following way: the first reading is ta-ken before loading the sample, the second one is taken immediately after first loading and the successive readings are taken:

during 10 days every 24 hours, during the successive 20 days every 48 hours, then 30 days every 5 days and finally every 10 days.

2.6.6. Loading time shall be determined individually for each case on the basis of test results until estabilishing the relationship between slip rate for members of the sample, and loading time. Minimum loading time is of 100 days.

2.6.7. Test procedure. The sample prepared for testing as in 2.3.3. shall be placed in testing machine. After gauges for measurement of slips between members of the sample are installed, so called, zero readings are taken, and then the sample shall be loaded as in 2.6.2. taking the respective readings. Later the sample is left for a time specified in 2.6.6. and the readings are taken as in 2.6.5.

2.6.8. Measurement of temperature and relative humidity of air shall be done continuously / equipment with automatic recording/.

2.6.9. Test results. After finishing the test on joints is necessary:

a/ calculate slips of their members:
- immediately after loading, u_a, mm,
- during loading , u_t, according with 2.6.5., mm,

b/ determine time to failure, if it occured during the tests, days,

c/ calculate deformation rate in relation to time, v_t, from the formula:

$$V_t = \frac{u_t - u_a}{t} \quad /mm/days/ \qquad /12/$$

where:

u_t , u_a - are from clause a/

t - loading time, days.

d/ the relationship between deformation rate , v_t, and time, t according with /12/ shall be drawn on bilogarithmic net. Time axis is that of abscissae and deformation rate axis is this of ordinates /mm/days/.

e/ approximate the curve as in clause d/ plotting e.g. straig line representing time- deformation rate relationship,

f/ determine the value of m from the equation e/:

$$\log v_t = m \cdot \log t + \log z \qquad /13/$$

where:

m- directivity factor of straight line on the diagram,

z- theoretical deformation rate v_t after one day of loading, mm/days,

g/ calculate the strain after 50 years , u_{k50}, from the formul

$$u_{k50} = z \cdot t_k^{/1+m/} + u_a \quad /mm/ \qquad /14/$$

where:

m , z - are from the formula /13/,

u_a - is from the clause a/,

t_k - 18250 days /50 years/.

h/ from the results for a serie of joints is necessary to calculate the respective mean value of u_{k50}.

2.7. Determination of the effect of connectors number

2.7.1. Static arrangement - acc. 2.3.6.

2.7.2. Sample - according with the respective sheet of this standard.

2.7.3. Method of test - acc. 2.4.2.

2.7.4. Test procedure - acc. 2.4.3.

2.7.5. Test results - acc. 2.4.4.

2.8. Additional tests - according with the corresponding sheets of this standard.

3. STRENGTH ASSESSMENT CRITERIA

3.1. Characteristic load carrying ability of joints is calculated from the formula:

$$F_c = F_n - t \cdot s \quad /N/ \qquad /15/$$

where:

F_{ns} - is from formula /3/, as F_{ni} shall be substituted all n-values obtained during the tests as in 2.4. and 2.5. /N/,

t - respective factors from tables of normal distribution at the significance level $\alpha = 0,05$

s - standard deviation calculated from the formula:

$$s = \sqrt{\frac{\sum_{i=1}^{i=k} /F_{ni} - F_{ns}/^2}{k - 1}} \quad /N/ \qquad /16/$$

3.2. Characteristic load carrying ability of connector per 1 infision shall be calculated from the formulae:

$$T_c = \frac{F_c}{m \cdot c} \quad /N/ \qquad /17/$$

or

$$T_c = \frac{F_c}{m \cdot A} \quad /N/ \qquad /18/$$

where:

F_c - acc. /15/,

m,n,A - acc. 2.4.4.

3.3. Modulus of flexibility of joints shall be calculated from the formula:

$$M = \frac{1}{k_{os}} \quad /N/mm/ \qquad /19/$$

3.4. Design load carrying ability of joints shall be calculated from the formulae:

$$F_o = \frac{F_c \cdot m_o}{\gamma m} \; /N/ \qquad /20/$$

$$F_o = \frac{F_c \cdot m_o}{2,25} \; /N/ \qquad /21/$$

$$F_o = M \cdot 1,5 \cdot m_o \; /N/ \qquad /22/$$

where:

$$\gamma m = \gamma m_1 \cdot \gamma m_2 \qquad /23/$$

$$\gamma m_1 = \frac{F_{ns}}{F_c} \qquad /24/$$

$$\gamma m_2 = 1,5 \quad \text{for } u_{k50} \leqslant 2,25$$

$$m_2 = 1,5 \cdot \frac{u_{k50}}{2,25} \quad \text{for } u_{k50} > 2,25$$

$$m_o = m_{o1} \cdot m_{o2} \cdot m_{o3} \qquad /25/$$

m_{o1} - factor determining the effect of connectors number from the respective sheet of this standard,

m_{o2} - factor determining the effect of joint manufacturing

m_{o2} - 0,9 for construction manufactured industrially,

m_{o2} - 0,8 for joint manufactured in another way,

m_{o3} - factor including additional factor affecting the load carrying ability of joints - according with the respective sheet of this standard,

F_c - from formula /15/,

M - from formula /19/.

As a load carrying ability of a joint the lower value from the formulae /20/, /21/, /22/ is taken.

3.5. Design load carrying ability of connector per 1 incision is calculated from the formulae:

$$T_o = \frac{F_o}{m \cdot n} \quad /N/ \tag{26}$$

or

$$T_o = \frac{F_o}{m \cdot A} \quad /N/ \tag{27}$$

where:

F_o - from 3.4.

m,n,A - from 2.4.4.

4. SELECTION OF TYPE OF TESTS

In dependence on field of application as in clause 2 of the standard BN-80/5059-04 sheet 00 the following types of tests are recommended:

- for the field of application as in 2a - tests acc. 2.4.
- for the field of application as in 2b - all the tests acc. 2
- for the field of application as in 2c - tests acc. 2.4. and 2.5.
- for the field of application as in 2d- tests acc. 2.4.

5. STRENGTH ASSESSMENT CRITERIA FOR JOINTS BEING REJECTED DURING ACCEPTANCE

The load carrying ability of a joint per 1 incision shall fulfill the following condition:

$$T_n \geqslant \frac{T \cdot \gamma_m}{M_{o2}} \quad /N/ \tag{28}$$

where:

T - load carrying ability of a connector per 1 incision, calculated according with PN-73/B-03150,

T_n - load carrying ability of a connector per 1 incision calculated on the basis of test results

$\gamma_m = \gamma_{m_1} \cdot \gamma_{m_2}$

γ_{m_1} = 1,25 for wood ,

γ_{m_1} = 1,50 for wood-based materials,

γ_{m_2} - partial safety factor determining the effect of long-term loads,

$\gamma_{m_2} = 1,50$

m_{o2} = 0,80 , factor acc. 3.4.

6. TEST REPORT

The report from every test performed shall contain:

- description of samples being tested,
- description of test procedure
- data concerning accuracy of measuring equipment,
- test daily register , with photographies, if needed,
- test results with discussion.

ADDITIONAL INFORMATIONS

ADDITIONAL INFORMATIONS

1. This standard has been prepared by: Building Joinery Research and Development Centre in Wołomin, Poland.

2. Reference standards:

PN-73/B-03150 - Timber structures. Static calculations and design.

PN-76/B-03001 - Building structures and soils. Design principles.

PN-73/B-06281 - Building pre-fabricated products from concrete. Methods of strenght tests.

3. Foreign standards:

CIB-RILEM Timber standard - 06. Test of mechanical fasteners test method for short-term testing. Draft.

4. Authors of this standard

Prof. Eng. - MICHNIEWICZ Wincenty

Ass.Prof.Eng. - DZIARNOWSKI Zbigniew

Eng. - SKALMOWSKA Danuta

Consultant: Ass.Prof. Eng. - NOŻYNSKI Władysław

Proposal.

Building members from wood and wood-based materials	INDUSTRY-WIDE STANDARD Structures from wood and wood-based materials. Method of tests and strength assessment criteria for joints with mechanical fasteners. Nailed joints.	BN-80 7159-04 Sheet 02 Supersedes Catalogue group VII 23

1. INTRODUCTION

1.1. Scope. This standard provides for methods of tests and strength assessment criteria for nailed joints in load-bearing structures from wood and wood-based materials under static loads.

1.2. Field of application: according with the clause 2 of BN-80/7159-04. Sheet 00.

1.3. Designations: In this standard the following symbols and designations are used:

a - thickness of joint members, mm,

a_1 - thickness of joint members, mm,

B - width of joint members, mm,

b - height of sample, mm,

c - dimension of a part of joint member, mm,

d - nail diameter, mm, cm.

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F- symbol of a force, N,

F_r - withdrawal force,

g - thickness of a panel from wood -based materials,mm,

γ_r - safety factor,

l - lenght of nail, mm,

l_p - lenght of part of nail in withdrawal,mm,

m_{o1} - factor corresponding to the effect of number of nails rows in a joint on its load carrying ability,

m_r - load carrying ability factor of nails for withdrawal,

m_{rc} characteristic value of m_r,

m_{ro} - design value of m_r,

m_{rs} - mean value of m_r,

m_{02} - factor corresponding to the effect of structure manufacturing on load carrying ability of joint,

p - number of rows of nails in a joint,

S - dimension of spacing of nails in a joint, mm,

S_1 - dimension of spacing of nails in a joint, mm,

S_2 - dimension of spacing of nails in a joint, mm,

S_3 - dimension of spacing of nails in a joint, mm,

T_{ns} - mean value of load carrying ability of nails per one incision, N,

T_{nsp} - value of T_{ns} for p rows of nails,N;

2. TESTS

2.1. Types of tests - acc. BN-80/7159-04 ,sheet 01, clause 2.1.

2.2. Specifications for samples /joints/ - acc. BN-80/7159,-04, sheet 01, clause 2.2.

2.3. Climate - acc. BN-80/7159-04, sheet 01, clause 2.3.

2.4. Investigation of load carrying ability of joints.

2.4.1. Static arrangement. The static arrangement, dimensions of samples, spacing of nails and distances between nails and edges of the sample shall be as given on fig. 1 and on table 1.

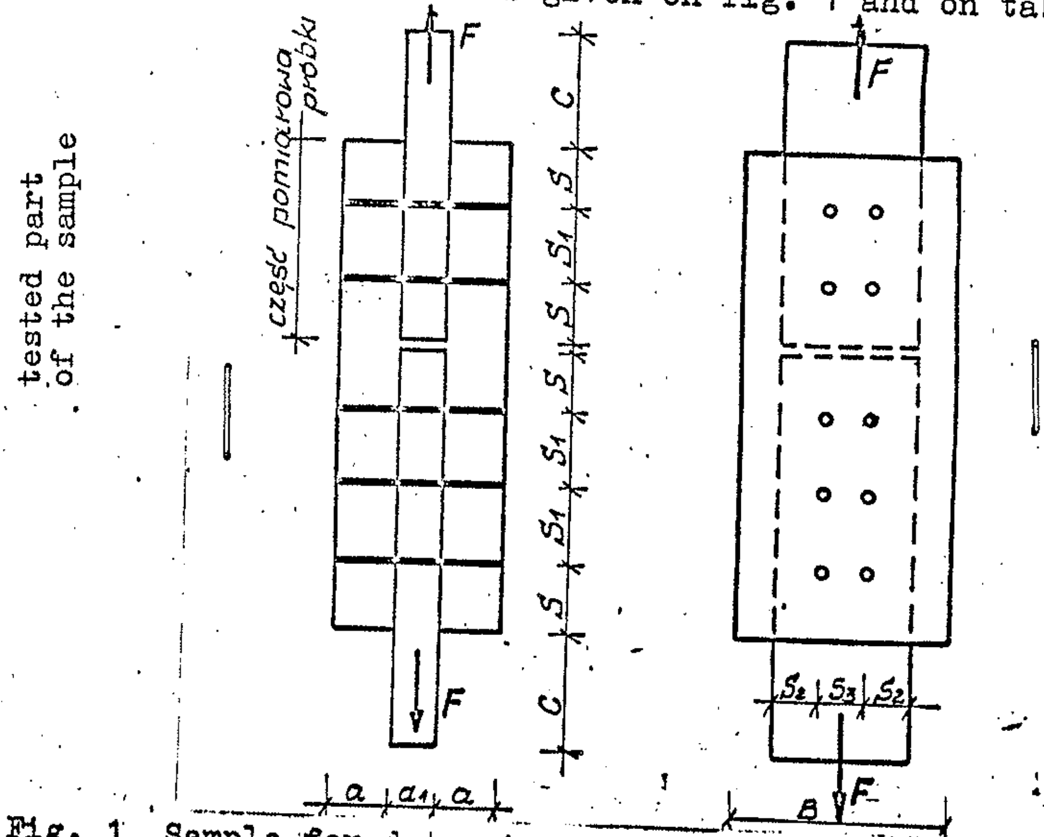

Fig. 1 Sample for determination of load carrying ability of joints

Table 1. Nominal dimensions of joints

Material / Designation acc. fig. 1 and 2	a	a_1	S	S_1	S_2	S_3	B
Wood	8d	6d	20d	25d	6d	6d	18d
Panels	g	g	20d	25d	6d	6d	18d

d - diameter of nails

g - thickness of panel or sheet

Nominal dimensions of joints from wood-based materials shall be checked having in consideration their strenght characteristics

2.4.2. Method of test - acc. BN-80/7159-04,sheet 01, clause 2.4.2.

2.4.3. Test procedure - acc. the above standard,clause 2.4.3.

2.4.4. Test results - acc. BN-80/7159-04,sheet 01,clause 2.4.4.

2.5. Determination of slips of jointed members

2.5.1. Static arrangement. Static arrangement, dimensions of samples, spacing of nails and distances between nails and edges of a sample shall be as in 2.4.1.

2.5.2. Method of tests - acc. BN-80/7159-04,sheet 01,clause 2.5.2.

2.5.3. Test procedure - acc. BN-80/7159-04, sheet 01,clause 2.5

2.5.4. Test results - acc. BN-80/7159-04, sheet 01,clause 2.5.4

2.6. Determination of the effect of loading time

2.6.1. Static arrangement - acc. 2.4.1.

2.6.2. Test method - acc. BN-80/7159-04,sheet 01, clause 2.6.2.

2.6.3. Climate - acc. BN-80/7159-04, sheet 01, clause 2.6.3.

2.6.4. Measurements - acc. BN-80/7159-04, sheet 01, clause 2.6.4

2.6.5. Measurement of slips - acc. BN-80/7159-04, sheet 01, clause 2.6.5.

2.6.6. Loading time - acc. BN-80/7159-04, sheet 01, clause 2.6.6

2.6.7. Test procedure - acc. BN-80/7159-04, sheet 01, clause 2.6.7.

The screws shall be driven into holes drilled previously having diameters in 2 mm smaller than their own diameters. These holes shall be drilled on a lenght of 0,8 l.

2.4.2. Test method - acc. BN-80/7159-04, sheet 01, clause 2.4.

2.4.3. Test procedure - acc. BN-80/7159-04, sheet 01, clause 2.4.3.

2.4.4. Test results -acc. BN-80/7159-04,sheet 01, clause 2.4.4

2.5. Determination of slips of jointed members

2.5.1. Static arrangement -Static arrangement, dimensions of samples, spacing of screws and distances between screws axis and edges of sample shall be as in 2.4.1.

2.5.2. Method of tests- acc. BN-80/7159-04,sheet 01,clause 2.5

2.5.3. Test procedure - acc. BN-80/7159-04,sheet 01,clause 2.5

2.5.4. Test results -acc. BN-80/7159-04,sheet 01,clause 2.5.4.

2.6. Determination of the effect of loading time - acc. BN-80/7159-04,sheet 01 ,clause 2.4.1.

2.7. Determination of the effect of connectors number

2.7.1. Static arrangement - as on fig. 3 and acc. 2.4.1.

2.7.2. Sample - acc. fig. 3 under the following conditions:

a/ every type of samples has a different number of rows tested part of the sample

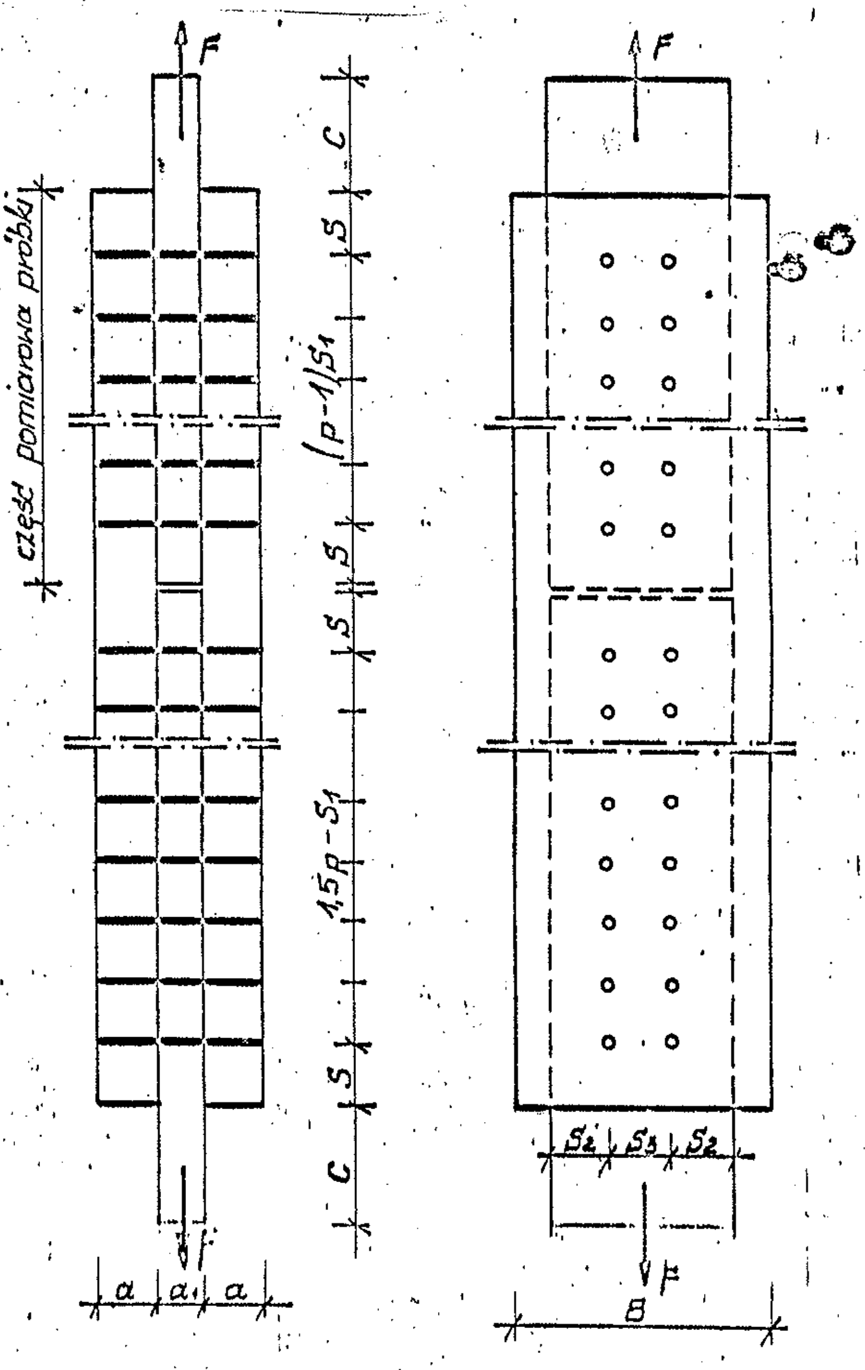

Fig. 2. Sample for determination of the effect of connectors number

2.7.5. <u>Test results</u> - acc. 2.7.5. -BN-80/7159-04, sheet 01 besides

d/ is necessary determine the value of m_{o1} for a variation of load carrying ability of nails per one incision in dependen on rows number, from the formula:

$$m_{o1} = \frac{T_{nsp}}{T_{ns}} = f\ /p/ \qquad /1/$$

where:

f /p/ - corelation relationship /function of regression develop on the basis of test results/. Straight-lined regressio is allowed.

T_{nsp} - from the formula /4/ from BN-80/7159-04, sheet 01 - mean load carrying ability of nails per 1 incision determined during the tests on joints having p nails rows

T_{ns} - from the formula /4/ from BN-80/7159-04, sheet 01, mean value of load carrying ability of nails per 1 incision, determined during tests on joints having p= 2.

2.8. Additional tests . An additional test to be carried out constitutes determination load carrying ability of nails from withdrawal force ; using timber having the following water uptake: A/ 12 ± 2% , B/ 23 ± 2%.

2.8.1. Static arrangement - according with that represented on fig. 3. Dimension of samples shall be as on the respective figure.

2.8.2. Method of test consists in withdrawing the nail from wood and determing the maximum withdrawal force - F_r.

2.8.3. Test procedure. The sample prepared for testing acc. BN-80/7159-04, clause 2.3.3./sheet 01/ shall be placed in testing machine and loaded as described in 2.8.2. After finishing the test water uptake of wood and its density shall be determined acc. BN-80/7159-04, sheet 01, clauses 2.3.4. and 2.3.8.

2.8.4. Test results. The following shall be calculated:

a/ for each sample - coeficient of load carrying ability of nails from withdrawal force - m_r , from the formula:

$$m_r = \frac{F_r}{d \cdot l_n} \quad /N/mm2/ \qquad /3/$$

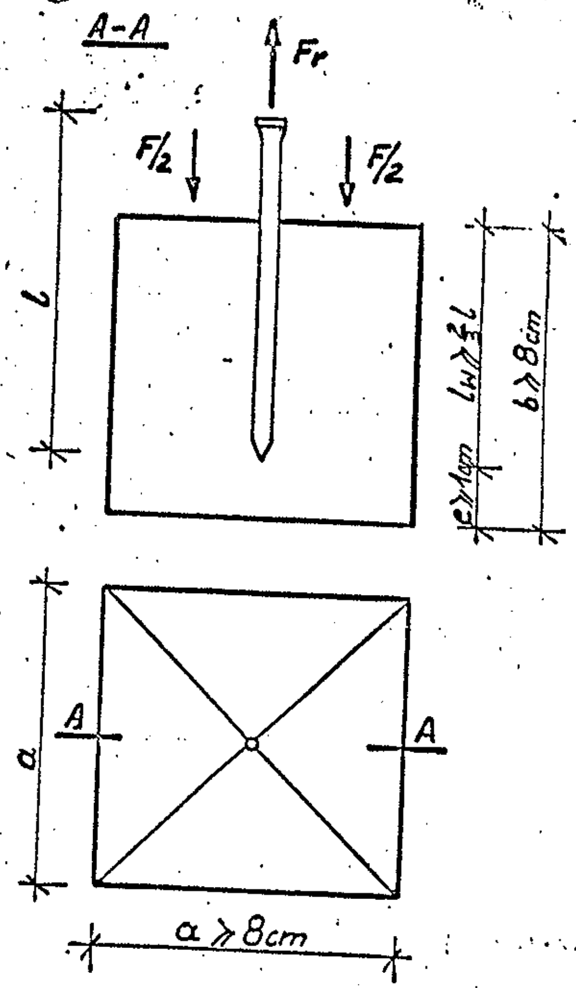

Fig.3 Sample for determining the withdrawal force

b/ for the whole serie the mean value of m^r shall be calculat from the formula:

$$m_{rs} = \frac{\sum_{i=1}^{i=k} m_{r_i}}{k} \quad /N/mm^2/ \qquad /4/$$

c/ characteristic value of m_r from the formula:

$$m_{rc} = m_{rs} - t \cdot s \; /N/mm^2/ \qquad /5/$$

d/ design value of m_r from the formula:

$$m_{ro} = \frac{m_{rc} \cdot m_{o2}}{\gamma_r} \quad /N/mm^2/ \qquad /6/$$

where:

d - diameter of nail , cm,

l_p - Lw - 1,5d

Lw - acc. fig. 3

t, s - from the formula /15/ - BN-80/7159-04, sheet 01

$$\gamma_r = \gamma_{r1} \cdot \gamma_{r2}$$

$$\gamma_{r1} = \frac{m_{rs}}{m_{rc}}$$

$$\gamma_{r2} = 1,5$$

m_{o2} - acc. BN-80/7159-04 sheet 01 clause 3.4.

ADDITIONAL INFORMATIONS

1. This standard has been prepared by: Building Joinery Rese and Development Centre in Wołomin, Poland.

2. Authors of the draft:

- Prof. Eng. - MICHNIEWICZ Wincenty
- Ass.Prof. Eng. - DZIARNOWSKI Zbigniew
- Eng. - SKALMOWSKA Danuta

Consultant: Ass.Prof. Eng. -NOŻYNSKI Władysław

Proposol.

Buildings members from wood and wood-based materials

INDUSTRY-WIDE STANDARD

Structures from wood and wood-based materials.
Methods of tests and strength assessment criteria for joints with mechanical fasteners.
Screw joints.

BN-80
7159-04
Sheet 03

Supersedes

Catalogue group
VII 39

1. INTRODUCTION

1.1. Scope. This standard provides for methods of tests and strength assessment criteria for screw joints used in load-bearing structures from wood and wood-based materials under static loads.

1.2. Field of application - acc. BN-80/7159-04, sheet 00, clause 2.

1.3. Designations: In this standard sheet the following symbols and designations are used:

a - dimension of joint member,

a_1 - dimension of joint member,

B - dimension of joint member,

b - height of sample,

c - dimension of joint member,

d - diameter of screw,

F - symbol of force, load,

F_r - withdrawing force of screw,

Prepared by Building Joinery Research and Development Centre having been approved by the Director of Building Joinery Union .... as an obligatory one since ....

g - dimension of joint members,

γ_r - safety factor,

l - lenght of screw,

m_{o1} - factor corresponding to the effect of number of screw rows in a joint on its load carrying ability,

m_{o2} - factor corresponding to the effect of structure manufacturing on load carrying ability of joints,

m_r - load carrying ability factor of screws for withdrawal

m_{rc} - characteristic value of m_r,

m_{rs} - mean value of m_r,

m_{ro} - design value of m_r,

p - number of csrews rows,

S - dimension of spacing of srews in a joint, mm,

S_1 - dimension of spacing of screws in a joint,

S_2 - dimension of spacing of screws in a joint,

S_3 - dimension of spacing of screws in a joint,

T_{ns} - mean value of load carrying abilfty of screws per one incision,

T_{nsp} - value of T_{ns} for p rows of screws.

2. TESTS

2.1. Types of tests - acc. BN-80/5059-04, sheet 01, clause 2.1.

2.2. Specifications for samples/ joints/ - acc. BN-80/5059-04, sheet 01, clause 2.2.

2.3. Climate - acc. BN-80/5059-04, sheet 01, clause 2.3.

2.4. <u>Determination of load carrying ability of joints</u>

2.4.1. <u>Static arrangement</u>. The static arrangement, dimensions of samples, spacing of screws and distances between screws axis and edges of the sample shall be as given on fig. 1 and on table. The samples as represented on fig.1 shall be used for screws having diameter d ≤ 10 mm. For the screws having larger diameters the corresponding samples shall be as represented on fig.2.

Tested part of sample

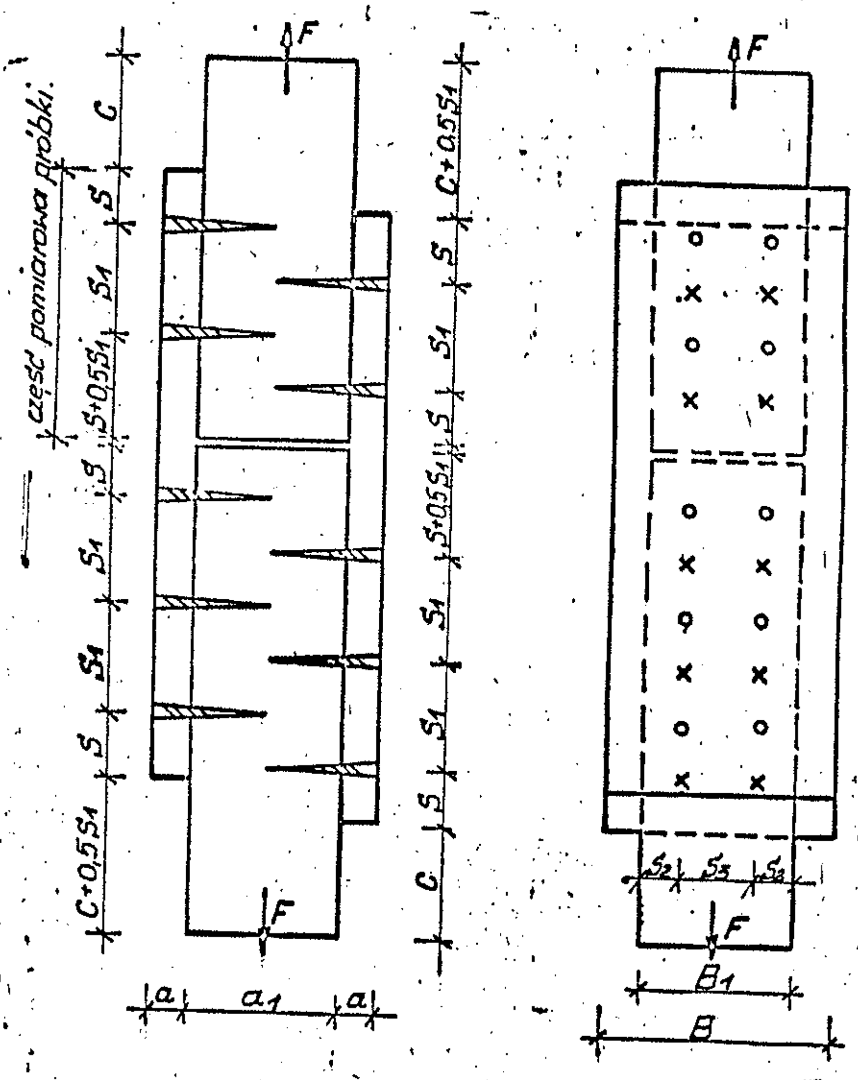

Fig. 1 Sample for determination of load carrying ability of joints with screws having diameters d ≤ 10 mm.

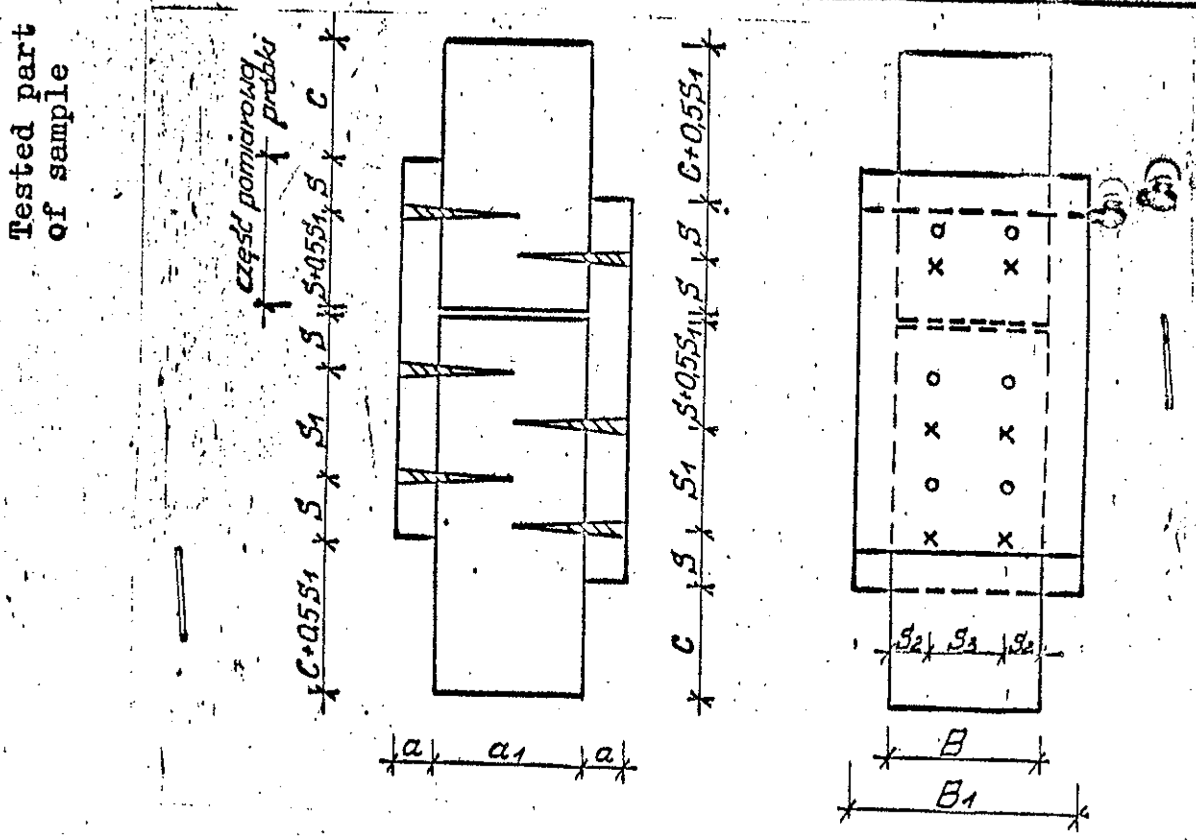

Fig.2 Sample for determination of load carrying ability of joints with screws having diameters d ≥ 10 mm

Table 1. Nominal dime-nsion of joints

Designation acc. fig. 1 and 2 / Material	a	a_1	S	S_1	S_2	S_3	B
Wood	3d	1d–10d	12d	24d	5d	5d	15d
Panels	g	1-g+2d g	12d	24d	5d	5d	15d

d - diameter of screw

l - lenght of screw

g - thickness of panel or sheet

Nominal dimension of joints members from wood-based materials shall be checked in view of their strength characteristics.

2.6.8. Measurement of temperature and relative humidity of air – acc. BN-80/5059-04 ,sheet 01, clause 2.6.8.

2.6.9. Test results – acc. BN-80/5059-04,sheet 01,clause 2.6.9.

2.7. Determination of the effect of connectors number

2.7.1. Static arrangement – as on fig. 1.

2.7.2. Sample – as on fig. 2 under the following conditions:

a/ every type of samples has a different number of rows in the tested part – p.

b/ number of nails rows in the tested part of sample shall be even,

c/ number of nails rows in the other part ofthe sample shall be of 1,5 p,

d/ spacing of nails and the distances between the nails and edges shall be as those specified in clause 2.4.1.-table 1.

e/ minimum number of samples for one test shall be of 5.

2.7.3. Method of test – acc. BN-80/5059-04, sheet 01, clause 2.7.3.

2.7.4. Test procedure – acc. BN-80/5059-04 ,sheet 01, clause 2.7.4.

c/ spacing of screws and distances between their axis and edges of sample shall as those given on table 1.

d/ minimum number of samples for 1 test shall be of 5.

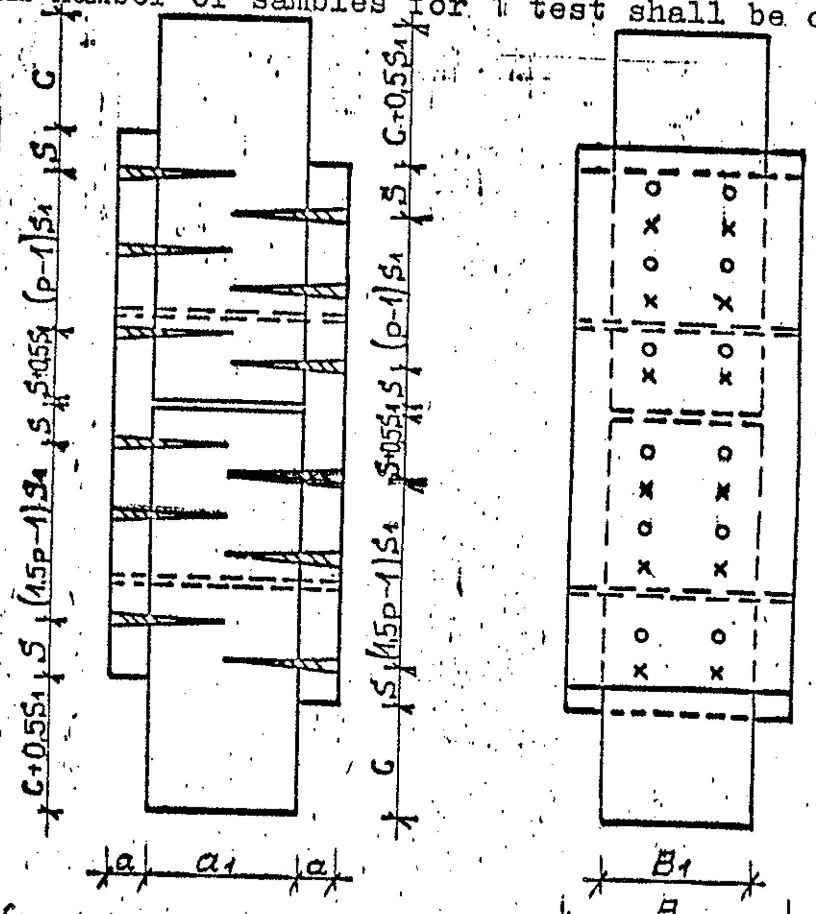

Fig.3 Sample for determination of the effect of connectors number.

2.7.3. Method of test - acc. BN-80/5059-04, sheet 01, clause 2.7.3.

2.7.4. Test procedure -acc. BN-80/5059-04 sheet 01, clause 2.7.4.

2.7.5. Test results - acc. BN-80/5059-04 sheet 01,clause 2.7.5. and additionally:

d/ is necessary determine the value of m_{01} from the formula /1/ of BN-80/5059-04, sheet 02, clause 2.7.5., substituing the respective values for screws.

2.8. Additional tests. Additionally the load carrying ability of screws from withdrawal force shall be determined.

2.8.1. Static arrangement. Static arrangement for test, dimensions of sample and distances between the screws axis and the edge of sample shall be as those on fig. 4.

For wood:

$a \geq 8$ cm
$e \geq 1$ cm
$c = l - lw$
$lw \geq 2/3\ l$
$b = lw + e$

For wood-based materials:

g - thickness of panel
$e \geq d$
$lw = g - e$
$c = l - lw \geq 1/3\ l$
$a \geq 8$ cm

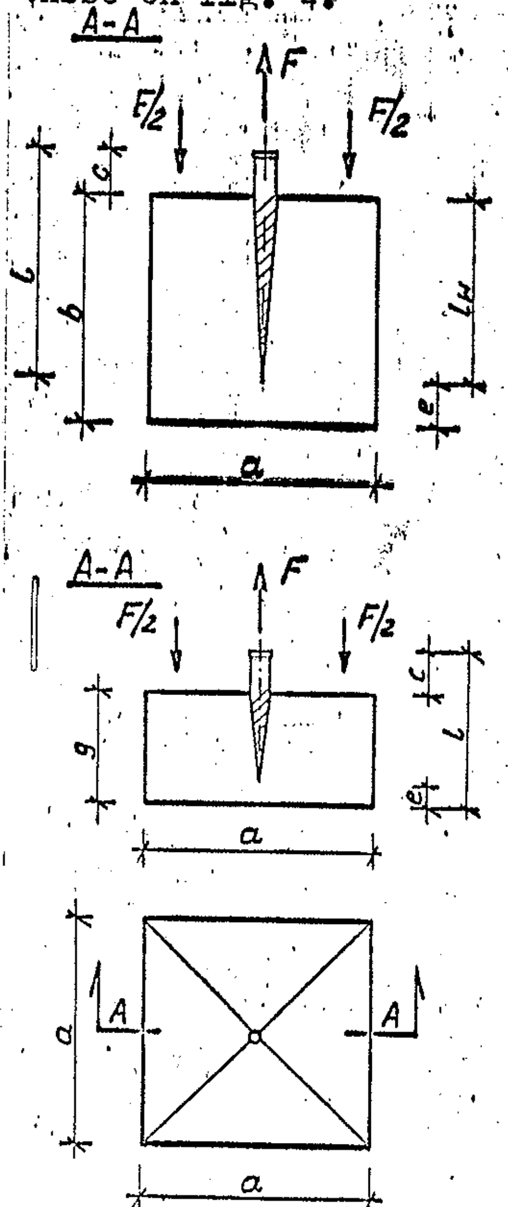

Fig. 4 Sample for determination of load carrying ability in witdrawal

2.8.2. Test method consist in extracting the screw from wood and determining of maximum withdrawing force $-F_r$.

2.8.3. Test procedure. The sample prepared as in BN-80/5059-04 sheet 01 clause 2.3.3. shall be placed in testing machine and loaded uniformely according with 2.8.2. After finishing the test is necessary determine the water uptake of the sample and its density according with BN-80/5059-04 ,sheet 01, clauses 2.3.4. and 2.3.8.

2.8.4. Test results. The following shall be calculated:for

a/ load carrying ability factor for withdrawal from the formula:

$$m_r = \frac{F_r}{\pi d \cdot lp} \quad /N/mm^2/ \qquad /1/$$

b/ mean value - m_{rs}

c/ characteristic value - m_{rc} - from the formula:

$$m_{rc} = m_{rs} - t \cdot s \quad /N/mm^2/ \qquad /2/$$

d/ design value - m_{ro} -from the formula:

$$m_{ro} = \frac{m_{rc} \cdot m_{o2}}{\gamma_r} \quad /N/mm^2/ \qquad /3/$$

where:

d - screw diameter

Lp -lw = 1,5d

lw - as on fig. 4

t,s - from the formula /15/-BN-80/5059-04 sheet 01

$\gamma_r = \gamma_{r1} \cdot \gamma_{r2}$

$\gamma_{r1} = \frac{m_{rs}}{m_{rc}}$

$\gamma_{r2} = 1,5$

m_{o2} - acc. BN-80/5059-04 sheet 01 ,clause 3.4.

ADDITIONAL INFORMATIONS

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1. This standard has been prepared by: Building Joinery Research and Development Centre in Wołomin, Poland.

2. Authors of draft:

- Prof. Eng. - MICHNIEWICZ Wincenty
- Ass.Prof. Eng. - DZIARNOWSKI Zbigniew
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Proposal

Building members from wood and wood-based materials	INDUSTRY-WIDE STANDARD Structures from wood and wood-based materials. Methods of tests and strength assessment criteria for joints with mechanical connectors. Bolted and screwed joints.	BN-80 7159-04 Sheet 04 Supersedes Catalogue group VII 39

1. INTRODUCTION

1.1. Scope. This standard provides for methods of tests and strength assessment criteria for bolted and screwed joints in load-bearing structures from wood and wood-based materials under static loads.

1.2. Field of application: - acc. BN-80/7159-04, sheet 00.

1.3. Designations:

a - dimension of joint member

a_1 - dimension of joint member,

B - dimension of joint member,

C - dimension of joint member,

d - diameter of bolt or screw,

F - symbol of force,

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g - dimension of joint member,

m_{o1} - factor determining the effect of screw and bolt rows number on load carrying ability of joint,

m_{o3} - factor determining the effect of grain direction in relation with force direction of action on load carrying ability of joint,

p - number of rows of bolts or screws,

s - spacing of bolts or screws in a joint,

S_1 - spacing of bolts or screws in a joint,

S_2 - spacing of bolts or screws in a joint,

S_3 - spacing of bolts or screws in a joint,

T_{ns} - mean value of load carrying ability of bolts or screws per one incision,

T_{nsa} - value of T_{ns} for a force acting perpendiculary to grain.

2. TESTS

2.1. Types of tests - acc. BN-80/ 159-04 sheet 01, clause 2.1.

2.2. Specifications for samples /joints/ - acc. BN-80/7159-04 sheet 01, clause 2.

2.3. Climate - acc. BN-80/7159-04 sheet 01, clause 2.3.

2.4. Determination of load carrying ability of joints

2.4.1. Static arrangement. Static arrangement, dimensions of samples, spacing of bolts and screws and distances between bolts and screws axis and edges of the sample shall be as those represented on fig. 1 and on table 1.

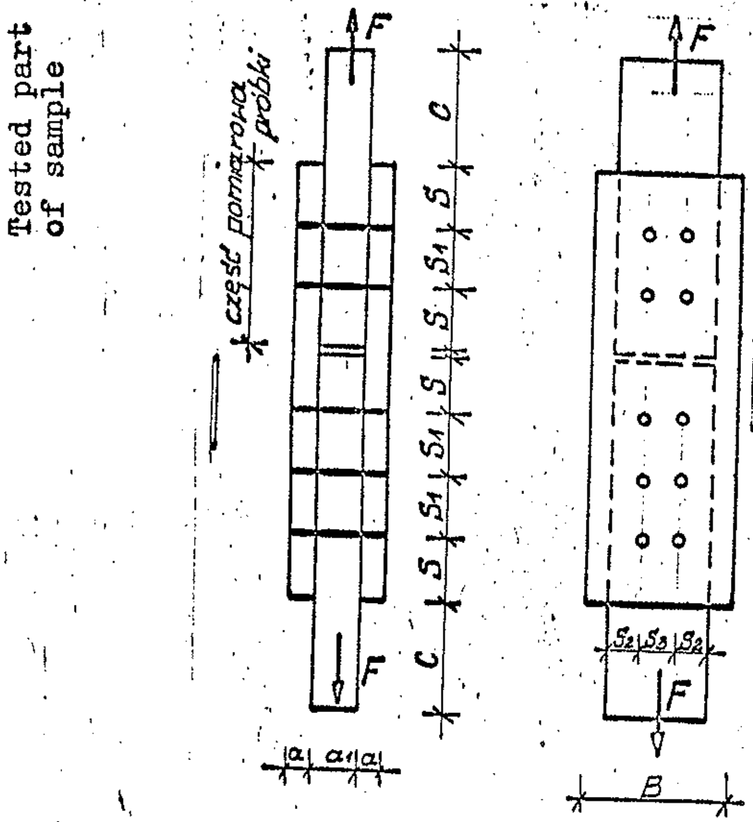


Fig. 1 Sample for determination of load carrying ability of symmetrical joints

Table 1. Nominal dimensions of joints

Material	Designation acc. fig. 1 and 2 Type of samples	a	a_1	S	S_1	S_2	S_3	B
wood	A	3d	3d	8d	8d	4d	5d	⩾13d
	B	2d	6d					
panels	A	g	g	8d	8d	4d	5d	⩾13d
	B	g	3g					

d - diameter of bolt or screw
g - thickness of panel or sheet

Nominal dimensions of joints from wood -based materials shall be checked having in consideration their strength characteristics. Both, bolts and screws shall be placed in having a diameter of 0,97 d.

2.4.2. Types of samples . For this test three types of samples shall be prepared having different forms and dimensions: types A and B as on fig. 1 and according with the table 1 , and type C as on fig. 2 and according with table 1. The corresponding dimensions are as follows: $a = 2d$ or $a = g$, $a_1 = 6d$ or $a_1 = 3\ g$.

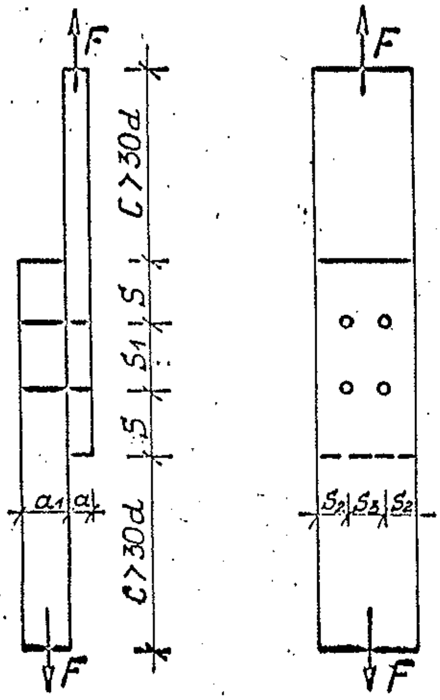


Fig. 2 Sample for determining load carrying ability of assymmetrical joints.

2.4.3. Method of test - acc. BN-80/1459-04 ,sheet 01 ,clause 2.4.2.

2.4.4. Test procedure - acc. BN-80/1459-04, sheet 01,clause 2.4.3.

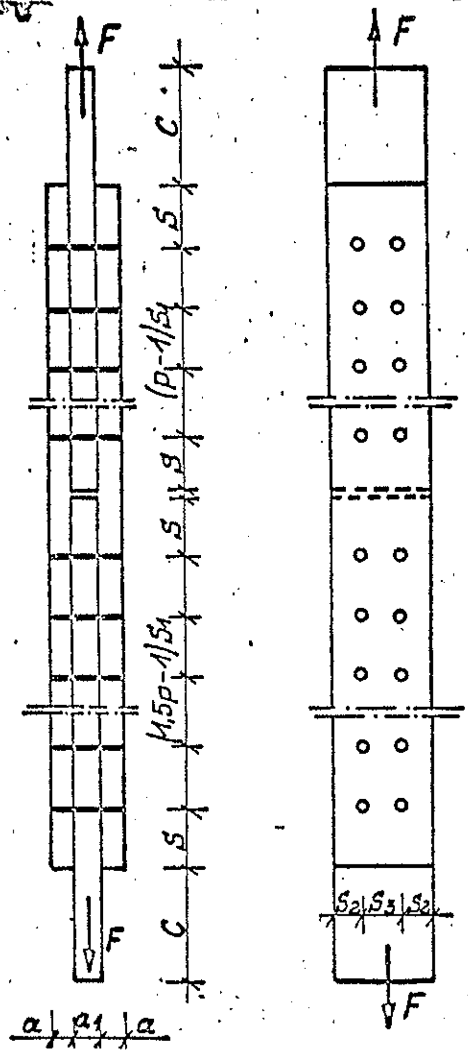
2.4.5. Test results - acc. BN-80/1459-04 ,sheet 01, clause 2.4.4.

2.5. Determination of slips of jointed members - acc. BN-80/7159-04, sheet 01, clause 2.5. Static arrangement and samples shall be as those in 2.4.1. and 2.4.2. of this sheet.

2.6. Determination of the effect of loading time - acc. BN-80/7159-04. The corresponding static arrangment and samples shall be as those in 2.4.1. and 2.4.2. of this sheet.

2.7. Determination of the effect of connectors number

2.7.1. Static arrangement - as on fig. 3.



a= 3d
a_1= 3d

Fig. 3 Sample for determining of the effect of connectors number

2.7.2. Sample - as on fig.3 under following conditions:

a/ every sample type has a different number of bolts and screws rows in its tested part - p ,

b/ number of bolts or screws rows in the other part of sample shall be as specified on table 1,

c/ minimum number of samples for testing shall be of 5.

2.7.3. Method of test - acc. BN-80/7159-04 ,sheet 01,clause 2.7.3.

2.7.4. Test procedure - acc. BN-80/7159-04,sheet 01, clause 2.7.4.

2.7.5. Test results - acc. BN-80/7159-04, sheet 01, clause 2.7.5. and besides is necessary:

d/ determine factor m_{01} - acc. BN-80/7159-04 ,sheet 02, clause 2.7.5.

2.8. Additional tests. As an additional test the load carrying ability of bolts and screws shall determined in direction perpendicular to grain in joints from wood or to grain of face veneer in joints from plywood or to main direction of panels from wood-based materials. The main direction for panels from wood-based materials is that along the panel.

2.8.1. Static arrangement. Static arrangement, dimensions of sample, spacing of bolts and screws and distances between bolts and screws axis from egdes of sample shall be as given on fig.4 and according with table 1. For bolts and screws having diameters up to 10 mm - sample A is taken as on fig. 5 and according with the table 1, for bolts and screws having diameters more than

10 mm - sample A is taken . Bolts and screws shall be placed in holes previously drilled having diameters of 0,97 d.

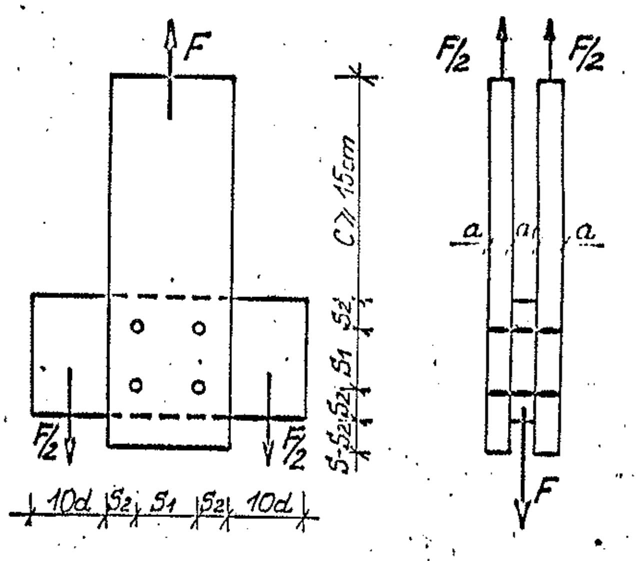


Fig. 4 Sample for determining load carrying ability of bolts and screws in direction perpendicular to grain for d ≤ 10 mm

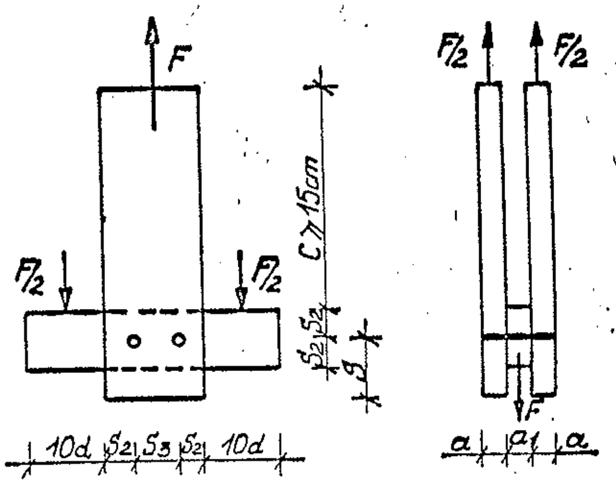


Fig. 5 Sample for determining of load carrying ability of bolts and screws in direction perpendicular to grain for d ≥ 10 mm.

2.8.2. Method of test - acc. BN-80/7141-59-04, sheet 01, clause 2.4.2.

2.8.3. Test procedure - acc. BN-80/7141 59-04 sheet 01, clause 2.4.3.

2.8.4. Test results - acc. BN-80/7141 59-04, sheet 01, clause 2.4.4. and determine the value of m_{o3} from the formulae:

$m_{o3} = 1,00$ - for direction paralel to grain of wood, plywood and panels,

$m_{o3} = m_{o3a}$ - for direction perpendicular to grain of wood,

$1,00 > m_{o3} > m_{o3a}$ - for other directions.

Linear interpolation

where:

$$m_{o3a} = \frac{T_{nsa}}{T_{ns}}$$

T_{nsa} - mean value of T_{ns} from the formula /4/ -BN-80/5059-04, sheet 01 and on the basis of results according with 2.8.4.

T_{ns} - mean value of T_{ns} determined as in 2.4.

ADDITIONAL INFORMATIONS

1. This standard has been prepared by: Building Joinery Research and Development Centre, Wołomin, Poland

2. Authors of draft:

- Prof. Eng. - MICHNIEWICZ Wincenty
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Consultant: Ass.Prof. Eng. - NOŻYNSKI Władysław

Proposal

Building members from wood and wood-based materials	INDUSTRY-WIDE STANDARD Structures from wood and wood-based materials. Test methods and strength assessment criteria for joints with mechanical fasteners. Joints with toothed rings.	BN-80 7159-04 Sheet 05 Supersedes Catalogue group VII 32

1. INTRODUCTION

1.1. Scope. This standard provides for methods of tests and strength assessment criteria for joints with toothed rings in load bearing structures from wood under static load.

1.2. Field of application - as in BN-80/7159-04, sheet 00, but exclusively for wooden structures.

1.3. Designations.

a - dimension of joint member,

b - diameter of screw,

d_o - diameter of toothed ring,

F - symbol of force

m_{o1} - factor of load carrying ability variation in relation with rings number in a joint,

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m_{o3} - factor determing the effect of grain direction in relation with direction of action on load carrying ability of a joint,

p - Number of groups of rings per joint,

S - spacing of rings in a joint,

S_1 - spacing of rings in a joint,

S_2 - spacing of rings in a joint,

T_{ns} - mean value of load carrying ability of rings in joint,

T_{nsa} - value of T_{ns} for direction of action perpendicular to grain.

2. TESTS

2.1. Types of tests -acc. BN-80/5059-04, sheet 01, clause 2.1.

2.2. Specifications for samples /joints/ - acc. BN-80/5059-04, sheet 01, clause 2.

2.3. Climate - acc. BN-80/5059-04, sheet 01, clause 2.3.

2.4. Determination of load carrying ability of joints

2.4.1. Static arrangement. Static arrangement for this tests, dimensions of samples, spacing of axis of rings, distances between rings axis and edges of samples shall be as given on fig. 1 and on table 1. Jointed members of sample shall be screwed together with screws with washers, placed on axis of each ring. The screws shall be placed in the holes drilled previously having diameters of 0,97d, where d - diameter of screws.

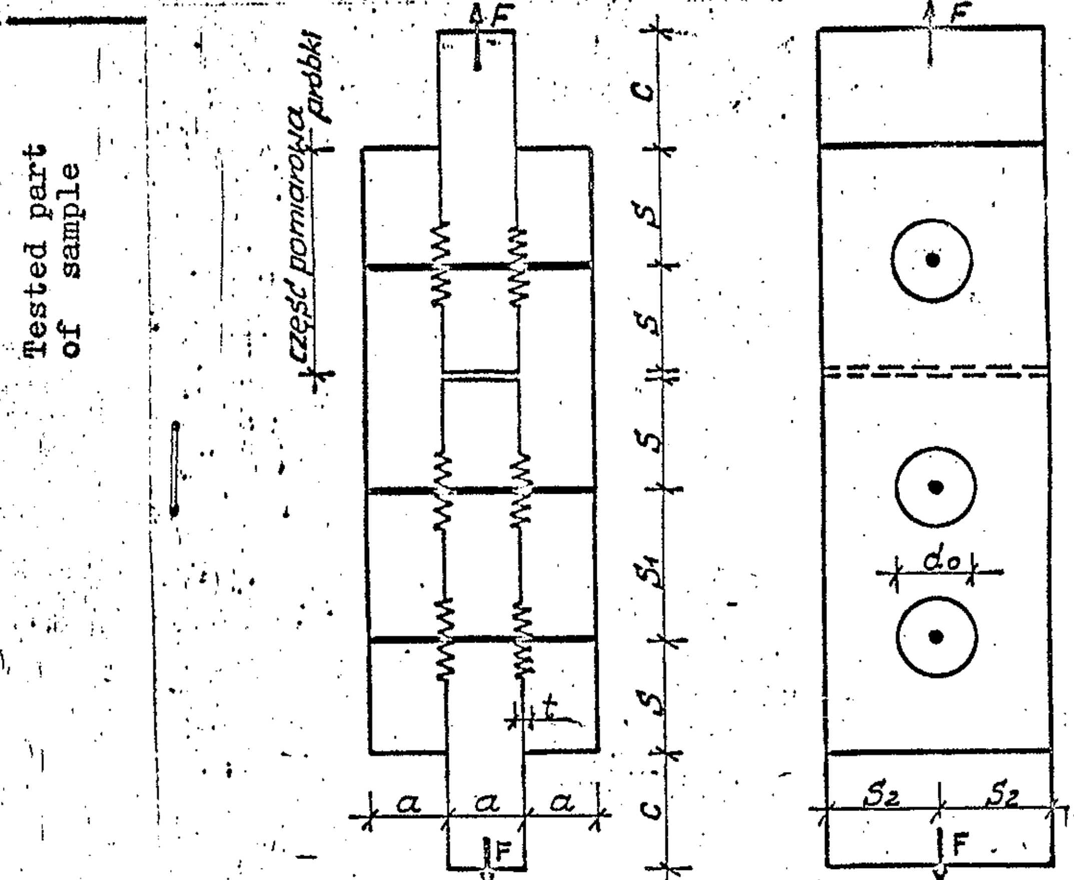

Fig. 1 Sample for determination of load carrying ability of joints

Table 1 . Nominal dimensions of joints

Designations as on fig. 1	a	s	s_1	s_2
values of above designations	3t 6cm	1,5 d_o	2d_o	$0,5d_o+t$ $0,5d_o+2,5$

Size of screws and washers for determined external diameter of rings shall comply with PN-73/B-03150.

2.4.2. <u>Test method, test procedure, test results</u> - acc. BN-80/5059-04 , sheet 01, clause 2.4.

2.5. <u>Determination of slips of jointed members</u> - acc. BN-80/5059-04 , sheet 01, clause 2.5. Static arrangement for this test acc. clause 2.4.1. of this standard.

2.6. <u>Determination of the effect of loading time</u> - acc. BN-80/5059-04, sheet 01, clause 2.6. Static arrangement - according to this sheet of the standard, clause 2.

2.7. <u>Determination of the effect of connectors number</u>

2.7.1. <u>Static arrangement</u> - as on fig. 2.

2.7.2. <u>Sample</u> - as on fig. 2 under following conditions:

a/ types of samples differs from each other by number of group of rings in the tested part of the sample - p ,

b/ spacing of rings axis and distances between rings axis and edges of samples shall be as on table 1,

c/ minimum number of types of samples being tested shall be of 3.

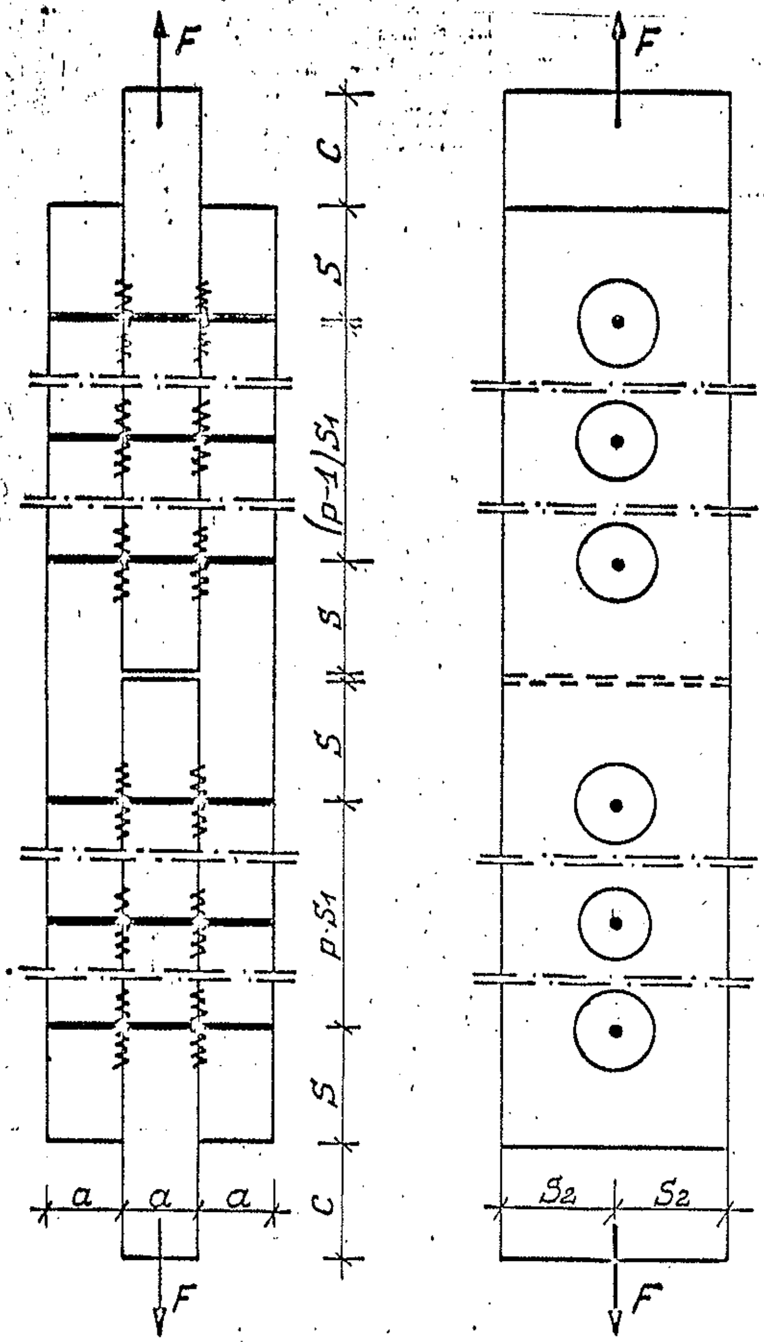

Fig. 2 . Sample for determination of the effect of connectors number

2.7.3. <u>Method of test</u> - acc. BN-80/5059-04,sheet 01,clause 2.7.3.

2.7.4. <u>Test procedure</u> - acc. BN-80/5059-04,sheet 01,clause 2.7.4.

2.7.5. <u>Test results</u> -acc. BN-80/5059-04, sheet 01,clause 2.7.5.

d/ factor of load carrying ability variation - m_{o1} - acc. BN-80/5059-04,sheet 01, clause 2.7.5.

2.8. Additional tests .As an additional test the load carrying ability of rings in direction perpendicular to grain shall be determined.

2.8.1. Static arrangement. Static arrangement for this test, dimensions of sample, spacing of rings axis and distances between ring axis and edges of sample shall be as those on fig.3 and on table 1. The screws shall be placed in the holes drilled previously acc. 2.4.1

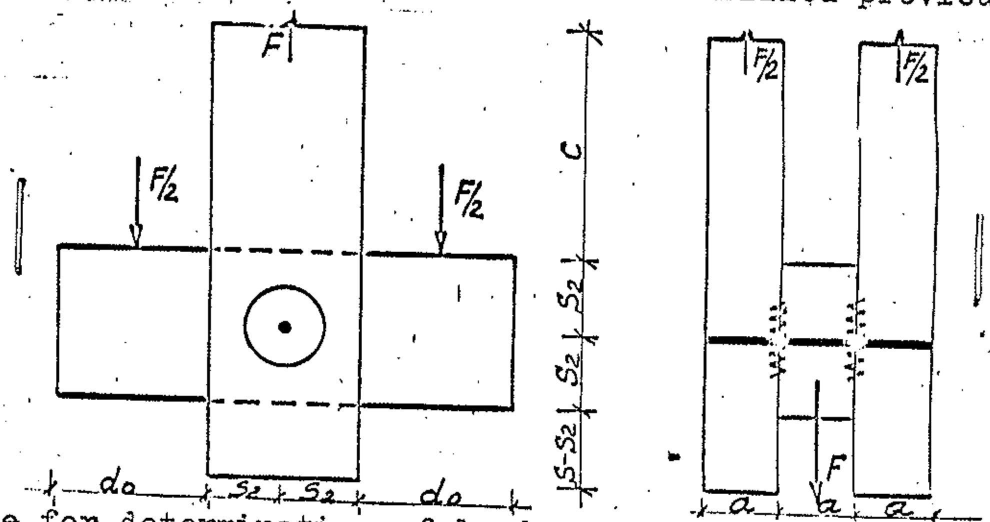

Fig.3 Sample for determination of load carrying ability of rings in direction perpendicular to grain.

2.8.2. Method of test -acc. BN-80/5059-04,sheet 01,clause 2.4.2

2.8.3. Test procedure- acc. BN-80/5059-04,sheet 01,clause 2.4.3

2.8.4. Test results - acc. BN-80/5059-04,sheet 01,clause 2.4.4. and besides is necessary:

d/ determine of m_{o3} value from the formulae:

$m_{o3} = 1,0$ for direction of action paralel to grain of joint,

$m_{o3} = \frac{T_{nsa}}{T_{ns}}$ for direction of action perpendicular to grain,

where:

T_{nsa} - mean value of T_{ns} calculated from the formula /4/ BN-80/5059-04 ,sheet 01,clause 2.8.

T_{ns} - value of T_{ns} calculated on the basis of test results acc. 2.4.

ADDITIONAL INFORMATIONS

1. This standard has been prepared by: Building Joinery Research and Development Centre, in Wołomin, Poland.

2. Authors of draft:

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WORKING COMMISSION W18 - TIMBER STRUCTURES

INVESTIGATION OF THE EFFECT OF NUMBER OF NAILS IN A JOINT ON ITS LOAD CARRYING ABILITY

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1. INVESTIGATION ON THE EFFECT OF NAILS NUMBER IN A JOINT ON ITS LOAD CARRYING ABILITY

1.1. Description of samples

The investigation of the effect of number of nails in a joint on its load carrying ability has been carried out in Poland for the first time. The investigation involved symetrical joints with GO and GTP nails 35/90 [1/]. The forms and dimensions of joints and arrangement of triangular cross-section nails in respect with load direction are illustrated on fig. 1.1.

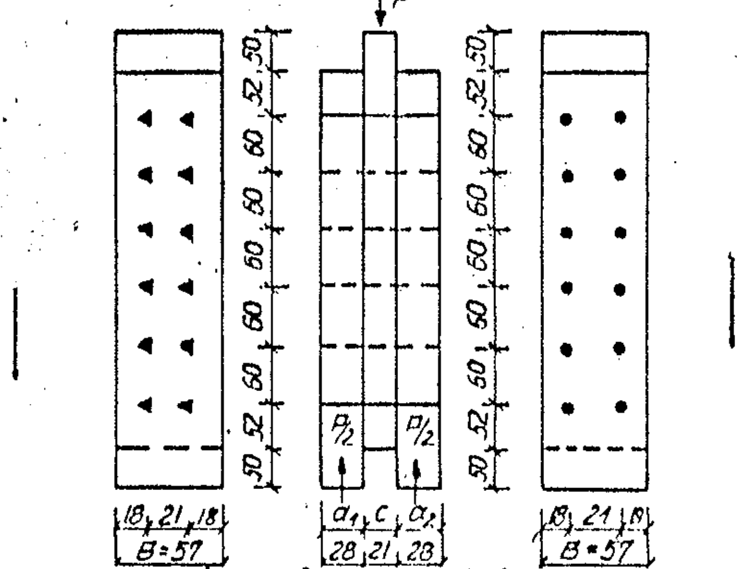


Fig. 1.1. Forms and dimensions of joints - in mm - for investigation of the effect on number of nails in a joint on its load carrying ability. Static arrangement.

1/ GO - nails having round cross-sections,
GTP - nails having triangular cross-sections,
35/90 - 10 /diameter/ / nails lenght, mm.

The study involved joints having the following number of rows of nails:

a/ one row of nails / 2 nails/,

b/ four rows of nails /8 nails/,

c/ six rows of nails /12 nails/,

d/ eight rows of nails /16 nails/.

For each test 20 joints have been prepared.

1.2. Designation of joints

Every joint has had a symbol involving:

- OST and TSI,
- 35 ,
- A,
- N , where N = 1 - 8 ,
- successive number of joint.

Where T means an investigation of the effect of nails number in a joint on its load-carrying ability; A - type of joint, N - number of rows of nails; O - nails having round cross-sections, T - nails having triangular cross-sections, S -symetrical joint.

1.3. Method of testing

Joints have been submitted to a compressive load action and maximum bearing load was determined with an accuracy of 100 N. Number of samples for each test has been 20 for every type of joint. Before the test has started, all joints were seasoned in a climate having relative humidity of 65 ± 2% and temperature of 20 ± 2°C.

Water uptake of timber in joint has been of 12 - 15%. In the course of investigation 5 timber samples havee been taken from each serie in order to determine its compressive strength, which was found to be of 53 - 62 MPa.

Thickness of joint members has been measured using slide caliper having an accuracy of 0,1 mm. Static arrangement of joints is illustrated on fig. I.1.

10.4. <u>Test results</u>

The mean values of the thickness of joints are given on table I.1.

<u>Table 1,1.</u>
Mean values of thickness of joints members in mm.

Designation of joint serie	a_1	c	a_2
OSI-35-A1	28,06	20,64	28,01
OSI-35-A4	27,93	20,70	27,78
OSI-35-A6	27,67	20,71	27,72
OSI-35-A8	27,55	20,73	27,56
TSI-35-A1	28,03	20,67	28,07
TSI-35-A4	27,68	20,74	27,49
TSI-35-A6	27,70	20,70	27,70
TSI-35-A8	27,68	20,87	27,65

The test results related with load carrying ability of joints in dependance on a number of rows of nails are given on table 1.2.

Table 1.2.

Test results from the investigation of the effect of number of nails on load carrying ability of joints - in N

Designation	$\overline{X}$ kG ≈ 10N	s kG ≈ 10N	v
OSI-35-A1	721	93	12,9
OSI-35-A4	2638	340	12,9
OSI-35-A6	3679	240	6,5
OSI-35-A8	4956	846	17,1
TSI-35-A1	790	82	10,4
TSI-35-A4	2925	281	9,6
TSI-35-A6	4121	407	9,9
TSI-35-A8	5155	933	18,1

where: $\overline{X}$ - mean value of load carrying ability of joints

s - standard deviation.

1.3. Conclusion about behaviour of joint during tests

The behaviour of joints has been observed in order to establish the character of joint failure. Three characteristic types of failure have been observed:

- slip of joint insert in relation with its covering plates without increment of load , as a result of maximum pressure stresses under nails being reached and bending of nails,
- characteristic failure of an insert as a result of surpassing of compressive stresses /fig. 1.2./,
- fracture of one of joints members along the line of nails row.

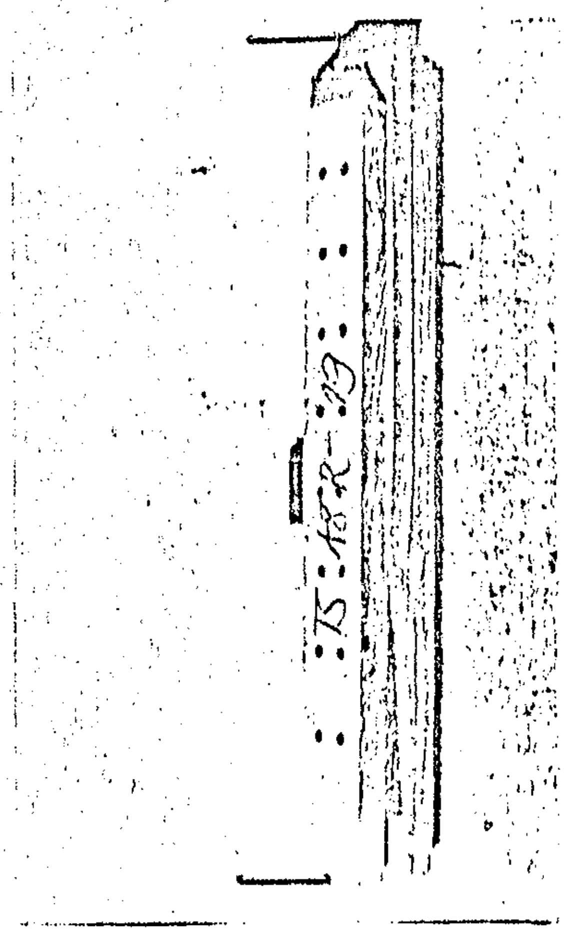


Fig.1.2. Characteristic failure of joint when maximum compressive stresses are surpassed.

1.6. Discussion of test results

In order to analyse the test results, load carrying ability of nails per one punched hole - Ng - in dependence of number of rows of nails in joints has been calculated from the formula

$$Ng = \frac{N}{k} \qquad 1.1.$$

where: k - number of holes punched by nails in a joint

$$K = 2p_1 = 4x \qquad 1.2.$$

where: p_1 - number of nails in a joint,

x - number of nails rows in a joint.

Ng values taken as means for each serie, are given in table 1.3.

Table 1.3.

Load carrying ability of a nail per 1 punched hole - Ng - and safety factor of joints - n - in dependence on number of nails rows in a joint

Designation of joints	Ng Kg = 10N	Ng, Kg = 10N $\overline{X} - S < m < \overline{X} + S$	n	Ng, n %
OSI-35-A1	180,0	169-191,5	4,83	100
OSI-35-A4	164,8	135-175	4,42	91,5
OSI-35-A6	153,2	149-158	4,11	85,2
OSI-35-A8	154,9	143-167	4,15	86,0
TSI-35-A1	197,5	188-207	5,60	100
TSI-35-A4	182,8	175-191	4,91	92,6
TSI-35-A6	171,8	164-180	4,86	87,0
TSI-35-A8	161,2	149-173	4,56	81,8

After calculating Ng, safety factor - n- for joints has been also derived, from the formula:

$$n = \frac{N}{T}$$

where:

N - $\overline{X}$ from table 1.2.

T - the minimum value T_d, T_a, T_c

T_d - k . 300 d^2 8.4.

T_a - k . 50 a d 8.5.

T_c - k . 40 c d 8.6.

d – diameter of a nail / for nails having triangular cross-sections – height of a triangle/

a = a_1 or a_2

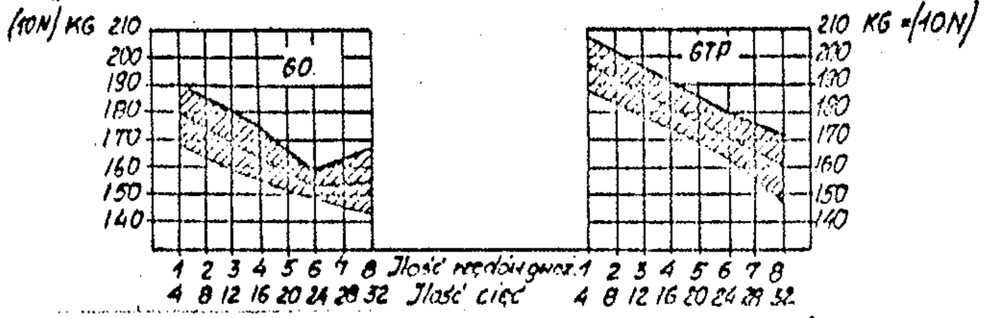
The values of safety factor are given on table 1.3., which also includes m range of means of Ng, where:

$$\mathcal{E} = \frac{t_{\alpha} \cdot s}{\sqrt{n}}$$

where: t_{α} – 1,96

s – standard deviation.

The relationship between Ng and number of rows of nails is represented on fig. 1.3. /considering also m range/. On diagram 1.3. a „cavity" on the dotted line for six rows of GO nails is shown. The cavity is due to a very little coefficient of variation, v for serie OST-35-A6 is 6,5% and for serie OST-35-A8 is 17,1%. In the test on serie TST coefficient of variation v has been not so reduced, and for this reason the line corresponding GPT nails is almost a straight one.



Number of rows of nails
Number of punched holes

Fig. 1.3. Relationship between Ng and number of rows of nails in a joint.

From the table 1.3. comparative coefficients have been calculated / ϱ_i , ϱ_n / from the formulae:

$$\varrho_i = \frac{N_g^{GTP}}{N_g^{GO}} \qquad \varrho_n = \frac{n^{GTP}}{n^{GO}} \quad . \qquad 1.3.$$

The values derived are given on table 1.4., which includes also a decrement of n value per 1 row of GO and GTP nails.

Table 1.4.

ϱ_t and ϱ_n and decrement of n value per 1 row of nails in relation with nails rows

Designation of joints	ϱ_t	ϱ_n	Designation of joints	Decrement of n value per 1 row of nails	Designation of joints	Decrement of n value per 1 row of nails
SI-35-A1	1,096	1,158	O-A1	0	T-A1	0
SI-35-A4	1,108	1,111	O-A4	0,0212	T-A4	0,0185
SI-35-A6	1,121	1,182	O-A6	0,0247	T-A6	0,0216
SI-35-A8	1,041	1,100	O-A8	0,0256	T-A8	0,0222

From the table 1.3. one can deduce that the nails GTP are more convenient for nailed joints in structural members than GO nails. Their load carrying ability per 1 punched hole is higher than the load carrying ability of GO nails per 1 punched hole, in 10%, and the respective safety factor is higher in 13%. Load carrying ability decrement per 1 punched hole in dependence on number of rows of nails in a joint is lower for GTP nails than GO ones, as one can see from table 1.4.

From diagram 1.4. one can deduce that load carrying ability decrement per 1 punched hole is linear and that load carrying ability of a nail per 1 punched hole is in inverse proportion to a number of rows of nails. Another conclusion that can be drawn from table 1.4. is that an additional row of nails in a joint shall decrease the load carrying ability of a nail per 1 punched hole in approximately 2% and the safety factor of a joint shall decrease in the same proportion.

For this very reason a decrement of load carrying ability of nails per 1 punched hole in dependence on number of nails rows shall be considered whilst designing a nailed joint. The value of correction factor - p - shall be determined by separate tests, but before performing them , this factor can be derived from the formula:

$$p = 0{,}02\ /z - 5/ \qquad 1.4.$$

where: $z > 5$ number of rows of nails in a joint,

0,02 - decrement of load carrying ability of nails per 1 row of nails in a joints, per 1 punched hole

1.7. Conclusions

1. Load carrying ability per 1 punched hole of GTP nails is higher than load carrying ability per 1 punched hole of GO nails in 10%.
2. Load carrying ability per 1 punched hole shall decrease with the increment of number of rows of nails in 2% per 1 row.
3. Factor of safety of joints for GTPnails is higher than corresponding safety factor for GO nails in general in 13%.
4. Factor of safety shall decrease with the increment of number of rows of nails in 2% per 1 row / for GTP nails in 1,9%, for GO nails in 2,3%/.
5. While designing a nailed joint factor p shall be introduced which can be calculated from the formula 1.4.

2. TESTS ON JOINTS UNDER LONG-TERM LOAD

2.1. Description of samples

The tests on nailed joints under long-term load have been performed in Poland for the first time. The tests involved symetrical joints having forms and dimensions as given on fig.8.4. and on table 2.1. with GO 35/90 and GTP nails.

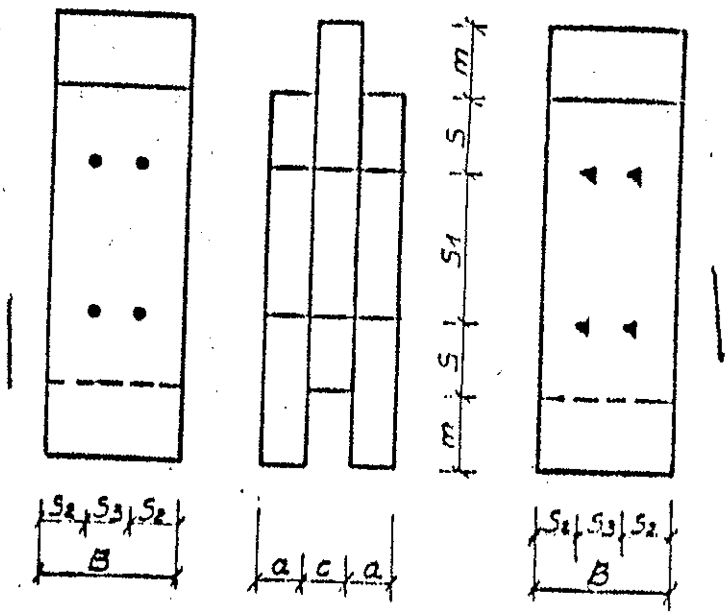


Table 2.1.

Designation and dimensions of symetrical joints for testing under long-term loads

Designation of samples	Dimensions of symetrical joints as in 8.4.							
	$a_{1,2}$	c	B	s	s_1	s_2	S_3	m
TA OA	28	17,5	57	52	60	18	21	60
TB OB	28	21	57	52	60	18	21	60
TC OC	28	24,5	57	52	60	18	21	60
TD OD	28	28	57	52	60	18	21	60

For testing four types of joints have been taken having inserts of different thickness /table 2.1./, three for each thickness /total number of joints - 24/.

2.2. Method of testing

Joints have been submitted to tensile load , as on fig.2.1. and slip of their members was measured after determined time under load.

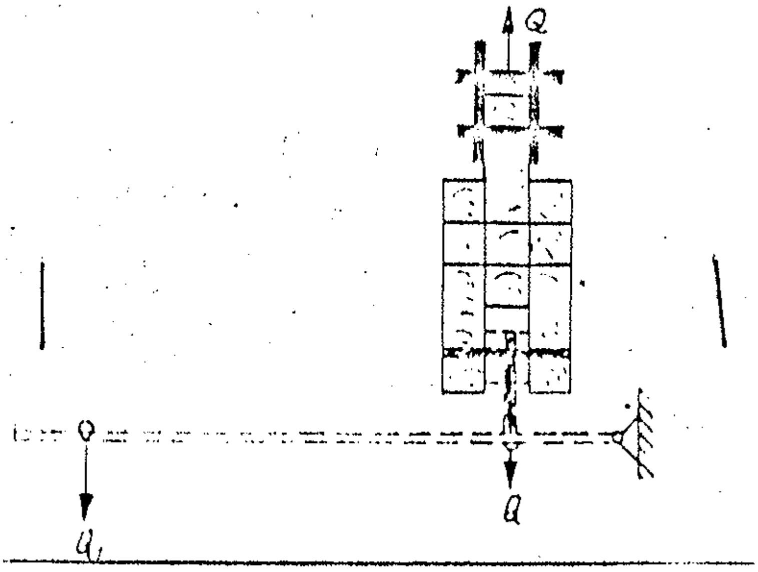


Fig. 2.1. Static diagram for testing joints under long-term load

For loading $/Q_1$ - on fig. 2.1./ metallic weights have been used from 0,20 up to 20 Kg /10N/. The load has been transmitted to joint by means of lever arm having leverage of 1:10 as on fig. 2.1.

The slip of members of joints , one in relation to the other, has been measured with dial gauges having an accuracy of 0,01 mm. To every joint two dial gauges have been attached. The loads applied, having in consideration the holding down the central member of joint, have been calculated from formula 8,6. and are given in table 2.2.

Table 2.2.

Loads applied to joints in Kg /1 Kg = 10 N/

Designation of joints	Loads Kg /10N/
TA	196
OA	196
TB	235
OB	235
TC	275
OC	275
TD	314
OD	314

Time of testing has been approximately of 3 months. During the tests the following climate has been mantained: relative humidity of 63% and temperature of 20%.

2.3. Test results

Slip values - s - for members of joints have been obtained as a function of loading time. Mean values of slip for three joints in relation with loading time are given on table 2.3. From the table 2.3. the rate of deformation of joints in time - v - has been derived using the formula:

$$v = \frac{u_t}{t} \qquad 2.1.$$

where: u_t - slip of members of joints after time ,t'- in mm

t - loading time /days/.

Table 2.3.

Mean values of slips of joints members in relation with loading time

Loading time days	Mean values of slips of joints members in mm							
	CA	TA	OB	TB	OC	TC	OD	TD
1	0,028	0,017	0,042	0,015	0,065	0,056	0,073	0,055
2	0,036	0,022	0,048	0,023	0,070	0,062	0,082	0,063
3	0,037	0,027	0,052	0,031	0,075	0,064	0,088	0,069
4	0,041	0,030	0,062	0,036	0,077	0,067	0,094	0,072
6	0,046	0,038	0,070	0,053	0,080	0,072	0,105	0,078
7	0,049	0,0435	0,079	0,0615	0,084	0,077	0,110	0,080
8	0,052	0,047	0,086	0,068	0,088	0,080	0,114	0,082
9	0,056	0,049	0,089	0,070	0,092	0,083	0,120	0,084
10	0,060	0,052	0,090	0,073	0,095	0,090	0,125	0,088
15	0,092	0,063	0,114	0,102	0,111	0,1125	0,1515	0,108
20	0,100	0,065	0,118	0,105	0,115	0,118	0,168	0,118
25	0,108	0,065	0,125	0,110	0,125	0,120	0,170	0,122
30	0,140	0,095	0,148	0,1155	0,132	0,126	0,177	0,128
40	0,160	0,145	0,182	0,138	0,142	0,133	0,192	0,142
50	0,165	0,150	0,189	0,145	0,155	0,138	0,195	0,144
60	0,167	0,158	0,190	0,149	0,160	0,140	0,198	0,150
70	0,168	0,158	0,198	0,150	0,164	0,140	0,200	0,152
80	0,168	0,160	0,205	0,152	0,169	0,143	0,208	0,158
90	0,170	0,160	0,213	0,155	0,169	0,145	0,216	0,159
100	0,172	0,162	0,220	0,157	0,170	0,148	0,220	0,160

The derived v values are given on table 2.4. and are illustrated as a curves in double logarithmic scale on fig. 2.3.
The equation of straight line in double logarithmic scale is as follows:

$$\log v = \Psi \log t + \log C$$

where: v - mean rate of deformation of joints in mm/24 h

Ψ - directivity factor of straight line,

c - rate of deformation - v - after one day of loading.

From formula 2.3. directivity factors have been calculated as well as c values, which are given on table 2.5.

Table 2.3.

Ψ and c values

Designation of joints	Ψ	c
TA	-0,52	0,017
OA	-0,54	0,024
TB	-0,60	0,026
OB	-0,62	0,039
TC	-0,75	0,050
OC	-0,76	0,060
TD	-0,74	0,051
OD	-0,75	0,073

Table 2.4.

Deformation rate of joints under long-term load in mm/24h

P - Kg	196	196	235	235	275	275	314	314
a	28	28	28	28	28	28	28	28
c	17,5	17,5	21	21	24,5	24,5	28	28
Designation	OA	TA	OB	TB	TC	OC	OD	TD
D a y s								
1	0,02800	0,01700	0,0420	0,0150	0,065	0,0560	0,0730	0,0550
2	0,01800	0,01100	0,0240	0,0115	0,035	0,0310	0,0410	0,0315
3	0,01230	0,00900	0,0173	0,0105	0,025	0,0213	0,0293	0,0230
4	0,01050	0,00750	0,0155	0,0090	0,0192	0,0167	0,0235	0,0180
5	0,00784	0,00634	0,0117	0,00884	0,0133	0,0120	0,0175	0,0130
7	0,00715	0,00622	0,0113	0,0088	0,0120	0,0110	0,0157	0,0114
8	0,00663	0,0059	0,0108	0,0085	0,0110	0,0100	0,0143	0,0102
	0,00622	0,00545	0,0099	0,0078	0,0102	0,00945	0,0133	0,0093
10	0,00600	0,00520	0,0090	0,0073	0,0095	0,00900	0,0125	0,0088
15	0,00613	0,00420	0,0076	0,0063	0,0074	0,00750	0,0101	0,0072
20	0,00500	0,00325	0,0053	0,00523	0,00575	0,00350	0,0084	0,0039
25	0,0043[illegible]	0,00260	0,0050	0,0044	0,0050	0,0048	0,0068	0,0049
30	0,00467	0,00316	0,00494	0,00385	0,0044	0,0042	0,0059	0,00427
40	0,004[illegible]	0,00363	0,00455	0,00345	0,00355	0,00332	0,0048	0,00355
50	0,00330	0,00300	0,00378	0,0029	0,00310	0,00277	0,0039	0,00288
60	0,00280	0,00263	0,00318	0,00243	0,00267	0,00233	0,0033	0,00250
70	0,00240	0,00226	0,00283	0,00214	0,00234	0,00200	0,00285	0,00217
80	0,00210	0,00200	0,00236	0,0019	0,00211	0,00179	0,0026	0,00198
90	0,00189	0,00178	0,00237	0,00172	0,00188	0,00161	0,0024	0,00177
100	0,00172	0,00162	0,00220	0,00157	0,00170	0,00148	0,0022	0,00160

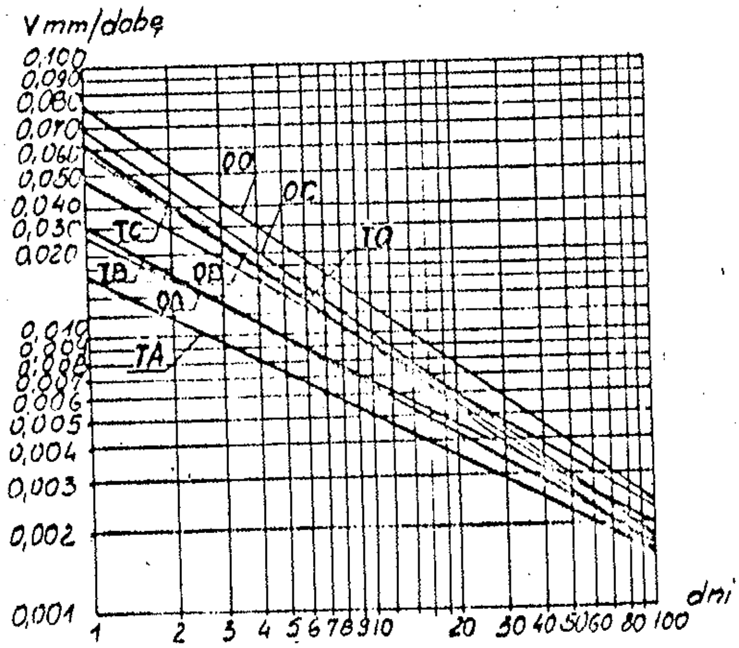


Fig. 2.3. Functions : Loading time - mean deformation rate for joints

2.4. Assessment of test results

During the tests on nailed joints with GO and GTP nails the relationship between deformation rate, e.g. slip rate of members of joints, and load value has been determined. This relationship is expressed by the formula 2.2. The mean deformation rate of joints shall be derived from the formula:

$$v = C\, t \tag{2.3.}$$

$$u_t = v \cdot t + u_o = c\, t^{1+\psi} + u_o \tag{2.4.}$$

where: u_t - deformation of joints after „t" time , mm,

u_o - initial deformation, mm.

Using the above formula deformations of joints have been calculated after 10, 30 and 50 years of loading, which are given on table 2.6.

Table 2.6.

Deformation of joints under long-term load after a period of 10 , 30 and 50 years

Designation of joints	$u_{t_{10}}$ mm	$u_{t_{30}}$ mm	$u_{t_{50}}$ mm
TA	0,889	1,494	1,8[illegible]5
OA	1,044	1,731	2,134
TB	0,718	1,099	1,313
OB	0,919	1,367	1,628
TC	0,438	0,565	0,623
OC	0,490	0,619	0,684
TD	0,481	0,624	0,681
OD	0,640	0,820	0,910

On the basis of table 2.6. comparisons have been made between joints with GTP and GO nails using the formula:

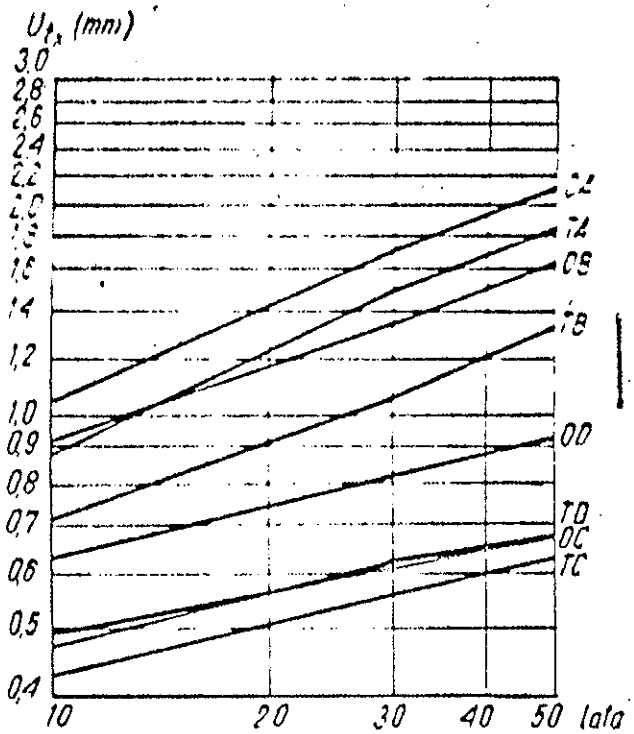
$$\varrho_u = \frac{u_{tx}^{GTP}}{u_{tx}^{GO}} \qquad 2.5.$$

The results of comparisons are given in table 2.7.

Table 2.7.

u-comparative factors

Designation of joint	ϱu_{t10}	ϱu_{t30}	ϱu_{t50}
A	0,851	0,862	0,864
B	0,780	0,804	0,807
C	0,895	0,912	0,911
D	0,752	0,761	0,748



years

Fig.2.4. Deformation of joints under long-term load

On fig. 2.4 the functions loading-time - deformation of joints are represented, calculated from the fomula 2.4.

After comparing joints after 10 up to 50 years of loading GTP - 2.7. - one can deduce that nails are more suitable for joints than GO nails, considering their lower deformation in 10-25%.

From the graphics on fig. 2.4. one can see that certain types of joints shall reach the allowable deformation of 1,5 mm after 25 years of loading, while others shall do not reach it even after 50 years. Considering this it would be appropiate to increase the load carrying ability of nails per punched hole, increasing all the same the load carrying ability of joints, considering besides the service time of structure in design. This problem deserves further investigation.

2.5. Conclusions

1. In wooden structural nailed joints submitted to a long-term load an increasing rate of deformation is continuously induced.
2. The deformation rate of nailed joints under long-term load depends on load value and loading time. It is in direct proportion to a loading time.
3. The rheological deformations of nailed joints depend on type of nails. Rheological deformations of joints with GTP nails are at least in 10% smaller than those of joints with GO nails.
4. GTP nails are more suitable for wooden joints in structural members even under long-term load , than GO nails.
5. Rheological phenomena related with nailed joints under long-term load, deserve further investigation in order to determine the design load carrying ability of joints /load carrying ability of nails per 1 punched hole/ or derive appropiate formulae for these phenomena, to be used in design procedure of wooden

structures.

6. Study on the effect of service time of joints under long-term load on safety factors in order to introduce them to design procedure in relation with service time of structure.
7. It shall be useful to introduce the service time in design procedure of wooden structures.

CIB-W18/13-7-3

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International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber

by

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OTANIEMI, FINLAND
JUNE 1980

SWEDISH FOREST PRODUCTS
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1980-02-19

INTERNATIONAL ACCEPTANCE OF MANUFACTURE, MARKING AND CONTROL OF FINGER-JOINTED STRUCTURAL TIMBER

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Introduction

Building is subject to regulations and codes in most countries. The codes also concern building material and components. Divergences between national codes may cause significant obstacles to the trade. Considerable efforts are made on various levels to harmonize rules. Such harmonization or standardization is particularly important for material and components traded as such or integrated in constructions.

Structural timber is the example dealt with here. It is still a material but the demands for an improved classification with respect to properties, as well as the trend against an extended initial end-use adaption, have made international agreements on this product more necessary than ever.

Although it is appreciated that such agreements ultimately are bilateral, they should be guided by international recommendations and standards. The agreement on a product will have to include

1) Definition and declaration of properties
2) Manufacture and control
3) Marking

The definition and declaration of properties should give any information of the products that the user needs. This is simplified if the product can be referred to classes in recognized classification systems, for example, rating in a series of structural grades. Marking is for identification. The mark itself seldom specifies the necessary properties, but is a code which by some record make such a specification possible.

The relation between the mark, the class and the properties (permitted stresses) is established by the code of practice in order to make it possible for the designer to prescribe grade.

The control has primarily the aim to guarantee that the product, subject to marking, will have - with stipulated confidence - the properties indicated by the mark. The rules for manufacturing and control concern the manufacturer, the inspecting agency and the authorities involved, but is hardly of any interest to the designer or even the builder or user.

Finger-joints for structural timber or finger-jointed structural timber

By the caption is made a distinction which may appear academic, but which in fact is of importance for the manufacturing rules, the control procedure and the marking. It is fundamentally a matter of responsibility and of administration of the control. First there is the product responsibility of the seller to the buyer. When a timber dealer sells structural timber to a builder, the latter will direct a claim for poor finger-joints or false grading to this dealer, no matter who has jointed respectively graded the timber. Naturally the dealer will claim backwards in his turn.

It is more complicated with the responsibility of a Building Authority (BA), a Control Board (CB), a Testing Agency (TA) and the Building Inspection (BI). All these parties may be involved the BA taking responsibility for the stresses in the Code of Practice, the CB for the administration and the TA for the performance of the control and the BI taking some responsibility by spot-checking that the timber is marked as specified. We may assume that the official authorization and manufacturing control have been committed either to a control

board or directly to a testing laboratory. This "Official Control Board" is a supplement to the Building Inspection for components the quality of which cannot easily be checked on the building site.
It is considered that the opposite party of the Control Board should primarily be that manufacturer which is the last to process the product in a way that the properties or the conditions for control are influenced. This manufacturer is authorized to use the official control sign in combination with (a code for) his name in the mark on the product. By making use of that right he will carry the chief responsibility to the CB for the product, that is, he will be responsible for the grading and the jointing as well as for planing and for a possible lamination. For those of the mentioned processes which he does not perform himself, his responsibility may well, by agreement with the CB, be limited to checking and recording official marks put on at a previous process by another manufacturer and, maybe, supervised by a different Control Board.

One advantage with such an integrated control is that the Building Authority directly knows from the mark where to start checking back on the crucial processes which led to the product. Also, the system of remarking is avoided. Such remarking is very open to mistakes, particularly when the remarking company buys his semi-manufactured timber from several different subcontractors, timber which may be graded or merely jointed or graded and jointed. This, and the fact that the user should preferably deal with jointed structural timber exactly as with unjointed timber, speaks for "finger-jointed structural timber" or just "structural timber" with one mark notifying grade (strength) and official control, both referring to wood grading as well as to the joints. However, other national systems of approval of control must be anticipated, by which grading and finger-jointing are dealt with completely separated.

. Standard measures of joints or performance requirements

The principal requirement of a joint in structural timber is that it shall not weaken the timber. This requirement is generally not met if it is applied on the part of the piece where the joint is. However, joints are not allowed in parts where natural defects have weakened the timber. The requirement is therefore that the joint must no weaken the timber to a strength lower than the strength stipulated for the grade of the unjointed timber.

It is not possible with present technics to join clean wood or straight grown timber of very high quality to the strength of the wood itself, that is, the joint efficiency will in this case always be less than unit. Low graded timber may be considered jointed with 100 percent efficiency if failure can be proved not to appear in the joints or close to them. Formally, one hundred percent efficiency can be claimed if the 5-percentile strength of a population of timber is not decreased by jointing. Although this criterion is applied in many countries to verify strength of joints, its significance is not clear without more precise definition.

If extended to all types of loads, climates and stresses the mentioned strength requirements constitute the performance requirements. As it is impossible to verify only by testing that they are continuously complied with, demands on the means and performance of the manufacturing have been introduced as a supplement to product testing. Further, there may be requirement of the design, that is, of one profile of the joint. Obviously some profiles are not suitable for all grades. Hence, there must be recommendations on joint-profiles for different structural grades and sometimes restrictions with respect to other end-use requirements.

Marking and remarking for information of properties, responsibility and control

In discussions on classification, codes, standards and quality control, the time spent on marking often seem to get out of proportion. Obviously there are many interests to look after, sometimes contradictionary. Here we will confine ourselves to such information which is necessary for the "user" receiving the timber, including manufacturers of prefabricated components, builders and the official control which represents the real "end-users", for example the owner of a house.

The usual case is that the user orders timber with properties in conformity with what is specified by the designer on a construction drawing. At receiving the timber he can check some properties visually, such as nominal dimensions, maybe species or species group, whether the timber is planed and if there are finger-joints. For other properties, like strength and structural classification he must rely on specifications in delivery notes or obtainable from marking.

New techniques of invisible or visible marking in code of timber pieces offers input of a practically unlimited amount of information individual for the piece, ready for output when needed for evaluation, grading or assigning to classes of a quantity of timber. For the near future and with respect to the need of a certain standardisation in the design of structures and particularly in the trade, we will remain at conservative marking.

By reference to acknowledged structural classes, defined by standardized claims on properties, the number of data transmitted by marks can be increased anyway. However, it is difficult to assign timber (and other material) of different kind to classes defined by "profiles" of several properties.

Classification systems for structural timber, which allow timber from different sources, of different species and graded by different methods to be assigned to the same class, have been discussed at international conferences for at least 30 years. Such systems (should preferably have been "such a system") are proposed in the CIB Timber engineering code and for British Standard. With an increasing interest for general classification, international approving of assignement to classes, marking rules and quality control will grow in importance. Standardisation of grading rules like the ECE recommendations, will facilitate such international co-operation considerably, but will not be vital.

The basic princips for marking are the same for ECE-graded timber and structural timber graded after other rules and the ECE-marking recommendations do not necessarily have to be changed. Still, the fact that they were developed separately for grading and finger-jointing make some improvement by co-ordination possible.

Integral and differential marking

It has already been indicated that one possibility is to mark separately for the grading and the finger-jointing. Another is to use a single mark for the product: Structural timber. Which alternative is chosen is much dependent on what the administrative routines of the quality control are. We will probably have to accept that international recommendations and standards for marking must allow the two alternatives and maybe combinations of them. They are here denoted SM = Separate Marking and IM = Integral Marking.

The ECE-standard stipulates that visually graded timber shall have the following information marked on one face (or for an interim period on edge or on end):

(G1) The company responsible for the grading
(G2) The grade of the piece
(G3) The species or species group
(G4) The control authority, where appropriate

Timber graded by machine shall have the same information and one supplementary information:

(G5) The licence number of the grading machine

The ECE-standard for finger-joints stipulates that each separate piece of timber shall be marked with:

(F1) The common ECE mark: FS10, FS8, FS6, FMS10, FMS8 or FMS6
(F2) The mark of the national approving and control authority
(F3) A mark to identify the manufacturer

We will here assume that the ECE-rules are based on the assumption that there should be two separate marks, the "G-mark" for grading and the "F-mark" for the finger-jointing". They give 5 + 3 = 8 basic informations. The user/builder will need three of them: The grade (G2) and the signs of the control authorities (G4), (G5). The building authority (aproving authority) will want to see the same three informations and, additionally, the codes of the respective manufacturers (G1) and (F3).

The ECE-grading was developed in order to standardize the grading performance. This is beneficial to grading companies which have to deal with many species. Unfortunately, the result in terms of structural rating might well deviate between species. In order to make it possible in practice to use the grading for structural (strength) rating, species or group of species (with approximately the same strength properties) must be specified. Thus, if the application of the ECE-grading standard is extended over a large range of species it is necessary to have sub-grades in terms of species group. If, for example, they will be applied on two groups of species, a grade S8 is divided into S8/1 and S8/2 and should be marked accordingly. From the users point of view it had been better to start with a classification system and assign the grades (possibly after adjustment of the grading rules) to the relevant classes. The grade mark (G2) should then be substituted by a class mark (for the user) plus a mark to indicate the grading rules applied, such as ECE or S for the ECE-rules, T for the T-rules, etc.

Three of the eight pieces of information - (G3), (G5) and (F1) - were not mentioned above as absolutely necessary. Specification of species in the marking of individual pieces is not generally adopted, but appears for example in UK. If species grouping with respect to strength properties is applied the grade-mark can be extended into a grade/species-group- (grade/subgrade-) mark. The information (G1) "The company responsible for the grading" should maybe be substituted by "The manufacturer and production line for which the use of the official control mark (G4) is issued". The licence number of the grading machine (G5) gives of course the production line. Still, without a date the value of this information at checking back on possible defects of the machine or its operation is limited. An appropriate grading code could be

Ψ MR/2 GW010-ECE MS8/2 (G)

saying that the piece was graded 1980, week no. 10 (GW010) by the manufacturer MR on his production line no. 2 (MR/2), by agreement subject to official approval and control and authorized to use the (official) control mark (Ψ). The licence number of the grading machine used on production line no. 2 will be on file at the Control Board. The machine has been set as stipulated for the ECE grade MS8 for species group 2.

If finger-jointed two weeks later by another manufacturer NN on line no. 1 approved and authorized by another control board to use the (official) control mark Φ the following code is added

-- ECE/FMS8/2- NN/1 W012 Φ (F)

Together the two codes, (G) + (F) give a rather extended mark. If we assume that the grade-notation S alwaysrefers to the ECE-grading rules we may delete the letters ECE. Let us also assume that the "sub-grading" by introducing species groups is not applied. In this case we can write the differential mark:

Ψ MR/2 GW010 - MS8--FMS8-NN/1 W012 (1)

or

Ψ MR (licence-number) GW10 - etc.

Of principal interest for the builder and inspector is the information Ψ, MS8 and Φ.

In this double mark (G) + (F) we have all the information stipulated by the ECE grading and fingerjointing standards but for the species or species group indication.

If the same manufacturer and the same approval/control body are responsible for the grading and the finger-jointing the following condensed mark could be used:

Ψ MR/2-MS8--FM S8 W012 Ψ (2)

or

Ψ MR/2-F MS8 - W012 (2a)

Reviewing the case where two different manufacturers are involved, that is, the mark (1) and (2), it is realized that in most cases the timber is planed immediately after finger-jointing.

In the planing any grading-code (G) is removed. It is then stipulated in the ECE Standard for Stress Grading that "the company carrying out the processing must reinstate the marking and, in addition, add its own identification mark". Actually, the information that the user/builder needs (grade and control board) he will find in the finger-jointing code (F) put on after the planing. The reason for the stipulation on reinstating the (G)-code thus must be to make it possible for the control-board to trace the grader. According to law the finger-jointing company will be primarily responsible to the buyer for the total product, including the grading, so he must also be able to trace the grading company. The finger-jointing/planing company (or merely planing company) can be a timber dealer or a terminal with many different suppliers, and several stamps must be kept up-to-date in store. The risk of mistakes in handling all these stamps and administrative and legal difficulties have resulted in doubt about remarking. The alternative is integral marking. In this case the approving board (control board) specifies that the finger-jointing/planing company keeps records by which the information, stipulated for the grading code, can be traced with the (F)-mark on the timber as starting point. Thus for example the following mark should be sufficient

Ψ MR/2 - FMS8 - W012

This is the mark already presented as (2a), but of generalized significance.

Ψ stands for approving of the product, including the control of the product. The product is structural sawn timber, which is graded, which can be unjointed or jointed and unplaned or planed. The right to use the approval/control sign (Ψ) is given the mentioned manufacturer (MR, production line no. 2) by an official, national institute or authority.

MR is the code for the manufacturer which carries out the last of the operations subject to control. It can be grading or jointing or planing.

FMS8 is the grade mark including the information that the timber is fingerjointed. (It might be claimed that the F can be deleted in the mark as this can be observed anyway, but there are reasons for including it).

W012 is the additional code necessary in order to produce from records the stipulated supplementary information, which is of particular interest when the product is claimed to be below standard. It is suggested that the standard code for the week of the last operation (together with the information on dimensions and grade) is applicable.

International co-operation on approval, certification and control

The evaluation of a building product may involve:

a. Description of the product, including quality.
b. Account of properties, including deviation.
c. Rules for manufacturing, including control rules.
d. Assessment for establishing compliance with regulations and other requirements.
e. Certification to standard.
f. Supervisory control
g. National administrative approval of the product, including the control, (a to f).
i. International adminstrative approval of the national approval, including the control, (g).

(a), (b), and (c) constitute a base for the assessment (d) and for the approval (g) - directly or indirectly through the assessment (d). Certification to standard (e) is principally an aid in establishing the compliance of the product "properties" (a, b, and c) with what is required for an approval, in other words, a method of simplifying the assessment which also gives less margin for judgement.

Although approval is a fairly general conception, it is here, with reference to fingerjointed structural wood, dealt with in the sense of an official administrative approval, applied by a national authority. The rules for grading and fingerjointing of structural timber concern the safety of buildings and it has been recommended in ECE "Guidelines for international recognition of acceptance certificates" that the ECE activities should concentrate, for the time being and as far as appropriate, on approvals to mandatory requirements. Obviously, international agreements on grading and fingerjointing of wood for other purposes, particularly joinery timber, are likewise important, but they are more a matter for the standard organizations (ISO).

It has been indicated in several ECE documents that the ultimate agreement on accepting approvals and quality control will in the foreseeable future be a concern of the countries, generally bilaterally and probably limited with respect to group of products. Something to look forward to would be "accepting without further formalities building materials and systems which are subject to an appropriate foreign approval confirmed by a national body". Governments were recommended to study the possibilities of such acceptance by the Fourth ECE Seminar on the Building Industry in its Policy Statement.

Thus, the ideal state, would be that an international approval (i), let be bilateral, is granted without an extensive debate about the conditions that qualified for the national approval. One condition is, of course, that a system of national administrative approvals and controls exists which is not always the case or they have a different status. This appears from the (draft) Directory on information centres and approval agencies made within the ECE/HBP Committee, Working Party on Building. The publication of the Directory has been delayed due to a political argument.

The progress towards the ideal state of more or less automatic mutual approvals between countries is fairly slow. However, it seems that an increased support from governments and authorities should be expected in future negotiations on approvals of products and quality control with reference to the work within ECE/HBP/WP.2.

It is appreciated that in order to facilitate agreements between countries, it is essential to know what data about the product and the national approval and control should be presented. Hence, application forms for approval of stress-graded and fingerjointed sawn softwood have been prepared within the ECE Timber Committee, Subgroup on Quality and Marking. The general acceptability of the proposed forms within the developing ideas of the ECE project on the international harmonization of approval and control rules for buildings and building products has been recognized by the Working Party on Building of the HBP Committee.

SWEDISH FOREST PRODUCTS
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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF JOINTS WITH NAIL PLATES - CALCULATION OF SLIP

by
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JUNE 1980

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SWEDISH FOREST PRODUCTS
RESEARCH LABORATORY

DESIGN OF JOINTS WITH NAIL PLATES - CALCULATION OF SLIP

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Introduction

The proposal of a Nordic group on the design of joints with nail plates in timber structures was presented in October 1979 as CIB-W18 paper No. 12-7-3. In that edition the subsection 2.6 Calculation of slip was considered incomplete and left out. The following draft for that subsection has not been finally adopted by the group, but it was decided that it should be presented for the CIB-W18 as a supplement to the mentioned principle paper.

2.6 Calculation of slip

2.61 Model

The slip between two jointed members (1 and 2) due to the force and moment transmitted is of two kinds: translation (δ) and rotation (θ). It is composed of the slip in the plate/timber joints (y_1 respectively y_2) and deformation in the plates between the grip areas and in the wood at contact between the members. The mentioned components of slip may be given separately or integrated into one value of total slip or correspondent slip modulus.

For calculation of slip in the servicability limit state the slip may generally be considered proportional to the force (moment), that is, a constant value of slip modulus is applied. The slip is modified with respect to combined action of force and moment.

2.62 Translation

The modulus for translation slip between the plate and the timber is defined by:

$$k = \tau/y \qquad (N/mm^3) \qquad (30)$$

where τ = F/A is the shear stress in the anchorage area (p/t-joint). The corresponding contribution to the mutual displacement between two jointed wood members (1 and 2) is

$$\delta = \tau_1/k_1 + \tau_2/k_2 \qquad (31)$$

Here $\tau_1 = F/A_1$ and $\tau_2 = F/A_2$ denote the shear stress in the plate (plates) and the respective wood members jointed[1].

The values of the moduli k_1 and k_2 are dependent on the angles between the directions of the force and the principle directions of the wood and the plates. However, in many cases it will give sufficiently accurate result if the approximation $k_1 = k_2 = k$ is introduced also when the connection is not symmetrical with respect to the (timber/ /timber-) joint. In the symmetrical case when $A_1 = A_2 = A$ and $\tau_1 = \tau_2 = \tau$ the mutual displacement of the members is

$$\delta = \frac{\tau}{k/2} = \frac{F/A}{k/2} \qquad (32)$$

Examples of displacement (translation and rotation) between jointed timbers are found in 2.65, Table 2.61.

2.63 Rotation

The modulus for rotation slip (θ) in the plate/timber joint is defined by

$$k_M = \tau_M/y = \frac{M/I_p}{y/r} = \frac{M}{\phi I_p} \qquad (\mathrm{N/mm^3}) \qquad (33)$$

---

1) In this supplement A (A_1, A_2) is denoting the total effective area of the plate/timber joint or joints available for transmitting the force from the plate or plates to the wood member. This is inconsistent to the principle paper in which A is used for the grip area of a single plate and 2A for the total grip area of two equal plates, one on each face of the timber. It is suggested that the denotation of the principle paper is changed.

In (33) γ denotes the tangential slip plate to wood at the distance r from the rotation centre (RC). I_p is the polar moment of inertia with respect to RC calculated from the total effective area of the (two) plate/timber joints available for transmitting the moment M.

The mutual rotation between two jointed wood members, when the deformation of the plates themselves is neglected, is the difference between the rotations of the respective members to the plate (plates):

$$\theta = \phi_1 - \phi_2 = \left(\frac{M}{k_M I_p}\right)_1 - \left(\frac{M}{k_M I_p}\right)_2 \qquad (34)$$

The moments (M_1 and M_2) in (34) may be defined <u>either</u> as moment (M_{CG}) in the centre of gravity of the grip area (plate/timber joint area) <u>or</u> as the moment (M_{RC}) in the centre of rotation. The distance between these two centres, CG and RC, depends on the relative dimensions of the force and the moment transmitted in the joint and their distribution on the plates respectively directly between the members. It seems logical to refer the torque and the polar moment of inertia to the same point (either CG or RC). Anyway, the modulus k_M may be used to some degree to compensate the approximation introduced by adopting CG as RC.

The value of the modulus for rotation, k_M, should be specified in the type approval of the plate. Examples of values for rotation and moduli are found in 2.65, Table 2.61.

2.64 Translation and rotation

When there is simultaneous action by a force and a moment in the point of the plate/timber joint to which the I_p in (33) is referred (for example the centre of gravity), the translation may still be calculated from 2.62, while the rotation ϕ_M calculated from 2.63 is modified by

$$\phi_{M,F} = \phi_M \, (1 + \gamma \, F/F_D) \qquad (35)$$

The value of the factor γ is given with respect to the type of plate and joint. The value γ = 1.5 has been propsed for symmetrical splices for which common types of plates are used, see 2.65.

2.65 Symmetrical joints for splicing

2.65.1 General

There are different proposals for application of (33) and (34) in 2.63, particularly when the angle generated by load is calculated for splices in beams or in the rafters and bottom chord of roof trusses. Such splices are in most cases of symmetrical design, Figure 2:7, and may be assumed symmetricaly loaded, that is, the contribution to the rotation by the shear force is neglected.

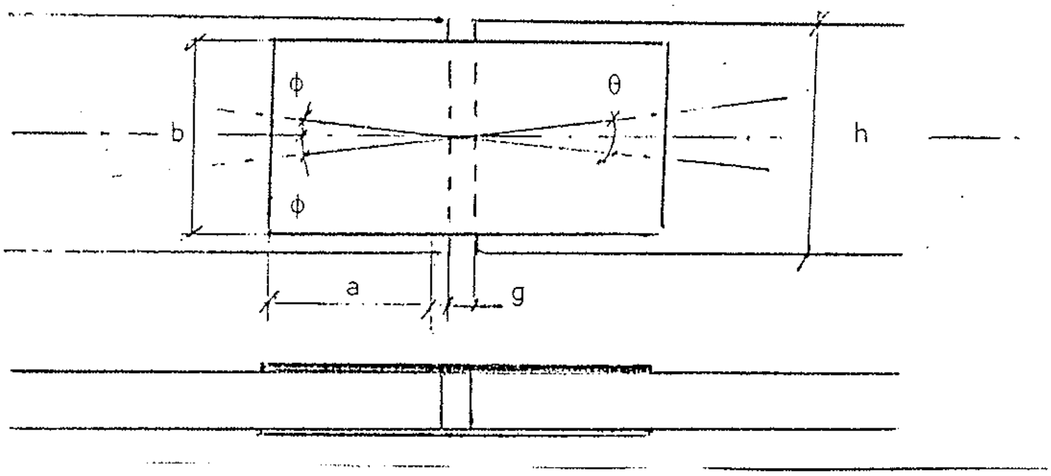

Figure 2:7 Symmetrical splicing with nail plates

The moment M and the axial force N generate an angle between the connected timber parts

$$\theta_{M,N} = \theta_g + \theta_M \ (1 + \gamma N/N_D) \tag{36}$$

Here θ_g is represents the part of the rotation generated when there is still a clearance between the parts, that is when the total load on the joint is carried by the plates. This part may be neglected when the joint is performed without gap or it may be included in the second term. N_D is the design value of the force N which appears along the centre line.

θ_M is the angle when N = 0 expressed by (33) which in this case (Figure 2:7) can be transformed to

$$\theta_M = 2\phi = \frac{M/A}{k_M/2} \cdot \frac{12}{d^2} \tag{37}$$

In (37) $A = 2ab$ denotes the total of the two pt-areas at each side of the joint and $d = \sqrt{a^2 + b^2}$ is the diagonal of the individual pt-area.

2.65.2 Approximations

The distance between the endsurfaces of the jointed timbers (δ_θ), to be compensated by the rotation θ, can be regarded as a total of the initial gap (g), a gap (positive or negative) generated by the normal force in the plates ($\delta_{N_p} = \delta_{N,M}$) and a "gap" which is equivalent to the compression of the wood (δ_c):

$$\delta_\theta = g + \delta_{N,M} + \delta_c \qquad (38)$$

$$\theta = \frac{\delta_\theta}{h/2} \qquad (39)$$

The following approximation can be used for $\delta_{N,M}$ at the serviceability limit state /4/ :

$$\delta_{N.M} = (N + \frac{M - M_D}{h/2}) \frac{2}{kA} \qquad (40)$$

M_D is the design value of the moment ($N = 0$). The values $M_D = 0.5\ M$ and $\delta_c = 0.3$ mm have been proposed as approximations /4/.

The method should be applicable for joints with a gap $g \leq 0.5$ mm.

An alternative method is examplified by the expressions in Table 2.61. It is based on (34) and a rotation by moment, expressed in the form

$$\theta_M = C \frac{h(h_g - h)}{3A} \cdot \frac{M}{M_D} \qquad (41)$$

It has been proposed /2/ that the ratio of the design values for moment and axial force in these joints (Figure 2:7) is approximated to $M_D/N_D = h/6$ in which case (41) can be transformed into

$$\theta_M = \frac{C}{N_D} (h_g - h)\ M/A \qquad (42)$$

The value of C/N_D is dependent on the type of plate used. The value of the h_g in the term $(h_g - h)$, which refers to a depth effect, has to be determined by experiment, cf. /2/.

Table 2.61 Translation δ and rotation θ in a symmetrical joint, Figure 2:7 ($y = \delta/2$ and $\phi = \theta/2$ between plates and respective timber).
The values refer to short term loading by a force N_a ($1.5\ N_a$) respectively a moment M_a ($1.5\ M_a$) transmitted in the joint. The value of N_a (M_a) is assumed to be 0.4 times the capacity (5-percentile strength). For long term loading, multiply the slip by a factor 2. The values are applicable for Gang-Nail gage 18, Hydro-Nail E, Structo-Nail T, Träförband T150 and other plates of similar type, and when the timber depth is limited to $125 \leq h \leq 225$ mm.

Level of force	Extension δ mm	Slip modulus k/2 N/mm^3
N_a 1,5 N_a	0,27 0,54	5,0 3,75
Level of moment	Mutual rotation θ at zero gap	Rotation modulus $k_M/2$ N/mm^3
M_a 1,5 M_a	(13-0,04 h) 10^{-5} M/A 1,4(13-0-04 h) 10^{-5} M/A	
Level of moment	Mutual rotation θ at 1,5 mm gap	
M_a	(26-0,04 h) 10^{-5} M/A or 18 · 10^{-5} M/A	5 500 · 12/d^2
1,5 M_a	25 · 10^{-5} M/A	4 000 · 12/d^2

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References:

/1/ Nordiska riktlinjer för träkonstruktioner, NKB-skrift Nr 13, Nov. 1970 (Nordic Recommendations for Structural Timber, NKB publication series, No. 13).

/2/ Edlund, G: Längdskarvning av träbalkar med spikplåtsförband. Byggforskningen Rapport R40:1971 (Longitudinal jointing of timber joists using nail plate connectors. National Swedish Building Research Report R40:1971).

/3/ Godkännanderegler nr 4 Spikplåtsförband (Rules for general approvals No. 4 Truss plate joints) Statens planverk 1974.

/4/ Feldborg, T & Johanson, M: Wood Trussed Rafter Design. SBI-report. Draft Feb-April 1980. Danish Building Research, Hørsholm.

SWEDISH FOREST PRODUCTS
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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF JOINTS WITH NAIL PLATES - THE HEEL JOINT

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JUNE 1980

DESIGN OF THE HEEL JOINT IN ROOF TRUSSES USING NAIL PLATES

Introduction

General design rules for joints with nail plates have been proposed in CIB-W18/12-7-3. The development of simple rules for specific joints were anticipated in section 2.5.

In connection with tests of roof trusses the author has investigated the heel design with respect to performance and calculation of the joints and their influence on modelling the truss for the static calculation. In this paper is presented a limited extraction from the principal report which has not yet been published /1/. The paper contains examples of design where the supports are eccentrically located in respect of the intersection of the upper and lower chords.

1 General

For the calculations of the joints is used a model based on plasticity theory or in agreement with the principal design rules proposed in the CIB-W18 paper. Simultaneously the use of a linear elastic model for the truss is accepted. This may not seem satisfactory but is part of a desirable simplification.

In the selected examples of design of the heel different assumptions on stiffness conditions are assumed. This appears from the shown model consisting of fictitious members connected either by hinges or stiffly. Semistiffness is simulated by the stiffness values ascribed to the members within the connection. Also the selected models are examples and do not exclude alternatives within one and the same connection design.

2 Simplification of design

At testing of roof trusses it is observed that the wood members are mutually displaced within the connection in a rather complicated manner. Hence, it is often difficult to state a force distribution which is both simple and realistic. Here a theory of plasticity is applied and it is in most cases assumed that the plates themselves do not transmit moment, that is, the stress in the plates in a section over the joint between the wood members is considered constant, see Figure 1. In some cases this section is assumed to be exposed to (constant) stress only partly. Corresponding conditions as to distribution are adopted for the compression stress between the wood members at contact. Friction between members is not taken in account.

In verifying that the stresses in the plate and in the wood members do not exceed the design values the general rules for design of joints proposed in the CIB-W18 paper are applied.

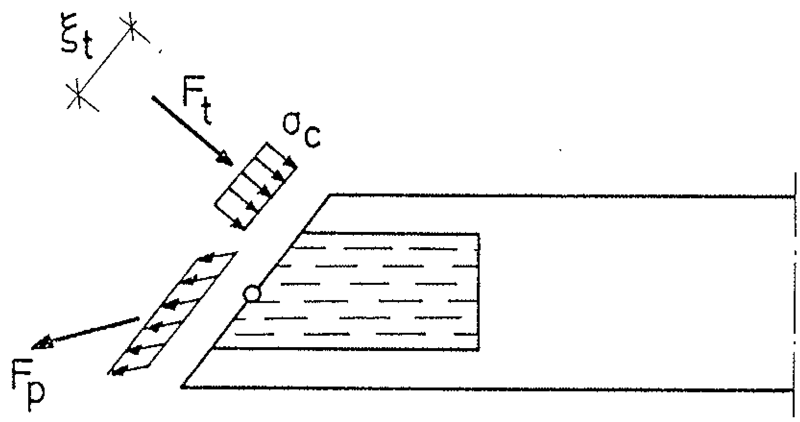

Figure 1

EXAMPLES

The following examples are provided with figures showing the design, a correspondig model, the forces transmitted between members, the distribution of forces on plates (F_p) and direct transmission between members (F_t), respectively the distribution of stresses.

The denotations appear from the figures. Suffix c is used to denote compression, t is for timber, p for plate and D is indicating design value.

The equations are derived from the equilibrium conditions and, when these are not sufficient, from stress conditions.

The difference in design of the examples should be clear from the figures.

Example 1 (Figure 2)

Equations of equilibrium

$$F_p = \frac{Q}{\cos(\alpha-\alpha_F)} \tag{7.4}$$

$$F_t = Q\ \mathrm{tg}(\alpha-\alpha_F) + N \tag{7.5}$$

$$e_t = \frac{M}{F_t} = \frac{M}{Q\ \mathrm{tg}(\alpha-\alpha_F) + N} \tag{7.6}$$

Design against plate failure

$$N_a = F_p \cos\alpha_F \tag{7.7}$$

$$N_b = F_p \sin\alpha_F \tag{7.8}$$

Design condition is

$$\left(\frac{F_p}{N_{aD}} \cos\alpha_F\right)^2 + \left(\frac{F_p}{N_{bD}} \sin\alpha_F\right)^2 \leq 1 \tag{7.9}$$

Design against wood failure

$$\sigma_c = \frac{F_t}{2(h_t-e_t)B} \leq f_c$$

where σ_c is the stress and f_c is the strength in compression perpendicular to the fibre direction. B denotes the thickness of the wood members.

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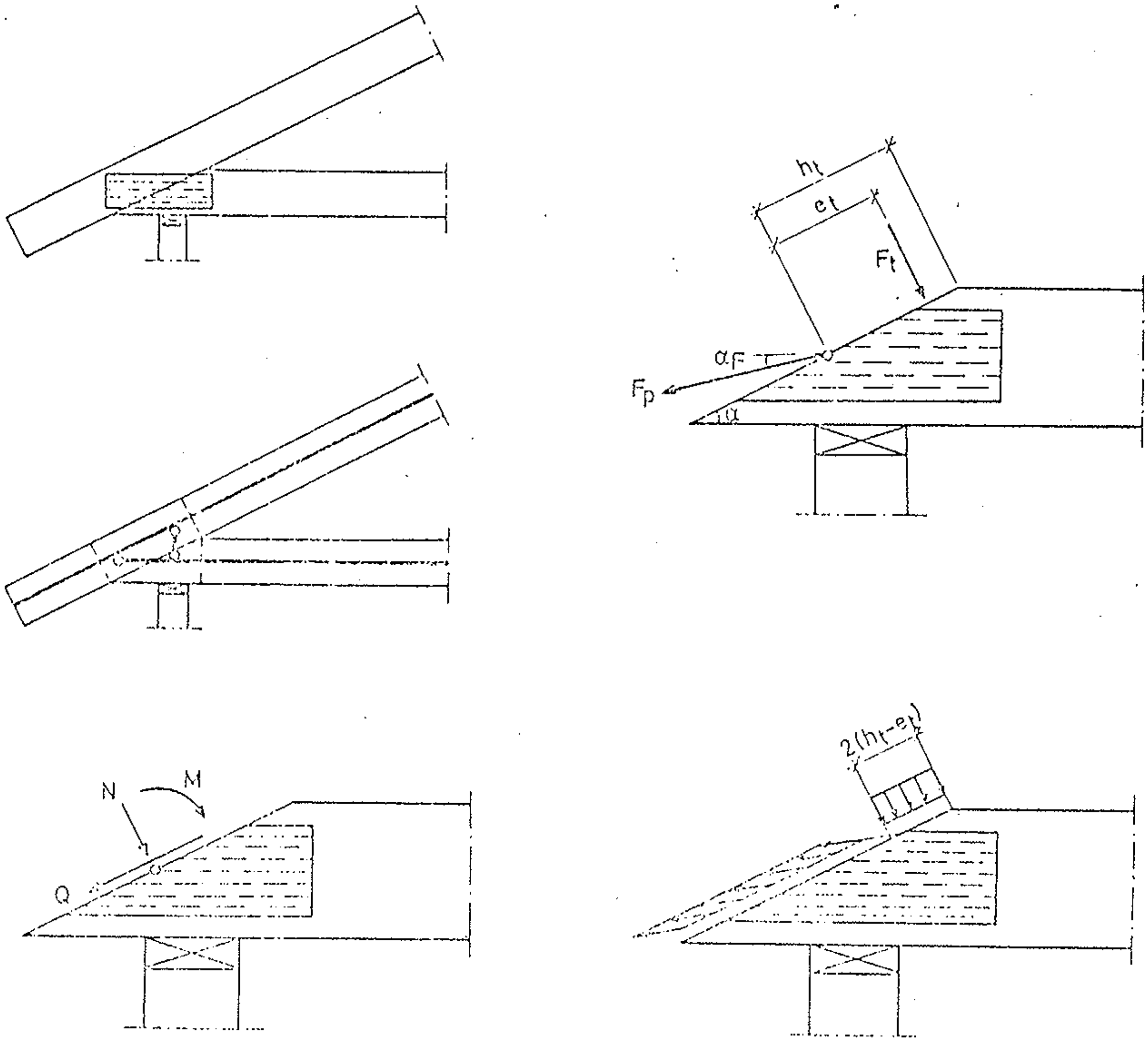

Figure 2

Example 2 (Figure 3)

Equilibrium equations

$$F_{p1} = \frac{H}{\cos\alpha_F} \tag{7.11}$$

$$F_v = V - Htg\alpha_F \tag{7.12}$$

$$F_t = F_v\cos\alpha = (V - Htg\alpha_F)\ \cos\alpha \tag{7.13}$$

$$F_{p2} = F_v\ \sin\alpha = (V - Htg\alpha_F)\ \sin\alpha \tag{7.14}$$

$$e = \frac{M}{F_t} = \frac{M}{(V - Htg\alpha_F)\ \cos\alpha} \tag{7.15}$$

Design against plate failure

The components of the force F_{p1} on the first plate are

$$N_a = H \tag{7.16}$$

$$N_b = H\ tg\ \alpha_F \tag{7.17}$$

The design condition is

$$\left(\frac{H}{N_{aD}}\right)^2 + \left(\frac{Htg\alpha_F}{N_{bD}}\right)^2 \leq 1 \tag{7.18}$$

The force on the second plate is

$$N_b = (V - Htg\ \alpha_F)\ \sin\alpha \tag{7.19}$$

Design condition is

$$\frac{(V - Htg\alpha_F)\sin\alpha}{N_{bD}} \le 1 \tag{7.20}$$

Design against wood failure

$$\sigma_c = \frac{(V - Htg\alpha_F)}{2(h_t - e_t)\cos\alpha\ B} \le f_c \tag{7.21}$$

where σ_c is the compression stress between the wedge and the bottom chord.

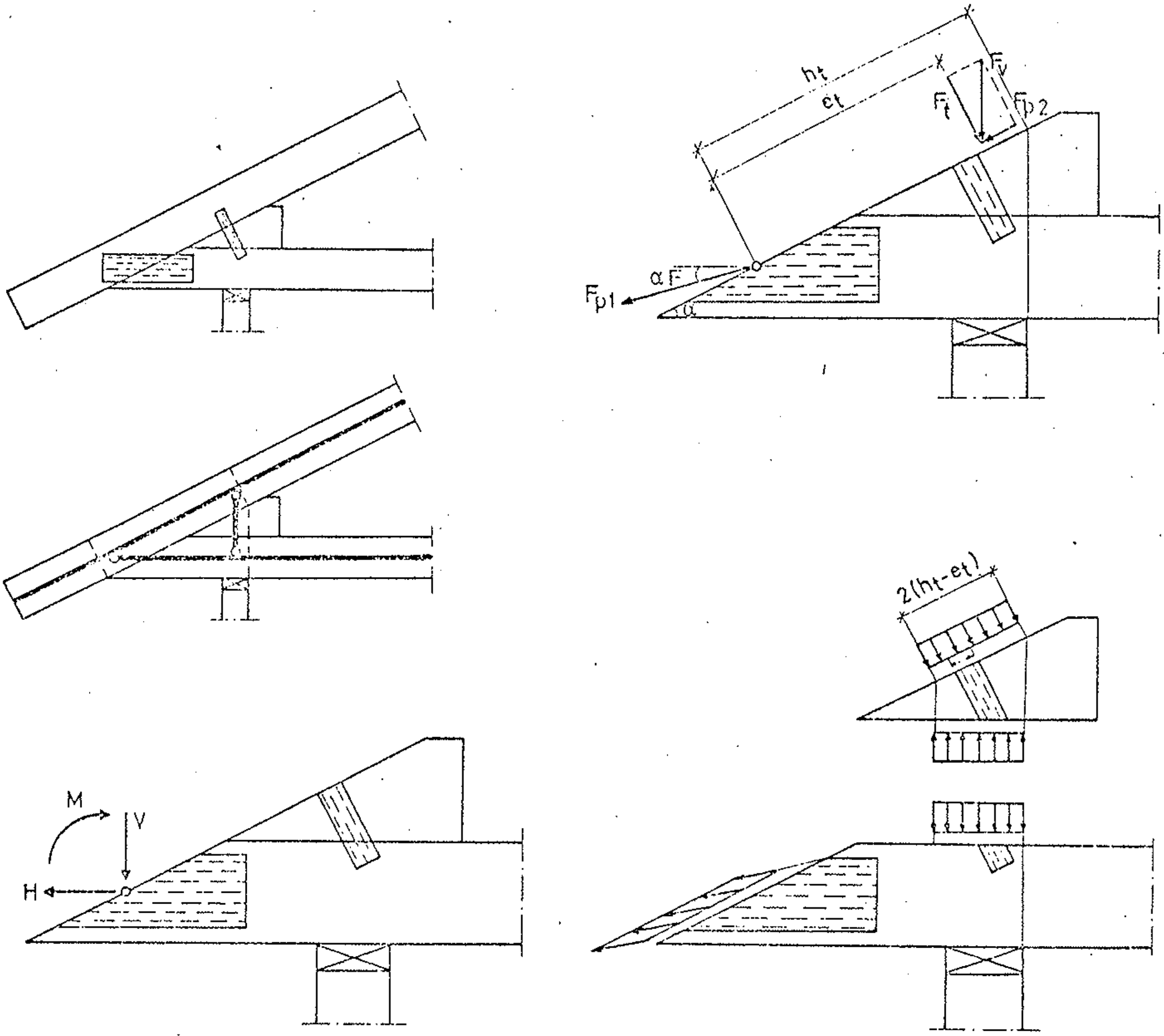

Figure 3

Example 3 (Figure 4)

Equilibrium equations

$$F_{p1} = H \tag{7.38}$$

$$F_{t1} + F_v = V \tag{7.39}$$

$$F_{t1}\, e_{t1} + F_v\, e_v = M \tag{7.40}$$

If the compression strength of the wood is utilized:

$$F_{t1} = 2(h_{t1} - e_{t1})\, Bf_c \tag{7.41}$$

$$F_v = 2(h_v - e_v)\, Bf_c \tag{7.42}$$

The vertical force F_v is divided into the components F_{t2} and F_{p2}

$$F_{t2} = F_v \cos\alpha \tag{7.43}$$

$$F_{p2} = F_v \sin\alpha \tag{7.44}$$

The unknown quantities in (7.39) are eliminated successively

$$e_v = \frac{1}{4}\left(h_{t1} + 3\,h_v - \frac{V}{Bf_c}\right) + \sqrt{\frac{1}{16}\left(h_{t1}+3h_v - \frac{V}{Bf_c}\right)^2 - \frac{4}{4Bf_c} - \frac{1}{2}\left(h_v - \frac{V}{2Bf_c}\right)\left(h_{t1}+h_v - \frac{V}{2Bf_c}\right)} \tag{7.45}$$

The force in the second plate F_{p2} is obtained if e_v is inserted in (7.42) and (7.44).

Further simplification

If it is assumed that the force F_{t1}, defined in Figure 4, does not contribute to the moment stabilizing the rafter ($e_{t1} = 0$), the eqs. (7.38)-(7.44) are replaced by

$$F_{p1} = H \tag{7.46}$$

$$F_{t1} + F_v = V \tag{7.47}$$

$$F_v \, e_v = M \tag{7.48}$$

$$F_v = 2\,(h_v - e_v)\,Bf_c \tag{7.49}$$

$$F_{t2} = F_v \cos\alpha \tag{7.50}$$

$$F_{p2} = F_v \sin\alpha \tag{7.51}$$

Inserting (7.49) in (7.48) gives

$$e_v = \frac{h_v}{2} + \sqrt{\frac{h_v^2}{4} - \frac{M}{2Bf_c}} \tag{7.52}$$

Reference

/1/ Källsner, B — Inverkan av indragna upplag på utformning och dimensionering av W- och WW-takstolar med spikplåtsförband. (Influence of support eccentricity on the design of W- and WW-trusses with nail plate connectors). Not yet published.

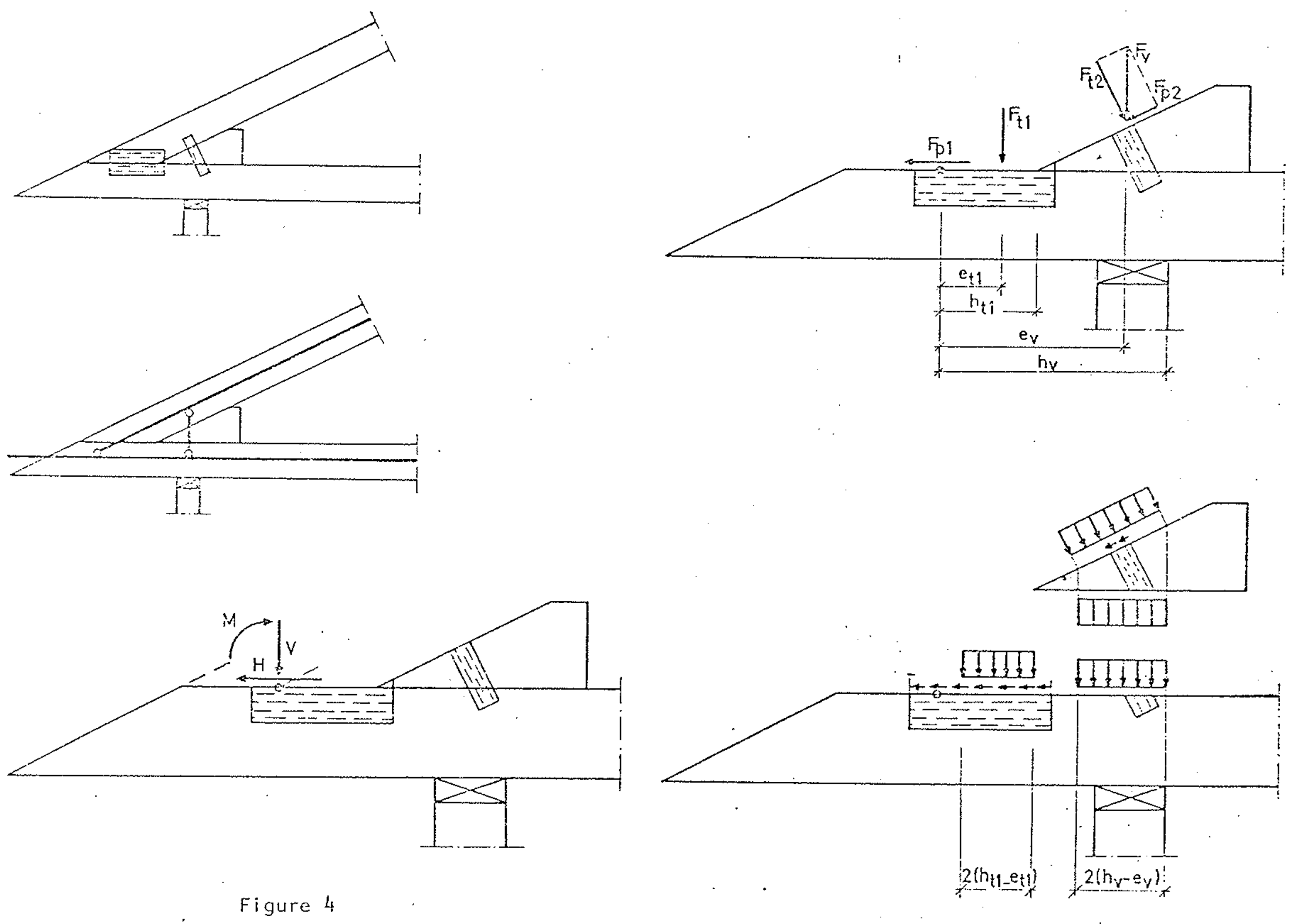

Figure 4

CIB-W18/13-7-6

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

NAIL DEFLECTION DATA FOR DESIGN

by

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JUNE 1980

C O N T E N T S

I l l u s t r a t i o n s

U N I T S

This paper refers to historical documents with test results given in imperial and technical metric units. For comparing the different results with each other and with the version of CP.112 that made use of some results, the most convenient units are used in the paper but the final conclusions are expressed in SI units.

The following table shows SWG sizes in millimetres as quoted in CP.112.

mm	SWG
2.64	12
2.95	11
3.25	10
3.66	9
4.06	8
4.47	7
4.88	6
5.38	5
5.89	4
6.40	3
7.94	-
9.52	-

NAIL DEFLECTION DATA FOR DESIGN

Part I - Solid to Solid Joints

The following paper is condensed from reports written in 1977 in relation to CP.112. It makes no reference to current work on fastener values derived for the revision of CP.112 which will appear as BS.5268.

A study of possible presentation methods for fastener test results has been made by Booth (1969) who suggests that in addition to the mean force-displacement curve the 1 in 100 and 1 in 40 curves should be provided. This suggestion assumed that in limit state design "deflection limits will have to be more clearly specified" than at present and that "force-displacement information for joints will be essential and analysis will be more complex, particularly for those joints which behave non-linearly".

It is not easy to think of design applications where exclusion limit force-displacement curves would be useful. They have an application in the precise calculation of the moment resistance of a nail group, but since there will usually be a considerable number of nails in the group the mean curve will be appropriate and not a minimum one. For a nailed ply-box beam Booth acknowledges that it would seem reasonable to use a mean force-displacement curve for the nail. He suggests the 1 in 100 value is probably more appropriate for a single fastener acting alone, but practical applications where the deflection is required are hard to imagine.

Even a mean force-displacement curve seems unlikely to find much application in practical design, and this view is supported by the absence of enquiries for the information that would be needed. It might be suggested that such curves are not applied because it is too difficult for designers to obtain and use the necessary data. But for a limited number of designers the data are in fact easily obtained and the fact that they have not been applied indicates the process would be too complicated and unrewarding for day-to-day design. Nevertheless for the present purposes it will be supposed that at least mean force-displacement curves are needed, to introduce a discussion of how they might be derived.

Fastener data for grouped timbers

If fastener data are given for grouped timbers, the tested timber should presumably be selected to have a characteristic density lower than any other in the group. This density will be taken as the lower boundary of the group and the fastener test results when divided by a suitable 'γ_m' value will be appropriate for designing structures with joints formed from a timber having the lowest characteristic density. For timbers in the same group with a higher characteristic density, the design loads calculated from the fastener test results will be conservative.

Similarly joints made from a higher timber in the group will deform less than indicated by the mean deformation quoted for fasteners tested in the lowest timber in the group. If a mean load-deflection curve is given for the lowest timber, its shape will be only roughly applicable to the higher ones. The shape would still give a roughly correct load distribution for calculating the moment resistance of a nail group if the nails were loaded to their full values, but in fact this will not be the case - they will be loaded only to the design values found permissible for the lowest timber, using only a limited portion of the true load-deflection diagram for the higher one. Because of this, the load distribution in a moment-resisting fastener group in the higher timber will actually be false. For such reasons if load-deflection curves are given at all the data of this kind would have to be very complicated.

FORCE DISPLACEMENT DATA FOR NAILED JOINTS

Rather than consider more sophisticated possibilities, it will be useful to see what simple data would satisfy the immediate needs of the designer for calculating the deflection of nailed structures and how it can be provided. Up till now perhaps the most readily available guidance in the United Kingdom has been the statement by Brock (1957) that "the slip under permissible load of two-member joints with nails in single shear should not exceed about 0.008 in. for joints made with 12-gauge nails, 0.016 in. for 7-gauge nails and 0.020 in. for 4-gauge nails". With supplementary information, this statement has been enough to give designers a rough notion of the order of deflection to be expected.

Long term displacements.

Brock's values were found from short-duration tests and considerably higher displacements will appear after a long period under full design load. Mack found that with initially green timber the displacement increases with time about sevenfold, and about four times if the timber is initially dry, most of the creep occurring during the first year (Mack 1972). For initially green timber under dead load it was concluded that an eventual displacement of about 1/16 in. should be allowed for with the design loads used in Australia (Mack 1963), and perhaps half this value would be appropriate for dry timber.

In service conditions in domestic roofs in the UK, these figures would perhaps be halved again because only half the total design load is continuous. As the published test results are limited and very variable between different test specimens, precise Code recommendations would not be realistic. Nor would it be realistic to attempt a refined study of, for example, the difference between Australian and UK timbers and loadings. It is suggested that a wording for the Code might warn that the eventual deflection under full design load might be 'several times' the figures tabulated, or alternatively that a crude figure such as 1 mm might be given as an allowance when estimating the deflection of structures; only the appropriate fraction of this would be applied, depending on the ratio of long-term to total load.

Mean curves for lowest timbers of groups in CP.112.

Brock's figures were derived for the 1 in 100 exclusion density of the timber tested. Load-displacement curves and slips at design load have been estimated roughly from his published report for the 1 in 100 exclusion density of the lowest timber in each fastener group of CP.112 but are not given here. They provide a link with the displacements quoted by Brock for three nail diameters in J3 timber, and will be available if a special need arises for data relating to the group boundaries. As already discussed, however, the need in ordinary design is for mean force-displacement figures and the 1 in 100 calculation would appear unnecessarily conservative.

The mean specific gravity for the lowest timber in each group has been provided by Brock in a draft British Standard for tests on fasteners. Slip moduli for the groups may be found by applying mathematical relationships given by Brock to provide load-displacement data at slips 0.006 in. and 0.030 in. from which load-displacement curves may be sketched for each nail diameter in timber of the four group specific gravities. From the resulting curves the slip δ at design load P may be found for each nail diameter, enabling P/δ to be plotted against nail diameter as shown for the J3 group in Fig.1.

Norén (1968) has proposed the relation k = 2700d for the short term slip modulus of fluted nails at working load. The corresponding expression for round wire nails would be

k = 2550d with k in kp/cm and d in cm

or k = 36,300d with k in lb/in and d in inches.

The latter line is shown in Fig.1 and may seem low in relation to the straight line sketched through Brock's results, which has the equation k = 7,560d in metric units or k = 107,500d in imperial. However if Norén's proposal is studied in relation to the highly-variable test results on which it is based, while bearing in mind that Brock's test specimens were tightly nailed, it appears that Brock's results are not inconsistent with the data quoted by Norén. An earlier proposal by Norén was k = 3700d which is also shown in Fig.1.

Application of Norén's Table 18 roughly confirmed the suggestion of 1 mm as the final displacement under the non-factored design load for the smaller-gauge nails in dry timber, with twice this value for large gauges.

Consideration should perhaps be given to taking the average values from diagrams such as Fig.1 rather than following the conservative dotted lines which for the J3 group have been found to give displacements similar to those for the 1 in 100 specific gravity. The briefest way to present the average results while avoiding an undue impression of precision is to quote the slip moduli as k = 7,560d etc for the short-term displacement.

The ratio of the long-term to the short-term slip derived from Norén's Table 18 is 3400/800 = 4.25. If this factor is applied throughout, the condensed average data for the four groups would be as follows when converted to SI units.

Group	Short-term slip modulus P/δ N/mm	Long-term slip modulus P/δ N/mm
J1	1200d	280d
J2	950d	220d
J3	740d	170d
J4	590d	140d

Slip moduli not related to diameter

If conservative values just below the crosses in Fig.1 and corresponding diagrams are selected, these would perhaps be

Group	Short-term slip modulus lb/in	Short-term slip modulus N/mm	Long-term slip modulus N/mm
J1	25,000	4300	1010
J2	20,000	3500	824
J3	15,000	2600	612
J4	10,000	1750	412

The figures in the right-hand column are again obtained by dividing by 4.25. This allows a figure to be obtained for comparison with the value 600 kg/cm in DIN 1052, which seems to correspond with the long-term value for J3 timber. Thus slip moduli for the four groups could be provided in the manner of DIN 1052 with values 400, 600, 800 and 1000 for groups J4 to J1 respectively. This would allow the subject to be covered very briefly in a Code catering fairly accurately for the more important smaller sizes of nail but underestimating the stiffness of the large diameters.

REVIEW AND RECOMMENDATIONS

The study from which the above notes derived took the following course:

(1) 1% exclusion load-deflection diagrams introduced a tentative suggestion of 1 mm for the long-term deflection of joints.

(2) As the study suggested mean data are required, the 1% exclusion basis seemed unnecessarily conservative, so deflections relating to the mean specific gravity of the lowest timber in group J3 were examined (Fig.1). When diameter-dependent slip moduli on Norén's conservative basis were introduced, short-term displacements similar to those for the 1% exclusion case were found because of the conservatism.

Similar results were reached in a different manner by Morris (1973). He drew 1% exclusion load-displacement curves for the J3 group and found that a slip modulus k = 350d N/mm suggested by the work of Norén, Niskanen and others yielded nail design loads very close to those in CP.112. This is the same as saying that the CP112 nail loads applied to the 1% exclusion curves will indicate a slip modulus k = 350d.

A diameter-dependent slip modulus was found for long-duration as well as short-term loading, following Noren's method.

(3) Average load-displacement data for all four groups were plotted as shown in Fig.1 for Group J3, and diameter-dependent slip modulus expressions were given for the average values shown by the plotted crosses. For the J3 group, the short-term slip modulus (740d) is about twice Morris's value (350d).

(4) In a fresh approach, a slip modulus not dependent on diameter was given from the diagrams. A conservative value just below the plotted crosses gave the figures 400, 600, 800 and 1000 N/mm for groups J1 to J4 respectively, the 600 for J3 agreeing with DIN 1052.

Conclusions

The writer favours the last proposal, long-term slip moduli of 400, 600, 800 and 1000 N/mm for groups J1 to J4. If diameter-dependent moduli are specially wanted then Morris's k = 350d for the short-term J3 values seems acceptable; it is not so conservative as Noren's latest recommendation but agrees with his earlier one. Corresponding values for the other groups and related long-term moduli can be worked out following the methods already indicated.

PART II - PLYWOOD TO SOLID JOINTS

The following notes describe how Fig 3 was prepared to compare several sets of deflection data for plywood-to-timber joints. It will be seen that the results vary widely, allowing only crude figures to be produced for use in design.

Nail loads for plywood-to-timber joints have been derived afresh for BS.5268, and there is no reference to this work in the notes below condensed from reports written in 1977.

Deflection data for Douglas fir.

The load-deflection curves in Fig.2 for Douglas fir plywood attached to Douglas fir framing have been available for many years and in the absence of better data have been applied more generally when a displacement under load has been needed. The curves, taken from DFPA laboratory report No.55 by D. Countryman have been applied only for estimating the deflection of racking-resistant wall panels under wind loading. For this purpose the factor 1.25 was formerly applied to the nail design load before using the curves, and more recently the factor 1.375 has been adopted for very short-term wind loading.

The corresponding slip moduli are plotted against nail diameter in Fig.14 using U.S. nail sizes rather than the U.K. equivalents. The group of points plotted is joined by dotted lines giving a shape reminiscent of the Great Bear and marked 'COUNTRYMAN, D.FIR'.

Variation of slip moduli with specific gravity (Morris)

For the purpose of the following calculations taking Morris's work further, his expression for Mack's load reduction factor will be adopted. The load-deflection curve for any nail diameter and known 'species factor' may be obtained by multiplying the reduction factor for a given displacement by a nail factor N and a species factor k_s according to the relationship

$$P = RNk_s$$

Morris has proposed the relationship

$$k_s = 350\ G$$

for the species factor and if this is also adopted then

$$P = 350\ GRN$$

For the J3 lower exclusion specific gravity the relationship will be

$$P = 350 \times 0.303\ RN$$

$$\text{i.e.}\ P = 106\ RN$$

where R may be found for any displacement from the curve showing the load reduction factor and N is the factor for a given nail diameter and length.

The value of slip modulus depends on the value of the design load which has not been formally established for different plywood constructions. Adopting a U.S. rule for the minimum plywood thickness for use with a given nail diameter, the nail design load will be the same as for nails in solid timber, giving for example for 2 in. 12-gauge nails in J3 timber,

$$40 = 106\ R$$

- since N = 1 for 2 in. 12-gauge nails.

Then R = 0.377, the corresponding value of δ is found from the curve to be 0.007 in. and the slip modulus may be worked out as $40/\delta = 40/0.007 = 5720\ \frac{lb}{in.}$

For any other specific gravity and nail size, the value of R may be found from

$$R = \frac{P}{350GN}$$

where P is the design nail load. If the slip corresponding to this value of R is δ_R, the slip modulus is P/δ_R.

Applying this procedure to a range of nail sizes for the J3 group gives the values of slip modulus plotted against nail diameter in Fig.3 for two thicknesses of plywood. The resulting lines are marked MORRIS G = 0.303, $\frac{3}{4}$" and $\frac{3}{8}$".

Redwood plywood nailed to redwood

A paper by Ingvar Jansson at the CIB joints symposium in 1965 gave load-deflection curves for redwood plywood nailed to solid redwood. The curves were fitted by expressions of the form

$$p = \alpha x^{\beta} - \gamma$$

and values for the coefficients α, β and γ are tabulated in the paper. These allow slip moduli to be calculated without making measurements on the load-deflection curves, but Jansson's suggested design loads when plotted against nail diameter are much higher than would be allowed by CP.112. The CP.112 design loads for the J3 group were plotted in the same diagram and consonant design loads corresponding to Jansson's nail diameters were derived by projecting downwards to this lower curve and then reading horizontally to the vertical axis.

The resulting values are used together with the tabulated values of α, β and γ to calculate the slip moduli plotted in Fig.14 and marked JANSSON-REDWOOD. Where several ply thicknesses were tested with a single nail diameter, the points are joined together by a vertical line. Also, points relating to the same plywood thickness are joined together.

Birch plywood nailed to redwood

Test results for Finnish Birch plywood are available from a paper by Hall at a 1970 conference in Sheffield, reporting tests by Lauricio at Imperial College (1967 thesis). Slip moduli tabulated in the paper for 12-gauge and 10-gauge nails are plotted in Fig.3. These are not based on deflection at design load but are $v_{0,4}$ values of the Stevin Laboratory test method; however both these values and the slip at design load relate to the initial steep part of the load-deflection curve where the curvature is not very great, so the plotted results give a reasonable indication of the slip moduli that would be expected.

The results for three ply thicknesses tested with 12-gauge nails are shown in Fig.3 by the vertical dotted line marked FINPLY-REDWOOD. For the same thicknesses with 10-gauge nails only the 9 mm point is shown at the top of the page; the values for 12 mm and 18 mm plywood run off the top of the graph.

Further results for Douglas fir.

A further set of test results for Douglas fir is provided by DFPA Laboratory report No.94, 'Nails in double shear' by Lyons. The results extracted are actually from single shear tests made to establish a basis of comparison for the double shear tests in the report's title.

The design loads for J2 timber that would be allowed in CP.112 for the nail sizes used by Lyons are applied in Figs.10-11 of report No.94 to calculate the slip moduli plotted in the bottom right-hand corner of Fig.3 and labelled LYONS D.FIR.

COMPARISON OF RESULTS (FIG. 3)

Values for redwood

Leaving out the two sets of results for Douglas fir, there are three items in Fig. 3 relating to redwood. Jansson's figures are for shimmed joints of redwood plywood nailed to redwood. Morris's lines predicting the performance of shimmed joints at the lower exclusion specific gravity of redwood seem to lie in roughly correct relation to those of Jansson but his thickness factor shows a much smaller effect of thickness than indicated by Jansson's results for 12 mm and 18 mm plywood at the 0.11 and 0.134 in. diameters.

The Finply-redwood results at 0.104 diameter (12-gauge) are surprisingly similar to Jansson's 0.11 in. diameter figures, so it is a pity that the Finply-redwood results for 0.128 in. diameter (10-gauge) are so high that they have to be left out of the discussion altogether. In these tests following the Stevin Laboratory method the joints were presumably shimmed so the three sets of results for redwood are all for shimmed single-shear joints although there are differences in the details of the specimens and test methods. The comparisons being made are so rough that minor differences are unimportant, but important effects could arise from differences in the manner in which deflection is measured.

Still concentrating on the three sets of points for redwood (Jansson, Finply, Morris), the lines radiating from the origin would provide relations of the form $k = md$ for each of the thicknesses 4, 7 (or 9), 12 and 18 mm. Each line is directed roughly towards the middle of the group of points for the corresponding thickness. The one for 12 mm relates to the two Jansson results and could be steeper if influenced by the Finply point for 12 mm ply.

Douglas fir

The results for Douglas fir are not sensibly related to those for redwood. Countryman's set from Fig.2 is below Jansson's for similar ply thicknesses and nail diameters although it refers to the J2 timber Douglas fir while Jansson's values are for redwood, a J3 timber.

The Lyons set is still more perplexing, lying so far below the Countryman set that a scale error of 2 would be suspected.

Consideration of all results together

If the Lyons set is left out it is possible to reach some sort of logical conclusion from this difficult comparison by lumping all the results together without trying to establish different slip moduli for each timber group. The radiating line marked '7' could be swung lower and called 9 mm, passing between Countryman's points for $\frac{3}{8}$ in. The '12' line would stay roughly in its present position and the '18' line could be rotated to a more conservative angle. These remarks assume the radiating lines should pass through the centres of their respective groups of plotted points, rather than the lower boundaries of the groups.

The resulting lines, indicated by short dotted lines marked (9), (12) and (18), have slopes giving the following diameter-dependent relationships for the short-term slip moduli:

Ply thickness mm	Short-term slip modulus lb/in	kg/cm	N/mm
9	55,500d	3890d	379d
12	77,500d	5450d	534d
18	100,000d	7030d	689d

Another alternative, again leaving out the Lyons set, is obtained by ignoring nail sizes lower than 12-gauge so that the Jansson value for 12 mm ply at 0.0906 diameter is left out. Then horizontal lines may be drawn as marked inside the vertical axis (9, 12, 15, 18). Two possibilities are shown. The one on the left indicates constant slip moduli for all diameters as follows:

Ply thickness	Slip modulus lb/in.
9 mm	6000
12 mm	9000
15 mm	12000
18 mm	15000

The value for 15 mm thickness would not allow for the 15 mm Douglas fir in Countryman's results. To include this and establish a more conservative value for the 18 mm thickness, the right-hand column of figures inside the vertical axis suggests the values:

Ply thickness	Slip modulus lb/in.
9 mm	6000
12 mm	8000
15 mm	10000
18 mm	12000

In the Code, long-term values based on these might be quoted as conservative for any kind of plywood in CP112 attached to softwood, with a note that much higher values may be found in practice.

LONG DURATION LOAD

Only very limited information is immediately to hand on the effect of long duration load on ply-to-solid joints. Mack's tests were on plywood of Australian group D species fastened to initially green messmate stringybark with 12-gauge nails.

For solid-to-solid green messmate stringybark with an overall displacement on loading of 0.006 in. for a 55 lb. nail (p.6, Table 2), Mack's Fig.4 upper diagram gives a final displacement of 0.05 in. for two joints and 0.15 for a third.

In the ply-to-solid construction, the initial displacement for 55 lb. load may be taken as 0.002 from Table 6 (p.14 - actually 60 lb.) and the final displacement in Table 8 (p.17) is 0.066 in. There is no mention of shims for these joints.

It is concluded that creep in the ply-to-solid joints is at least as severe as in the solid-to-solid construction, and it must be presumed that this will also be the case for dry constructions. For the present, then, the factor of 4.25 applied to solid-to-solid joints in the first section of this report will be adopted also for plywood-to-solid joints. The long-term slip moduli derived from the last suggestion above will then be as follows:

Ply thickness	Short-term slip modulus lb/in	Short-term slip modulus N/mm	Long-term slip modulus N/mm
9 mm	6000	1050	247
12 mm	8000	1400	329
15 mm	10000	1750	412
18 mm	12000	2100	494

Possible round values for the figures in the right-hand column are 250, 330, 400, 500 N/mm for ply thicknesses 9, 12, 15 and 18 mm respectively, compared with the value 600 N/mm suggested for solid-to-solid J3 nailed joints in the previous section.

PRACTICAL APPLICATION OF SHORT-TERM SLIP MODULI

The short-term moduli have been taken as directly applicable in the design of ply-sheathed wall panels against wind loading. The displacements for the few small nail sizes used in this application do not vary very greatly with nail diameter and the difference found from the diameter-dependent relationships is not very different from that using a constant value of slip modulus for each plywood thickness.

Bearing in mind the variability of the test results, the differences seemed small enough to allow only a single value to be quoted for all nail diameters used in a given thickness of plywood as follows:-

Ply thickness mm	Displacement at design load mm
9	0.25
12	0.18
18	0.14

In a publication giving these figures, a note as follows appears below the table:-

> "For nails loaded to full design load, incorporating the factor 1.25 for wind loading, the following nail slip values may be applied. These are conservative estimates taken from very variable test results, and much lower values may be found in practice."

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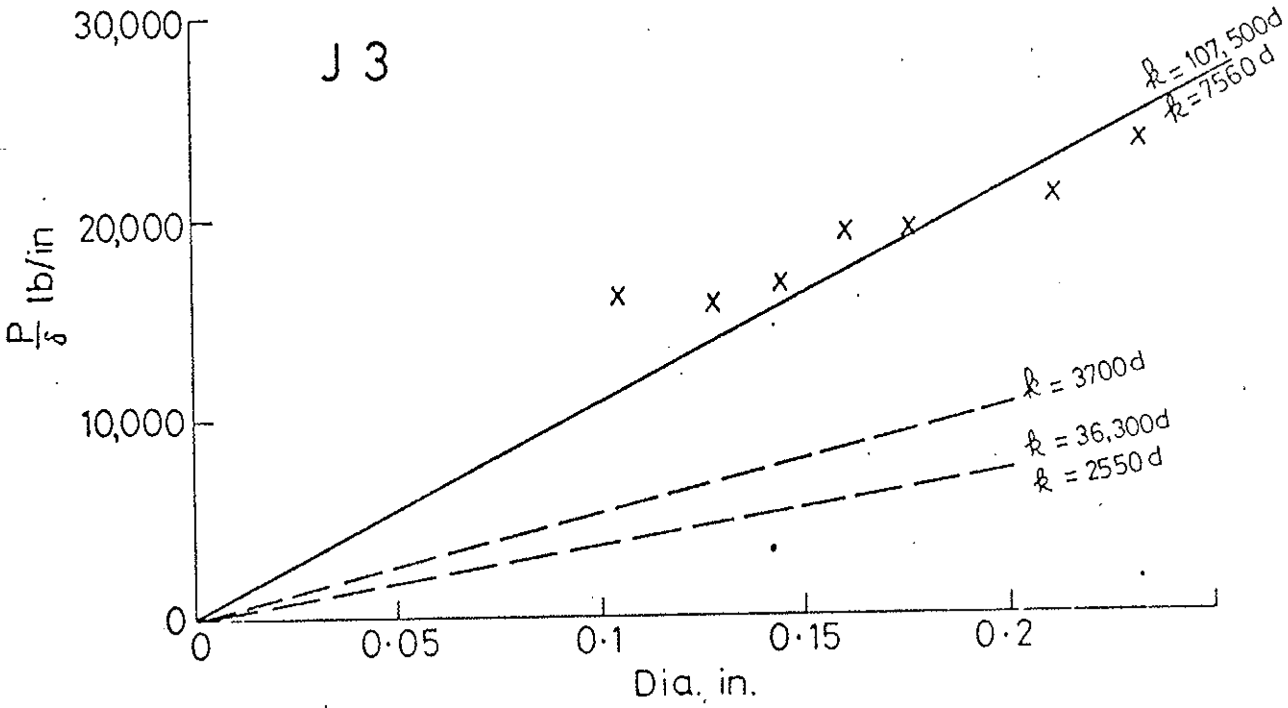

J 3
30,000
20,000
10,000
0
$\frac{P}{\delta}$ lb/in
0
0·05
0·1
0·15
0·2
Dia., in.
k = 107,500d
k = 7560 d
k = 3700d
k = 36,300d
k = 2550 d

FIG. 1.

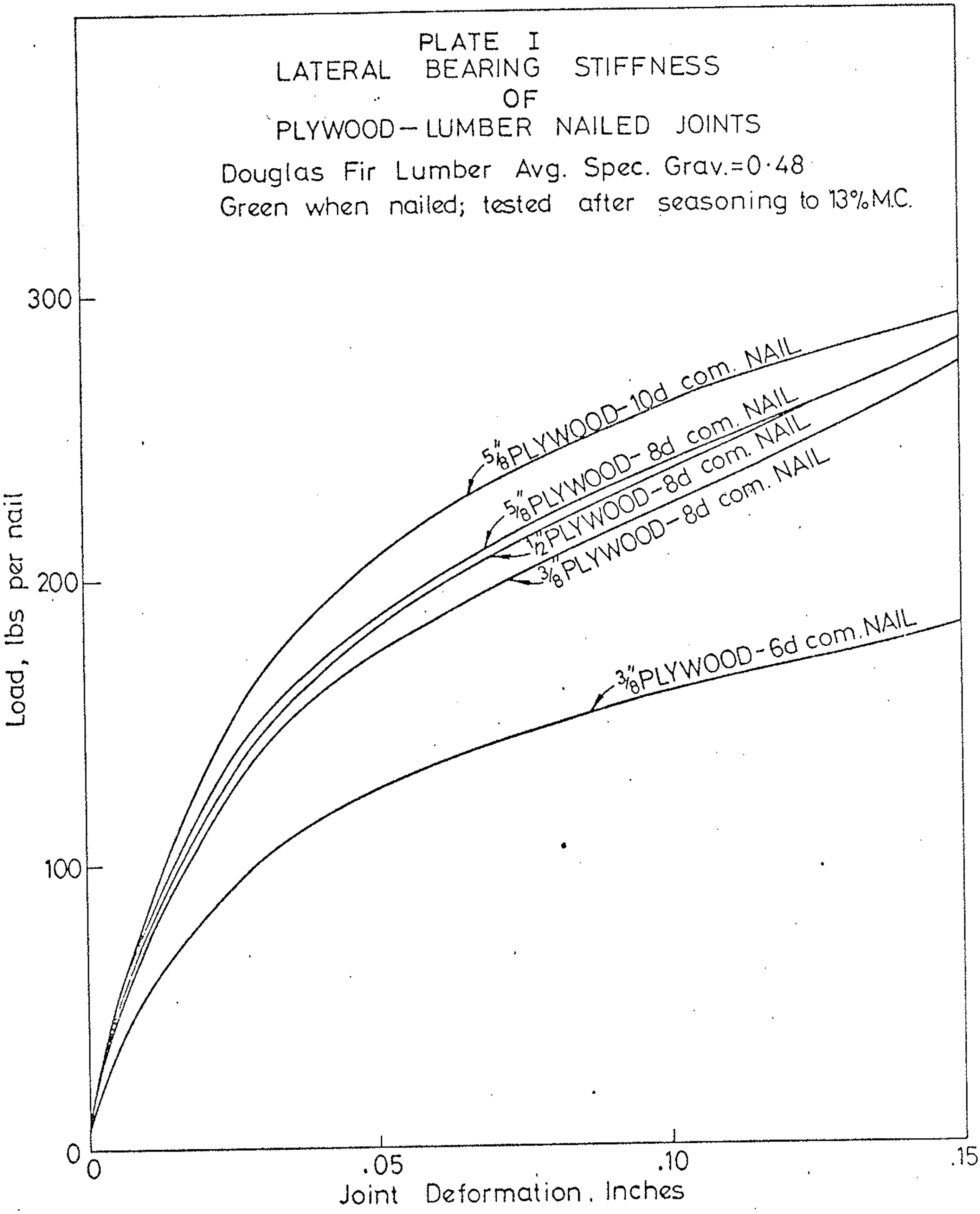
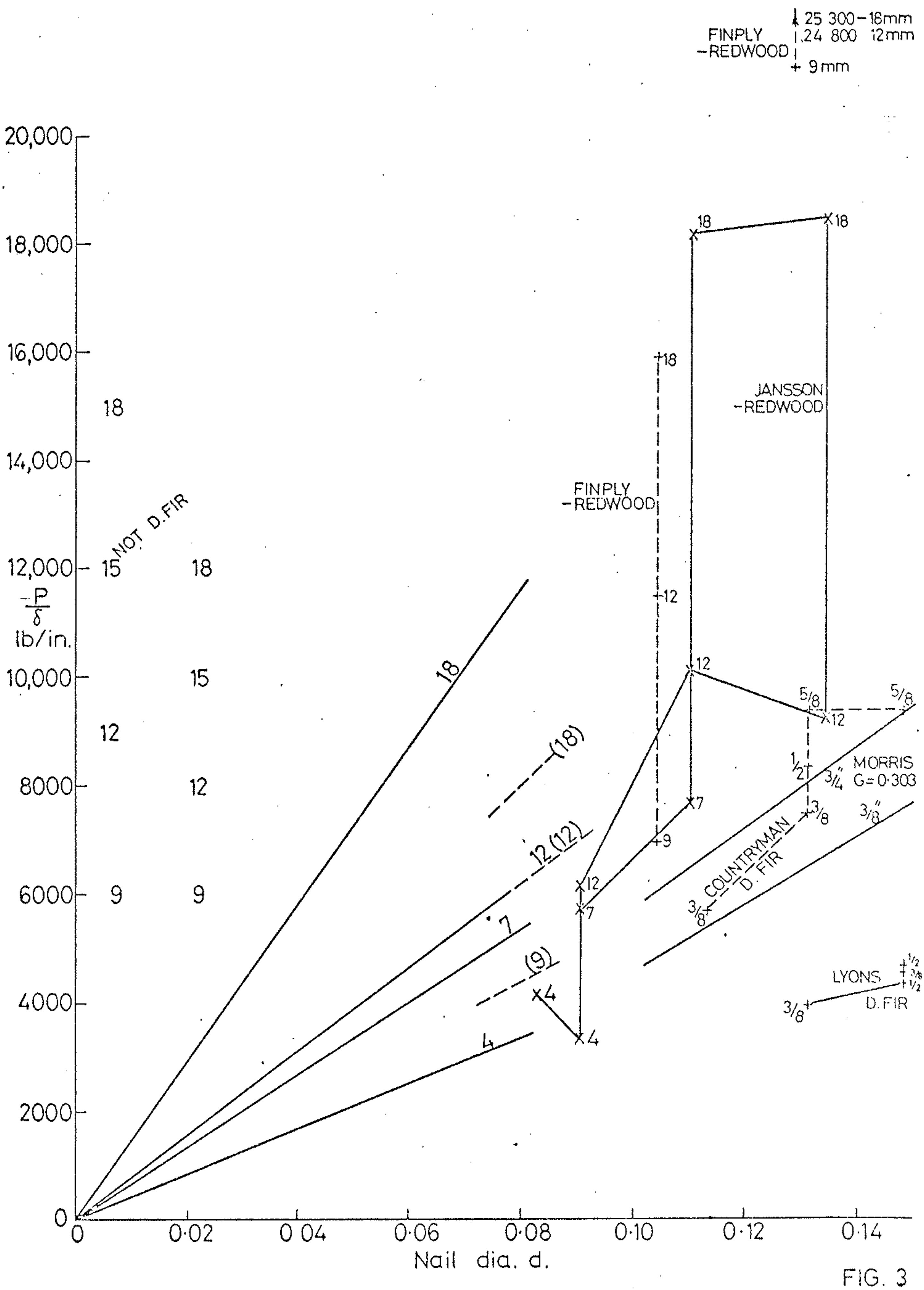


PLATE I
LATERAL BEARING STIFFNESS
OF
PLYWOOD-LUMBER NAILED JOINTS
Douglas Fir Lumber Avg. Spec. Grav.=0·48
Green when nailed; tested after seasoning to 13% M.C.
5/8" PLYWOOD-10d com. NAIL
5/8" PLYWOOD-8d com. NAIL
1/2" PLYWOOD-8d com. NAIL
3/8" PLYWOOD-8d com. NAIL
3/8" PLYWOOD-6d com. NAIL
Load, lbs per nail
0
100
200
300
0
.05
.10
.15
Joint Deformation, Inches

FIG. 2.



25 300 – 18mm
24 800 12mm
FINPLY
–REDWOOD
9 mm
20,000
18,000
16,000
14,000
12,000
10,000
8000
6000
4000
2000
0
$\frac{P}{\delta}$
lb/in.
NOT D.FIR
18
15
12
9
18
15
12
9
18
(18)
12 (12)
7
(9)
4
FINPLY
–REDWOOD
JANSSON
–REDWOOD
MORRIS
G = 0·303
COUNTRYMAN
D. FIR
LYONS
D. FIR
0
0·02
0·04
0·06
0·08
0·10
0·12
0·14
Nail dia. d.

FIG. 3

CIB-W18/13-7-7

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TEST ON BOLTED JOINTS

by

P Vermeyden

Stevin Laboratory

THE NETHERLANDS

OTANIEMI, FINLAND

JUNE 1980

TECHNISCHE HOGESCHOOL DELFT

AFDELING DER CIVIELE TECHNIEK

Report 4-80-1 Onderzoek B-2

Test on bolted joints

Translation of 4-63-6 B-2

march 1980 Ir. P. Vermeyden

STEVIN-LABORATORIUM

HOUTCONSTRUCTIES

STEVIN-LABORATORIUM
van de afdeling der
Weg- en Waterbouwkunde
der
TECHNISCHE HOGESCHOOL

Report 4-80-1 B-2

Test on bolted joints

March 1980

Translation of 4-63-6 B2

By Ir. P. Vermeyden

Contents

I. Introduction

A limited number of tests was performed as a follow-up to the literature research described in Report B 1*) relating to the strength of bolted joints. The object of the tests was to verify or to supplement the available data in some respects and to obtain insight into the magnitude of the displacements that occur, on the one hand, and into the scatter of the failure loads, on the other.
As the ultimate object of this research is to establish at set of regulations for bolted joints, the results of the tests will be compared with the theoretical strength values given in report B 1 and with foreign regulations.

*) Report 4-62-2-B-1. Literature research concerning the strength of bolted joints. February 1962.

II. Review of the tests performed

II.1. Schedule of tests

The tests that were performed can be divided into five main groups, with the reference designations SBa, SBb, SBc, SBd and SBe. Each main group is subdivided into series, each comprising five specimens which are all alike. This schedule of tests is presented in Table 1. Each letter in this table signifies that five specimens of the dimensions indicated were tested.

Table 1: Schedule of tests.

bolt diameter d (inch)			$\frac{1}{2}$		$\frac{3}{4}$	
thickness of middle timber m (mm)			38	58	38	58
load angle	number of axes	arrangement				
T 0°	1		a	a c	b d	b
	2	oo		c		
		o°		c		
	4	oo oo		c		
		oooo		c	d	
T 30°	1			e		
	2	o°		e		
T 60°	1			e		
	2	o°		e		

In all, 15 x 5 = 75 tests were performed. Of these, the tests in the two main groups "a" and "b" were carried out in conjunction with the investigation of joints formed with toothed plate connectors. The timber used for these bolted joints corresponded to that used for certain toothed plate joints. The timber dimensions and the washer sizes used with the bolts were also so chosen as to correspond to the toothed plate joints. For the results of this comparative research the reader is referred to the report on the plate connector tests which is to be published in due course.

The specimens SBa and SBb were tested in August 1962, the others (SBc, SBd and SBe) in October and November of that year.

II.2. Description of the specimens

All the specimens are symmetrically constructed tensile joints. The "straight" joints (Fig. 1) comprise a middle timber (thickness m) and two side timbers (each of thickness z); the "oblique" joints are constructed as shown in Fig. 2, where the horizontal tension members are loaded by the bolts with a force acting at an angle α on one side and at an agle of 90° on the other side. Drawings of the various test specimens are given in Appendix 5.

Two different bolt diameters are used, namely, $d = \frac{1}{2}$" and $d = \frac{3}{4}$". The timber thickness is so shosen that the slenderness ratio $\lambda = m/d$ approximately has the values 2, 3 and 4.5, having regard to the difference in the action of the bolt in the joint (cf. report B 1).

In most cases the timber width has been given the minimum value, while the distances to the unloaded and to the loaded edge are $r_o = 3d$ and $r_b = 4d$ respectively, while in the oblique loaded joints the centroid of the group of bolts is always on the centre-line of the member.

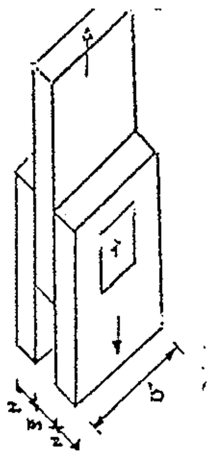


Fig. 1: Construction of straight joints

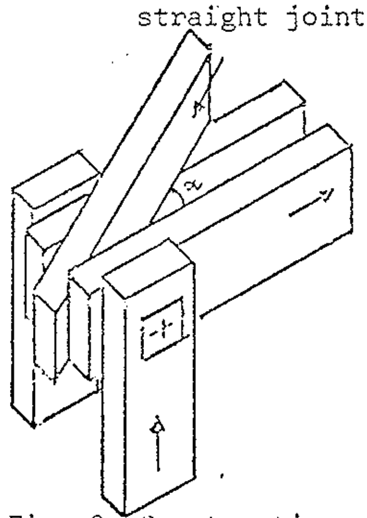


Fig. 2: Construction of oblique joints

In the quadriaxial joints the width has been made larger than 6d so as not to have high tensile stresses in the weakened timber section. The dimensions chosen for the washers are as follows: 40 mm x 40 mm x 3 mm washers for ½" bolts*), 60 mm x 60 mm x 4 mm washers for ¾" bolts. Some of the dimensions in the specimens SBa and SBb are different because these specimens were tested in conjunction with specimens formed with toothed plate timber connectors, as already mentioned; the joints of the two types were "matched" for the sake of comparability. Some data relating to the specimens are given in Table 2.

Table 2: Dimensions of the specimens.

Specimen no. SB	bolt diameter d inch	washers mm	load angle	number of axes	Arrangement	$d_{//}$ mm	$d_{\perp}'$ mm	timber dimensions m	z	b	λ
a 1- 5	½	63.63.4	T 0°	1				38	19	95	3,0
6-10								58	29	95	4,6
b 1- 5	¾	76.76.5	T 0°	1				58	29	120	3,0
6-10								38	25	120	2,0
c 1- 5	½	40.40.4	T 0°	1				58	29	76	4,6
6-10				2	oo	89	0			76	
11-15				2	o°	67	24			102	
16-20				4	oo oo	89	51			127	
21-25				4	oooo	89	0			127	
d 1- 5	¾	60.60.4	T 0°	1				38	19	115	2,0
6-10				4	oooo	134	0			145	
e 1- 5	½	40.40.4	T 30°	1				58	')	101	4,6
6-10			T 30°	2	o°	73	20			120	
11-15			T 30°	1						101	
16-20			T 60°	2	o°	57	53			133	

') Central compression member 95 mm; vertical legs each 47 mm.

*) Actually, 40 mm x 40 mm x 4 mm washers were supplied and used, however.

The end distance (to the loaded end) has been taken as 7d, and this dimension has also been adopted for d//, while d⊥ has been taken as 4d. (The symbols // and ⊥ denote parallel to, and perpendicular to, the grain respectively). The bolts in the biaxial joints are in a staggered arrangement determined in accordance with fig. 3.

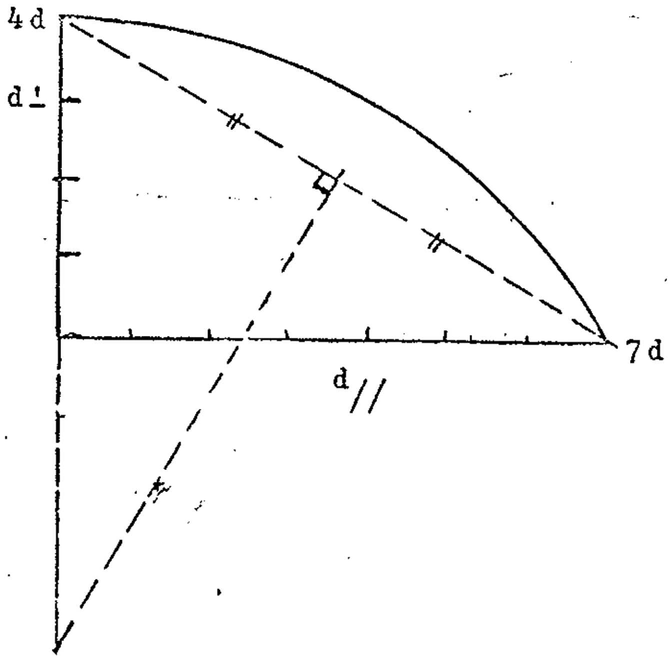


Fig. 3: Diagram for determining the positions of bolts

Basing oneself on d// = 7d and d ⊥ = 4d, the curve in that diagram has been obtained in the same manner as in the positioning diagram for ring connectors (see documentation sheet*)). The region situated within this curve gives combinations of d// and d⊥ which are not permissible in a joint.

*) Documentation no. 4: Ring connectors H.V.I., Amsterdam, 1961.

The bolt holes have been formed by drilling; the drills have the diameters in Table 3. Measurements have shown the diameter of the actual hole to be on an average about 0.3 mm larger than that of the drill employed.

Table 3: Diameters of the drills used for forming the bolt holes.

bolt diameter	drill diameter	for specimen no.
$\frac{1}{2}$	14	a
	13	c, e
$\frac{3}{4}$	20	b, d

In constructing the joints the bolts are tightened. Prior to testing, they are slackened and then lightly tightened by hand to the "loosely tight" condition. The object of this procedure is to eliminate as far as possible the effect of timber-on-timber friction at the start of testing. In an actual structure friction may, after all, also be absent as a result of shrinkage of the timber.

The straight loaded joints were all tested in the 50-tonne Amsler tensile testing machine. The oblique joints were tested in the testing rig devised for the purpose. All these tests were performed in accordance with standard procedure. In the case of the oblique joints the load was measured with the aid of a load cell installed on the line of the central compressive member.

III. Material properties

III.1. Timber

The timber used for the specimens was whitewood conforming to the test requirements for structural timber (grade "constructiehout"). It was purchased in two lots, namely, the timber for the specimens SBa and SBb at the same time as that for the timber connector tests in June 1962, and the timber for the other specimens in August 1962. The moisture content of the timber on delivery was about 19% on an average (12-25%). At the time of testing the average was 13% (10-15%) in the straight and 15% (13-18%) in the oblique joints. The annual rings were 1-3 mm in width.
After testing, specimens for investigating the timber properties were prepared from the middle timber and also from the two side timbers of a number of straight joints. The properties investigated were the compressive strength, the shear strength, the cleavage strength and the crushing strength. Drawings of these specimens are given in Appendix 6. The test results are given in Appendix 1, while the mean values are listed in Table 4. This table also includes the coefficients of variation, i.e., the quotients of the standard deviation of the individual result and the associated mean.

Table 4: Average strength properties of the timber*)

property	average	coefficient of variation
compressive strength $\sigma_{d\ max}$	564 kgf/cm^2	12%
crushing strength $\sigma_{s\ max}$	367 kgf/cm^2	16%
shear strength τ_{max}	78 kgf/cm^2	15%
cleavage strength S_{max}	38 kgf/cm'	16%

*) Average values of the results obtained with the timber of specimens SBb 1 to 5, SBc 6 to 25, SBd 6 to 10.

In Graph 1 the compressive strengths have been plotted against the crushing strengths for the same member in a number of joints. The mean value obtained for the ratio ξ of maximum crushing strength to maximum compressive strength is $\xi = \sigma_{s\ max}/\sigma_{d\ max} = 0.65$. Graph 2 presents the histograms of the strength properties, to which Table 4 also relates.

III.2. Bolts

The yield point of bolts simular to those used in the specimen joints was determined in a three-point bending test. The load-deflection diagrams of $\frac{1}{2}$", $\frac{5}{8}$" and $\frac{3}{4}$" bolts are presented in Graph 3. Each curve is the average of five tests and comprises two branches: an "elastic" portion below the yield point and a "plastic" portion above it. In the diagrams the two portions are schematized as straight lines; their intersection is taken as representing the load at which the plastic hinge develops.
In all the tests the span was l = 107 mm; the mean value of the diameter, as determined from measurements, was d = 12.7 mm for the $\frac{1}{2}$" bolts, d = 16.3 mm for the 5/8" bolts, and d = 18.8 mm for the $\frac{3}{4}$" bolts. In a flexurally loaded bolt the cross-section will undergo yielding when a plastic hinge develops, i.e., when the yield stress has been reached in all parts of the section. The value of the yield point is then expressed by:

$$\sigma_v = \frac{mv}{w_{pl}} = \frac{\frac{1}{4}.P_v.1}{1.7.\frac{\pi}{32}.d^3} = \frac{0.25.10.7}{0.167\ d^3}.P_v = \frac{16\ P_v}{d^3}$$

The values thus found (each being the average of five tests) are summarized in Table 5.

Table 5: Average value of the yield point σ_v from flexural tests.

bolt diameter d inch	yield point σ_v kgf/cm^2
$\frac{1}{2}$	2540
5/8	3050
$\frac{3}{4}$	2920

A number of $\frac{1}{2}$" bolts were moreover loaded in tension. In these tests the average values obtained for the yield point and the tensile strength were 2900 kgf/cm^2 and 3780 kgf/cm^2 respectively. Measurements revealed that the diameter of the bolts employed in the specimen joints was in an average 0.3 mm less than the nominal value, whereas the diameter of the bolt holes was on an average 0.3 mm more than the nominal drill diameter. The scatter displayed by these values is fairly large.

IV Results of the tests of the joints

IV.1. Straight uniaxial joints

As appears from tables 1 and 2, in all 6 x 5 = 30 uniaxial specimens were tested. The results, averaged per series, are given in table 6, which also gives dimensions of the specimens. Appendix 2 contains the results for the specimens individually.

Table 6: Results for straight uniaxial joints

λ			2,0		3,0		4,6	
Serie SB			d 1-5	b 6-10	a 1-5	b 1-5	c 1-5	a 6-10
Dimensions: bolt diameter d		inch	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{2}$
washers		mm	60.60.4	76.76.5	63.63.4	76.76.5	40.40.4	63.63.4
drill diameter for hole (nominal)		mm	20	20	14	20	13	14
timber dimensions	m	mm	38	38	38	58	58	58
	z	mm	19	25	19	29	29	29
	b	mm	115	120	95	120	76	95
Failure load: average per serie		kgf	2135	2858	1428	3572	2686	2633
coefficient of variation			11%	15%	5%	11%	3%	11%
Displacements:							0,14.	
$e_{0.4}$	average	mm	0.17	0.19	0.18	0.23	0.14	0.31
	coef. of var.		10%	10%	15%	11%	27%	39%
$v_{0.4}$	average	mm	0,92 0.92	1.09	0.87	1.37	1.93	1.87
	coef. of var.		22%	30%	22%	20%	17%	34%
$v_{0.6}$	average	mm	1.32	1.51	1.38	2.14	3.00	4.06
	coef. of var.		20%	25%	13%	14%	14%	24%
$v_{0.8}$	average		2.32	2.55	2.31	3.44	5.13	6.6
	coef. of var.		22%	32%	10%	14%	12%	18%
a	average		2.02	0.92	2.04	1.89	- 0.07	1.95
	coef. of var.		14%	59%	23%	68%	38%	24%
Type of failure: crushing only	timber		-	2m	3m	3m	-	5m
cracking, splitting	timber		2m, 1z	1m	2m	-	4m	-
shearing	timber		2z	2m	-	2m	1m	-
shape of bolt			straight	straight	straight	straight	bent	bent

In compiling Table 6, the value of the slenderness ratio λ (= m/d) was used as the criterion of classification. This was done because, on the basis of the theory in report B 1, it is presumed that the behaviour and the strength on the joint are dependent on λ. This will be further considered in chapter V.

There is very minor difference (to be further neglected here) in timber width between the two series of joints with $\frac{3}{4}$" bolts and λ = 2.0, and also a difference in the thickness of the side timbers and in the dimensions of the washers. As a result of these two last-mentioned differences the restraining moment of the bolt in the side timber in the specimens of series SNb 6-10 may exceed that in the specimens SBd 1-5. Hence the restraint is more effective in the specimens of series b. With slender bolts (high value of λ) this would result in an increase in strength (higher loadbearing capacity: see Report B 1); however, with bolts characterized by λ = 2.0, which are far from slender, this has theoretically no effect.

All the joints were given the same treatment before being tested: the bolts were slackened and then tightened to the "loosely tight" condition in order to eliminate the effect of friction as far as possible. It therefore appears unlikely that the difference in failure load between the two series is attributable to the difference in side timber thickness and washer dimensions. More probably, the difference in failure load is due to the variation in the properties of the timber. This assumption is supported by the data in Table 4, which indicate a very low value of the crushing strength for the side timber of series d 1-5, while the values of the shear strength and cleavage strength of the timber of this series (70 kgf/cm^2 and 38 kgf/cm respectively) are lower than the corresponding values of the middle timber of series b 6-10 (82 kgf/cm^2 and 43 kgf/cm respectively).

More particulary the difference in crushing strength (average 278 kgf/cm^2 for the side timber of series d 1-5, and 349 kgf/cm^2 for the middle timber of series b 6-10) appears important, since the crushing strength of these joints will in most cases be the determining property, even though crushing does in a number of cases initiate splitting or cracking and perhaps also shearing.
The two series with ½" bolts and λ = 4.6 differ drom each other in the width of timber, in the washer dimensions and in the bolt hole diameter. There is virtually no difference in failure load. The form of failure displayed by the specimens does differ, however, this probably being due to the difference in crushing strength (as indicated in Table 4) of the middle timber in the two series (average 372 kgf/cm^2 in series c 1-5, as against 313 kgf/cm^2 in series a 6-10), for practically equal values of the cleavage strength of the middle timber (34 and 35 kgf/cm).
The difference in timber width or bolt hole diameter was not expected to have a perceptible effect on the failure load. However, the differences in washer dimensions could, in the case of these fairly slender bolts in series a 6-10, result in more effective restraint and thus give rise to a higher failure load.
It is quite possible, however, that this increase in the "frictional contribution" to the failure load has been cancelled by the reduction in loadbearing capacity due to the lower crushing strength in series a 6-10.
The two series with bolts having a slenderness ratio λ = 3.0 give results which are in reasonably good agreement. This is evident from the fact that the average failure loads of the two series differ little when divided by λd^2: $p/\lambda d^2$ = 295 kgf/cm^2 for series a 1-5; $P/\lambda d^2$ = 326 kgf/cm^2 for series b 1-5. For further information see chapter V.

In the case of the joints with $\lambda = 3.0$ the specimens with $\frac{3}{4}$" bolts display larger displacements ($v_{0.4}$, $v_{0.6}$, $v_{0.8}$) than those with $\frac{1}{2}$" bolts, i.e., the displacement increases with bolt diameter.
The joints with $\frac{3}{4}$" bolts and $\lambda = 2.0$ display smaller displacements than those with $\frac{1}{2}$" bolts and $\lambda = 4.6$. This could mean that in this respect the slenderness ratio would be a more important factor than the bolt diameter. The number of tests is too small, however, to enable reliable conclusions to be drawn from them.
The effect of the diameter of the bolt hole upon the initial displacement is again manifest in Table 7. In this table the value a (which is the difference between the measured displacement ast 40% of failure load and the value $v_{0.4}$) is compared with the difference between average bolt hole diameter d_g and bolt diameter d_b (the clearance in the hole). As indicated in III.2, the hole diameter d_g is equal to the drill diameter plus 0.3 mm, while d_b is equal to the nominal diameter d minus 0.3 mm. In examining these figures the large amount of scatter should be borne in mind.

Table 7: Initial deformation "a" and existing clearance

Series	SB	d 1-5	b 6-10	b 1-5	a 1-5	a 6-10	c 1-5
d	inch	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$
drill diameter (nominal)	mm	20	20	20	14	14	13
$d_g - d_b$	mm	1.5	1.5	1.5	1.9	1.9	1.9
a	mm	2.0	0.9	1.9	2.0	2.0	-0.1
$a + v_{0.4}$	mm	2.9	2.0	3.3	2.9	3.8	1.9

Table 7 also indicates the value of $a + v_{0.4}$ per series, which has some relation to the displacement that can be expected to occur in a joint at working load. Entirely as expected, reduced clearance in the bolt hole results in less displacement: the magnitude of a as well as of $(a + v_{0.4})$ decreases.

As regards the type of failure of the joints it is to be noted that in the joints comprising ½" bolts with 40 mm x 40 mm x 4 mm washers these washers cut into the wood at the edges and became tilted in consequence (see Photo 8). The larger washers used in otherwise similar joints did not display this effect. The washers (large and small) were bent a little, but only slightly. The shape of the bolt in the joints with bolts of slenderness ratio $\lambda = 4.6$ was as shown in Fig. 4 and in Photo 1, where the "plastic hinges" at B had as yet hardly developed.

Photos 3 to 6 show joints with ¾" bolts and ½" bolts of slenderness ratio $\lambda = 3.0$. It is clearly seen that the bolt remains practically straight. The deformation in the middle timber is found always to be substantially greater than that in the side timber, although in both cases the sum of the thicknesses of the two side timbers is equal to the thickness of the middle timber.

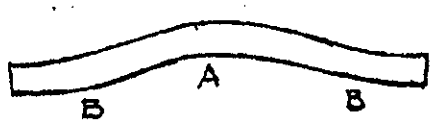


Fig. 4: Shape of the bolt in joints with $\lambda = 4.6$.

IV.2. Straight multiaxial joints

Two groups of multiaxial straight joints were tested: the specimens SBc with ½" bolts and $\lambda = 4.6$ and the specimens SBd with ¾" bolts and $\lambda = 2.0$. For the dimensions see Table 2. The results of all the individual joints are presented in Appendix 3. The average results per series are summarized in Table 8 for the joints with ½" bolts and in Table 9 for those with ¾" bolts. For comparison, the results for the uniaxial joints belonging to the group were likewise mentioned.

Table 8: Results for ½" bolted joints SBc ($\lambda = 4.6$)

Series		SB	c 1-5	c 6-10	c 11-15	c 16-20	c 21-25
number of axes			1	2	2	4	4
positioning			o	oo	o°	oo oo	oooo
Failure load: average per series		kgf	2686	5332	4897	9224	6917
coefficient of variation			3%	8%	5%	7%	21%
Displacements:							
$e_{0.4}$	average	mm	0.14	0.19	0.32	0.33	0.32
	coef. of var.		27%	8%	19%	9%	20%
$v_{0.4}$	average	mm	1.93	1.57	2.08	1.36	1.32
	coef. of var.		17%	7%	22%	8%	11%
$v_{0.6}$	average	mm	3.00	2.76	3.98	2.69	2.05
	coef. of var.		14%	7%	18%	1%	8%
$v_{0.8}$	average	mm	5.13	4.71	7.12	4.68	3.23
	coef. of var.		12%	9%	17%	7%	7%
a	average	mm	-0.07	0.29	1.16	1.28	0.71
	coef. of var.		38%	92%	30%	16%	40%
Type of failure*): Crushing only			-	-	-	-	-
cracking, splitting			5m	1m 5z	3m 4z	2m 4z	3m 2z
shearing			2m	2m 1z	2z	3m	1z
shape of bolt			bent	bent	bent	bent	bent

*) The number of times that a phenomenon occurs is stated; more than one phenomenon may occur in one specimen (cf. Appendix 3).

Table 9: Results for $\frac{3}{4}$" bolted joints SBd (λ = 2.0)

Series			d 1-5	d 6-10
number of axes			1	4
positioning			o	oooo
Failure load: average per series		kgf	2135	9188
coefficient of variation			11%	16%
Displacements:				
$e_{0.4}$	average	mm	0.17	0.27
	coef. of var.		10%	9%
$v_{0.4}$	average	mm	0.92	0.86
	coef. of var.		22%	28%
$v_{0.6}$	average	mm	1.32	1.38
	coef. of var.		20%	15%
$v_{0.8}$	average	mm	2.32	2.10
	coef. of var.		22%	12%
a	average	mm	2.02	2.02
	coef. of var.		14%	30%
Type of failure*): crushing only			-	-
cracking, splitting			2m 2z	1m 4z
shearing			2z	1m 5z
tensile fracture			-	1z
shape of bolt			straight	straight

*) See footnote to Table 8.

In the failure load is divided by the number of bolts in the joint concerned, and if the average failure load of the straight uniaxial joint is taken as equal to unity, the ratios presented in Table 10 are obtained. The number of axes successively occurring in the direction of the force (j) is also stated in this table.

Table 10: Effect of the number of axes on the failure load

λ	4.6				2.0
Series	c 6-10	c 11-15	c 16-20	c 21-25	c 6-10
number of axes	2	2	4	4	4
arrangement	oo	o^{o}	oo oo	oooo	oooo
average failure load series/ average failure load 1-a	0.99	0.91	0.86	0.64	1.08
j	2	2	2	4	4

In determining this, the value j = 2 for $d_{//} > 2\, d_{\perp}$ has been adopted for a biaxial joint, in analogy with the rule for ring connectors (see Fig. 5). In the case of the slender bolts there is found to be reduction in the strength (loadbearing capacity) of the joint; in the case of the bolts with λ = 2.0 this is not so.

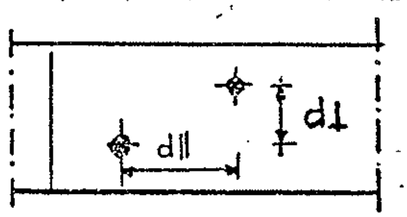


Fig. 5.

The displacements of the various joints do not differ much from one another. With the slender bolts there would seem to be some reduction of the values of v in the quadriaxial joints. Against this, the value of a for these joints is somewhat higher. With the $\frac{3}{4}$" bolts there is very good agreement between the values of v and a of uniaxial and quadriaxial joints. Why there should be a fairly large difference in $e_{0.4}$ is not clear. Some values are compared in Table 11. See Table 7 for the meaning of these data.
Some idea of the degree of co-operation of the bolts in a multiaxial joint is given by the photographs of the quadriaxial joints in series SBc 21-25 and series SBd 6-10. In the case of SBc 21-25 ($\frac{1}{2}$" bolts with $\lambda = 4.6$) the shape of the bolts located one behind the other was photographed after failure of the joint. The extent to which the bolts is bound up with the magnitude of the failure load. In the most cases there is fairly good agreement in shape between the four bolts of one and the same joint. As appears from Photo 10, there is some difference between the ultimate bolt hole dimensions - more than in the joints in series SBd 6-10 ("" bolts with $\lambda = 2.0$), as Photo 12 shows. All the bolts in SBd 6-10 remained practically straight. The conclusion that the bolts - at least towards the end of the test - participate in not too widely differing a manner in the transmission of force would appear justified for series SBd 6-10. but less to for series SBc 21-25. The values found for the failure loads (as compared with those for the uniaxial joints) confirm this.

<u>Table 11</u>: Initial deformation "a", direct displacement $v_{0.4}$ and clearance.

Series	SB	c 1-5	c 6-10	c 11-15	c 16-20	c 21-25	d 1-5	d 6-10
d	inch	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{3}{4}$
Number of axes		1	2	2	4	4	1	4
arrangement		○	○○	○○	○○ ○○	○○○○	○	○○○○
drill diameter (nominal)	mm	13	13	13	13	13	20	20
a	mm	-0.1	0.3	1.2	1.3	0.7	2.0	2.0
$d_g - d_b$	mm	0.9	0.9	0.9	0.9	0.9	1.5	1.5
$a + v_{0.4}$	mm	1.9	1.9	3.2	2.6	2.0	2.9	2.9

IV.3. Oblique joints

All the oblique joints are formed with ½" bolts. A joint of this kind comprises (see fig. 6): a 47 mm thick leg, a 58 mm thick obliquely loaded member, a 95 mm thick straight loaded central member, a 58 mm thick obliquely loaded member, and a 47 mm thick leg. Thus each specimen is symmetrical with respect to a plane through the middle of the straight loaded central member. On either side of this plane of symmetry there is a joint comprising an obliquely loaded middle timber (58 mm thick) and straight loaded side timber (47 mm thick). The slenderness ratio of the bolt is therefore $\lambda = 58/12.7 = 4.6$.

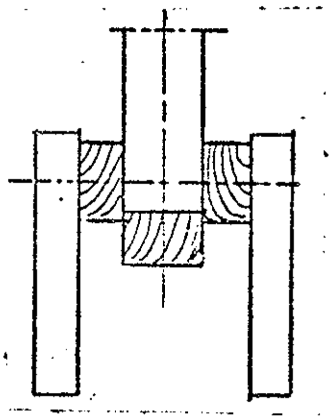


Fig. 6: Rear view of oblique joint

For comparison the series SBc 1-5 and 11-15 may be condidered; these comprise straight unixial and biaxial joints in which only the thickness of the side timber and the timber width differ from the corresponding dimensions in the SBe specimens. For these and other dimensions see Table 2.

The results of the tests are given in Appendix 4, while the average results per series are summarized in Table 12. It is to be noted that in the case of the oblique joints the values of v and e stated here have been calculated from measurements performed with dial gauges indicating the displacements of the straight loaded central (compression) member in its axial direction in relation to the obliquely loaded tension members (see Fig. 7: dial gauges d). In addition, the displacement of the obliquely loaded tension members in their axial direction in relation to the central member was measured (Fig. 7: dial gauge c).

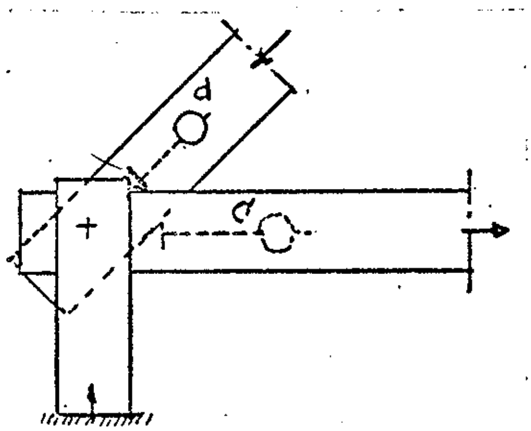


Fig. 7: Measurement of displacements in oblique joints.

Table 12: Results for oblique joints

Series		SB	c 1-5	e 1-5	e 11-15	c 11-15	e 6-10	e 16-20
load angle		α	0°	30°	60°	0°	30°	60°
member of axes			1	1	1	2	2	2
Failure load: average per series		kgf	2686	2903	3450	4897	5592	5698
Coefficient of variation			3%	3%	7%	5%	3%	5%
Displacements:								
$e_{0.4}$	average	mm	0.14	0.39	0.59	0.32	0.33	0.36
	coef. of var.		27%	14%	33%	19%	12%	29%
$v_{0.4}$	average	mm	1.93	2.37	4.39	2.08	2.31	3.89
	coef. of var.		17%	21%	26%	22%	11%	13%
$v_{0.6}$	average	mm	3.00	5.30	9.06	3.98	4.73	7.70
	coef. of var.		14%	14%	17%	18%	9%	10%
$v_{0.8}$	average	mm	5.13	10.42	13.16	7.12	10.09	12.88
	coef. of var.		12%	11%	14%	17%	9%	6%
a	average	mm	-0.07	-0.39	-0.95	1.16	-0.35	-0.67
	coef. of var.		38%	57%	27%	30%	47%	12%
Type of failure*: crushing only			-	1 b.v. 2	4	-	-	-
cracking, splitting			5m	2m	1z	3m 4z	4m	5m
shearing			2m	1m	-	2z	4m	-
shape of bolt			bent	bent	bent	bent	bent	bent

*) See footnote to Table 8
b.v. = failure occurred outside the joint

Based on the measurements obtained with gauge c, the mean values and coefficient of variation for e and v are given in Table 13. For $\alpha = 30^\circ$ ir appears that tha ratio between the displacements measured with the gauges c and d, theoretically linked by $c = d \cos \alpha$, has an average value of 0.77 for the uniaxial as well as the biaxial joints.

Table 13: Displacements measured at gauges c

Series SB			e 1-5	e 11-15	e 6-10	e 16-20
load angle			30°	60°	30°	60°
number of axes			1	1	2	2
Displacements:						
$e_{0.4}$	average	mm	0.30	0.13	0.28	0.06
	coef. of var.		20%	28%	12%	57%
$v_{0.4}$	average	mm	1.87	1.42	1.81	1.53
	coef. of var.		22%	24%	14%	12%
$v_{0.6}$	average	mm	3.68	2.80	3.70	2.65
	coef. of var.		22%	22%	13%	9%
$v_{0.8}$	average	mm	8.34	4.64	7.3	4.08
	coef. of var.		9%	35%		6%

This would correspond to cos 40° instead of actually cos 30° = 0.87. Again, for α = 60° the average value of the ratio is 0.33, corresponding to cos 71° instead of actually cos 60° = 0.5. In the joints with α = 60° a rotation of the member in relation to one another was measured, so that α, also in the biaxial joints, underwent an increase of about 7°. In the biaxial joints with α = 30° no such rotation was measured. The difference between the theoretical and the actual value of the ratio of the measurements obtained with the gauges c and d is in part attributable to this rotation; the rest of the differences is presumably due to a translational displacements of the tension member in the vertical direction (upwards).

The average resultant direction of displacement of the tension member in relation to the central compression member is approximately: $\beta = 40^{\circ}$ for $\alpha = 30^{\circ}$, and $\beta = 70^{\circ}$ for $\alpha = 60^{\circ}$ (see Photos 17 and 18). The figures given for the oblique joints in Table 12 are again given in Table 14, but now as ratios with respect to the corresponding straight joints.

Table 14: Effect of the load angle

Series SB	e 1-5	e 11-15	e 6-10	e 16-20
α	30°	60°	30°	60°
number of axes	1	1	2^{*}	2^{**}
P_{br}	1.08	1.28	1.14	1.16
$e_{0.4}$	2.8	4.2	1.03	1.13
$v_{0.4}$	1.23	2.28	1.11	1.87
$v_{0.6}$	1.77	3.02	1.19	1.93
$v_{0.8}$	2.03	2.57	1.42	1.81

* $j = 2$;
** $j = 1$ ($d_{//} < 2 \cdot d_{\perp}$)

The ratios for the failure load P_{br} are 1.08 and 1.14, average 1.11, for $\alpha = 30^{\circ}$, while they are 1.28 and 1.16, average 1.22, for $\alpha = 60^{\circ}$. The values found for $e_{0.4}$ were widely divergent. The uniaxial joints here compare very unfavourably with the biaxial joints. The values for $v_{0.4}$ of the joints with $\alpha = 30^{\circ}$ are, on average, 1.17 times the corresponding values of the straight joints.

Since the initial deformations a of the oblique joints are smaller than those of the straight joints, as appears from Table 12, it can be presumed that, for equal loading, the directly occurring displacements of straight and oblique joint with $\alpha = 30^\circ$ are of equal magnitude. In the event of overloading the displacements increase more rapidly in the oblique than in the straight joints.
In the joints with $\alpha = 60^\circ$ the initial displacement a is even smaller, but the values $v_{0.4}$ have considerably increased. In this case there is a greater increase in the displacement than would be deduced from the higher failure load.
It is also to be noted that the failure loads of the oblique joints are the loads at 15 mm displacement (average of the measurements at dial gauges d); with further continuation of the test in most cases there was some further (small) increase in load. In the case of the joints with $\alpha = 60^\circ$ the test was, especially with the uniaxial joints, somewhat complicated by the rotation of the members in relation to one another, including more particularly the tilting of the legs (vertical members). Because of these difficulties, it was not always possible exactly to adopt the load at 15 mm displacement as the measured failure load; The resulting differences are not of any substantial magnitude, however.
The shape of the bolts in the joints is as shown in Photos 16 and 17; doubly curved in each side of the plane of symmetry through the middle, corresponding to the shape to be expected with $\lambda = 4.6$ and the side timber thickness employed (see further Section V). The washers in these joints remained flat, but did penetrate deep into the timber (Photo 14), locally destroying it. This indicates that the thickness of the washers was sufficient, but too great in relation to the other dimensions.
Photos 18 and 19 show the deformation of the obliquely loaded timber of the horizontal tension members for loading at 60° and 90° respectively.

V Discussion of the results

V.1. Summary of the theory relating to the magnitude of the failure load

The theoretical determination of the distribution of force in bolted conntections and the calculation of the failure load, as set forth in report B 1, will be briefly summarized below, using the following notation:

- $\sigma_{s\ max}$ - crushing strength of the timber;
- σ_v - (real) yield point of the bolt; if determined from a three-point flexural test, the full plastic moment should be adopted: $\sigma_v = M/W_{pl}$, where $W_{pl.} = 1.7\ W = 0.167\ d^3$; $S = \sqrt{\sigma_v/\sigma_{s\ max}}$
- λ - slenderness ratio of the bolt: $\lambda = m/d$;
- ψ - ratio of thicknesses of side and middle timber: $\psi = z/m$;
- μ - timber-om-timber coefficient of friction;
- γ - factor indicating the extent to which axial forces may occur in the bolt; together with μ, this factor determines the magnitude of P_w;
- w - factor indicating the degree of restraint (fixity) of the bolt in the side timber: $w = 0$ corresponds to non-restraint, $w = 1$ to fully effective restraint;
- P_v - that portion of the failure load which is contributed by forces perpendicular to the axis of the bolt;
- P_w - that portion of the failure load which is contributed by timber-on-timber friction, i.e., by the axial force in the bolt;
- P_{br} - failure load (ultimate load) of the joint: $P_{br} = P_v + P_w$.

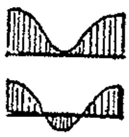
The data given in Chapters IV, V and X of Report B 1 are reproduced here in tabular form. Table 15 gives the magnitude of the failure load according to Johansen's theory; Tables 16 and 17 give the corresponding information according to Fahlbusch and Norén respectively. These data relate to joints comprising two side timbers and one middle timber connected by one bolt.

Table 15: Theoretical failure load per bolt, according to Johansen

shape of bolt	valid for			$P_{br} = P_v + P_w;\ P_w = \gamma \cdot \sigma_v \cdot \mu d^2$	
	* ψ	λ	w	P_v	γ
1. —	$\geqq 0.5$	$\lambda \leqq \frac{1.63}{\sqrt{(3-2\psi)(1+2\psi)}} \cdot S$		$\sigma_{s\ max} \cdot \lambda d^2$	$\gamma = 0$
	$\leqq 0.5$	$\lambda \leqq \frac{0.41}{\psi} \cdot S$		$\sigma_{s\ max} \cdot 2\psi\lambda d^2$	
2. ⌒	$\geqq 0.5$	$\frac{1.63}{\sqrt{(3-2\psi)(1+2\psi)}} \cdot S \leqq \lambda \leqq \frac{1.4}{\psi} S$	w = 0	$\frac{2}{3}\sigma_{s\ max}\psi\lambda d^2 (2\sqrt{1+\frac{0.5}{\psi^2\lambda^2} \cdot S^2} - 1)$ approximation $(0.19\ \sigma_{s\ max}\ \psi^2\lambda^2 + 0.78\ \sigma_v) d^2 \cdot \frac{1}{S}$	$\gamma \geqq 0$
	$\leqq 0.5$	$\frac{0.41}{\psi} \cdot S \leqq \lambda \leqq \frac{0.41}{\psi} \cdot S$			
3. ∩	$\geqq 0.5$	$\lambda \geqq 1.15 \cdot S$	w = 1	$1.15\ d^2 \sqrt{\sigma_v \cdot \sigma_{s\ max}}$	$\gamma \geqq 0$
	$\leqq 0.5$	$\lambda \geqq \frac{0.58}{\psi} \cdot S$			
	all	$\lambda \geqq \frac{1.4}{\psi} \cdot S$			

* $\psi \geqq 0.5$: $m \leqq 2z$, therefore m is decisive; $\psi \leqq 0.5$: z decisive

Table 16: Theoretical failure load per bolt, according to Fahlbusch

distribrution of crushing pressure along the bolt	valid for λ	$P_{br} = P_v + P_w;\ P_w = \gamma\, \sigma_v\ \mu\, d^2$ **	
		P_v	γ
	$\lambda \leqq 1.6 \cdot S$	$\dfrac{\sigma_{s\ max}}{1 + \dfrac{\lambda^4}{6.75} \cdot \dfrac{1}{S^4}} \cdot \lambda\, d^2$	$\gamma \geqq 0$
	$\lambda \geqq 1.6 \cdot S$	$0{,}8\ d^2 \sqrt{\sigma_v \cdot \sigma_{s\ max}}$	$\gamma > 0$

** Fahlbusch proposes that, to be on the safe side, P_w should not be taken into account in determining the permissible load.

Table 17: Theoretical failure load per bolt, according to Norén

	valid for		P_{br}
	ψ	λ	
1.	$\geqq 0.5$	$\lambda \leqq 1.4 \cdot S$	$\sigma_{s\ max} \cdot \lambda\ d^2$
	$\leqq 0.5$	$\lambda \leqq \dfrac{0.7}{\psi} \cdot S$	$\sigma_{s\ max} \cdot \psi\, \lambda\, d^2$
2.	$\geqq 0.5$	$\lambda \geqq 1.4 \cdot S$	$1.4\ d^2 \sqrt{\sigma_v \cdot \sigma_{s\ max}}$
	$\leqq 0.5$	$\lambda \geqq \dfrac{0.7}{\psi} \cdot S$	

If different values are found for the crushing strengths of the middle and the side timber, the values of $\sigma_{s\ max}$ to be substituted into the formulas can, according to Norén. be determined from:

$$\sigma_{s\ max\ eq} = \frac{2 \cdot \sigma_{s\ max\ m} \cdot \sigma_{s\ max.z}}{\sigma_{s\ max\ m} + \sigma_{s\ max.z}}$$

V.2. Comparison of experimental results with the theory

The results of tests on uniaxial straight joints given in Table 6 will now be compared with the failure loads predicted on the basis of the above-mentioned theories.

Series SBd 1-5: $\lambda = 2.0;\ \psi = 0.5;\ d = 19.1$ mm

$$\sigma_{s\ max\ eq} = \frac{2.361.278}{361 + 278} = 314\ \text{kgf/cm}^2$$

$$\sigma_v = 2920\ \text{kgf/cm}^2$$

$$S = \sqrt{\frac{2920}{314}} = 3.05$$

Johansen: $\lambda = 2.0 < \frac{0.41}{0.5}.3.05$

$$P_v = 314.2.0.1.91^2 = 2290\ \text{kgf}.P_w = 0$$

Fahlbusch: $\lambda < 1.6.3.05$

$$P_v = \frac{314}{1 + \frac{2.04}{6.75}.\frac{1}{3.05^4}}.2.0.1.91^2 = 2230\ \text{kgf}.P_w = 0$$

Norén: same as Johansen

Test result: $P_{br} = 2135$ kgf

We similary obtain:

Series SBb 6-10: $\lambda = 2.0$; $\psi = 0.66$; $s = 2.86$

Johansen $P_{br} = 2610$ kgf Norén same

Fahlbusch $P_{br} = 2520$ kgf

Test result $P_{br} = 2850$ kgf

Series SBa 6-10: $\lambda = 3{,}0$; $\psi = 0{,}5$; $S = 2{,}70$

Johansen $P_v = 1260$ kgf $P \approx 0$

Fahlbusch $P_v = 1390$ kgf $P \approx 0$

Norén $P_{br} = 1690$ kgf

Test result $P_{br} = 1428$ kgf

Series SBb 1-5: $\lambda = 3{,}0$; $\psi = 0{,}5$; $S = 2{,}85$

Johansen $P_v = 3100$ kgf $P_w \approx 0$

Fahlbusch $P_v = 3320$ kgf $P_w \approx 0$

Norén $P_{br} = 3950$ kgf

Test result $P_{br} = 3572$ kgf

Series SBc 1-5: $\lambda = 4.6$; $\psi = 0{,}5$; $S = 2{,}6$

Johansen ($w < 1$, therefore analysis method 2): $P_v = 1450$ kgf

$P_w = 4110\ \gamma\mu$ kgf

Fahlbusch $P_v = 1260$ kgf $P_w = 4110\ \gamma\mu$ kgf

Norén $P_{br} = 2210$ kgf

Test result $P_{br} = 2686$ kgf

Series SBa 6-10: $\gamma = 4.6$; $\psi = 0.5$; $S = 2.75$

Johansen $P_v = 1350$ kgf $P_w = 4110\ \gamma\mu$ kgf

Fahlbusch $P_v = 1200$ kgf $P_w = 4110\ \gamma\mu$ kgf

Norén $P_{br} = 2100$ kgf

Test result $P_{br} = 2633$ kgf

In Graph 4 the average failure loads, determined experimentally for the series with $\lambda = 2.0$ and $\lambda = 3.0$, have been plotted against the calculated values of $P_{br} = P_v + P_w$. There is found to be good agreement between experiment and theory.

For the two series with $\lambda = 4.6$ the situation is more complex in that here a factor $\gamma\mu$ occurs in the theoretical value of P_{br}. It emerges that good agreement between experimental and theoretical values is obtained on substitution of $\gamma\mu = 0.3$.

In the above treatment of the subject the average values of $\sigma_{s\ max}$ and σ_v for each series have always been employed. If constant values are adopted for these — here they have been chosen as $\sigma_{s\ max}$ = 350 kgf/cm^2 and σ_v = 2750 kgf/cm^2 (so that S = 2.8) — and if $\gamma\mu = 0.3$ is furthermore introduced, the theoretical failure loads will then have the values listed in Table 18.

In that table the average values of the test results have again been included, for comparison. These values have been plotted against one another in Graph 5. Graph 6 gives the failure loads of the individual joints, compared with the theoretical values according to Fahlbusch, as indicated in Table 18. The magnitude of the scatter (dispersion, represented by the coefficient of variation, has been calculated for this distribution and for the distributions of the individual joints compared with the theoretical values according to Johansen and according to Norén. This has been done separately for the 20 joints for which $\lambda = 2.0$ or 3.0 and for all the straight uniaxial joints, the object being to ascertain the favourable or unfavourable effect (if any) of the assumption $\gamma\mu = 0.3$. Table 19 gives the coefficients of variation and also the quotients of the average failure load determined experimentally and the average theoretical value.

Table 18: Comparison of theoretical and measured failure load ($\sigma_{s\ max}$ = 350 kgf/cm^2; σ_v = 2750 kgf/cm^2; $\gamma\mu$ = 0.3), in kgf.

λ	2.0		3.0		4.6	
Series	d 1-5	b 6-10	a 1-5	b 1-5	c 1-5	a 6-10
theoretical failure load according to:						
Johansen	2550	2550	1300	2950	2750	2750
Fahlbusch	2460	2460	1420	3210	2600	2600
Norén	2550	2550	1690	3830	2220	2220
average failure load from tests	2135	2858	1428	3572	2686	2633

Table 19: Comparison of theoretical and measured failure load ($\sigma_{s\ max}$ = 350 kgf/cm^2; σ_v = 2750 kgf/cm^2; $\gamma\mu$ = 0.3) and the magnitude of the coefficient of variation.

	λ = 2.0; 3.0 20 joints		λ = 2.0; 3.0; 4.6 30 joints	
	$\frac{\overline{P}_{test}}{\overline{P}_{br\ theor.}}$	coefficient of variation	$\frac{\overline{P}_{test}}{\overline{P}_{br\ theor.}}$	coefficient of variation
Johansen	1.07	16.9%	1.03	15.0%
Fahlbusch	1.03	15.1%	1.03	13.1%
Norén	0.93	14.6%	1.02	18.0%

It appears from Tables 18 en 19 that there is fairly good agreement between the various theoretical failure loads themselves and also between these and the values found experimentally. The failure loads calculated on the basis of Fahlbusch's theory tie up, on an average, most satisfactorily with the test results. In the following, only Fahlbusch's theory will therefore be used, but this choice must not be taken expressly to indicate a preference for the method of analysis to be adopted.

The data obtained from the tests on multiaxial and oblique joints have been compared with the theoretical failure loads according to Fahlbusch. For this purpose some corrections have been introduced on the basis of the results reported in Chapter IV.

Table 10 gives the effect of the number of axes on the failure load. On an average, it follows from this that for a joint of series c with two axes behind each other in the direction of the force ($j = 2$) the ratio of the average failure load for "multiaxial" to that for "uniaxial" is 0.92, which can suitably be rounded off to 0.9. For $j = 4$ this ratio is found to be 0.64, say 0.65. In series d for $j = 4$ the ratio is 1.08, say 1.0. In order to eliminate the effect of the number of axes, the respective failure loads have been divided by one of these ratio values. The same has been done with the failure loads of the oblique joints. Having regard to what has been said in IV.3, the failure loads of the joints with $\alpha = 30^\circ$ are divided by 1.11 and those of joints with $\alpha = 60^\circ$ by 1.22.

Graph 7 gives the histrograms of the ratios between the theoretical and the measured failure load of, successively, the straight uniaxial joints, the straight multiaxial joints, the oblique joints and all the joints together. The average measured failure load of all the specimens turns out to be 1.02 times the theoretical value calculated by Fahlbusch's analysis; the coefficient of variation is 12.5%.
On comparison of the results of the tests with the values obtained with Fahlbusch's theory, values of between 12 and 15% are found for the coefficient of variation of the failure load.
Since a value of 15% is found for a fairly homogeneous group of joints to which no corrections have been applied, as is apparent from Table 19, it would appear advisable to adopt this value for the coefficient of variation of the load.

Note

In report N-1*) a description is given of tests performed on straight uniaxial bolted joints with $d = \frac{1}{2}"$, $m = 70$ mm and $z = 18$ mm (series SBA 1 to 5). The results found in those tests have been checked against the theory outlined above.

$$\lambda = 70/12.7 = 5.5; \ \psi = 18/70 = 0.26.$$

$\sigma_{s\ max}$ and σ_v have not been seperately determined; assuming $\sigma_{s\ max} = 350$ kgf/cm^2 and $\sigma_v = 2750$ kgf/cm^2, we obtain $S = \sqrt{2750/350} = 2.8$.

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*)

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Johansen: $\lambda = 5{,}5 > \frac{0.41}{0.26} . 2.8 = 4.4$

$< \frac{1.4}{0.26} . 2.8 = 15.0$

$< \frac{0.58}{0.26} . 2.8 = 6.2$

$$P_v = \frac{2}{3} . 350.0.26.5.5.1.27^2 \; (2 \sqrt{1 + \frac{0.5}{0.26^2.5.5^2} . 2.8^2} - 1) = 1300 \text{ kgf}$$

$$P_w = \gamma.2750.\mu.\ 1.27^2 = 4440\ \gamma.\mu.\text{kgf}. \quad P_{br} = 1300 + 4440\,\gamma\,\mu \text{ kgf}$$

Fahlbusch: $\lambda = 5.5 > 1.6 . 2.8 = 4.5$

$$P_v = 0.8 . 1.27^2 \sqrt{350.2750} = 1270 \text{ kgf}$$

$$P_w = 4440\ \gamma\,\mu \text{ kgf} \qquad P_{br} = 1270 + 4440\ \gamma\,\mu \text{ kgf}$$

Norén: $\gamma = 5.5 < \frac{0.7}{0.26} . 2.8 = 7.5$

$$P_{br} = 350 . 2 . 0.26 . 5.5 . 1.27^2 = 1610 \text{ kgf}$$

Test result: $P_{br} = 3040$ kgf

Hence it follows that for $P_w = 3040 - 1270 = 1770$ kgf $4440\,\gamma\mu$, the value of $\gamma\mu$ is 0.4. So with $\lambda\mu = 5.5$ the value found for $\gamma\mu$ is somewhat higher than with $\gamma\mu = 4.6$, which is reasonable.

V.3. Timber stresses, weakening of members

Various data relating to the joints tested are additionally given in Table 20, namely, at the failure load of the joint: the tensile stresses in the unweakened section of the middle timber (σ_{gross}) and in weakened section of the middle timber (σ_{net}), the force per mating surface at failure load (P_{br}/F_a)*, the area of the weakened section (F_{net}) and the extent of weakening ($F_{net}:F_{gross}$). By mating surface F_a is understood the area of contact of the connected members. Since $\psi > 0.5$ for all the joints, it will suffice to refer all the values to the middle timber, as this timber has a cross-section which is either equal to, or smaller than, that of the side timber.

Table 20: Timber stresses, weakening of members, and force per mating surface at failure load, average per series.

Series		σ_{gross} kgf/cm²	σ_{net} kgf/cm²	F_{net} cm²	$\frac{F_{net}}{F_{gross}}$	$\frac{P_{br}}{F_a}$ kgf/cm²
SB a	1- 5	39.5	45.6	31.3	0.87	4.3
	6-10	47,8	55.1	47.7	0.87	7.8
b	1- 5	51.3	61.1	58.5	0.84	5.5
	6-10	62.6	74.4	38.4	0.84	4.4
c	1- 5	61.0	73.3	36.7	0.83	10.0
	6-10	120.9	145.2	36.7	0.83	13.1
	11-15	82.8	94.6	51.8	0.87	9.8
	16-20	125.2	156.6	58.9	0.80	13.6
	21-25	93.5	104.4	66.3	0.90	6.1
d	1- 5	48.8	58.6	36.4	0.83	3.5
	6-10	166.8	192.3	47.8	0.87	4.7
e	1- 5	49.6	56.7	51.2	0.87	7.6
	6-10	80.4	89.9	62.3	0.89	9.7
	11-15	58.9	67.4	51.2	0.87	15.5
	16-20	73.8	91.3	62.4	0.81	13.9

*) This quantity is one measure for the effectiveness of joints

It is apparent from this table that uniaxial bolted joints give rise to very low stresses in the connected members; only multiaxial joints give serviceable results in this respect. The force to be transmitted per unit area of mating surface shows a similar pattern. It is to be noted that the values given in the table relate to failure load; in order to obtain the corresponding values for permissible load, all the values in the table must be divided by a "factor of safety". It emerges that the weakening of the members is between 10% and 20% of the gross section.

VI Conclusions

The following conclusions can be drawn from the results obtained from testing 75 bolted timber joints, namely 30 straight uniaxial, 25 straight multiaxial and 20 oblique joints, and from a comparison of these results with the theory described in Report B 1:

(1) The timber (fir) employed for the straight joints is found to have an average crushing strength $\sigma_{s\ max} = 367$ kgf/cm^2; a design value of 350 kgf/cm^2 has been adopted.

(2) The bolts employed in the test specimens have a yield point σ_v averaging 2540 and 2920 kgf/cm^2 for $\frac{1}{2}$" and $\frac{3}{4}$" bolts respectively; a value of $\sigma_v = 2750$ has been adopted.

(3) The failure loads of the straight uniaxial joints with $\lambda = 2.0$ and 3.0, in which the proportion P_w of the failure load transmitted by friction can be taken as zero, are in good agreement with the predicted values calculated with the theories of Johansen, Fahlbusch or Norén. Fahlbusch's appears to tie up most satisfactorily with the tests.

(4) In the straight uniaxial joints with $\lambda = 4.6$ there occurs yielding of the bolt, while the contribution by friction also plays a part in determining the failure load. With the assumption $\gamma\mu = 0.3$ a result is obtained that is in good agreement with the measured values.

(5) If the number of bolts located one behind the other in the direction of the force in a straight multiaxial joint is j, the failure load per bolt in such a joint is found to be, on an average, egual to δ times the failure load of a uniaxial joint; the values given in Table 21 have been found for this coefficient δ.

Table 21: Reduction coefficient δ for multiaxial straight joints.

j	2	4
$\gamma = 2.0$	–	1.0
$\gamma = 4.6$	0.9	0.65

(6) The effect of the angle α of the load upon the failure load of joints with $\lambda = 4.6$ is, irrespective of the number of axes per joint, equivalent to an average failure load increase by a factor 1.11 for $\alpha = 30^{\circ}$ and of 1.22 for $\alpha = 60^{\circ}$ as compared with corresponding straight joints.

(7) The coefficient of variation of the failure load, determined from the tests, is about 15%.

(8) The effect of the diameter of the bolt holes upon the initial displacement a is appreciable; it is advisble to use a drill whose nominal diameter is the same as the nominal diameter of the bolts.

(9) The displacements occurring under load are large; inclusive of initial displacements their magnitude at 40% of failure load is between 2 and 4 mm (depending on the difference between bolt diameter and hole diameter). The effect of the number of axes upon this displacement is not demonstrable, but the effect of the load angle is manifest: in the case of oblique joints with $\alpha = 30^{\circ}$ the displacements are not yet increased whereas for $\alpha = 60^{\circ}$ there is a substantial increase.

Delft, March 1963
ir. P. Vermeyden

Appendix 1: Summary of timber properties.

Timber of specimen no.	middle or side timber	compressive strength $\sigma_{d\ max}$		crushing strength $\sigma_{s\ max}$		shear strength τ_{max}		cleavage strength s_{max}	
		average kgf/cm^2	coefficient of variation	average kgf/cm^2	coefficient of variation	average kgf/cm^2	coefficient of variation	average kgf/cm^2	coefficient of variation
SBa 1- 5	m z	530 540	10% 14%	331 364	10% 12%	70.5 66.4	16% 10%	33.7 36.8	11% 9%
SBa 6-10	m z	383 448	12% 9%	313 369	13% 11%	67.5 70.3	11% 17%	33.8 41.1	12% 17%
SBb 1- 5	m z	560 535	11% 10%	364 358	15% 19%	70.8 83.1	13% 16%	35.4 35.1	13% 14%
SBb 6-10	m z	501 495	8% 13%	349 367	15% 12%	82.2 71.2	17% 20%	43.4 40.1	13% 13%
SBc 1-5	m z	481 518	6% 8%	372 382	7% 7%	83.0* 83.3	20% 22%	35.4* 43.8	8% 15%
SBc 6-10	m z	643 597	11% 8%	412 348	8% 14%	86.0 87.7	9% 9%	41.6 42.7	6% 13%
SBc 11-15	m z	524 507	5% 16%	359 322	11% 20%	77.2 78.3	6% 18%	36.6 35.5	7% 20%
SBc 16-20	m z	528 585	7% 16%	346 434	4% 10%	65.6 83.2	9% 10%	33.2 38.5	11% 9%
SBc 21-25	m z	487 625	8% 5%	353 401	9% 6%	72.0 67.1	16% 11%	39.0 40.1	32% 17%
SBd 1- 5	m z	452 404	2% 4%	361 278	3% 6%	79.8 69.6	22% 14%	37.0 38.4	16% 11%
SBd 6-10	m z	645 501	7% 7%	453 307	11% 9%	78.6 78.9	20% 9%	38.2 36.1	18% 14%

*) SBc 3 not included

Appendix 2: Results of tests on straight uniaxial joints.

Specimen no. SB	failure load kgf	$e_{0.4}$ mm	$v_{0.4}$ mm	$v_{0.6}$ mm	$v_{0.8}$ mm	a mm	type of failure
a 1	1350	0.15	0.55	1.08	1.97	2.80	(O + S) m
2	1380	0.18	1.02	1.50	2.48	1.79	Sm
3	1410	0.18	0.96	1.52	2.42	1.71	Sm
4	1520	0.18	0.93	1.38	2.18	2.10	Om
5	1480	0.22	0.90	1.43	2.49	1.80	Om
a 6	2975	0.49	2.38	4.73	6.93	1.17	Om
7	2400	0.34	2.26	4.72	7.53	2.07	Om
8	2750	0.18	0.81	2.38	4.57	1.94	Om
9	2290	0.31	2.08	4.26	7.22	2.23	Om
10	2750	0.21	1.79	4.20	6.71	2.34	Om
b 1	3180	0.19	1.53	2.34	3.68	0.84	(A + S) m
2	3760	0.22	1.74	2.43	3.75	1.43	Om (+ Sm)
3	3560	0.24	1.15	1.82	3.01	0.67	Om
4	4150	0.23	1.06	1.82	2.87	3.07	Am
5	3210	0.26	1.39	2.29	3.92	3.42	Om
b 6	3490	0.22	0.65	0.96	1.62	1.74	Am
7	2760	0.19	0.86	1.28	2.06	0.26	Am
8	2760	0.17	1.32	1.89	2.88	0.93	Om
9	2450	0.19	1.42	1.72	3.76	0.94	Om
10	2830	0.17	1.21	1.68	2.44	0.72	Sm
c 1	2725	0.10	1.96	2.96	5.04	-0.43	Sm
2	2610	0.16	2.20	3.41	5.85	-0.23	Sm
3	2610	0.18	2.26	3.44	5.61	-0.23	(S + A) m
4	2745	0.16	1.72	2.66	4.47	0.08	(S + A) m
5	2740	0.09	1.50	2.54	4.66	0.47	Sm
d 1	1890	0.18	1.17	1.59	2.52	2.11	Az
2	2210	0.15	0.82	1.11	1.85	2.06	(A + S) zz
3	2485	0.18	0.71	1.07	1.96	1.63	S m
4	2000	.	.	.	.	.	S m
5	2090	0.18	0.97	1.49	2.93	2.29	S z

* Key: A = shearing
S = cracking, splitting
O = crushing
T = tensile fracture

of the (m = middle timber
z = side timber)

Appendix 3: Results of tests on straight multiaxial joints

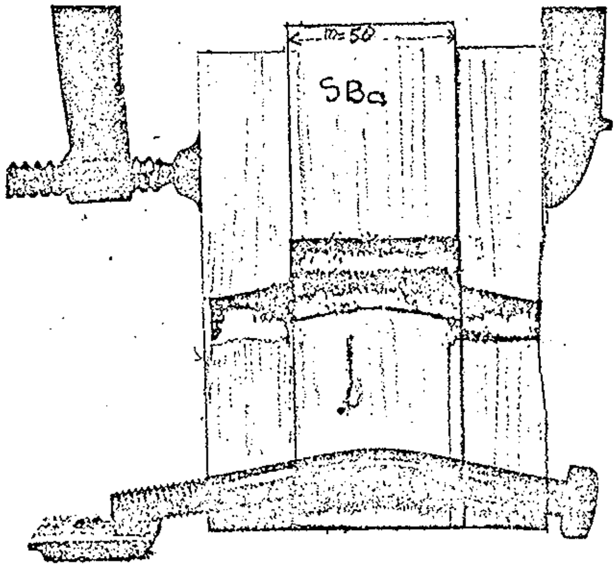
Specimen no. SB	failure load kgf	$e_{0.4}$ mm	$v_{0.4}$ mm	$v_{0.6}$ mm	$v_{0.8}$ mm	a mm	Type of failure
c 6	5640	0.17	1.55	2.52	4.13	0.18	S m zz
7	5000	0.21	1.40	2.63	4.69	0.75	S zz
8	5280	0.20	1.69	2.88	4.76	0.19	A m Sz
9	4860	0.20	1.62	2.96	5.31	0.23	S zz
10	5880	0.20	1.61	2.81	4.67	0.09	A mz Sz
c 11	4900	0.36	1.85	3.66	6.27	0.88	S m z
12	4900	0.34	2.61	4.83	8.60	1.14	(A + S)z, Sz
13	5000	0.31	2.21	4.05	7.61	1.37	S m
14	4515	0.35	2.31	4.34	•	0.80	(A + S) z
15	5170	0.21	1.41	2.99	6.00	1.64	S m z
c 16	8350	0.30	1.47	2.74	4.69	1.47	S m zz
17	9760	0.36	1.43	2.70	5.18	1.04	A m Szz
18	8730	0.32	1.19	2.66	4.68	1.42	A m
19	9550	0.31	1.35	2.67	4.31	1.39	S m z
20	9730	0.36	1.34	2.68	4.56	1.04	A m Sz
c 21	6625	0.40	1.49	2.25	3.23	0.53	A zz
22	6270	0.29	1.12	1.86	3.12	1.22	S m
23	7660	0.37	1.31	2.15	3.56	0.62	S zz
24	5050	0.24	1.44	2.07	•	0.52	S m
25	8980	0.30	1.25	1.92	3.01	0.67	S m zz
d 6	9840	0.30	1.24(?)	1.62	2.47	1.27	(A + S)m zz
7	9800	0.25	0.72	1.20	1.97	2.39	A z
8	10000	0.23	0.61	1.20	1.89	2.30	(A + S)zz
9	9800	0.27	0.87	1.32	2.08	1.48	(A + S)z
10	6500	0.27	0.86	1.58	•	2.62	Tz,(A + S)z

* See footnote to Appendix 2.

Appendix 4: Results of tests on oblique joints.

Specimen no. SB	failure load kgf	$e_{0.4}$ mm	$v_{0.4}$ mm	$v_{0.6}$ mm	$v_{0.8}$ mm	a mm	Type of failure
e 1	3040	0.32	1.51	4.01	9.10	-0.37	0
2	2870	0.40	2.52	5.45	9.46	-0.07	Sm
3	2840	0.44	2.79	5.73	11.45	-0.60	buiten verb.
4	2810	0.35	2.68	5.89	11.50	-0.60	(S + A)m
5	2890	0.44	2.33	5.41	10.58	-0.30	0
e 6	5420	0.36	2.38	4.74	9.80	-0.13	(S + A)mm
7	5400	0.37	2.70	5.36	11.47	-0.57	A mm
8	5700	0.29	2.23	4.68	9.78	-0.28	(S + A)mm
9	5650	0.28	2.19	4.71	10.40	-0.35	Am, Sm
10	5790	0.36	2.04	4.16	9.01	-0.41	S mm
e 11	3380	0.66	5.46	10.52	13.18	-1.18	0
12	3630	0.37	2.67	7.07	10.48	-0.66	0
13	3800	0.69	3.81	7.80	12.45	-0.94	0
14	3250	0.83	5.14	10.33	15.39	-1.23	Szz
15	3190	0.42	4.89	9.56	14.30	-0.75	0
e 16	5540	0.42	4.05	8.16	13.47	-0.68	Sm
17	5920	0.23	3.10	6.57	12.21	-0.76	Sm
18	5660	0.49	3.96	8.04	12.93	-0.66	Sm
19	5320	0.35	4.43	8.50	13.69	-0.54	Sm
20	6050	0.30	3.92	7.22	12.09	-0.71	Sm

* See footnote to Appendix 2



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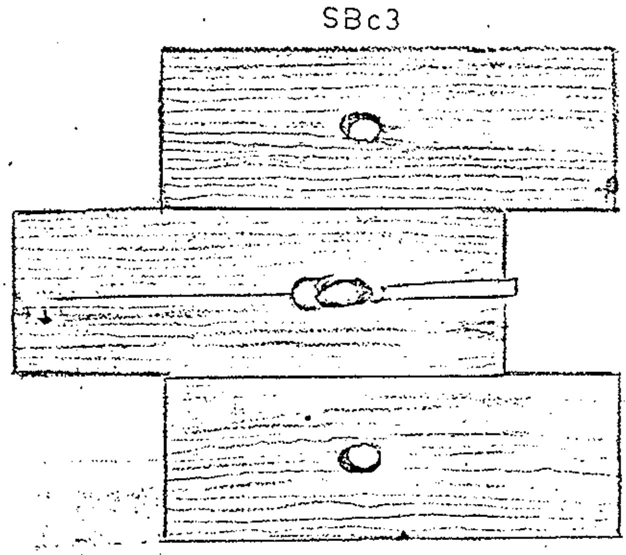

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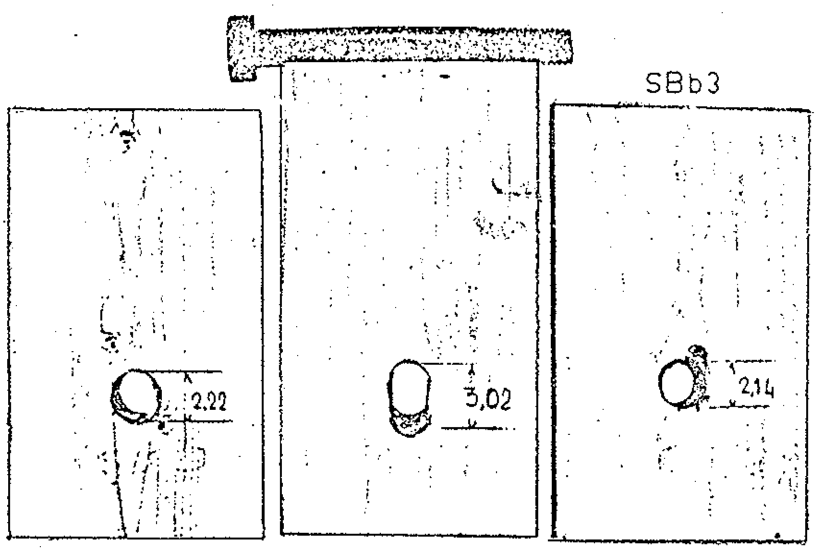

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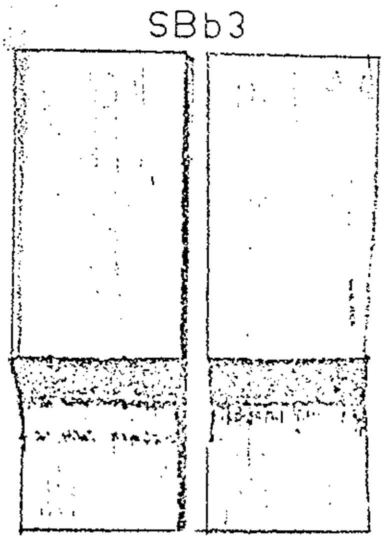

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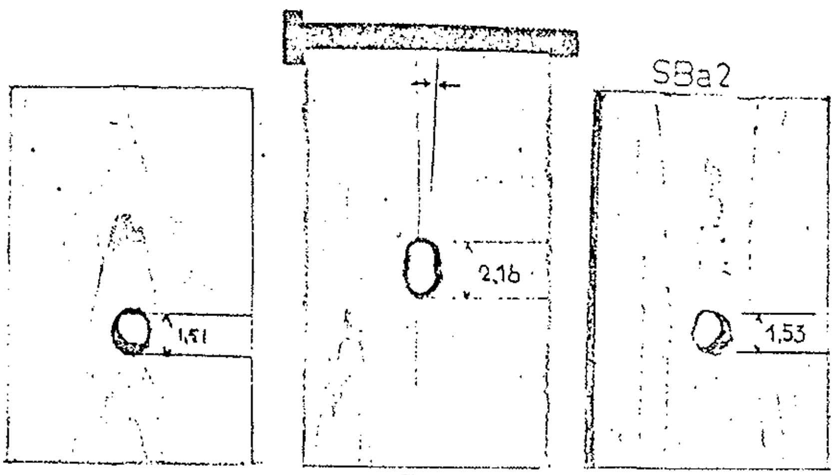

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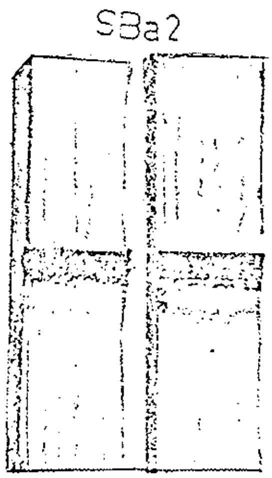

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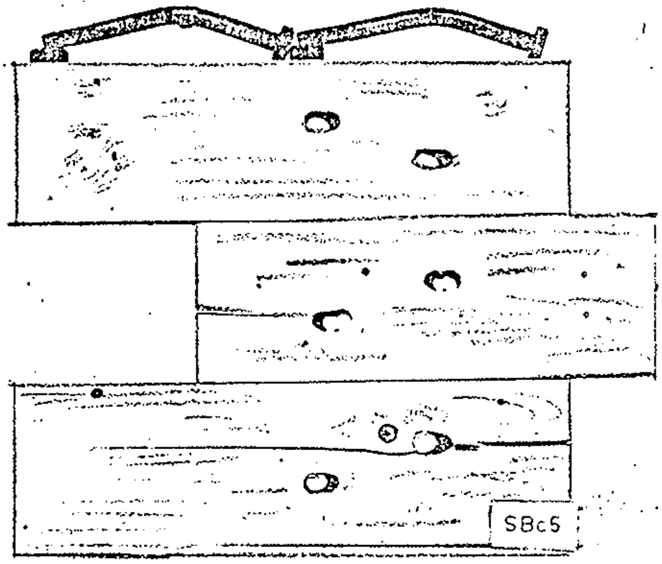

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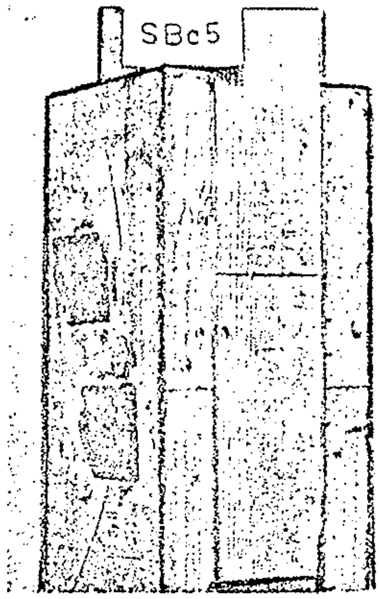

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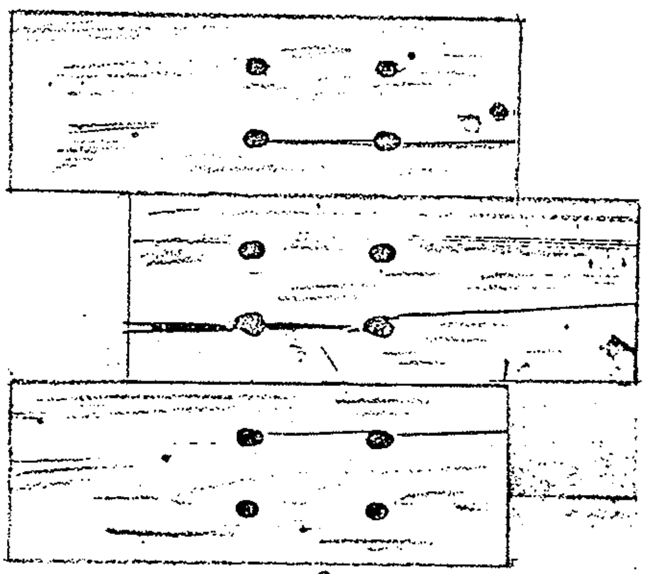

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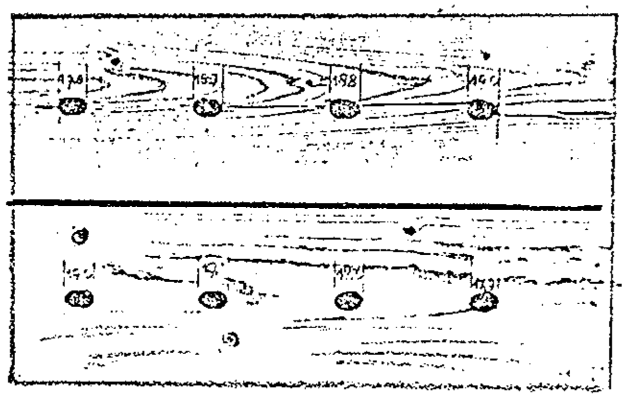

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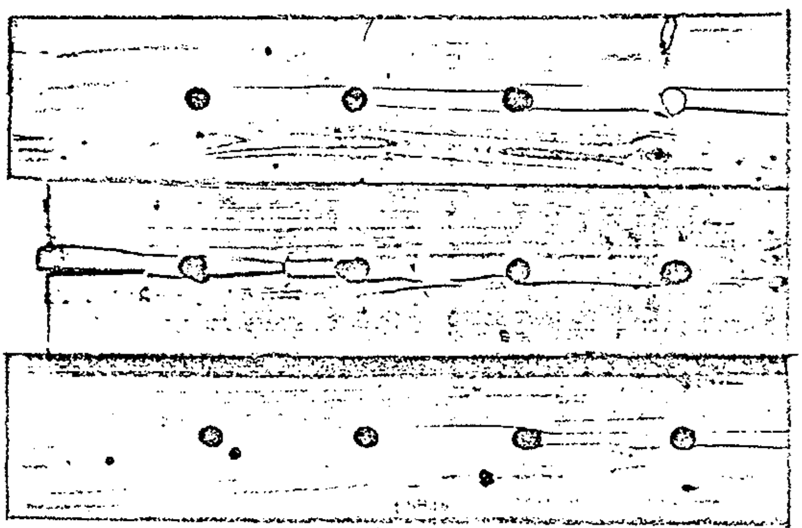

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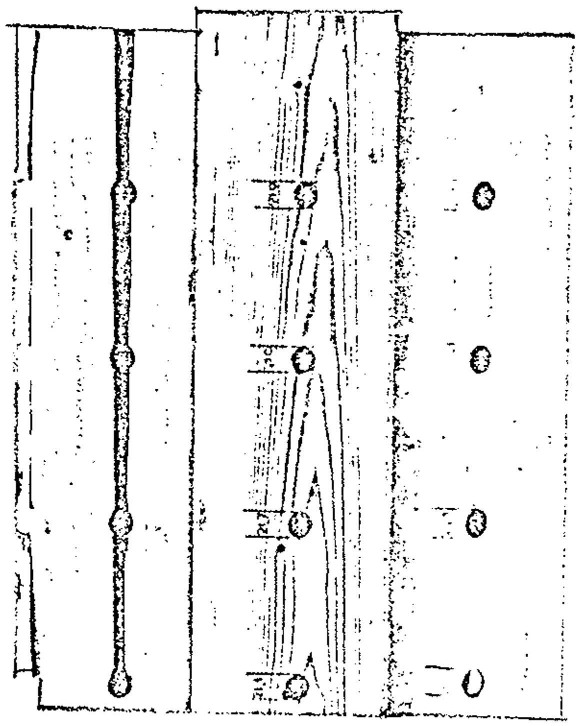

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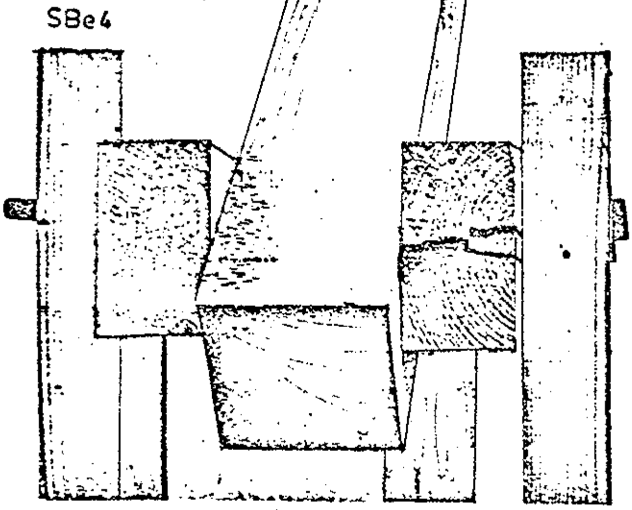

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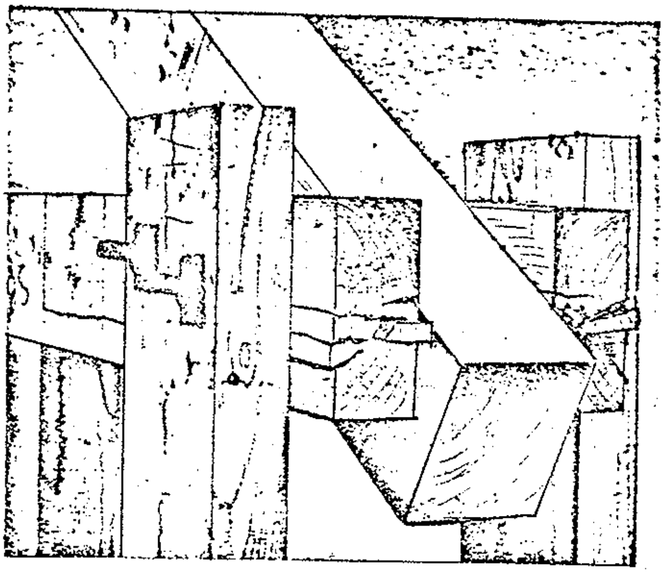

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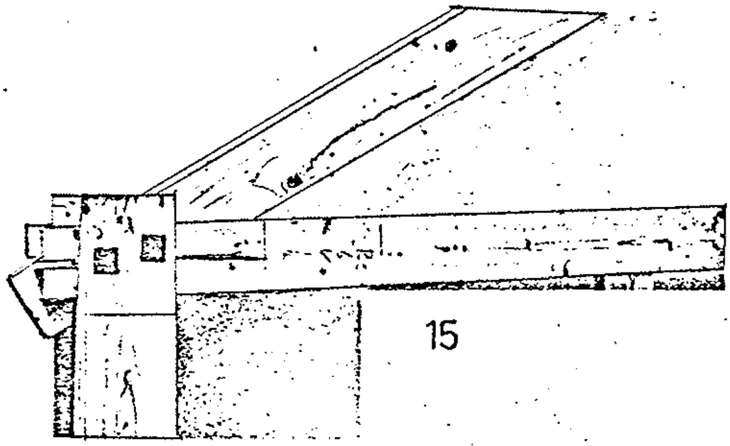

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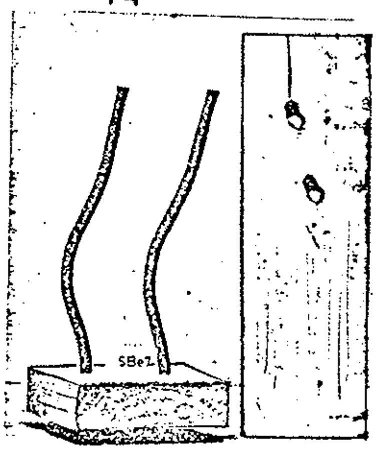

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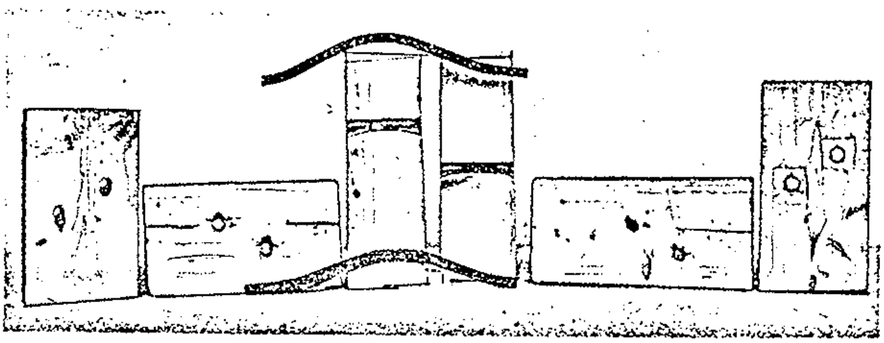

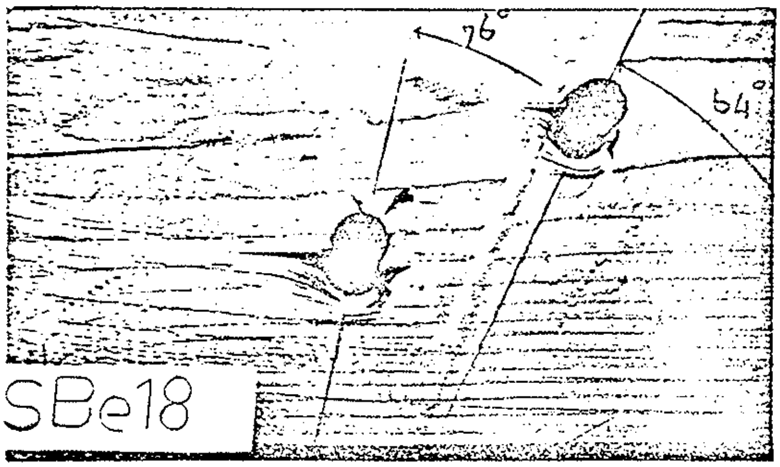

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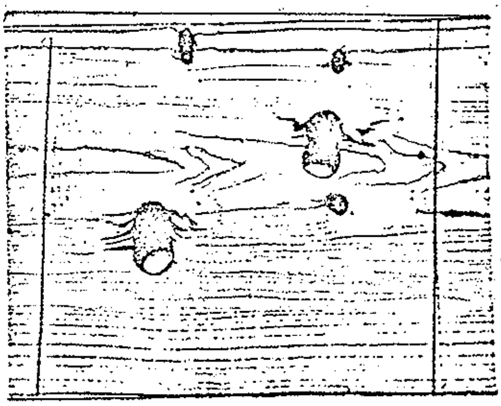

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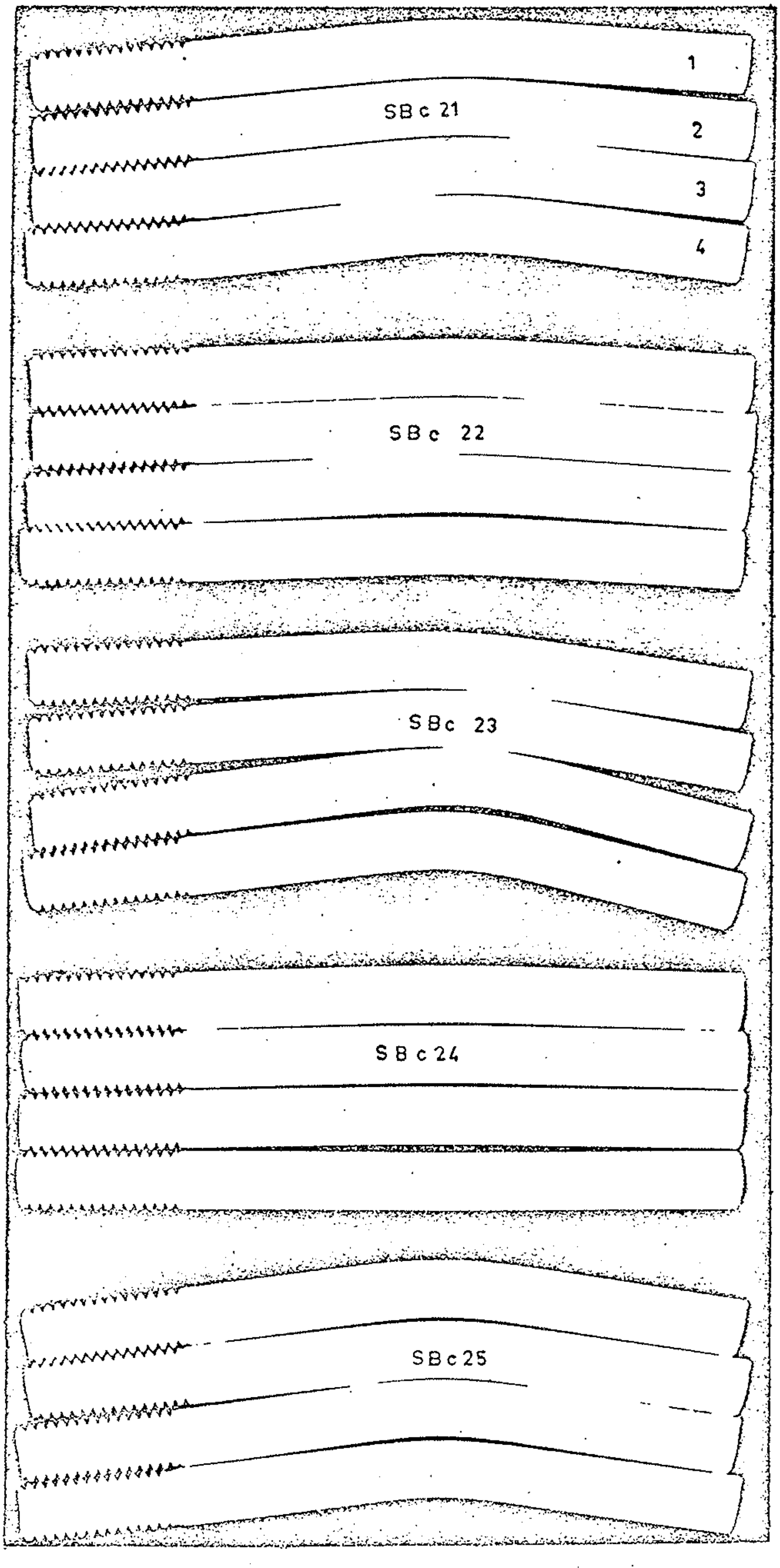
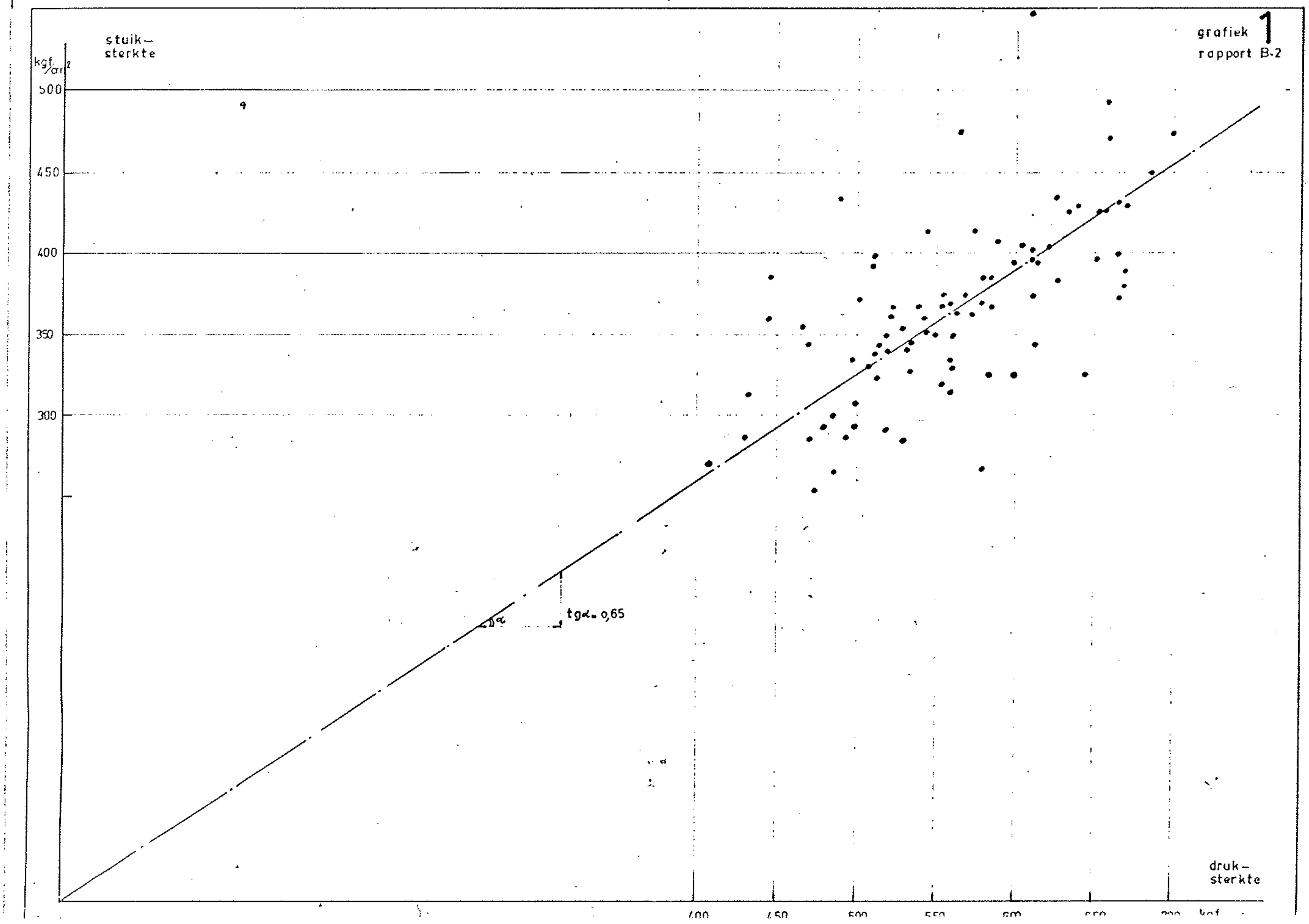
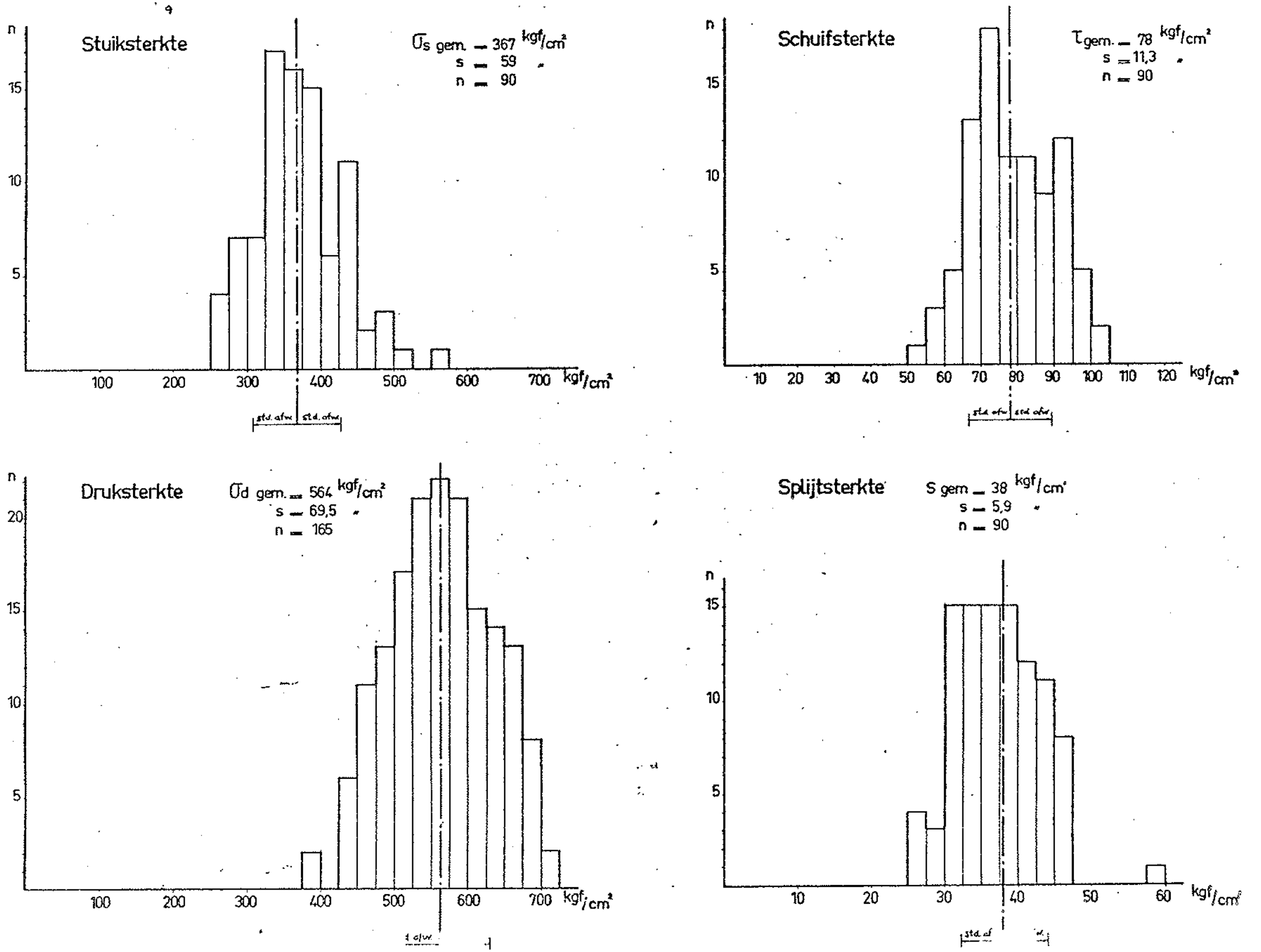
Rapport B_2

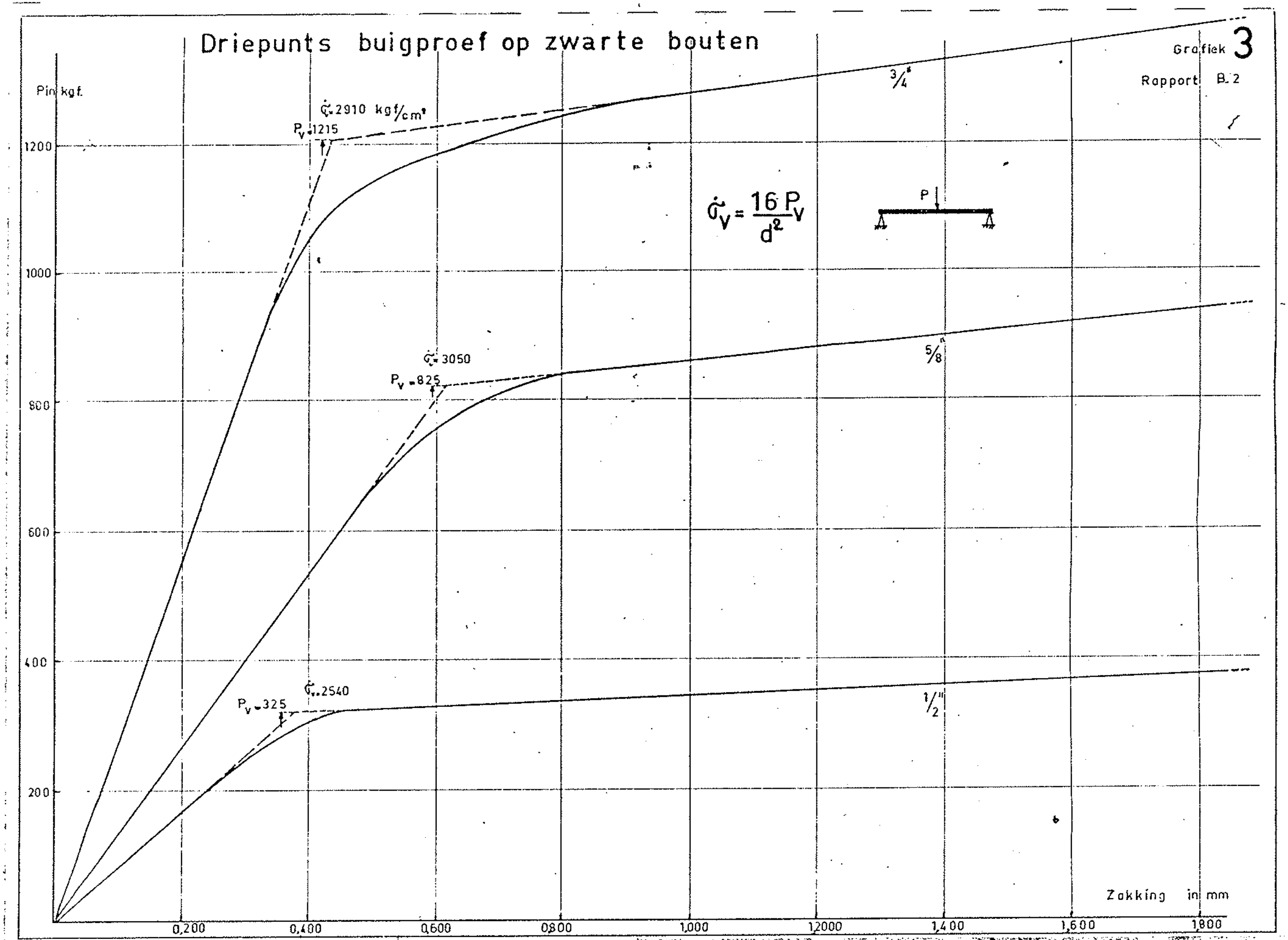


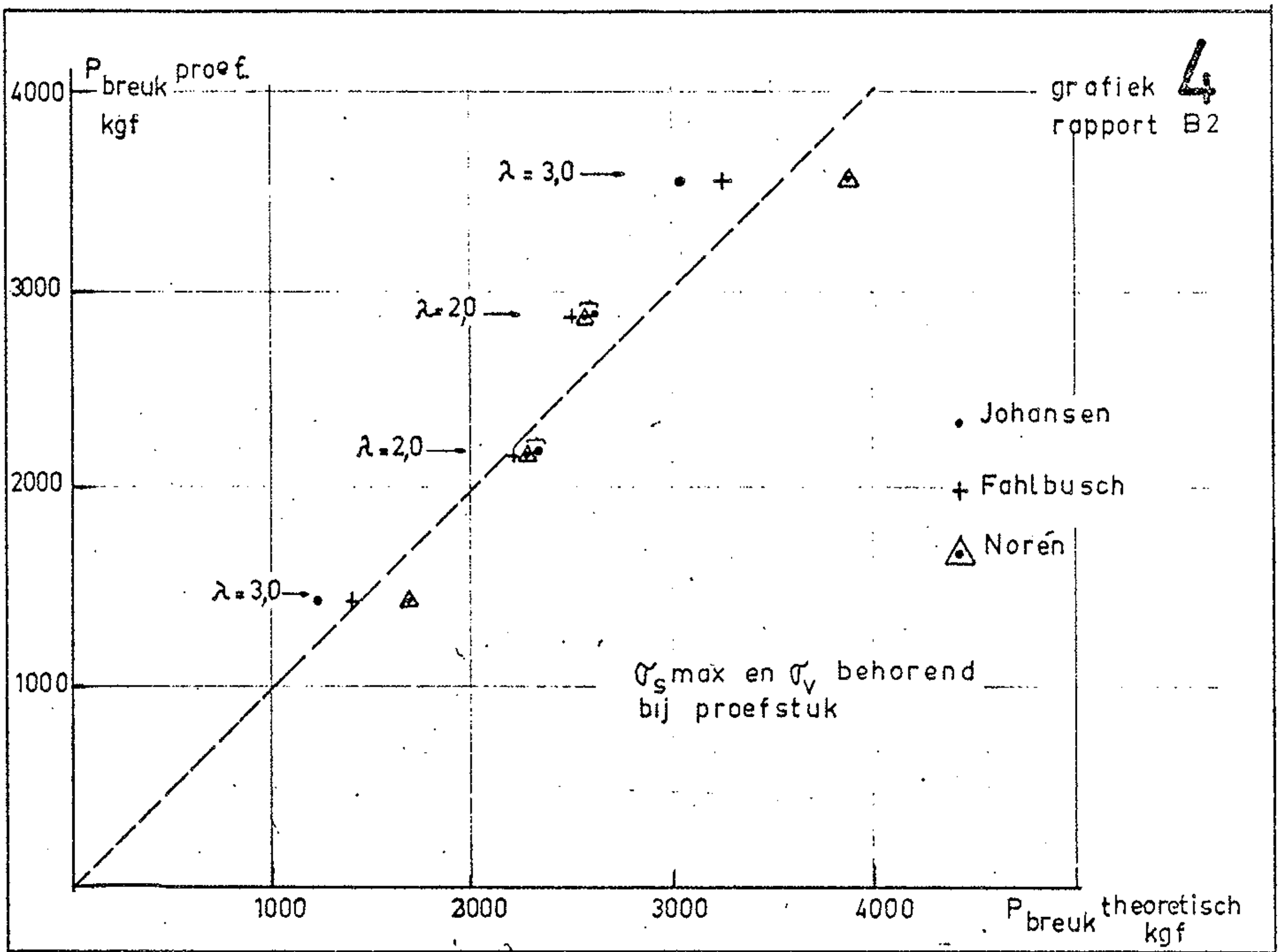
Vervorming van de bouten van de 4 assige verbinding


grafiek 1
rapport B-2
stuik-
sterkte
kgf/cm2
500
450
400
350
300
tgα = 0,65
α
druk-
sterkte
400
450
500
550
600
650
700
kgf

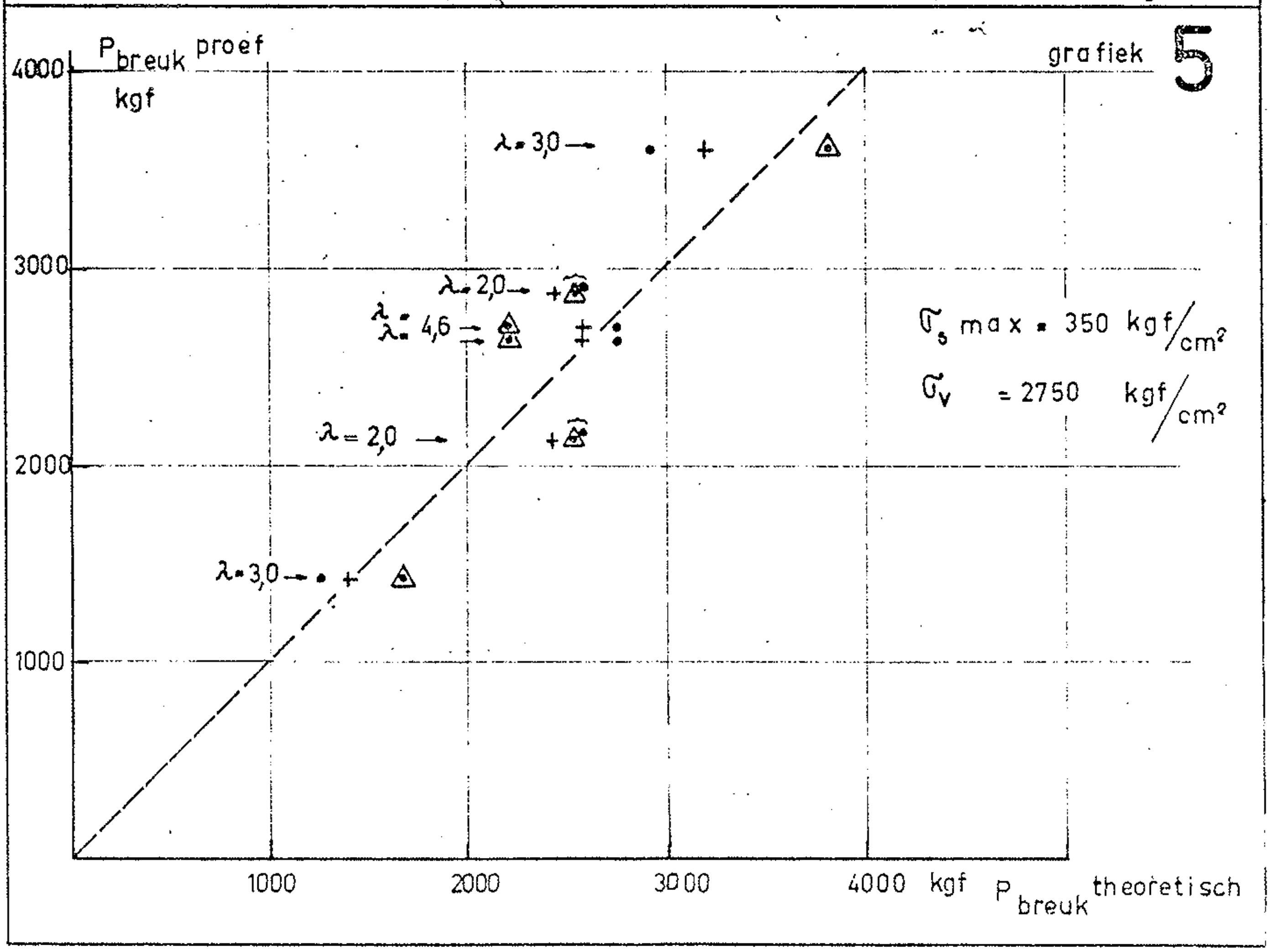
HISTOGRAMMEN VAN DE STERKTE-EIGENSCHAPPEN VAN HET HOUT.




Driepunts buigproef op zwarte bouten
Grafiek 3
Rapport B.2
P in kgf.
1200
1000
800
600
400
200
0,200
0,400
0,600
0,800
1,000
1,200
1,400
1,600
1,800
Zakking in mm
$\dot{\sigma}_v = 2910$ kgf/cm²
$P_v = 1215$
$\dot{\sigma}_v = 3050$
$P_v = 825$
$\dot{\sigma}_v = 2540$
$P_v = 325$
3/4"
5/8"
1/2"
$\dot{\sigma}_v = \frac{16 P_v}{d^2}$
P

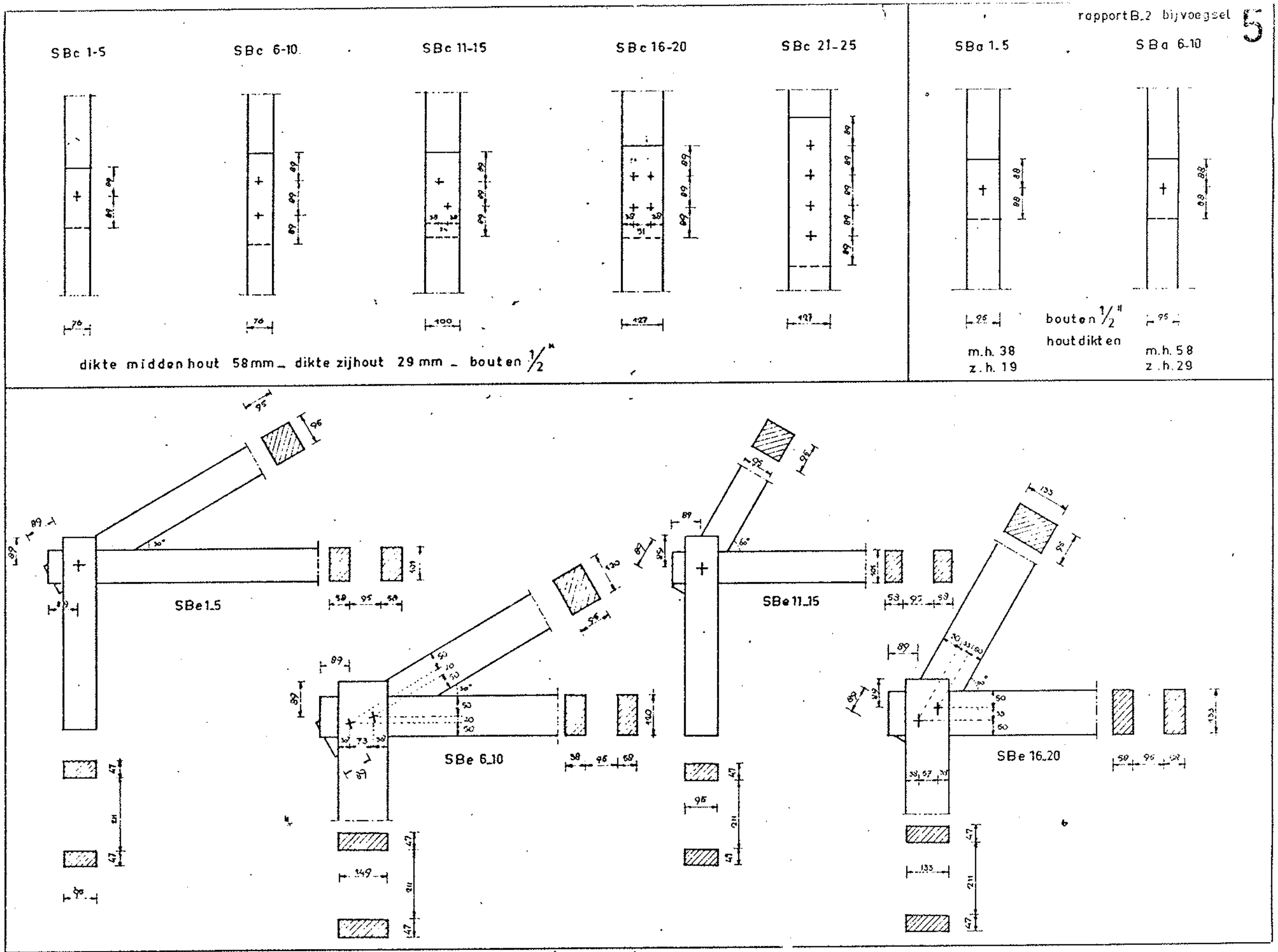


grafiek 4
rapport B2
P_{breuk} proef
kgf
4000
3000
2000
1000
λ = 3,0
λ = 2,0
λ = 2,0
λ = 3,0
Johansen
Fahlbusch
Norén
σ_s max en σ_v behorend
bij proefstuk
1000
2000
3000
4000
P_{breuk} theoretisch
kgf



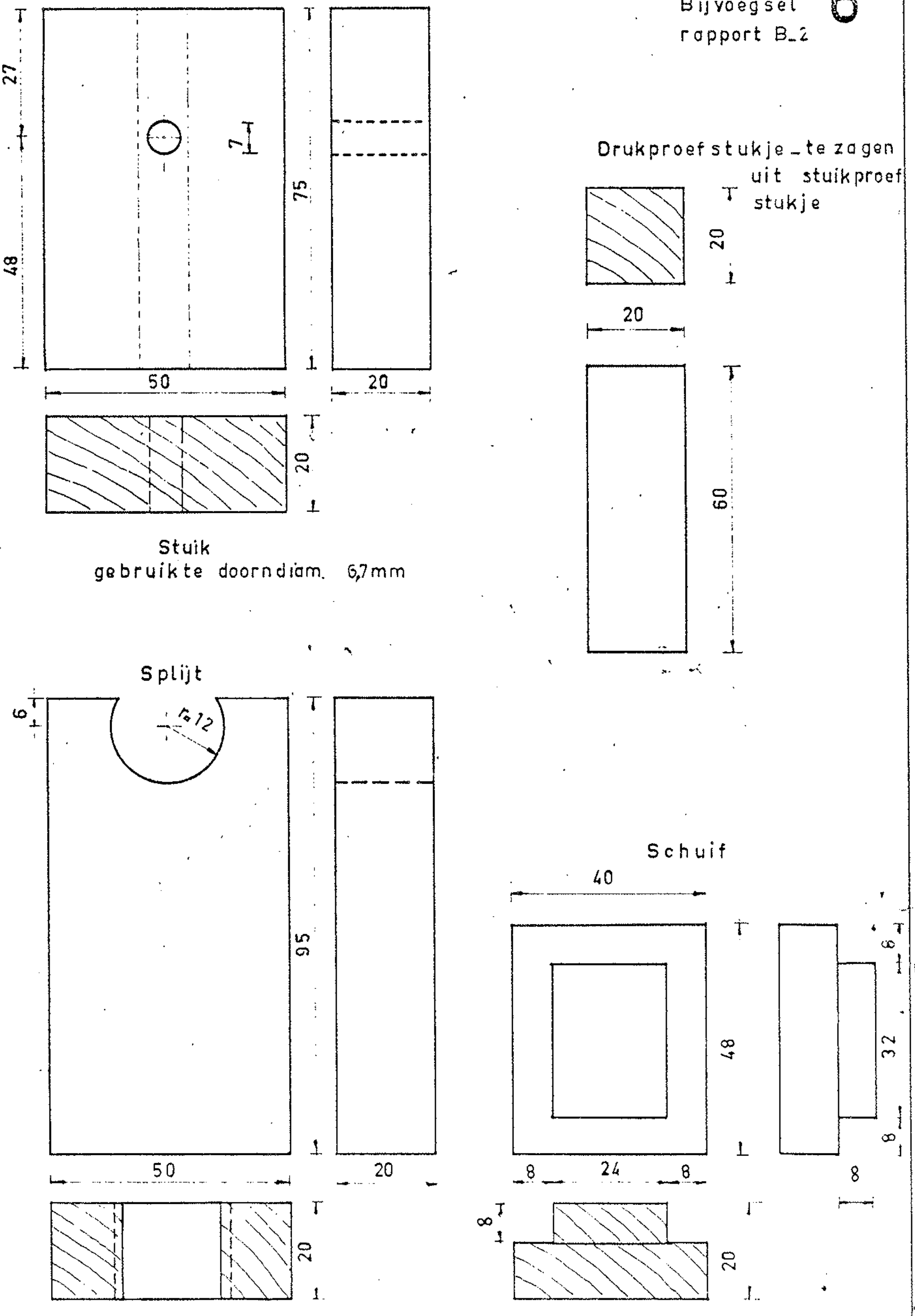
grafiek 5
P_{breuk} proef
kgf
4000
3000
2000
1000
λ = 3,0
λ = 2,0
λ = 4,6
λ = 2,0
λ = 3,0
σ_s max = 350 kgf/cm²
σ_v = 2750 kgf/cm²
1000
2000
3000
4000 kgf
P_{breuk} theoretisch

rapport B.2 bijvoegsel 5



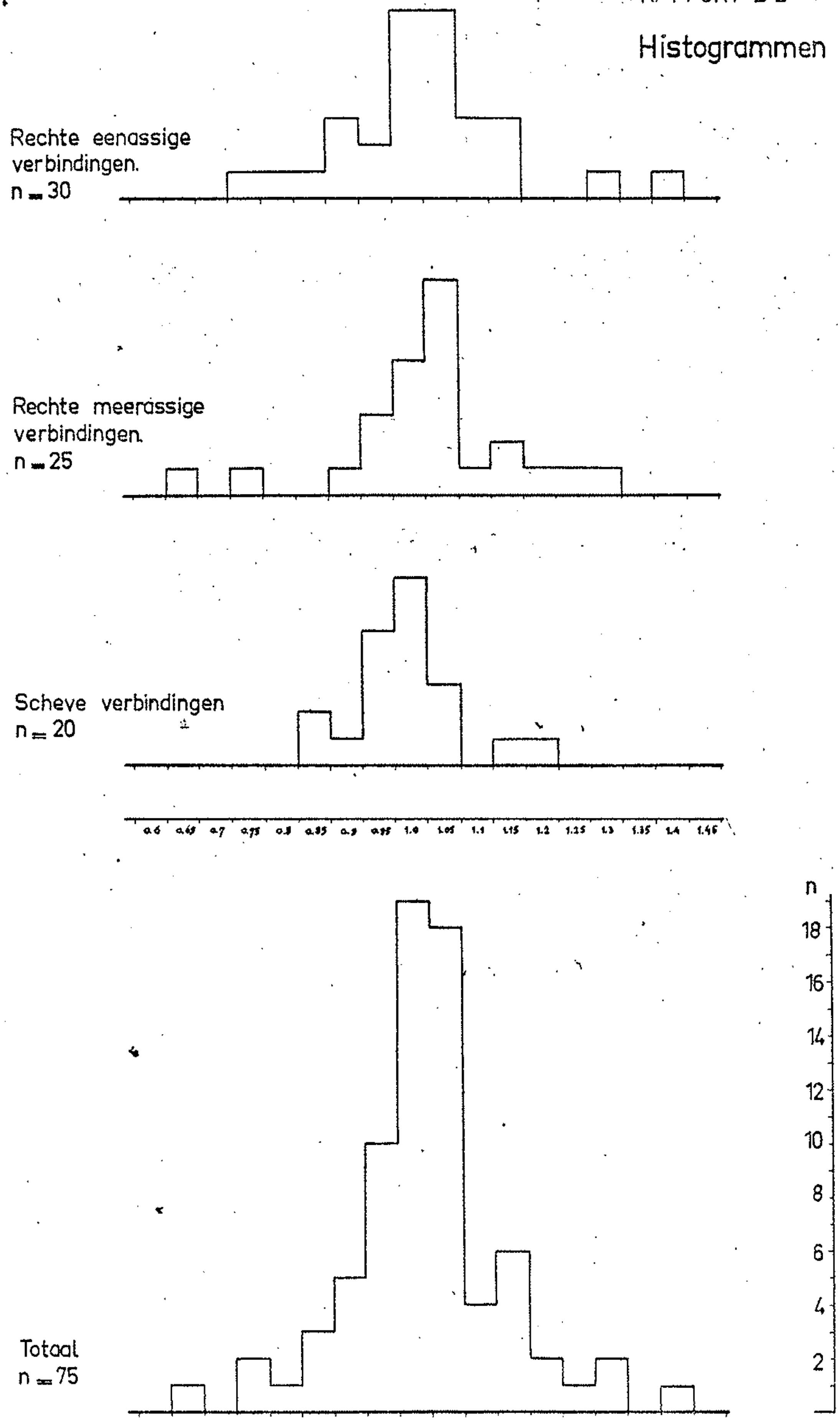
Bijvoegsel
rapport B_2

6



RAPPORT B-2 GRAFIEK 7.

Histogrammen van $\frac{P_{br\ proef}}{P_{br\ theor}}$



CIB-W18/13-7-8

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENTS TO PAPER CIB-W18/12-7-3 'DESIGN OF JOINTS WITH NAIL-PLATES'

OTANIEMI, FINLAND

JUNE 1980

BN 1980-06-04

Comments to paper CIB-W18/12-7-3
DESIGN OF JOINTS WITH NAIL PLATES

1 There are two editions of the paper:
That distributed in Bordeaux and that appearing in the proceedings from the Bordeaux meeting (12). The corrections mentioned when the first edition was presented are made in the latter edition.

2 Further corrections and alterations are suggested:

α_F is replaced by α_{XF}
α_2 is replaced by α_{Fa} (or $2\pi - \alpha_{aF}$)
α_1 and α_3 are deleted (in 1.2.2)
β is replaced by $\alpha - \pi/2$

The text and figures are changed accordingly, for example, at the bottom of page 18: ...... as a function of the angle's behaviour, the plate force and plate and fibre directions.

The denotation A should generally be used as the total effective grip area on one side of the (timber/timber) joint, cf foot note (1) page 2 of paper CIB-W18/13-7-4.

The next last sentence of p17 is changed to:
'It is denoted A and generally is a total of two areas (A/2) from plates on each face of the timber.'

The factor 2 in eqs (16), (18) and (21) is deleted.

3 The alterations suggested above may be anticipated in a third edition (draft). In this also the sub-section 2.6 Calculation of slip, will be incorporated, cf paper CIB-W18/13-7-4.

It is further suggested that the sub-section 2.5.2 (pages 26 and 27) are replaced by the examples from Källsner's paper CIB-W18/13-7-5 and references to reports by M Johansen respectively B Källsner.

4 The validity of 2.4.3 shear, eq (29) still has to be verified.

Otaniemi June 1980
Bengt Noren

CIB-W18/13-7-9

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STRENGTH OF FINGER JOINTS

by

H J Larsen
Aalborg University Centre
DENMARK

OTANIEMI, FINLAND
JUNE 1980

INSTITUTTET FOR BYGNINGSTEKNIK

INSTITUTE OF BUILDING TECHNOLOGY AND STRUCTURAL ENGINEERING

AALBORG UNIVERSITETSCENTER · AUC · AALBORG · DANMARK

Prepared for seminar on

THE PRODUCTION, MARKETING AND USE OF FINGER-JOINTED SAWNWOOD

Hamar, Norway, September 1980

H. J. LARSEN

STRENGTH OF FINGER JOINTS

MAY 1980

IFB/A 8002

1. SUMMARY

The results from three test series on finger joints are briefly summarized.

In section 2 tensile testing of 80 finger jointed glulam lamellas of spruce is reported. The mean value of the tensile strength varied between 26 MPa (ECE grade 6) and 32 MPa (ECE grade 10) with a coefficient of variation of 15-20 per cent. The 5-percentiles (characteristic values) varied correspondingly between 18 and 20 MPa. The mean strength is approximately 80 per cent of the strength of unjointed lamellas, the 5-percentiles, however, differ less because of a smaller coefficient of variation. The 5-percentiles assumed in CIB-W18-Structural Timber Design Code vary between 9 and 20 MPa.

In section 3 bending and tensile tests on finger jointed lamellas from 10 different glulam factories are reported. The investigation had a twofold purpose.

The first was on matched samples to compare bending strengths as determined rather crudely in the factories as part of the quality control with laboratory values. The second was to compare bending and tensile strengths.

The test results showed great variation (coefficients of variation for the bending strength were about 20 per cent, for the tensile strength 25-30 per cent), and the ratio between the two bending strengths and between the tensile strength and the bending strength of the individual test specimens varied strongly (coefficient of variation about 20-30 per cent).

On the other hand there was, perhaps with one exception, very good agreement between the mean values of the bending strength determined by the factory and at the laboratory, and it can thus be concluded that the factories carry out the testing satisfactorily.

Also a rather constant ratio between the mean values of the tensile and bending strength was found; the tensile strength was about 60 per cent of the bending strength. This corresponds to the theoretical value derived in appendix 1.

For the tensile strength the 5-percentiles varied for the individual factories between 7 and 24 MPa, for all test results as a whole the value was 17 MPa. The value assumed in the Danish Timber Code for the grade in question is 27 MPa.

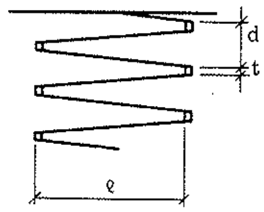
Section 4 concerns bending and tensile tests on 40 finger jointed pieces of structural timber. A ratio of 0.65 was found between the tensile and the bending strength. The 5-percentile in bending was 31 MPa, in tension 18 MPa, again to be compared with the assumed 27 MPa.

The conclusion is that the present practice where the same grades and strength values for unjointed and finger jointed timber are assumed should probably be changed taking into account a lower ratio between tensile and bending strength for the higher grades of finger jointed timber.

2. TENSILE STRENGTH OF GLULAM LAMELLAS

As part of an investigation into the strength and stiffness of glued laminated beams the tensile strength of 180 unjointed and 80 finger jointed lamellas were determined. The complete test report is given in [Larsen, 1978].

The boards were of Swedish spruce graded according to the ECE-rules [ECE, 1977]. The cross-sections were 33 × 140 mm planed. The finger length was 20 mm and corresponded to DIN 68140, cf. figure 2.1. The fingers were visible on the wide side.



ℓ	d	t
20	6	1
10	4	1

Figure 2.1 Finger profile. Measurements in mm.

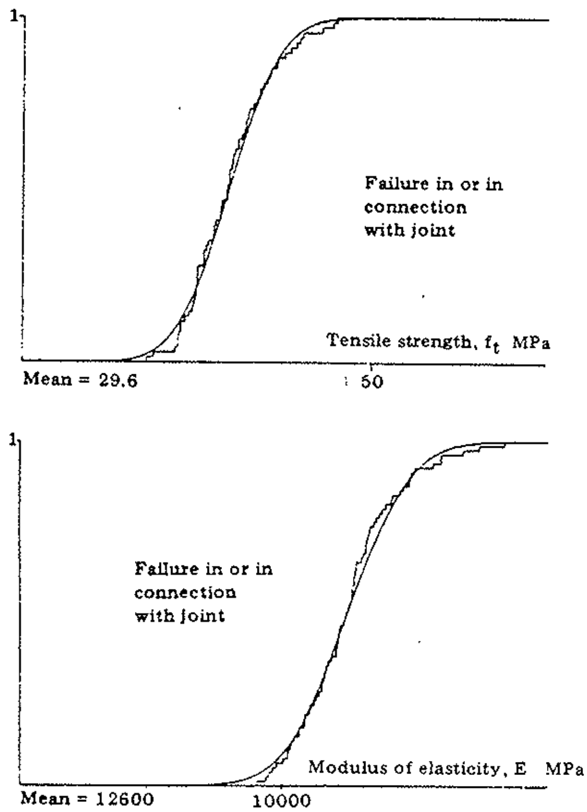


Figure 2.2. Distribution functions (step functions) for finger jointed boards. For comparison the corresponding normal distributions are drawn.

Table 2. Tensile strength and stiffness of spruce

	ECE-class 10	ECE-class 8	ECE-class 6	Failure not in joint	Failure in or in connection with joint	Failure in joint	All
Finger jointed boards							
Density, ρ							
Mean value					0.437	0.436	0.437
Tensile strength, f_t							
Mean value, MPa	31.53	28.68	26.14	24.4	29.64	31.51	29.21
Coefficient of variation, pct.	19	16	15		19	20	21
Minimum value, MPa				18.0	17.8	21.9	17.8
5-percentile, MPa	20.4	19.3	18.4		19.3	20.0	18.4
Modulus of elasticity, E							
Mean value, MPa					12560		12480
Coefficient of variation, pct.					15		15
Minimum value, MPa					9370		9030
Unjointed boards							
Density, ρ							
Mean value, MPa	0.437	0.415	0.433				0.431
Tensile strength, f_t							
Mean value, MPa	40.56	35.06	31.04				34.73
Coefficient of variation, pct.	21	24	25				27
5-percentile, MPa	23.3	19.7	17.4				18.5
Modulus of elasticity, E							
Mean value, MPa	12940	12430	12090				12370

The moisture content was approximately 12 per cent.

The testing was made in the testing machine described in [Andersen et al., 1977]. The grips of the machine are not hinged, and the action thus corresponds to uniform strain over the cross-section. A ramp load was used with approximately 5 minutes to failure. The modulus of elasticity was measured at midspan over a length of 400 mm, for the finger jointed boards the measurements were taken across the joint.

The test results for the finger jointed boards are summarized in figures 2.2 and table 2 together with some values for unjointed boards for comparison. The failure occurred in the finger joint in more than 90 per cent of the tests.

The test distributions were compared with normal distributions, log-normal distributions and 3-parameter Weibull distributions. There is good agreement between the test distributions and the theoretical distributions, the Weibull distribution being perhaps a little better than the other two.

The 5-percentile (the characteristic values) are in accordance with the CIB-Structural Timber Design Code [CIB-W18, 1979] calculated corresponding to the normal distribution and a confidence level of 75 per cent. The mean strength is lower than for unjointed boards (the ratio is app. 0.80). Due to a lower coefficient of variation this is generally not reflected in the 5-percentiles. For comparison it can be mentioned that the CIB-code assumes the following 5-percentiles: 20 MPa for ECE 10, 16 MPa for ECE 8 and 9 MPa for ECE 6. The values are met by the tested joints.

3. BENDING AND TENSILE STRENGTHS, GLULAM LAMELLAS

As part of the internal control factories producing structural glulam are required to test at least 3 finger joints a day per production line. The joints are tested in bending as shown in figure 3.1 with the distance a between

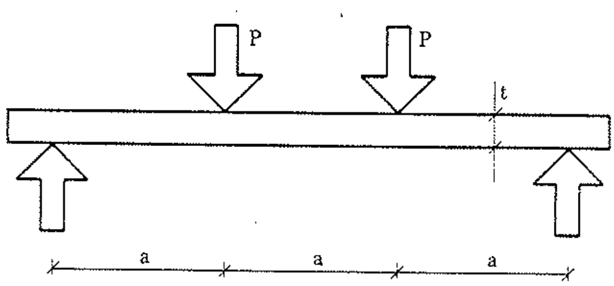


Figure 3.1

5t and 8t. The test apparatus is rather primitive, consisting of a manually operated hydraulic jack with a manometer, and the time to failure is 10-20 seconds.

Tests were made for the Danish Glulam Control in cooperation with Teknologisk Institut (Danish Institute of Technology) and had a twofold purpose:

- To evaluate the testing at the glulam factories.
- To get information about the bending and tensile strength of finger jointed glulam lamellas.

The complete rest report is given in [Larsen, 1979].

Ten factories were included in the investigation (in total there are 11 factories, but one had a very limited production at the time of the investigation).

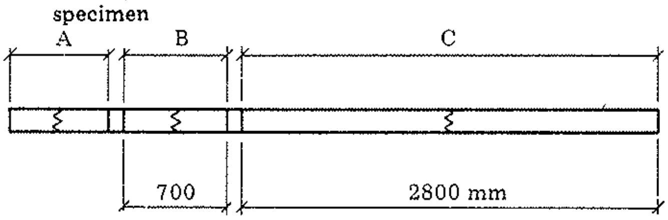


Figure 3.2

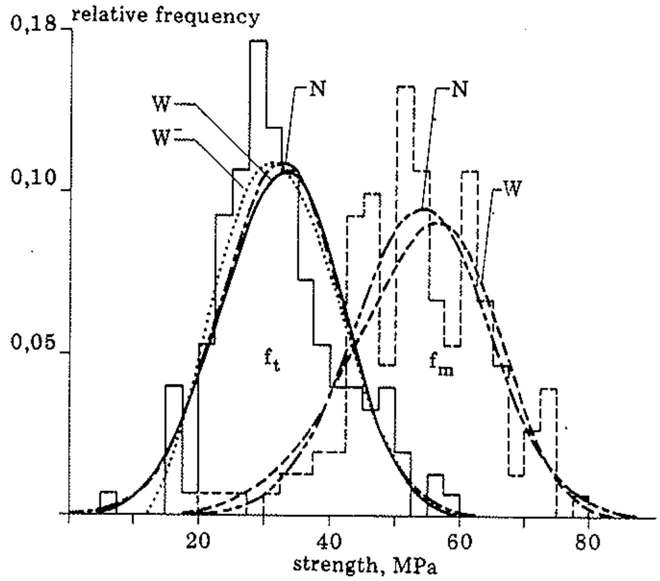


Figure 3.3. Distribution of test results and the corresponding theoretical distributions: N = Normal distribution. L = Log normal distribution. W = Weibull distribution. W^- as W but with the lowest value disregarded.

Table 3

Factory No.	1	2	3	4	5	6	7	8	9	10	All
Specimen A:											
Bending strength, $f_{m,own}$, MPa	55.1	51.8	53.9	48.1	60.6	47.9	51.4	58.8	57.9	52.4	53.8
Coefficient of variation, pct.	12	16	14	20	26	9	16	8	15	12	17
Specimen B:											
Bending strength, f_m, MPa	52.9	51.3	55.8	46.4	62.2	58.6	45.2	60.4	50.4	56.0	53.9
Coefficient of variation, pct.	14	13	18	20	15	13	11	19	27	15	19
Minimum value, MPa	42.3	42.9	39.7	20.6	43.9	34.2	37.5	31.9	24.2	45.6	
5-percentile, MPa	38.5	37.7	35.4	27.6	43.1	43.6	35.6	36.9	23.4	39.8	36.2
Specimen C:											
Tensile strength, f_t, MPa	29.1	25.3	30.9	29.6	34.4	39.4	27.7	38.4	28.2	40.9	32.4
				(30.2)	(34.2)		(28.5)	(38.4)	(28.4)	(42.6)	(32.5)
Coefficient of variation, pct.	18	16	28	25	28	20	23	19	35	23	28
				(27)	(32)		(20)	(20)	(36)	(22)	(29)
Minimum value, MPa	20.7	17.1	16.8	16.8	15.6	28.2	16.9	27.1	7.0	28.1	
				(16.8)	(15.6)		(20.3)	(27.1)	(7.0)	(29.1)	
5-percentile, MPa	18.9	17.5	13.7	15.0	15.2	23.8	15.1	24.0	8.4	22.5	16.9
				(13.8)	(11.9)		(20.8)	(23.3)	(7.3)	(23.7)	(16.9)
Specific density, ρ *)	0.397	0.411	0.453	0.423	0.443	0.437	0.428	0.433	0.437	0.426	0.429
Coefficient of variation, pct.	8	18	7	8	8	11	8	9	12	10	10
$f_m/f_{m,own}$	0.97	1.01	1.05	1.00	1.06	1.24	0.90	1.03	0.88	1.08	1.02
Coefficient of variation, pct.	18	19	18	26	20	16	20	20	30	16	22
f_t/f_m	0.56	0.50	0.56	0.67	0.56	0.69	0.62	0.66	0.60	0.73	0.62
				(0.67)	(0.56)		(0.64)	(0.62)	(0.61)	(0.76)	(0.61)
Coefficient of variation, pct.	22	18	25	34	28	29	24	27	43	21	30
				(37)	(30)		(23)	(18)	(46)	(21)	(30)

*)based on dry weight and volume corresponding to a moisture content of 12 per cent.

Numbers in brackets ( ) apply to tensile rupture in or mostly in connection with joint, the other numbers apply to all tests.

Three times a day during a one-week period a test specimen as shown in figure 3.2 was produced, giving 15 test specimens per factory.

The thickness of the boards was 33 mm planed, the widths varied between 70 mm and 195 mm planed. The moisture content was 10-14 per cent. The test specimens were handled, cured, planed, etc. as the ordinary lamellas. The wood quality was T300 according to the Nordic T-grading system, approximately corresponding to ECE 10 and the better half of ECE 8.

Each test specimen contained three finger joints marked A, B, and C.

Joints B were tested in bending with a = 7t (figure 3.1) at Teknologisk Institut in ramp loading with a time to failure of approximately 3 minutes.

Joints C were tested in tension at Aalborg University Centre in ramp loading with approximately 3 minutes to failure. The test results are summarized in figure 3.3 and table 3.

In the tensile tests more than 90 per cent of the failures occurred in or in connection with the finger joints. The preparation of the test results has been made for all test results as well as for those only where the failure occurred in the joint. In most cases it was advantageous to include all the test results.

Two finger types were included in the investigations both corresponding to DIN 68140, cf. figure 2.1. The factories 1-9 use 20 mm fingers, factory No. 10 uses 10 mm fingers. In all cases the finger profiles are visible on the wide side. Six factories use hot curing, five of them by radio-frequency heating of the glue in the finished joint, the sixth by radio-frequency heating of the fingers before application of the glue. As far as the remaining four factories are concerned the completed lamellas rest at least 24 hours before they are handled. Application of glue is made manually on one end or mechanically on one or both ends. The investigation gives no basis for preferring one method to the other as long as the equipment and the procedure is fit for the finger jointing method chosen.

The mean value of the bending strength is about 54 MPa with a coefficient of variation of about 20 per cent. The ratio $f_m/f_{m,own}$ between the bending strength determined in the laboratory (f_m) and that determined by the factories ($f_{m,own}$) is, apart from that of factory No. 6, equal to 1.00, which indicates that the testing equipment has been correctly calibrated and that the primitive loading procedure is insignificant. As regards factory No. 6 it is possible that an error has been made in the force measuring, although the possibility of a coincidence cannot be dismissed (the probability, however, is only 5-10 per cent).

The requirements made in [DS 413, 1974] for the bending strengths of glulam lamellas T300 are either that all values should be above 39 MPa or that the characteristic value of

The requirements made in [3] for the bending strengths of glulam lamellas T300 are either that all values should be above 39 MPa or that the characteristic value of the test the test set should be above this value. For test series of 15 the characteristic value - under the assumption of normal distribution - is equal to

$$(m - 2s) = m(1 - \frac{2v}{100})$$

where m is the mean value, s the standard deviation and v the coefficient of variation.

All values from the testing at the factories are above the required 39 MPa which can hardly be surprising.

The testing in the laboratory showed that 9 values from 5 factories were below the required value and only for one of them the required characteristic value was above the 39 MPa, and this only when the Weibull distribution was used.

The tensile strengths derived can be compared with the fact that the Danish Timber Code for structural timber T300 assumes a characteristic short-term strength of 27 MPa and for the next grade, T200, 18.5 MPa. Thus, the jointed lamellas just fulfil the requirements for T200. For glulam made from T300 a characteristic tensile strength of about 35 MPa is assumed. The values for ECE10 and ECE 8 are according to the CIB-code 20 and 16 MPa, respectively. The requirements for ECE 8 (but not ECE 10) are met by the individual factories. The vary low value - 7 MPa - for factory No. 9 should be noted.

The ratio f_t/f_m between the tensile and bending strength is on average 0.62 and for the individual series there is great variation, from 0.50 to 0.69. While the ratio between the group results is thus rather constant this does not apply to the individual tests, cf. figure 3.4. Perhaps this was

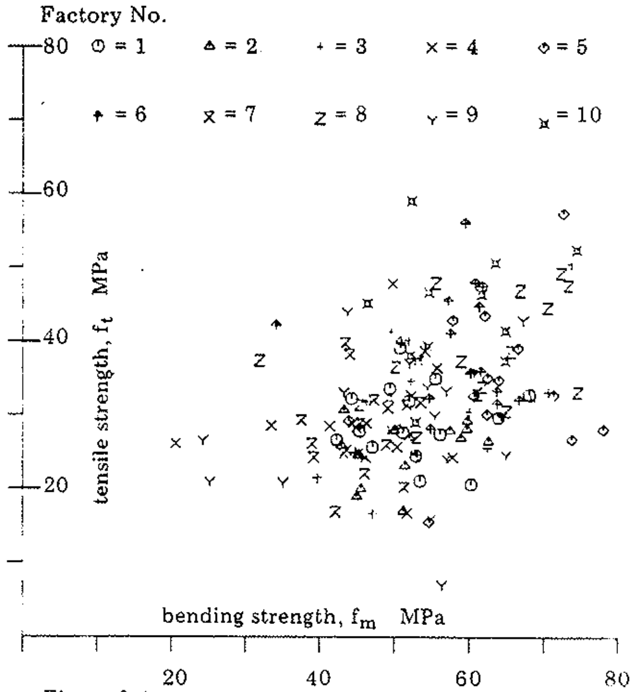


Figure 3.4

not to be expected when it is taken into consideration that the coefficient of variation for the bending strength is about 20 per cent and for the tensile strength 25 - 30 per cent.

A ratio of 0.60 - 0.65 between the tensile strength and the bending strength is what would be expected according to the theory in Appendix 1.

In figure 3.5 corresponding values of density (minimum values for the two members in a joint) and the tensile strengths are shown. It can be seen directly that there is no great dependence between the values. The picture of the bending strength has not been given here but it is quite analogous.

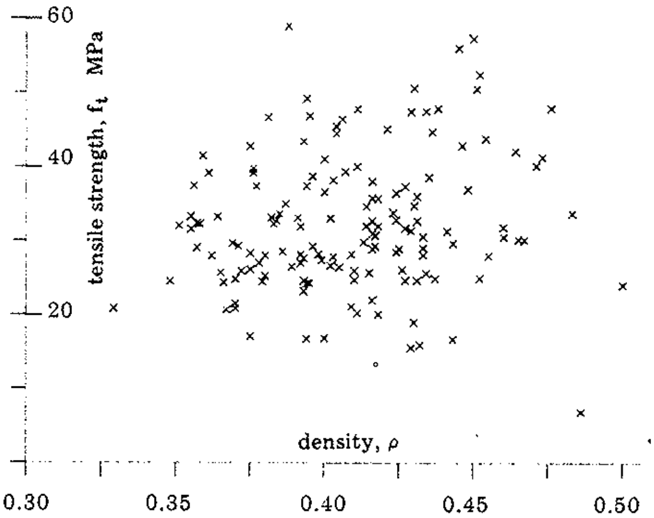


Figure 3.5

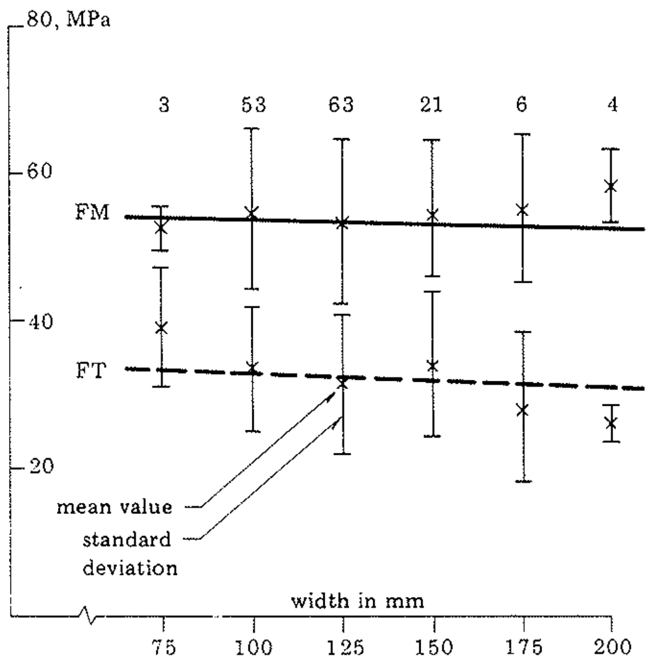


Figure 3.6. The bending strength, f_m, and the tensile strength, f_t, dependent on the width of the lamellas. The values give the number of specimens.

Figure 3.6 shows the dependence of the strengths on the width of the raw timber (the real widths are 5-10 mm smaller). It seems that the strengths tend to decrease with increasing width, but the number of wide specimens is so low that there is no statistic significance.

4. BENDING AND TENSILE STRENGTHS, STRUCTURAL TIMBER

The test pieces were of pine of quality T300 with a cross-section of approximately 50 × 200 mm. The finger joints corresponded to figure 2.1 with ℓ = 10 mm. The profile was visible on the wide side.

20 specimens were tested in flatwise bending with a = 350 mm, cf. figure 3.1. The specimens were not planed before testing. The bending strength based on the nominal cross-sectional area had a mean of f_m = 44.9 MPa with a coefficient of variation of 16 per cent corresponding to a 5-percentile of approximately 31 MPa. 19 of the failures were caused by the finger joint, only 1 was outside the joint. The mean value of the modulus of elasticity was 11050 MPa with a coefficient of variation of 7 per cent.

20 specimens were tested in tension. The test specimens were planed to approximately 48 × 197. The tensile strengths f_t based on the actual areas had a mean of 29.3 MPa with a coefficient of variation of 19 per cent corresponding to a 5-percentile of approximately 18 MPa. The Danish Code assumes 27 MPa for T300 and 18 for T200. 16 of the failures were caused by the finger joint, 1 of the failures was at the grips of the testing machine.

The ratio f_t/f_m is approximately 0.65, i.e. the same as found for the glulam lamellas and in accordance with the expected theoretical value, cf. the appendix.

5. LITERATURE

Andersen, H. et al., 1977: Description of tensile test machine for wood. Institute of Building Technology and Structural Engineering. Aalborg University Centre. Report No. 7705.

Larsen, H. J., 1978: Forsøg med limtræslameller (Tests with glulam lamellas, in Danish). Institute of Building Technology and Structural Engineering. Aalborg University Centre, Report No. 7803.

Larsen, H. J., 1979: Bøjnings- og trækstyrke af fingerskarringer (Bending and tensile strength of finger joints, in Danish). Institute of Building Technology and Structural Engineering, Aalborg University Centre, Report No. 7902.

CIB-W18 1979: CIB-Structural Timber Design Code, Fourth Draft.

ECE, 1977: Recommended standard for stress grading (Annex 1 to ECE/TIM/WP.3/AC.3/8).

DS 413, 1974: Danish Standard DS 413, Code of Practice for the Structural use of Timber (English edition).

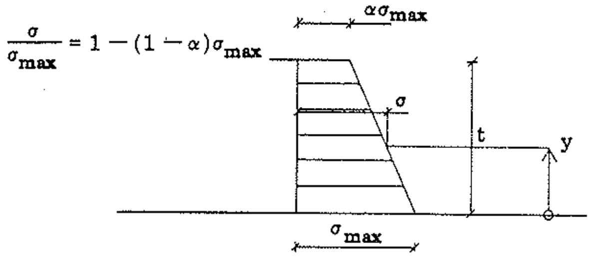


Figure A.1

APPENDIX 1. THE RATIO BETWEEN TENSILE AND BENDING STRENGTH

Reference is made to e.g. Johnson, A. I.: Strength safety and economical dimensions of structures. Statens Kommitte för Byggnadsforskning, Medd. No. 22, Stockholm, 1953.

A weakest link failure theory is assumed.

The tensile strength of a unit volume is assumed to be Weibull distributed with expectation value μ and form parameter k.

For a volume with length ℓ (finger length) width b and thickness t^* and uniformly stressed in tension with the stress σ_{max} the probability of survival is G^*, and

$$\ell n G^* = -\left(\frac{\sigma_{max}}{\mu}\Gamma\left(1+\frac{1}{k}\right)\right)^k \ell b t^*$$

Γ is the gamma function.

For a volume of ℓbt stressed with a varying tensile stress as shown the probability of survival is G

$$\ell n G = -\left(\frac{\sigma_{max}}{\mu}\Gamma\left(1+\frac{1}{k}\right)\right)^k \ell b \int_0^t \left(1-(1-\alpha)\frac{y}{t}\right)^k dy$$

$$= -\left(\frac{\sigma_{max}}{\mu}\Gamma\left(1+\frac{1}{k}\right)\right)^k \ell bt \frac{1-\alpha^{k+1}}{(1-\alpha)(1+k)}$$

If the two volumes shall have the same probability of survival then

$$\frac{t^*}{t} = \frac{1-\alpha^{k+1}}{(1-\alpha)(1+k)}$$

t^* is the equivalent thickness.

The expectation value of the strength of a volume corresponding to t^* as compared to the expectation value of a volume with thickness t is

$$r = \left(\frac{t}{t^*}\right)^{1/k}$$

For k = 5.0, a value usually assumed for wood, r-values are found as shown. For $\alpha < 0$ only the part with tensile stresses is taken into account.

The ratio between the strength in pure tension and pure bending ($\alpha = -1.0$) is thus theoretically 0.50 - 0.67.

Table A.1

	k = 5			k = 3	k = 7
α	t^*/t	r	1/r	1/r	1/r
0.75	0.55	1.13	0.89	0.88	0.89
0.50	0.33	1.25	0.80	0.78	0.82
0.25	0.22	1.35	0.74	0.69	0.77
0	0.17	1.43	0.70	0.63	0.74
− 1.0	0.08	1.64	0.61	0.50	0.67

CIB-W18/13-12-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

GLULAM STANDARD PART 3
GLUED TIMBER STRUCTURES: PERFORMANCE
(4th draft)

OTANIEMI, FINLAND
JUNE 1980

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DES SYNDICATS DE FABRICANTS DE MENUISERIES
INDUSTRIELLES DE BATIMENT
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« GLULAM »

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November 1979

4th DRAFT

PART 3

Approved on the meeting 26.October 1979
Mulhouse

GLUED TIMBER STRUCTURES
PERFORMANCE

Preface:

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

There are three parts of the GLULAM-STANDARD:

Part 1 : GLUED TIMBER STRUCTURES
REQUIREMENTS FOR TIMBER

Part 2 : GLUED TIMBER STRUCTURES
RATING

Part 3 : GLUED TIMBER STRUCTURES
PERFORMANCE

1) Scope:

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

Deviations from the requirements of this code and the use of materials and methods of design or construction of wood structures not covered by this code are permitted when the validity is substantiated by analytical and engineering principles or reliable test data, or both.

2. Performance

2.1 General rules for design and installation of structural elements.

2.1.1 Structural elements should be designed avoiding as far as possible stresses perpendicular to the glue-line being a consequence of exterior loads.

2.1.2 With curved beams the thickness of their lamellae should be determined depending on the ratio r/t. See Part 2.
Thereby " r " is the radius of curvature of the individual lamella and " t " is its thickness.

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

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

2.1.5 In the final state as well as in the states of assembly sufficient precautions against upsetting should be provided by fastenings or similar measures. With vertically loaded beams having rectangular cross-section, the ratio should not exceed $h : B \leq 10$, if no additional precautions for stabilization are taken

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

2.2 End joints of lamellae:

Generally, end joints should be made by finger jointing. During manufacture, finger joints should be tested at random.
In the tension zones of beams, within one lamella, the distance between the end joints should not be smaller than 1.50m, otherwise special measures should be

taken to warrant the quality of the finger joints. For the transmission of loads, butt end jointing is not permissible.

2.3 Arrangement of the lamellae:

(1) In general, the quality requirements for laminated beams which consist of individual components having smaller cross-sections, need to be related only to the unit as a whole but not to the indivdual parts. However, the components being situated in the tension zone must by themselves correspond to the chosen quality class.
In laminated beams, which are strained by bending this applies to all lamellae in the outer 1/8 of the depth of the beam and at least to the two outer lamellae of the tensile zone; just so the upper of the cross- section, however at least two lamellae, must be made of boards without end joints or of lamellae with finger joints.
End joints in the interior of a laminated structural element which is shiefly strained by bending or compression may be butt joints. These end joints in neighbouring lamellae should be staggered ad distances of at least 50 cm.

(2) With glued laminated timber, the cross-section should ne made of at least 4 lamellae.

2.4 Glues:

Only glues beeing adequate for this field of application and beeing tested by an authorized institution may be used.
The use of thermoplastic glues is not permissible.
The shear strength of the glue must be at least as high as that of the wood, and at least correspond to the instructions of DIN 68141 or AFNOR.
The glues should be resistent against moisture as well as against chemical or other impacts. Therefore, as a rule, synthetic resin glues should be used.

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

2.5 Conditions of gluing:

2.5.1 Only well planed lamellae should be glued, deviations from the intended thickness of up to $\pm$ 0.2mm are tolerated [1)]

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

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

2.5.4 Temperatur and humidity:
The room temperatur during pressing should be at least as high as 18° C. Prior to gluing, the wood which is glued should be conditioned to room temperature in time.
The relative humidity in the gluing room should not be less than 40% and not more than 80%.
With radio frequency gluing, the related rules should be obtained.

Footnotes:

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

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

2.6 Supervision:

2.6.1 Supervision by the manufacturer himself:

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

The gluings carried out should be plotted in data sheet (gluing records), where all the gluing work is recorded showing kind and amount of the same.

The gluing records, which may be replaced by automatic records (EDV) as well, should comprise the following data:

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

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

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

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

For the supervision of the production shear-tests in the plain of the gluing area are recommended.

2.6.2 Supervision by competent authorities:

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

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

If the structural elements have more severe service conditions due to moisture a suitable surface protection is needed but it must be warranted that it doesn't interfere with the glue.
A corresponding protection against humidity during storage and erection has to be provided.
In general, wood preservatives should be applied only after gluing, as far as no guarantee is given that no risk is involved if they are applied earlier.

2.8 Marking of glued structural elements:

Glued structural elements should be given durable markings showing manufacturer and number of production.

3) Reference to other standards:

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

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TRUSS DESIGN METHOD FOR CIB TIMBER CODE

by

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OTANIEMI, FINLAND

JUNE 1980

TRUSS-DESIGN METHOD FOR CIB-TIMBER CODE

BY

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The Structural Research Laboratory
The Technical University of Denmark

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

Working Commission W18-Timber Structures.

Foreword.

At the meeting of CIB W18 in Perth, Scotland, June 1978, a sub-group was formed with the task of producing a section for the CIB-Timber Code on the design of trusses, especially with regard to the use of nail-plates.

This paper is meant as a proposal for the section "Trusses" in the CIB-Timber Code.

It is recognised that at present a rigorous theoretical analysis of forces, moments and deflections will generally not be possible because of the lack of necessary data in sophisticated computerized method.

Therefore, three methods of analytical design are given. The structural design methods are of increasing complexity, but allow for sophisticated analysis where sufficiently reliable data are established from test.

7. TRUSSES

7.0 General

The design of a trussed rafter must take the following effect and aspects of truss behaviour into consideration: eccentricities, slip and rotation of joints, strength and stiffness variation in joints and elements and stress-redistribution (ductility).

7.1 Design methods

The structural adequacy of a trussed rafter design may be established by one of the methods given below:

A. Pin-joint analysis.

B. Linear Frame analysis

C. Non-linear Frame analysis (limit state analysis)

Pin-joint analysis.

The axial forces are determined from a statically determinant mathematical model which assumes hinges in all nodal points. The moments in continuous members are determined from moments coefficients. The coefficients are estimated and will in general take into account rigidity of joints and ductility of timber, and strength and stiffness variation in members.

Linear Frame analysis.

The non-determinant mathematical model used must be capable of reproducing the effect from eccentricities and semi-rigidity of joint.

The fictitious members used in the model to simulate these effects must be determined by joint test.

The model used in the analysis must be checked by full-scale test to ensure reasonably realistic behaviour compared with the real structure.

Limit-state analysis.

The non-determinant mathematical model used in the calculation must be capable of reproducing the effect of non-linear behaviour of members and joints.

The calculation method can be a step by step analysis or a plastic-collapse mechanism method. A check for sufficient yield and moment rotation capacity must be performed to ensure the correct solution of the ultimate load-carrying capacity.

Any of the design methods must produce the following design parameters for use in the design criteria of the individual members and joints in the truss.

1. Axial forces.
2. Bending moments.
3. Effective buckling length.
4. Deflections.

7.2 Members

The design of members should be in accordance with the recommendations of section 5.1. Each member should be capable of safely resisting the appropriate axial forces and bending moment..

Compression members should be designed for in-plane bending and compression and out-of-plane compression. The effective buckling length should be taken as the distance between points of contraflexure.

7.3 Joints

All joints should be designed to transfer axial forces and bending moments which are assumed to be in the design of the members joined. Joint moment capacity can only be disregarded if the members' fixity in the joint is assumed pinned, and the eccentricity is small.

7.4 Deflections

In addition to deflection due to axial forces and bending moments in members, allowance should be made for deflection due to joint slip. The total deflection of any joint of the truss can be obtained from the following formula:

$$\delta_t = \Sigma \frac{N_a N_o}{EA} l + \int \frac{M_a M_o}{EI} dx + \Sigma \frac{GF_p}{S}$$

where the three contributions arise from axial forces, bending moments and slip in joints.

List of References

1. British Standard Instruction BS 5268: Part 3 THE STRUCTURAL USE OF TIMBER TRUSSED RAFTERS FOR ROOFS (Draft March 1980).

2. Cahier 111 au Centre Technique du Bois: RECOMMANDATIONS POUR LE CALCUL DES CHARPENTES INDUSTRIALISEES ASSEMBLEES PAR CONNECTEURS OU GOUSSETS 1978.

3. Truss Plate Institute
DESIGN SPECIFICATION FOR METAL PLATE CONNECTED WOOD TRUSSES 1978.

4. Institute for Industrial Research and Standards.
Irish standard specification: TIMBER TRUSSED RAFTERS FOR ROOFS. I.S. 193P: 1978.

CIB-W18/13-14-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TRUSSED RAFTERS, STATIC MODELS

by

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DENMARK

OTANIEMI, FINLAND
JUNE 1980

TRUSSED RAFTERS, STATIC MODELS

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Background

February 1979 there was held a meeting at the Danish Building Research Institute with participants from Finland, Norway, Sweden, and Denmark. It was decided to form a working group, which should propose guidelines for static models of trussed rafters. The members are: Erik Aasheim, NTI, Norway. Bo Källsner, STFI, Sweden. Tuomo Poutanen, Tammersfors Tekniska Högskola, Finland. Hilmer Riberholt, ABK, DTH, Denmark.

This working group (named the model group) cooperates with another nordic group which deals with the connections (the plate group). Up to now there has been formed no group which deals with the strength control of the timber. The working groups primaryly deal with trusses assembled with toothed plates.

Introduction

There exist several research reports on the strength and stiffness of timber trusses and almost just as many methods to analyse. There occurs of course a lot of effects in trussed rafters which should be taken into account, and every single effect gives often cause to a new method. If one adds to this all the effects due to workmanship one gets a rather complicated problem.

At the moment in praxis there is emploied some simple analysis methods, f.ex. based on a pinned joint model and some tricks with moment coefficients. These models have been employed succesfully for many years on the truss types used up to now. But the types of trusses are changing and how does this influence the analysis methods?

At the meeting February 1979 it was agreed to set up a general analysis method and compare it with the test results in the available research reports. The general analysis method will be based on a frame model which will be made as simple as possible, but still be able to reflect the most important effects in trussed rafters. A comparison with test results and analyses with more refined methods will reveal the most important effects which must be taken into account.

It is envisaged that the amount of work in setting up and testing a general analysis method is considerable. But if one wants to attach importance to the guidelines these must be elaborated carefully, and special attention must be paid to the implemention in praxis.

A brief description of the basic ideas in the nordic group.

The rest of this paper contains an abstract of "Retningslinier for statiske modeller af træspær. Andet udkast. November 1979". (English translation: Guidelines for static models of timber trussed rafters. Second draft. November 1979).

Of practical reasons the dimensioning of a truss has been divided into:

1. Analysis of forces and moments.

2. Strength control of timber.

3. Strength control of joints (plates).

The analysis of forces and moments depends in general of the timber sizes and the stiffness of the joints. Introductory investigations indicates that it is sufficient to take the joint stiffness into account in a rough way; the joints are modelled as either pinned joints or moment stiff joints.

The guidelines will be prepared with reference to 2 methods. Under the preparation of the guidelines there will be a need of more refined methods. The methods are:

1. Simple practical methods for f.ex. hand calculation of certain often used truss types.

2. General practical method based on a linear elastic frame model.

3. Realistic methods for scientific use, f.ex. for the calibration of method 1 and 2.

It is the intention to set up general guidelines for method 2 and to propose simple practical methods for a few common used truss types. The simple methods will be calibrated so that they give the same results (dimensions of timber and plates) as method 2.

Some essential assumptions for a general practical method, (method 2).

GEOMETRY

As a principal rule it must be valid that outside the joints the system linies of the beam elements must coincide with the centre lines of the lumber.

Where the incoming systemlinies do not cross each other in the same point, fictitious beam elements must be introduced in order to connect the beam elements. Under centain conditions the fictitious beam elements may be omitted.

PHYSICAL ASSUMPTIONS

All beam elements are assumed linear elastic. The stiffness of the fictitious beam elements is normally set to a large value, but in some cases specific values are prescribed.

The joints are normally modelled as either pinned or stiff joints. The real deformations in the joints are not taken into account in the force and moment analysis, partly of practical reasons and partly due to lack of information. In the calculation of the deflection of the truss the deformation in the joints can of course be included.

STATIC MODELS OF THE JOINTS

As a principal rule the joints are assumed to act as pinned joints. There is given some guidelines for the placing of the charniers relative to the geometry of the joint, see enclosure 1.

In same cases the joints are assumed to be modelled most correctly as moment stiff joints, see enclosure 1.

SUPPORTS

Unless the supports are designed in a special way they are assumed to act as pinned supports, eventually with a suitable degree of freedom.

LOADINGS

The loads must principally act on the truss in the form and position as they occur in reality.

The cords are often loaded with a row of single loads with equidistant spacing. When the spacing is less than 2/5 of the distance between the nodes (joints) it is accepted to let the single forces be equivalent to a line load. (The error in the maximum moments is less than 10%).

If it occurs, it is essential to take into account that single forces, among these the reactions, in reality act eccentric on the joints in a truss. Eccentricities may cause very large moments in the chords.

STATICS

It is acceptable to carry out the analysis of forces and moments with a first order theory, e.g. where the equations of equilibrium are set up in the non-deformed state. The second order effects must be taken into account in other ways. There will be given guidelines for the effective column lengths, based on comparisons with results of a geometric non-linear frame program.

MISCELLANEOUS

In order to assure that the trusses will act as assumed by the analysis of forces and moments, there will be set up some requirements to the production, the transport, the stability in the finished building, connections to light partition walls and so on.

Simple practical methods

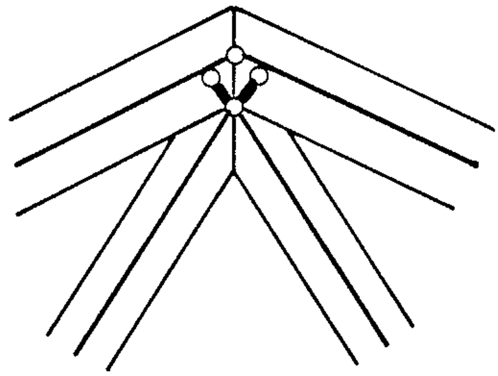
Up to now the moment coefficient method has been emploied succesfully, and it looks like that it still will be applicable to some types of trusses. The moment coefficients must be determined so that they give the same result as the frame model gives.

In some types of trusses the moment coefficients are not useable, f.ex. trusses supported eccentrically at the heel joint. In this case some simplified formulas can give a practical tool for hand calculations, see f.ex. Wood trussed rafter design, Marius Johansen et al, Danish Building Research Institute, 1980.

4 PLADEAREALER

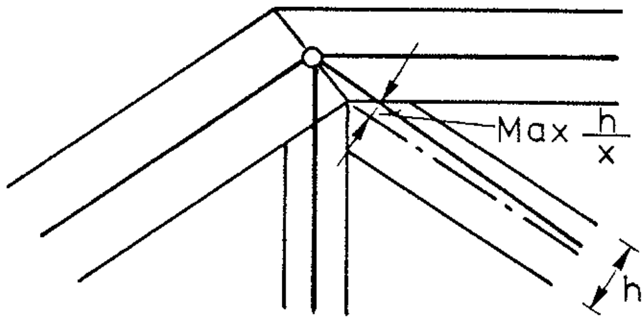
F

Kipsamling



G

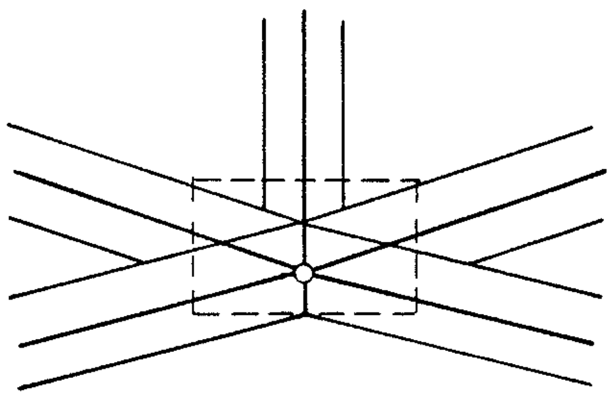
Samling i valm



5 PLADEAREALER

H

Samling i saksespær.



ENCLOSURE 1 Abstract of: Retningslinier for statiske modeller for træspær.

B

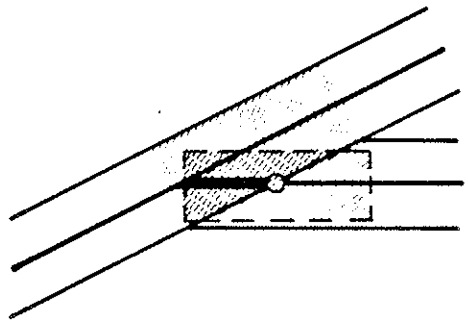
Vilkårlig skarring.

Træk: Pladerne skal overføre hele kraften.

Tryk: Pladerne skal kunne overføre 2/3 af kraften

C

Knæk i hoved eller fod.

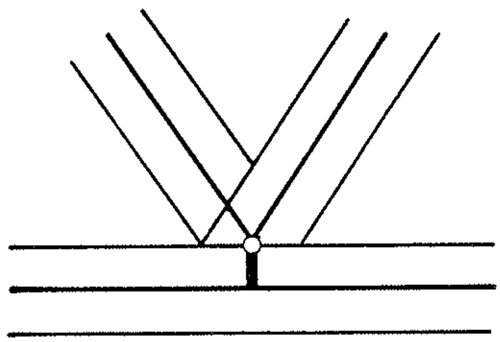


3 PLADEAREALER

D

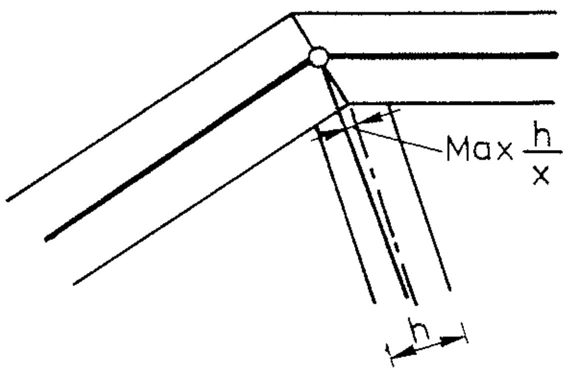
K-forbindelse.

To diagonaler - hoved eller fod



E

Samling i valm



Examples of moment stiff connections

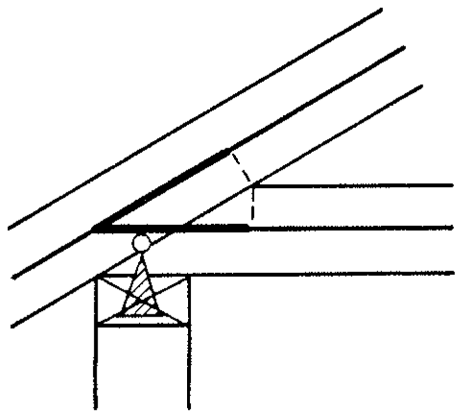
5.2 Eksempler på ueftergivelige samlinger.

I det følgende er der angivet eksempler på ueftergivelige samlinger, eller samlinger, der har omtrent samme virkemåde.

Der skal tilstræbes en samling af type A

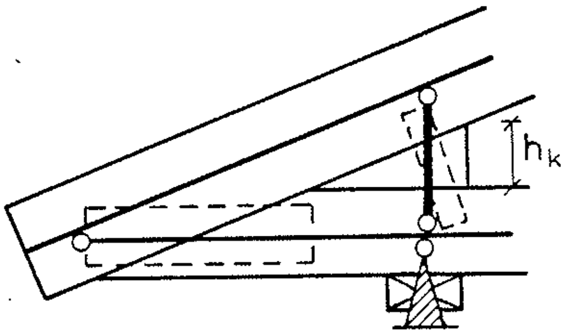
A Heel joint

Tagfod. Dersom samlingen ved tagfod påvirkes med en væsentlig ydre kraft, f.eks. understøtningsreaktionen, der skaber tryk i kontaktfladen mellem trædelene, så kan følgende modeller anvendes. Det er en forudsætning, at kraftens angrebslinie skærer igennem kontaktfladen.



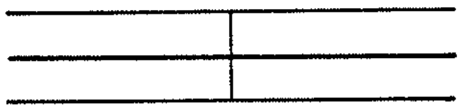
A Heel joint

Hvor indrykningen er stor skal der anbringes en kile, så remmen er dækket. Dersom højden af kilen h_k er større end xxx mm, skal fiberretningen i kilen være vinkelret på enten hoved eller fod. Kilen skal altid fastholdes af en separat tandplade, der skal kunne overføre en kraft på xxx.



B Scarf

Skarringer i hoved og fod. Ders.m disse lægges højst xxx fra momentnulpunkterne eller dersom de dimensioneres for den beregnede normalkraft samt 1,x gange det beregnede moment, så kan samlingen antages at være ueftergivelig.



CIB-W18/13-14-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMPARISON OF 3 TRUSS MODELS DESIGNED BY DIFFERENT ASSUMPTIONS FOR SLIP AND E-MODULUS

by

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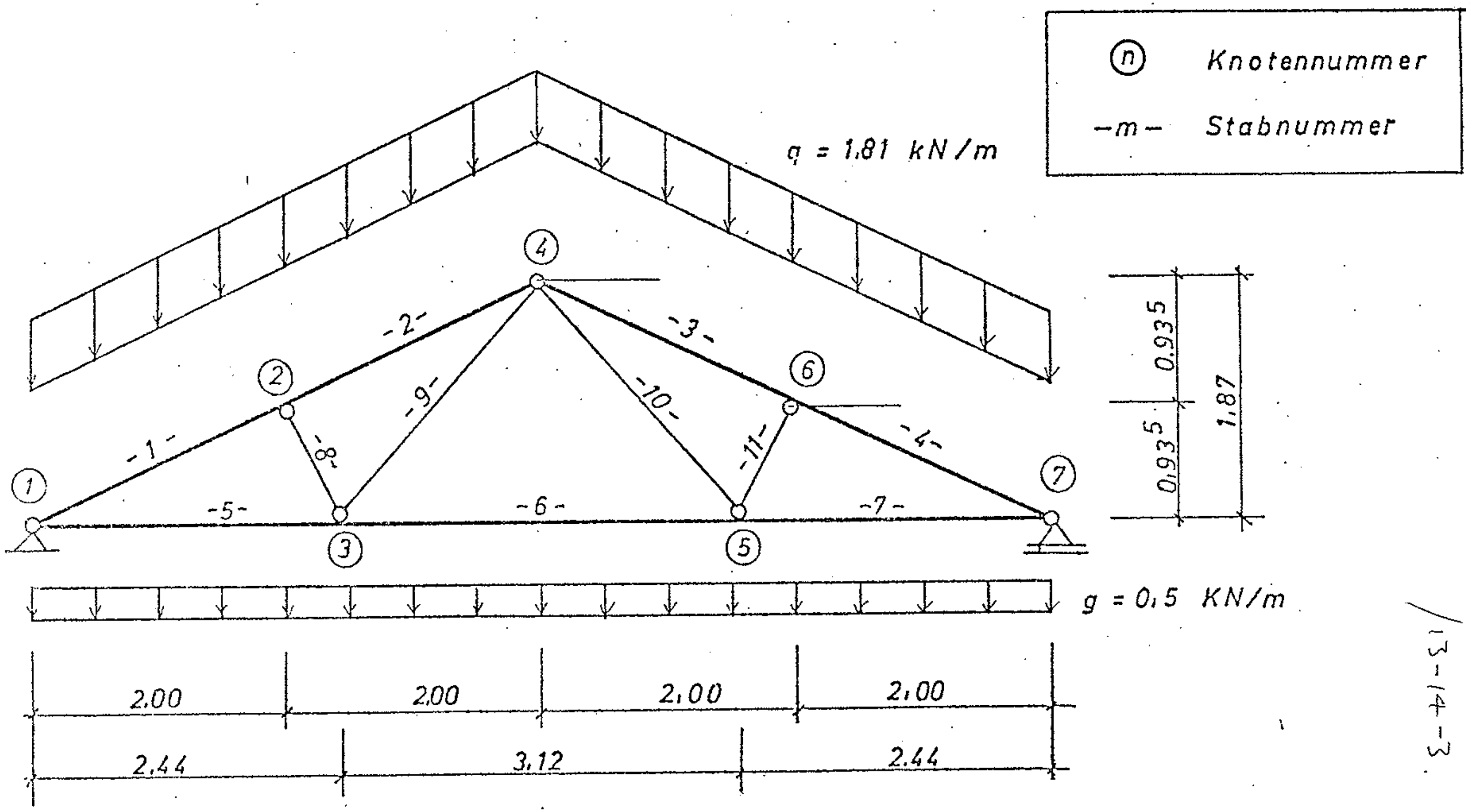
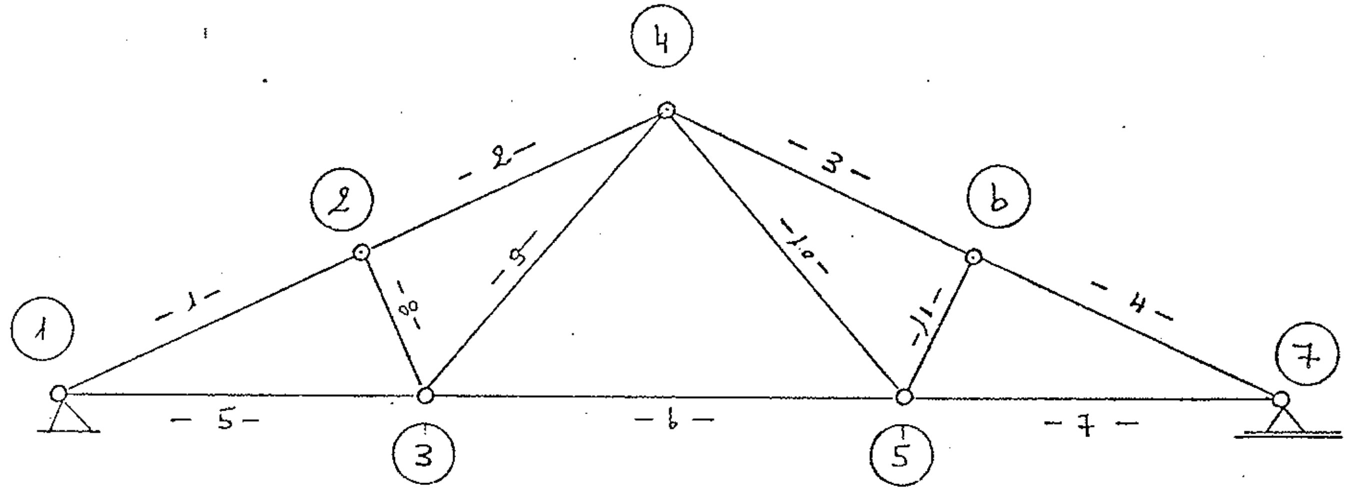
FEDERAL REPUBLIC OF GERMANY

OTANIEMI, FINLAND
JUNE 1980

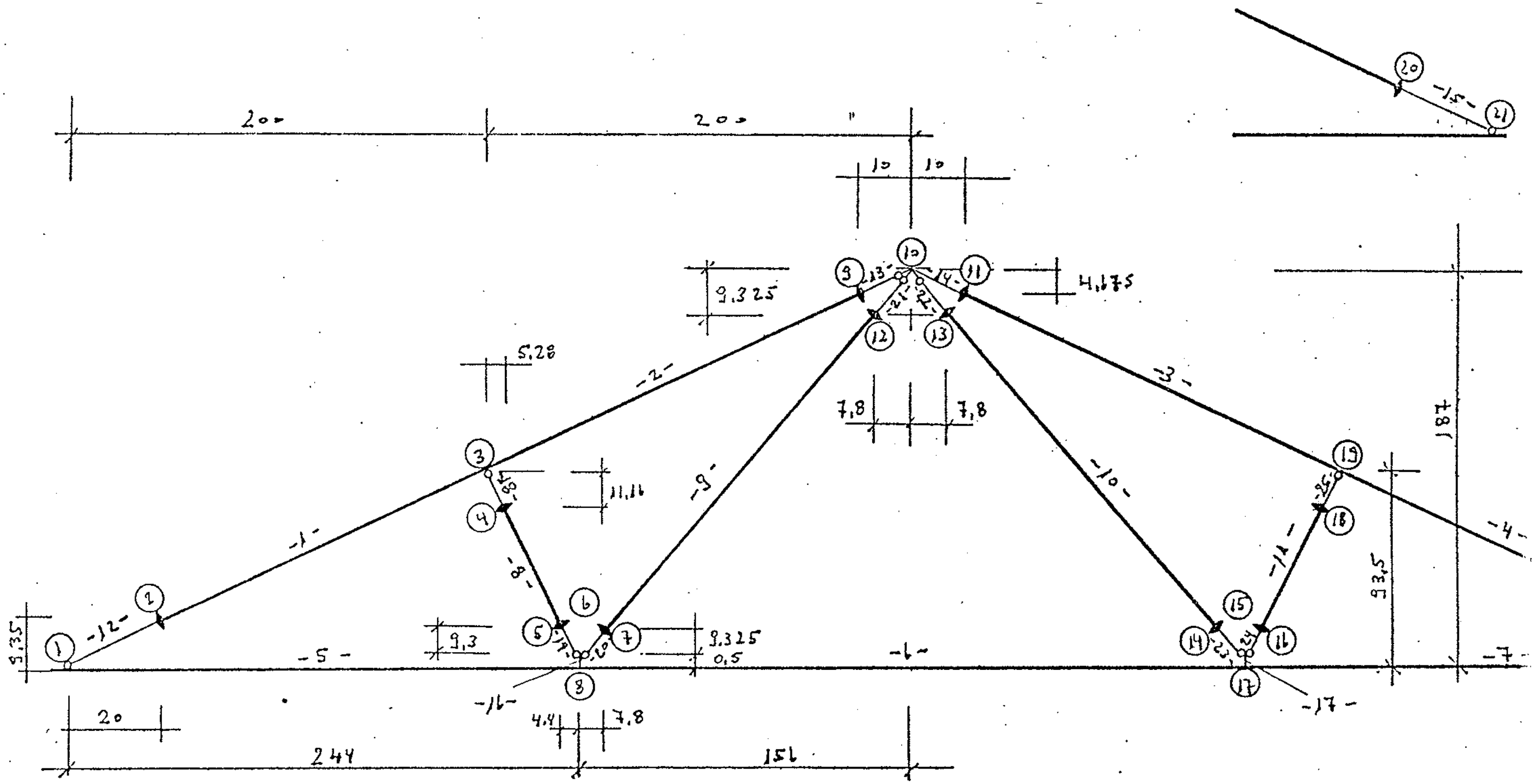


<u>Binder System 2</u>



Binder System 1

Bei der Berechnung des Biegemoments des Obergurtes und des Untergurtes wurden diese als Durchlaufträger auf "starren Stützen" betrachtet.



Binder System 3

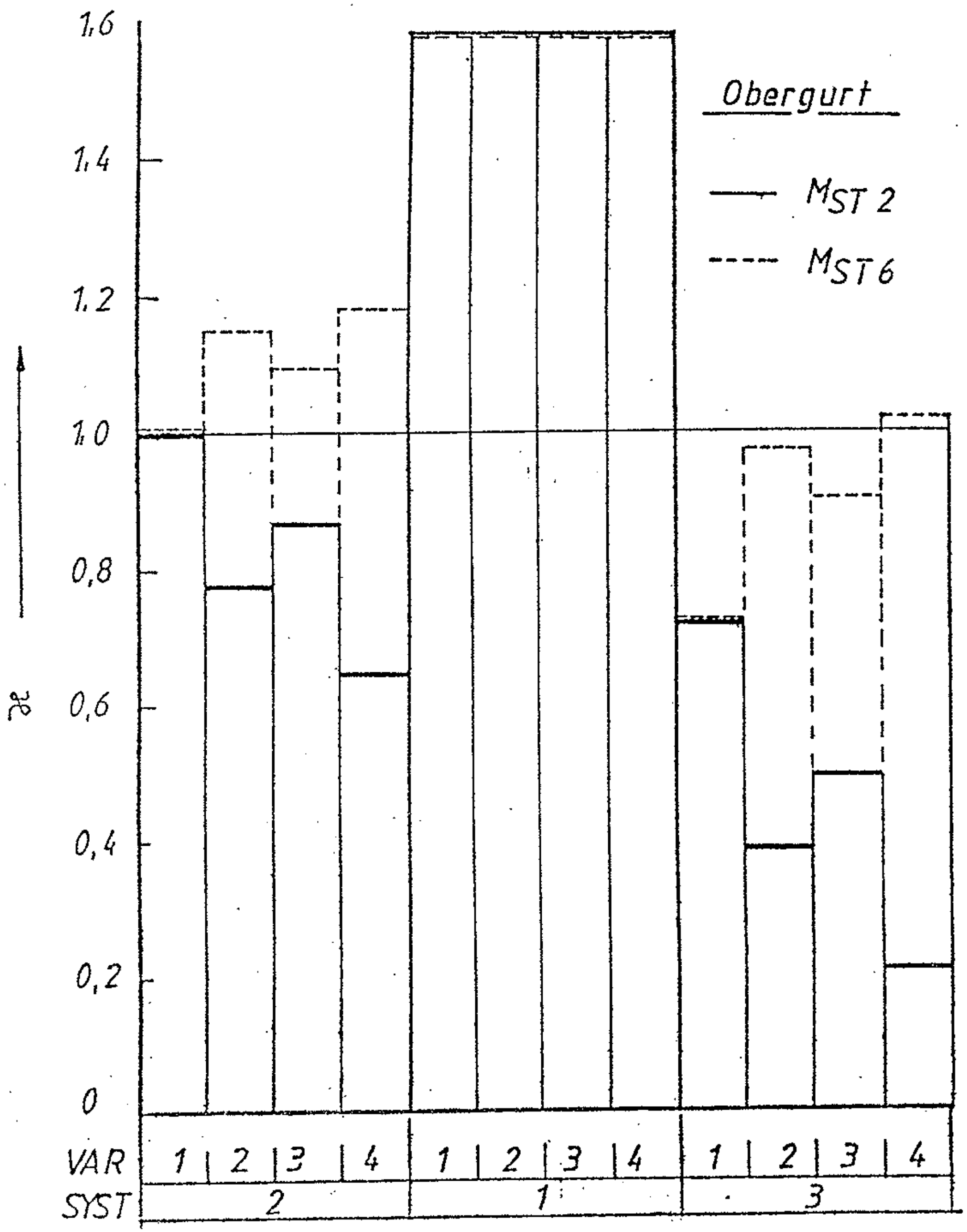
E-Modul in N/mm^2

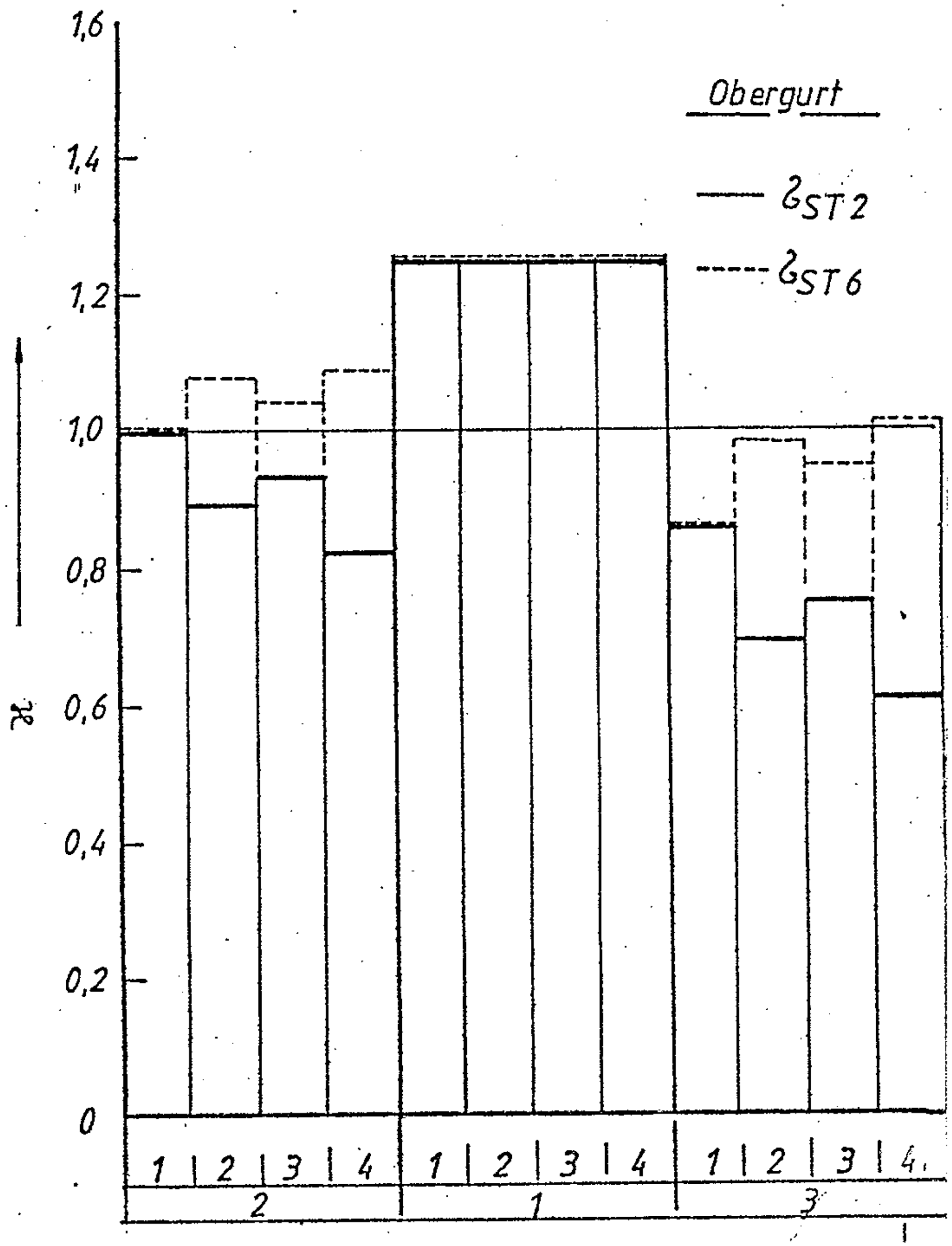
Stab	Var. 1	Var. 2	Var. 3	Var. 4
O1	10.000	16.000	16.000	16.000
O2	10.000	16.000	16.000	16.000
O3	10.000	6.000	6.000	6.000
O4	10.000	6.000	6.000	6.000
U1	10.000	10.000	10.000	10.000
U2	10.000	10.000	10.000	10.000
U3	10.000	10.000	10.000	10.000
D1	10.000	10.000	16.000	6.000
D2	10.000	10.000	16.000	6.000
D3	10.000	10.000	6.000	16.000
D4	10.000	10.000	6.000	16.000

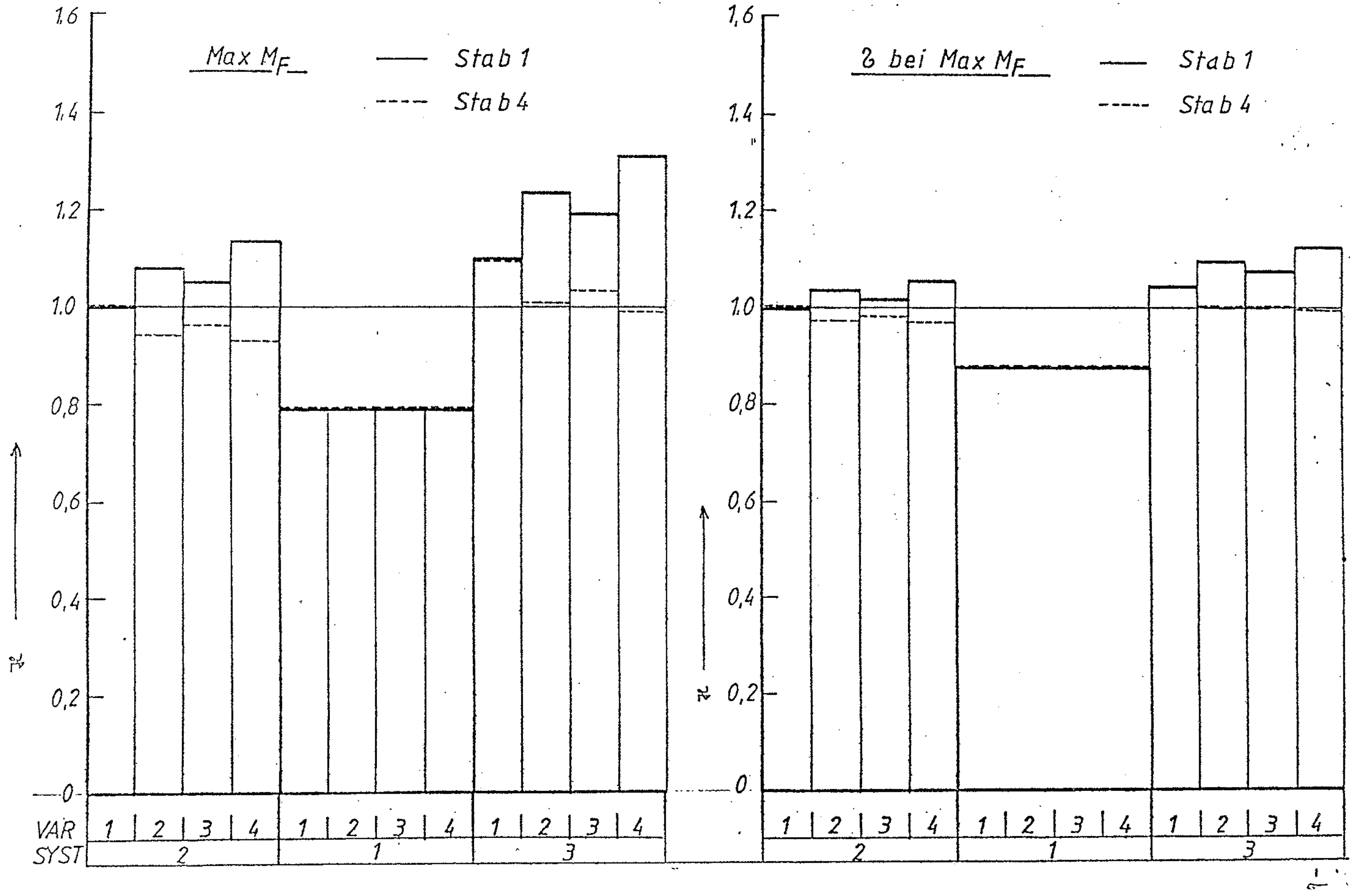
Bei der Darstellung der Momente, Stabkräfte und Spannungen wurden alle Größen auf die Werte vom Grundsystem (System 2 VAR 1) bezogen.

$$\Rightarrow \quad \mathcal{H} = \frac{\text{im jeweiligen System vorh. Schnittgr. bzw Spannu}}{\text{Schnittgr. bzw Spannung im Grundsystem}}$$

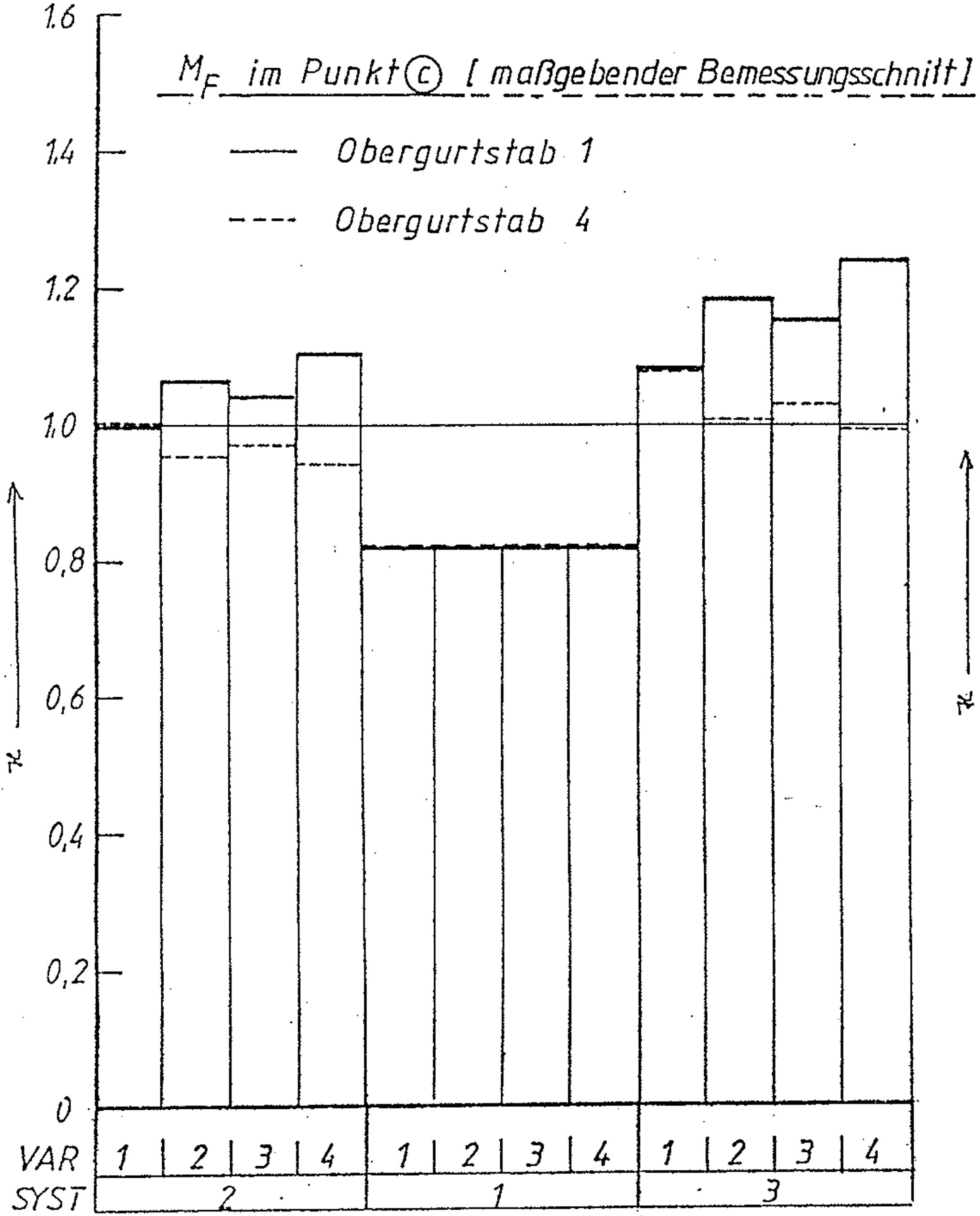
Die in den Diagrammen verwendeten Bezeichnungen beziehen sich auf die Knoten bzw Stabnummerierung von System 1 und 2


Obergurt
$M_{ST\,2}$
$M_{ST\,6}$
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
ϰ
VAR
SYST
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3

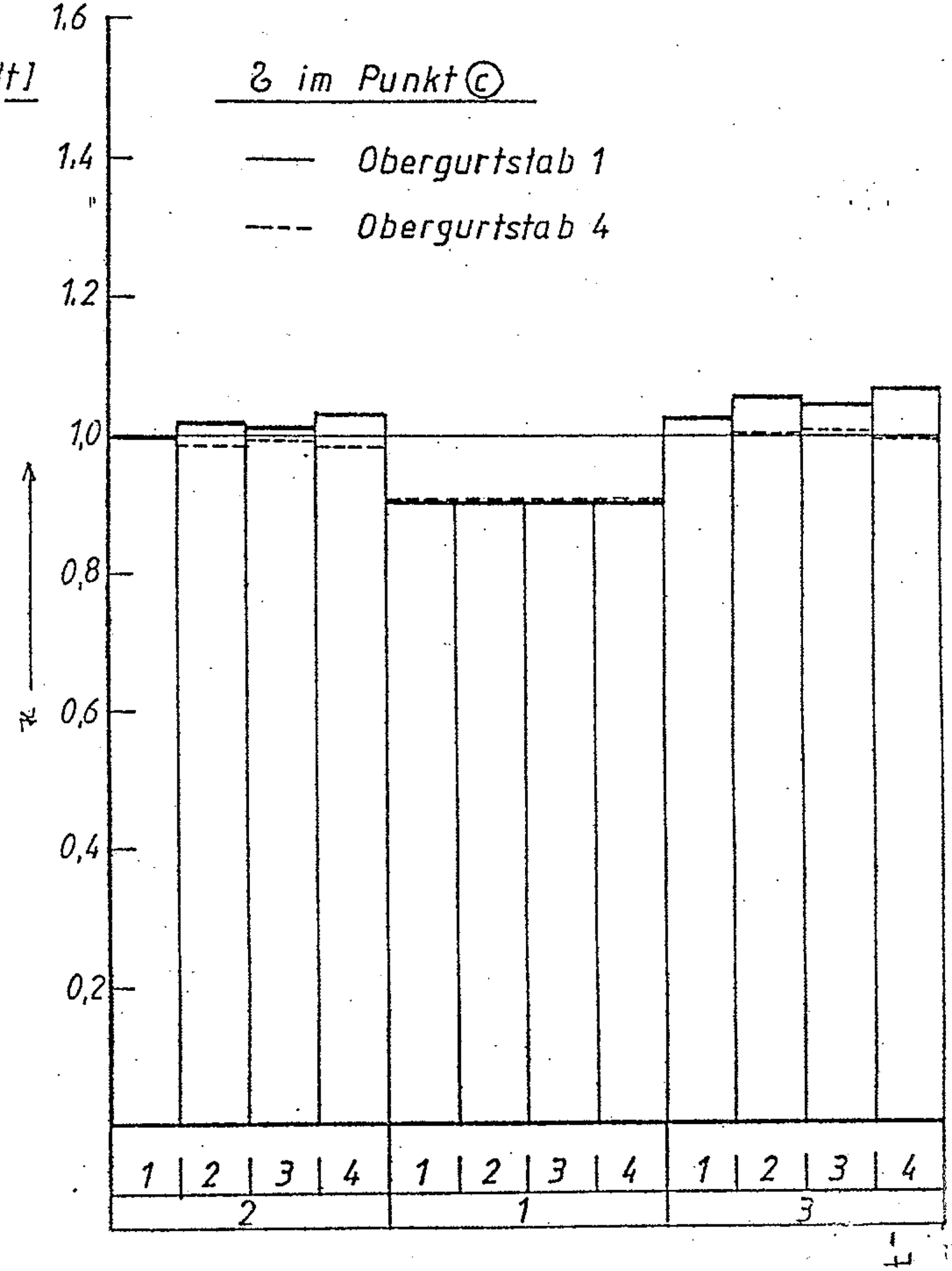

Obergurt
$\tau_{ST\,2}$
$\tau_{ST\,6}$
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
ϰ
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



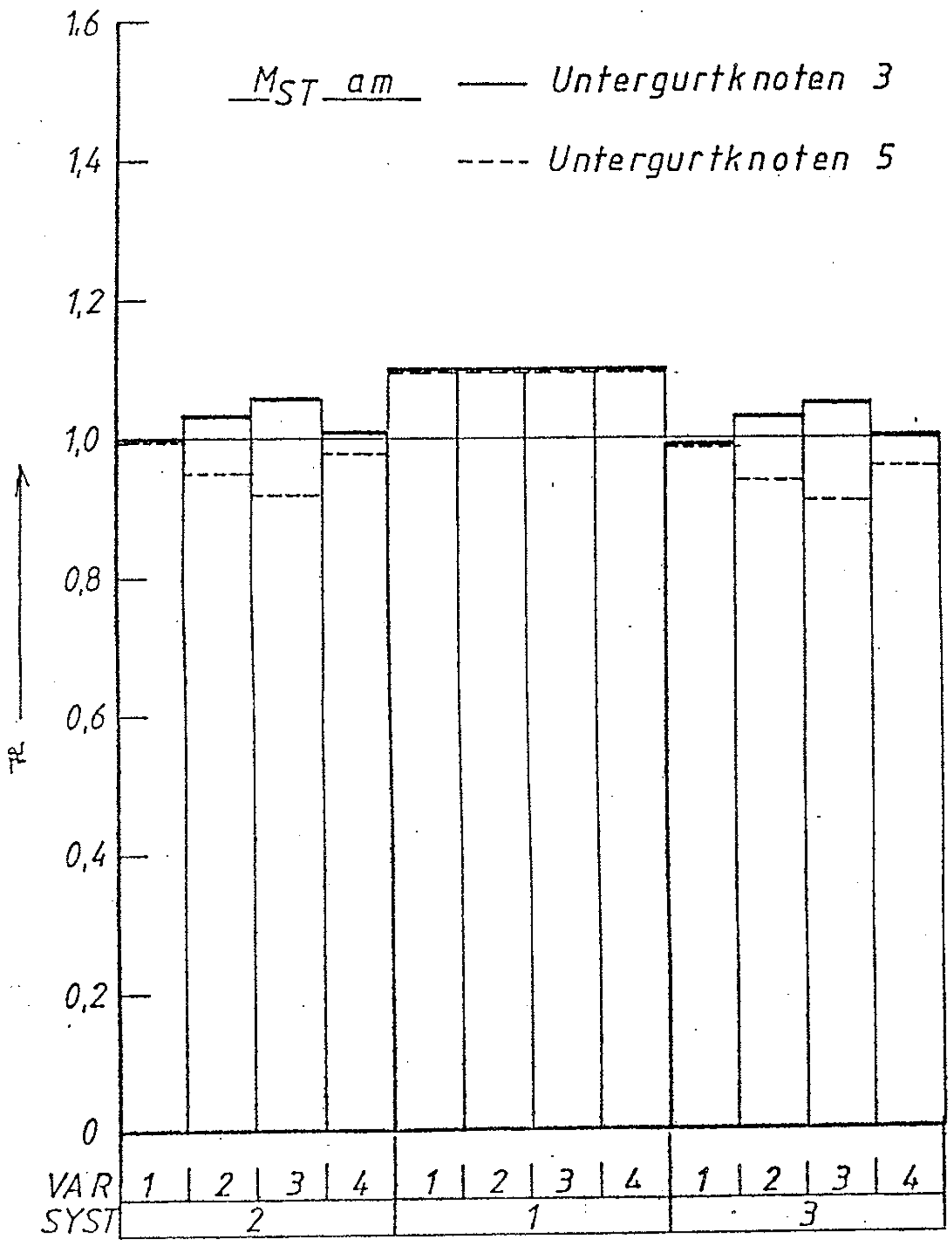
Max M_F
Stab 1
Stab 4
ϰ
1,6
1,4
1,2
1.0
0,8
0,6
0,4
0,2
0
VAR
SYST
ϐ bei Max M_F
Stab 1
Stab 4



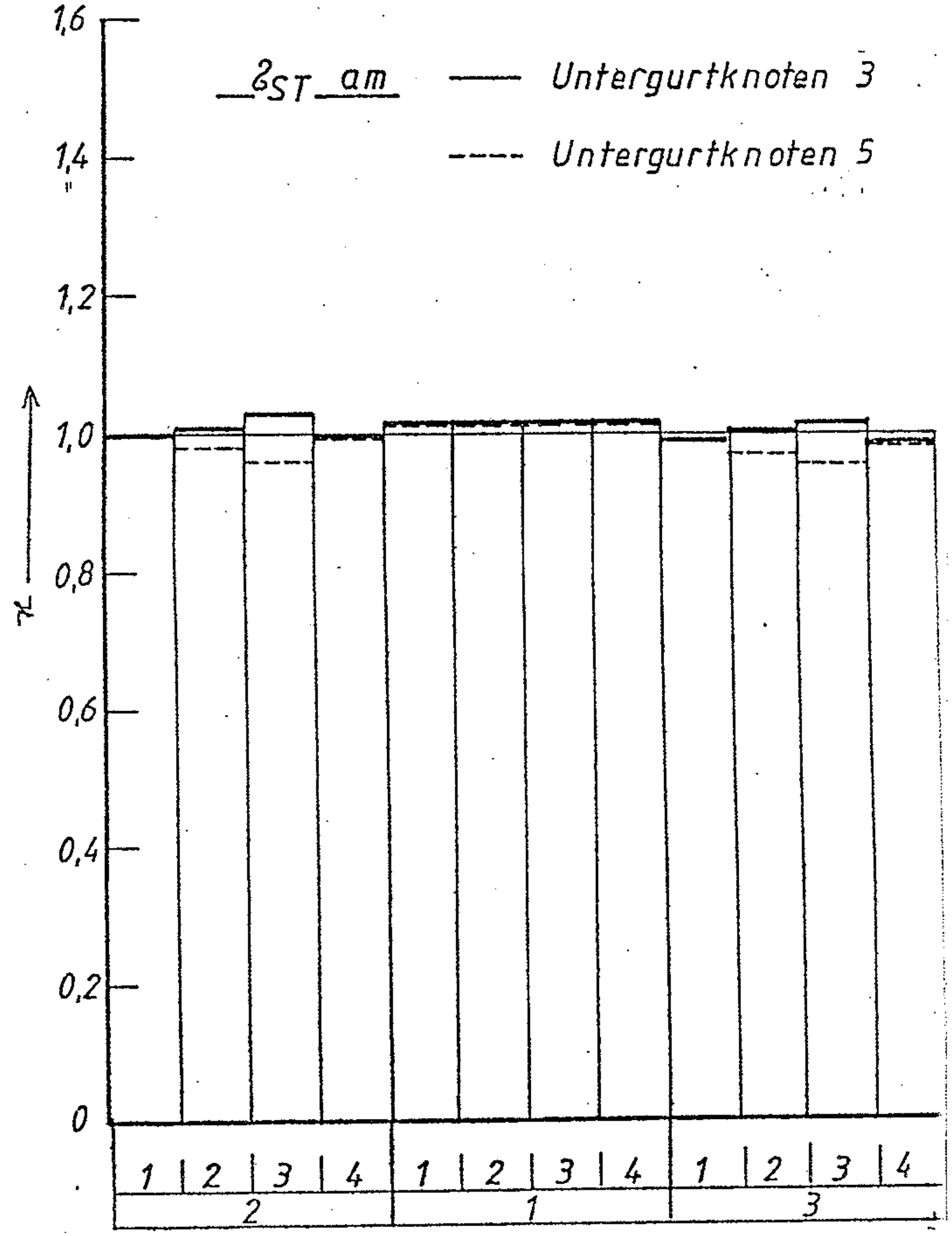
M_F im Punkt c [maßgebender Bemessungsschnitt]
Obergurtstab 1
Obergurtstab 4
1.6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
ϰ
VAR
SYST
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



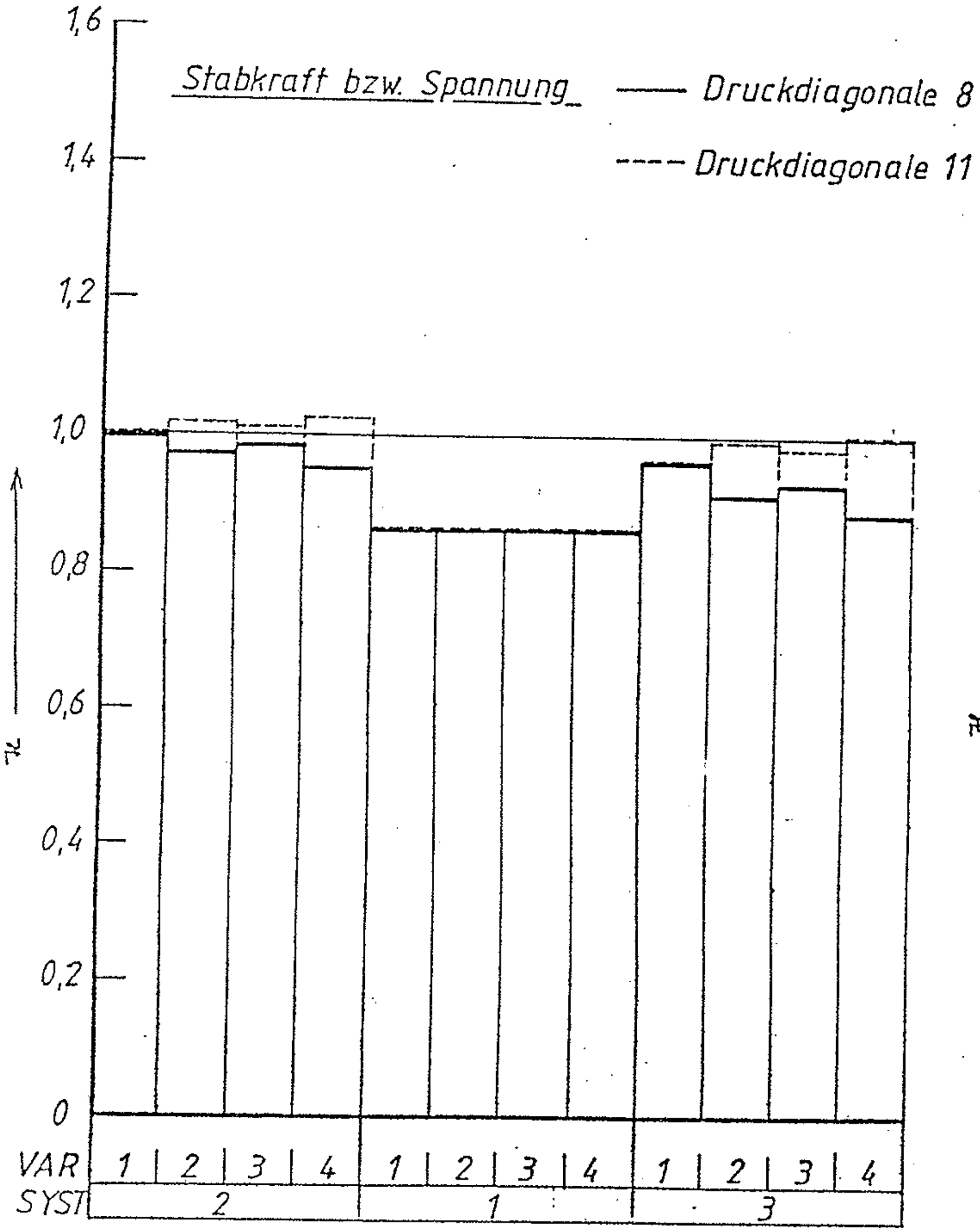
ϑ im Punkt c
Obergurtstab 1
Obergurtstab 4
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
ϰ
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3

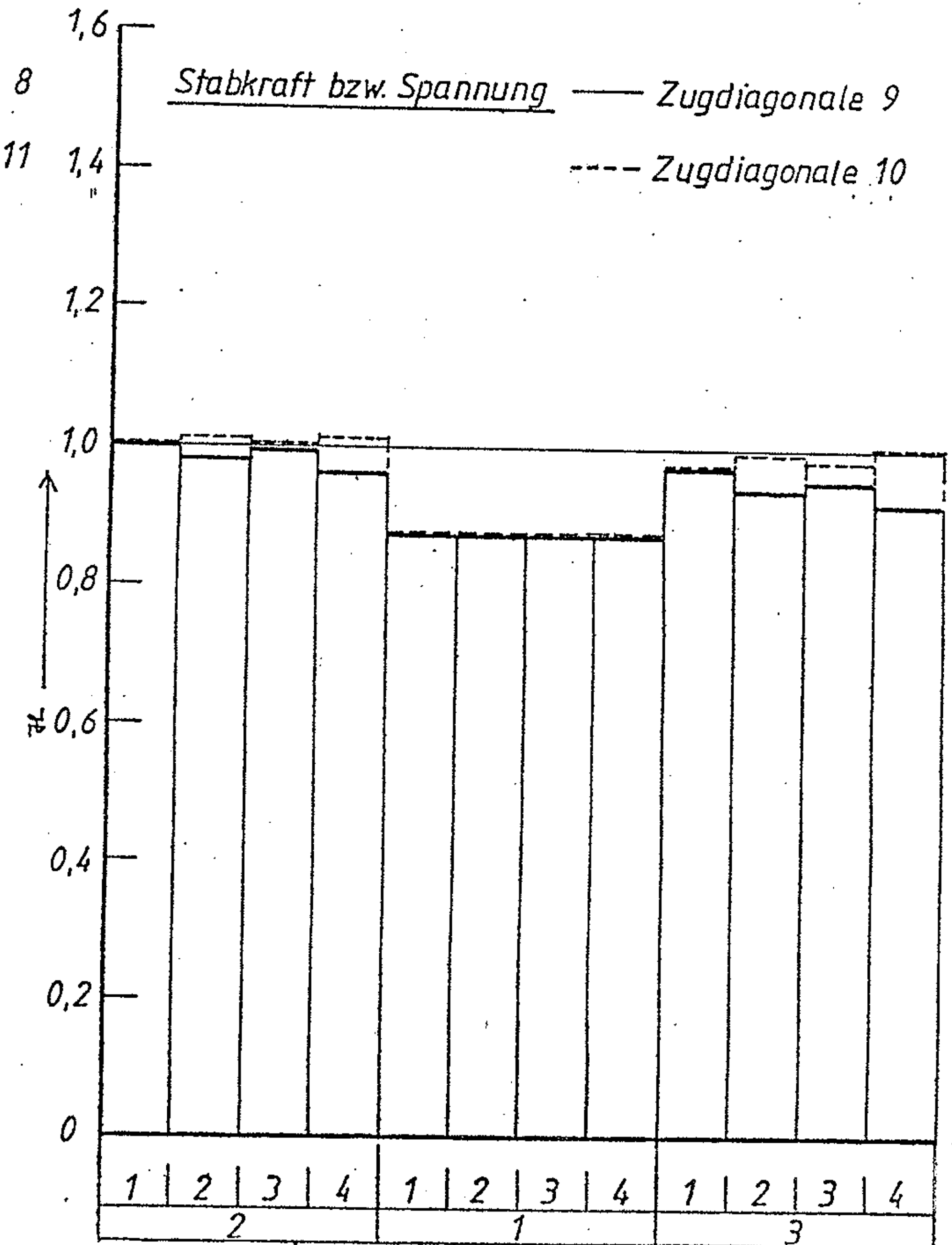


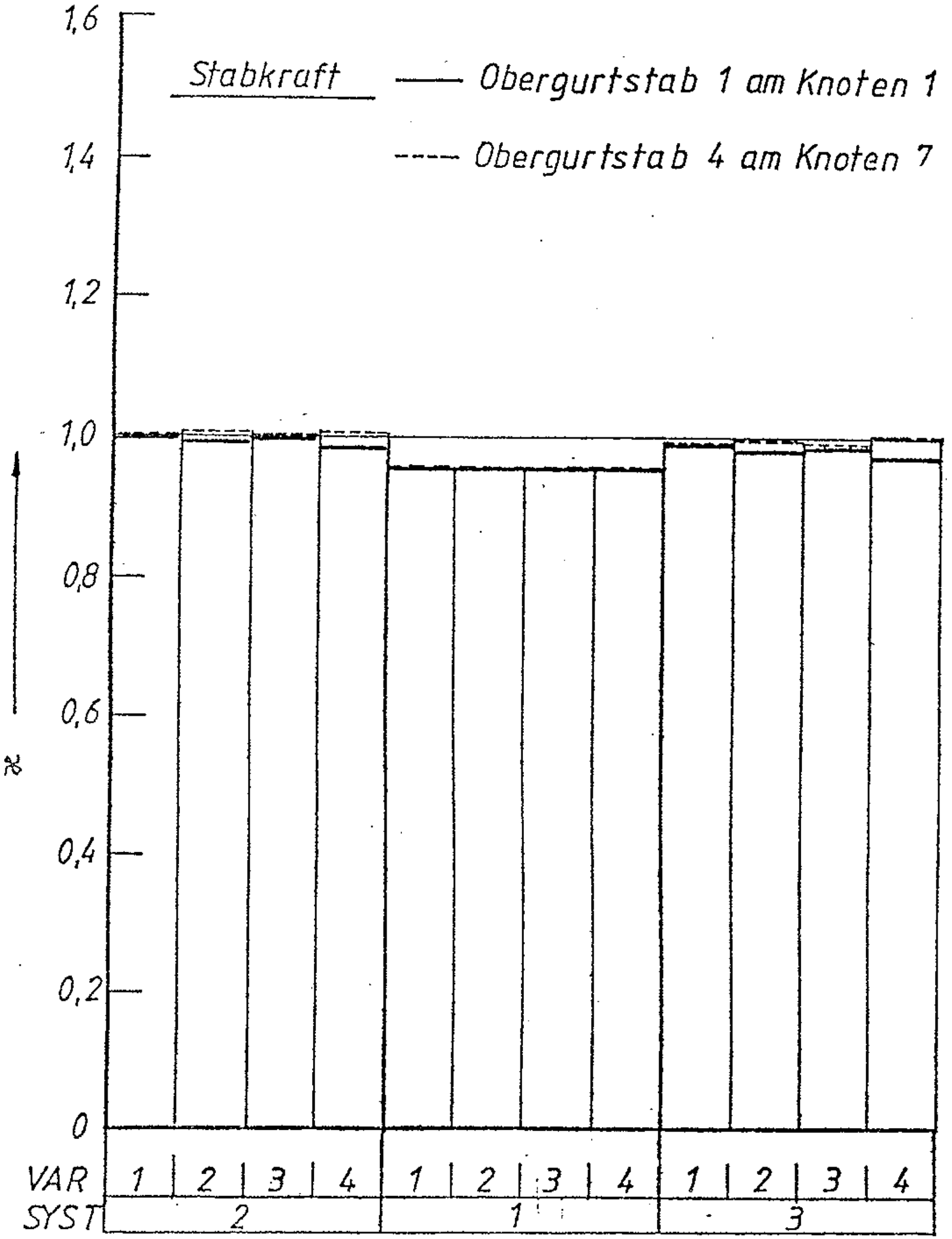
M_{ST} am
Untergurtknoten 3
Untergurtknoten 5
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
$\varkappa$
VAR
SYST
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



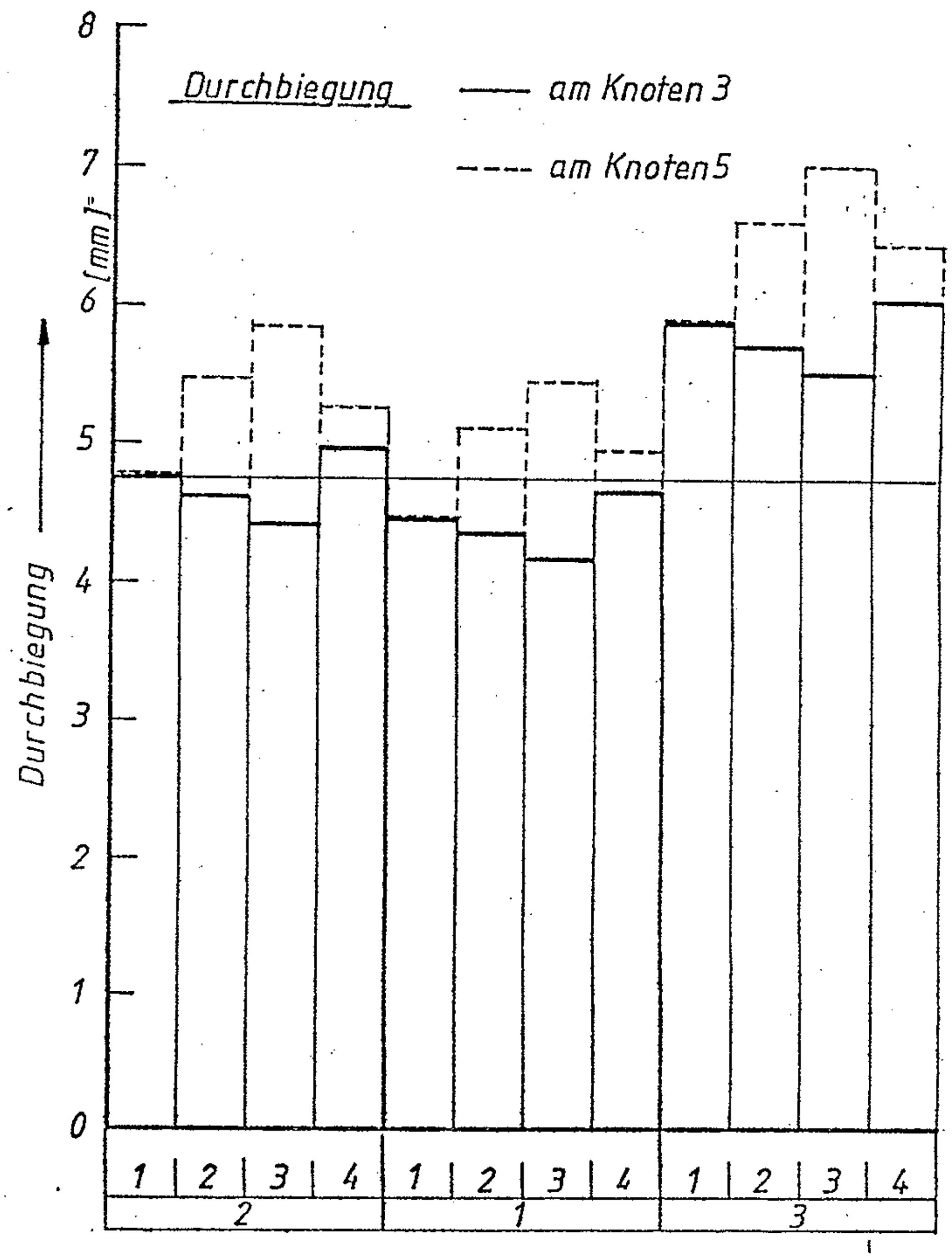
ϱ_{ST} am
Untergurtknoten 3
Untergurtknoten 5
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
$\varkappa$
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3


1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
Stabkraft bzw. Spannung
Druckdiagonale 8
Druckdiagonale 11
ϰ
VAR 1 2 3 4 1 2 3 4 1 2 3 4
SYST 2 1 3

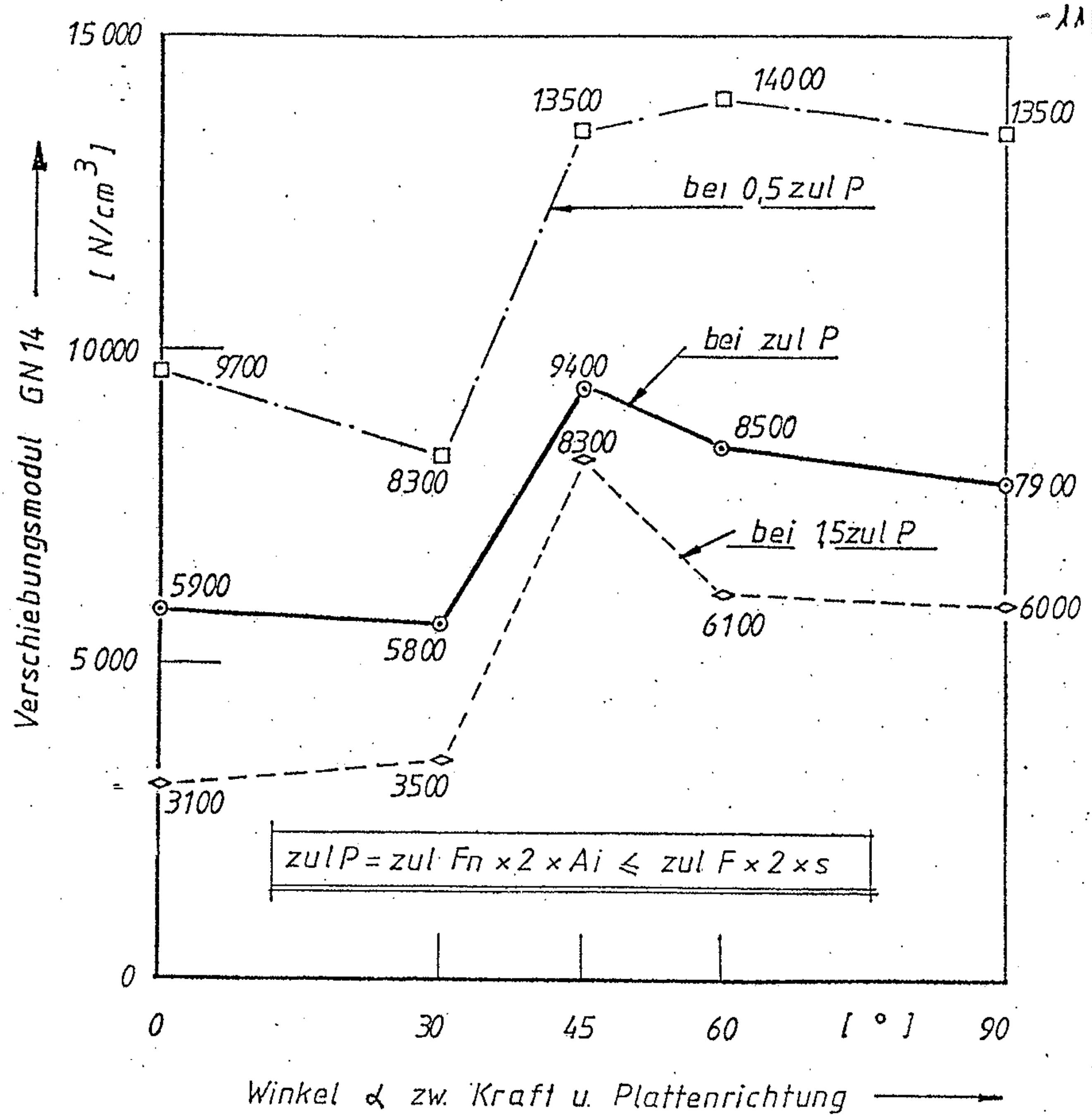

1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
Stabkraft bzw. Spannung
Zugdiagonale 9
Zugdiagonale 10
ϰ
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



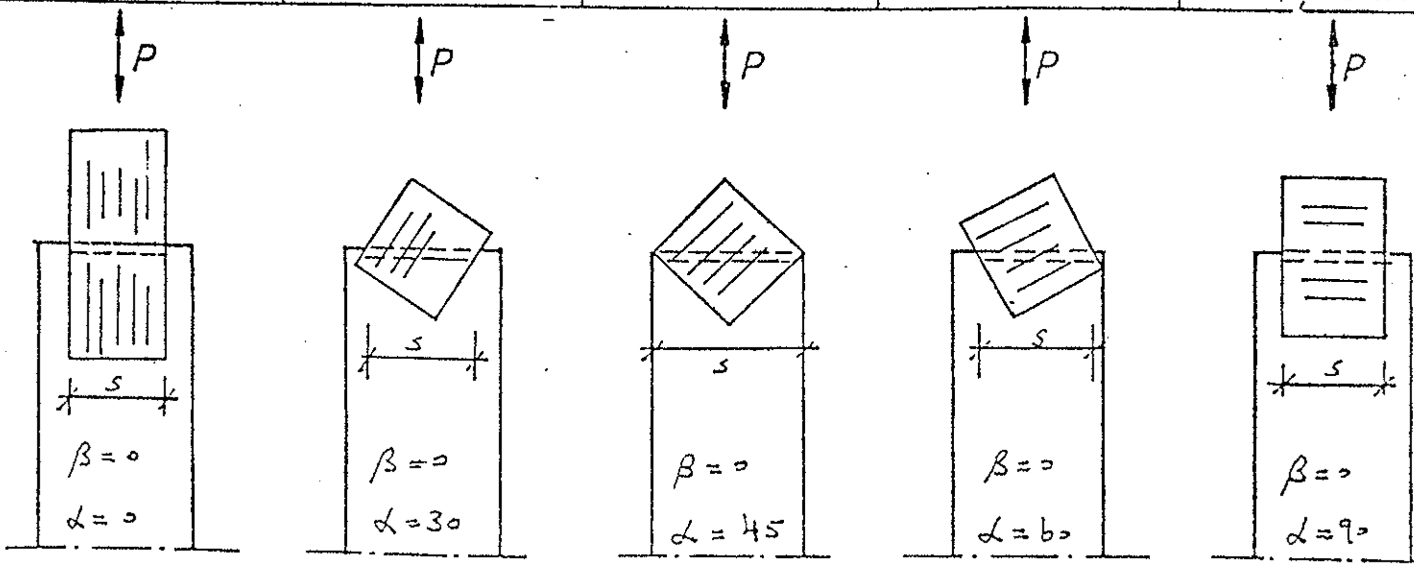
Stabkraft
Obergurtstab 1 am Knoten 1
Obergurtstab 4 am Knoten 7
1,6
1,4
1,2
1,0
0,8
0,6
0,4
0,2
0
ϰ
VAR
SYST
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



Durchbiegung
am Knoten 3
am Knoten 5
8
7
6
5
4
3
2
1
0
Durchbiegung
[mm]
1 2 3 4 1 2 3 4 1 2 3 4
2 1 3



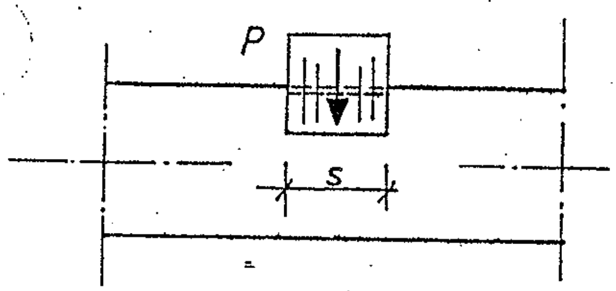
zul Fn = 120	104	96	88	72 N/cm
zul F = 1580	880	530	530	530 N/cm

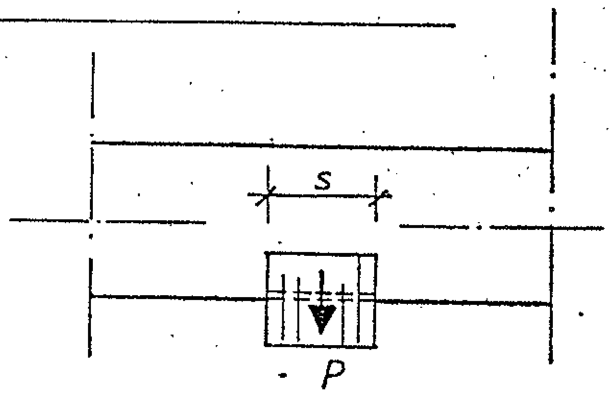


$\beta = 90^\circ \quad \alpha = 0^\circ$

$$\boxed{zul\,P = 60 \times 2 \times A_i \leq 1580 \times 2 \times s}$$

C' [zul P] = 6100 N/cm^3	2600 N/cm^3
C' [0,5 zul P] = 6800 N/cm^3	3600 N/cm^3
C' [1,5 zul P] = 4200 N/cm^3	2100 N/cm^3

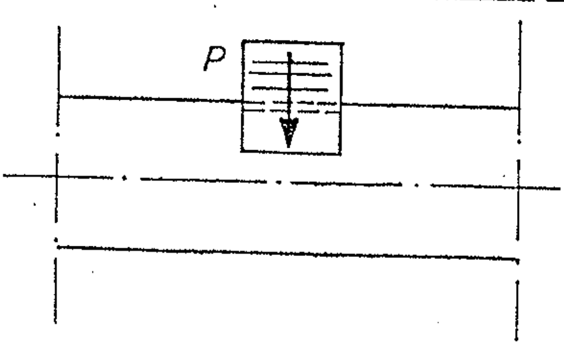


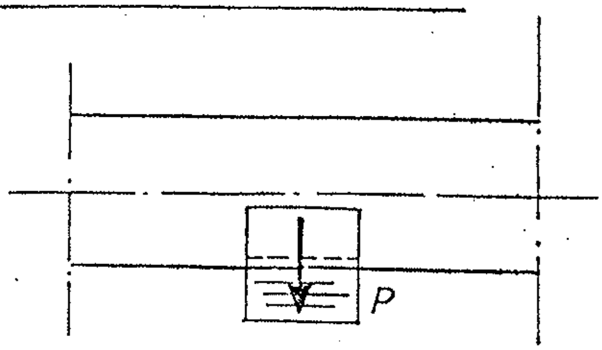


$\beta = 90^\circ \quad \alpha = 90^\circ$

$$\boxed{zul\,P = 60 \times 2 \times A_i \leq 530 \times 2 \times s}$$

C' [zul P] = 6700 N/cm^3	4400 N/cm^3
C' [0,5 zul P] = 7200 N/cm^3	4800 N/cm^3
C' [1,5 zul P] = 6700 N/cm^3	4000 N/cm^3





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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TIMBER AND WOOD-BASED PRODUCTS STRUCTURES. PANELS FOR ROOF COVERINGS. METHODS OF TESTING AND STRENGTH ASSESSMENT CRITERIA. POLISH STANDARD BN-78/7159-03

OTANIEMI, FINLAND
JUNE 1980

BUILDING	INDUSTRY-WIDE STANDARD	BN-78
JOINERY	Timber and wood-based products structures. Panels for roof coverings. Methods of testing	7159-03
PRODUCTS	and strength assessment criteria.	

I N D E X

1. INTRODUCTION

2. SPECIFICATIONS FOR TESTING

This standard has been developed by Building Joinery Research and Development Centre, having been approved by the Director of Building Joinery Union on 8-ght December 1978 as a mandatory one since 1-st July 1979r.

3. TYPES AND METHODS OF TESTING AND ASSESSMENT CRITERIA

3.1. Rigidity and elasticity tests

3.1.1. Static arrangement

3.1.2. Types and points of load application

3.1.3. Measured characteristics

3.1.4. Points of measurement of characteristics

3.1.5. Test principle

3.1.6. Test procedure

3.1.7. Determination of water content in components of a member

3.1.8. Test results.

3.1.9. Test results assessment

3.1.10. Criteria for assessment of rigidity and elasticity

3.2. Determination of flexural strength

3.2.1. Static arrangement

3.2.2. Measured characteristics

3.2.3. Test principle

3.2.4. Test procedure

3.2.5. Test results

3.2.6. Test results assessment

3.2.7. Criteria for assessment of flexural strength

3.3. Determination of impact strength

3.3.1. Static arrangement

3.3.2. Points of load application

3.3.3. Test principle

3.3.4. Test procedure

3.3.5. Test results

3.3.6. Test results assessment

3.3.7. Criteria for assessment of impact strength

3.4. Determination of long-term load resistance

3.4.1. Static arrangement

3.4.2. Type and value of load

3.4.3. Time of load action

3.4.4. Measured characteristics

3.4.5. Principle of deflection measurement

3.4.6. Test principle

3.4.7. Test procedure

3.4.8. Principle of determination of temperature and relative humidity of air

1. INTRODUCTION

1.1. Scope. This standard concerns strength tests and criteria for assessment of wood and wood-based panels for roof coverings.

1.2. Field of application. This standard shall be applied for:

a assessment of new panels/prototype ones/, determination of their detailed characteristics and performance,

b quality inspection of serialized production,

c sampling acceptance inspection,

d quality inspection of elements rejected during acceptance

1.3. Definitions.

1.3.1. Measured characteristics - characteristics to be measured eg deflection, action, water content.

1.3.2. Zero reading - initial reading at the start of the test.

1.3.3. Bilogarithmic net - double logarithmic net having abscissa and ordinate in logarithmic scale.

1.3.4. Rigidity of a member - its deformation resistance under actual short-term loads.

1.3.5. Elasticity of a member - its permanent deformation resistance under varying in-service load.

1.3.6. Characteristic load - acc. PN-74/B-02009.

1.3.7. Design load - acc. PN-74/B-02009.

1.3.8. Permanent load - acc. PN-74/B-02009.

2. SPECIFICATIONS FOR TESTING

2.1. Technical specifications.

2.1.1. Specifications for a panel to be tested. A panel to be tested shall comply with standard specification or technical documentation as far as variability of its dimensions, form as well as types of characteristics of its components are concerned.

2.1.2. Testing equipment. Testing machines, gauges, structure of supports and auxiliaries shall comply with PN-73/B-06281. Another testing equipment shall be allowed also but always having the same accuracy of measurement of loads and readings.

2.2. Climatic conditions. The test shall be performed in closed rooms, isolated from direct effect of weather. The temperature of 17-23°C and relative humidity of air of 50-60% are recommended. When particular specifications are required other climatic conditions shall be allowed.

2.3. Preparation for testing. The preparation of a member for testing is as follows:

a inspection of its accordance with the specifications as in 2.1.1.

b description involving type, structure, composition, deviation from specifications as in 2.1.1., quality of finishing and other data, which can influence the test results, eg if treated or not, type of preservative,

c determination of dimensions and mass of a member,

d determination of a form, number and situation of joints,

e determination of type and number of connectors like nails, screws, glue and assembly of joint eg by pneumatic or manual nailing,

f marking of a member not to be mistaken with another one,

g determination and marking of points for measurement and application of loads,

h determination of water content in components of a member,

i seasoning in accordance with 2.2. until constant water content shall be reached, but at least for 6 days.

2.4. Static arrangement.

2.4.1. Support diagram. The static arrangement of a panel shall correspond to its technical for-design or to its real in-service conditions.

A substitute of static arrangement/simplified one/is allowed if there is a possibility of transforming the results to a panel required.

2.4.2. Loading diagram. The type and points of load application shall correspond to its technical for-design, to respective standard or in-service conditions. An alternative loading diagram shall be allowed if there is a possibility of transforming the results to a diagram required.

2.5. Minimum number of panels for testing. For all the tests as in 1.2. a and d n_{min} = 3 panels, for tests as in 1.2.b a number of panels shall correspond to that customary used for quality inspection and for tests as in 1.2.c a number of panels shall correspond to that used for determined method of acceptance.

2.6. Determination of water content in components shall be done using a drier and a balance or hygrometers having an accuracy of 2%.

3. TYPES AND METHODS OF TESTING AND ASSESSMENT CRITERIA

3.1 Rigidity and elasticity tests

3.1.1. Static arrangement for tests acc. 2.4.

3.1.2. Type and points of load application acc. 2.4.2.

3.1.3. Measured characteristics. During investigations the following characteristics shall be determined.

a actual deflection under characteristic load,

b actual deflection under design load,

c deflection after 24 hours under design load,

d deflection after unloading,

3.1.4. Points of measurements shall be determined by the scientist before an investigation is started. Those are points corresponding to maximum value of measured characteristics, specified in technical for-design or by established static calculations analysis. There is no possibility of determining them in theoretic way but only on the basis of preliminary tests.

3.1.5. Test principle. The test consist in applying loads and measuring deflections so produced.

3.1.6. Test procedure. A member shall be placed on supports as in static arrangement in 2.4.1.

Gauges for measurement of characteristics shall be installed as in 3.1.3. in the points specified in 3.1.4., then zero readings are taken. Afterwards the members shall be loaded with characteristic load in accordance with its for-design or corresponding standard, and again readings are taken. The design load shall be applied taking respective readings. Under this load a member is left for 24 hours and readings are taken. Later the member is loaded until permanent load is reached and again readings are taken. The successive readings are taken after 2 hours from unloading. If gauges used are able to record data continuously, the assessment of recording is done after the test is over.

3.1.7. Determination of water content in components shall be done as 2.6. immediately after finishing the tests.

3.1.8. Test results. Taking an advantage of gauges indications the following shall be derived:

a total deflection of a member U_a - mm, under characteristic load/considering also its dead load/ from the formula:

$$U_a = U_{bl} + U_l \qquad /1/$$

where:

U_{bl} - deflection of a member under characteristic load without considering its dead load, mm,

U_l - deflection of a member under its dead load, mm,

$$U_l = \frac{q_l}{q_c} \cdot U_{bl}$$

where:

q_l - dead load of a member, Pa,

q_c - characteristic load, Pa.

b total deflection of a member, U_b, mm, under design load is calculated from the formula:

$$U_b = U_{b2} + U_1 \qquad /2/$$

where:

U_{b2} - deflection indicated by gauges under design load without considering the dead load of a member, mm,

U_1 - from formula /1/.

c permanent deflection of a member U_c, mm, is calculated from the formula:

$$U_c = W_k - W_z \qquad /3/$$

where:

W_k - indications of gauges after 2 hours from unloading, mm,

W_z - indications of gauges before starting the test, so called, zero readings, mm.

3.1.9. Test results assessment. To assess the rigidity and elasticity of a member is necessary:

a calculate allowable deflection coefficient, r_d, from the formula:

$$r_d = \frac{U_a}{U_{dop}} \qquad /4/$$

where:

U_a - is from the formula /1/,

U_{dop} - allowable deflection in accordance with PN-73/B-03150.

b calculate permanent deflection coefficient, r_t, from the formula:

$$r_t = \frac{U_c}{U_a} \qquad /5/$$

where:

U_c - from formula /3/,

U_a - from formula /1/.

c derive load-deflection curve plotting U_a /formula 1/ and /formula 2/.

3.1.10. Criteria for assessment of rigidity and elasticity of a member

a $r_d \leq 1{,}0$

b $r_t \leq 0{,}20$

3.2. Determination of flexural strength

3.2.1. Static arrangement - acc. 2.4.

3.2.2. Measured characteristics. In order to determine flexural strength the following characteristics shall be measured:

a failure load value,

b maximum deflection corresponding to failure load.

3.2.3. Test principle. This consists of imposing on a member a gradually increasing load until failure shall occur and measuring deflection under this load.

3.2.4. Test procedure. The member prepared for the test as in 2.3. shall be placed on supports as illustrated on static arrangement 2.4.1. After the gauges are fixed as in 3.1.4. zero readings are taken as 3.2.2. and then the member is gradually loaded with increasing load until failure. For loading ballast weights are used, up to 25% of characteristic load, determined by technical for-design or by respective standard. Every next load shall be applied after 15 minutes from the anterior one. After failure is necessary determine: failure load value, water content in components of a member as in 2.6. and estimate deflection under failure load.

3.2.5. Test results. On the basis of measurements the following shall be derived:

a estimate maximum deflection under failure load, U_n, mm, from the formula:

$$U_n = \frac{q_n}{qb} \cdot U_b \qquad /6/$$

where:

q_n - failure load including dead load, Pa,

q_b - test load /without dead load/ corresponding to maximum deflection, Pa,

U_b - deflection indicated by gauges in the test loads range, mm.

b failure load - shall be determined with an accuracy of 3% according with PN-73/B-06281.

3.2.6. Test results assessment. In order to determine flexural strength of a member, safety coefficient,s, shall be calculated from the formula:

$$s = \frac{q_n}{q_c} \qquad /7/$$

where:

q_c - characteristic load, determined in for-design or standards,

q_n - from formula /6/.

3.2.7. Criteria for assessment of flexural strength.

$s \geqslant s_{dop}$; s - from formula /7/;

s_{dop} - allowable safety factor calculate from the following formula:

$$s_{dop} = \frac{n_o}{p \cdot m} \qquad /8/$$

where:

$$n_o = \frac{\sum_i n_i \cdot q_{ci}}{\sum_i q_{ci}}$$

ni - load factor acc. PN-74/B-02009 and PN-74/B-02010.

q_{ci} - characteristic loads acc. PN-74/B-02009.

m - correction factor acc. PN-73/B-03150; before corresponding values will be introduced into standard, this factor for wood-based materials shall be taken as for solid wood but not exceeding 1,0 p - limiting stress coefficient for a member made from solid wood

p = 0,6, from plywood p = 0,5, from fibreboard p = 0,4, from other wood based materials p = 0,3, from composite members

p shall be taken as for extreme fiber in it.

3.3. Determination of impact strength

3.3.1. Static arrangement - acc. 2.4.1.

3.3.2. Points of load application. The points of application of dynamic and static loads shall be determined individually for each case in relation with the structure of a member to be tested. The point of application shall be most weak ones considering local or total strength of a member, eg in the middle span between ribs. The area of static load application shall be of 0,5 m^2.

3.3.3. Test principle. The test consist in application of dynamic load, determination of failure and resistance against concentrated loads.

3.3.4. Test procedure. The test is performed in two stages. The member prepared for testing as in 2.3. shall be placed on supports as on static arrangement 2.4.1. In the first stage it shall be submitted to 3 impacts of a building paper roll having a mass of 45 kg, falling down from 1,5 m, measured in its centre of gravity, in a point determined in 3.3.2., what shall be done as follows: during the first impact the axis of a roll shall be perpendicular to a tested member, during the second on the axis shall be parallel to its shorter side and during the third one parallel to its longer side. Then failure is stated. In the second stage the member shall be loaded with 2 kN static load imposed to its surface as in 3.3.2. and the failure or behaviour under load stated.
After the test is finished water content shall be determined in component of the member as in 2.6.

3.3.5. Test results. Failure of a member or its absence shall be stated.

3.3.6. Test results assessment. The assessment consists in describing the failure of a member.

3.3.7. Criteria for assessment of impact strength. During the two stages of the test, the member shall not be totally destroyed.

3.4. Determination of long-term loads resistance

3.4.1. Static arrangement - acc. 2.4.

3.4.2. Type and value of load shall correspond to design load determined in for-design or respective standards.

3.4.3. Time of load action is determined individually in relation with test results and the test is conducted until there will be a possibility of establishing an estimate relation between deflection velocity variations and loading time. Minimum time of load action, if a member have not failed before, is 100 days.

3.4.4. Measured characteristics. Testing the member under long-term load the following characteristics shall be determined:

a deflection in time,

b failure load, if a member has failed,

c time to failure, if a member has failed,

d temperature and relative humidity of air in testing room.

3.4.5. Principle of deflection measurement. Deflection shall be measured continuously /continuously recorded/. Periodical readings are also allowed following the suggestions:

- the first reading shall be taken before loading the member,
- the second reading shall be taken immediately after loading,
- the following readings shall be taken at least:
  - every 24 hours for 10 days,
  - every 48 hours for 20 days,
  - every 5 days for 30 days,
- and then readings shall be taken at least every 10 days.

3.4.6. Test principle. The test consists in loading the member with a load having determined value and establishing the measured characteristics.

3.4.7. Test procedure. A member prepared for test as described in 2.3. is placed on supports according to static diagram as in 2.4.1. Then gauges are installed for measuring the characteristics as in 3.4.4. Afterwards zero readings shall be taken in the points specified in 3.1.4. The member is loaded with ballast weights up to value determined in 3.4.2. and respective readings are taken. The member is left under load for a time determined in 3.4.3. recording indications of gauges, or readings are taken during loading as in 3.4.5. After the time required the member is unloaded and corresponding readings are taken. The readings are taken then after 24 hours from unloading. The test being finished, water content in components of a member is determined as in 2.6.

3.4.8. Principle of determination of temperature and relative humidity of air. Temperature and relative humidity of air shall be recorded continuously.

3.4.9. Test results. After the test is over the following deflections of a member shall be recorded in mm:

a after loading,

b during loading at intervals as in 3.4.5.,

c after unloading,

d after 24 hours since unloading.

If failure occurs its character shall be described and points of failure defined.

3.4. Test results assessment. In order to determine long-term resistance of a member is necessary:

a calculate deflection velocity in time, V_t, mm/24 h, from the formula:

$$V_t = \frac{U_t - U_o}{t} \qquad /9/$$

where:

U_t - deflection after a time, t, mm

U_o - initial deflection of a member after loading, mm,

t - loading time, days, being equal at least to a time as in 3.4.3., b plot a relation time, t - deflection velocity, V_t, in bilogarithmic scale, abscissae axis having time in days scale and ordinate axis having deflection velocity scale in mm/24 h,

c approximate the graphic so obtained acc. b/ to get a direct line of time - deflection velocity relation,

d determine an estimate mathematical data m from the formula for direct line as in c/:

$$\lg V_t = m \lg t + \lg z \qquad /10/$$

where:

m - directivity factor of a direct line/tangent of an angle of its inclination in relation with abscissae axis,

z - theoretic deflection velocity, V_t, after 1 day from loading mm/24 h.

e determine in-service deflection, U_e, mm, and servicing time of a member, t_e, until failure, in days, from the formulae:

$$U_o = z \cdot t_e^{1+m} + U_o \qquad /11/$$

$$t_e = \sqrt[1+m]{\frac{U_n - U_o}{z}} \qquad /12/$$

where:

t_e - planned or designed time of service of a member until failure, days,

U_o - acc. formula /9/,

U_n - acc. formula /6/,

m and z - acc. formula /10/.

f derive permanent deflection factor r_{td} from the formula

$$r_{td} = \frac{U_{trw}}{U_o} \qquad /13/$$

where:

U_{trw} - deflection after 24 hours from unloading

U_o - from formula /9/.

3.4.11. Criteria for assessment of long-term load resistance.

a $U_e \leqslant \frac{U_n}{3}$

b $U_e \leqslant u_{edop}$ or $t_e \geqslant m\ 3t_p$

c $r_{td} \leqslant 0{,}5$

where:

U_{edop} - allowable deflection in final stage of service of a member, determined by for-design or standards, mm,

t_p - servicing time of a member, if determined by technical documentation.

Other symbols from formulae: /6/, /11/, /12/ and /13/.

3.5. Determination of characteristic load resistance of a member having wetted surface

3.5.1. Static arrangement - acc. 2.4.1.

3.5.2. Type and value of load and mode of its application in accordance with the for-design or respective standard for a characteristic load.

3.5.3. Measured characteristics. Is necessary to describe a mode of failure and supported load.

3.5.4. Test principle. The test consist in wetting the whole surface of a member, imposing the characteristic load and stating of eventual failure.

3.5.5. Test procedure. The member prepared for test as in 2.3. is placed on supports as in 2.4.1. To the edges of a member a strap of building paper is attached, 5 cm deep, protecting it against water leaking. The recipient so formed is filled up with water up to 2 cm and left for 24 hours. Water shall be added to maintain the member surface always under water. After 24 hours of wetting, water shall be removed and a member loaded with ballast until imposing characteristic load as in 3.5.2. After 0,5 hour the state of a member shall be defined. If a member is destroyed earlier, failure load shall be determined.

3.5.6. Test results. Failure or its absence shall be stated.

3.5.7. Tests results assessment. Grade of failure shall be described.

3.5.8. Criteria for assessment of characteristic load resistance of a member having wetted surface. The member shall not fail under a load smaller than characteristic one, determined by for-design or respective standard.

4. SELECTION OF TYPES OF TESTS AND TEST REPORT

4.1. Selection of type of test. Type and method of testing shall be determined for each case as in 1.2. For assessment of members as in 1.2.a all the tests included in this standard are recommended, and for quality assessment as in 1.2.b-d tests as in 3.1. and 3.3. are recommended.

4.2. Test report. Every test report shall include:

a description of a member as in 2.3.a-h,

b description of static arrangement as in 2.4.,

c data concerning accuracy of gauges used,

d description of test performance /with photographs if possible/,

e specification of test results.

Additional information

ADDITIONAL INFORMATION

1. This standard has been developed by: Building Joinery Research and Development Centre - Wolomin

2. Reference standards

   PN-74/B-02009 - Loads in static calculations. Permanent and mobile loads.

   PN-70/B-02010 - Loads in static calculations. Snow loads.

   PN-77/B-02011 - Loads in static calculations. Wind loads.

   PN-76/B-03001 - Building structures and soils. Design principles.

   PN-73/B-03150 - Timber structures. Static calculations and design.

   PN-73/B-06281 - Building prefabricated products from concrete. Methods of strength tests.

3. Foreign standards

   ASTM-Et2-E74a - Standard methods of conducting strength tests on panels for building constructions.

4. Authors of this standard: Ass. Prof. Z. Zdziarnowski
   Dr Eng. J. Oliferowics
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CIB-W18/13-17-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ON TESTING WHETHER A PRESCRIBED EXCLUSION LIMIT IS ATTAINED

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JUNE 1980

On Testing Whether a Prescribed Exclusion Limit is Attained

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INTRODUCTION

The problem addressed is that of testing whether a prescribed exclusion limit (fifth percentile) is attained in a well-defined population of plywood panels. This population can be envisaged as, and may in actuality be, a shipment. It is assumed that sampling of the shipment is effectively at random, and that the sample size is a negligible fraction of the population (shipment).

Several strategies will be considered:

(1) Fixed sample size, normality assumed

(2) Sequential sampling, normality assumed

(3) Fixed sample size, distribution free

(4) Sequential sampling, distribution free.

For (1) and (2) a study of the robustness against departures from normality will also be undertaken.

Sequential sampling with normality assumed was considered by the writer in a Supplemental Report (May, 1977). In this report the development of the operating-characteristic curves is not correct; an approximation of the unconditional distribution of the statistic $\bar{x}$ - ks (see below) was used instead of the conditional distribution given that a decision had not been reached at a previous stage. The implementation of the scheme is not affected but the operating characteristics are in error, although it is difficult to ascertain the magnitude of the error. Indeed, the curves obtained may well be reasonable approximations.

A sequential strategy, based on the normality assumption, thus needs redevelopment. Mathematically this is not an easy task. Since operational sample sizes are urgently required, a fixed-sample size strategy will be considered, firstly, with sequential methods to be discussed in a subsequent note.

THEORY

Let us assume that the quantity of interest is normally distributed, and we can obtain a random sample of size n. Then, a lower 95%

tolerance limit with specified confidence is given by

$$\bar{x} - ks$$

$$(\bar{x} = \sum^{n} x_i/n, \quad s^2 = \sum^{n} (x_i - \bar{x})^2/(n - 1))$$

where $k = k(n,\gamma)$ is chosen so that

$$P(\bar{x} - ks \leqslant x_\alpha) = \gamma$$

where x_α is the 5th percentile of the population. The statement, $P(\bar{x} - ks \leqslant x_\alpha) = \gamma$ is equivalent to

$$P(F(\bar{x} - ks) \leqslant F(x_\alpha)) = \gamma$$

i.e. $$P(F(\bar{x} - ks) \leqslant 0.05) = \gamma$$

where $F(\cdot)$ is the cumulative distribution function of the variable of interest. In words, the probability that at most 5% of the population falls below $\bar{x} - ks$ is γ (specified). Tables of k are available for selected values of γ and sample size, n.

Suppose we take $\gamma (0 < \gamma < 1)$ large, i.e. there is a high probability that $\bar{x} - ks$ will not exceed the true fifth percentile. Then, if x_c is some specified critical value, and if $\bar{x} - ks > x_c$ we can be reasonably confident that the true fifth percentile also exceeds x_c, i.e. that the shipment complies with the requirement. (x_c falls below $\bar{x} - ks$ which, with sufficiently high probability, falls below x_α).

Conversely, let us take γ small; the probability that $\bar{x} - ks$ exceeds the fifth percentile is now large, i.e.

$$P(\bar{x} - ks > x_\alpha) = 1 - P(\bar{x} - ks \leqslant x_\alpha) = 1 - \gamma .$$

Suppose that $\bar{x} - ks < x_c$, i.e. $\bar{x} - ks$ falls below the critical value, we could then be reasonably confident that the shipment does <u>not</u> comply with the requirement (x_c lies above $\bar{x} - ks$ which, with sufficiently high probability, is greater than x_α).

Let us write these values of k as k_1 and k_2, respectively ($k_1 \geqslant k_2$) and the associated values of γ as γ_1 and γ_2, respectively.

Note that if $\bar{x} - k_1 s < x_c < \bar{x} - k_1 s$ we are in a state of indecision, i.e. we are not sufficiently confident that the shipment complies or does not comply with the requirement.

It remains to quantify what is meant by "reasonably confident" or "sufficiently confident".

If the sample value $\bar{x} - k_1 s$ actually falls above x_α (unknown) we run the risk of incorrectly judging that a shipment complies, conversely, if $\bar{x} - k_2 s$ falls below x_α we run the risk of incorrectly judging that a shipment fails to comply.

Such incorrect judgements have different consequences. For example, if a shipment that does not meet the requirement is incorrectly passed, there is a higher proportion of weaker material than allowed and hence an increased chance of structural failure. The cost of such failure will, of course, depend on the end use. On the other hand, the incorrect rejection of a shipment that meets the requirement while not affecting the purchaser, would result in economic loss to the supplier; indeed the rejection of one shipment may jeopardize a supplier's chances of further supplying a particular market.

No such risks surround the acceptance of a complying sample, but if this is the result of overcompliance there is inefficient use of the material, an aspect which also demands consideration in a time of dwindling resources.

The quantities γ_1, γ_2 have to be chosen, in conjunction with the sample size, to reduce these risks to acceptable levels. This, in turn depends on the value ascribed to the potential consequences, and to the costs of sampling and testing. Note that, because of the asymmetry of the consequences, one would not necessarily chose $\gamma_1 = 1 - \gamma_2$.

The operating characteristics, here expressed as the relationships between the probability of acceptance/rejection and the actual shipment 5th percentile, provide a rational basis for choosing a strategy. Operating characteristic curves have therefore been developed and are presented for selected values of γ_1, γ_2 and for several sample sizes. The critical value of x_c has been taken as 4305 (= 2.1 x 2050, 2050 being the current allowable value for 5 + plies, face grain parallel to span) and the coefficient of variation as 20% (based on available data of equivalent material).

The operating characteristics have been obtained as follows:

$$P(\bar{x} - ks < x_c) = P\left(\frac{\bar{x} - x_c}{s} < k\right)$$

and under the normality assumption $t' = \sqrt{n}\,(\bar{x} - x_c)/s$ is distributed as a non-central t with non-centrality parameter $\sqrt{n}(\mu - x_c)/\sigma$ and $(n - 1)$ degrees of freedom.

Accordingly $P(\frac{\bar{x} - x_c}{s} < k) = P(t' < \sqrt{n}\,k)$. The latter has been plotted against x_α, the values being obtained from an in-house computer program for the distribution function of the non-central t. Figs. 1.1-5.9 give the operating characteristics for all combinations of

$\gamma = 0.01, 0.05, 0.10, 0.25, 0.50, 0.75, 0.90, 0.95, 0.99$

and $n = 10, 20, 40, 80$ and 160.

INTERPRETATION

Let us start with Fig. 1.1 (n = 10). γ is small so that this figure represents

$$P(\bar{x} - k_2 s < x_c).$$

If $\bar{x} - k_2 s < x_c$ we would wish to reject, and the probability of rejection is given as the right-hand ordinate. Then, if $x_\alpha = x_c = 4305$

$$P(\bar{x} - k_2 s \leq x_c) = P(\bar{x} - k_2 s \leq x_\alpha) = 0.01 \ (= \gamma_2)$$

Clearly, for $x_\alpha < x_c$, $P(\bar{x} - k_2 s \leq x_c) > 0.01$ and, indeed, $P(\bar{x} - k_2 s < x_c)$ increases as x_α decreases.

The complement of this is Fig. 1.9. γ is large (0.99) so that this figure then represents

$$P(\bar{x} - k_1 s \leq x_c).$$

If $\bar{x} - k_1 s > x_c$ we would wish to accept, and the probability of acceptance is given as the left-hand ordinate. If $x_\alpha = x_c = 4305$

$$P(\bar{x} - k_1 s > x_c) = P(\bar{x} - k_1 s > x_\alpha)$$

$$= 1 - P(\bar{x} - k_1 s \leq x_\alpha) = 1 - 0.99 = 0.01.$$

Clearly, for $x_\alpha > x_c$, $P(\bar{x} - k_1 s > x_c) > 0.01$ and $P(\bar{x} - k_1 s > x_c)$ increases as x_α increases.

We take γ_1 to be large so that there is a low probability of accepting a shipment if $x_\alpha < x_c$. The probability of accepting if $x_\alpha < x_c$ is clearly less than 1% (Fig. 1.9). Likewise, we take γ_2 small for there to be a low probability of rejecting the shipment if $x_\alpha > x_c$. Clearly this is < 1% (Fig. 1.1).

A consequence of these no doubt desirably low probabilities is that the probability of accepting a shipment is also low when $x_\alpha > x_c$ unless x_α is very much greater than x_c, and the probability of rejecting is also low unless x_α is very much less than x_c. Indeed, with sample size 10, for a wide range of x_α, the outcome would most likely be indecisive.

For example, a shipment with $x_\alpha = 3430$ would have a 50% chance of rejection (but no chance of acceptance) while a shipment would have to have $x_\alpha > 10330$ to have better than a 50% chance of acceptance (with no chance of rejection).

We will use this interval (3430, 10330) as a measure of the width of a "zone of indecision".

It would be surprising if the strategy represented by Figs 1.1, 1.9 were regarded as viable. The zone of indecision can be reduced by (1) increasing the sample size and/or (2) allowing greater risks of wrong decisions, i.e. decreasing (increasing) γ_1 (γ_2). Consider therefore Figs. 1.2, 1.8 ($n = 10$).

The probability of rejection with $x_\alpha > x_c$ has increased (as has the probability of acceptance with $x_\alpha < x_c$), although depending on the costs, these risks may not be damagingly large. The probability of rejection with $x_\alpha > x_c$ (= 4305) is less than 5% and is less than 1% for $x_\alpha > 4700$. The probability of acceptance with $x_\alpha < 4305$ is also less than 5% and is less than 1% for $x_\alpha < 3800$. Our measure of the width of the zone of indecision is reduced to (3600, 6550). This still seems uncomfortably wide unless substantial overcompliance were the accepted practice.

The operating characteristics for $\gamma_2 = 0.10$, $\gamma_1 = 0.90$ are given in Figs. 1.3 and 1.7, respectively ($n = 10$). The probabilities of incorrect acceptance/rejection are farther increased; for there to be less than a 1% chance of incorrectly rejecting a shipment x_α must now exceed 4900 and for there to be less than a 1% chance of incorrectly accepting a shipment x_α must now fall below 3610. On the other hand, the zone of indecision is now measured as (3720, 5735). There is, however, a new complication, at $x_\alpha = 3720$ there is a 50% chance of rejection, but unlike the situation where $\gamma_2 = 1 - \gamma_1 < 0.05$, where the probability of accepting at this boundary value was close to zero, the probability of acceptance is here approaching 2%.

The operating characteristics for $\gamma_2 = 0.25$, $\gamma_1 = 0.75$ are given in Figs. 1.4, 1.6, respectively (n = 10). The trends noted above continue.

The operating characteristic for $\gamma_2 = \gamma_1 = 0.50$ is given in Fig. 1.5. Here there is, of course, no zone of indecision, however the probability of incorrect decision may be intolerably large.

The zone of indecision can also be eliminated by taking $\gamma_1 = \gamma_2$ ($\neq 0.5$) but here the probability of incorrect decision will exceed 50% for some values of x_α. This also seems intolerable unless we are indifferent to the outcome for x_α just below or just above x_c.

There is, of course, no requirement that $\gamma_2 = 1 - \gamma_1$, so that any one of Figs. 1.1 through 1.5 can be used in combination with any one of Figs. 1.5 through 1.9. One is also free to choose values for γ other than those illustrated here.

If an acceptable strategy cannot be found with n = 10 there is no alternative but to increase the sample size. Operating characteristics are given for n = 20, 40, 80 and 160 in Figs. 2.1 through 5.9. These may be interpreted in the same manner as Figs. 1.1 through 1.9.

In Fig. 6 we plot the values of x_α above which the probability of incorrectly rejecting the shipment is less than 1% against sample size for specified values of γ_2. Also, in Fig. 6 we plot the values below which the probability of incorrectly accepting the shipment is less than 1% for specified values of γ_1. This is repeated in Figs. 7 and 8 but for probability less than 5% and 10%.

To illustrate a use of these figures, suppose that, because of the incorporation of a safety factor in the allowable load, we feel that no harm would be done if x_α fell below x_c by no more than 10%. Then we might specify very low probability of acceptance (less than 1%, say) for x_α below 0.9 x 4305 = 3875. From Fig. 6 we see that this can be attained with the following combinations of γ_1 and n.

γ_1	n
0.95	$2^{0.45} \times 10 = 13.7 \simeq 14$
0.90	$2^{1.37} \times 10 = 25.8 \simeq 26$
0.75	$2^{2.40} \times 10 = 52.8 \simeq 53$
0.50	$2^{3.35} \times 10 = 102.0 \simeq 102$

Conversely, we may feel that by far the greatest part of production will have x_a well in excess of x_c, so that the actual rate of erroneous rejection of complying shipments would be low, and hence we may tolerate a relatively high probability of erroneous rejection for shipments for which x_a just exceeds x_c. Suppose, for example, we allow a 10% erroneous rejection rate when x_a exceeds x_c by 5%, i.e. for $x_a = 1.05 \times 4305 = 4520$. Fig. 8 shows that this can be achieved with:

γ_2	n
0.25	$2^{1.65} \times 10 = 31.4 \simeq 32$
0.50	$2^{3.80} \times 10 = 139.3 \simeq 140$

These are boundary points; clearly we will reduce the erroneous acceptance/rejection rate by fixing n and increasing (decreasing) γ_1 (γ_2) or by fixing γ_1 (γ_2) and increasing n.

Note, however, that the probability of indecision will also be affected. Recall that for fixed n the probability of indecision decreases with decreasing γ_1 and increasing γ_2, so that we cannot have the best of all worlds. To give some feeling for how the probability of indecision changes with sample size, the width of the zones of indecision are illustrated in Fig. 9.

Figs. 6 through 9 show that at some point the improvement brought about by increasing the sample size is not commensurate with the cost of sampling and testing.

The above operating characteristics are based on a coefficient of variation of 20%. The coefficient of variation is unknown - hence one should examine the effect of the coefficient of variation on the operating characteristics. In Fig. 10.1-10.3 we gave the operating characteristic for coefficients of variation of 15%, 20% and 25%, for $\gamma = 0.05$, 0.50 and 0.95, n = 40.

From these figures it is clear that incorrect specification of the coefficient of variation can have an appreciable effect on the operating characteristic. Indeed, reduction of the coefficient of variation from 20% to 15% is virtually equivalent to doubling the sample size from 40 to 80, while an increase in the coefficient of variation to 25% has an effect very similar to halving the sampling size to 20.

In conclusion, it is emphasized that it is not the intent of this note to recommend any particular strategy, but rather to put forward information which coupled with knowledge of costs, etc. will enable a suitable strategy to be selected.

May 1980

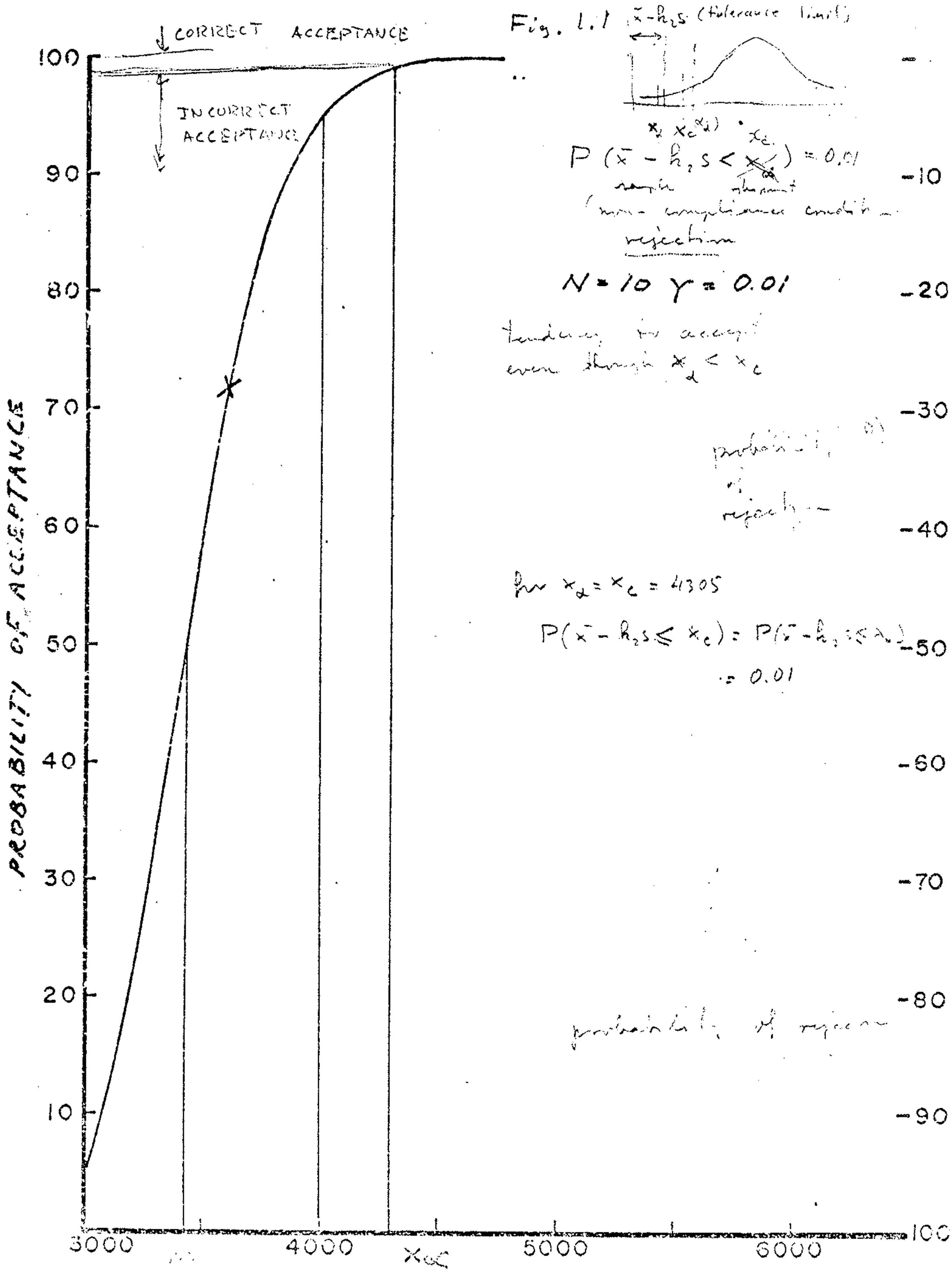
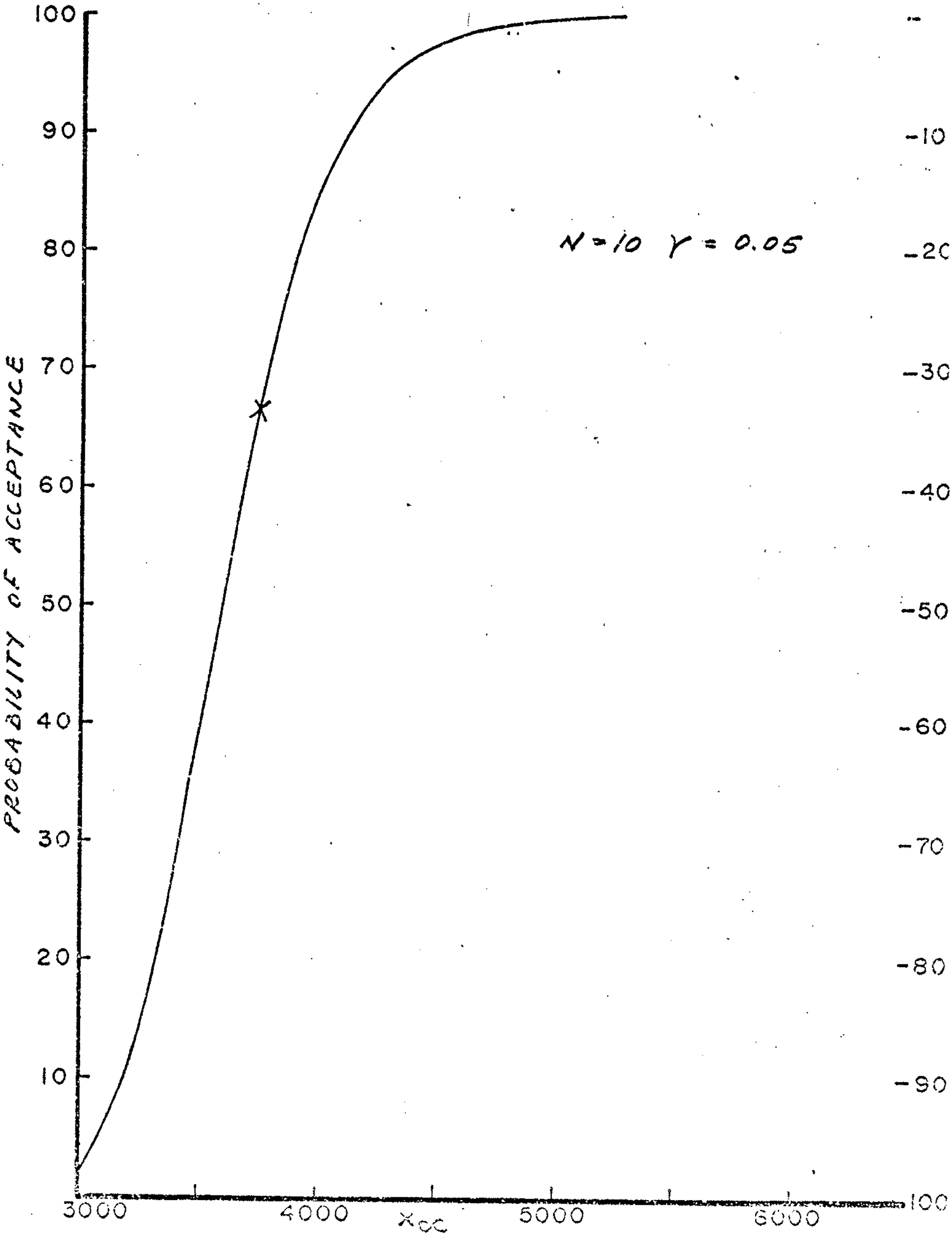


Fig. 1.1
CORRECT ACCEPTANCE
INCORRECT ACCEPTANCE
$P(\bar{x} - k_2 s < x_c) = 0.01$
rejection
$N = 10 \quad \gamma = 0.01$
tendency to accept even though $x_\alpha < x_c$
probability of rejection
for $x_\alpha = x_c = 4305$
$P(\bar{x} - k_2 s \leq x_c) = P(\bar{x} - k_2 s \leq x_\alpha) = 0.01$
probability of rejection
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
3000
4000
5000
6000
x_α
10
20
30
40
50
60
70
80
90
100

Fig. 1.2



100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
PROBABILITY OF ACCEPTANCE
N = 10 γ = 0.05
3000
4000
5000
6000
X_{oc}

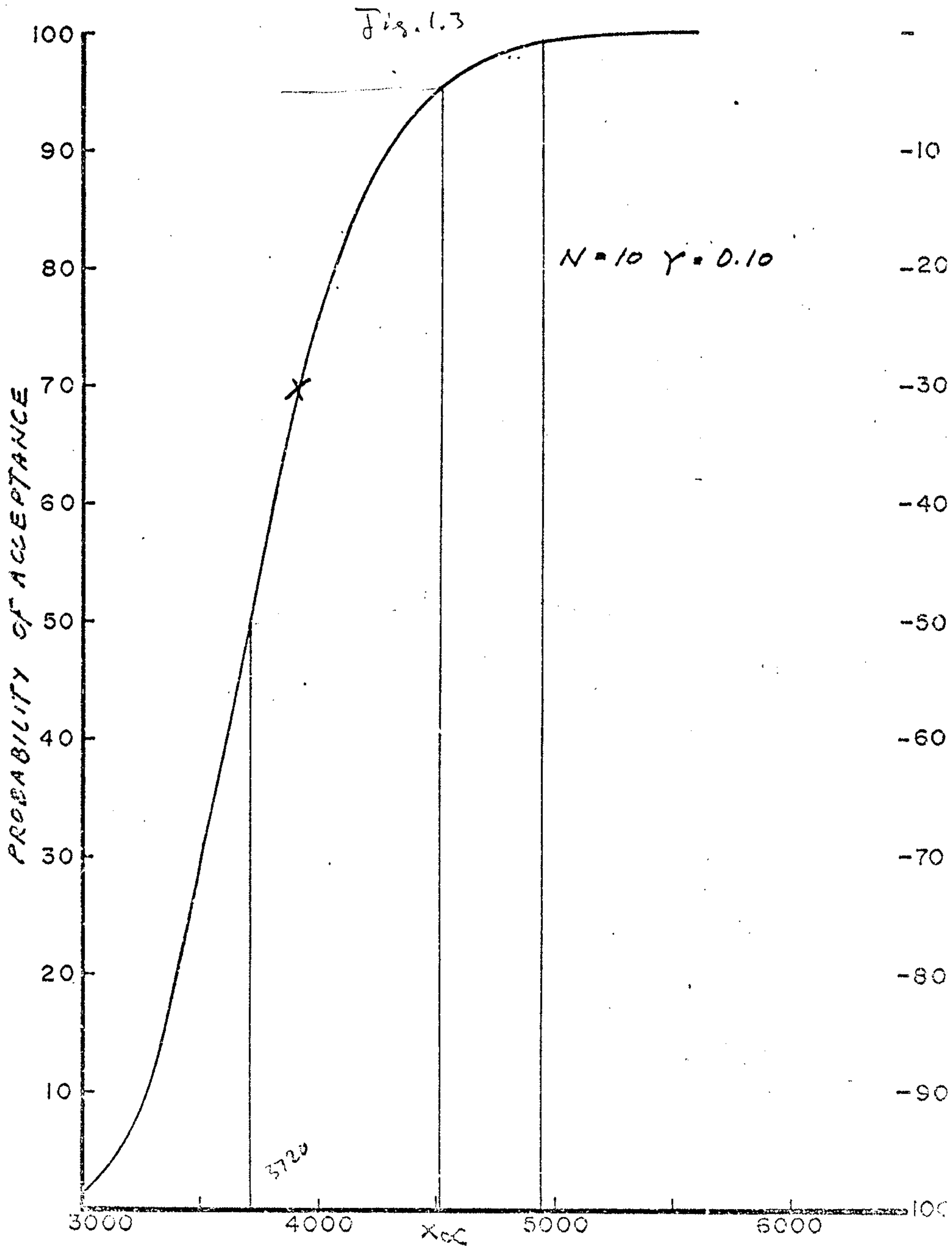
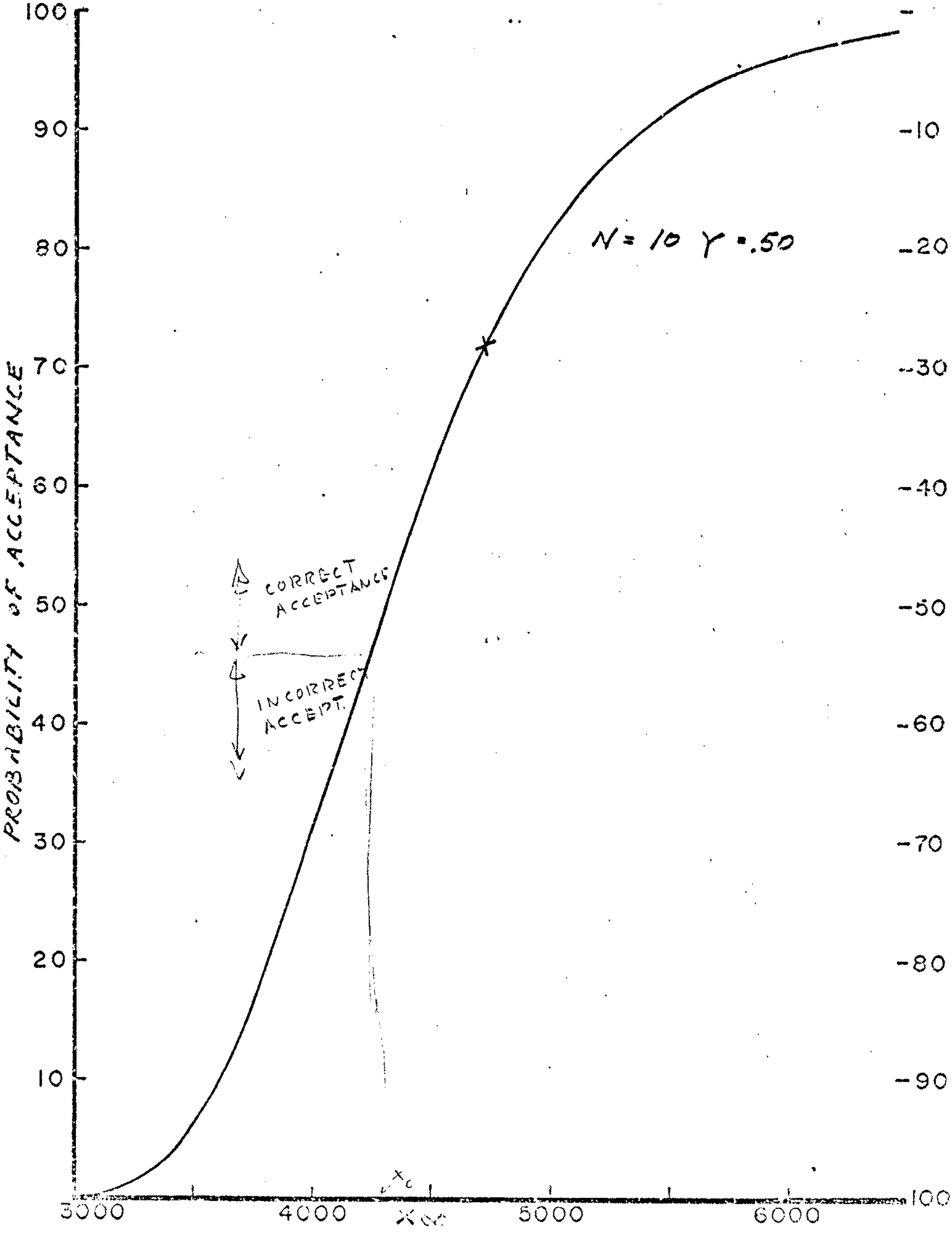


Fig. 1.3
N = 10 γ = 0.10
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3720
3000
4000
5000
6000
XoC

Fig. 1.5


PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
N = 10 Y = .50
CORRECT ACCEPTANCE
INCORRECT ACCEPT.
x_c
3000
4000
5000
6000

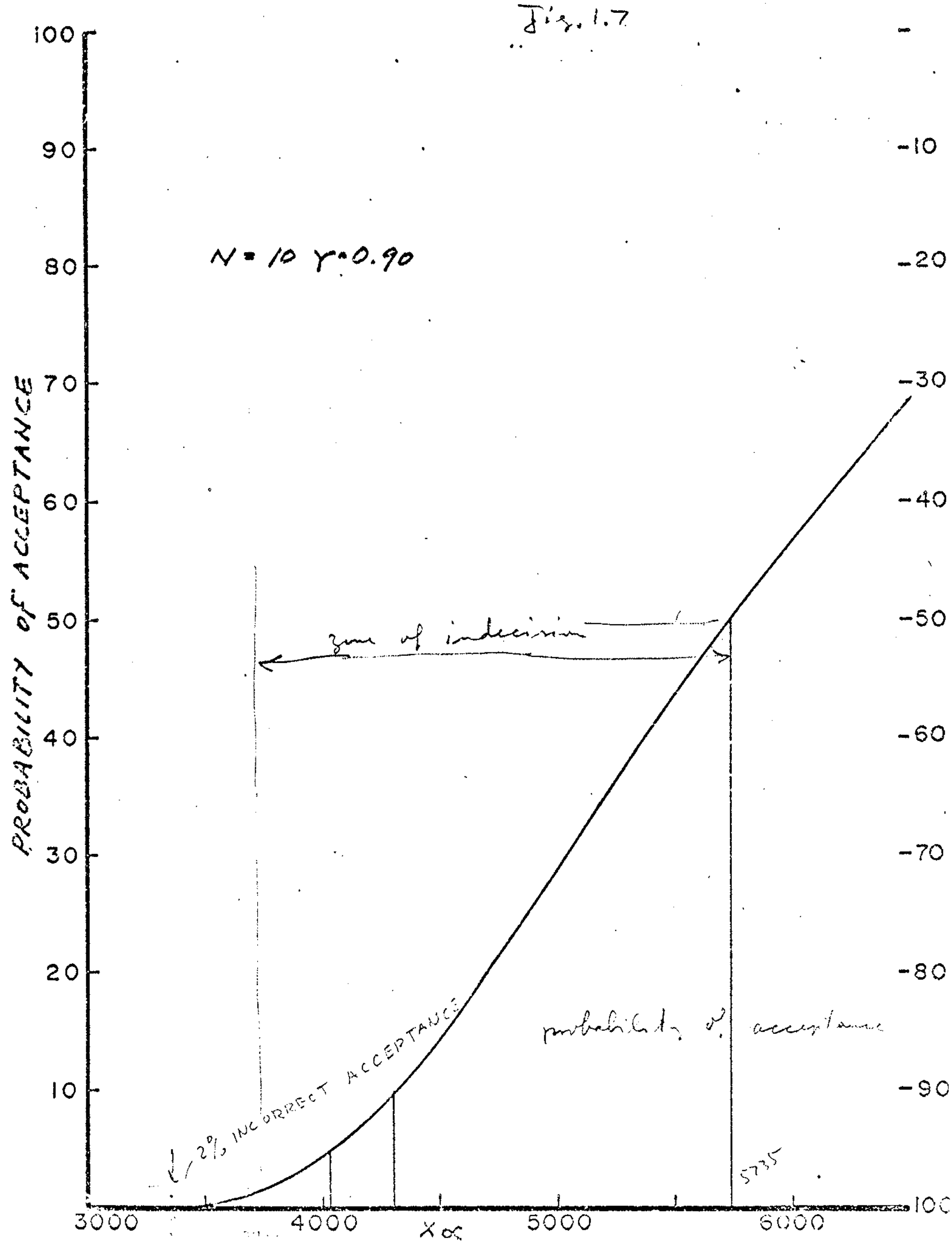
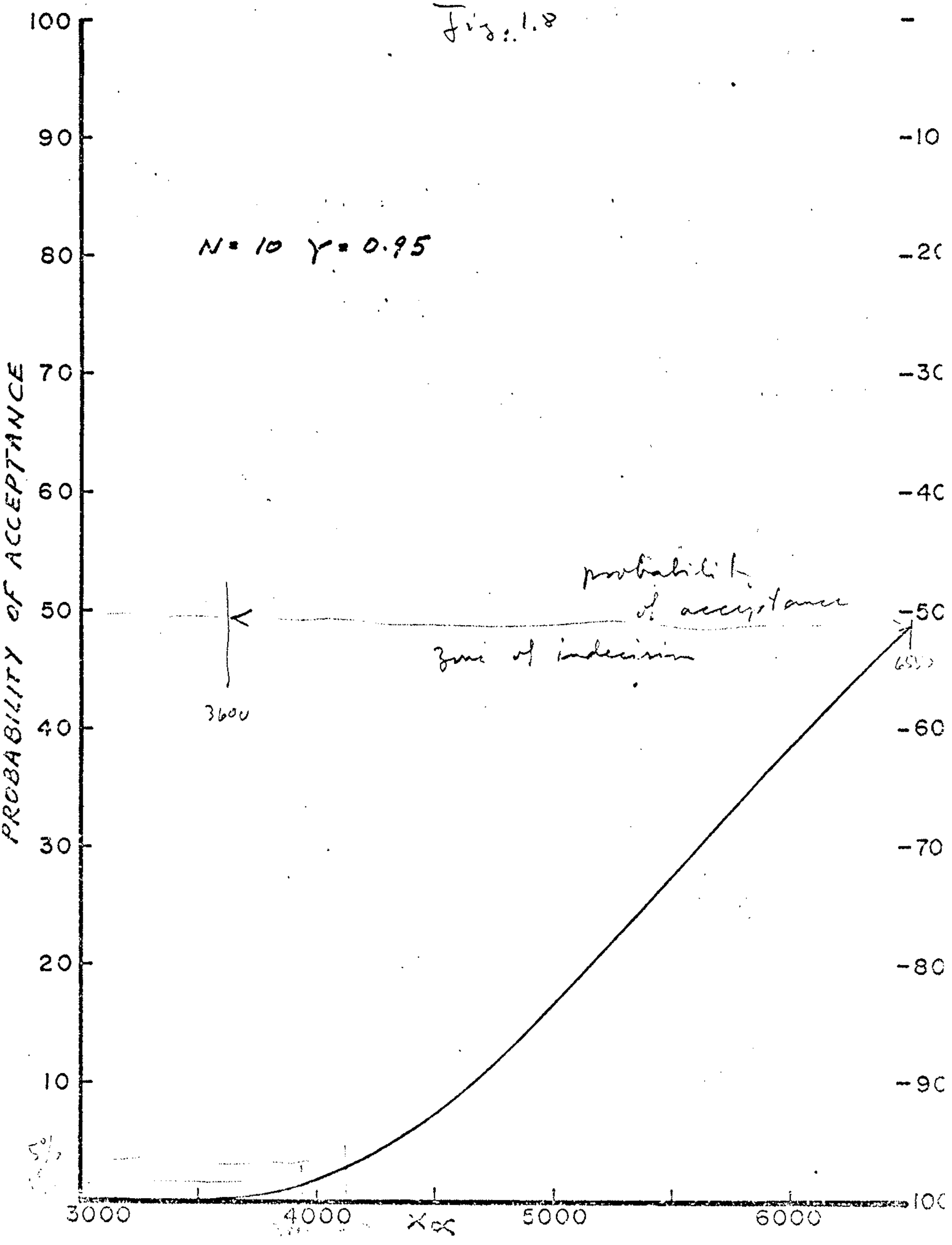


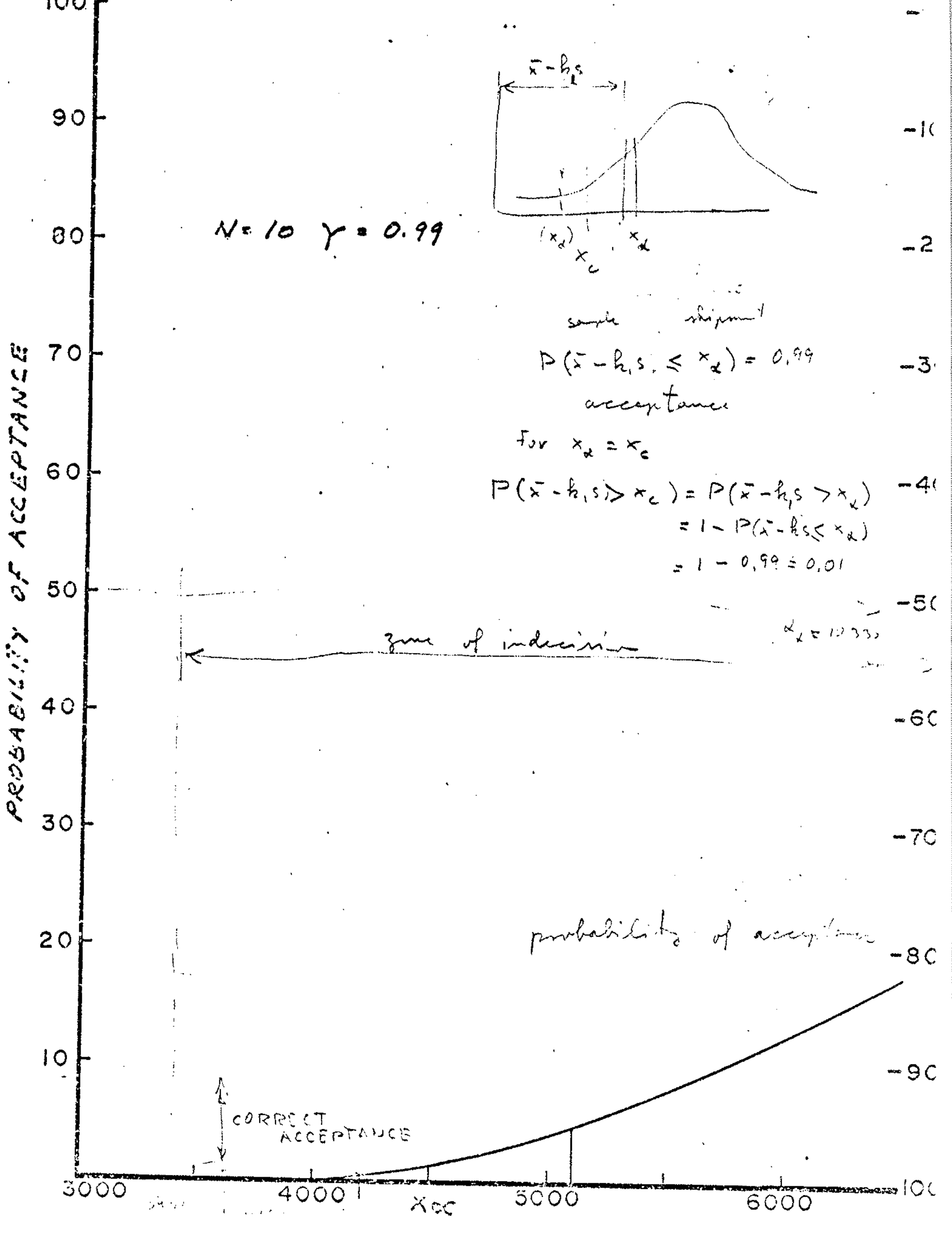
Fig. 1.7
N = 10 γ = 0.90
PROBABILITY of ACCEPTANCE
zone of indecision
2% INCORRECT ACCEPTANCE
probability of acceptance
5735
X_α
3000
4000
5000
6000
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100

Fig. 1.8

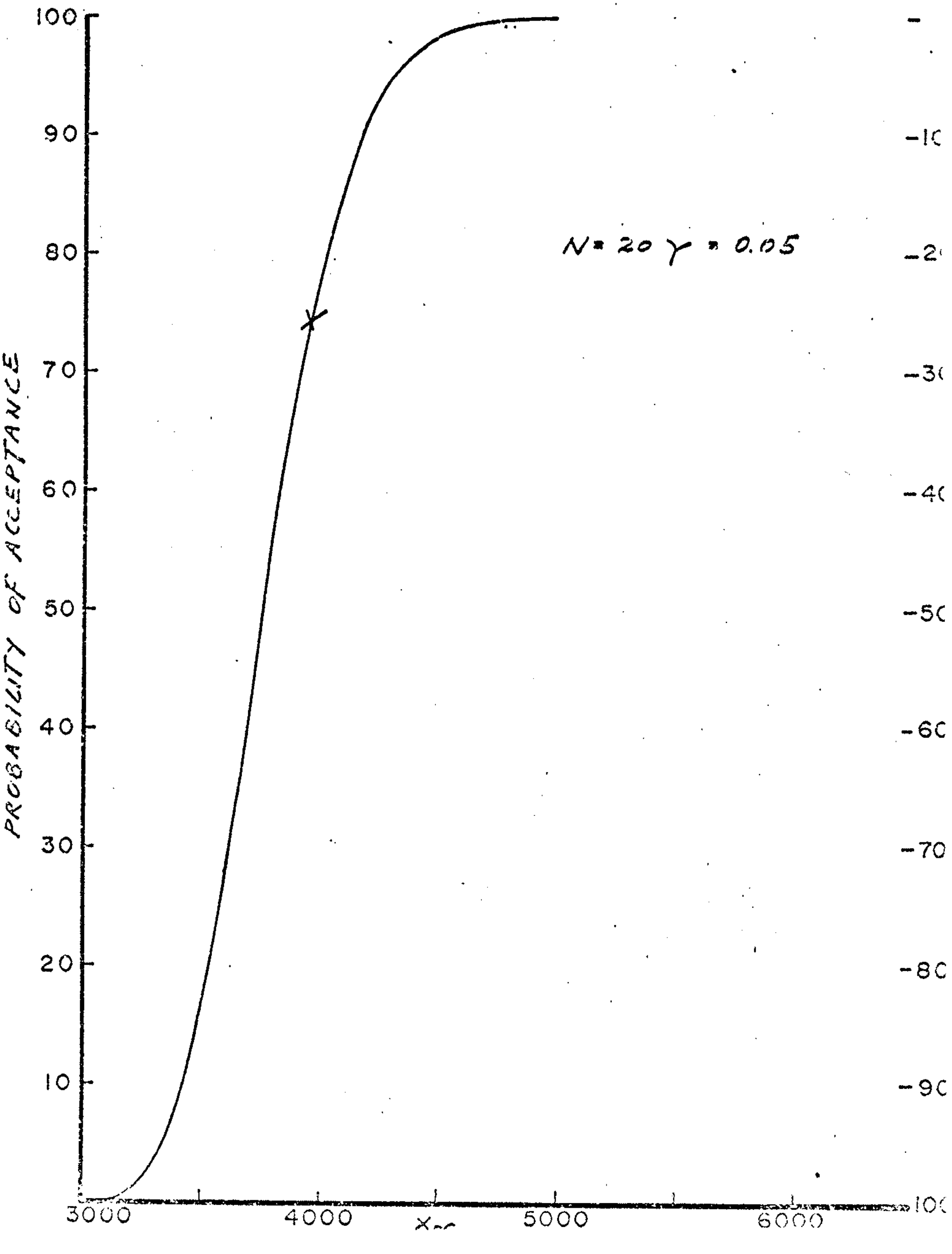


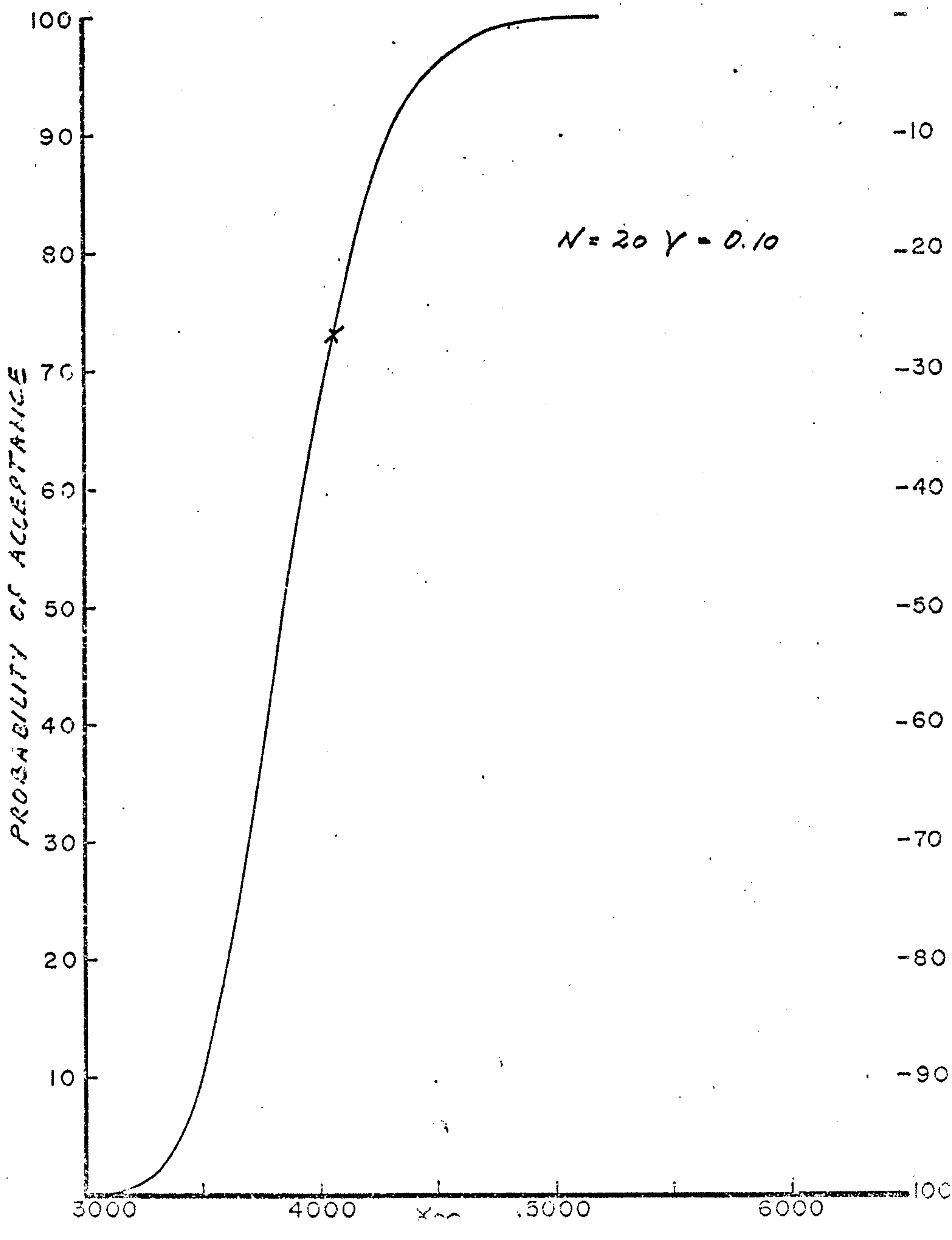
N = 10 γ = 0.95
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
5%
10
20
30
40
50
60
70
80
90
100
probability of acceptance
zone of indecision
3600
6500
3000
4000
5000
6000
X_α

FIG 1.9

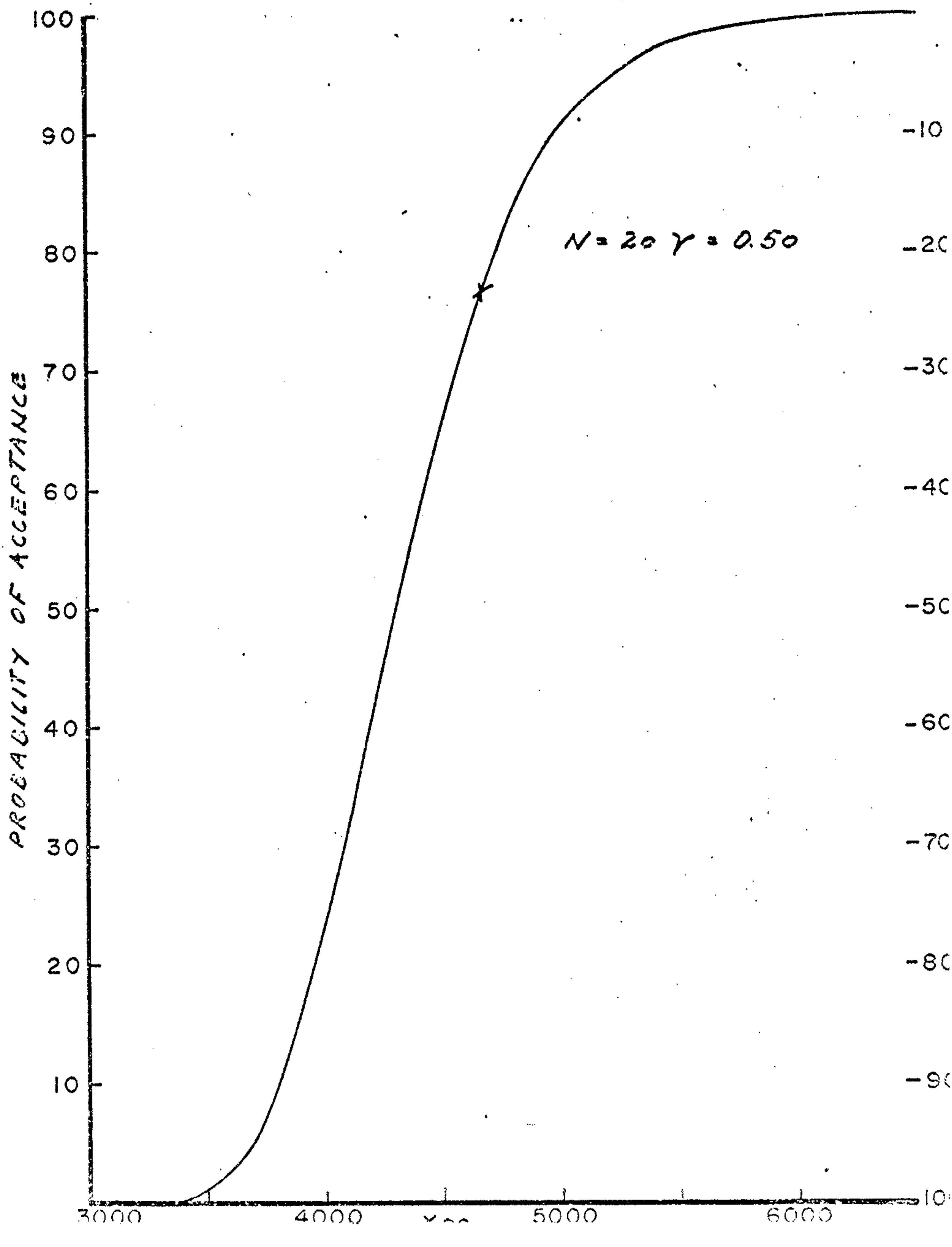


PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3000
4000
5000
6000
Xcc
N = 10 γ = 0.99
x̄ - k₁s
(x_α)
x_c
x_α
sample
shipment
P(x̄ - k₁s ≤ x_α) = 0.99
acceptance
for x_α = x_c
P(x̄ - k₁s > x_c) = P(x̄ - k₁s > x_α)
= 1 - P(x̄ - k₁s ≤ x_α)
= 1 - 0.99 = 0.01
zone of indecision
probability of acceptance
CORRECT ACCEPTANCE

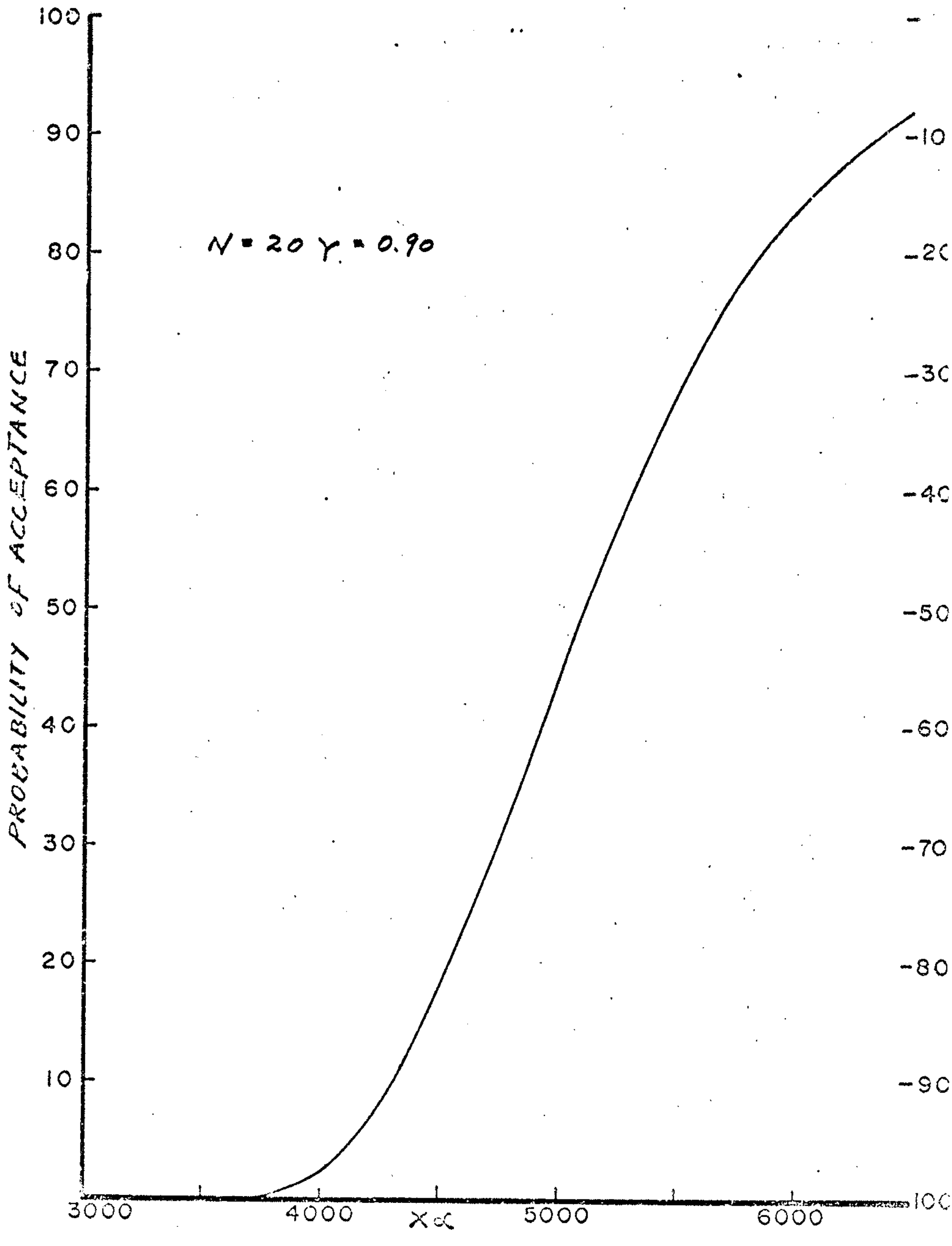

PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
3000
4000
5000
6000
N = 20 γ = 0.05



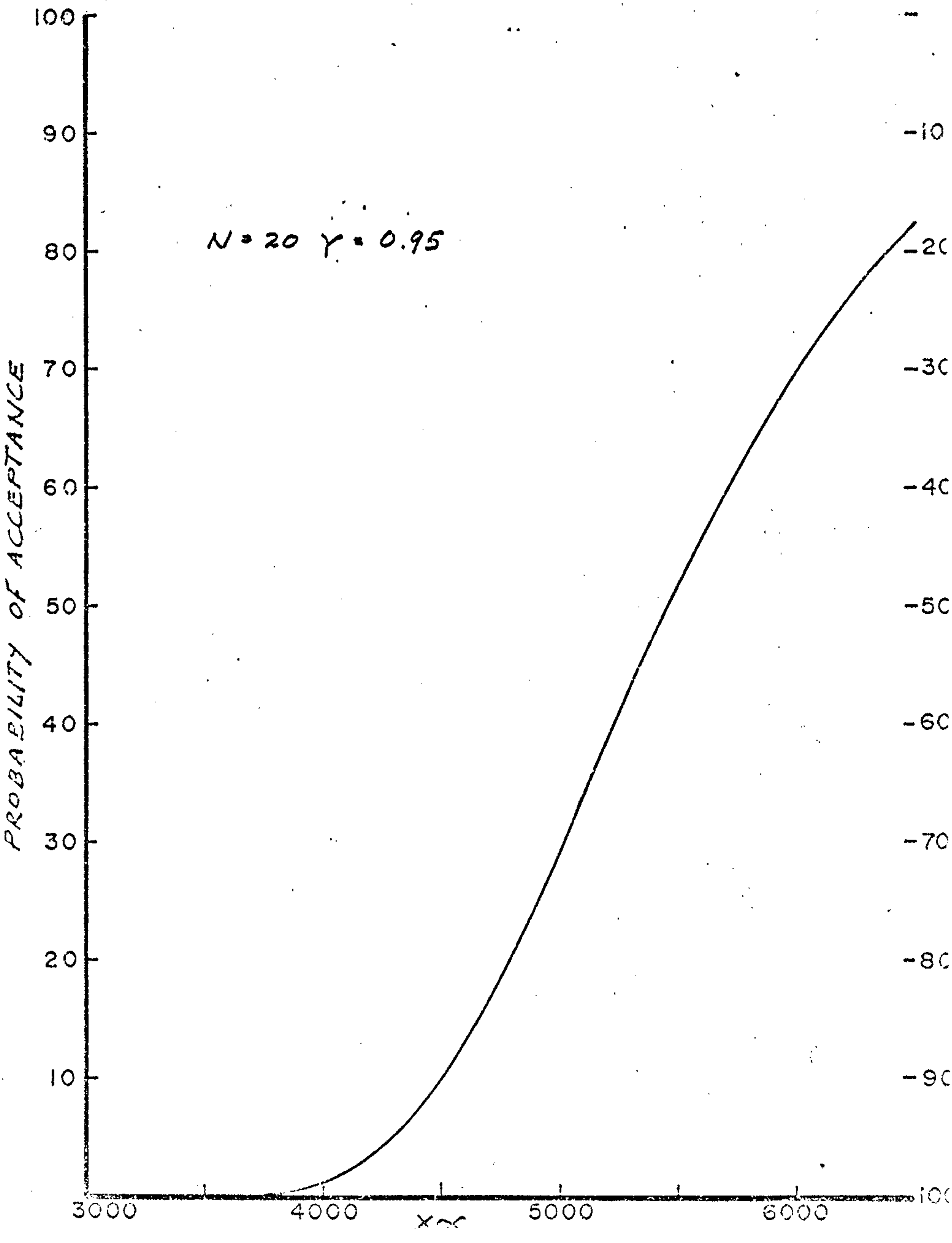
PROBABILITY OF ACCEPTANCE
N = 20 γ = 0.10
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3000
4000
5000
6000



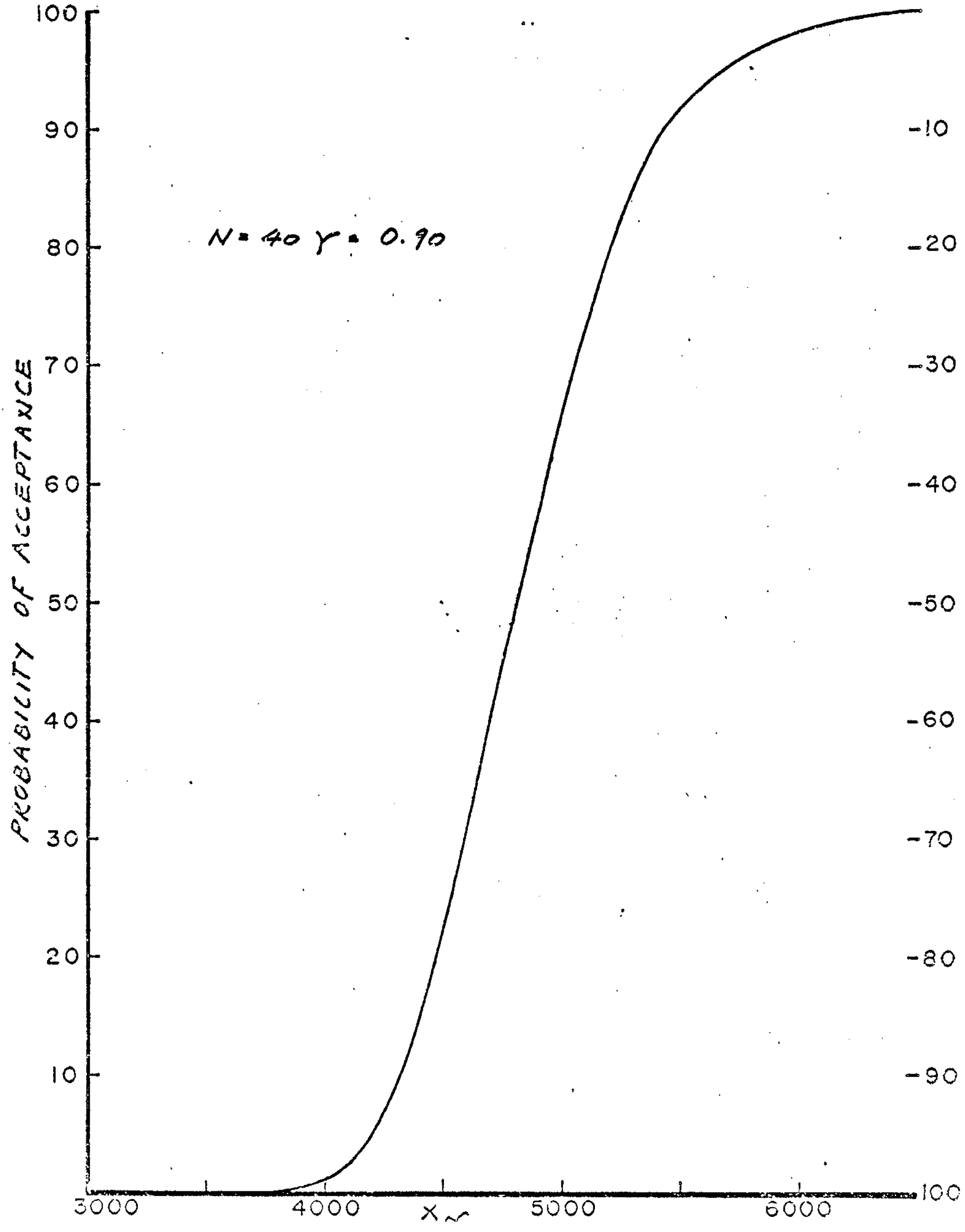
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
N = 20 γ = 0.50
-10
-20
-30
-40
-50
-60
-70
-80
-90
3000
4000
5000
6000



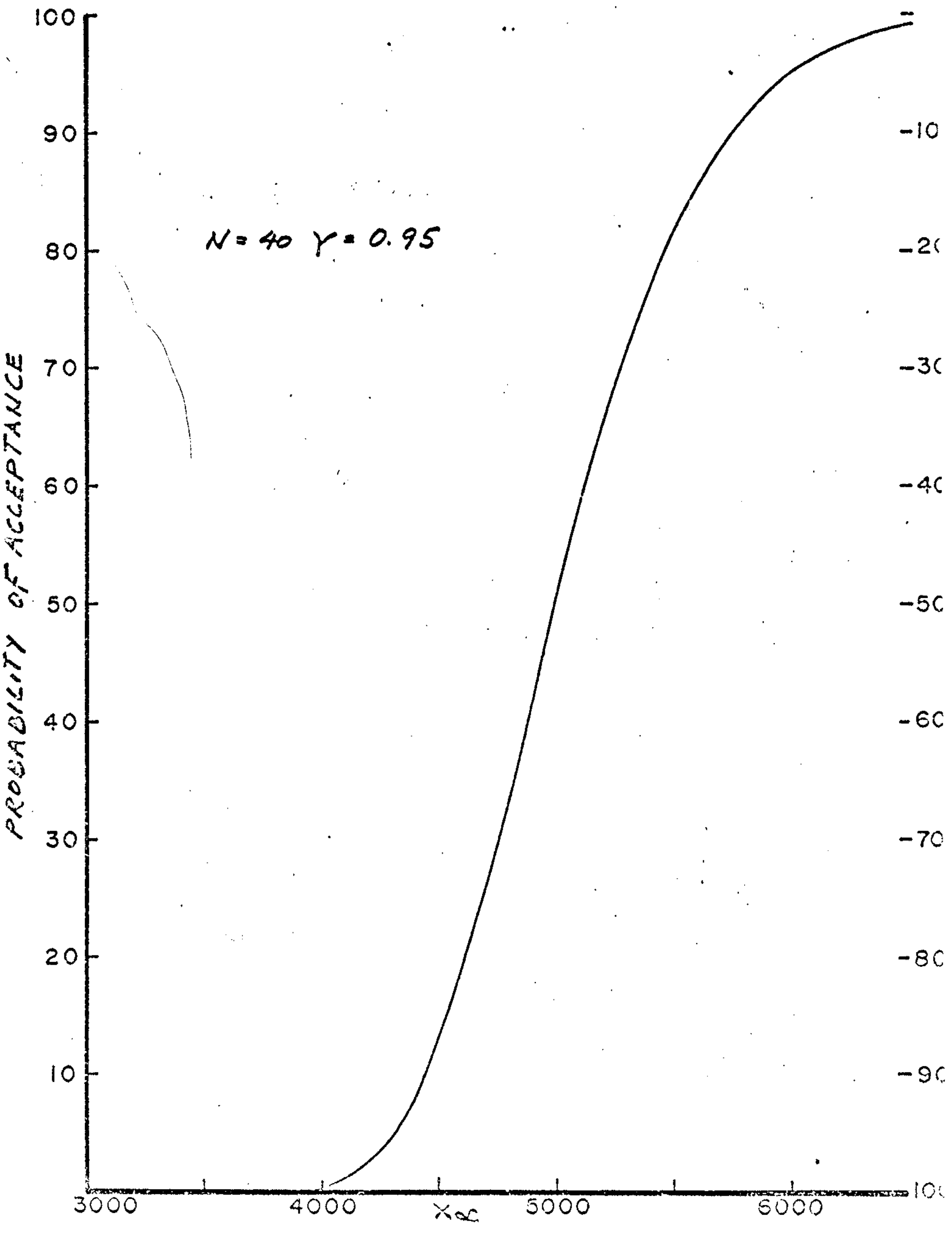
N = 20 γ = 0.90
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
3000
4000
5000
6000
Xα
10
20
30
40
50
60
70
80
90
100



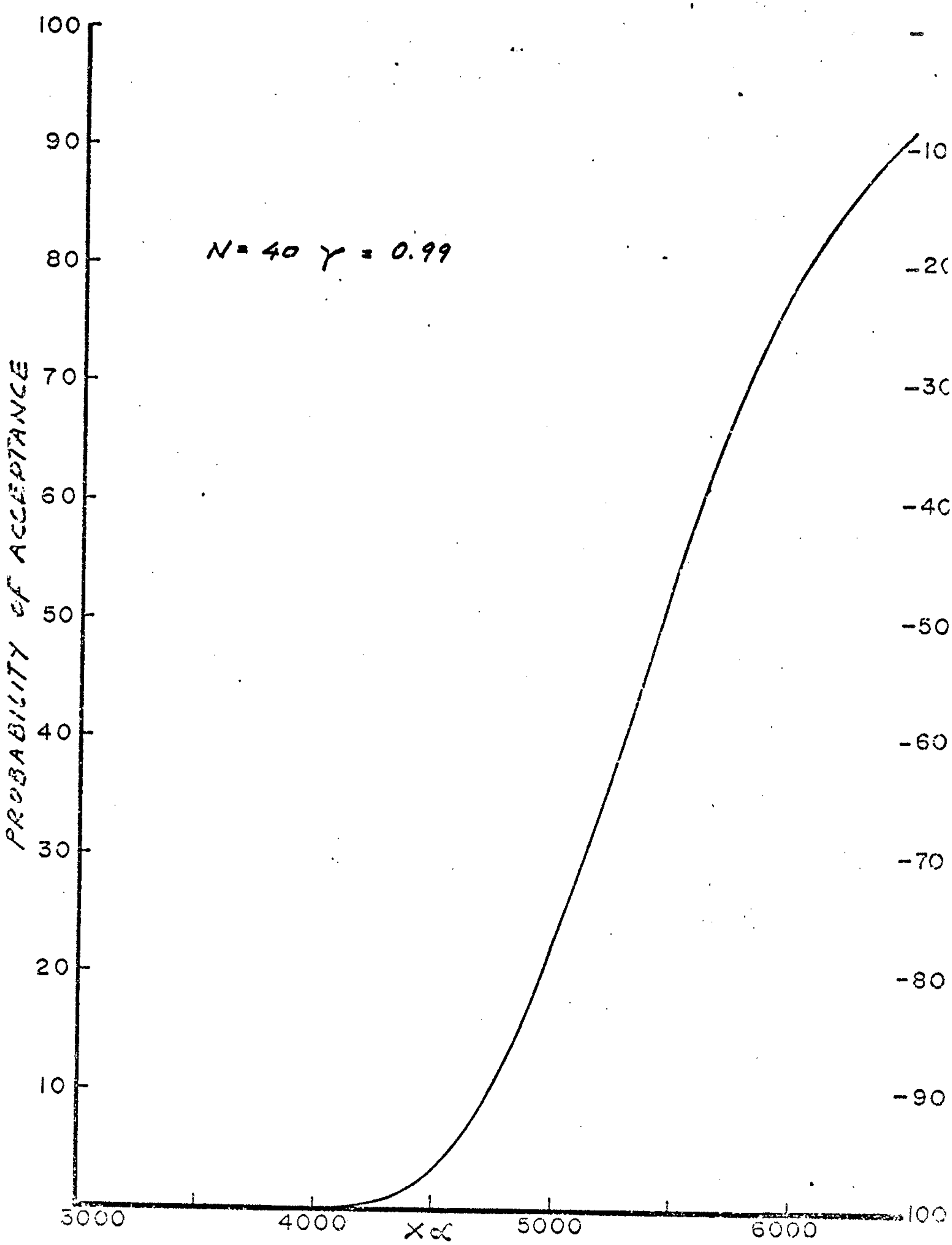
PROBABILITY OF ACCEPTANCE
N = 20 γ = 0.95
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
3000
4000
5000
6000

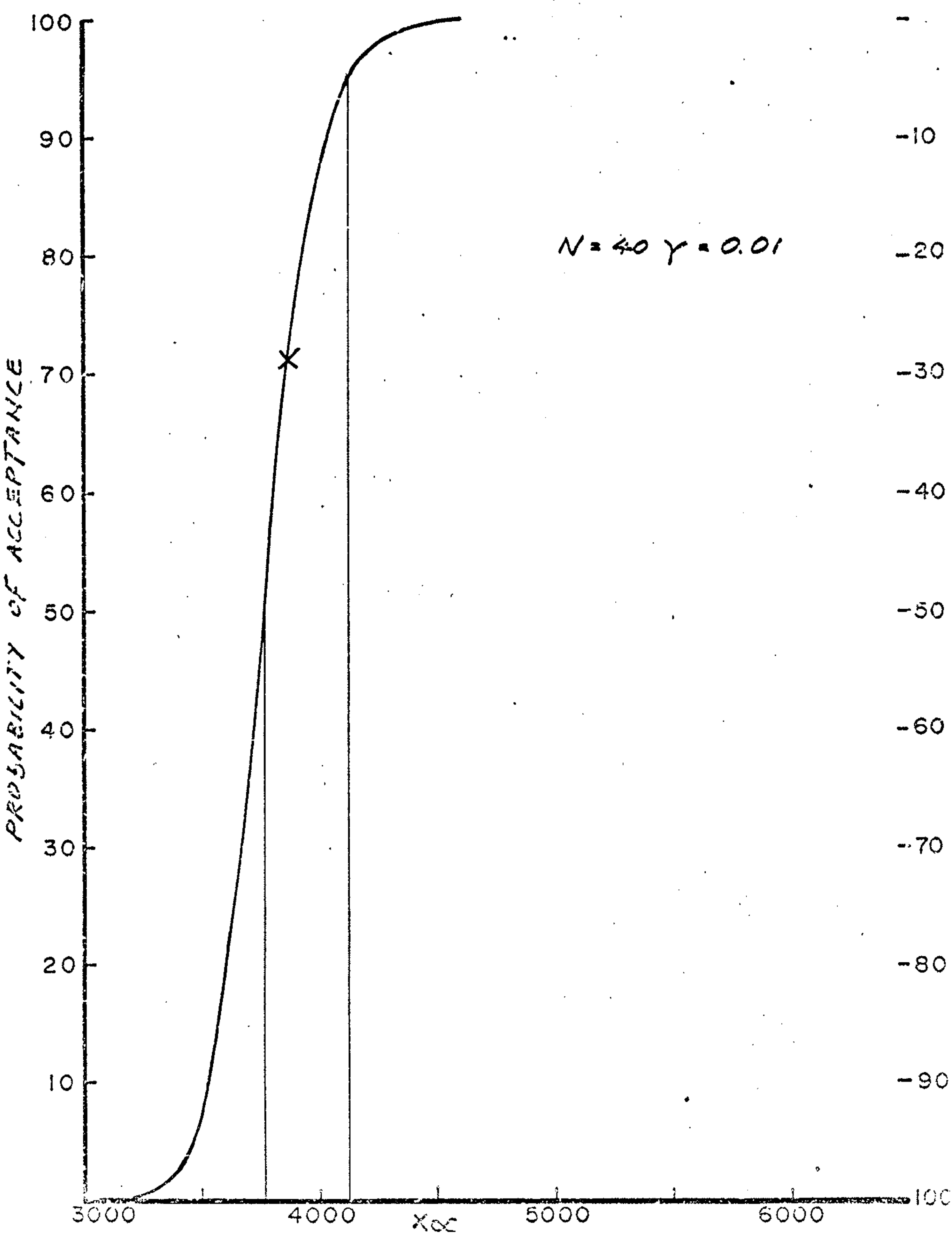


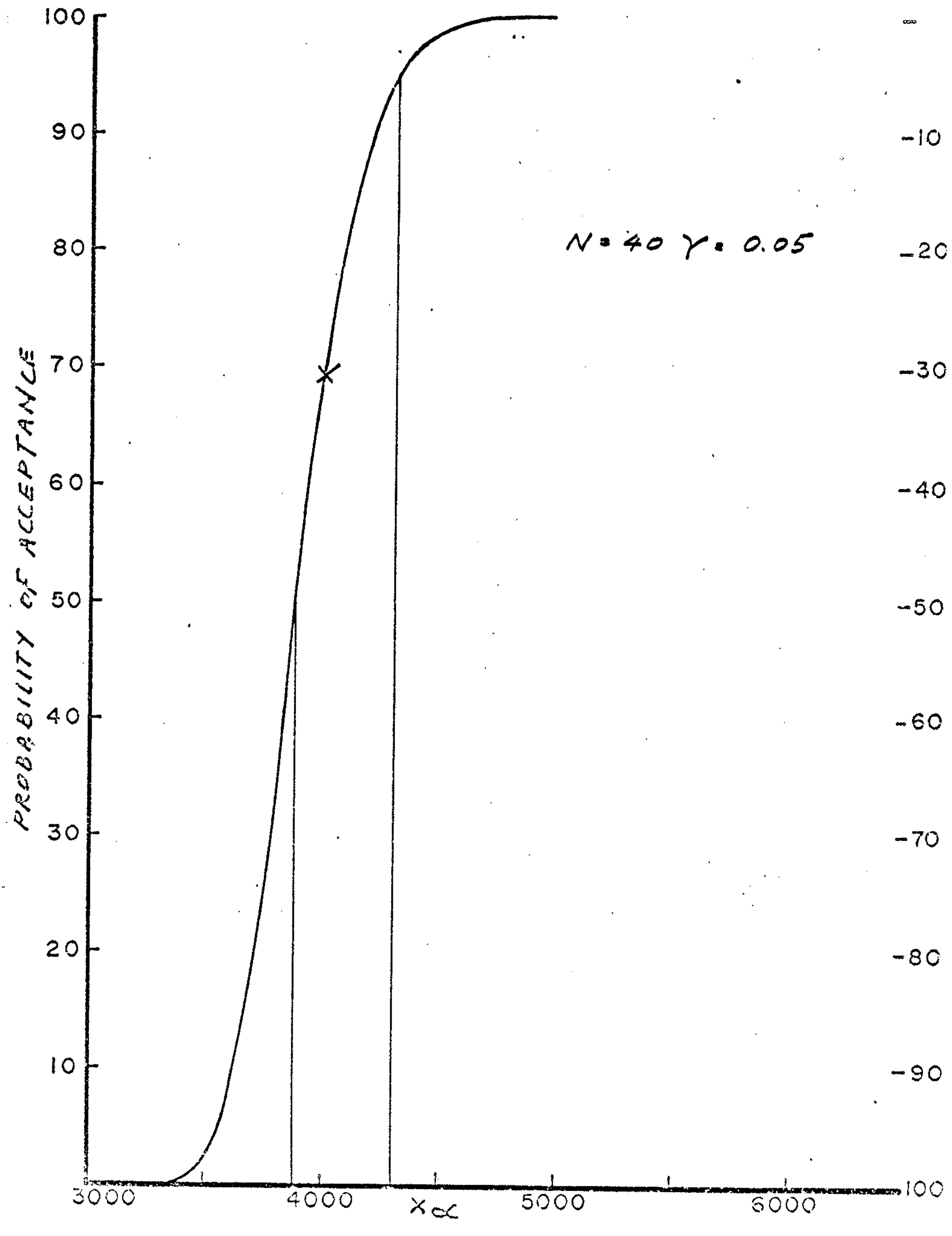
N = 40 γ = 0.90
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
3000
4000
5000
6000
X



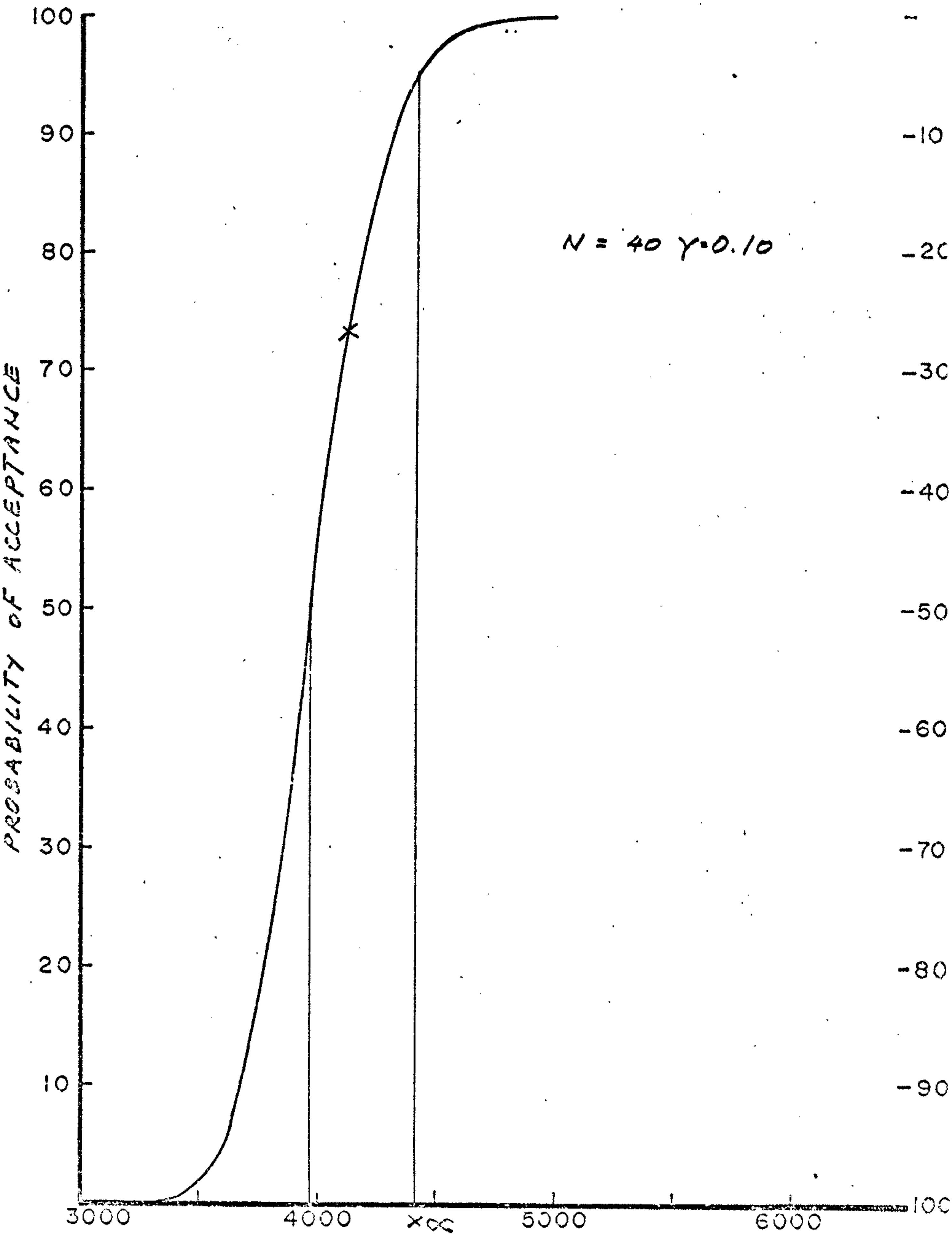
N = 40 γ = 0.95
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
3000
4000
5000
6000


100
90
80
70
60
50
40
30
20
10
PROBABILITY OF ACCEPTANCE
N = 40 γ = 0.99
3000
4000
5000
6000
Xα
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100

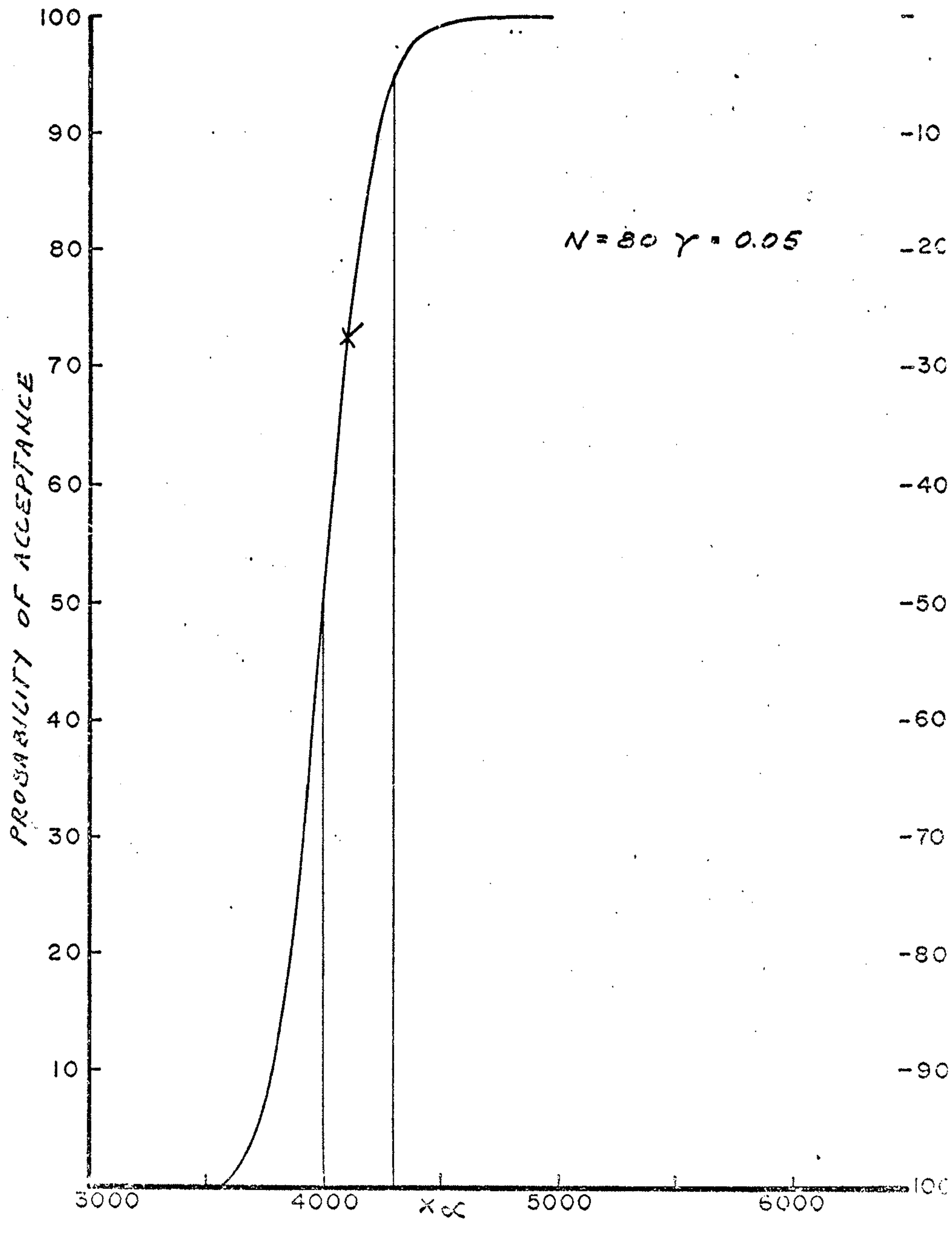

PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3000
4000
5000
6000
X_α
$N = 40 \ \gamma = 0.01$



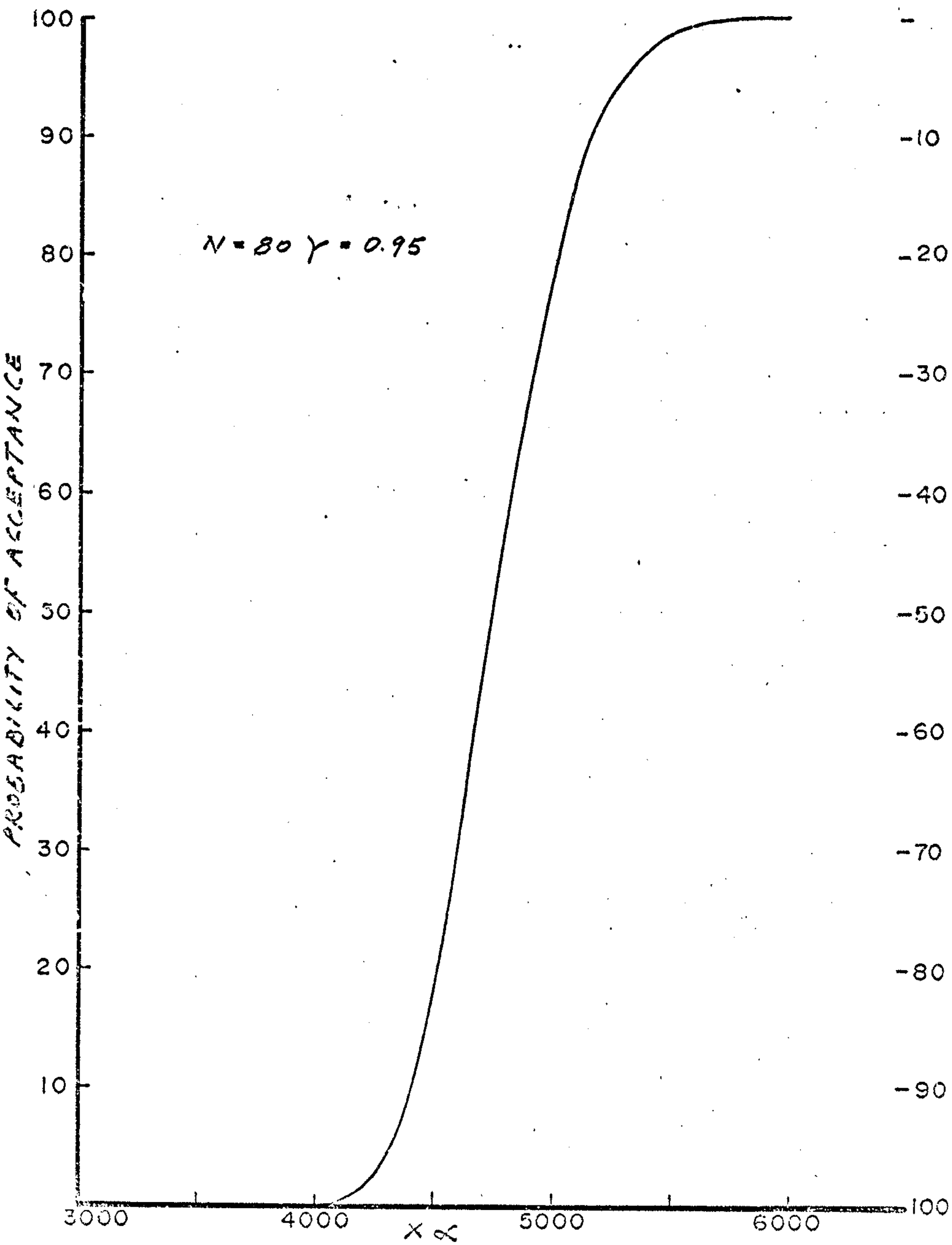
N = 40 γ = 0.05
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
3000
4000
5000
6000
X_α



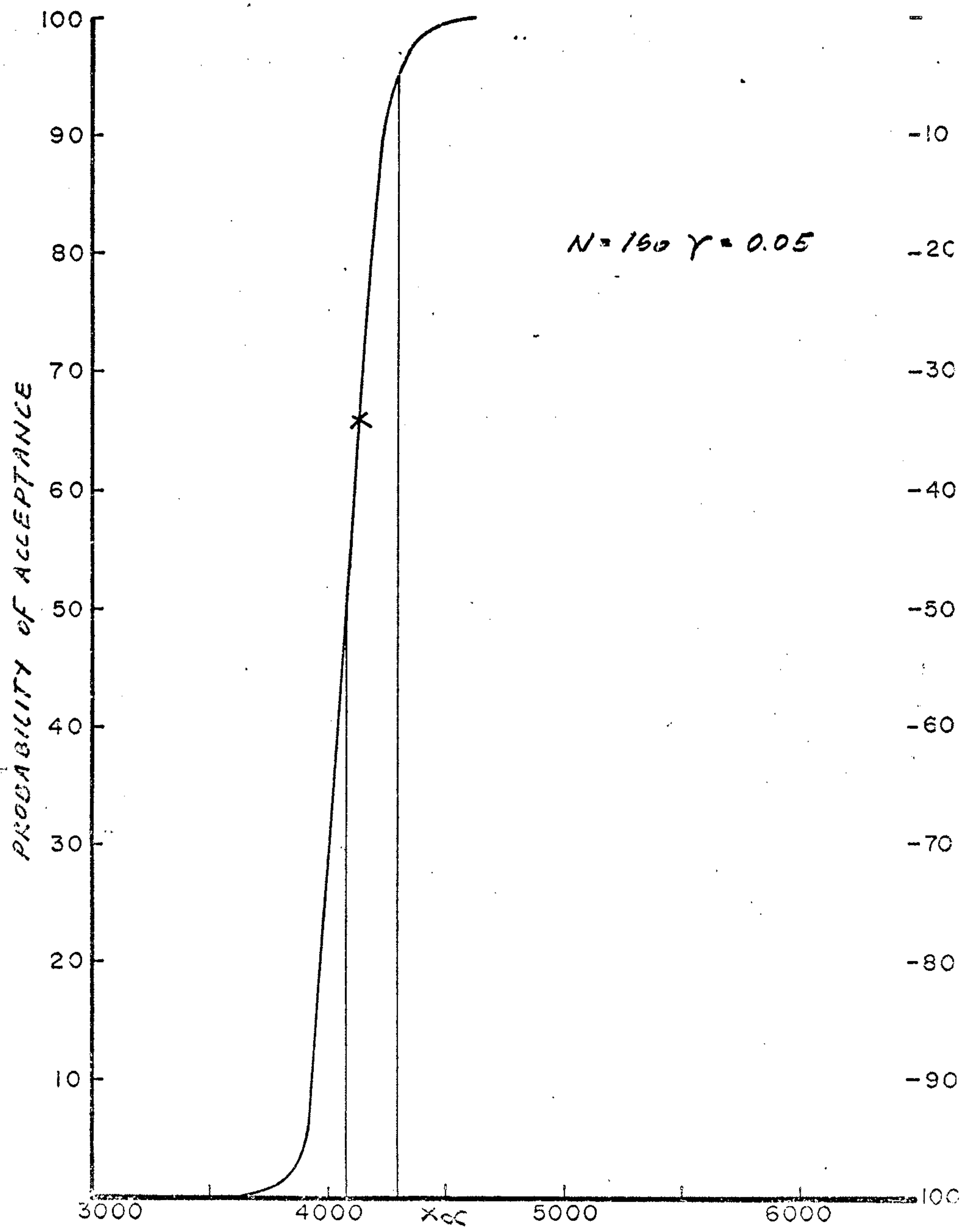
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
N = 40 γ=0.10
3000
4000
Xα
5000
6000



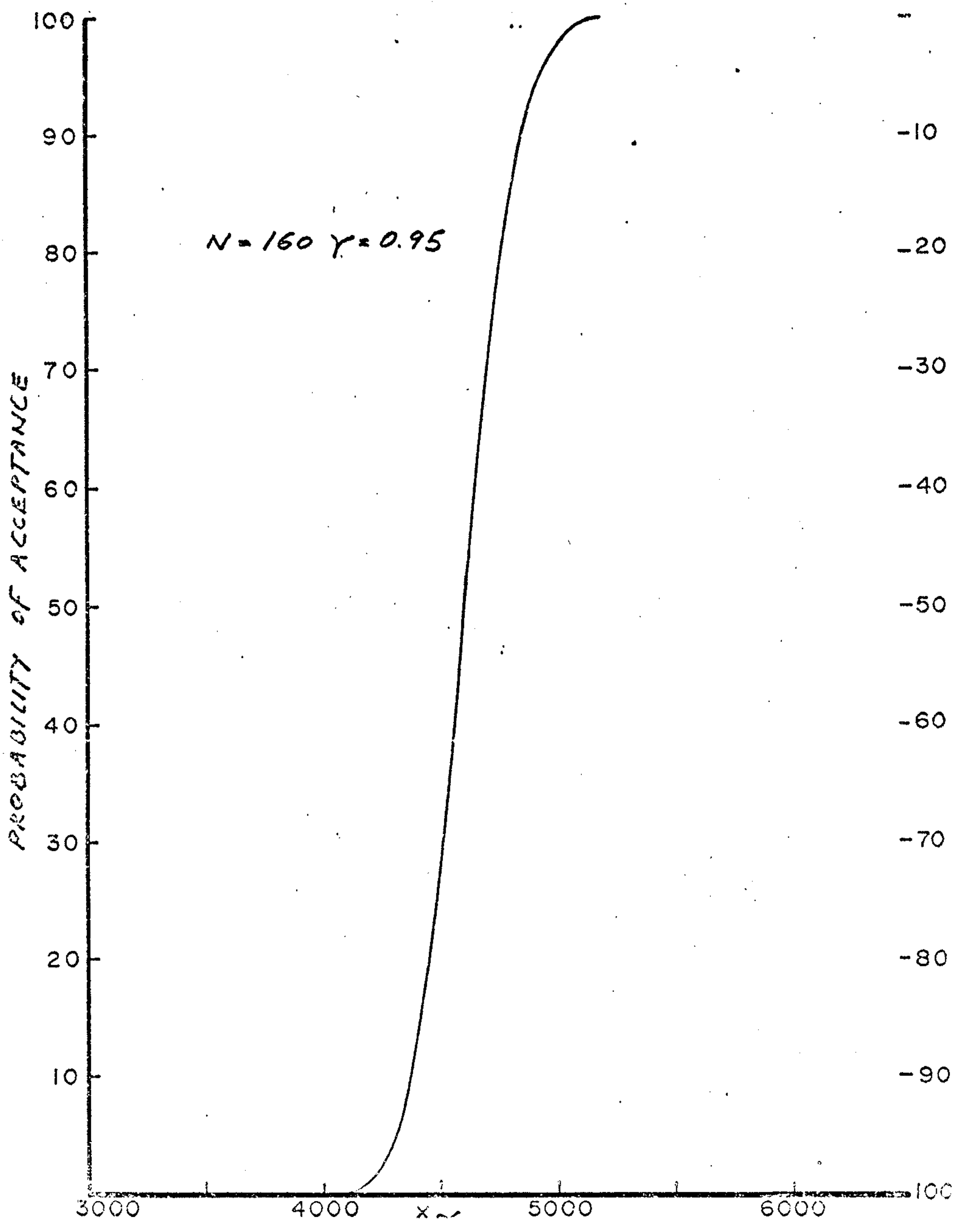
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
PROBABILITY OF ACCEPTANCE
N = 80 γ = 0.05
3000
4000
5000
6000
Xα



N = 80 γ = 0.95
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3000
4000
5000
6000
X α

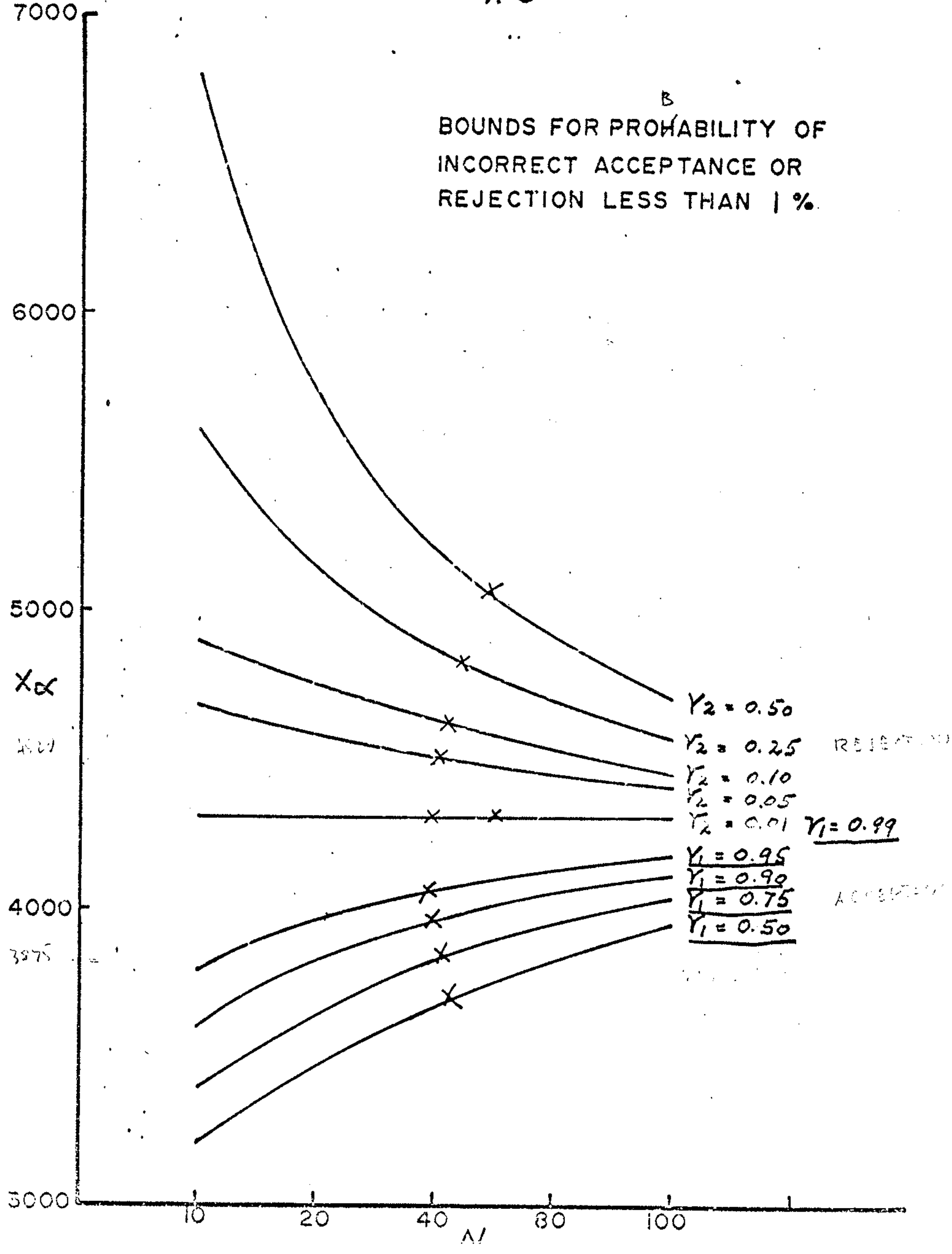


100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
N = 150 γ = 0.05
PROBABILITY OF ACCEPTANCE
3000
4000
5000
6000
$X_{\alpha C}$



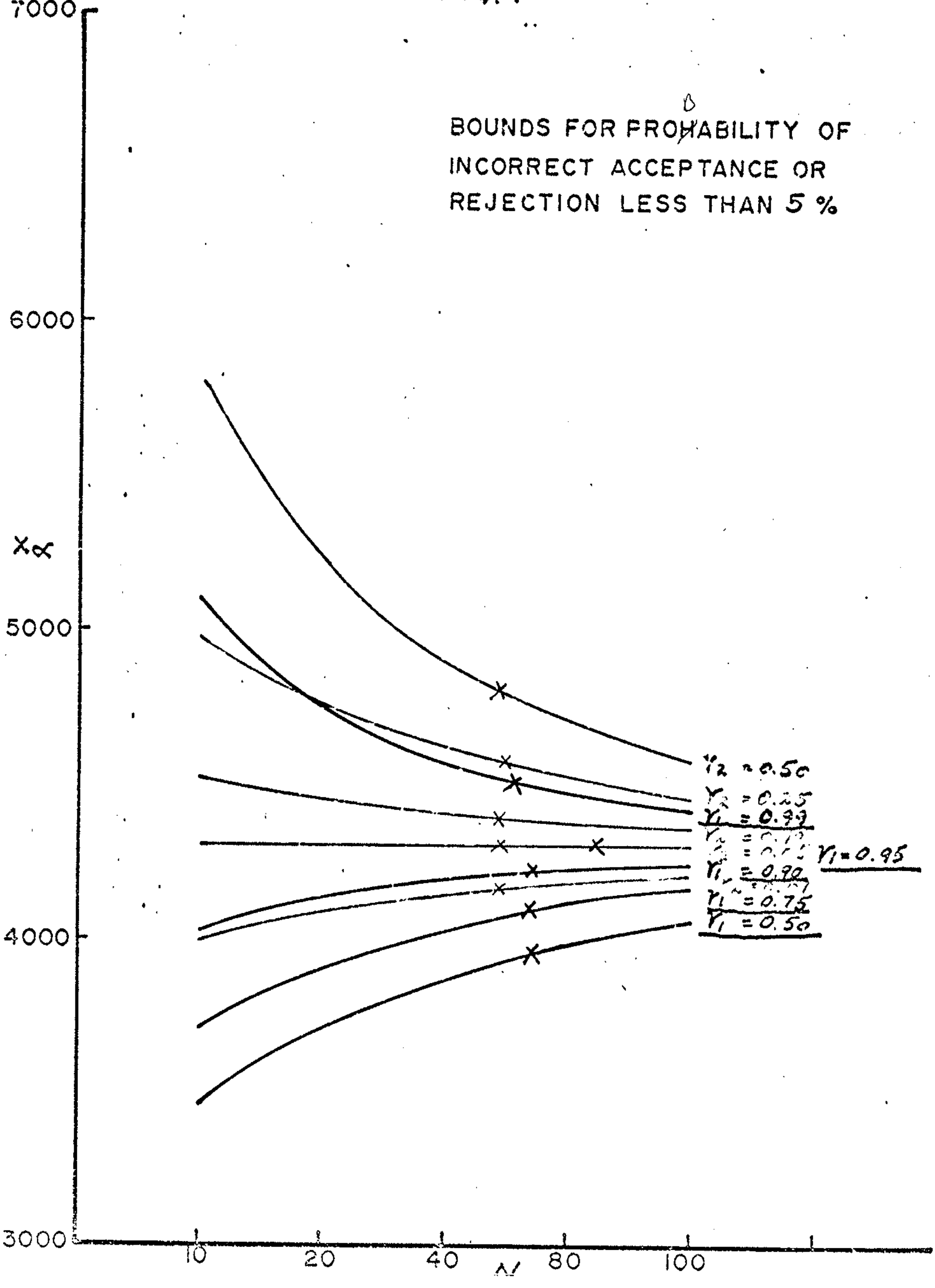
N = 160 Y = 0.95
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
3000
4000
5000
6000

FIG. 6



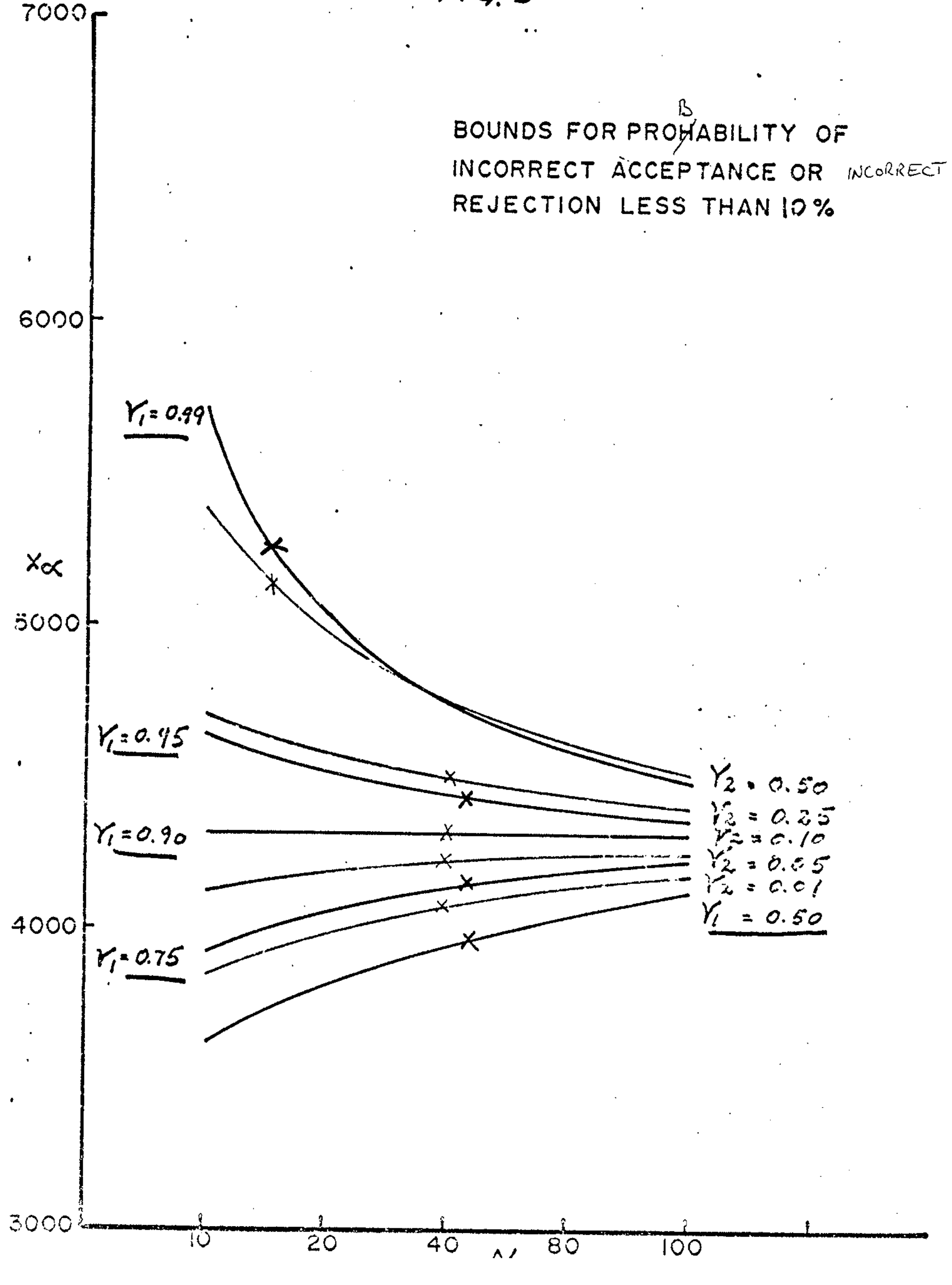
BOUNDS FOR PROBABILITY OF INCORRECT ACCEPTANCE OR REJECTION LESS THAN 1 %
7000
6000
5000
4000
3000
X_α
$\gamma_2 = 0.50$
$\gamma_2 = 0.25$
$\gamma_2 = 0.10$
$\gamma_2 = 0.05$
$\gamma_2 = 0.01$
$\gamma_1 = 0.99$
$\gamma_1 = 0.95$
$\gamma_1 = 0.90$
$\gamma_1 = 0.75$
$\gamma_1 = 0.50$
10
20
40
80
100
N

FIG. 7



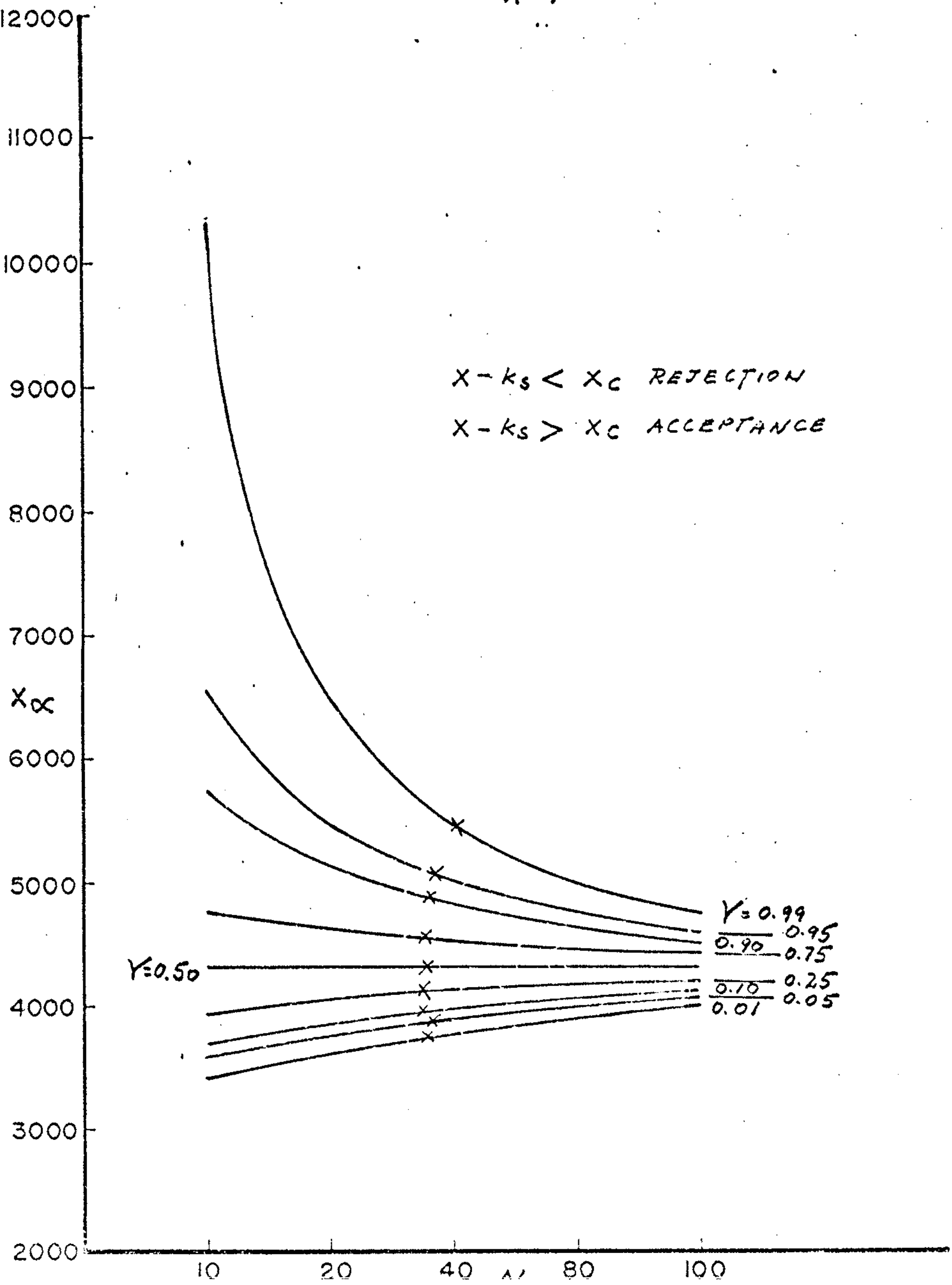
BOUNDS FOR PROBABILITY OF
INCORRECT ACCEPTANCE OR
REJECTION LESS THAN 5 %
7000
6000
5000
4000
3000
X_α
10
20
40
80
100
N
$\gamma_2 = 0.50$
$\gamma_2 = 0.25$
$\gamma_1 = 0.99$
$\gamma_1 = 0.95$
$\gamma_1 = 0.90$
$\gamma_1 = 0.75$
$\gamma_1 = 0.50$

FIG. 8



BOUNDS FOR PROBABILITY OF INCORRECT ACCEPTANCE OR INCORRECT REJECTION LESS THAN 10%
7000
6000
5000
4000
3000
x_α
$Y_1 = 0.99$
$Y_1 = 0.95$
$Y_1 = 0.90$
$Y_1 = 0.75$
$Y_2 = 0.50$
$Y_2 = 0.25$
$Y_2 = 0.10$
$Y_2 = 0.05$
$Y_2 = 0.01$
$Y_1 = 0.50$
10
20
40
80
100
N

FIG. 9



12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
Xα
X - ks < Xc REJECTION
X - ks > Xc ACCEPTANCE
Y=0.50
Y= 0.99
0.95
0.90
0.75
0.25
0.10
0.05
0.01
10
20
40
N
80
100

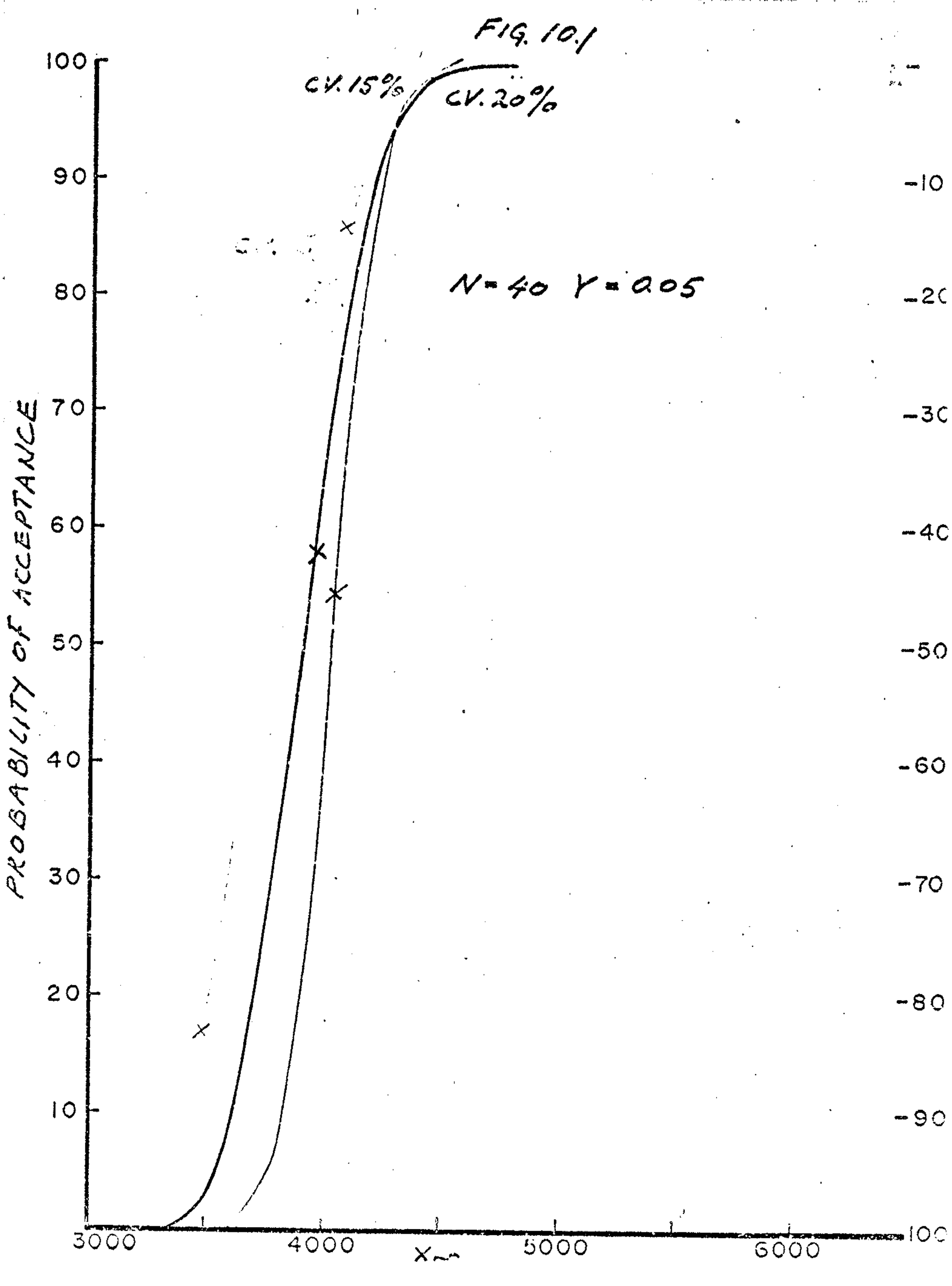
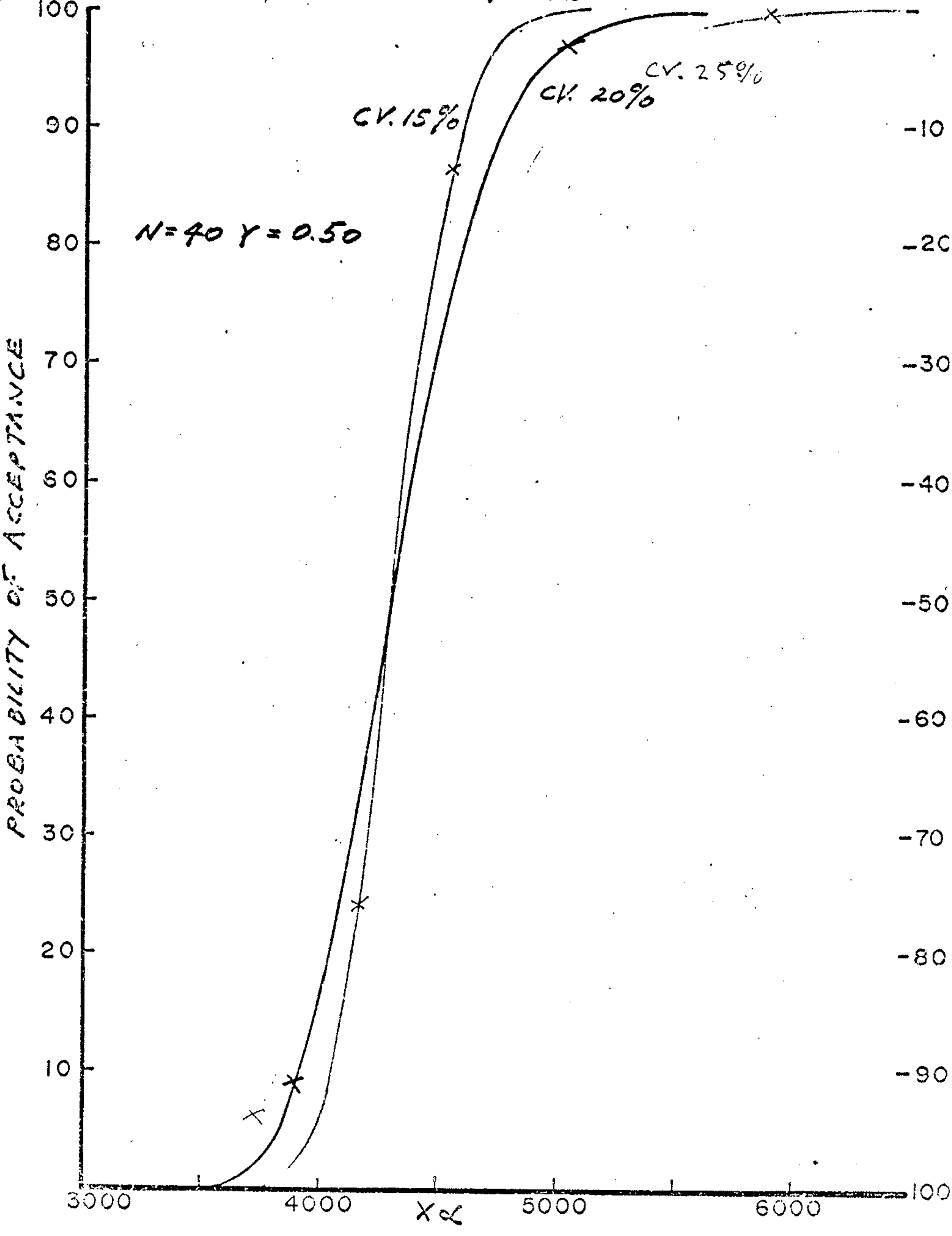


FIG. 10.1
C.V. 15%
C.V. 20%
N = 40 Y = 0.05
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
3000
4000
5000
6000

FIG. 10.2


100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
3000
4000
5000
6000
Xα
PROBABILITY OF ACCEPTANCE
N=40 Y=0.50
CV. 15%
CV. 20%
CV. 25%

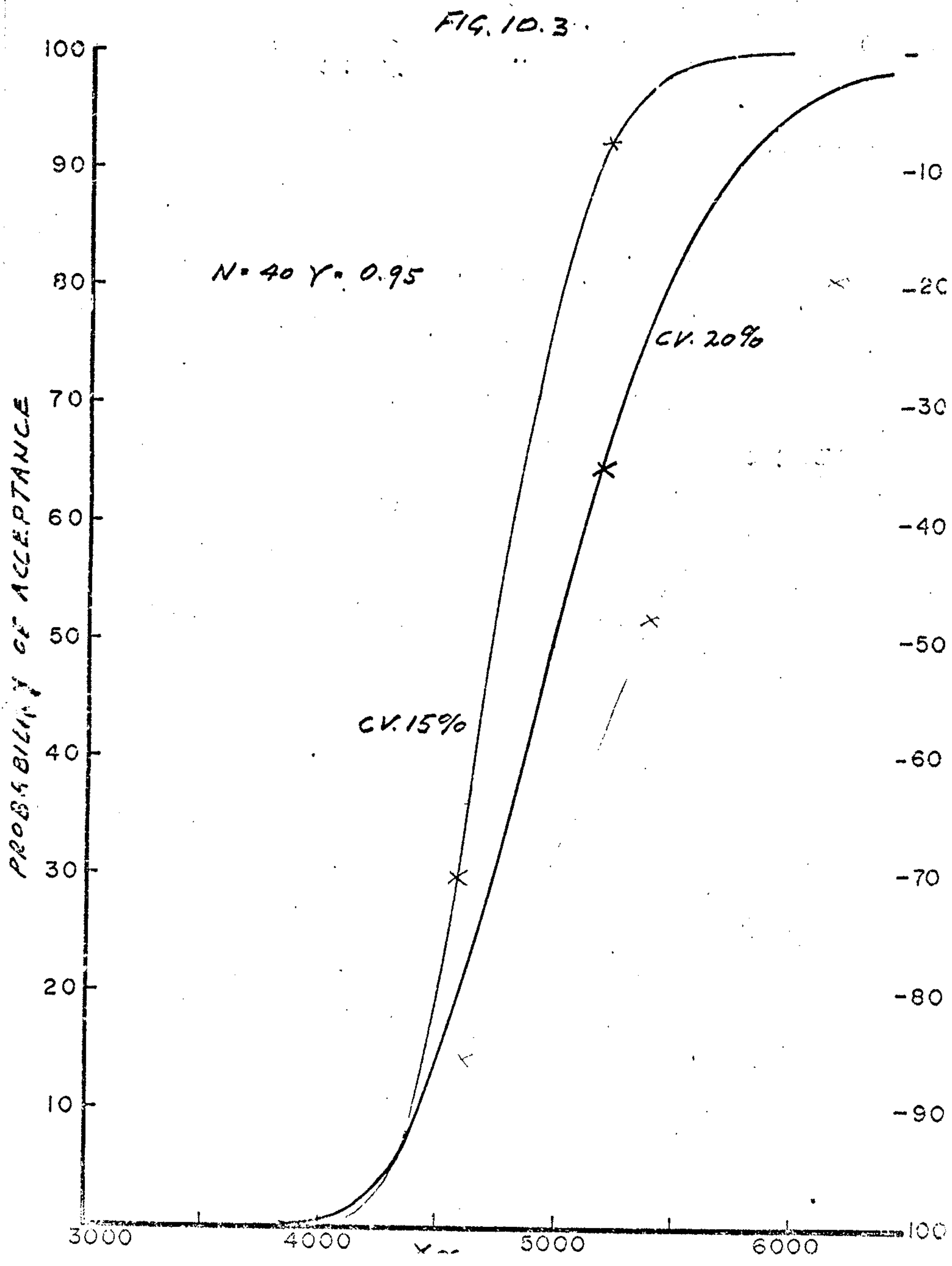


FIG. 10.3
N = 40 Y = 0.95
CV. 15%
CV. 20%
PROBABILITY OF ACCEPTANCE
100
90
80
70
60
50
40
30
20
10
-10
-20
-30
-40
-50
-60
-70
-80
-90
100
3000
4000
5000
6000

CIB-W18/13-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

AMENDMENTS TO CIB STRUCTURAL TIMBER DESIGN CODE
(Fourth Draft)

OTANIEMI, FINLAND
JUNE 1980

CONSEIL INTERNATIONAL DU BATIMENT • INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING GROUP W18
TIMBER STRUCTURES

AGREED CHANGES

CIB
STRUCTURAL TIMBER DESIGN CODE

BASED ON COMMENTS FROM

ISO TC/165 - Ottawa, September 1979
CIB-W18 - Bordeaux, October 1979

Fourth draft, June 1979

LIST OF CONTENTS

Addition:

9. CALCULATION OF FIRE RESISTANCE

1.4 Notations

Addition:

Main symbols

γ Partial coefficient (load factor, material factor)

Subscripts

a Accidental

d Design

f Load (on γ)

k Characteristic

m Material property (on γ)

Subscripts are omitted whenever possible without confusion.

: As an example (5.1.1.1 a): $\sigma_t \leq k_{size,0} \cdot f_{t,0}$ is to be read as $\sigma_{t,d} \leq k_{size,0} \cdot f_{t,0,d}$.

2.1.1 Characteristic values

Change:

The characteristic strength and stiffness values given in this code for timber and wood-based materials are defined as lower 5-percentile values ~~(i.e. 95% of all possible test results exceed the characteristic value).~~ directly applicable to a load duration of 3 to 5 mins. at a temperature of 20 ± 3°C and relative humidity of 0.65 ± 0.02. Where the characteristic values are estimated from a limited number of tests the estimate shall be made with a confidence level of 0.75.

the population

2.2 Climate classes

Change:

The name will be changed to **Moisture classes.**

Addition:

: Based on the moisture properties of ordinary softwoods Figure 2.2 shows the moisture class dependent on
: temperature and relative humidity.

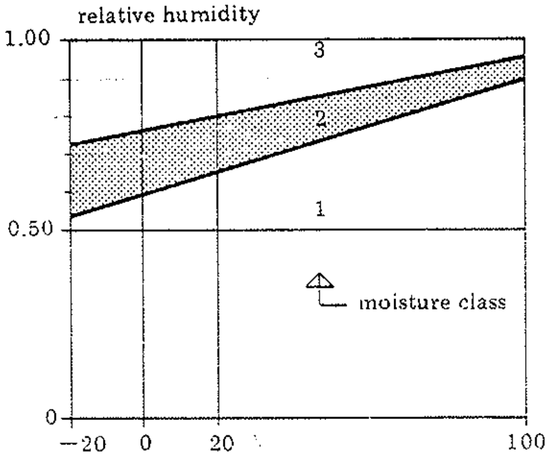


Figure 2.2

2.3 Load-duration classes

Change:

The name of the load-duration class »normal» will be changed to long-term

The examples in Table 2.3b are for permanent *structures* (not buildings only)

3. BASIC DESIGN RULES

Change:

Most of Chapter 3, which is mainly included for information, will be transferred from »normal print» (main text) to »small print».

3.0 General

Addition:

The design can be based on calculations, on testing or on a combination hereof.

Change:

when exposed to an accidental load

The main structure should normally be designed in such a way that it should not subsequently be damaged to an extent disproportionate to the extent of the original incident. This requirement may be achieved by

due to an accidental load.

a) designing the structure in such a way that if any single load-bearing member becomes incapable of carrying load this will not cause collapse of the whole structure or any significant part of it, or

b) where necessary, ensuring (by design or by protective measures) that no essential load-bearing member can be made ineffective as a result of an accidental load

4.1.1 Standard strength classes

Addition:

The standard strength classes can be used also for round timber (poles).

Change:

The lines:

: 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 $f_{t,0}/f_m$.

are transferred from »small print» to »normal print» (main text).

4.3.1 Standard glulam strength classes

Change:

The lines:

: 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.

are deleted.

Addition:

It is emphasized that the introduction of standard strength classes does not prevent the introduction of other grades.

4.7 Steel parts

Change:

Nails, screws, and other steel parts should as a minimum be protected against corrosion according to Table 4.7. The protection is described in relation to ISO 2081, Electroplated coatings of zink on iron or steel, but other protection systems may be used. The requirements for protection against corrosion may be relaxed where surface corrosion will not significantly reduce the load-carrying capacity.

Table 4.7 Minimum protection against corrosion

climate class	nails, screws and bolts	other steel parts
1	none[1)]	none[1)]
	none - Fe/Zn 5c[2)]	Fe/Zn 5c
2	Fe/Zn 12c	Fe/Zn 12c
3	Fe/Zn 25c[3)]	Fe/Zn 25c[3)]

1) In permanently heated buildings without artificial humidifying.

2) Only for nails with $d \leqslant 2.8$ mm.

3) Under severe conditions: Fe/Zn 40c or Hot dip zink coatings.

: The consideration for the finish of the structures may call for stricter rules for corrosion protection, especially
: in climate class 2. Attention is drawn to the fact that certain woods, e.g. oak, and some treatments may have
: a corroding effect, and other protection could be called for.

5.1.0 Characteristic values

Addition:

In Table 5.1.0a a column on SC 38 will be added (the values will be the same as for SC 38 in Table 5.2.0a).

Change:

assumed to be

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

Change:

and

5.1.1 Tension members, compression members, beams[2)] ~~and columns~~

Addition:

2) All stresses and strength values in this chapter are design values, cf. Chapter 1.4 and 3.3.

5.1.1.0 General

Change:

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary for nails ~~and screws~~ with a diameter of 5 mm or less, and without predrilling.

5.1.1.7 Compression and bending with column effect

Change:

On Fig. 5.1.1.7 »E_0, f_{c0}» should be »E_0/f_{c0}».

Change:

Table 5.1.1.7 Relative effective length of compression members

Condition of end restraint	ℓ_c/ℓ
Restrained at both ends in position and against rotation	0.7
Restrained at both ends in position and one end against rotation	0.85
Restrained at both ends in position but not against rotation	1.0
Restrained at one end in position and against rotation and at the other end against rotation but not in position	1.5
Restrained at one end in position and against rotation and free at the other end	~~2.5~~ ≧2

5.2.0 Characteristic strength and stiffness values

Change:

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

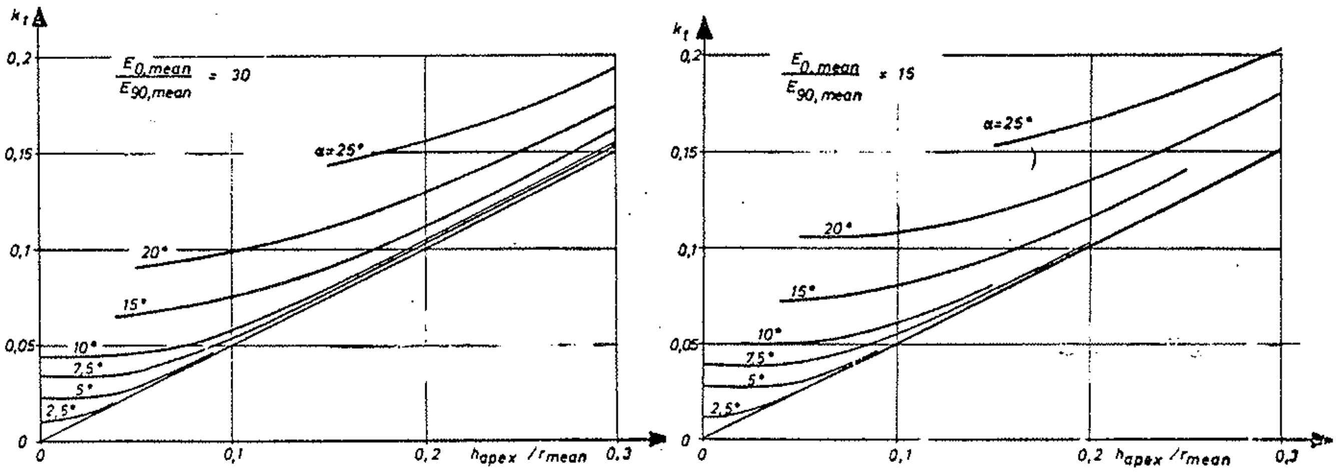
5.2.2 ~~Cambered~~ Tapered beams

Change:

$$k_{size,90} = \begin{cases} \dfrac{\cancel{0.5}}{V^{0.2}} & \text{for uniformly distributed load} \quad \to 2.0 \\ \dfrac{\cancel{0.85}}{V^{0.2}} & \text{for other loading} \quad \to 1.4 \end{cases} \qquad (5.2.2\ b)$$

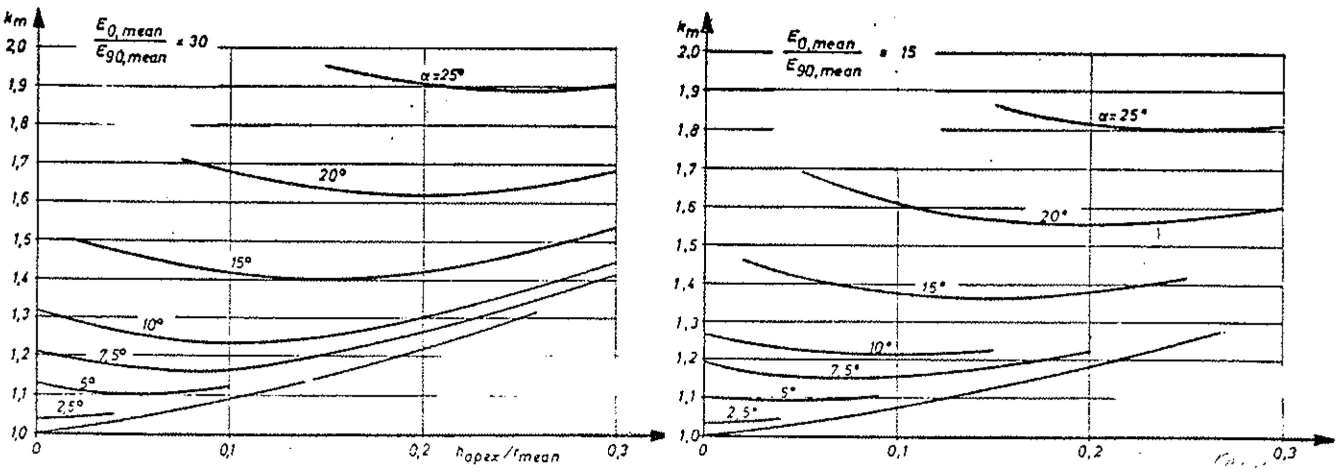
Change:

Figure 5.2.2.c:



Change:

Figure 5.2.2.d:



5.2.3 Curved beams

Change:

$$k_{size,90} = \begin{cases} \dfrac{\cancel{0.4}}{V^{0.2}} & \text{for uniformly distributed load} \quad 1.6 \\ \dfrac{\cancel{0.3}}{V^{0.2}} & \text{for other loading} \quad 1.2 \end{cases} \qquad (5.2.3\ e)$$

6.1.0 Load-carrying capacities, general

Change:

: In cases where the slip is important lower values should be used.
: Attention is drawn to the fact that certain fasteners, e.g. nails, bolts without connectors and bolts with split ring
: or shear-plate connectors, have only inferior strength and will reveal great slip when exposed to heavy stresses with
: frequently alternating directions or vibrating load

6.1.1.1 Laterally loaded nails

Timber-to-timber joints

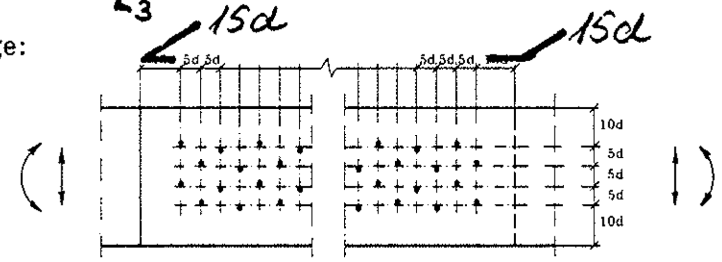
Addition:

The slip at serviceability load is about 0.1d.

Change:

: If ~~ℓ~~ 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.

Change:



Steel-to-timber joints

Addition:

No staggering is envisaged.

6.1.1.2 Axially loaded nails

Change:

~~The length of the point is denoted ℓ_p.~~ $\ell \geqq 4d$ is assumed.

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

6.1.1.3 Staples

Change:

The general rules for nailed joints apply with the two staple legs acting as two nails with the same diameter. For timber under the crown the angle between the crown and the grain direction should not be less than 30°.

6.1.2 Bolts and dowels

Addition:

Bolts

Change:

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).

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

Addition:

The slip at serviceability load is about 0.1d + 1 mm.

Change:

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~~ d):

$$F = \min \begin{cases} 6.5(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2\ \cancel{g)}\ f \\ 12.5 k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2\ \cancel{h)}\ g \\ 25 k_1 t_1 d & (6.1.2\ \cancel{i)}\ h \\ \cancel{26d^2 + 1.5 k_1 t_1 d} & \cancel{(6.1.2\ j)} \\ 45d^2 \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.2\ \cancel{k)}\ i \end{cases}$$

Addition:

Dowels

The rules for bolted joints apply, but the load-carrying capacities for bolted joints may be multiplied by 1.25.

The slip at serviceability load is about 0.1 d.

6.1.3.1 Laterally loaded screws

Timber to timber

Change:

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

Addition:

The slip at serviceability load is about 0.1 d.

6.1.3.2 Withdrawal loads of screws

Change:

$$F = \cancel{(f_0 + fd)}\, f_{screw} (\ell_t - d) \qquad (6.1.3.2\ a)$$

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~~ f_{screw}	~~and~~ is a parameter~~s~~ dependent on among other things the shape of the screw and timber species and grade.

6.1.4 Connectors

Change:

The characteristic load-carrying capacity and deformation characteristics of joints with connectors are in general determined by testing.

By the testing consideration should be given to the influence of among other things:

- The angle between force direction and grain.
- The diameter of bolts or screws.
- The dimensions of the members.
- The mutual distances and distances to end and edge.
- The manufacturing conditions.

When a load is applied at an angle to the grain direction it should be shown that the condition (6.1.2 f) is satisfied. In this case b_e is the distance from loaded edge to farthest edge of the connectors, see fig. 6.1.4.

The rules for bolts should be complied with and the minimum distances between connectors should be sufficient to prevent splitting or shearing of the timber.

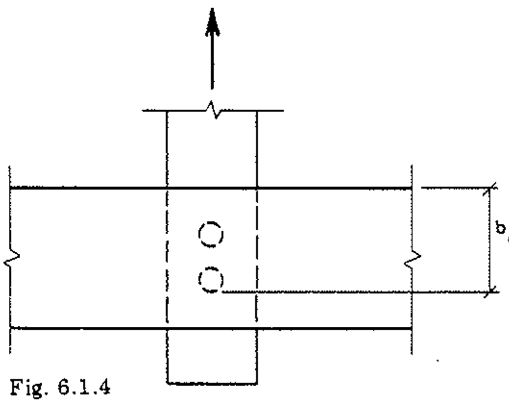


Fig. 6.1.4

If a connector is to be used with several bolt diameters the investigation should comprise at least maximum and minimum bolt diameter and it can be assumed that the load-carrying capacity of the joint F_{joint} is

$$F_{joint} = F_{bolt} + F_{conn}$$

F_{bolt} is the load-carrying capacity of the bolt (or screw) calculated as stated in 6.1.2 (or 6.1.3), and F_{conn} is the contribution from the connector.

7.1.1 Thin-webbed beams

Change:

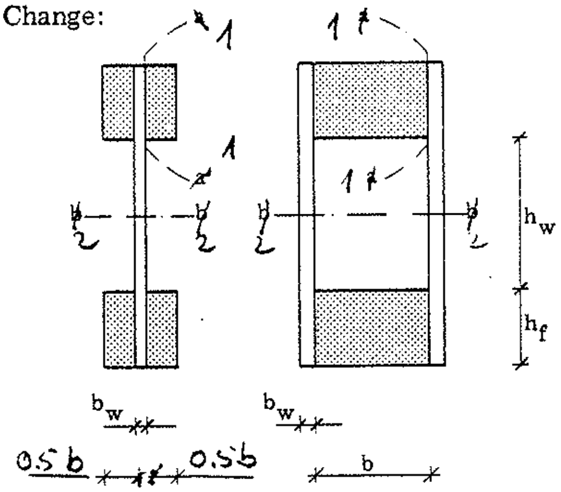


Fig. 7.1.1 a

Change:

The shear stresses may be assumed uniformly distributed over the width of the sections ~~a-a~~ 1-1 and ~~b-b~~ 2-2 shown in fig. 7.1.1 a.

7.1.2 Thin-flanged beams (stiffened plates)

Change:

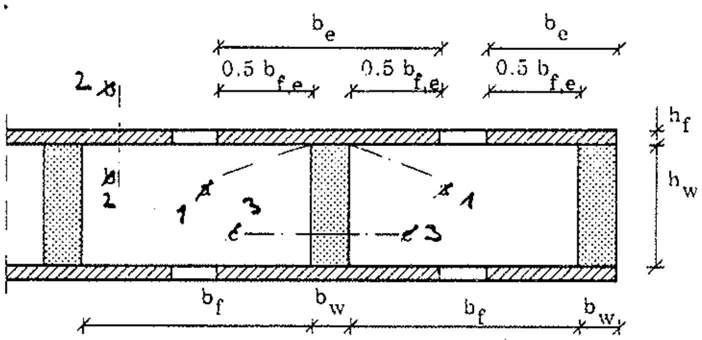


Fig. 7.1.2

Change:

The shear stresses may be assumed uniformly distributed over the width of the sections ~~a-a~~ 1-1, ~~b-b~~ 2-2 and ~~c-c~~ 3-3 shown in fig. 7.1.2.

7.2 Mechanically jointed components

Addition:

The values in table 7.2 are provisional.

8.3 Joints

Change:

~~At least 2 dowels should be used in a joint.~~ The minimum dowel diameter is 8 mm. The tolerances on the dowel diameter are $-0/+\ 0.1$ mm and the pre-bored holes in timber members should have the same diameter as the dowel. ~~The dowels should be at least 2d longer than the total thickness of the joint.~~

Change:

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.

CIB-W18/13-100-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB STRUCTURAL TIMBER DESIGN CODE

Chapter 9: Performance in Fire

OTANIEMI, FINLAND
JUNE 1980

9 PERFORMANCE IN FIRE

9.0 General

The recommendations in this chapter give methods of assessing the performance of timber members in fire.

For the purposes of this chapter the definition of 'stability' shall be taken as:

- a the ability to sustain the applied load throughout the period of the fire test, and
- b in the case of flexural members, the ability to restrict deflection during the fire test to one-thirtieth of the span.

Charring may be assumed to occur at a steady rate in the fire resistance test described in IS 0000 and the timber beneath the charred layer may be assumed to retain its original strength. These characteristics make it possible to predict the performance of timber components in fire, reducing the need for testing.

The stability criteria of IS 0000 are applicable to structural timber components. Where members are built into, or form part of, a fire resisting construction, the insulation and integrity requirements of that standard may also be applicable.

9.1 Rates of charring

9.1.1 Fire retardants

Timber treatments, including impregnation to retard the surface spread of flame, should not be assumed to affect the charring rate.

9.1.2 Solid members

Calculation of the residual section of solid members should be based on the values given in Table 9.1. These values should be modified in the case of fully exposed columns and tension members as set out in 9.2.2a and 9.2.3.2a respectively.

Table 9.1 Notional rate of charring for the calculation of residual section

Species	Charring in 30 min	Charring in 60 min
(a) Western red cedar	25	50
(b) Other softwoods	20	40
(c) Oak, utile, keruing (gurjun) teak, greenheart, jarrah	15	30

Note: Linear interpolation or extrapolation is permissible for periods between 15 min and 90 min.

: Notional charring rates for particular species and for longer periods of time may be established.

9.1.3 Glued laminated members

The charring rates given in 9.1.1 may be applied to members laminated with the following thermosetting phenolic and aminoplastic synthetic resin adhesives: resorcinol-formaldehyde, urea-formaldehyde, and urea-melamine-formaldehyde.

: When other adhesives are to be used their performance in fire should be verified by tests.

9.1.4 Finger joints

Finger joints manufactured using the adhesives given in 9.1.3 may be considered to char at the rates given in Table 9.1.

9.1.5 Sections built up with metal fasteners (see Fig 9.2)

The charring rates in 9.1.1 may only be applied to the section as a whole if metal fasteners on which the structural performance of the built-up member depends are fully protected from the effects of fire (see 9.2.4.2).
Where such protection is not given, local structural weaknesses may occur and the member can only be assessed for fire performance by applying the residual section calculation (assuming charring on all faces of each component of the built-up member) or by conducting a fire resistance test.

9.1.6 Increased rate of charring on exposed arrises

Arrises will become progressively rounded during fire exposure. The radius of this rounding will be equal to the depth of the charring and the centre will lie equidistant from the two aspect faces at a distance of twice the charring depth (see Fig 9.1).

For periods of exposure not exceeding 30 min, where the least dimension of the residual section is not less than 50 mm, rounding is insignificant and may be disregarded.

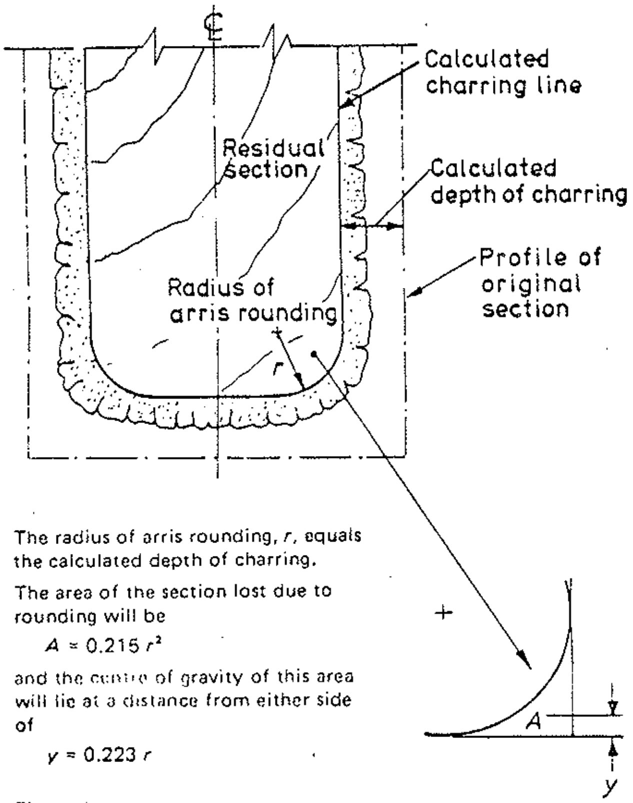


Fig 9.1 Radius of arris rounding

9.2 Design requirements

9.2.1 Flexural members

9.2.1.1 *Stability criteria*

'(a) Strength. The residual section should be such that the member will support the appropriate loads that would be applied if the component were tested in accordance with the requirements of IS 0000 to either the maximum permissible design load or the loads which the member is required to support in normal service'.

(b) Deflection. The deflection under the appropriate design load should not exceed 1/30 of the clear span. Consideration should be given to the effect of deflection on the stability and integrity of other parts of the structure.

9.2.1.2 *Assessment of fire resistance*

(a) Residual section. The residual section should be computed by subtracting from the appropriate faces the notional amount of charring assumed to occur during the required period of fire exposure, making allowance for the rounding on the exposed arrises, where necessary.

(b) Strength. The load-bearing capacity of a flexural member should be calculated in accordance with normal practice, using the residual section and characteristic stresses given in Chapter 4 when the minimum initial breadth of the section is 70 mm or greater; and 0.9 times the characteristic stress when this dimension is less than 70 mm.

(c) Deflection. Deflections should be calculated using the residual section and the dry value of the modulus of elasticity, taking the mean or minimum values, as used in the original design. The resulting deflection should not exceed the limit defined in 9.2.1.1.

9.2.2 Compression members

9.2.2.1 *Stability criterion*

The residual section should be such that the member will support the appropriate loads such as would be applied if the component were tested in accordance with the requirements of IS 0000 to either the maximum design compressive load or loads based on those which the member is required to support in normal service.

9.2.2.2 *Assessment of fire resistance*

(a) A column that is exposed to the fire on all faces (including a column which abuts on or forms part of a wall that does not have fire resistance, as in Figs 9.3b and 9.4b) should be assumed to char equally on all faces during the whole period of fire exposures. To determine the residual section of such columns, the rates of charring given in Table 9.1 should be multiplied by 1.25.

Where a column abuts on or forms part of a wall which provides fire resistance from either side not less than that of the column, charring on all faces is unlikely. Calculations should therefore be based on charring of the column occurring on the side of the wall on which the column has the greater surface exposure, using the rates of charring given in Table 9.1 (see Figs 9.3a and 9.4a).

Care should be taken to ensure that the junctions between the wall and the column will be adequate as a barrier to fire so that the integrity of the construction is unimpaired.

Where a column abuts on or forms part of a wall, which is required to provide fire resistance from one side only (such as in an external wall) and which has fire resistance not less than the column, charring on the faces of the column which can be exposed to fire need only be considered and the rates of charring given in Table 9.1 should be used. In establishing the vulnerable column faces, due regard should be given to the protection afforded by the walling materials.

Care should be taken to ensure that the junctions between the wall and the column will be adequate as a barrier to fire so that the integrity of the construction is unimpaired.

(b) No angular restraint at the ends (as distinct from positional restraint) should be assumed in determining the effective length of residual column sections unless consideration of the residual joint (as indicated in 9.2.4) shows that a degree of restraint would be provided.

(c) The maximum slenderness ratio based on the residual section should not exceed 200 and the stress modification factor for long-term loading for the slenderness ratio of the residual column should be derived from clause 5.1.1.7.

(d) The strength of a compression member should be calculated in accordance with 5.1.1.2 or 5.1.1.7, as appropriate, using the residual section and the appropriate characteristic stresses or the stress derived in 9.2.1.2b.

9.2.3 Tension members

9.2.3.1 *Stability criteria*

The residual section should be such that the member will support the appropriate loads.

9.2.3.2 *Assessment of fire resistance*

(a) To determine the residual section of a tension member the rates of charring given in Table 9.1 should be multiplied by 1.25.

(b) The load-bearing capacity of a tension member should be calculated in accordance with normal practice using the residual section and the characteristic stress.

(c) The load-bearing capacity of a tension member subject to bending should be calculated in accordance with 5.1.1.6 of this code using the permissible stresses derived in 9.2.1.2b and 9.2.3.2b.

9.2.4 Joints

9.2.4.1 *General*

The charring rates given in Table 9.1 may be applied provided that in all cases the faces of the abutting pieces of timber are held in close contact and that special attention is paid to the placement or protection of metal fasteners and components (see 9.2.4.2 and 9.2.4.3).

The methods of calculation given previously are directly applicable to the performance of individual flexural, tension and/or compression members. Junctions between members may be particularly vulnerable to the effects of fire and require special consideration. Where a compressive force is transferred by direct timber-to-timber bearing, the loss in strength of the joint is unlikely to be significant where members have been designed in accordance with the recommendations of this code.

However, where a structure is designed to have joints that transfer forces from one member to another, special account should be taken of the behaviour of such joints. An assessment should be made of the residual timber after the specified period, with particular attention to the effects of any metal connectors and the probability of rounding at abutting arrises (as indicated in 9.1.6). In redundant structures, charring may alter the relative stiffness of various parts of the structure and result in a redistribution of forces, and account should be taken of complete or partial yielding of the joints as this may change the structural action. The structure with redistributed forces should be assessed for fire resistance as detailed in 9.2.1, 9.2.2 and 9.2.3.

9.2.4.2 *Metal fasteners*
Where any part of a nail, screw or bolt becomes exposed to heating during a fire, rapid heat conduction will lead to localized charring and loss of anchorage. Where this effect is likely to lead to the failure of a structural member which is required to have fire resistance, protection of the fastener should be provided by any one of the following methods.

a Ensuring that every part of the fastener is embedded in the timber so that it remains within the residual section as shown in Fig 9.2. Any holes should be fully and securely plugged with timber glued in position. Advice on the use of alternative plugging materials should be sought from an appropriate authority.

b Covering the exposed part of the fastener with a suitable protecting material, eg timber, plasterboard, asbestos insulation board, or equivalent. Special attention should be paid to the fixing of such protection to ensure that it remains in position for the required period of fire resistance. Nails, screws or staples may be used in this case to fix this insulation.

c Any appropriate combination of the methods outlined in (a) and (b).

9.2.4.3 *Steel hangers for joists or beams*
Where steel hangers are fully protected for the required period of fire resistance either by a ceiling membrane or locally by a protecting material, they will be satisfactory in fire.

For floor construction up to and including 30 min fire resistance, joist hangers of the strap or shoe type formed from 1 mm steel, may be used with ceiling construction which affords 20 min protection, eg 12 mm plasterboard.

For floor construction up to and including 30 min fire resistance, joist hangers of the substantial shoe type with gusset or strap bracing, formed from at least 3 mm steel, may be used without protection.

For 1 h fire resisting floors, a ceiling has to be used affording at least 45 min protection, eg 31 mm plasterboard.

9.2.4.4 *Metal plates and other metal connectors*
Metal connectors and metal connector plates may be used without restriction in trussed rafter construction when no fire resistance requirements exist. When a member incorporating exposed nail plates is required to have fire resistance, the provisions of 9.2.4.2 apply.

When the bolts of other types of metal connectors, eg toothed plates, split rings etc, are likely to become exposed during a fire, additional protection as outlined in 9.2.4.2 should be provided. All other types of joints should be referred to an appropriate authority.

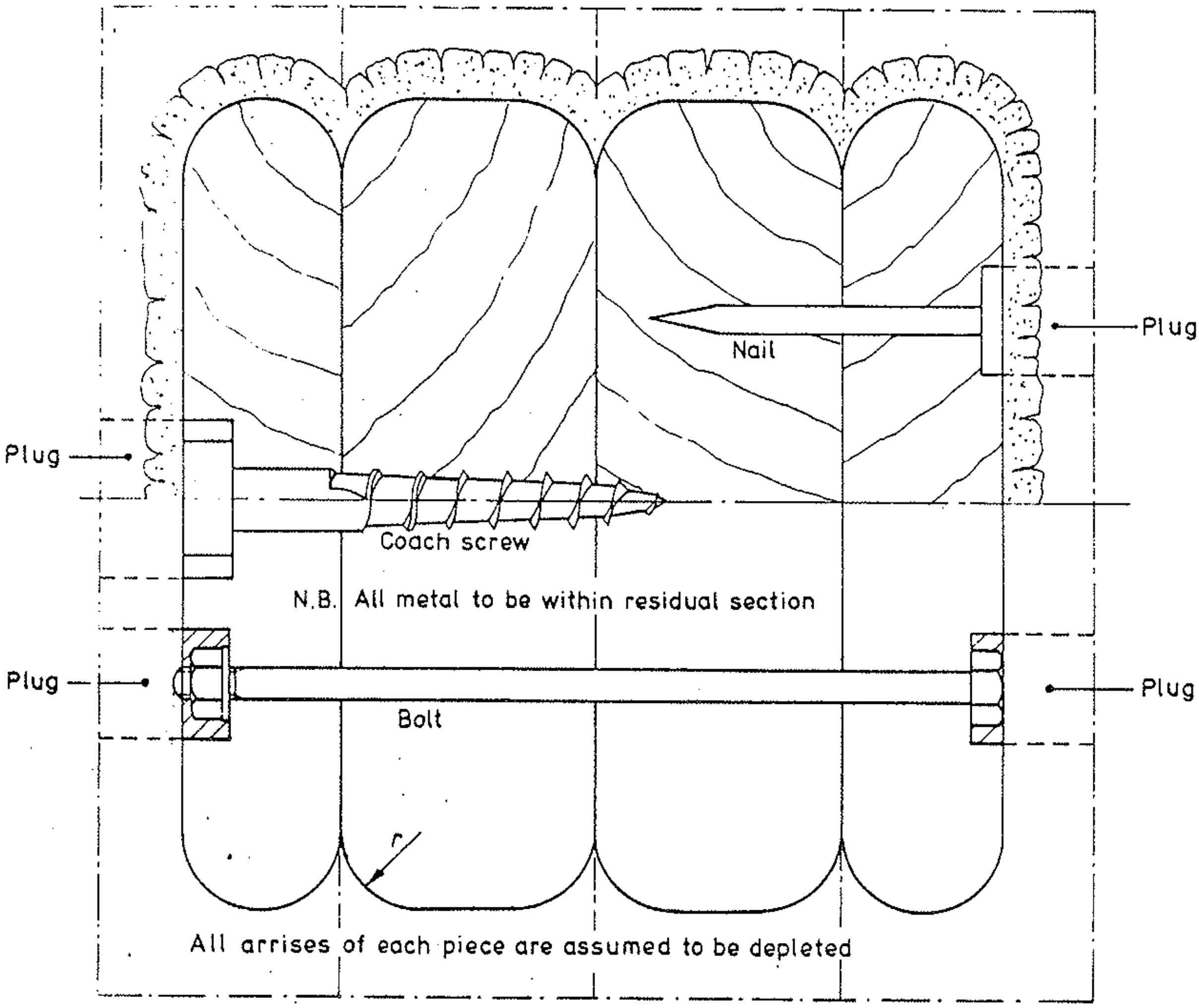


Fig 9.2 Sections built up with metal fasteners

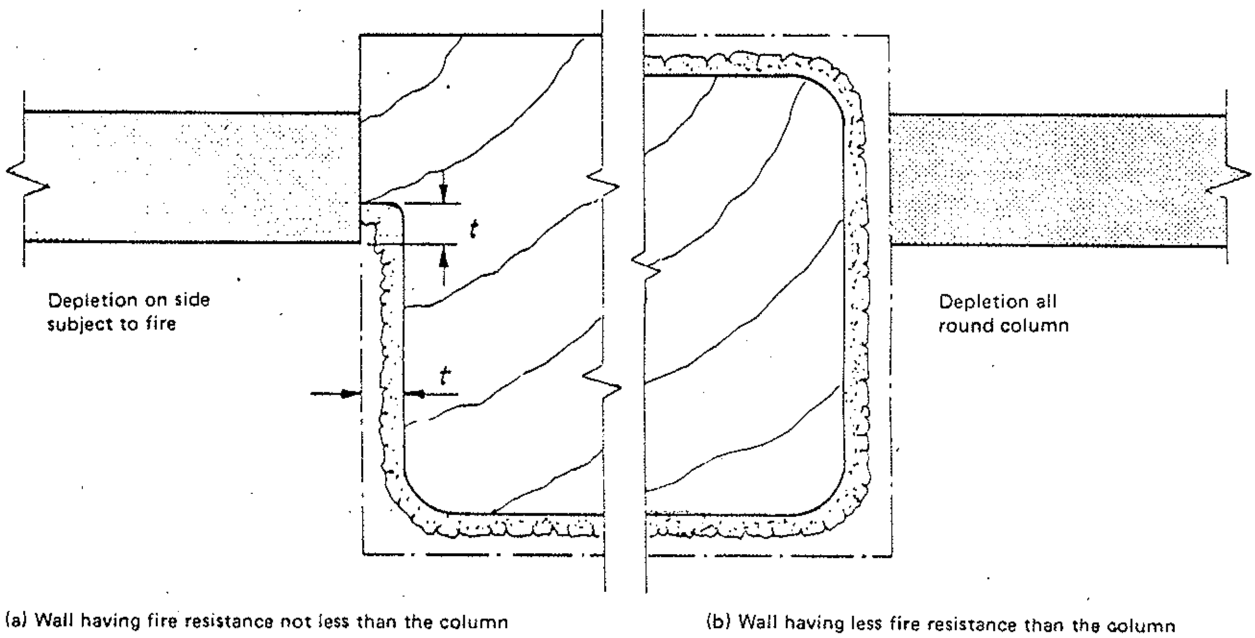
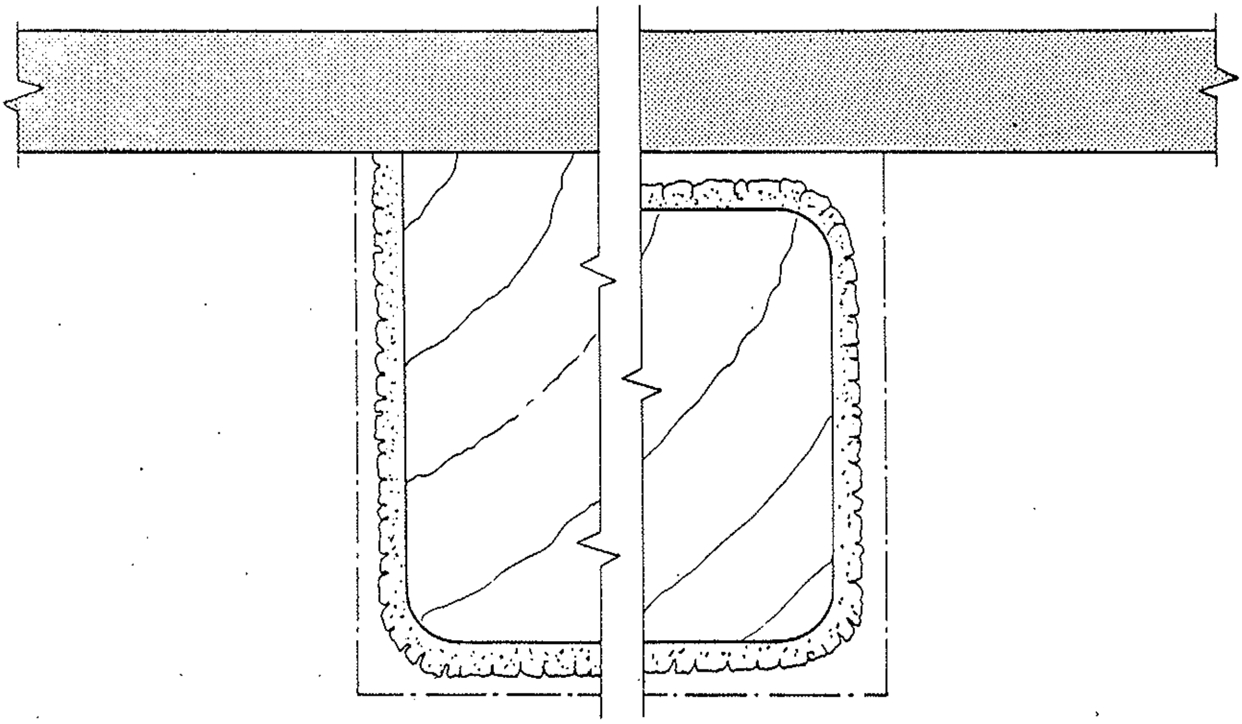


Fig 9.3 Columns built into walls



(a) Wall having fire resistance not less than the column

(b) Wall having less fire resistance than the column

Fig 9.4 Columns abutting on walls

CIB-W18/13-100-3a

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENTS ON CIB STRUCTURAL TIMBER DESIGN CODE

B Bany, Poland
N I Bovim, Norway
J Kuipers, The Netherlands
W Nożyński, Poland
G O Marchi, Italy
L Monfort, Belgium

OTANIEMI, FINLAND
JUNE 1980

Proposals and comments have been received from

B. Bany	(BB)
N. I. Bovim	(NB)
Jan Kuipers	(JK)
W. Nożyński	(WN)
G. O. Marchi	(GM)
L. Monfort	(LM)

In the following the present text (4. draft + agreed changes) is given in the left column and the proposals and comments received in the right column together with my proposals and comments etc., which will hopefully facilitate the discussions. An exception is the proposal from L. Montfort, which is of a general character. The original letter is in the left column, a rough translation into English on the right page. A proposal for a new lay-out of chapters 5.2.2 and 5.2.3 is given in an Annex.

H. J. Larsen

<u>General</u>

<u>WN:</u>

I have a pleasure to send You the titles of subjects which I suggest to enclose in the Document: "CIB structural timber design code":

1. Wood-based materials and related problems of structures design.
2. Directives for designing of structures under serviceability limit state - problem of deformations /deflections/.
3. Determination of buckling lenghts of bar structures /arcs-frameworks-latticeworks/ - buckling factors.
4. Directives for calculation of counter-bracing /strutting/.
5. Designing of reinforced and prestressed timber constructions.
6. Designing of composite construstions /wooden-steel/ and from other materials /trussed beam et cetera/.
7. Directives for calculation of framework corners.
8. Calculation of glued-laminated joints e.g. finger-jointing of solid members.

<u>Comment</u>

To be discussed.

Pref.Ir.L.MONTFORT

Bruxelles II50,3-I-80
30 Corniche verte

conc. TIMBER CODE C.I.B

PROPOSITION POUR UNE DISTRIBUTION PLUS LOGIQUE DES COEFFICIENTS " k "

I- Schéma de vérification

Il n'y a pas de doute quant au schéma de vérification d'un état limite ultime . Les phases sont les suivantes :

I.I- Détermination des charges de calcul (design loads) en partant des valeurs caractéristiques des actions

$F_d = \gamma_f \psi_o F_k$ et leurs combinaisons .

I.2- Calculs à l'aide des formules de la Résistance des matériaux des éléments de réduction M, N, T, M_t et des contraintes correspondantes : $\sigma_c, \sigma_t, \sigma_m, \tau$

I.3- Détermination des résistances de calcul du matériau (design strength) en partant de leurs valeurs caractéristiques

$$f_c, f_t, f_m, f_v = \frac{f_k}{\gamma_m}$$

I.4- Vérification d'inéquations du type :

$$\sigma_c \leq f_c \quad \sigma_t \leq f_t \quad \sigma_m \leq f_m \quad \tau \leq f_v$$

2- Problèmes particuliers au bois

Le problème est de savoir où il faut logiquement faire intervenir ce qui est particulier au bois et ne se retrouve pas dans les autres matériaux : influence de la durée de la charge, influence de l'humidité, influence des dimensions de la section etc..

2.I- En ce qui concerne la durée de la charge et l'influence de l'humidité, j'estime légitime de les faire intervenir dans la fixation des valeurs caractéristiques comme celà a été fait jusqu'ici dans le T.C. C.I.B. ,car les essais sur le bois sont " rapides " et se font sur des échantillons à I5% d'humidité.

Je propose néanmoins ,pour éviter toute confusion,que les valeurs caractéristiques du tabl.5.I.0.a soient appelées " résistances caractéristiques de base" . Par application du tableau 5.I.0.b ,on obtient les "résistances caractéristiques

PROPOSAL FOR A MORE LOGICAL USE OF THE FACTORES "k"

I. Verification sequence

Undoubtedly the verification sequence for a limit state is the following:

I.I Determination of the design loads caused by the characteristic loads

$F_d = \gamma \psi_o F_k$ and their combinations

I.2 Calculation by the formulas from engineering mechanics and the properties of the materials the internal forces M, N, T, M_z and the corresponding stresses σ_c, σ_t, σ_m, τ.

I.3 Determination of the design strengths of the materials derived from their characteristic values

$$f_c, f_t, f_m, f_v = \frac{f_k}{\gamma_m}$$

I.4 Verification of inequation of the form

$\sigma_c < f_t$ ....

2. Special problems in timber

The problem is to decide where phenomena not found for other materials but only for wood should be taken into account: The effect of load duration and moisture content, size effects, etc.

2.1 As regards the time and moisture effect I find it justified to take them into account by the definition of characteristic values as done in the present draft of CIB Timber Code because the test is quick and made at a moisture content of 15 pct.

All the same I propose to avoid any cofusion to call the characteristic values in Table 5.1.0 a "basic" characteristic values. By application of Table 5.1.0.b design characteristic values are

de calcul" qui sont à comparer aux contraintes σ et τ .
Je suis en outre partisan de séparer dans le tabl.5.I.O.b. l'influence de la durée et l'influence du climat .

2.2- Mais les considération de forme et de dimension de la section comme aussi ce qui concerne le flambage devraient intervenir dans le calcul des contraintes σ et τ (phase I.2 du schéma) et ne devraient pas affecter les résistances de calcul f_c, f_t, f_m, f_v (phase I.3 du schéma) ,comme celà a été fait jusqu'ici dans le T.C. C.I.B.
Il s'agit en effet de corriger,en fonction du comportement du bois,les contraintes issues de la résistance des matériaux classique basées sur des hypothèses d'homogénéité et d'isotropie qui ne sont pas vérifiées pour le bois. Quant au flambage, c'est aussi un problème de calcul des contraintes.
J'estime donc qu'il ne faudrait pas écrire au §.5.I.I.I

$$\sigma_t \leq k_{size} \cdot f_t$$

mais $$\sigma_t = \frac{N}{A \cdot k_{size}}$$

de sorte que la vérification soit toujours $\sigma_t \leq f_t$.
De même,au §.5.I.I.2[a] on devrait écrire $\sigma_c = \dfrac{N}{A \cdot k_{bearing}} \leq f_c$

et au §.5.I.I.2[b] : $\sigma_m = \dfrac{M}{J \cdot k_{depth} \cdot k_{inst}} \leq f_m$ etc...

Cette façon de procéder est d'ailleurs traditionnelle dans la vérification au flambage puisqu'on a toujours écrit :

$\dfrac{N}{\varphi A} \leq \sigma'_{adm}$ (en Belgique avec $\varphi > 1$)

$\omega \cdot \dfrac{N}{A} \leq \sigma'_{adm}$ (en Allemagne avec $\omega < 1$)

la inte admissible σ'_{adm} étant la même qu'en l'absence de flambage.

3- Ces propositions de modification de la présentation des formules se justifient par la logique,par le souci d'éviter des confusions et d'assurer un parallélisme dans les calculs relatifs aux divers matériaux.

obtained. I further advocate to separate in Table 5.1.0 b the factor for load duration and moisture class.

2.2 On the other hand, the effect from form and size and also the effects of fire should be taken into account by the stresses (phase I.2 in the sequence) and not as done at present in the CIB Timber Code by modifications of the design strengths f_c, f_t, f_m, f_v (phase I.3 in the sequence).

What should really be done is taking into account the properties of timber by modifying the stresses calculated by the classical theory of elasticity for homogenous, isotropic material, a theory not verified for timber. In the case of fire there are further problems in calculation of the stresses.

I'm therefore of the opinion that in 5.1.1.1

$$\sigma_t < k_{size}\, f_t$$

should be replaced by $\sigma_t = \dfrac{N}{A\, k_{size}}$

Equally 5.1.1.2 a should be written as $\sigma_c = \dfrac{N}{A \cdot k_{bearing}} < f_c$

and 5.1.1.2 b : $\sigma_m = \dfrac{M}{W \cdot k_{depth} \cdot k_{inst}} < f_m$, etc.

This is by the way the same format as is traditionally used for colums, where we have always written

$\dfrac{N}{\rho A} < \sigma'_{adm}$ (In Belgium, with $\rho > 1$)

$\omega \dfrac{N}{A} < \sigma'_{adm}$ (In Germany with $\omega < 1$)

The permissible stres σ'_{adm} is the same as in the case without column effect.

3. The proposal for a change in the presentation of the formula is logical, will avoid confusion and ensure harmony with the design of the various other materials.

Draft code	Comments
FOREWORD **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. **1.5 Definitions** (will be prepared at completion of the work)	JK: proposed amendment The draftcode makes reference to other documents which have been submitted to ISO/TC165 for comment. They were prepared by the joint committee RILEM/CIB-3TT: Testing methods for Timber, and published after acceptance by CIB-W18 as Recommendations in "Materials and Structures". These are: . Timber Structures - Joints ..... Fasteners; 3TT-1, Mat.&Struct. Vol.12 no . Timber Structures - Plywood .... Properties; Draft 3TT-2. Mat & Struct. Vol. 11 no 62 . Timber Structures - T. in Struct.Sizes ..... Properties; 3TT-3, Mat. & Struct. Vol. 11 no 66 Other documents relating to test methods will be prepared by the same or other joint RILEM/CIB committees; documents relating to the sampling of test specimens ..... Propsosal: Accept.
2.2 Climate classes *Climate class 1* The climate class is characterized by a moisture content in the materials corresponding to a temperature of 20 ± 3°C and a relative humidity of the surrounding air never exceeding 0.80 and only exceptionally, and then only for short periods (less than a week), exceeding 0.65. The following structures can be included in this class: - structures in outer walls in permanently heated buildings where the structures are protected by a well-ventilated tight cladding.	JK: As an example of Climate Class 1 "structures in outer walls in permanently heated buildings ..." are mentioned. Is this correct? or are all structures in centrally (not permanently!) heated buildings (not only the studs etc. in outer walls) included? Proposal: Delete the comments. BB: The name "Climate classes" will be changed to "Moisture classes" /loock point.2.2./ It would be better to provide the same changes to tables 4.7 and 5.1.0.b. Comment: Yes

3.0 General

WN: I have some doubts concerning chapter 3: Does it concern timber structures or reinforced concrete and steel structures too especially as far as coefficients values are concerned?
If it is true, they shall be enclosed in a separate standard for limit states concepts.

Comment: Chapter 3 is almost entirely outside the scope of CIB-W18 and it is in fact not necessary for the rest of the code.
Hopefully ISO or other international bodies will soon produce agreed basic documents. They will, however, not contain numerical values of the safety factors, the determination of these is left to the governments.

Change:

The main structure should normally be designed in such a way that it when exposed to an accidental load should not subsequently be damaged to an extent disproportionate to the extent of the original incident. This requirement may be achieved by

a) designing the structure in such a way that if any single load-bearing member due to an accidental load becomes incapable of carrying load this will not cause collapse of the whole structure or any significant part of it, or

b) where necessary, ensuring (by design or by protective measures) that no essential load-bearing member can be made ineffective as a result of an accidental load

JK: We doubt if the proposed change has the intended effect, i.e. that it remains doubtful if for instance a tied arch can be made. The introduction of the accidental load even seems to strengthen the requirement. Because we have no better formulation our proposal is to delete this requirement.

Comment: As above, most tied arches are covered by b).

3.1.2 Serviceability limit states

Serviceability limit states are related to the criteria governing normal use.

Serviceability limit states may for example correspond to:

- local buckling of thin plates (for example in thin webs or flanges) without rupture.

JK: Is it necessary to forbid here local buckling without rupture if the efficient use of a structure or its appearance will not be affected?
Proposal: delete this example.

Comment: Most codes at present regard buckling as an ultimate limit state. The example is included to draw attention to the possibility of a more varied treatment.

4.1 Solid structural timber

4.1.0 General

Strength and stiffness parameters shall be determined by standardized short-term tests in accordance with ISO/TC 165: Timber structures - Timber in structural sizes - Determination of some physical and mechanical properties. The test specimens shall contain a grade-determining defect - preferably knots - in the zone with maximum force or bending moment.

JK: The last sentence about a grade-determining defect occurring in the zone with max. force or moment belongs to a (CIB-?) document about sampling. It should be deleted here.

Comment: It can be deleted when the sampling paper is produced, until then it is necessary to define the basis of the values unambiguously.

4.3.1 Standard glulam strength classes[1)]

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

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

BB: I mean, that the users of this code must have some more informations about the values given in table 4.3.1.
It is very good that the glulam members will subject the strength classification, but what about the sizes of test pieces for determination the main propertios of glulam members.
It would be good to put a short information that this tests should be taken according the CIB - RILEM recomendations given in standart.
"Timber in structural sizes. Determination of some physical and mechanical properties.

Comment: Is a separate standard needed ?

4.1.1 Standard strength classes[1)]

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

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

JK: The last sentence above the table 4.1.1.: "For maching stress-rated timber it should further be shown that the characteristic tensile strength, ft,o, is not less than given in the table" is not clear and needs explanation. Why is this extra requirement needed and not in the case of visual grading? Is there any evidence that visual grading gives a better correlation than machine grading with tensile strength?

Comment: It is a general experience that f_t/f_m is lower for machine-stress graded than for visually graded timber.

4.4 Wood-based sheet materials

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

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

JK: As long as this ISO document does not exist reference could be made to the Recommendation published in Materials and Structures.

Comment: Yes, cf Foreword

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

		SC15	SC19	SC24	SC30
characteristic values (for strength calculations)					
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

JK: Reference is made to our comment "Annex 2 to our letter 1979-09-10" incorporated in [illegible], especially to point c) and d). This last item could be formulated more clearly, maybe, as follows:

d) In table 5.1.0.a two groups of E-values have been given, for strength and for deformation calculations resp. This splitting up is only acceptable if it is related to corresponding calculation theories or models; the differences in table 5 however are not based on the application of two such theories.

In the case of imposed deformations the resulting stresses will follow from the strains and the E-value: a too low value of E may result in an unsafe structure.

Although it is realized that, for instance for buckling calculations, there is a need for a "low" E-value this can be reached by other means. In NEN 3852 we calculate the deformation of a column with initial curvature U_0 as

$$U = \frac{\frac{\pi^2 E}{3\lambda^2}}{\frac{\pi^2 E}{3\lambda^2} - 1{,}2\, F/A} = U_0.$$

The introduction of $\frac{E}{3}$ takes into account both a characteristic E-value ($= {}^2/_3\, E_{mean}$) and a creep factor of $\phi = 1$ ($E_\omega = \frac{E}{1+\phi}$).

Suggestion: delete in the tables 5.10.a and 5.2.0 the additions "(for strength calculations)" and "(for deformation calculations)". On the left side it could be said that: "characteristic values of E and G must/should/could be used for the calculations of the deformation of a single member or for instance for the buckling of a single compression member."

Such an explanation also prevents the calculation of the deformations with low E-values where this is not necessary, for instance for roofs, trusses, etc., where not a single member determines this deformation.

→ Page 8

Proposal: Delete in Table 5.1.0 a and 5.2.0 "(for strength calculations)" and "(for deformation calculations) and change Table 5.1.0 b accordingly, but without adding the explanations to the left hand page.

The rules concerning the use of characteristic values and mean values should be given in connection with chapter 3.

Table 5.1.0 b.

: Attention is drawn to the fact that deformations 2-3 times those calculated with the values given may be expected if green timber is allowed to dry under design load.

JK: Suggestion for the last sentence, below the table:
"Attention is drawn ..... values given must be expected ......".

Comment: Yes

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

JK: In 5.1.1.1.c the equal-sign in $V \leq 0.02\ m^3$ and $V \geq 0.02\ m^3$ should be deleted in one of both.

. One equal-sign in formula 5.1.1.3.b be deleted.

5.1.1.3 Bending

The bending stresses, σ_m, calculated according to the theory of elasticity shall satisfy

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

k_{dept} is a factor (≤ 1) taking into account the reduced strength of deep sections:

JK: . We will not introduce a k_{depth} for solid timber; our largest member height is normally not more than ca. 300 mm, giving $k_{depth} \approx 0.96$.

. We postpone our eventual further comments until we have studied the instability problems more thoroughly.

5.1.1.7 Compression and bending with column effect (columns)

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

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

Table 5.1.1.7 Relative effective length of compression members

Condition of end restraint	ℓ_c/ℓ
Restrained at both ends in position and against rotation	0.7
Restrained at both ends in position and one end against rotation	0.85
Restrained at both ends in position but not against rotation	1.0
Restrained at one end in position and against rotation and at the other end against rotation but not in position	1.5
Restrained at one end in position and against rotation and free at the other end	2.5

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

. Suggestion: The length should be judged according to the theory of elasticity and is therefore dependent on the end conditions of the member.
For straight members the values of l_c can be taken from table 5.1.1.7; a fixed end condition may be reached by gluing if the construction to which the member is glued is stiff too. The actual length of the member is denoted l.

→ Page 9

Table 5.1.1.7 Relative effective length l_{ef}/l of compression members

character of end restraints	measure of rotational end restraint	
	fixed	mechanical fasteners
....	0,5	0,7
....	0,7	0,85
....	1	1
....	1	1,5
....	2	$\geq$ 2

. Remark: some guide-lines for the effective length of more complex structures could be easy; see DIN 1052 chapter 7 or NEN 3852 - 4.5.

Proposal: **Leave out § 1 i.e. give the general principles without mentioning the theory of elasticity. Amend the rest as proposed.**

Comment: **It is not that easy. The rules in DIN and NEN are at best dubious often completely misleading.**

5.1.2 Tapered beams

For a tapered beam with rectangular cross-section with the grain direction parallel to one of the contours and $\alpha <$ 20° the bending stresses in the outermost fibres can be calculated as

$$\sigma_{m,0} = (1 + 3.7\tan^2\alpha)\frac{6M}{bh^2} \quad (5.1.2\ a)$$

$$\sigma_{m,\alpha} = (1 - 3.7\tan^2\alpha)\frac{6M}{bh^2} \quad (5.1.2\ b)$$

The stresses should satisfy the following conditions

$$\sigma_{m,0} < f_m \quad (5.1.2\ c)$$

$$\sigma_{m,\alpha} < \frac{f_m}{\sqrt{1 + (\frac{f_m}{f_v}\tan\alpha)^2 + (\frac{f_m}{f_{90}}\tan^2\alpha)^2}} \quad (5.1.2\ d)$$

where $f_{90} = f_{t,90}$ or $f_{90} = f_{c,90}$ depending on the sign of the stresses.

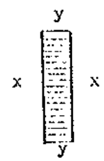
JK: Suggestion: give the formula for $5^0 \leq \alpha \leq 20^0$

5.2 Glued laminated members

5.2.0 Characteristic strength and stiffness values

. See comments to 5.1.0

. For laminated members it can be expected that the mean value of E is the same as for the wood from which it was made; the standard deviation however will be reduced and a characteristic value be therefore higher than for solid timber. This may especially be of importance for buckling calculations of laminated structures about the y-y-axes.

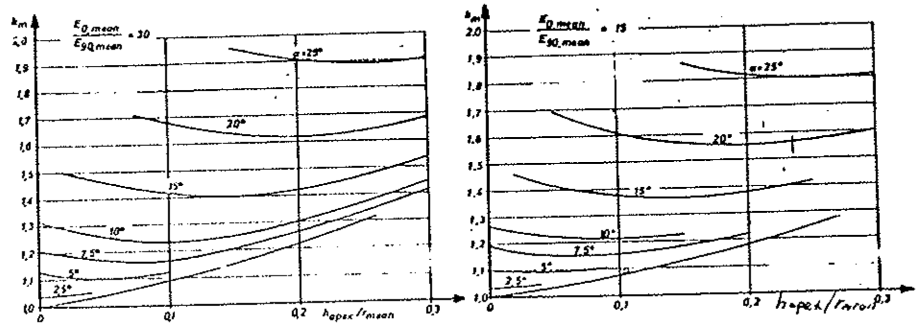


Comment: **Yes, probably**

5.2.2 Cambered beams

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

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



: 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.

JK: We suggest to split up the clause as follows:

5.2.2 Tapered beams

5.2.2.0 General

This section applies ..... etc.

5.2.2.1 Tensile stresses

The influence of ..... etc.

including fig. 5.2.2.d

5.2.2.2 Deflection

In deflection calculations .....

Comments: **No objections.**

A more comprehensive proposal for a revision of the lay out of chapter 5.2.2 and 5.2.3 is given as an Annex.

WN: to the clause 5.2.2. it seems necessary to give formulae for the curves on drawings enclosed,

Comments: **Yes, if possible.**

5.2.3 Curved beams

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

We suggest to split up the clause:

5.2.3.0 General

This section applies .....

5.2.3.1 Reduction of strength

The ratio .....

5.2.3.2 Distribution of bending stresses

In heavily curved beams .....

Comments: **No objections.**

A more comprehensive proposal is given as an Annex.

6. JOINTS

6.0 General

. . .

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

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

JK: . Suggestion:

6.1.1. Combination of different types of fasteners.

6.1.1.0 In a joint normally only one type of fastener should be used. If this is not possible only maximum 30% of the load on the joint may be taken by a second type. The slip characteristics of this second type of fastener must be of sufficient resemblan with that of the main fastener-type to ensure combined action. Joined action of glue and mechanical fasteners normally cannot be expected.

6.1.1.1 The strengthening fastneners should be designed for 1,5 times the load that they are supposed to bear.

Comments: **Are there any reasons for having two restrictions (max 30%, 1,5 times)?**

6.1.0 Load-carrying capacities, general

: In cases where the slip is important lower values should be used.

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

Where nothing else is stated the load-carrying capacities for the load-duration classes and climate classes defined in sections 2.2 and 2.3 are found by multiplication by the factors given in table 5.1.0 b.

JK: . The first small printed remark: "In cases were the slip is important lower values should be used." should be omitted . Instead slip-values should be added for all fasteners mentioned.

Proposal: **Delete the remark.**

→ Page 12

6.1.1.1 *Laterally loaded nails*

Timber-to-timber joints

The characteristic load-carrying capacity in N per shear plane can be determined by:

$$F = k_{nail} d^{\alpha_{nail}} \quad (6.1.1.1\ a)$$

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

If there are only one or two nails in a joint the values according to formula (6.1.1.1 a) are multiplied by 0.5.

For more than 10 nails in line the load-carrying capacity of the extra nails should be reduced by 1/3.

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

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

$$\alpha_{nail} = 1.7$$
$$k_{nail} = 200\sqrt{\rho} \quad (6.1.1.1\ b)$$

: where ρ is the specific gravity defined in section 2.1. No preboring is assumed.

JK: 6.1.2 Alternating direction of loading. Joints loaded in two directions with extreme values F_1 and F_2 respectively, must be designed for both loads. If the larger of the values $|F_1|$ and $|F_2|$ is less than the smaller one, the joint must be calculated also on a load of 1,3 times this larger value. This complication is not necessary if the alternating load is due to short or seldom occurring loading, such as extreme high wind loading.

Comment: It is proposed to design for 1,5 max $\begin{cases}(F_1)\\(F_2)\end{cases}$?

WN: Using the formula 6.1.1.1.a together with formula 6.1.1.1.b. to excessive load carrying ability are obtained, what J represent on table.

Load carrying ability of nails in N calculated from formula 6.1.1.1.a for different q and for comparison load carrying ability of nails calculated from polish standard PN-73/B-03150.

q \ d	2,8	3,0	3,5	4,0
PN-73/B-03150	370	435	570	720
q = 0,36	691	776	1009	1267
q = 0,4	728	811	1064	1333
q = 0,5	815	907	1189	1491
q = 0,6	892	993	1303	1633

The values calculated so are in 80% - 140% greater than calculated from PN-73/B-03150, which is similar to DIN.

Are these formulae correct ones?

Comment: The CIB-values are characteristic values (short-term, no safety factor). The Polish and German values are permissible (long-term, with safety factor).

→ Page 13

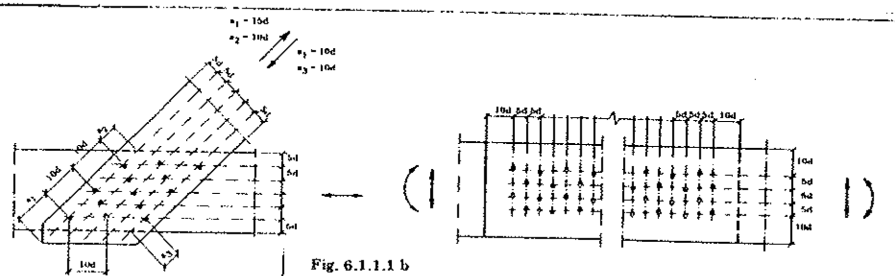


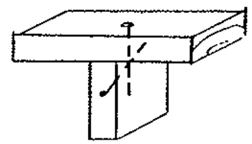
Fig. 6.1.1.1 b

WN: Reduction of load carrying ability of nails in 1/3 when their number in joint exceeds 10 is in my opinion to big. J propose to reduce in 10% to 20 nails, and then in 20% to more than 20.

JK: For more than 10 nails in line we understand that the extra nails load-carrying capacity should be multiplied by 0,67; is this right?

Comment: **The load-carrying capacity of the nails in excess of 10 only is taken as 2/3.**

WN: The frase "Nails in end grain should normally not be considered capable of transmitting force" is not clear for me. Is the meaning: the exterior row of nails in a joint do not transmitt force /for instance/?



Comment: **No. It is nails e.g. as shown.**

JK: . Nails in end grain have to transmit forces, e.q. in timber frame house structures.

Comment: **Are there really strength requirements ?**

JK: . The loaded edge distance should be: $(5 + \frac{\alpha}{20°})d$ for $\alpha < 60°$, and 8d for $\alpha \geq 60°$.

Question: A_4?

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

: Table 6.1.1.2

JK: density → specific gravity?

Answer: Yes.

6.1.2 Bolts and dowels

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

NB: The table can be replaced by

$$k = \frac{k_{90}}{k_{90} \cos^2 v + \sin^2 v}$$

where $k_{90} = 0,45 + 8\, d^{-1,5}$ (d in mm).

JK: A reduction for more bolts in line should be introduced; our formula is $k_n = \frac{6}{n+5}$. Bolts are supposed to be "in line" if $d_{\perp} < 0,5\, d_{//}$.

JK: . In a definition of dowels something has to be said about diameter of predrilling and tolerances.

Comment: Reference is made to 8.3

6.1.4 Connectors

JK: We understand that the national codes can and have to fill up this clause independently, which due to the availability of different types in different places may be the most acceptable way.

7.1 Glued components

7.1.1 Thin-webbed beams

Date:	Chapter:	Page:
79.06.01	Annex 7A	3

3. ASSUMPTIONS

The member is assumed either loaded in the Z-direction giving moments about the Y-axis, or with an axial load acting in the gravity axis.

The case where both bending moment and axial force are acting (beam columns) at the same time is only treated in a special case, cf. section 7.

The conditions are assumed linear-elastic and the following relation between the load on a fastener and the slip is assumed to apply.

$$F = Ku \qquad (1)$$

All members are assumed to have the same modulus of elasticity, but the expressions may be extended to apply also to cross-sections where the cross-section members have different properties by transforming of the cross-section sizes in relation to their stiffness after the usual methods.

WN: Chapter 7 - pages 2 and 3 - in the text following the table it seems necessary to give some cases of application of the text.

JK: In this chapter it should be clear that in some formulas difference must be made for the E and G-values of flanges and webs.

Comment: Yes, it should be.

WN: The textof anclosures should be contained in the standard.

General We had not yet the time to study the annexes thoroughly enough and will proceed our comments later on.

ANNEX

Proposal for a new lay-out of 5.2

Chapter 5.1.2 is transferred to chapter 5.2.2. This means that the number af decimals in the headings of chapter 5.1 can be reduced, i.e. 5.1.1.5 → 5.1.5

Chapter 5.2.2 is replaced by the following chapter 5.2.2 + 5.2.4

Chapter 5.2.3 is maintained.

5.2.2 Tapered beams

This section applies to single tapered beams (Figure 5.2.2a) and double tapered beams (Figure 5.2.2b) with rectangular cross-section.

For the double tapered beams the shear forces are assumed small near the apex.

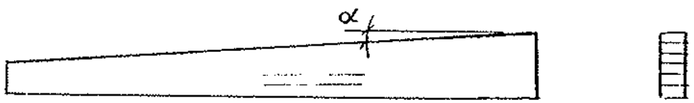


Figure 5.2.2a Tapered beam

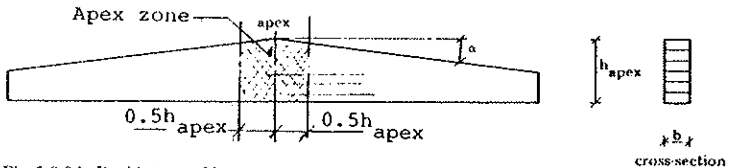


Fig. 5.2.2 b Double tapered beam

For the tapered beams and for the double tapered beams outside of the apex zone the bending stesses in the outermost fibres should satisfy the condition (5.1.1.6a). By determinig the stresses the influence from the taper should be taken into account.

In the apex zone the bending stresses should satisfy the following condition

$$\sigma_m \leq f_m \qquad (5.2.2a)$$

In the apex zone

the tensile stresses perpendicularly to the grain should satisfy the conditions 5.1.1.1 b, i.e.

$$\sigma_t < k^c_{size,90} f_{t,90} \qquad (5.2.2b)$$

with

$$k_{size,90} = \begin{cases} \cancel{\frac{0.5}{\sqrt{0.2}}} \quad \frac{2.2}{(bh^2)^{0.2}} & \text{for uniformly distributed load} \\ \cancel{\frac{0.35}{\sqrt{0.2}}} \quad \frac{1.5}{(bh^2)^{0.2}} & \text{for other loading} \end{cases} \qquad (5.2.2c)$$

$k_{size,90}$ should not be taken greater than corresponding to ~~V = 0.02 m³~~. $bh^2 = 0.03\ m^3$.
(b and h in m)

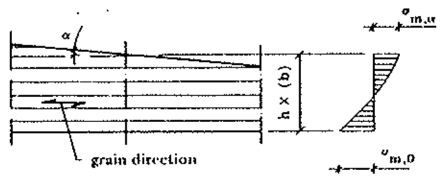


Fig. 5.2.2c

For a tapered beam with rectangular cross-section with the grain direction parallel to one of the contours and $\alpha <$ 20° the bending stresses in the outermost fibres can be calculated as

$$\sigma_{m,0} = (1 + 3.7 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.2.2d)$$

$$\sigma_{m,\alpha} = (1 - 3.7 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.2.2e)$$

The stresses should satisfy the following conditions

$$\sigma_{m,0} < f_m \qquad (5.2.2f)$$

$$\sigma_{m,\alpha} < \frac{f_m}{\sqrt{1 + (\frac{f_m}{f_v} \tan\alpha)^2 + (\frac{f_m}{f_{90}} \tan^2 \alpha)^2}} \qquad (5.2.2g)$$

where $f_{90} = f_{t,90}$ or $f_{90} = f_{c,90}$ depending on the sign of the stresses.

In the apex zone the greatest bending stresses can be calculated as

$$\sigma_m = (1 + 1.4\tan\alpha + 5.4\tan^2\alpha)\ \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2i)$$

and the greatest tensile stresses perpendicularly to the grain as

$$\sigma_t = 0.2\tan\alpha \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.2j)$$

5.2.4 Cambered beams

This section applies to cambered beams as shown in Figure 5.2.4a with rectangular cross-section. The shear forces are assumed small near the apex.

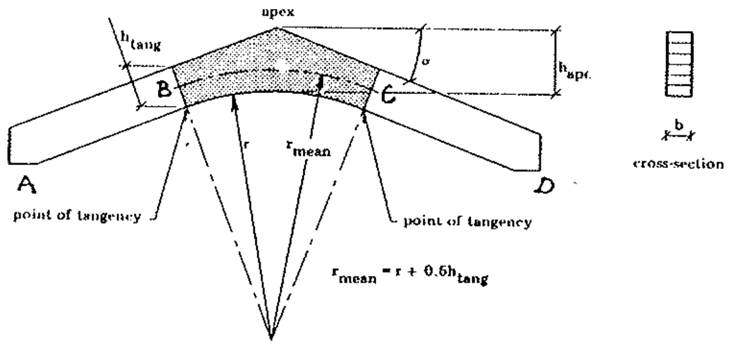


Fig. 5.2.4a Double tapered curved beam

For the straight parts (A-B and C-D) section 5.2.2 applies if the stresses from axial forces are taken into account.
In the apex zone (C-D) the conditions (5.2.2a) - (5.2.2c) apply with

$$k_{size,90} = \begin{cases} \dfrac{\cancel{0.6}}{V^{0.2}} & \text{for uniformly distributed load} \longrightarrow 2.0 \\ \dfrac{\cancel{0.86}}{V^{0.2}} & \text{for other loading} \longrightarrow 1.4 \end{cases} \qquad (5.2.4a)$$

V is the beam volume in m^3 between the points of tangency (corresponding to the shaded area in fig. 5.2.2 a). V shall, however, not be taken as less than $V = 0.6\ bh^2_{apex}$ or less than $0.02\ m^3$.

The bending stresses and the tensile stresses perpendicularly to the grain in the apex zone can be calculated as

$$\sigma_{m,apex} = k_{m,apex}\sigma_{m,curv} \qquad (5.2.4b)$$
$$\sigma_{t,apex} = k_{t,apex}\sigma_{t,curv} \qquad (5.2.4c)$$

where $\sigma_{m,curv}$ and $\sigma_{t,curv}$ are the corresponding stresses in a curv beam (Section 5.2.3) with $h = h_{tang}$. $k_{m,apex}$ and $k_{t,apex}$ are given in Figures 5.2.4b and 5.2.4c.

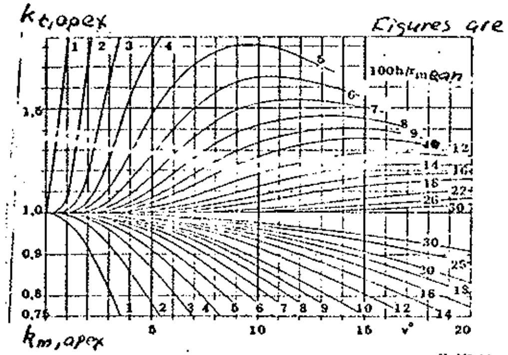


Figure 5.2.4b $k_{m,apex}$ and $k_{t,apex}$ for $E_{0,mean}/E_{90,mean} = 30$

In preparation

Figure 5.2.4c $k_{m,apex}$ and $k_{t,apex}$ for $E_{0,mean}/E_{90,mean} = 15$

CIB-W18/13-100-3b

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB Structural Timber Design Code

Further Proposals and Comments: received April 1980

OTANIEMI, FINLAND
JUNE 1980

3.3 Verification of design

The verification .... partial coefficients

Proposal: Introduce the possibility of verification of the design by means of level-2 (First Order, Second Moment) and/or level-3 methods with accepted failure probability.

This is outside the scope of the CIB-W18 Code.
All the same a better formulation is:
"The verification is usually made according to ..."

3.4 Analyses

For the ultimate limit states linear elastic, non-linear elastic and plastic theories may be applied ......

Proposal 1: For each of these theories give the basic principles and assumptions.

Proposal 1: Impossible; there are many elastic and plastic theories.

Proposal 2: To investigate the internal force distribution and ultimate load carrying capacity of the structure, allowace should be made for experimental and empirical design methods.

Proposal 2: To be discussed.

4.0 General

It must be shown .... are satisfactory.

Proposal: Refer to relevant requirements or subcodes.

In general they do not exist.

Ir. W.R.A. Meyer

Memo 80-30-bg-Myr/Vso, dd. 1 april 1980

Comments on the 4th draft of CIB Structural Timber Design Code

Section

1. INTRODUCTION

Proposal: Change the title of this chapter in:

GENERAL REGULATIONS

No objections.

2.1.1 Characteristic values

Comments: In this section mention is made of a conditioned temperature of 20 $\pm$ 3 °C and a relative humidity of 0.65 $\pm$ 0.02. These values do unhappily and unnecessarily differ from the values mentioned in for instance:

ISO/TC 165 N49.

Joint committee RILEM/CIB-3TT "Testing methods of timber"

6. Conditioning of test specimens

temp. 23 $\pm$ 3 °C and relative humidity of 0.60 $\pm$ 0.02.

idem: ISO/TC 165 N50 7

Proposal: In the code no conditions for carrying out the tests should be specified but only reference will be made to some classification systems.

It has been agreed to follow the relevant ISO-standard (ISO 554, 1976) where the following standard atmospheres are given:

23°C/50% rh - 27°C/65% rh - 20°C/65% rh.

3.0 General

Structures should ...... exceeded

Proposal: Replace "exceeded" in "reached".

The wording should be in accordance with eg CEB/JCSS Volume 1.

Furthermore ......

...... accident.

Comments: This part of 3.0 do not belong in a Timber Code but for instance in "Common Unified Rules for different types of construction and material.

Yes.

CIB-W18/13-100-3c

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB Structural Timber Design Code:

Comments from British Standards Institution Committee CSB 32/2 -

Working Stresses for Timber

OTANIEMI, FINLAND

JUNE 1980

CIB STRUCTURAL TIMBER DESIGN CODE: COMMENTS FROM BRITISH STANDARDS INSTITUTION COMMITTEE CSB 32/2 - WORKING STRESSES FOR TIMBER

Contents:	Either a section on load/prototype testing should be provided or more positive references to suitable test standards and how they should be interpreted should be provided.
Clause 2.1.1:	The tolerance of ± 0.02 on relative humidity is impracticable. Propose ± 0.05. The tolerance on temperature could be reduced to ± 2°C. These proposals conform with the ordinary tolerances of ISO 554-1976 'Standard Atmospheres for Conditioning and/or Testing'. The term 'relative density' should replace 'specific gravity'.
Clause 2.2:	The requirements for relative humidity are difficult to interpret.
Table 2.3a:	'long-term' (amended from 'normal') would be better defined as 'medium-term'.
cf Table 2.3a and Table 3.3.1a:	In Table 2.3a ' permanent' refers to load duration while in Table 3.3.1a the same term implies 'constant'. Is a dual classification of duration and frequency required for load?
Table 2.3b:	Characteristic loads are not defined.
Chapter 3:	It is understood that the content of this Chapter is to be revised.
Table 4.1.1:	The examples of strength classes given in this Table would not be acceptable. There are insufficient classes and the range is too limited to cover many hardwoods. Alternative classes cannot be proposed until standard methods of sampling and analysis have been agreed.
Clause 4.2.0:	Proposed rewording of first paragraph: "Finger jointed structural timber should be manufactured in accordance with rules and controls which are no less stringent than those of 'UN/ECE Recommended Standard for Finger Jointing in Structural Coniferous Sawn Timber' (UN/ECE TIM/WP.3/AC3/8 - Annex II)".
Clause 4.3.1:	The strength classes given as examples in this Clause are unacceptable.
Clause 4.4:	The RILEM/CIB test standard should be referred to rather than the ISO/TC 165 document. Some guidance should be provided on how the test results are to be interpreted.
Table 5.1.0a:	These strength classes are unacceptable.

Table 5.2.0:	These strength classes are unacceptable. What is the origin of : 'rolling shear = $f_v/2$'?
Chapter 7, page 2:	Replace third line by: 'It must be shown that the webs do not buckle, or that the whole of the forces can be resisted by the diagonal tension strength of the webs'.
Table 7.1.1:	It is not clear how the values of h_{max} have been derived. The values will depend on the safety factors to be adopted for design (for stability), and as these factors may not be fixed in the final CIB Code it may be advisable for this Table to be omitted. In any case the values for h_{max} for particleboards and fibreboards will vary widely because of the large range of elastic modulus values associated with different grades of these materials.

CIB-W18/13-100- 4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB STRUCTURAL TIMBER DESIGN CODE

Proposal for section 6.1.5. Nail plates

Nils Ivar Bovim, Norsk Treteknisk Institutt, Norway

OTANIEMI, FINLAND

JUNE 1980

CIB - STRUCTURAL TIMBER DESIGN CODE

The proposed section for joints with nail-plates is based on the work of a Nordic group with the following members: M. Johansen, Denmark, B. Norén, Sweden, T. Poutanen, Finland, and N. I. Bovim, Norway. The work of the group is summarized of B. Norén in the CIB-W18/12-7-3.

Proposed CIB Timber Code section 6.1.5 Nail plates

6.1.5.1 Joint model

In joints with nail-plates the forces are transmitted between the connected members partly by the plates and partly by compression and corresponding friction in the contact area between the connected members - if contact could occur. The proportion of these parts depend on the model chosen for the behaviour of the joints.

The model must give forces and moments transmitted in the joints which are in equilibrium with the forces and moments acting at the connection boundaries. The model for calculation of joints should correspond reasonably with the model for the entire structure, especially with respect to deformations. A basic model for a joint with nail-plates is described in CIB-W18/12-7-3.

6.1.5.2 Load-carrying capacity

The load-carrying capacity of a certain type of nail-plate should be derived from tests carried out in accordance with RILEM/CIB-3TT Testing standard No. 1, Annex A1 Punched metal plate fasteners.

Characteristic values are to be given in a plate certificate, type approval etc.

The load-carrying capacity of joints with nail-plates are limited by the following design criteria A, B and C:

A. Design against plate failure

The capacity of the plates to carry a force F_p and a moment M_p should be verified with respect to the strength of the plate itself.

CIB-W18/12-7-3 gives the following limiting condition:

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

N_a and N_b are the components of F_p in the principle plate directions a and b.

N_{aD} and N_{bD} denote design values for plate strength in a and b directions, calculated as stated in the plate certificate o.e.

It is assumed that F_p and M_p generate a uniform stress in the effective cross section of the plate, and that transmission of compression in the plates is neglected when there is contact between the connected timber members.

Actual plate dimensions should be reduced when the punched nail holes interfere with the plate edge.

B. Design against plate grip failure

The capacity of the grip area to carry the force F_p and the moment M_{RC} applied on the centre of rotation (RC) should be verified.

The grip area A is the effective contact area between the nail-plate and the wood member inside certain narrow zones along the timber edges.

The width of these zones are to be given in the plate certificate o.e.

The shear stress generated by the force F_p and the moment M_{RC} may be assumed to have a constant value over the grip area, corresponding to full plasticity.

CIB-W18/12-7-3 gives the following limiting conditions for the load-carrying capacity of the grip area:

$$F_p \leqslant P_{BD}$$
$$M_{RC} \leqslant M_{RCD}$$

$F_{pD} = A \cdot \tau_D(\alpha)$ where $\tau_D(\alpha)$ is the design value of the shear stress. $\tau_D(\alpha)$ is given in the plate certificate o.e. as a function of the angle of fibre direction to plate force (F_p) and the angle of plate force (F_p) to the a-direction of the plate.

The design value for M_{RC} is determined from

$$M_{RCD} = \tau_D \int^{A} r_{RC} \cdot dA$$

where r_{RC} is the distance from the rotation centre RC and $\tau_D = \tau_D$ ($\alpha = 0$) if not said otherwise in the certificate.

CIB-W18/13-102-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

PROGRAM OF STANDARDISATION WORK INVOLVING TIMBER STRUCTURES AND WOOD-BASED PRODUCTS IN POLAND

OTANIEMI, FINLAND
JUNE 1980

P r o g r a m

Of standardisation work involving timber structures and wood-based products in Poland.

We are planning to develop the following standards:

1. Reviewed standard: Dimensional co-ordination in building. Dimensional tolerances shall be introduced for members and structures from timber and wood-based products in Poland.
2. Methods of testing and criteria for assessment of strength for
   a wall members,
   b roof members,
   c trussed rafters,
   d beams,
   e columns, frames, arcs.
3. Methods for testing of joints and fasteners
   a joints with mechanical fasteners /to be finished soon/,
   b glued-laminated members,
   c joints with nailed plates type Gang No 1.
4. Stress graded sawn building timber.
5. Amendments to standard: PN-73/B-03150: Timber structures. Statical calculations and design.

CIB-W18/13-103-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

REPORT FROM ISO TC/165

by

Mrs A Sørensen

Secretariat ISO/TC 165 - Timber Structures

OTANIEMI, FINLAND

JUNE 1980