Klaus Hemmer

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB - W18

MEETING ELEVEN

VIENNA, AUSTRIA

MARCH 1979

CONTENTS

1 LIST OF DELEGATES

AUSTRIA

E Armbruster	European Federation of Building Joinery Manufacturers, Wien

BELGIUM

A Egerup	Automated Building Components, Brussels
L Montfort	Institute National du Logement, Brussels

CANADA

R O Foschi	Western Forest Products Laboratory, Vancouver
C R Wilson	Council of Forest Industries of British Columbia, Vancouver

DENMARK

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

EIRE

P R Colclough	Institute for Industrial Research and Standards, Dublin

FEDERAL REPUBLIC OF GERMANY

J Ehlbeck	Universität Karlsruhe, Karlsruhe
K Hemmer	Universität Karlsruhe, Karlsruhe
D Henrici	Universität München, München
H Kolb	Otto-Graf Institut, Stuttgart
K Möhler	Universität Karlsruhe, Karlsruhe

FINLAND

E Pennala	Helsinki University of Technology, Espoo

FRANCE

P E H Crubilé	Centre Technique du Bois, Paris

NETHERLANDS

J Kuipers	Stevin Laboratory, Delft
W R A Meyer	Instituut TNO, Delft

NEW ZEALAND

H Bier	New Zealand Forest Service, Wellington

NORWAY

N I Bovim	Norsk Treteknisk Institutt, Oslo

POLAND

W Marosz	Union of Building Joinery Industry, Warsaw
Z Mielczarek	Technical University of Szczecin, Szczecin
W Nożyński	Centralny Osrodek Badowczo, Laskowa

SWEDEN

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

UNITED KINGDOM

	L G Booth	Imperial College of Science and Technology, London
	H J Burgess	Timber Research and Development Association, High Wycombe
	W T Curry	Building Research Establishment, Princes Risborough
	R Marsh	Ove Arup and Partners, London
(1)	J G Sunley	Timber Research and Development Association, High Wycombe
(2)	J R Tory	Building Research Establishment, Princes Risborough

UNITED STATES OF AMERICA

D H Brown	American Plywood Association, Tacoma
W L Galligan	Forest Products Laboratory, Madison
P Taylor	American Plywood Association, Tacoma

INTERNATIONAL STANDARDS ORGANISATION

Mrs A Sørensen	Secretariat ISO/TC165, Denmark

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

2 CHAIRMAN'S INTRODUCTION

DR HARTL, President of Sub-Commission GLULAM welcomed CIB-W18 to Vienna. He told the delegates that their work was being closely followed by the members of GLULAM and he welcomed the opportunity to assist in international co-operation in the field of timber engineering.

The meeting was opened by MR SUNLEY, the co-ordinator of CIB-W18. He drew attention to the large number of delegates from more countries than had previously been represented at a meeting; presuming that this was indicative of the increasing interest and importance of the work of the group. He thanked sub-commission GLULAM and Osterreichischer Leimbauverband for the opportunity to meet in Vienna.

3 CO-OPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: PROF LARSEN told delegates that the next meeting of ISO/TC 165 Timber Structures would take place on 24-27 September 1979 in Ottawa. At that meeting they hoped to discuss the CIB Structural Timber Design Code, the testing of joints, and how to proceed with testing standards for plywood and structural sized timber. He said that there were problems concerning these last two subjects that had to be resolved with ISO/TC 139 and ISO/TC 55 respectively. PROF LARSEN provided paper CIB-W18/11-7-1 "A draft proposal for an International Standard: ISO Document ISO/TC 165 N 38E" as one of the documents to be discussed in Ottawa. He invited comments on this document either in Ottawa or in writing when the Draft International Standard was circulated.

RILEM 3-TT: DR KUIPERS reported that the CIB-W18/RILEM 3-TT group had completed and passed to ISO/TC 165 the main joint-testing standard but they had not yet completed annexe B and C dealing with nails and staples. At their meeting, that had immediately preceded this W18 meeting, they had discussed the plywood testing standard. They still had some drafting work to do on that standard but expected to have it ready for ISO/TC 165 by September. DR KUIPERS said that they had made several unsuccessful attempts to communicate with Professor Noack, the chairman of TC 139, about plywood testing. The meeting was reminded that, in accordance with RILEM policy, 3-TT would have to be disbanded. However, a new group would be formed to consider the testing of other board materials, including particle and hardboards, and this group would invite participation from the ISO technical committees at present responsible for those materials.

IUFRO S5.02: MR SUNLEY told delegates that the next meeting of IUFRO S5.02 Wood Engineering would be included in the all-group-5 meeting that would take place in Oxford, England on 8-22 April 1980. Further details of the meeting are available from the Princes Risborough Laboratory, England. Professor Madsen of the University of British Columbia, Canada, is now the co-ordinator for the IUFRO Wood Engineering Group.

FEMIB/GLULAM: DR ARMBRUSTER said that sub-commission GLULAM was continuing its work on the production of a code for glued-laminated timber. They now planned to produce their code in three parts: Part 1 'Requirements for Timber' had been produced (CIB-W18/8-12-3); Part 2 would deal with design; Part 3 of this code was almost complete and would soon be presented to CIB-W18. Part 2 was more difficult and they were now beginning to analyse the answers to questionnaires on standard design procedures that had been circulated to their member countries. FEMIB wished to co-operate with CIB-W18 and would present their code to W18 when it was completed.

PROF LARSEN expressed concern at the efforts of FEMIB to produce a glulam design code independent of the main body of the code. He pointed out that there was already a chapter in the CIB Code on glulam design.

PROF LARSEN distributed paper CIB-W18/11-102-1 'Eurocodes'. He suggested that there should be no attempt within the European Economic Community to produce a timber code that did not take full account of a wider international code and the competent body for both of these needs was CIB-W18.

DR HARTL, the president of sub-commission GLULAM, told delegates that there should be no conflict between the work of CIB-W18 and sub-commission GLULAM. It was the intention of his organisation to present their work to W18 as a constructive contribution towards a unified approach within Europe and within a wider international scene.

The chairman reminded the meeting that W18 was not exclusively European and that their work towards harmonisation had a wider international appeal. He suggested that some European delegates might get together within the coming days to discuss EEC problems and to clarify what action they might take to improve communications with the EEC and CEI-BOIS.

Delegates were also informed of the following meetings:

12 Nov 1979	Trussed rafter design, St Louis, USA
9-14 June 1980	International Association for Shell and Space Structures, Ooloo, Finland
Sept 1980	International Association for Bridge and Structural Engineering, Vienna.

4 CIB CODE

The meeting considered Chapter 6 of paper CIB-W18/9-100-1 'CIB Timber Code (Second Draft)'. The following comments and amendments were made:

Title of Chapter 6: To be changed to 'JOINTS'

Chapter 6, General: PROF LARSEN said that he was not entirely satisfied with this chapter because it was too detailed to deal with all species satisfactorily.

DR KUIPERS said that he would be producing written comments on this chapter. The general content was satisfactory but Dutch data on nails and connectors was significantly different from that given in this chapter.

Section 6.0.1: MR EHLBECK suggested that 'general behaviour' should replace 'load-carrying capacities' in the first paragraph.

DR FOSCHI asked for the last six words of the last paragraph to be deleted.

MR GALLIGAN said that there should be general notes on the effect of fire retardants, preservatives and environmental changes on fasteners.

Section 6.0.2: It was agreed that 'characteristic' should be deleted from the title.

MR CURRY and MR EHLBECK were not satisfied that the contents of table 6.0.2 was suitable for bolts and connectors.

Section 6.1.1: MR EHLBECK suggested that equation 6.1.1.1a should be:

$F = k_{nail}\ d^m$, and an equation 6.1.1.1c should give $1.5 \leq m \leq 2.0$ for softwoods. The sentence 'A joint should contain at least 2 nails' is to be deleted and a comment on the facing page is to be provided.

The lower half of Chapter 6 page 2 is to be brought in to the main text.

PROF LARSEN agreed to consider the introduction of a minimum thickness for the steel in steel-to-timber joints.

MR RIBERHOLT suggested that for steel-to-timber joints there should be another value of k_{nail}.

DR KUIPERS asked for reduction factors for multiple in-line fasteners.

MR EHLBECK drew attention to the absence of information on joint deformation. He agreed to provide a proposal for consideration at the next meeting.

Section 6.1.1.2: MR GALLIGAN pointed out that the first line was contradictory since characteristic values relate to one climate class.

Section 6.1.1.3: PROF LARSEN said that he would adopt Prof Möhler's recommendations from paper CIB-W18/9-7-2 "Staples" for the Code.

Section 6.1.2: PROF LARSEN said that he understood that Mr Johansen was investigating the differences between the formulae in this section and those in CP 112 and CSA 086 (the UK and Canadian Codes).

PROF LARSEN agreed to reconsider the wording of the first main text paragraph below table 6.1.2.

Section 6.1.3: DR BOOTH suggested that in 6.1.3.1 a further mode of failure should be included since the behaviour of screws in hardwoods was not adequately covered.

The assumptions at the foot of page 6 are to have their print size reduced.

PROF LARSEN agreed with MR BOVIM that in glued laminated members without checks, screws in the end grain might carry load. PROF LARSEN said he would include a note to this effect on the facing page.

Section 6.1.4: It was agreed that it would not be feasible to tabulate data for proprietary items such as connectors but a comment on the facing page should be provided.

Section 6.1.5: MR CURRY observed that since there were so many types of nail plate it would not be possible for this section to provide detailed recommendations. Since plates were not manufactured to comply with a standard it would only be possible to specify that they should be tested in accordance with a standard.

DR NOREN said that he would provide a short paper setting out recommendations for this section. PROF MOHLER agreed to contribute.

5 CIB STRUCTURAL TIMBER DESIGN CODE (Third Draft)

The meeting considered the comments that had been received on the CIB Structural Timber Design Code (Paper CIB-W18/11-100-1). The comments considered were from Finland, USSR, Australia, Poland and The Netherlands and are contained in paper CIB-W18/11-100-2.

The following comments and changes were made:

List of Contents: DR KUIPERS presented paper CIB-W18/11-100-2(e) on behalf of Dr Meyer, TNO, The Netherlands. He said that this comment was not on the content of the Code but on its structure. The Code should be arranged in a more logical order for the convenience of the designing engineer. The present Dutch code had a format similar to the CIB code and they had received complaints from practising engineers about the illogical order of presentation.

PROF LARSEN said he felt that Dr Meyer had misused the Canadian comments made to ISO TC 165 in September 1976 and Canada had in fact been represented by a practising engineer on the code drafting committee in March 1978. At that meeting Dr Meyer's proposals had been considered and rejected. PROF LARSEN also drew attention to Dr Meyer's eight page index for less than forty pages of code. He did not consider that an aid to simple usage. PROF LARSEN said that little could be achieved by a 'logical' format. Structures could not be designed by rote; designing engineers had to be conversant with the whole code content.

The format and content of the CIB Code could not control national codes said MR CURRY. In the UK it was desirable to follow the general pattern set by other material codes and this policy would persist regardless of the format of the CIB Code.

MR MARSH did not see much advantage in Dr Meyer's format. As a practising engineer dealing with several materials in many countries he considered that a consistent presentation between materials and countries was more important than the order of presentation.

The chairman summed up the general feeling of the meeting by stating that for such a radical change to be introduced there would have to be a very strong feeling for Dr Meyer's proposal. Since that feeling was not evident the existing format would be retained.

Chapter 1 (USSR): Notation is in accordance with ISO Standard 3898.

Section 2.1.1 (Poland, USSR, Finland):	Delete 'very short term load condition'. The temperature and humidity are those corresponding to climate class 1.
	The volume 0.02 m^3 is to be retained since it is only a calibration reference value.
Section 2.2 (Pol (Poland, USSR, Australia):	MR MAROSZ pointed out that 23°C was not an ISO standard temperature, but a Swedish proposal to ISO.
	It was agreed that the climate classes should, if possible, conform to ISO Standard IS 554:1976. If this is not possible the standard temperature is to be taken as 20°C and the humidity for class 1 as 0.65.
	PROF LARSEN suggested that increasing the number of climate classes would not be worthwhile since there would not be significant changes in material properties between the classes.
Section 2.3 (Poland, USSR):	It was agreed that no changes should be made.
Section 4.1.0 (Australia):	Agreed with the comment but considered it to be already covered.
Section 4.1.1:	PROF LARSEN, in answer to MR GALLIGAN, said that not all characteristic values would be established by testing. Some would be interpolated from other results.
	MR SUNLEY asked for this to be made clear on the facing page of 2.1.1.
	MR BROWN and MR GALLIGAN thought that such a note should appear in the body of the Code.
	MR CURRY said that the relations between bending and tension strengths suggested by table 4.1.1 would not always be valid. For machine grading there was a good relation between bending and tension strength but for visual grading the relativities could be different.
	PROF LARSEN explained to the meeting that the strength classes he had written into the code were intended as examples and did not purport to cover the whole range of hardwood and softwood structural timbers. He said that he expected the problems of dealing with a very wide range of species to be discussed more fully at the next meeting of ISO/TC 165. His main intention at this stage of the Code's evolution had been to introduce the system of strength classes.
Table 4.3.1:	'37.5' is to be replaced by '38'.
Section 4.4:	Since there are no fixed relations between the properties of plywood it had not been possible to define strength classes for plywood, said PROF LARSEN.
	MR GALLIGAN asked for the characteristic strength of plywood to be related to a standard width in section 2.1.1.

PROF LARSEN said that he would insert a note to the effect that there were sampling and testing standards and that modification factors would become available to translate test results into characteristic values.

Table 4.7: Delete climate class 0. Add a note to the effect that no protection is required in permanently heated building without humidifying.

Table 5.1.0b: The factors for strength calculations are to be changed. A footnote will be inserted beneath this table about constructing in the green condition for eventual use in a dry condition.

Section 5.1.1.3: PROF LARSEN explained that creep buckling was taken into account by the use of lower modulus of elasticity values.

PROF LARSEN agreed to change Chapter 5 page 4 to take into account the comments received from Dr Meyer (CIB-W18/11-100-2g).

Section 5.1.1.4: DR HENRICI told the meeting that in his experimental work he had found high stress concentrations in sharp corners but that it would be difficult to produce a rule to allow for this effect. He offered to produce a paper for the next meeting.

PROF LARSEN will include a comment on the facing page to this section and would take account of the literature available from Australia, Germany and Canada.

Section 5.1.1.7: DR KUIPERS drew attention to Dr Meyer's comments.

PROF LARSEN said that he considered the column design methods satisfactory and the curves drawn in Fig 5.1.1.7 were simple examples from various national codes. He accepted the Finnish comment.

Section 5.1.2: Title to be changed to 'Tapered beams'.

Section 5.2.2: MR HEMMER offered to produce formulae as an alternative to Fig 5.2.2c and Fig 5.2.2d although he pointed out that they could not be too exact.

DR FOSCHI observed that the diagrams were apparently based on pure bending which did not reflect a very genuine loading situation.

Section 7.1.2: In response to a request from MR BROWN, PROF LARSEN said he would try to improve the text to make clear the load sharing between the inner and outer parts of a panel.

Section 7.1.3: PROF LARSEN said that a reference to Annexe 7A would only be valid for stiff connections. He said that he would be giving further consideration to the other comment from Poland.

Section 7.2: Bolts are not permitted without connectors.
In table 7.2, replace '0.002' by '0.02'.

Section 8.3: In paragraph five, replace '2 mm' by '1 mm'.

Annexe 7A: In the title, replace 'BEAMS AND COLUMNS' by 'MEMBERS'.

General: MR KOLB asked if it would be possible to give some numerical or other identification to the modification factors in the Code.

PROF LARSEN told the meeting that he would be writing to each of the individuals that had submitted comments on the Code. He would also produce a fourth draft of the Code and suggested that it should be formally submitted to ISO/TC 165.

MR EHLBECK said that there were too many gaps in the Code for it to be submitted as a finished document.

DR NOREN and MR BOVIM pointed out that the closing date for comments on the Code to CIB-W18 was 1 September 1979.

It was not necessary at this time, said PROF LARSEN, to have a complete document and TC 165 would not attempt to fill the gaps that existed in the Code; that was the function of CIB-W18. Nor should the submission of the Code to TC 165 inhibit comment. PROF LARSEN assured delegates that he expected the Sept 1979 meeting of TC 165 to be primarily concerned with how to deal with the Code. He envisaged several international standards eminating from the Code; perhaps on climate classes, load-duration classes and other topics that could be dealt with individually.

It was agreed that Prof Larsen should produce a fourth draft of the Code and should submit it to ISO/TC 165 for consideration at their meeting in September 1979.

6 CIB STRUCTURAL TIMBER DESIGN CODE: CHAPTER 3

Paper CIB-W18/11-100-3 'CIB Structural Timber Design Code: Chapter 3' was presented by PROF LARSEN. He also distributed 'Safety Design of Timber Structures' (CIB-W18/11-1-1) which he said would explain some of his thinking in drafting Chapter 3.

MR RIBERHOLT asked why fire and progressive collapse were mentioned under the General heading. He thought that fire should be in section 3.2 Actions.

PROF LARSEN said that the Code made no attempt to design against fire. Design for the effects of fire was a governmental requirement that could overrule the Code.

MR SUNLEY said that part of the United Kingdom timber code was concerned with fire resistance. Both he and MR GALLIGAN offered to make contributions to Chapter 3 with respect to fire resistance.

MR BOVIM asked for high compression perpendicular to the grain stresses to be treated as serviceability limit states.

In answer to MR BURGESS, PROF LARSEN said that ψ factors could not take into account the time depdenence of loads; they were only to allow for the probability of having two or more loads simultaneously at their maximum values.

The title of section 3.3 is to be changed to 'Verification of Design'.

DR BOOTH suggested that γ_{n1} should be in small print, similar to γ_{m1}.

DR FOSCHI asked for the first part of section 3.3.2 to be redrafted.

7 STRESSES FOR SOLID AND GLUED-LAMINATED TIMBER

PROF MOHLER introduced paper CIB-W18/11-6-2 'Stresses Perpendicular to the Grain'. He pointed out that the volume effect on tension perpendicular to the grain stresses was significant and should not be ignored.

MR HEMMER suggested that the Code should contain stress values for a standard volume and modification factors for larger volumes.

PROF LARSEN agreed that either factors or a formula could be included in the Code and that there could be a limiting minimum volume below which they would not be applicable. However, he was awaiting confirmation of the factors to be used from Dr Barrett of the Canadian Western Forest Products Laboratory.

DR FOSCHI agreed to check the factors to be included in the code.

MR HEMMER presented paper CIB-W18/11-6-3: explaining that it was an extension of an earlier paper CIB-W18/9-6-4 'Consideration of Combined Stresses for Lumber and Glued-Laminated Timber'. He asked Prof Larsen to consider it when the next draft of the CIB Code was being prepared.

Paper CIB-W18/11-10-1 'Tapered Timber Beams' was introduced by MR RIBERHOLT who said that he had been able to demonstrate that tapered glulam beams were linear elastic up to brittle fracture. He said that it was possible to calculate the radial stresses on circumferential sections and also the principal stresses. He recommended the use of the Norris interaction formula.

MR HEMMER said that since the slopes of tapered beams were usually quite small there was very little difference between perpendicular and circumferential stresses. It was also much more convenient for the engineer to simply divide moment by section modulus.

PROF MOHLER said that tests involving combined shear and tension and shear and compression stresses indicated that the Norris formula was probably safe in compression but less safe in tension.

PROF LARSEN agreed to reconsider the content of section 5.1.2 of the CIB Structural Timber Design Code.

8 STRESSED SKIN PANELS

Paper CIB-W18/11-4-1 'Analysis of Plywood Stressed Skin Panels with Rigid or Semi-Rigid Connections' was presented by MR BURGESS. He told delegates that the paper was an example of the practical application of Annexe 7A of the CIB Code. He agreed with PROF LARSEN that the basic assumptions for the paper were the same as for Annexe 7A and that this was an uncontroversial extension of the general principles given in that annexe. He pointed out that the paper dealt with non-symmetric sections and calculated a 'true' neutral axis rather than assuming no slip and adopting an idealised neutral axis.

9 TRUSSED RAFTERS

DR EGERUP, introducing paper CIB-W18/11-14-1 'Design of Metal Plate Connected Wood Trusses', told the meeting that he had studied three existing codes and a draft revision of the UK Code in assessing existing design methods. He explained the differences that existed between their design procedures and said that the more complex procedures tended to be more conservative than the simple methods and none was entirely satisfactory. There was a need for two design procedures, continued DR EGERUP; a simple, conservative, every day method suitable for hand calculations and a more complex but less conservative computerised method for the calculation of span tables. He would eventually like to see a plastic design method evolve that would allow for the ductility of timber.

MR RIBERHOLT said that no one really knew how to analyse a truss. Eccentricity at joints was seldom considered and the variability in the stiffness of members was probably much more significant than the stiffness of joints.

One of the major problems in attempting to analyse a rafter, said PROF MOHLER, was that it was a continuous beam and the true bending moment diagram was not known over the centre support. It did not follow the theoretical pattern and the difficulties were increased when the supports were considered to be elastic. In addition to these problems there was the difficulty of taking into account the variability within and between the members.

In Canada, said DR FOSCHI, truss design was based on the conservative assumption of pin-joints with an arbitrary factor of 1.3 to relate the design to test results. He foresaw difficulties in how Codes could be written to accept fully computerised methods of design.

MR SUNLEY and MR CURRY said that in the UK nearly all on-site failures that had occurred could be attributed to workmanship. Whatever design method was adopted it should not assume that the truss fabrication was perfect.

It should be remembered that design rules for trusses should be the same as for other structures said PROF LARSEN. He asked if a short paper could be produced setting out the design principles for turssed rafters.

MR RIBERHOLT suggested that the problem of analysis could be divided into three stages: static analysis of the structure; timber design peculiarities and plates and joints.

It was agreed that Dr Egerup should form a working group to draft design rules for trussed rafters and to recommend methods of analysis.

10 PLYWOOD

DR NOREN introduced "Sampling of Plywood for Testing Strength" (Paper CIB-W18/11-4-3). He said that in this second draft he had omitted reference to the evaluation of test results since that would eventually be the subject of a separate standard. He agreed with MR GALLIGAN that the reference to the period of time in which the plywood is produced implied that time dependent variables were adequately represented whereas the intention had been only to draw attention to the importance of time to the sampling.

PROF LARSEN asked if different lay-ups could be present in the same population. He suggested that a large part of section 3 belonged in section 2.

MR TAYLOR drew attention to the number of panels that would need to be tested to adequately represent some populations. With only 2 species, 17 mills, 3 layups and 6 thicknesses, testing 15 panels from each of these combinations would involve more than nine thousand panels.

MR CURRY said that it should not be necessary to test all lay-ups of a given thickness of plywood when the properties for some could reasonably be inferred from others.

DR NOREN agreed with Ms Taylor and Mr Curry saying that it was very difficult to precisely define a population and some judgement would have to be exercised on how to to pool sub-populations to limit the extent of testing programmes. There would also be instances where characteristic property values would be required for mixed lay-ups and sources of supply and this would present difficulties for the standard on the evaluation of results.

It was agreed that the plywood sub-committee of DR NOREN, DR BOOTH, DR KUIPERS and DR WILSON should provide a further draft for the plywood sampling standard and they should also consider the evaluation and analysis of test results.

DR WILSON circulated paper CIB-W18/11-4-2 'A Comparison of Plywood Modulus of Rigidity Determined by the ASTM and RILEM/CIB-3TT Test Methods". He said that the tests referred to in the paper indicated significant differences between the RILEM/CIB-3TT test method and that given in ASTM. Furthermore they showed that there would be difficulties in deriving general modification factors to convert data from one test method to the other.

The meeting agreed with a recommendation from DR BOOTH that further testing was required to fully investigate Dr Wilson's preliminary findings and urged that COFI should be encouraged to continue their work in this field.

11 TESTING STRUCTURAL-SIZED TIMBER

MR GALLIGAN, introducing paper CIB-W18/11-6-1 'Evaluation of Lumber Properties in the United States' outlined the extent of the resources that were currently involved in research into light-frame construction and material properties in the US. He spoke of the sampling methods that had been adopted and how work was being carried out to find which were the most suitable methods of analysis for the data. He anticipated that statisticians would again advocate non-parametric statistics for the estimation of lower exclusion levels and expected a revision of ASTM D2555 to permit both parametric and non-parametric statistics to be used. MR GALLIGAN also told delegates of the so far unsuccessful efforts that were being made to establish confidence limits for the Weibull distribution. In answer to PROF THUNELL, MR GALLIGAN said that the grading rules in the US were essentially the same for all structural softwoods. He agreed that there would be differences between species in the size and shape of knots and in the percentage of juvenile wood and the growth rate, and therefore massive sampling was required to adequately cover all these variables. In the longer term there were plans to revise the grading rules, concluded MR GALLIGAN.

MR FOSCHI told the meeting that at the Western Forest Products Laboratory there were plans to repeat some of the procedures adopted by Madison. He drew attention to the fact that the material was tested in the green condition and suggested that projects were required to evaluate moisture content and temperature effects.

12 GLUED-LAMINATED TIMBER

Paper CIB-W18/9-12-1 'Experiments to Provide for Elevated Forces at the Supports of Wooden Beams, with Particular Regard to Shearing Stresses and Long-term Loadings' was briefly introduced by DR ARMBRUSTER. He explained that although the idea of reinforcing the bearing areas of beams was not new the paper was the result of an investigation into how the reinforcing could be provided by dowels. He pointed out that although it had been shown that the performance in compression perpendicular to the grain had been improved there had been no improvement in shear behaviour. This was contrary to the finds from earlier tests carried out by Professor Möhler.

PROF MOHLER said that the results from the tests reported in this paper were not directly comparable with his own since there had been differences in the modes of failure.

13 HARDBOARD

PROFESSOR NOZINSKI circulated for information, three papers on the testing of hardboard:

CIB-W18/11-13-1	Tests on Laminated Beams from Hardboard Under Short and Long-term Load
CIB-W18/11-13-2	Determination of Deformation of Special Densified Hardboard Under Long-term Load and Varying Temperature and Humidity Conditions.
CIB-W18/11-13-3	Determination of Deformation of Hardboard Under Long-term Load in Changing Climate.

14 NEXT MEETING AND OTHER BUSINESS

MR SUNLEY informed the meeting that Mr Crubilé had kindly offered to arrange the next meeting in France, probably in Bordeaux. The dates for the next meeting were fixed as 15-19 October 1979.

Several members have requested earlier notification of meetings and as an invitation has been received from Mr Saarelainen of Finland the meeting-after-next has tentatively been agreed for 2-6 June 1980 at Otaniemi, Finland.

Finally the chairman closed the meeting and on behalf of all delegates thanked MR ARMBRUSTER and MR WOLF, as representatives of Sub-commission Glulam and Osterreichischer Leimbauverband, for the arrangements they had made for the meeting and for the hospitality that had been extended to the participants.

15 PAPERS PRESENTED AT THE MEETING

CIB-W18/11-1-1	Safety Design of Timber Structures - H J Larsen
CIB-W18/11-4-1	Analysis of Plywood Stressed Skin Panels with Rigid or Semi-rigid Connections - I Smith
CIB-W18/11-4-2	A Comparison of Plywood Modulus of Rigidity Determined by the ASTM and RILEM/CIB-3TT Test Methods - C R Wilson and A V Parasin
CIB-W18/11-4-3	Sampling of Plywood for Testing Strength - B Noren
CIB-W18/11-6-1	Evaluation of Lumber Properties in the United States - W L Galligan and J H Haskell
CIB-W18/11-6-2	Stresses Perpendicular to Grain - K Möhler
CIB-W18/11-6-3	Consideration of Combined Stresses for Lumber and Glued Laminated Timber (Addition to Paper CIB-W18/9-6-4) - K Möhler
CIB-W18/11-7-1	A Draft Proposal for an International Standard: ISO Document ISO/TC165N 38E
CIB-W18/11-10-1	Tapered Timber Beams - H Riberholt
CIB-W18/11-13-1	Tests on Laminated Beams from Hardboard under Short and Long-term Load - W Nożyński
CIB-W18/11-13-2	Determination of Deformation of Special Densified Hardboard Under Long-term Load and Varying Temperature and Humidity Conditions - W Halfar
CIB-W18/11-13-3	Determination of Deformation of Hardboard Under Long-term Load in Changing Climate - W Halfar
CIB-W18/11-14-1	Design of Metal Plate Connected Wood Trusses - A R Egerup
CIB-W18/11-100-1	CIB Structural Timber Design Code (Third Draft)
CIB-W18/11-100-2	Comments Received on the CIB Code a U Saarelainen - Finland b Y M Ivanov - Union of Soviet Socialist Republics c R H Leicester - Australia d W Nożyński - Poland e W R A Meyer - The Netherlands (List of Contents) f P Beckman; R Marsh - United Kingdom g W R A Meyer - The Netherlands (Centrically and eccentrically loaded construction elements) h W R A Meyer - The Netherlands (Lateral buckling)
CIB-W18/11-100-3	CIB Structural Timber Design Code; Chapter 3
CIB-W18/11-102/1	Eurocodes - H J Larsen

CURRENT LIST OF CIB-W18 TECHNICAL PAPERS 79.05.01

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

Subjects are denoted by the following numerical classification:

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

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

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

LIMIT STATE DESIGN

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

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

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

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

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

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

1 Symbols for Use in Structural Timber Design

PLYWOOD

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

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

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

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

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

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

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

6-4-2 In-grade versus Small Clear Testing of Plywood - C R Wilson

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

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

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

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

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

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

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

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

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

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

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

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

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

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

STRESS GRADING

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

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

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

LOAD SHARING

DURATION OF LOAD

TIMBER BEAMS

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

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

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

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

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

11-10-1 Tapered Timber Beams - H Riberholt

ENVIRONMENTAL CONDITIONS

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

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

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

LAMINATED MEMBERS

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

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

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

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

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

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

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

PARTICLE AND FIBRE BUILDING BOARDS

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

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

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

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

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

STRUCTURAL DESIGN CODES

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

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

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

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

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

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

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

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

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

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

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

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

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

11-102-1 Eurocodes - H J Larsen

INTERNATIONAL STANDARDS ORGANISATION

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

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

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

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

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

TRUSSED RAFTERS

4-9-1	Paper 14 Long Term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen
6-9-3	Deflection of Trussed Rafters under Alternating Loading During a Year - T Feldborg and M Johansen
7-2-1	Lateral Bracing of Timber Struts - J A Simon
9-14-1	Timber Trusses - Code Related Problems - T F Williams
9-7-1	The Design of Truss-Plate Joints - F J Keenan
10-14-1	Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon
11-14-1	Design of Metal Plate Connected Wood Trusses - A R Egerup

CIB TIMBER CODE

2-100-1	Paper 2 A Framework for the Production of an International Code of Practice for the Structural Use of Timber - W T Curry
5-100-1	Design of Solid Timber Columns - H J Larsen
5-100-2	A Draft Outline of a Code of Practice for Timber Structures - L G Booth
6-100-1	Comments on Document 5-100-1; Design of Timber Columns - H J Larsen
6-100-2	A CIB Timber Code - H J Larsen
7-100-1	CIB Timber Code Chapter 5.3 Mechanical Fasteners; CIB Timber Standard 06 and 07 - H J Larsen
8-100-1	CIB Timber Code: List of Contents (Second Draft) - H J Larsen
9-100-1	The CIB Timber Code (Second Draft)
11-100-1	CIB Structural Timber Design Code (Third Draft)
11-100-2	Comments Received on the CIB Code

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

JOINT COMMITTEE ON STRUCTURAL SAFETY

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

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

CIB PROGRAMME, POLICY AND MEETINGS

1-105-1 Paper 1 A Note on International Organisations active in the Field of Utilisation of Timber - P Sonnemans

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

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

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

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

CIB-W18/11-1-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAFETY DESIGN OF TIMBER STRUCTURES

by

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SAFETY DESIGN OF TIMBER STRUCTURES

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SAFETY DESIGN OF TIMBER STRUCTURES

H. J. LARSEN*

SUMMARY

In recent years great efforts have been made to create a foundation for internationally agreed uniform load- and safety regulations based on the so-called limit-state methods. According to these methods certain limit states may not be exceeded. The limit states may relate to the load-carrying capacity of the structures (ultimate limit states) or to their function in normal use (serviceability limit states). As part of the method statistics are used to evaluate strengths and loads.

Significant contributions to clarify the theoretical and formal foundation have been made by CEB and NKB, whose proposals have been harmonized both mutually and with similar East European proposals into the so-called Östlund proposal. It is expected that general agreement can be reached upon this proposal.

In all available proposals it is assumed that the safety can be ensured by a system of partial coefficients, i.e. by multiplying characteristic loads and dividing strength parameters (strength values and moduli of elasticity) by partial coefficients. Is is possible to obtain a uniform safety level if the partial coefficients are assumed to depend on the type of load (permanent or variable load) and on the accuracy with which the material properties are known.

For calibration of the partial coefficients the use of a so-called level-2 method is assumed. The method is based on postulated distributions for load (usually the normal distribution) and strength parameters (usually the log-normal distribution), and therefore, the calculated failure risks are formal values and they are often given as so-called safety indices.

In principle the ultimate objective is to calculate the true failure risks, but the calculations can only be carried out in few cases, and the methods are of no importance for codes. This is due partly to lack of the necessary information about the true distribution of loads and strengths, and partly to technical calculation problems.

The proposals for load- and safety regulations aim only at code writers, and it is then the task of the different technological fields, e.g. concrete, steel masonry, timber, to transform them into user-oriented code rules. As a contribution to this transformation in the timber field a proposal how the safety section can be drawn up in the CIB Structural Timber Design Code has been worked out.

The proposal does not take into consideration the size of the partial coefficients, since their determination is the responsibility of the national authorities. However, reference is made to the proposals available from CEB and NKB.

INTRODUCTION

The great efforts made in recent years in connection with safety problems in load-carrying structures have been directed towards setting up and testing the theoretical basis as well as towards the adjustment of the theories to practical use in codes. This paper will especially deal with the latter aspect.

Although it is necessary to look at the problems as a whole, they can in principle be divided into the following main groups:

- determination of loads on a statistical basis,
- determination of strength- and stiffness parameters on a statistical basis,
- clarification of those limit states that are not to be exceeded,
- ensuring adequate safety against reaching a limit state.

The design methods worked out in this connection are often denoted limit state design, a designation, however, which is also often used for the special safety method, called the partial coefficient method or the load-factor method.

Moreover, it should immediately be noted that it is the systematic treatment and theoretical clarification of the subjects, rather than the subjects themselves, which is new:

- For many years loads have been determined on a statistical basis (for natural loads, for example, on the basis of meteorological data).

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- In the calculation with permissible stresses, the ultimate limit state has merely been that the stresses calculated according to the theory of elasticity should not exceed a certain limit - the permissible stress.
- Permissible stresses have indeed been determined from the mean value of the strengths, but in determining the safety factors the coefficient of variation has been taken into account.
- Partial coefficient systems have been used for many years. In Denmark they were thus introduced in the design of prestressed concrete as early as 1949 and some years later they were used generally. In the East European countries they have been used for about 25 years.

BASIC WORK

In Western Europe CEB*) has dominated the work, from 1971 through JCSS which is a joint bode for CEB, CECM, CIB, FIP, IABSE and RILEM. The CEB-domination has been political as well as technical.

The work was temporarily finished in 1976 with the CEB Bulletin 116, »Common Unified Rules», which is included as volume I in the publication »International System of Unified Standard Codes of Practice for Structures». In the following it is therefore denoted »CEB, volume I». Further, CEB's concrete code is included in the system as volume II, CECM's steel code as volume III and CIB's Structural Timber Design Code as volume V.

As a background for harmonization of the building regulations in the Nordic countries and for their participation in the work of JCSS a committee on load and safety was set up by NKB. The committee has worked out a Nordic counterpart to CEB, vol. I, namely »Recommendations on load and safety codes, 1978». In the following it is denoted NKB-Recommendations. In the course of time the difference between the CEB and NKB proposals has been reduced. The most significant devations are details of the system of partial coefficients and NKB has fewer representative load parameters and simpler load combinations.

The East European countries have also dealt with the problems through the cooperative body CMEA. In 1977, in an attempt to remove the differences that still remain, though many were removed through the cooperation with ISO, EEC asked JCSS to prepare a proposal for an internationally harmonized code. The result was »General Principles on Reliability of Structural Design», May 1978, which had mainly been worked out by Professor L. Östlund of Sweden, and is therefore normally known as the Östlund proposal. Generally, it is clearer and more coherent than the others, but it also leaves out a number of details, for example throwing a veil over areas where the traditions of Eastern and Western Europe differ. It looks as if general agreement can be reached upon this proposal and thus it will probably very soon be transformed into an ISO-standard to replace International Standard ISO-2394: General Principles for the Verification of the Safety of Structures.

One of the main objects of the EEC is the removal of trade obstacles e.g. dissimilar building codes and safety systems. Not only do these obstruct trade between the member countries in connection with building, (and especially such related services as consulting and contracting), but they also affect competitive power in e.g. the Middle East, where the lack of intereuropean codes gives rise to dissatisfaction. This is partly because the Middle Eastern countries do not want to bind themselves to a single country's code and partly because comparisons between the different quotations become difficult.

EEC has therefore begun drawing up so-called Euro-Codes, which are expected to become valid in all member countries in addition to the national codes, so that the consultants and contractors for work, which in accordance with the EEC-rules is to be put out to international tenders, can use either Euro-Codes or the codes of the building country.

So far the commission has set up a steering group and a working group for Euro-Code No. 1, General Load- and Safety Regulations, to be drawn up on

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*) The meaning of the abbreviations for organizations etc. used in the text is given below in alphabetical order:

CEB: Euro-International Committee for Concrete.
CECM: European Convention for Constructional Steelwork.
CMEA: Council for Mutual Economic Aid.
CIB: International Council for Building Research Studies and Documentation.
ECE: Economic Commission for Europe under UN/UNESCO.
EEC: European Economic Community.
FIP: International Federation for Prestressing.
IABSE: International Association for Bridge and Structural Engineering.
ISO: International Organization for Standardization.
JCSS: The Joint Committee on Structural Safety.
NKB: Nordic Building Regulations Committee.
RILEM: International Union of Testing and Research Laboratories for Materials and Structures.

the basis of CEB, vol. I and the Östlund proposal.

In the middle of 1979 the work on Euro-Code No. 1 is expected to be so far advanced that the work on Euro-Code No. 3: Steel Structures, can begin in August 1979. The work on Euro-Code No. 5 Timber Structures, is provisionally planned to begin around the beginning of 1980, when a CIB-proposal is expected to be available.

Recipients of the proposals

All the proposals are intended for use by code writers and not by designers, and they are not to be regarded as some kind of a functional model code.

The main object of the proposals has been to create a common precise frame of reference for the persons who have participated in the work in this rather unbroken field. Many sections are characterized by the fact that their final forms are compromises reached after long discussions. These compromises have not in all cases been carried through in other sections.

The next phase is to transform the proposals into user-oriented operational code chapters. It would undoubtedly have been a great advantage if the work on CEB-volume I had concluded with a total rewriting - as done in the Östlund proposal - and if it had been accompanied by a proposal for the drawing up of the load and safety chapters in the structural design codes.

CEB-volume I has in fact been converted in practice in International System of Unified Standard Codes of Practice for Structures, Volume II: CEB-FIP Model Code for Concrete Structures (in the following denoted CEB-concrete), but firstly there has been such confusion of the load and safety section and the concrete rules that is is difficult to transfer the rules to other materials and secondly, there are in many fields incomprehensible divergences - both material and linguistic - between CEB-volume I and CEB-concrete.

Therefore, it is necessary to work out general sections for each material with the risk that they differ from one code to another.

Limit states

An essential element has been a clear description of the limit states the exceeding of which means that the structure no longer fulfils its function. These are generally classified as ultimate limit states corresponding to exhaustion of the load-carrying capacity (of the whole structure or parts of it) and as serviceability limit states where the structure no longer fulfils its assumed function in normal use.

The requirement that the relevant limit states are not exceeded is normally followed by a requirement for a reasonable robustness, for example, collapse of a single structural member (as a result, perhaps of an accident) should not cause collapse of a significant part of the structure (security against progressive collapse).

Levels of limit state design

In CEB-volume I, for example, three levels are identified at which the structural safety may be treated. They are:

Level 1:
A semi-probabilistic process in which the probabilistic aspects are treated specifically in defining the characteristic values of loads or actions and strengths of materials and these are then associated with partial factors, the values for which, although stated explicitly, should be derived, whenever possible, from a consideration of the probability aspects.

Level 2:
A design process in which the loads or actions and the strengths of materials and sections are represented by their known, or postulated distributions, (defined in terms of relevant parameters such as types, mean and standard deviation) and some reliability level is accepted. It is thus a probabilistic design process.

Level 3:
A design process based upon an »exact» probabilistic analysis for the entire structural system, using a full distributional approach, with safety levels based on some stated failure probability interpreted in the sense of relative frequency.

In connection with these quotations from CEB-volume I a number of explanatory comments has been given, including:

Level 1:
A design method in which appropriate levels of structural reliability are provided, on a structural element (member) basis, by the specification of a number of partial safety factors. - - - - - - - - - - -
- -
Clearly Level 1 is, and should continue to be, the basis for most design codes. Simplifications within Level 1 are possible and indeed desirable in the presentation of the various clauses.

Level 2:

- -
- - - - - Reliability levels can be defined only by safety indices or equivalent »operational» probabilities and not in a relative frequency sense. - - -
- Level 2 should be used principally in assessing appropriate values for the partial safety factors in Level 1; hence it is intended primarily as a tool for code drafting committees. In certain specialised fields it may be the most appropriate design procedure, e.g. pylons for overhead electricity cables.

As seen from the above it is assumed that the determination of structural reliability is made on the basis of a system of partial coefficients, meaning that the previously dominant system, calculation with permissible stresses, is abandoned.

The choice of the partial coefficient method is due to the fact that it is the simplest method by which a reasonably uniform safety level can be ensured. This cannot be achieved by the use of permissible stresses, as will immediately be seen by comparing the risk of failure for two prestressed concrete structures, one with dominating dead load and one with negligible dead load:

For prestressed concrete structures the necessary dimensions are, as is known, determined almost exclusively by the variable load, while the permanent load is of only secondary importance. (It was in fact the problems in connection with pre-stressed concrete which led to the experimental introduction of the partial coefficient system 30 years ago in Denmark).

The partial coefficient method

In this section a very brief description of the partial coefficient method will be given.

It is assumed that a calculation model for the structure has been set up, and that the condition for the actual limit state not being exceeded is written as

$$\theta(G, Q_1, Q_2 \ldots, f_1, f_2, \ldots, a \ldots, \mu \ldots, C \ldots) \geqslant 0 \qquad (1)$$

where G denotes the dead load and Q_1 and Q_2 denote variable actions. f_1 and f_2 are characteristic strength parameters. a represents the geometric parameters. μ are quantities covering the uncertainties of the calculation model. C are constants including preselected design constraints.

The design criterion will then be

$$\theta(G_d, Q_{1d}, Q_{2d}, \ldots, f_{1d}, f_{2d}, \ldots, a_d \ldots, \mu_d \ldots, C \ldots) \geqslant 0 \qquad (2)$$

Subscript d denotes »design»-value.

The design value of the actions are obtained by multiplication of the respective actions with partial coefficients:

$$G_d = \gamma_g G,\ Q_{1d} = \gamma_{q1} Q_1, Q_{2d} = \gamma_{q2} Q_2 \ldots \quad (3)$$

The design values are obtained from the characteristic values by division by partial coefficients:

$$f_{1d} = f_1/\gamma_{m1},\ f_{2d} = f_2/\gamma_{m2} \ldots \qquad (4)$$

In practice, the difference between the partial coefficient method and the permissible stress method is merely that multiplied design load values are used in the former. It is therefore surprising that it is commonly thought to be very complicated.

The reason might be that in CEB-volume I the description of the system has been interwoven with a description of the complicated load- and load-combination system which has been chosen. This system is used by a few countries already - in connection with permissible stresses. In the series of consequence calculations carried out in connection with CEB-concrete most countries have pointed out that it greatly increases the calculation work because of the many load cases, and nothing much is gained. A simplification is therefore absolutely necessary.

Use of level-2 and level-3 methods

As seen from the description of the different levels it is assumed that level 2 can be used principally for calibration of the partial coefficients to ensure a uniform safety level. Level 2 will be directly applicable only in exceptional cases. In fact, only a few countries intend to mention the possibility at all in their codes.

It might seem peculiar that calibration is not attempted by a level-3 method. By this method the risk of failure of a structure can be determined, a value which is directly comprehensible and fit for e.g. economical optimizations.

The primary reason is the so-called tail-sensitivity problem, i.e. the calculated risks of failure are very sensitive to the distribution functions for the extreme values (high load values/low strength values). For most materials the distribution function is not known with sufficient accuracy and as for the actions there is no possibility of ever knowing them. Therefore, postulated distribution functions must be set up and the calculated risks of failure become formal measures of risk. They are purely theoretically-calculated quantities which can be used to compare structures, in which some of the parameters vary, and also, they can be used to calibrate the partial coefficients, but they cannot be identified with the frequency with which accidents or collapse occur in practice.

In this connection it is pointed out that exceeding an ultimate limit state does not necessarily cause collapse of the structure.

The results are usually given in the form of a safety index β and not as a probability of failure in order to signify that the calculations are formal.

A primitive description of the safety index can be given in connection with equation (1). Since the variables are random variables, θ is also a random variable with the mean value θ_{mean} and the standard deviation s. Then the safety index is

$$\beta = \theta_{mean}/s \tag{5}$$

It can be shown*) that the safety index is only a meaningful measure of safety if the basic variables are normally distributed or belong to the normal family of distributions. Therefore, log-normal distribution of strengths and normal distribution of actions are assumed in the NKB-proposal.

For problems with many random variables the tail-sensitivity phenomenon decreases. On the other hand, however, the necessary integrations of probability densities become an impractical task except for very special cases.

Though it is indeed possible in individual cases to calculate something approaching a true probability of failure by level-3 methods the calculations are so complicated and the results so impossible to generalize that they are useless for most practical purposes.

Example of code text

From various quarters doubts have been expressed whether CEB-volume I etc. is a suitable basis for the general sections of a user-oriented code, and especially, whether the system of partial coefficients is so simple that it is reasonable to use it.

To show that the doubts are groundless a proposal for the framing of the safety chapter of a structural timber code is given in the following.

The proposal is written so that it can be fitted directly into CIB-Structural Timber Design Code, whose definition of characteristic material parameters, i.e. lower 5-percentile values, is used.

A few comments on the proposal are given below.

In the code text proper, no values of the partial coefficients have been given. Their determination is a national matter based on a socio-economic optimization, taking into account the national levels of welfare, wages, material prices, etc. In a guiding text information is given of the partial coefficients proposed by CEB, in CEB-volume I and CEB-concrete, and by NKB.

For the actions the most significant deviation is that the NKB-proposal gives the value of the partial coefficient equal to 1.0 for permanent load in almost all cases. The primary reason for the choice of this value, which by the way facilitates the calculations considerably, is that this is the only possibility of getting a consistent design system which can also be used in foundation engineering where the earth on one hand is a building material and on the other is a significant loadgiving element. This is a fact which has often been demonstrated by foundation engineers, but since they have not been involved in the CEB-work it has been disregarded.

The recommendation to use a differentiated safety level based on the consequences of failure has been utilized (see section 3.3.2), a possibility which has been recommended in all the proposals, but has not, however, been used in CEB-concrete. From comparisons it has been assumed that CEB-concrete corresponds to failure-class »serious».

The CEB-value for γ_m corresponds to using 10-percentiles for concrete. For timber, where 5-percentiles are used a CEB-value of $\gamma_m = 1.4$ is deemed suitable.

For most structures the dimensions are proportional to

$$\alpha = \gamma_m(\gamma_g G + \gamma_q Q) =$$

$$\gamma_g \gamma_m (G + Q)\left(1 + \frac{Q}{G+Q}\left(\frac{\gamma_q}{\gamma_g} - 1\right)\right)$$

α, according to the CEB- and NKB-proposals, is shown in the figure.

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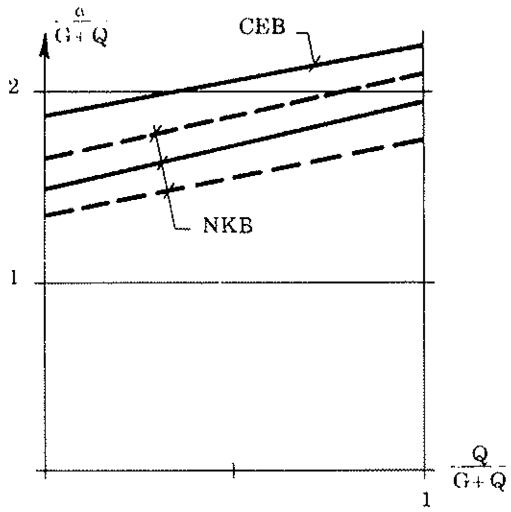
*) Ove Ditlevsen: Fundamentals of second moment structural reliability theory. International research seminar on safety of structures under dynamic loading. Trondheim, Norway, June 1977.

$\alpha/(G + Q)$ can be directly compared to the traditional safety values, provided uniform load regulations are used and provided 5-percentiles are used as characteristic values.



SECRETARY'S NOTE:

Chapter 3 of the CIB Structural Timber Design Code, which formed an annex to this paper, is reproduced elsewhere in these proceedings as Paper CIB-W18/11-100-3

CIB-W18/11-4-1

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WORKING COMMISSION W18 - TIMBER STRUCTURES

ANALYSIS OF PLYWOOD STRESSED SKIN PANELS WITH RIGID OR SEMI-RIGID CONNECTIONS

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ABSTRACT

This paper gives a method of analysis for both single and double skin stressed skin panels, with either rigid or semi rigid connections, which is suitable for hand calculation using an electronic calculator. By means of numerical examples it is demonstrated that there is good agreement between the method of analysis presented and alternative methods of analysis formulated by other workers.

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CONTENTS

NOTATION

a_i	$= \left(\frac{E_{yi}}{2G_{xyi}} - \nu_{xyi}\right)$
A_{ei}	= effective area of plate for skin i
A_s	= area of stringer
b	= breadth of stringer
b_e	= effective breadth of stringer (double skin panels)
b_N	= width of fastener pattern
B	= breadth of plate
B_{ei}	= effective breadth of plate for skin i
d	= depth of stringer
E_s	= Young's modulus for stringer, parallel to grain
E_{yi}	= Young's modulus for skin i in the Y direction
E_sI_s	= local flexural rigidity for stringer
$E_{yi}I_{pi}$	= local flexural rigidity for skin i
EI	$= E_sI_s + \sum E_{yi}I_{pi}$
EI_B	= flexural rigidity of the equivalent beam
G_{xyi}	= shear modulus for skin i, parallel and perpendicular to the Y direction
h	$= d + t_1/2 + t_2/2$
k_i	= slip modulus for fasteners connecting skin i to the stringers
L	= effective span of panel
L_s	= clear spacing between stringers
M	= bending moment at y due to applied load
M_o	= bending moment at $y = 0$ due to applied load
ν_{xyi} & ν_{yxi}	= Poisson's ratios for skin i, where the second subscripts denote the direction of the force
N_s	= number of stringers in the panel
p	= transverse load/mm run/ repeating unit
q_i	= shear flow at the interfaces between skin i and the stringers at y
q_{imax}	= maximum shear flow at the interfaces between skin i and the stringers (at $y = \pm L/2$)
s_i	= spacing of fasteners connecting skin i to the stringers
$\sigma_{pi\,omax}$	= maximum normal stress in skin i, parallel to stringers
σ_{somax}	= maximum normal stress in stringers, parallel to grain
t_i	= thickness skin i
τ_{yzsmax}	= maximum horizontal shear stress in stringers (at $y = \pm L/2$)
V	= vertical shear force due to applied load at y
V_{max}	= maximum vertical shear force due to applied load (at $y = \pm L/2$)
w	= transverse panel deflection at y
w_o	= transverse panel deflection at $y = 0$
$\bar{z}$	= vertical distance from the neutral axis to the top extreme fibres of skin 1
z_i	= vertical distance from the neutral axis to the mid plane of skin i
Z_i	= vertical distance from the mid plane of the stringer to the mid plane of skin i

1.0 Introduction

Plywood stressed skin panels consist of plywood sheets glued or mechanically fastened to the top or to both top and bottom surfaces of longitudinal timber stringers. The whole assembly acts as an integral section to resist bending, provided that the joints between the plywood and stringers are sufficiently rigid to prevent excessive slip due to longitudinal shear force induced between the plywood and the stringers. Structural economies are made possible by the utilisation of both plywood and stringers to resist bending. The size of a stringer for a given span can be less than that required for a simple joist, or a lower grade material can be used.

This paper presents a method of analysis for both single and double skin stressed skin panels, with either rigid or semi rigid connections, which is suitable for hand calculation using an electronic calculator. The analysis can be used to solve for panel deflections, and for stresses in the plywood skin(s) and timber stringers. Panels assembled by gluing are assumed to have rigid connections, and panels assembled using nails or staples are assumed to have semi rigid connections.

Using numerical examples it is demonstrated in Section 4.0 that there is good agreement between the method of analysis presented in Section 3.0 and alternative methods of analysis formulated by other workers.

Fig.1 shows the general arrangements for single and double skin stressed skin panel assemblies. Further details of construction and manufacture are given in TRADA Information Bulletin E/IB/22 {9}.

2.0 Approximate Method for Determining the Effective Breadth of Plywood Skins in Stressed Skin Panels

In the analysis which follows each skin of a stressed skin panel is assumed to act as an orthotropic plate in a state of plane stress. The distribution of stress across the width of the skin is therefore independent of its thickness.

In elementary beam theory it is assumed that normal stresses are evenly distributed across the width of a beam. The true normal stress distribution across the width of the skin of a stressed skin panel is of the form shown in Fig.2. The deviation of the normal stress distribution across the width of the skin from that predicted by elementary beam theory is known as "shear lag". The net effect of "shear lag" is a reduction in the normal stresses in the skin at locations on the cross-sections remote from the stringers, as compared with those predicted by elementary beam theory, and an increase in transverse panel displacements. A convenient representation for design purposes is to consider a stressed skin panel as consisting of a series of parallel T or I beams, each flange of these beams consisting of a width of uniformly stressed plate which, when acting in conjunction with the associated stringer contributes the same amount of flexural rigidity to the panel as would the whole of the corresponding width of non-uniformly stressed skin. The width of this notional uniformly stressed plate is termed its effective breadth, B_e.

The expressions for effective breadth, (18), (19) and (19A), are for simply supported panels with a uniformly distributed transverse load.

2.1 Analysis of Plane Stress

Consideration of the equilibrium of the element of plate $\delta x \delta y$ shown in Fig.3 yields;

$$\frac{\partial N_x}{\partial x} + \frac{\partial N_{yx}}{\partial y} = 0$$

and

$$\frac{\partial N_y}{\partial y} + \frac{\partial N_{xy}}{\partial x} = 0 \qquad (1)$$

where: N_x and N_y are the normal forces per unit length of element in the X and Y directions respectively, and $N_{xy} (= N_{yx})$ is the shear force per unit length of element, ($N = \sigma t$, where t = plate thickness, and σ denotes stress per unit area).

Equations (1) are satisfied by a function $\phi(x,y)$, Airy's stress function, if it is defined by;

$$N_x = \frac{\partial^2\phi}{\partial y^2} ; \quad N_y = \frac{\partial^2\phi}{\partial x^2} ; \quad N_{xy} = -\frac{\partial^2\phi}{\partial x \partial y} \qquad \text{———— (2)}$$

If the point (x,y) undergoes displacements u and v in the directions of X and Y respectively, Fig.4, the strains are given by;

$$\varepsilon_x = \frac{\partial u}{\partial x} ; \quad \varepsilon_y = \frac{\partial v}{\partial y} ; \quad \gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \qquad \text{———— (3)}$$

where: ε_x and ε_y are the normal strains in the X and Y directions respectively, and γ_{xy} is the shear strain.

Eliminating u and v the equation of compatibility of strains becomes:

$$\frac{\partial^2\varepsilon_x}{\partial y^2} + \frac{\partial^2\varepsilon_y}{\partial x^2} - \frac{\partial^2\gamma_{xy}}{\partial x \partial y} = 0 \qquad \text{———— (4)}$$

The equations of elasticity for the plate are;

$$\begin{aligned} \varepsilon_x &= \frac{1}{t}\left(\frac{N_x}{E_x} - \nu_{xy}\frac{N_y}{E_y}\right) \\ \varepsilon_y &= \frac{1}{t}\left(\frac{N_y}{E_y} - \nu_{yx}\frac{N_x}{E_x}\right) \\ \gamma_{xy} &= \frac{N_{xy}}{t\,G_{xy}} \end{aligned} \qquad \text{———— (5)}$$

where: E_x and E_y are Young's Modulus for the plate in the X and Y directions respectively, ν_{xy} and ν_{yx} are Poisson's Ratios (where the second subscript denotes the direction of the force), and G_{xy} is the shear modulus.

Substituting (2) and (5) in (4) yields the basic differential equation of plane stress in an orthotropic plate;

$$\frac{\partial^4\phi}{\partial x^4} + 2a\frac{\partial^4\phi}{\partial x^2\partial y^2} + b\frac{\partial^4\phi}{\partial y^4} = 0 \qquad \text{———— (6)}$$

where:

$$\frac{\nu_{xy}}{E_y} = \frac{\nu_{yx}}{E_x} , \qquad \frac{\partial^4\phi}{\partial x^2\partial y^2} = \frac{\partial^4\phi}{\partial y^2\partial x^2} ,$$

$$a = \left(\frac{E_y}{2G_{xy}} - \nu_{xy}\right) , \quad \text{and} \quad b = \frac{E_y}{E_x}$$

2.2 Effective Breadth of Plate for a Simply Supported Panel, with Multiple Stringers, with a Uniformly Distributed Transverse Load

Consider the orthotropic plate in Fig.5 loaded with edge shear forces:

$$N_{xy} = \pm ky \text{ , at } x = \pm B/2 \qquad (7)$$

The edge shear forces approximate to those developed at the edges of each skin of a stressed skin panel, with multiple stringers, which is simply supported at both ends and carries a uniformly distributed transverse load, provided it is assumed plane sections in the skin remain approximately plane after bending, and plane sections in the stringers remain plane after bending.

Assume Airy's stress function $\phi(x,y)$ is a fourth order polynomial in y and x , Gubenko {5}, of the form:

$$\phi = C_1 y^4 + C_2 x^4 + C_3 y^2 + C_4 x^2 + C_5 y^2 x^2 \qquad (8)$$

Substitution of the derivatives of ϕ in (2) yields:

$$\begin{aligned} N_y &= 12 C_2 x^2 + 2C_4 + 2C_5 y^2 \\ N_x &= 12 C_1 y^2 + 2C_3 + 2C_5 x^2 \\ N_{xy} &= -4 C_5 y x \end{aligned} \qquad (9)$$

Equating the expressions for N_{xy} from (7) and (9)

$$N_{xy} = N_{B/2\,y} = ky = -2C_5 By$$

therefore:

$$C_5 = -k/2B \qquad (10)$$

It is assumed that $N_x = 0$ at $x = \pm B/2$ for all values of y ; therefore:

$$C_1 = 0$$

Substituting in (9) for C_1 and C_5 yields;

$$N_x = 0 = 2C_3 - kB/4 \quad , \text{at } x = \pm B/2$$

therefore:

$$C_3 = kB/8 \qquad (11)$$

Substitution of the derivatives of ϕ in (6) yields;

$$8C_5 a + 24 C_2 = 0$$

and substituting for C_5 from (10):

$$C_2 = ka/6B \qquad (12)$$

At $y = 0$, the total normal force F_0 is given by;

$$F_0 = \int_{-B/2}^{B/2} 12 C_2 x^2 + 2C_4 \,] dx = C_2 B^3 + 2C_4 B \qquad (13)$$

this must equal the load applied at the edges by the shear forces, therefore:

$$F_0 = 2\int_0^{L/2} N_{B/2\,y}\, dy = kL^2/4 \qquad (14)$$

Equating (13) and (14) and substituting for C_2 from (12) yields:

$$C_4 = kL^2/8B - kaB/12 \qquad (15)$$

When $y = 0$ substitution for C_2, from (12), & C_4, from (15), in (9) yields:

$$N_{yo} = 2kax^2/B + kL^2/4B - kaB/6 \qquad (16)$$

N_{yo} has its greatest value when $x = \pm B/2$ therefore:

$$N_{yomax} = kaB/3 + kL^2/4B \qquad (17)$$

In the case where the skin is rigidly connected to the stringers, Fig.6, $B = L_s$ and the effective breadth at $y = 0$ is given by:

$$B_e = b + F_o/N_{yomax} = b + kL^2/4\left(\frac{kaL_s}{3} + \frac{kL^2}{4L_s}\right) = b + \frac{L}{\frac{4a}{3}\left(\frac{L_s}{L}\right) + \left(\frac{L}{L_s}\right)} \qquad (18)$$

where: b = width of stringer

L_s = clear spacing between stringers

In the case where the skin is semi rigidly connected to each stringer by multiple rows of equally spaced fasteners, with equal slip moduli, the effective breadth at $y = 0$ is given by:

$$B_e = b_N + \frac{L}{\frac{4a}{3}\left(\frac{B-b_N}{L}\right) + \left(\frac{L}{B-b_N}\right)} \qquad (19)$$

where: B = stringer spacing = $b + L_s$

b_N = width of fastener pattern i.e. distance across the stringer between outer rows of fasteners.

Where the skin is semi rigidly connected to each stringer by a single row of equally spaced fasteners, with equal slip moduli along the line of the vertical axis of the stringer, the effective breadth at $y = 0$ is found by putting $b_N = 0$ in (19) to give:

$$B_e = \frac{L}{\frac{4a}{3}\left(\frac{B}{L}\right) + \left(\frac{L}{B}\right)} \qquad (19A)$$

where: $B = b + L_s$

Fig.7 shows the relationship between B_e/B and B/L where B_e is calculated using (19A) for 6.5mm Canadian Douglas Fir Plywood.

APPENDIX I provides a quantitative comparison between (19A) and two alternative expressions for calculating the effective breadth at $y = 0$ for a simply supported panel carrying a uniformly distributed transverse load.

If the transverse load is applied directly to the skin the foregoing analysis is only valid if the transverse deformation of the skin is small.

2.3 Relationship Between Effective Breadth and y for a Simply Supported Panel, with Multiple Stringers, with a Uniformly Distributed Transverse Load

From (9)

$$N_y = 12C_2x^2 + 2C_4 + 2C_5y^2$$

Substitution for C_2, from (12), C_4, from (15), and C_5, from (10), yields:

$$N_y = 2kax^2/B + kL^2/4B - kaB/6 - ky^2/B \qquad (20)$$

For all values of y, (20) is a maximum at $x = \pm B/2$, therefore:

$$N_{ymax} = kaB/2 + kL^2/4B - kaB/6 - ky^2/B$$
$$= kaB/3 + kL^2/4B - ky^2/B \qquad (21)$$

The total normal force at y is given by:

$$F = 2\int_y^{L/2} N_{B/2\,y}\, dy$$

Substituting for $N_{B/2\,y}$ from (7):

$$F = 2\int_y^{L/2} ky\, dy = kL^2/4 - ky^2 \qquad (22)$$

If the skin is rigidly connected to the stringers $B = L_s$ and the effective breadth is given by

$$B_e = b + F/N_{ymax} = b + \left(\frac{L^2}{4} - y^2\right)\Big/\left(\frac{aL_s}{3} + \frac{L^2}{4L_s} - \frac{y^2}{L_s}\right) \qquad (23)$$

which reduces to (18) when $y = 0$.

Where the skin is semi rigidly connected to each stringer by multiple rows of equally spaced fasteners with equal slip moduli the effective breadth is given by

$$B_e = b_N + \left(\frac{L^2}{4} - y^2\right)\Big/\left(\frac{a(B-b_N)}{3} + \frac{L^2}{4(B-b_N)} - \frac{y^2}{(B-b_N)}\right) \qquad (24)$$

where: $B = b + L_s$

and this reduces to (19) when $y = 0$.

If the skin is semi rigidly connected to each stringer by a single row of equally spaced fasteners, $b_N = 0$ and the effective breadth is given by

$$B_e = \left(\frac{L^2}{4} - y^2\right)\Big/\left(\frac{aB}{3} + \frac{L^2}{4B} - \frac{y^2}{B}\right) \qquad (24A)$$

where: $B = b + L_s$

which reduces to (19A) when $y = 0$.

Fig.8 shows the relationship between effective breadth, B_e, and y for a plywood skin of a typical simply supported panel with a uniformly distributed transverse load.

It can be seen from Fig.8 that the effective breadth is approximately constant over the portion of the span where the bending moment is largest. It is convenient for the purposes of structural analysis to take the effective breadth as constant throughout the length of the panel. If it is further assumed that the effective breadth of the skin is equal to its effective breadth at mid-span, the overall effect upon the accuracy of a structural analysis is not excessive. Adoption of the above assumptions results in a slight underestimation of the transverse panel displacement and the normal stresses at locations adjacent to the panel supports and an overestimation of the maximum shear flow at the skin to stringer interfaces.

It is assumed in the following analysis, Section 3.0, that the effective breadth is constant throughout the length of the panel and equal in magnitude to the effective breadth at mid-span.

3.0 Analysis of Simply Supported Stressed Skin Panels with a Uniformly Distributed Transverse Load

In the analysis which follows a stressed skin panel is considered as consisting of a top and/or bottom plate in a state of plane stress and stringers stressed due to bending, with composite action enforced between each plate and the stringers. The state of stress in each plate is assumed to be described by equations (1) to (6).

It is assumed that the transverse panel deflection due to bending is small and small deflection theory is valid.

The analysis can be applied to stressed skin panels in which each plate is either rigidly or semi rigidly connected to the stringers. It is assumed that a rigid connection is achieved in panels assembled by gluing and that a semi rigid connection is achieved in panels assembled with nails or staples.

In panels with semi rigid connections slip will occur at the interfaces between the plates and stringers. The concept of slip modulus is used in the determination of the slip at the interfaces, and the following assumptions are made:

(a) shear transfer is continuous at all points in the interfaces.
(b) the fastener spacing is constant throughout the length of the panel, and all fasteners have the same slip modulus.
(c) the load/slip characteristic for the fasteners is linear.
(d) there is no separation between plates and stringers.

If Q is the load per fastener at slip γ and the fasteners are spaced s apart the load per unit length is given by:

$$q = Q/s \qquad (25)$$

and the slip modulus is given by:

$$k = qs/\gamma \qquad (26)$$

When $\gamma = 0$, k is infinite.

3.1 Single Skin Panel

Fig.9 shows the repeating unit of which single skin stressed skin panels with multiple stringers are composed.

Fig.10 shows an element of the repeating unit dy long. The state of stress in the components of the repeating unit can be represented by moments at the centroid of the plate M_{p1} and at the centroid of the stringer M_s and a couple of intensity $F_1 Z_1$. Consideration of the equilibrium of the element yields:

$$\frac{dF_1}{dy} = q_1$$

and
$$M = M_{p1} + M_s - F_1 Z_1 \qquad (27)$$

For equal curvature of the plate and stringer:

$$\frac{M_s}{E_s I_s} = \frac{M_{p1}}{E_{y1} I_{p1}} = \frac{M + F_1 Z_1}{EI} = \frac{1}{r} = \frac{d^2 w}{dy^2} \qquad (28)$$

where:

$EI = E_s I_s + E_{y1} I_{p1}$

r = radius of curvature for the panel

w = transverse panel displacement at y

E_s and E_{y1} are respectively the Young's Moduli for the stringer and plate in the Y direction and I_s and I_{p1} are respectively the local second moments of areas for the stringer and plate.

If the panel is simply supported at both ends and carries a uniformly distributed transverse load of p per unit length, M the moment at y due to the applied load is given by:

$$M = -\frac{pL^2}{8} + \frac{py^2}{2} \qquad (29)$$

The effective area of plate A_{e1} is given by:

$$A_{e1} = t_1 B_{e1} \qquad (30)$$

where: B_{e1} is given by (18) or (19) or (19A) depending upon the type of connection at the plate to stringer interfaces.

The effective section of the repeating unit can be represented by an equivalent T beam as shown by Fig.11.

The analysis for a single skin stressed skin panel with multiple stringers can be considered as a special case of the analysis for a double skin stressed skin panel with multiple stringers, described in Section 3.2, in which :

$$t_2 = A_{e2} = F_2 = M_{p2} = 0$$

Example 2, Section 4.2 shows the application of the theory developed in Section 3.2 to the analysis of a single skin stressed skin panel with multiple stringers.

3.2 Double Skin Panel

Fig.12 shows the repeating unit of which double skin stressed skin panels with multiple stringers are composed.

Fig.13 shows an element of the repeating unit dy long. The state of stress in the components of the repeating unit can be represented by moments at the centroids of the plates M_{p1} and M_{p2} and at the centroid of the stringer M_s and two couples of intensity F_1Z_1 and F_2Z_2 respectively.

Consideration of the equilibrium of the element yields:

$$\frac{dF_i}{dy} = q_i$$

and $$M = M_{p1} + M_{p2} + M_s - F_1 Z_1 - F_2 Z_2 \qquad (31)$$

For equal curvature of the plates and stringers:

$$\frac{M_s}{E_s I_s} = \frac{M_{p1}}{E_{y1} I_{p1}} = \frac{M_{p2}}{E_{y2} I_{p2}} = \frac{M + F_1 Z_1 + F_2 Z_2}{EI} = \frac{1}{r} = \frac{d^2 w}{dy^2} \qquad (32)$$

where: $EI = E_s I_s + E_{y1} I_{p1} + E_{y2} I_{p2}$

r = radius of curvature for the panel

w = transverse panel displacement at y

E_s ,E_{y1} ,and E_{y2} are respectively the Young's Moduli for the stringer, plate 1, and plate 2 in the Y direction and I_s ,I_{p1} ,and I_{p2} are respectively the local second moments of areas for the stringer, plate 1, and plate 2.

If the panel is simply supported at both ends and carries a uniformly distributed transverse load, of p per unit length, M the moment at y due to the applied load is given by:

$$M = -\frac{pL^2}{8} + \frac{py^2}{2} \qquad (33)$$

The effective area of plate i , A_{ei} , is given by :

$$A_{ei} = t_i B_{ei} \qquad (34)$$

where: B_{ei} is given by (18) or (19) or (19A) depending upon the type of connection at the plate to stringer interfaces. The effective section of the repeating unit can be represented by an equivalent I beam as shown by Fig.14.

Once the effective section of the equivalent I beam for the repeating unit has been determined, double skin stressed skin panels with rigid connections can be analysed using simple beam theory, as shown in Example 3, Section 4.3, and are not considered further in this section. Consideration is restricted to panels with semi rigid connections which are analysed by applying built-up beam theory to the equivalent I beam for the repeating unit.

The following two assumptions are made:

1) the direction of the applied load is such that the top flange, (flange 1) of the beam is loaded in compression, parallel to the y axis.

2) w, the transverse beam displacement at y, is the same for each of the beam components, (web, flange 1 and flange 2).

Fig.15 shows a portion of an equivalent I beam deflected under the action of a transverse load acting in the $-z$ direction.
Examination of Fig. 15 yields:

$$e_i + \gamma_i + \frac{dw}{dy} z_i = 0 \qquad (35)$$

where:
e_i = axial extension at the mid plane of flange i at y
γ_i = slip at interface i at y
z_i = vertical distance from the neutral axis to the mid plane of flange i

Combining (26) and (31):

$$q_i = \frac{dF_i}{dy} = \frac{k_i \gamma_i}{s_i} \qquad (36)$$

therefore:

$$F_i = \frac{k_i}{s_i} \int_y^{L/2} \gamma_i \, dy \qquad (37)$$

The axial strain at the mid plane of flange i, at y, is given by:

$$\frac{de_i}{dy} = \frac{F_i}{E_{yi} A_{ei}} = \frac{k_i}{s_i E_{yi} A_{ei}} \int_y^{L/2} \gamma_i \, dy \qquad (38)$$

therefore:

$$\frac{d^2 e_i}{dy^2} = - \frac{k_i \gamma_i}{s_i E_{yi} A_{ei}} \qquad (39)$$

Rearranging (39) yields:

$$\gamma_i = - \frac{s_i E_{yi} A_{ei}}{k_i} \frac{d^2 e_i}{dy^2} \qquad (40)$$

Substituting in (35) for γ_i from (40) yields:

$$e_i - \frac{s_i E_{yi} A_{ei}}{k_i} \frac{d^2 e_i}{dy^2} + \frac{dw}{dy} z_i = 0 \qquad (41)$$

The following sinusoidal loading and displacement definitions are assumed:

$$M = M_o \cos \frac{\pi y}{L} \qquad (42)$$

$$e_i = e_{iL/2} \sin \frac{\pi y}{L}$$

$$\frac{de_i}{dy} = \frac{\pi}{L} e_{iL/2} \cos \frac{\pi y}{L}$$

$$\frac{d^2 e_i}{dy^2} = -\frac{\pi^2}{L^2} e_{iL/2} \sin \frac{\pi y}{L} \qquad (43)$$

$$w = w_o \cos \frac{\pi y}{L}$$

$$\frac{dw}{dy} = -\frac{\pi}{L} w_o \sin \frac{\pi y}{L}$$

$$\frac{d^2 w}{dy^2} = -\frac{\pi^2}{L^2} w_o \cos \frac{\pi y}{L} \qquad (44)$$

where: $M_o = -\frac{pL^2}{8}$ as defined by (33)

$e_{iL/2}$ = axial extension at the mid plane of flange i at $y = L/2$

w_o = transverse beam displacement at $y = 0$

Fig. 16 shows the form of the assumed distribution of the transverse load, and the form of the assumed distribution of the transverse beam displacement.

APPENDIX II provides a quantitative comparison of the loading and displacement definitions for the beam, for a uniformly distributed transverse load, and for a sinusoidally distributed transverse load.

Substituting in (41) for e_i and $\frac{d^2 e_i}{dy^2}$, form (43), and for $\frac{dw}{dy}$, from (44), yields:

$$e_{iL/2} + \frac{\pi^2 e_{iL/2}\, s_i E_{yi} A_{ei}}{L^2 k_i} - \frac{\pi w_o z_i}{L} = 0$$

therefore:

$$e_{i\,L/2}\left[1+\frac{\pi^2 s_i E_{yi} A_{ei}}{L^2 k_i}\right] = \frac{\pi w_o z_i}{L}$$

$$e_{i\,L/2}\left[1+K_i\right] = \frac{\pi w_o z_i}{L} \qquad (45)$$

where:

$$K_i = \frac{\pi^2 s_i E_{yi} A_{ei}}{L^2 k_i}$$

Rearranging (45) yields:

$$\frac{e_{i\,L/2}}{w_o} = \frac{z_i \,\pi}{[1+K_i]\,L} \qquad (46)$$

Substituting into (32) the expression for M from (42), F_1 and F_2 from (38) and (43), and $\frac{d^2w}{dy^2}$ from (44) yields after rearrangement

$$M_o + E_{y1}A_{e1}\frac{\pi}{L}e_{1L/2}Z_1 + E_{y2}A_{e2}\frac{\pi}{L}e_{2L/2}Z_2 + \frac{\pi^2}{L^2}w_o EI = 0$$

therefore:

$$M_o = -\left[E_{y1}A_{e1}\frac{\pi}{L}e_{1L/2}Z_1 + E_{y2}A_{e2}\frac{\pi}{L}e_{2L/2}Z_2 + \frac{\pi^2}{L^2}w_o EI\right]$$

$$\frac{M_o}{w_o} = -\left[E_{y1}A_{e1}\frac{\pi}{L}\frac{e_{1L/2}}{w_o}Z_1 + E_{y2}A_{e2}\frac{\pi}{L}\frac{e_{2L/2}}{w_o}Z_2 + \frac{\pi^2}{L^2}EI\right] \qquad (47)$$

Substituting $\frac{e_{iL/2}}{w_o}$ from (46) and rearranging yields:

$$\frac{M_o}{w_o} = -\frac{\pi^2}{L^2}\left[\frac{E_{y1}A_{e1}Z_1 z_1}{1+K_1} + \frac{E_{y2}A_{e2}Z_2 z_2}{1+K_2} + EI\right] \qquad (48)$$

Let EI_B be the flexural rigidity of a simply supported beam with a sinusoidally distributed transverse load.

$$\frac{M}{EI_B} = \frac{1}{r} = \frac{d^2w}{dy^2} \qquad (49)$$

where M is defined by (42); therefore

$$\frac{d^2w}{dy^2} = \frac{M_o}{EI_B}\cos\frac{\pi y}{L}$$

$$\frac{dw}{dy} = \frac{L M_o}{\pi EI_B}\sin\frac{\pi y}{L} + C_1$$

When $y=0$, $\frac{dw}{dy}=0$, therefore $C_1=0$.

$$w = -\frac{L^2 M_o}{\pi^2 EI_B}\cos\frac{\pi y}{L} + C_2 \qquad (50)$$

When $y=\pm L/2$, $w=0$, therefore $C_2=0$.

The maximum transverse beam displacement is given by:

$$w_{max} = w_o = -\frac{L^2 M_o}{\pi^2 EI_B} \qquad (51)$$

Rearranging (51) yields:

$$\frac{M_o}{w_o} = -\frac{\pi^2 EI_B}{L^2} \qquad (52)$$

If EI_B is taken to be the flexural rigidity of the equivalent I beam; equating (48) and (52) yields:

$$EI_B = \frac{E_{y1}A_{e1}Z_1 z_1}{1+K_1} + \frac{E_{y2}A_{e2}Z_2 z_2}{1+K_2} + EI \qquad (53)$$

For a beam section which is symmetrical about its mid plane (53) reduces to:

$$EI_B = \frac{2E_y A_e Z^2}{1+K} + EI \qquad (54)$$

where:

$E_{y1} = E_{y2} = E_y$

$A_{e1} = A_{e2} = A_e$

$z_1 = Z_1 = z_2 = Z_2 = Z$

and $K_1 = K_2 = K$

Note: The expression for flexural rigidity given in (53) can be used in the determination of the Euler buckling load, P_E, for a built-up column with semi rigid connections. The bending moment at the buckling load is given by $-P_E w$, (the load is assumed to be applied in the plane of the neutral axis of the section), therefore M_o in (48) is given by: $M_o = -P_E w_o$, which yields:

$$P_E = \frac{\pi^2}{L^2}\left[\frac{E_{y1}A_{e1}Z_1 z_1}{1+K_1} + \frac{E_{y2}A_{e2}Z_2 z_2}{1+K_2} + EI\right] = \frac{\pi^2 EI_c}{L^2}$$

where: EI_c is the flexural rigidity of the built-up column. Therefore;

$$EI_c = \frac{E_{y1}A_{e1}Z_1 z_1}{1+K_1} + \frac{E_{y2}A_{e2}Z_2 z_2}{1+K_2} + EI$$

which is the same as the expression for the flexural rigidity of a built-up beam.

Before the maximum normal stresses, in the beam components can be determined the magnitudes of F_{10}, F_{20}, M_{p10}, M_{p20} and M_{s0} must be ascertained. Substituting w_o from (51) into (46) and rearranging yields:

$$e_{iL/2} = -\frac{z_i L M_o}{[1+K_i]\pi EI_B} \qquad (55)$$

Substituting (55) and (43) in (38) and rearranging, yields:

$$F_i = -\frac{z_i M_o E_{yi} A_{ei}}{[1+K_i] EI_B} \cos\frac{\pi y}{L} \qquad (56)$$

Therefore:

$$F_{io} = -\frac{z_i M_o E_{yi} A_{ei}}{[1+K_i]EI_B} \qquad (57)$$

M_{p10}, M_{p20}, and M_{s0} are found by substituting for F_{10} and F_{20}, from (57), in (32).

The maximum normal stresses in the web and flanges of the beam are given by

$$\bar{\sigma}_{so} = -\frac{F_{10}+F_{20}}{A_s} \mp \frac{M_{so} d}{2I_s}$$

and

$$\bar{\sigma}_{pio} = \frac{F_{io}}{A_{ei}} \mp \frac{M_{pio} t_i}{2 I_{pi}} \qquad (58)$$

where: $\bar{\sigma}_{so}$ and $\bar{\sigma}_{pio}$ are the normal stresses in the extreme fibres of the web and flange i respectively at $y = 0$.

The maximum shear flow, q_{imax}, at the interface between the web and flange i can be found by the method described below.

Combining (36) and (38):

$$q_i = \frac{dF_i}{dy} = \frac{d}{dy}\left(\frac{de_i}{dy}\right) E_{yi} A_{ei}$$
$$= \frac{d^2 e_i}{dy^2} E_{yi} A_{ei} \qquad (59)$$

Substituting for $\frac{d^2 e_i}{dy^2}$, from (43), and for $e_{iL/2}$, from (55), yields:

$$q_i = \frac{\pi z_i M_o E_{yi} A_{ei}}{L[1+K_i] E I_B} \sin \frac{\pi y}{L}$$

Inserting F_{io} from (57):

$$q_i = -\frac{\pi F_{io}}{L} \sin \frac{\pi y}{L} \qquad (60)$$

Therefore q_{imax} occurs at $y = \pm L/2$.

$$q_{imax} = \mp \frac{\pi F_{io}}{L} \qquad [y = \pm L/2] \qquad (61)$$

Equations (60) and (61) describe the shear flow at the interface between the web and flange i for a beam with a sinusoidally distributed transverse load. Corresponding equations for a beam with a uniformly distributed transverse load may be derived as follows:

From (22)

$$F_i = k_i L^2/4 - k_i y^2$$

, for a uniformly distributed transverse load; and from (31)

$$q_i = \frac{dF_i}{dy}$$

therefore:

$$q_i = -2k_i y \qquad (59A)$$

The value of k_i is found using (14) in which:

$$F_{io} = k_i L^2/4$$

Combining (14) and (59A) yields:

$$q_i = -\frac{8F_{io}\,y}{L^2} \qquad (60A)$$

Therefore q_{imax} occurs at $y = \pm L/2$.

$$q_{imax} = \mp \frac{4F_{io}}{L} \qquad [y = \pm L/2] \qquad (61A)$$

Comparison of (60) with (60A) and (61) with (61A) serves to illustrate that although the assumption of sinusoidal transverse displacement and transverse load distributions yields acceptable approximations for the purpose of determining transverse deflection and normal stress for a simply supported beam with a uniformly distributed transverse load, it does not yield an acceptable approximation of the distribution of shear force along such a beam.

In order that a beam which is unsymmetrical about its mid plane might be analysed using equations (53) to (61A) it is necessary to determine the location of the neutral axis for the beam and hence the values of z_1 and z_2.

The neutral axis can be located as described below:
The location of the neutral axis of an equivalent I beam with semi rigid connections is constant throughout its length and can be found by considering the normal stress distribution in the beam at $y = 0$.
The total normal compressive force, C, and the total normal tensile force, T, at the cross-section $y = 0$ are given by:

$$C = F_{10} - \frac{M_0 b (z_1 - t_1/2)^2 E_s}{2\, E I_B} \qquad (62)$$

and

$$T = F_{20} + \frac{M_0 b (z_2 + t_2/2)^2 E_s}{2\, E I_B} \qquad (63)$$

Substituting for F_{10} and F_{20} from (57) yields:

$$C = -\frac{M_0}{E I_B}\left[\frac{z_1 E_{y1} A_{e1}}{1+K_1} + \frac{b(z_1 - t_1/2)^2 E_s}{2}\right] \qquad (64)$$

and

$$T = -\frac{M_0}{E I_B}\left[\frac{z_2 E_{y2} A_{e2}}{1+K_2} - \frac{b(z_2 + t_2/2)^2 E_s}{2}\right] \qquad (65)$$

If the beam is subjected to bending forces only, there is no direct axial load on the beam and the total normal tensile force at a cross-section must equal the total normal compressive force at that cross-section.
Equating (64) and (65) and rearranging:

$$\frac{T}{C} = -1 \cdot 0 = \frac{\left[\frac{E_{y2} A_{e2} z_2}{1+K_2} - \frac{b E_s (z_2 + t_2/2)^2}{2}\right]}{\left[\frac{E_{y1} A_{e1} z_1}{1+K_1} + \frac{b E_s (z_1 - t_1/2)^2}{2}\right]}$$

$$= \frac{\phi_1 z_2 - \phi_2 (z_2 + t_2/2)^2}{\phi_3 z_1 + \phi_2 (z_1 - t_1/2)^2} \qquad (66)$$

where: $\phi_1 = \dfrac{E_{y2} A_{e2}}{1+K_2}$

$\phi_2 = \dfrac{b E_s}{2}$

and $\phi_3 = \dfrac{E_{y1} A_{e1}}{1+K_1}$

Geometrical considerations require that:

$$z_1 - z_2 = d + t_1/2 + t_2/2 = h \qquad (67)$$

therefore:

$$z_2 = z_1 - h \qquad (68)$$

Substituting in (66) for z_2 from (68) yields:

$$-1 \cdot 0 = \frac{\phi_1 (z_1 - h) - \phi_2 (z_1 - h + t_2/2)^2}{\phi_3 z_1 + \phi_2 (z_1 - t_1/2)^2}$$

Expanding squared terms and rearranging yields:

$$\zeta_1 = \frac{\phi_1 h + \phi_2 (h^2 - ht_2 + t_2^2/4 - t_1^2/4)}{\phi_1 + \phi_2 (2h - t_2 - t_1) + \phi_3}$$

Substituting for h, from (67), in bracketed terms, yields:

$$\zeta_1 = \frac{\phi_1 h + \phi_2 (d^2 + dt_1)}{\phi_1 + 2d\phi_2 + \phi_3} \qquad (69)$$

Similarly it can be shown that:

$$\zeta_2 = -\frac{\phi_3 h + \phi_2 (d^2 + dt_2)}{\phi_3 + 2d\phi_2 + \phi_1} \qquad (70)$$

Replacing ϕ_1, ϕ_2, and ϕ_3 in (69) and (70) yields:

$$\zeta_1 = \frac{\dfrac{E_{y2} A_{e2} h}{1 + K_2} + \dfrac{E_s A_s (d + t_1)}{2}}{\dfrac{E_{y2} A_{e2}}{1 + K_2} + E_s A_s + \dfrac{E_{y1} A_{e1}}{1 + K_1}} \qquad (69A)$$

$$\zeta_2 = -\frac{\dfrac{E_{y1} A_{e1} h}{1 + K_1} + \dfrac{E_s A_s (d + t_2)}{2}}{\dfrac{E_{y1} A_{e1}}{1 + K_1} + E_s A_s + \dfrac{E_{y2} A_{e2}}{1 + K_2}} \qquad (70A)$$

It can be seen that the above two expressions are modified forms of the expressions for a beam with rigid connections where ζ_1 and ζ_2 are found by taking moments of the beam cross-section about the mid planes of flanges 1 and 2 respectively.

In the case of a single skin stressed skin panel with multiple stringers $t_2 = A_{e2} = \phi_1 = 0$, and (69) and (70) reduce to:

$$\zeta_1 = \frac{\phi_2 (d^2 + dt_1)}{2d\phi_2 + \phi_3} \qquad (71)$$

$$\zeta_2 = -\frac{\phi_3 h + \phi_2 d^2}{\phi_3 + 2d\phi_2} \qquad (72)$$

4.0 Numerical Examples

In the examples which follow the skins of the stressed skin panels are of 'Selected Sheathing Tight Face, Select Sheathing and Sheathing Grades of Canadian Douglas Fir Plywood'. The grain of the external plies is assumed to be orientated parallel to the stringers. The dimensions and the elastic and shear moduli for the plywood are taken from 'Draft Amendment No.4 to CP.112: Part 2: 1971, Tables 43 and 44'.

The stringers of the stressed skin panels in the examples are assumed to be of softwood, Group S2 - GS Grade. The modulus of elasticity for the stringers is taken from 'Draft Amendment No.4 to CP112: Part 2: 1971, Table 11 (Dry Stresses and Elastic Moduli)'.

Note on the Effective Width of Stringers

Wardle and Peek {9} made the following recommendation concerning the design of stressed skin panels:-

> "In panels with both top and bottom skin the bridging and bottom skin act to distribute load laterally between stringers and the calculation can be based on section properties and loads for full panel width. In single skin panels the absence of a bottom skin renders the bridging ineffective in distributing the load transversely and the interior stringers of the panel carry a greater proportion of the load than the edge stringers. In this case the calculation should be based on a tee beam consisting of an interior stringer plus a plywood flange of width equal to the distance centre to centre of stringers, carrying the load appropriate to the flange width".

In the examples which follow the effective width of stringer for the repeating unit of a single skin stressed skin panel, Fig.9, is taken to be equal to the actual width of a stringer. For a double skin stressed skin panel with N_s stringers of equal dimensions the effective width of stringer, b_e, for the repeating unit, Fig.12, is given by:

$$b_e = \frac{N_s}{N_s - 1} b \qquad (73)$$

4.1 Example 1 Single Skin Stressed Skin Panel with Rigid Connections

Calculate, the maximum transverse panel deflection, the maximum normal stress in the skin and in the stringers, the maximum shear flow at the skin to stringer interfaces, and the maximum horizontal shear stress in the stringers, for the single skin stressed skin floor panel shown in Fig.17. The panel carries a uniformly distributed transverse load of 1.0 N/mm run/repeating unit acting vertically downwards, ($-z$ direction), and has an effective span of 5.0m.

B_{e1} the effective breadth of skin 1, is dependent upon the panel dimensions and the magnitudes of the elastic constants E_{y1}, G_{xy1}, and ν_{xy1}. It is assumed that ν_{xy1} is negligible compared to $E_{y1}/2G_{xy1}$ and that constant a_1 is given by:

$$a_1 = \frac{E_{y1}}{2\,G_{xy1}} = \frac{9300}{2 \times 500} = 9.30$$

For a panel where the skin is rigidly connected to the stringers B_{e1} is given by (18)

$$B_{e1} = b + \frac{L}{\frac{4a_1}{3}\left(\frac{L_s}{L}\right) + \left(\frac{L}{L_s}\right)}$$

$$= 35 + \frac{5000}{\frac{4 \times 9.30}{3}\left(\frac{365}{5000}\right) + \left(\frac{5000}{365}\right)} = \underline{377.376\,mm}$$

Location of Neutral Axis for Equivalent T Beam, Fig.11.

Taking moments about top edge of skin 1:

$$\bar{z} = \frac{\sum EAz}{\sum EA}$$

$$= \frac{9300 \times 377.376 \times 12.2^2 \times 0.5 + 8000 \times 145 \times 35 \times 84.7}{9300 \times 377.376 \times 12.2 + 8000 \times 145 \times 35}$$

$$= \underline{44.355\,mm}$$

Flexural Rigidities

$$E_s I_s = \frac{8000 \times 35 \times 145^3}{12} = 71.135 \times 10^9\ Nmm^2$$

$$E_{y1} I_{p1} = \frac{9300 \times 377.376 \times 12.2^3}{12} = 0.531 \times 10^9\ Nmm^2$$

$$EI = E_s I_s + E_{y1} I_{p1} = 71.666 \times 10^9\ Nmm^2$$

The flexural rigidity of the equivalent T beam, EI_B is given by:

$$EI_B = EI + \sum E_i A_i z_i^2$$

$$= 71.666 \times 10^9 + 9300 \times 377.376 \times 12.2 \times 38.255^2$$

$$+ 8000 \times 145 \times 35 \times 40.345^2$$

$$= \underline{200.412 \times 10^9\ Nmm^2}$$

Application of Simple Beam Theory to a Single Skin Stressed Skin Panel with Rigid Connections

For a stressed skin panel where the skin is rigidly connected to the stringers the structural calculations for the panel as a whole can be performed by applying simple beam theory to the equivalent T beam, Fig.11.

M_o ,bending moment at $y = 0$,is given by:

$$M_o = -\frac{pL^2}{8} = \frac{1 \times 5000^2}{8} = 3{\cdot}125 \times 10^6 \text{ Nmm}$$

Maximum Transverse Panel Deflection

$$w_{max} = w_o = \frac{5pL^4}{384\, EI_B}$$

$$= -\frac{5 \times 1 \times 5000^4}{384 \times 200{\cdot}412 \times 10^9}$$

$$= \underline{-40{\cdot}606 \text{ mm}}$$

Maximum Normal Stresses in Skin and Stringers

Skin 1

$$\sigma_{p1omax} = -\frac{M_o z_1 E_{y1}}{EI_B}$$

$$= -\frac{3{\cdot}125 \times 10^6 \times 44{\cdot}355 \times 9300}{200{\cdot}412 \times 10^9}$$

$$= \underline{-6{\cdot}432 \text{ N/mm}^2}$$

Fig.18 shows the distribution of the normal stress at the mid plane of skin 1, at $y = 0$,where σ_{p1o} is defined by (16).

Stringers

$$\sigma_{somax} = -\frac{M_o z_2 E_s}{EI_B}$$

$$= \frac{3{\cdot}125 \times 10^6 \times 112{\cdot}845 \times 8000}{200{\cdot}412 \times 10^9}$$

$$= \underline{14{\cdot}077 \text{ N/mm}^2}$$

Maximum Shear Flow at the Skin to Stringer Interfaces

$$q_{1max} = \frac{V_{max}\; EQ_{p1max}}{EI_B}$$

where:

$$V_{max} = \mp \frac{pL}{2} = \pm\ 2500 \text{ N} \qquad [y = \pm L/2]$$

and

$$EQ_{p1max} = E_{y1} A_{e1} z_1$$

$$= 9300 \times 377{\cdot}376 \times 12{\cdot}2 \times 38{\cdot}255$$

$$= 1{\cdot}638 \times 10^9 \text{ Nmm}$$

Therefore:

$$q_{1max} = \pm \frac{2500 \times 1{\cdot}638 \times 10^9}{200{\cdot}412 \times 10^9} = \underline{\pm\ 20{\cdot}433 \text{ N/mm}} \qquad [y = \pm L/2]$$

(see Fig.10 for sign convention)

Maximum Horizontal Shear Stress in the Stringers

$$\tau_{yzsmax} = \frac{V_{max}\ EQ_{smax}}{EI_B\ b}$$

where:

$$V_{max} = \mp \frac{pL}{2} = \pm\ 2500\,N \qquad [y = \pm L/2]$$

and $$EQ_{smax} = \frac{E_s b z_2^2}{2}$$

$$= \frac{8000 \times 35 \times 112{\cdot}845^2}{2}$$

$$= 1{\cdot}783 \times 10^9\ Nmm$$

Therefore:

$$\tau_{yzsmax} = \pm\ \frac{2500 \times 1{\cdot}783 \times 10^9}{200{\cdot}412 \times 10^9 \times 35}$$

$$= \underline{\pm\ 0{\cdot}635\ N/mm^2} \qquad [y = \pm L/2]$$

Summary Example 1

Table 1

Parameter	Simple Beam Theory (Equivalent T Beam)
w_{max}	- 40.606 mm
σ_{p10max}	- 6.432 N/mm^2
σ_{somax}	14.077 N/mm^2
q_{1max}	± 20.433 N/mm
τ_{yzsmax}	± 0.635 N/mm^2

4.2 Example 2 Single Skin Stressed Skin Panel with Semi Rigid Connections

Calculate, the maximum transverse panel deflection, the maximum normal stress in the skin and in the stringers, the maximum shear flow at the skin to stringer interfaces, and the maximum horizontal shear stress in the stringers, for the single skin stressed skin floor panel shown in Fig.17. The skin is semi rigidly connected to each stringer by a single row of equally spaced fasteners, with equal slip moduli, along the vertical axis of the stringer. The fasteners have a spacing, s_1, of 25mm and a slip modulus, k_1, of 329 N/mm. The panel carries a uniformly distributed transverse load of 1.0 N/mm run/repeating unit acting vertically downwards, ($-z$ direction), and has an effective span of 5.0 m.

B_{e1} the effective breadth of skin 1, is dependent upon the panel dimensions and the magnitudes of the elastic constants E_{y1}, G_{xy1}, and ν_{xy1}. It is assumed that ν_{xy1} is negligible compared to $E_{y1}/2G_{xy1}$ and that constant a_1 is given by:

$$a_1 = \frac{E_{y1}}{2G_{xy1}} = \frac{9300}{2 \times 500} = 9.30$$

For a panel where the skin is semi rigidly connected to each stringer by a single row of equally spaced fasteners, with equal slip moduli, along the vertical axis of the stringer B_{e1} is given by (19A).

$$B_{e1} = \frac{L}{\frac{4a_1}{3}\left(\frac{B}{L}\right)+\left(\frac{L}{B}\right)} \qquad \text{where } B = b + L_s = 400\,\text{mm}$$

$$= \frac{5000}{\frac{4 \times 9.30}{3}\left(\frac{400}{5000}\right)+\left(\frac{5000}{400}\right)} = \underline{370.590\,\text{mm}}$$

Geometric Properties

$$t_1 = 12.20\,\text{mm}, \quad t_2 = 0.0$$

$$Z_1 = 78.60\,\text{mm}, \quad Z_2 = -72.50\,\text{mm}$$

$$Z_1 - Z_2 = h = 151.10\,\text{mm}$$

$$A_{e1} = 370.590 \times 12.20 = 4521\,\text{mm}^2$$

$$A_{e2} = 0.0$$

$$A_s = 145 \times 35 = 5075\,\text{mm}^2$$

Flexural Rigidities

$$E_sI_s = \frac{8000 \times 35 \times 145^3}{12} = 71.135 \times 10^9\,\text{Nmm}^2$$

$$E_{y1}I_{p1} = \frac{9300 \times 370.590 \times 12.20^3}{12} = 0.522 \times 10^9\,\text{Nmm}^2$$

$$E_{y2}I_{p2} = 0.0$$

$$EI = E_sI_s + E_{y1}I_{p1} + E_{y2}I_{p2} = 71.657 \times 10^9\,\text{Nmm}^2$$

The equivalent beam Fig.11 is unsymmetrical about its mid plane therefore ζ_1 and ζ_2 must be determined using (71) and (72) or (67) and either (71) or (72).

Constants:

$$K_i = \frac{\pi^2 s_i E_{yi} A_{ei}}{L^2 k_i}$$

therefore:

$$K_1 = \frac{\pi^2 \times 25 \times 9300 \times 4521}{5000^2 \times 329}$$

$$= 1.2613$$

and $K_2 = 0.0 \qquad [A_{e2} = 0.0]$

$$\phi_1 = 0.0$$

$$\phi_2 = \frac{bE_s}{2} = \frac{35 \times 8000}{2} = 0.1400 \times 10^6 \ N/mm$$

$$\phi_3 = \frac{E_{y1} A_{e1}}{1 + K_1} = \frac{9300 \times 4521}{2.2613} = 18.5934 \times 10^6 \ N$$

From (71):

$$\zeta_1 = \frac{\phi_2 (d^2 + dt_1)}{2d\phi_2 + \phi_3}$$

$$= \frac{0.1400 \times 10^6 (145^2 + 145 \times 12.2)}{2 \times 145 \times 0.1400 \times 10^6 + 18.5934 \times 10^6}$$

$$= \underline{53.911 mm}$$

Substituting in (68) for ζ_1 yields :

$$\zeta_2 = \zeta_1 - h$$

$$= 53.911 - 151.1$$

$$= \underline{-97.189 mm}$$

Check value of ζ_2 by substituting in (72):

$$\zeta_2 = -\frac{\phi_3 h + \phi_2 d^2}{\phi_3 + 2d\phi_2}$$

$$= -\frac{18.5934 \times 10^6 \times 151.1 + 0.1400 \times 10^6 \times 145^2}{18.5934 \times 10^6 + 2 \times 145 \times 0.1400 \times 10^6}$$

$$= \underline{-97.189 mm}$$

The following values for ζ_1 and ζ_2 are used in the solution of this problem:

$$\zeta_1 = 53.9 mm$$

$$\zeta_2 = -97.2 mm$$

The flexural rigidity for the equivalent beam, EI_B, is found by substituting in (53):

$$EI_B = \frac{E_{y1} A_{e1} Z_1 \zeta_1}{1 + K_1} + \cancel{\frac{E_{y2} A_{e2} Z_2 \zeta_2}{1 + K_2}}^{\,0.0\ (A_{e2} = 0.0)} + EI$$

$$= \frac{9300 \times 4521 \times 78.6 \times 53.9}{2.2613} + 71.657 \times 10^9$$

$$= \underline{150.429 \times 10^9 \ Nmm^2}$$

M_o bending moment at $y = 0$ is given by:

$$M_o = -\frac{pL^2}{8} = \frac{1 \times 5000^2}{8} = 3{\cdot}125 \times 10^6 \text{ Nmm}$$

Substituting in (57) yields:

$$F_{10} = -\frac{\bar{z}_1 M_o E_{y1} A_{e1}}{[1+K_1] E I_B}$$

$$= -\frac{53{\cdot}9 \times 3{\cdot}125 \times 10^6 \times 9300 \times 4521}{2{\cdot}2613 \times 150{\cdot}429 \times 10^9}$$

$$= -20819 \text{ N}$$

Maximum Transverse Panel Deflection

The maximum transverse panel deflection is given by (51):

$$w_{max} = w_o = -\frac{L^2 M_o}{\pi^2 E I_B}$$

$$= -\frac{5000^2 \times 3{\cdot}125 \times 10^6}{\pi^2 \times 150{\cdot}429 \times 10^9}$$

$$= \underline{-52{\cdot}621 \text{mm}}$$

Maximum Normal Stresses in Skin and Stringers

The maximum normal stresses in the skin and stringers are given by (58):

$$\sigma_{p1omax} = \bar{\sigma}_{p1o} = \frac{F_{1o}}{A_{e1}} - \frac{M_{p1o} t_1}{2 I_{p1}}$$

and

$$\sigma_{somax} = \bar{\sigma}_{so} = -\frac{F_{1o}}{A_s} + \frac{M_{so} d}{2 I_s}$$

From (28):

$$\frac{M_{so}}{E_s I_s} = \frac{M_{p1o}}{E_{y1} I_{p1}} = \frac{M_o + F_1 Z_1}{EI}$$

Substituting for M_o and F_{1o} in (28) yields:

$$\frac{M_{so}}{E_s I_s} = \frac{M_{p1o}}{E_{y1} I_{p1}} = \frac{3{\cdot}125 \times 10^6 - 20819 \times 78{\cdot}6}{71{\cdot}657 \times 10^9} = 20{\cdot}774 \times 10^{-6} \text{ mm}^{-1}$$

Therefore:

$$\frac{M_{so}}{I_s} = 20{\cdot}774 \times 10^{-6} \times 8000 = 0{\cdot}1662 \text{ Nmm}^{-3}$$

$$\frac{M_{p1o}}{I_{p1}} = 20{\cdot}774 \times 10^{-6} \times 9300 = 0{\cdot}1932 \text{ Nmm}^{-3}$$

and

$$\sigma_{p1omax} = \frac{-20819}{4521} - \frac{0{\cdot}1932 \times 12{\cdot}2}{2}$$

$$= \underline{-5{\cdot}783 \text{ N/mm}^2}$$

$$\sigma_{somax} = \frac{20819}{5075} + \frac{0{\cdot}1662 \times 145}{2}$$

$$= \underline{16{\cdot}152 \text{ N/mm}^2}$$

Maximum Shear Flow at the Skin to Stringer Interfaces

The maximum shear flow at the skin to stringer interfaces is given by (61A):

$$q_{1max} = \mp \frac{4F_{10}}{L} \qquad [y = \pm L/2]$$

$$= \pm \frac{4 \times 20819}{5000}$$

$$= \underline{\pm 16.655 \text{ N/mm}} \qquad [y = \pm L/2]$$

(see Fig.10 for sign convention)

Maximum Horizontal Shear Stress in the Stringers

The maximum horizontal shear stress in the stringers is given by:

$$\tau_{yz\,smax} = \frac{V_{max}\, EQ_{smax}}{EI_B\, b}$$

where:

$$V_{max} = \mp \frac{pL}{2} = \pm 2500\text{N} \qquad [y = \pm L/2]$$

and

$$EQ_{smax} = \frac{E_s b z_2^2}{2}$$

$$= \frac{8000 \times 35 \times 97.2^2}{2}$$

$$= 1.323 \times 10^9 \text{ Nmm}$$

Therefore:

$$\tau_{yz\,smax} = \pm \frac{2500 \times 1.323 \times 10^9}{150.429 \times 10^9 \times 35}$$

$$= \underline{\pm 0.628 \text{ N/mm}^2} \qquad [y = \pm L/2]$$

Summary Example 2 — Table 2

Parameter	Panel Analysis (Equivalent T Beam)
w_{max}	- 52.621 mm
σ_{p10max}	- 5.783 N/mm^2
σ_{s0max}	16.152 N/mm^2
q_{1max}	± 16.655 N/mm
τ_{yzsmax}	± 0.628 N/mm^2

4.3 Example 3 Double Skin Stressed Skin Panel with Rigid Connections

Calculate, the maximum transverse panel deflection, the maximum normal stress in the skins and in the stringers, the maximum shear flows at the skin to stringer interfaces, and the maximum horizontal shear stress in the stringers, for the double skin stressed skin floor panel shown in Fig.19. The panel carries a uniformly distributed transverse load of 1.0 N/mm run/repeating unit acting vertically downwards, ($-z$ direction), and has an effective span of 5.0 m.

B_{ei} the effective breadth of skin i is dependant upon the panel dimensions and the magnitudes of the elastic constants E_{yi}, G_{Txyi}, and ν_{xyi}. It is assumed that ν_{xyi} is negligible compared to $E_{yi}/2G_{Txyi}$ and that constant a_i is given by:

$$a_i = \frac{E_{yi}}{2G_{Txyi}}$$

therefore:

$$a_1 = \frac{E_{y1}}{2G_{Txy1}} = \frac{8160}{2\times 500} = 8.16$$

and

$$a_2 = \frac{E_{y2}}{2G_{Txy2}} = \frac{9300}{2\times 500} = 9.30$$

The effective breadth of the stringers b_e is given by (73):

$$b_e = \frac{N_s}{N_s-1}b = \frac{4\times 35}{3} = 46.667\,\text{mm}$$

For a panel where the skins are rigidly connected to the stringers B_{ei} is given by (18).

$$B_{ei} = b + \frac{L}{\frac{4a_i}{3}\left(\frac{L_s}{L}\right)+\left(\frac{L}{L_s}\right)} \qquad \text{where: } b = b_e = 46.667\,\text{mm}$$

Therefore:

$$B_{e1} = 46.667 + \frac{5000}{\frac{4\times 8.16}{3}\left(\frac{365}{5000}\right)+\left(\frac{5000}{365}\right)} = \underline{391.664\,\text{mm}}$$

$$B_{e2} = 46.667 + \frac{5000}{\frac{4\times 9.30}{3}\left(\frac{365}{5000}\right)+\left(\frac{5000}{365}\right)} = \underline{389.043\,\text{mm}}$$

Location of Neutral Axis for Equivalent I Beam, Fig.14

Taking moments about top edge of skin 1:

$$\bar{z} = \frac{\sum EAz}{\sum EA}$$

$$= \frac{8160\times 391.664\times 18.6^2\times 0.5 + 8000\times 145\times 46.667\times 91.1 + 9300\times 389.043\times 12.2\times 169.7}{8160\times 391.664\times 18.6 + 8000\times 145\times 46.667 + 9300\times 389.043\times 12.2}$$

$$= \underline{82.267\,\text{mm}}$$

Flexural Rigidities

$$E_sI_s = \frac{8000 \times 46{\cdot}667 \times 145^3}{12} = 94{\cdot}847 \times 10^9 \text{ Nmm}^2$$

$$E_{y1}I_{p1} = \frac{8160 \times 391{\cdot}664 \times 18{\cdot}6^3}{12} = 1{\cdot}714 \times 10^9 \text{ Nmm}^2$$

$$E_{y2}I_{p2} = \frac{9300 \times 389{\cdot}043 \times 12{\cdot}2^3}{12} = 0{\cdot}547 \times 10^9 \text{ Nmm}^2$$

$$EI = E_sI_s + E_{y1}I_{p1} + E_{y2}I_{p2} = 97{\cdot}108 \times 10^9 \text{ Nmm}^2$$

The flexural rigidity of the equivalent I beam, EI_B, is given by:

$$EI_B = EI + \sum E_i A_{ei} z_i^2$$

$$= 97{\cdot}108 \times 10^9 + 8160 \times 391{\cdot}664 \times 18{\cdot}6 \times 72{\cdot}967^2$$
$$+ 8000 \times 145 \times 46{\cdot}667 \times 8{\cdot}833^2$$
$$+ 9300 \times 389{\cdot}043 \times 12{\cdot}2 \times 87{\cdot}433^2$$

$$= \underline{755{\cdot}265 \times 10^9 \text{ Nmm}^2}$$

Application of Simple Beam Theory to a Double Skin Stressed Skin Panel with Rigid Connections

M_o, bending moment at $y = 0$, is given by:

$$M_o = -\frac{pL^2}{8} = \frac{1 \times 5000^2}{8} = 3{\cdot}125 \times 10^6 \text{ Nmm}$$

Maximum Transverse Panel Deflection

$$w_{max} = w_o = \frac{5pL^4}{384\ EI_B}$$

$$= -\frac{5 \times 1 \times 5000^4}{384 \times 755{\cdot}265 \times 10^9}$$

$$= \underline{-10{\cdot}775 \text{ mm}}$$

Maximum Normal Stresses in Skins and Stringers

Skin 1

$$\sigma_{p1omax} = -\frac{M_o \bar{z} E_{y1}}{EI_B}$$

$$= -\frac{3{\cdot}125 \times 10^6 \times 82{\cdot}267 \times 8160}{755{\cdot}265 \times 10^9}$$

$$= \underline{-2{\cdot}778 \text{ N/mm}^2}$$

Skin 2

$$\sigma_{p2omax} = -\frac{M_o(\bar{z}-d-t_1-t_2)E_{y2}}{EI_B}$$

$$= \frac{3.125\times10^6\times93.533\times9300}{755.265\times10^9}$$

$$= \underline{3.599\ N/mm^2}$$

Stringers

$$\sigma_{somax} = -\frac{M_o(\bar{z}-d-t_1)E_s}{EI_B}$$

$$= \frac{3.125\times10^6\times81.333\times8000}{755.265\times10^9}$$

$$= \underline{2.692\ N/mm^2}$$

Maximum Shear Flows at the Skin to Stringer Interfaces

$$q_{imax} = \frac{V_{max}\,EQ_{pimax}}{EI_B}$$

where:

$$V_{max} = \mp\frac{pL}{2} = \pm\,2500\,N \qquad [y=\pm L/2]$$

and

$$EQ_{pimax} = E_{yi}A_{ei}z_i$$

which yields:

$$EQ_{p1max} = 8160\times391.664\times18.6\times72.967$$
$$= 4.338\times10^9\ Nmm$$

$$EQ_{p2max} = -9300\times389.043\times12.2\times87.433$$
$$= -3.859\times10^9\ Nmm$$

Therefore:

$$q_{1max} = \pm\frac{2500\times4.338\times10^9}{755.265\times10^9}$$

$$= \underline{\pm\,14.359\ N/mm} \qquad [y=\pm L/2]$$

and

$$q_{2max} = \mp\frac{2500\times3.859\times10^9}{755.265\times10^9}$$

$$= \underline{\mp\,12.774\ N/mm} \qquad [y=\pm L/2]$$

(see Fig.13 for sign convention)

Maximum Horizontal Shear Stress in the Stringers

$$\tau_{yzsmax} = \frac{V_{max}\, EQ_{smax}}{EI_B\, b_e}$$

where:

$$V_{max} = \mp \frac{pL}{2} = \pm 2500\,N \qquad [y = \pm L/2]$$

and

$$EQ_{smax} = E_{y1} A_{e1} z_1 + \frac{E_s b_e (\bar{z} - t_1)^2}{2}$$

$$= 8160 \times 391 \cdot 664 \times 18 \cdot 6 \times 72 \cdot 967$$
$$+ 8000 \times 46 \cdot 667 \times 63 \cdot 667^2 \times 0 \cdot 5$$
$$= 5 \cdot 094 \times 10^9\ Nmm$$

Therefore:

$$\tau_{yzsmax} = \pm \frac{2500 \times 5 \cdot 094 \times 10^9}{755 \cdot 265 \times 10^9 \times 46 \cdot 667}$$

$$= \underline{\pm 0 \cdot 361\ N/mm^2} \qquad [y = \pm L/2]$$

Summary Example 3 — Table 3

Parameter	Simple Beam Theory. (Equivalent I Beam)	C.O.F.I. Method
w_{max}	- 10.775 mm	- 10.561 mm
σ_{p10max}	- 2.778 N/mm^2	- 2.723 N/mm^2
σ_{p20max}	3.599 N/mm^2	3.527 N/mm^2
σ_{s0max}	2.692 N/mm^2	2.638 N/mm^2
q_{1max}	± 14.359 N/mm	± 14.409 N/mm
q_{2max}	∓ 12.774 N/mm	∓ 12.902 N/mm
τ_{yzsmax}	± 0.361 N/mm^2	± 0.360 N/mm^2

4.4 Example 4 Double Skin Stressed Skin Panel with Semi Rigid Connections

Calculate,the maximum transverse panel deflection, the maximum normal stress in the skins and in the stringers, the maximum shear flows at the skin to stringer interfaces, and the maximum horizontal shear stress in the stringers, for the double skin stressed skin floor panel shown in Fig.19. Both skins are semi ridigly connected to each stringer by a single row of equally spaced fasteners of equal slip moduli along the vertical axis of the stringer. The fasteners for both skins have a spacing of 25mm, ($s_1 = s_2 = 25$mm). The fasteners for the top skin have a slip modulus , k_1 ,of 494 N/mm, and the fasteners for the bottom skin have a slip modulus, k_2, of 329 N/mm. The panel carries a uniformly distributed transverse load of 1.0 N/mm run/repeating unit acting vertically downwards ($-z$ direction) and has an effective span of 5.0 m.

B_{ei} the effective breadth of skin i is dependent upon the panel dimensions and the magnitudes of the elastic constants E_{yi} , G_{xyi} , and ν_{xyi} .
It is assumed that ν_{xyi} is negligible compared to $E_{yi}/2G_{xyi}$, and that constant a_i is given by:

$$a_i = \frac{E_{yi}}{2G_{xyi}}$$

therefore:

$$a_1 = \frac{E_{y1}}{2G_{xy1}} = \frac{8160}{2 \times 500} = 8{\cdot}16$$

and

$$a_2 = \frac{E_{y2}}{2G_{xy2}} = \frac{9300}{2 \times 500} = 9{\cdot}30$$

B_{ei} is given by:

$$B_{ei} = \frac{L}{\frac{4a_i}{3}\left(\frac{B}{L}\right) + \left(\frac{L}{B}\right)} + \frac{b}{N_s - 1}$$

where:

$B = b + L_s$
N_s = number of stringers in panel

Therefore:

$$B_{e1} = \frac{5000}{\frac{4 \times 8{\cdot}16}{3}\left(\frac{400}{5000}\right) + \left(\frac{5000}{400}\right)} + \frac{35}{3}$$

$$= \underline{385{\cdot}627 \text{ mm}}$$

$$B_{e2} = \frac{5000}{\frac{4 \times 9{\cdot}30}{3}\left(\frac{400}{5000}\right) + \left(\frac{5000}{400}\right)} + \frac{35}{3}$$

$$= \underline{382{\cdot}257 \text{ mm}}$$

The effective breadth of the stringers b_e is given by (73):

$$b_e = \frac{N_s}{N_s - 1} b = \frac{4 \times 35}{3} = 46{\cdot}667 \text{ mm}$$

Geometric Properties

$$t_1 = 18.60 \text{ mm} \quad , \quad t_2 = 12.20 \text{ mm}$$

$$Z_1 = 81.80 \text{ mm} \quad , \quad Z_2 = -78.60 \text{ mm}$$

$$Z_1 - Z_2 = h = 160.40 \text{ mm}$$

$$A_{e1} = 385.627 \times 18.6 = 7173 \text{ mm}^2$$

$$A_{e2} = 382.257 \times 12.2 = 4664 \text{ mm}^2$$

$$A_s = 145 \times 46.667 = 6767 \text{ mm}^2$$

Flexural Rigidities

$$E_s I_s = \frac{8000 \times 46{\cdot}667 \times 145^3}{12} = 94{\cdot}847 \times 10^9 \text{ Nmm}^2$$

$$E_{y1} I_{p1} = \frac{8160 \times 385{\cdot}627 \times 18{\cdot}6^3}{12} = 1{\cdot}687 \times 10^9 \text{ Nmm}^2$$

$$E_{y2} I_{p2} = \frac{9300 \times 382{\cdot}257 \times 12{\cdot}2^3}{12} = 0{\cdot}538 \times 10^9 \text{ Nmm}^2$$

$$EI = E_s I_s + E_{y1} I_{p1} + E_{y2} I_{p2} = 97{\cdot}072 \times 10^9 \text{ Nmm}^2$$

The equivalent beam Fig.14 is unsymmetrical about its mid plane, therefore ζ_1 and ζ_2 must be determined using (69) and (70) or (67) and either (69) or (70).

Constants:

$$K_i = \frac{\pi^2 s_{ei} E_{yi} A_{ei}}{L^2 k_i}$$

where: s_{ei} = effective spacing of fasteners connecting skin i to the stringers

$$= \frac{N_s - 1}{N_s} s_i = \frac{3 \times 25}{4} = 18{\cdot}75 \text{ mm}$$

therefore:

$$K_1 = \frac{\pi^2 \times 18{\cdot}75 \times 8160 \times 7173}{5000^2 \times 494} = 0{\cdot}8771$$

and

$$K_2 = \frac{\pi^2 \times 18{\cdot}75 \times 9300 \times 4664}{5000^2 \times 329} = 0{\cdot}9759$$

$$\phi_1 = \frac{E_{y2} A_{e2}}{1 + K_2} = \frac{9300 \times 4664}{1{\cdot}9759} = 21{\cdot}9521 \times 10^6 \text{ N}$$

$$\phi_2 = \frac{b E_s}{2} = \frac{46{\cdot}667 \times 8000}{2} = 0{\cdot}1867 \times 10^6 \text{ N/mm}$$

$$\phi_3 = \frac{E_{y1} A_{e1}}{1 + K_1} = \frac{8160 \times 7173}{1{\cdot}8771} = 31{\cdot}1820 \times 10^6 \text{ N}$$

From (69):

$$\zeta_1 = \frac{\phi_1 h + \phi_2 (d^2 + d t_1)}{\phi_1 + 2d\,\phi_2 + \phi_3}$$

$$= \frac{21{\cdot}9521 \times 10^6 \times 160{\cdot}4 + 0{\cdot}1867 \times 10^6 (145^2 + 145 \times 18{\cdot}6)}{21{\cdot}9521 \times 10^6 + 2 \times 145 \times 0{\cdot}1867 \times 10^6 + 31{\cdot}1820 \times 10^6} = \underline{74{\cdot}107 \text{ mm}}$$

Substituting in (68) for z_1, yields:

$$\begin{aligned} z_2 &= z_1 - h \\ &= 74{\cdot}107 - 160{\cdot}4 \\ &= \underline{-86{\cdot}293\ mm} \end{aligned}$$

Check value of z_2 by substituting in (70).

$$z_2 = -\frac{\phi_3 h + \phi_2 (d^2 + dt_2)}{\phi_3 + 2d\phi_2 + \phi_1}$$

$$= -\frac{31{\cdot}1820 \times 10^6 \times 160{\cdot}4 + 0{\cdot}1867 \times 10^6 (145^2 + 145 \times 12{\cdot}2)}{31{\cdot}1820 \times 10^6 + 2 \times 145 \times 0{\cdot}1867 \times 10^6 + 21{\cdot}9521 \times 10^6} = \underline{-86{\cdot}293\ mm}$$

The following values for z_1 and z_2 are used in the solution of this problem:

$$z_1 = 74{\cdot}11\ mm$$

$$z_2 = -86{\cdot}29\ mm$$

The flexural rigidity for the equivalent beam , EI_B, is found by substituting in (53):

$$EI_B = \frac{E_{y1} A_{e1} Z_1 z_1}{1 + K_1} + \frac{E_{y2} A_{e2} Z_2 z_2}{1 + K_2} + EI$$

$$= \frac{8160 \times 7173 \times 81{\cdot}8 \times 74{\cdot}11}{1{\cdot}8771} + \frac{9300 \times 4664 \times 78{\cdot}6 \times 86{\cdot}29}{1{\cdot}9759} + 97{\cdot}072 \times 10^9$$

$$= \underline{434{\cdot}991 \times 10^9\ Nmm^2}$$

M_o, bending moment at $y = 0$, is given by :

$$M_o = -\frac{pL^2}{8} = \frac{1 \times 5000^2}{8} = 3{\cdot}125 \times 10^6\ Nmm$$

From (57);

$$F_{io} = -\frac{z_i M_o E_{yi} A_{ei}}{(1 + K_i)\, EI_B}$$

therefore:

$$F_{10} = -\frac{74{\cdot}11 \times 3{\cdot}125 \times 10^6 \times 8160 \times 7173}{1{\cdot}8771 \times 434{\cdot}991 \times 10^9}$$

$$= -16602\ N$$

and

$$F_{20} = \frac{86{\cdot}29 \times 3{\cdot}125 \times 10^6 \times 9300 \times 4664}{1{\cdot}9759 \times 434{\cdot}991 \times 10^9}$$

$$= 13608\ N$$

Maximum Transverse Panel Deflection

The maximum transverse panel deflection is given by (51):

$$w_{max} = w_o = -\frac{L^2 M_o}{\pi^2 E I_B}$$

$$= -\frac{5000^2 \times 3.125\times10^6}{\pi^2 \times 434.991\times10^9}$$

$$= \underline{-18.197\ mm}$$

Maximum Normal Stresses in Skins and Stringers

The maximum normal stresses in the skins and stringers are given by (58):

$$\sigma_{p1omax} = \bar{\sigma}_{p1o} = \frac{F_{1o}}{A_{e1}} - \frac{M_{p1o} t_1}{2 I_{p1}}$$

$$\sigma_{p2omax} = \bar{\sigma}_{p2o} = \frac{F_{2o}}{A_{e2}} + \frac{M_{p2o} t_2}{2 I_{p2}}$$

$$\sigma_{somax} = \bar{\sigma}_{so} = -\frac{F_{1o}+F_{2o}}{A_s} + \frac{M_{so} d}{2 I_s}$$

From (32):

$$\frac{M_{so}}{E_s I_s} = \frac{M_{p1o}}{E_{y1} I_{p1}} = \frac{M_{p2o}}{E_{y2} I_{p2}} = \frac{M_o + F_{1o} Z_1 + F_{2o} Z_2}{E I}$$

Substituting for M_o , F_{1o} , and F_{2o} in (32) yields:

$$\frac{M_{so}}{E_s I_s} = \frac{M_{p1o}}{E_{y1} I_{p1}} = \frac{M_{p2o}}{E_{y2} I_{p2}} = \frac{3.125\times10^6 - 16602\times81.8 - 13608\times78.6}{97.072\times10^9}$$

$$= 7.184\times10^{-6}\ mm^{-1}$$

Therefore:

$$\frac{M_{so}}{I_s} = 7.184\times10^{-6}\times 8000 = 57.472\times10^{-3}\ Nmm^{-3}$$

$$\frac{M_{p1o}}{I_{p1}} = 7.184\times10^{-6}\times 8160 = 58.621\times10^{-3}\ Nmm^{-3}$$

$$\frac{M_{p2o}}{I_{p2}} = 7.184\times10^{-6}\times 9300 = 66.811\times10^{-3}\ Nmm^{-3}$$

and

$$\sigma_{p1omax} = -\frac{16602}{7173} - \frac{58.621\times10^{-3}\times18.6}{2} = \underline{-2.860\ N/mm^2}$$

$$\sigma_{p2omax} = \frac{13608}{4664} + \frac{66.811\times10^{-3}\times12.2}{2} = \underline{3.325\ N/mm^2}$$

$$\sigma_{somax} = -\frac{-16602+13608}{6767} + \frac{57.472\times10^{-3}\times14.5}{2} = \underline{4.609\ N/mm^2}$$

Maximum Shear Flows at the Skin to Stringer Interfaces

The maximum shear flows at the skin to stringer interfaces are given by (61A):

$$q_{imax} = \mp\frac{4F_{io}}{L} \qquad [y=\pm L/2]$$

therefore:

$$q_{1max} = \pm\frac{4\times16602}{5000} = \underline{\pm 13.282\ N/mm} \qquad [y=\pm L/2]$$

and

$$q_{2max} = \mp\frac{4\times13608}{5000} = \underline{\mp 10.886\ N/mm} \qquad [y=\pm L/2]$$

(see Fig.13 for sign convention)

Maximum Horizontal Shear Stress in the Stringers

The vertical shear force per stringer due to normal stresses in the stringer is given by:

$$V_s = V\frac{EI_{ss}}{EI_B}$$

where: V = the vertical shear force per repeating unit at y,

and $$EI_{ss} = E_sI_s + b_e d\left(z_1 - t_1/2 - d/2\right)^2 E_s$$

$$= 94{\cdot}847\times10^9 + 46{\cdot}667\times145\times7{\cdot}69^2\times8000$$

$$= 98{\cdot}048\times10^9 \ \mathrm{Nmm}^2$$

$$V_{max} = \mp\frac{pL}{2} = \pm\ 2500\ \mathrm{N} \qquad [y = \pm L/2]$$

therefore:

$$V_{smax} = V_{max}\frac{EI_{ss}}{EI_B} = \pm\frac{2500\times98{\cdot}048\times10^9}{434{\cdot}991\times10^9}$$

$$= \underline{\pm\ 563{\cdot}5\ \mathrm{N}} \qquad [y = \pm L/2]$$

The maximum horizontal shear stress in the stringers is given by:

$$\tau_{yzsmax} = \frac{V_{smax}\,EQ_{ss}}{EI_{ss}\,b_e} + \frac{q_{1max}}{b_e}$$

where:

$$EQ_{ss} = \frac{b_e(z_1 - t_1/2)^2 E_s}{2} = \frac{46{\cdot}667\times64{\cdot}81^2\times8000}{2}$$

$$= 0{\cdot}784\times10^9\ \mathrm{Nmm}$$

Therefore:

$$\tau_{yzsmax} = \pm\frac{563{\cdot}5\times0{\cdot}784\times10^9}{98{\cdot}048\times10^9\times46{\cdot}667} \pm \frac{13{\cdot}282}{46{\cdot}667}$$

$$= \underline{\pm\ 0{\cdot}381\ \mathrm{N/mm}^2} \qquad [y = \pm L/2]$$

Summary Example 4 — Table 4

Parameter	Panel Analysis (Equivalent I Beam)
w_{max}	- 18.197 mm
σ_{p1omax}	- 2.860 N/mm^2
σ_{p2omax}	3.325 N/mm^2
σ_{somax}	4.609 N/mm^2
q_{1max}	$\pm$13.282 N/mm
q_{2max}	$\mp$10.886 N/mm
τ_{yzsmax}	$\pm$ 0.381 N/mm^2

4.5 Comments on Numerical Examples

Each of the stressed skin floor panels in Examples 1 to 4 have 35 x 145 softwood stringers, Group S2 - GS Grade, spaced at 400 mm centres and an effective span of 5.0 m.

Consider a simply supported floor joist of the same material and dimensions as the stringers of the floor panels in Examples 1 to 4 with an effective span of 5.0 m. If this floor joist carries a uniformly distributed transverse load equal in intensity to the transverse load applied to each repeating unit of the floor panels in Examples 1 to 4, (-1 N/mm run/repeating unit), its maximum transverse deflection is:

$$w_{max} = w_o = \frac{5pL^4}{384 E_s I_s}$$

where: $E_s I_s = 71 \cdot 135 \times 10^9 \ \mathrm{Nmm^2}$ [Example 1]

Therefore:

$$w_{max} = -\frac{5 \times 1 \times 5000^4}{384 \times 71 \cdot 135 \times 10^9}$$

$$= \underline{-114 \cdot 402 \ \mathrm{mm}}$$

Table 5 shows a comparison between the maximum transverse deflection for the above floor joist and each of the floor panels in Examples 1 to 4. It can be deduced from Table 5 that considerable structural economies can be achieved by utilising both the plywood skins and timber stringers of a stressed skin panel to resist bending as opposed to utilising only the timber stringers to resist bending.

Table 5

Panel	w_{max}
Example 1	- 40.606 mm
Example 2	- 52.621 mm
Example 3	- 10.775 mm
Example 4	- 18.197 mm
Simple Joist	-114.402 mm

A method of analysis applicable to the design of single skin stressed skin floor panels with semi rigid connections is presented by Larsen in Section 2 of 'The Design of Built-up Timber Columns' {6} and is applicable to Example 2. When used in conjunction with the concept of an equivalent T beam the above mentioned method of analysis gives values for maximum transverse panel deflection and maximum stresses identical to those calculated using the method of analysis presented in this paper.

A design method for double skin stressed skin panels with rigid connections (in which the skins are of Canadian Douglas Fir Plywood) is presented in Section 8 of the 'Plywood Construction Manual' produced by C.O.F.I. {10}. This design method is based on the work of Foschi {4} and is applicable to Example 3.

Table 3 shows a comparison,between the maximum transverse panel deflection and the maximum stresses calculated using the above mentioned C.O.F.I. design method,and the corresponding deflection and stresses calculated by using simple beam theory applied to the equivalent I beam for the panel,for the stressed skin floor panel in Example 3. Examination of Table 3 shows that there is good agreement between the corresponding numerical results for the two methods of analysis.

5.0 Conclusions

The most important feature of the method of stressed skin panel analysis presented in this paper is that both single and double skin panels, with either rigid or semi rigid connections, can be analysed using an electronic calculator.

This method of analysis is not restricted to panels which are symmetrical about their mid planes.

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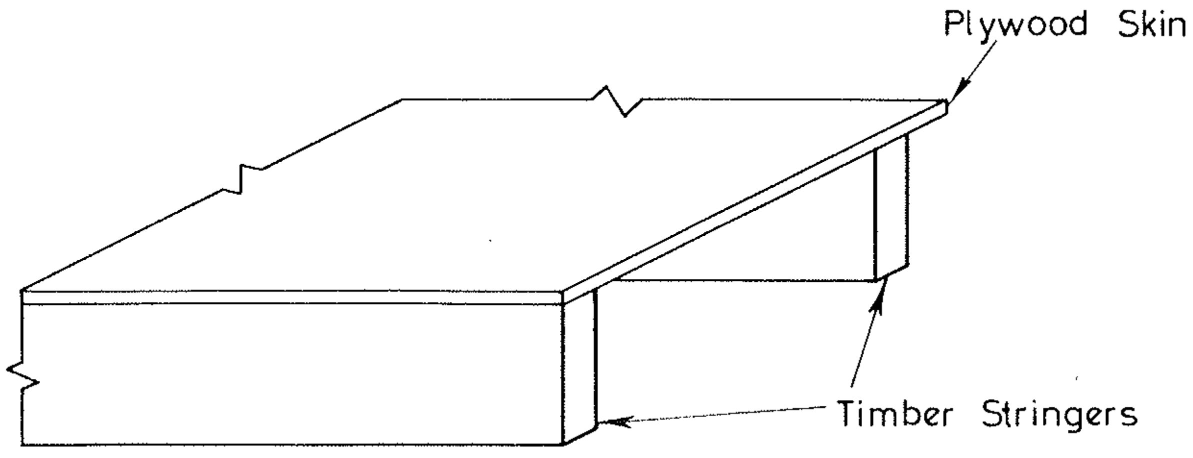
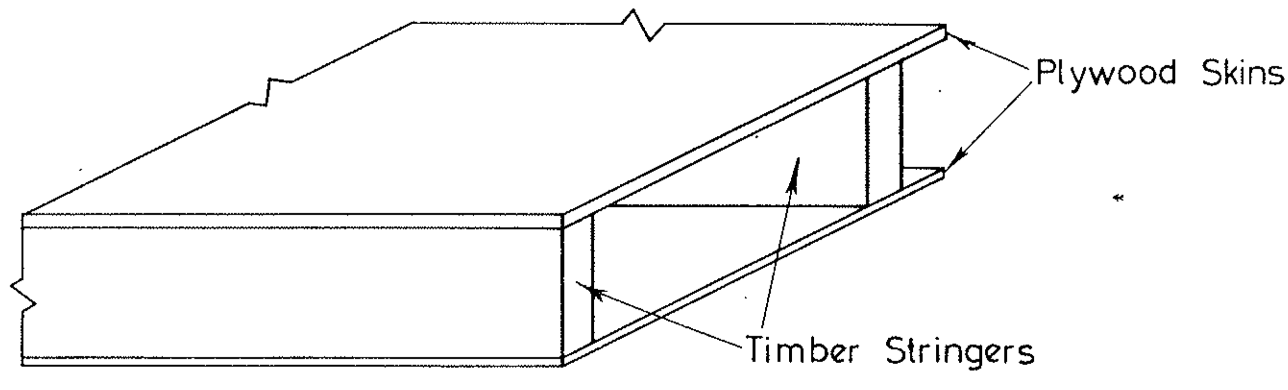
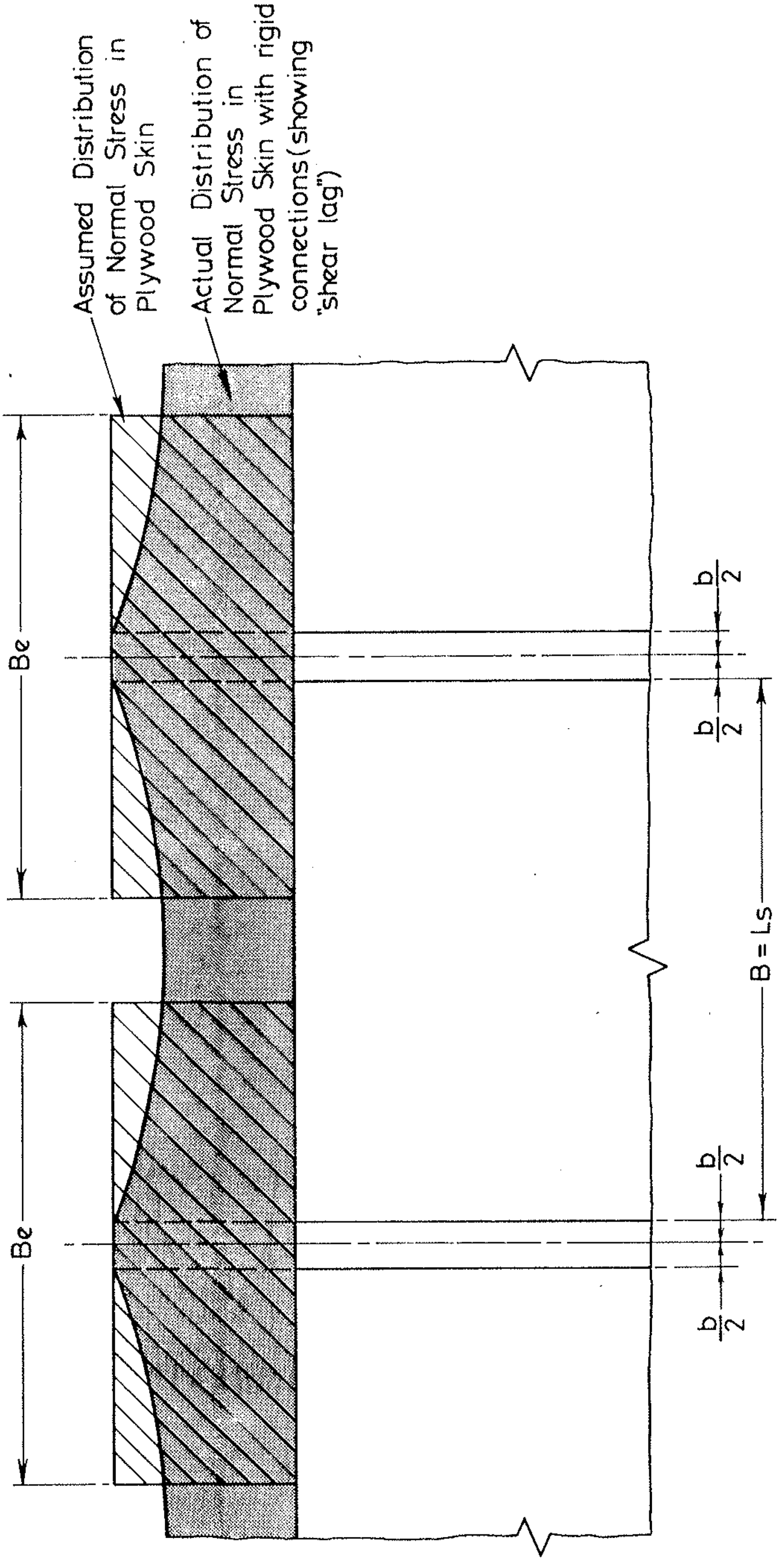
Fig. 1.



Single Skin Stressed Skin Panel



Double Skin Stressed Skin Panel



Assumed Distribution of Normal Stress in Plywood Skin
Actual Distribution of Normal Stress in Plywood Skin with rigid connections (showing "shear lag")
Be
Be
b/2
b/2
b/2
b/2
B = Ls

Fig. 2.

Fig 3

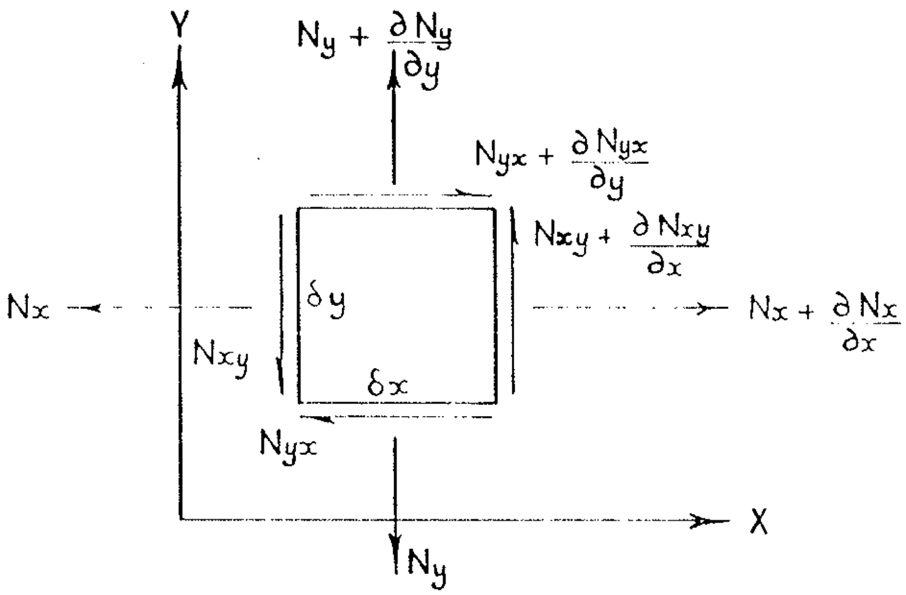

Y
Ny + ∂Ny/∂y
Nyx + ∂Nyx/∂y
Nxy + ∂Nxy/∂x
Nx
δy
Nx + ∂Nx/∂x
Nxy
δx
Nyx
X
Ny

Fig. 4.

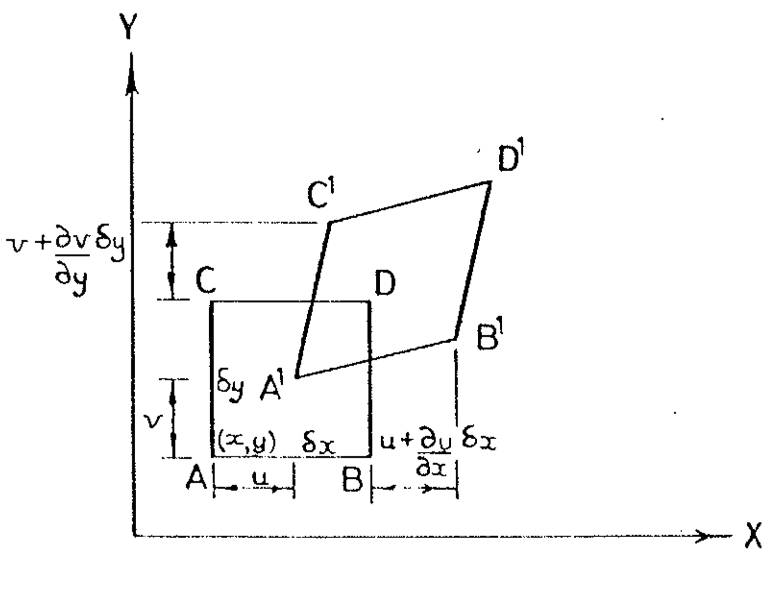

Y
D1
C1
v + ∂v/∂y δy
C
D
B1
δy
A1
v
(x,y)
δx
u + ∂u/∂x δx
A
u
B
X

Fig. 5.

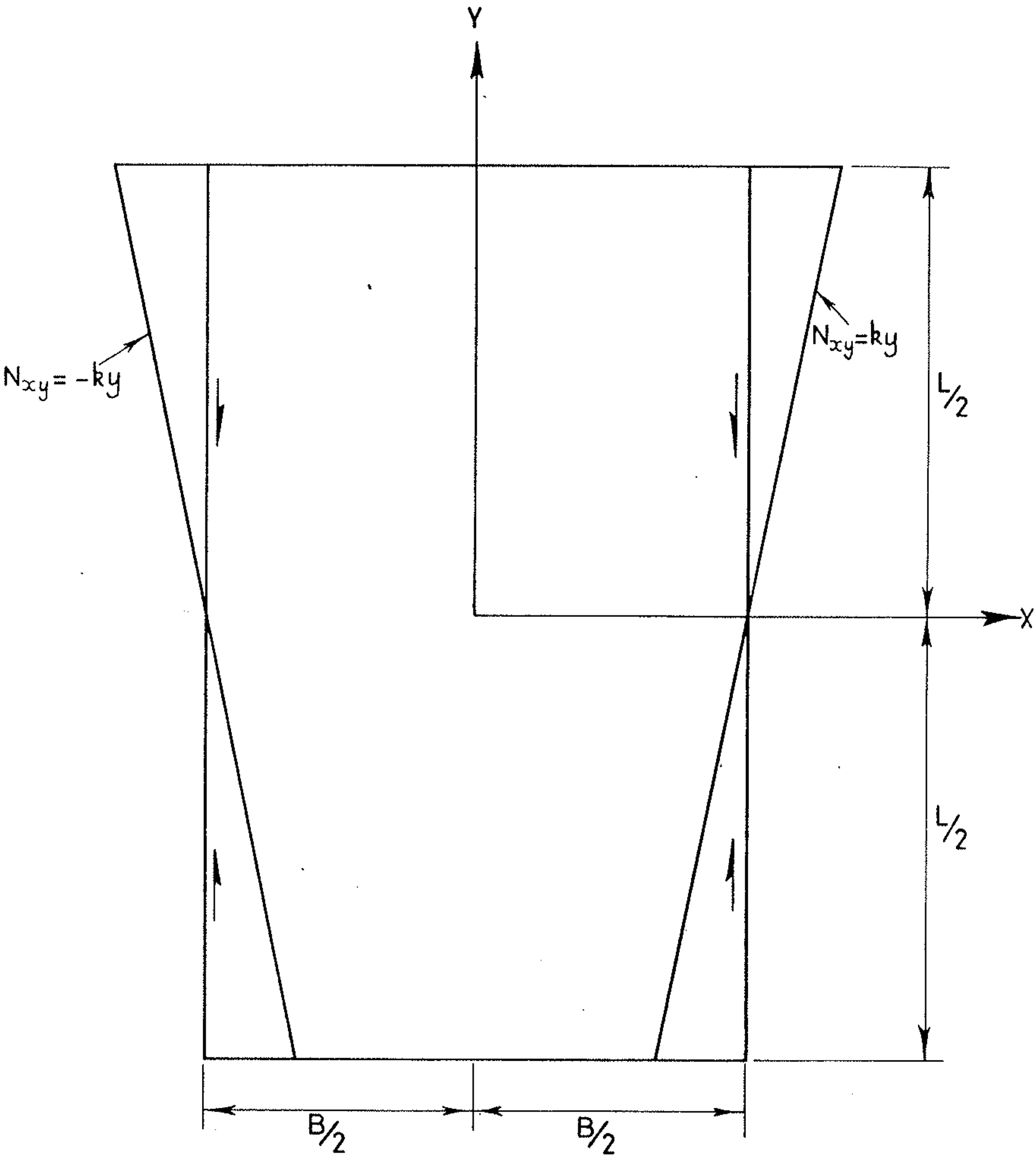
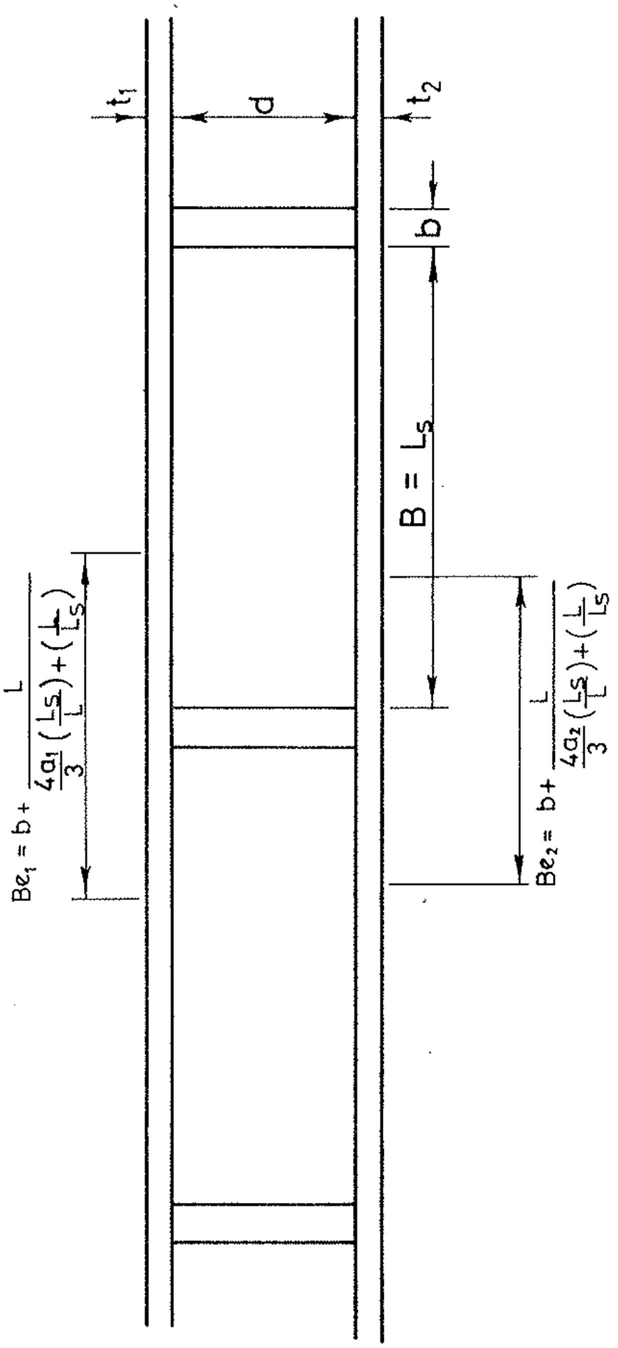


Fig. 6.



t_1
d
t_2
b
$B = L_s$
$Be_1 = b + \frac{L}{\frac{4a_1}{3}\left(\frac{L_s}{L}\right)+\left(\frac{L}{L_s}\right)}$
$Be_2 = b + \frac{L}{\frac{4a_2}{3}\left(\frac{L_s}{L}\right)+\left(\frac{L}{L_s}\right)}$

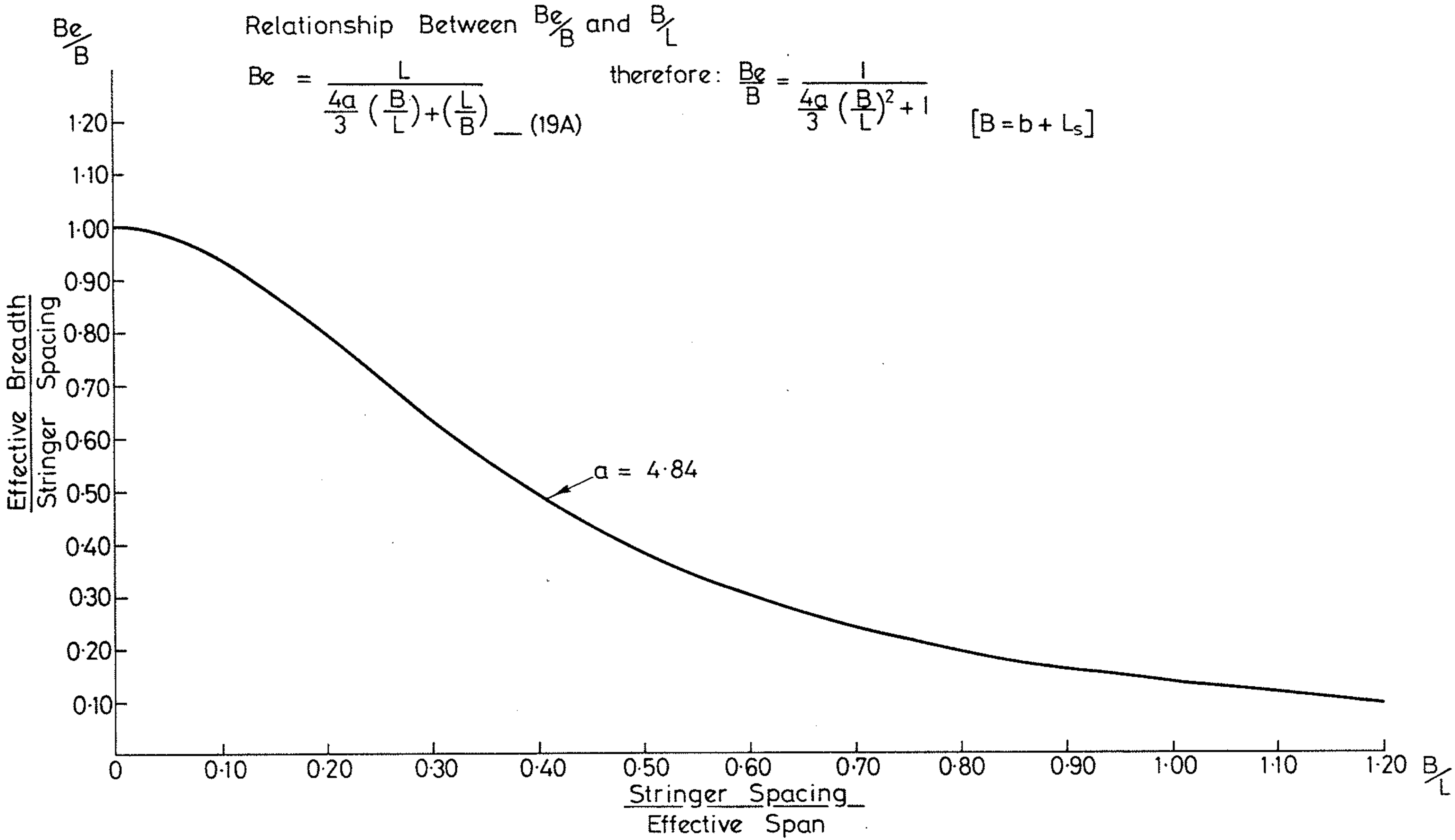

Relationship Between B_e/B and B/L
$B_e = \dfrac{L}{\frac{4a}{3}\left(\frac{B}{L}\right)+\left(\frac{L}{B}\right)}$ ___ (19A)
therefore: $\dfrac{B_e}{B} = \dfrac{1}{\frac{4a}{3}\left(\frac{B}{L}\right)^2+1}$
$[B = b + L_s]$
B_e/B
$\dfrac{\text{Effective Breadth}}{\text{Stringer Spacing}}$
1·20
1·10
1·00
0·90
0·80
0·70
0·60
0·50
0·40
0·30
0·20
0·10
0
0·10
0·20
0·30
0·40
0·50
0·60
0·70
0·80
0·90
1·00
1·10
1·20
B/L
$\dfrac{\text{Stringer Spacing}}{\text{Effective Span}}$
$a = 4{\cdot}84$

Fig. 7.

Relationship Between Effective Breadth Be and y for a Simply Supported Panel with a Uniformly Distributed Transverse Load

$$Be = \left[\frac{L^2}{4} - y^2\right]\left[\frac{1}{\frac{aB}{3} + \frac{L^2}{4B} - \frac{y^2}{B}}\right] \quad \text{——} (24A) \quad [B = b + L_s]$$

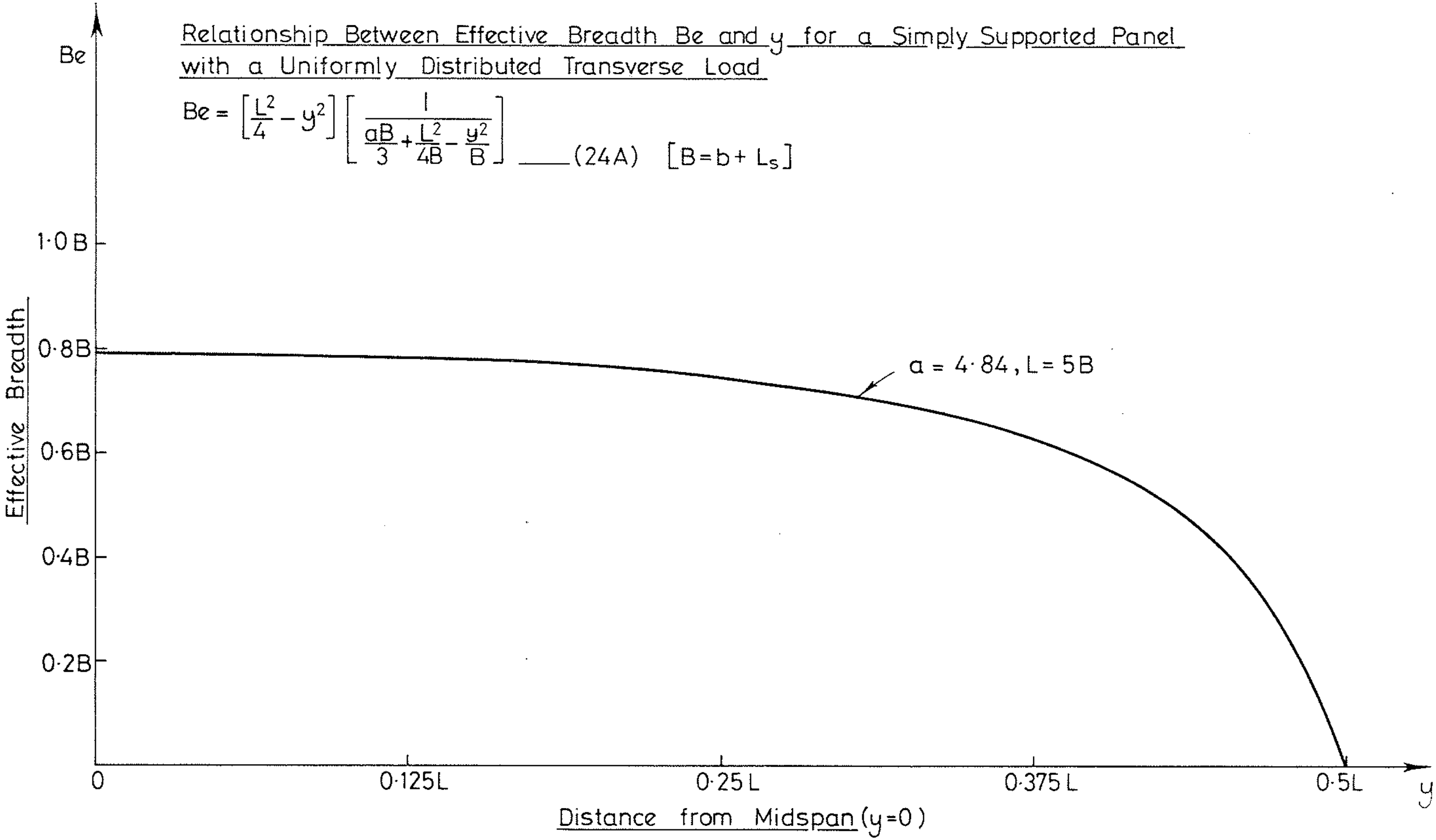
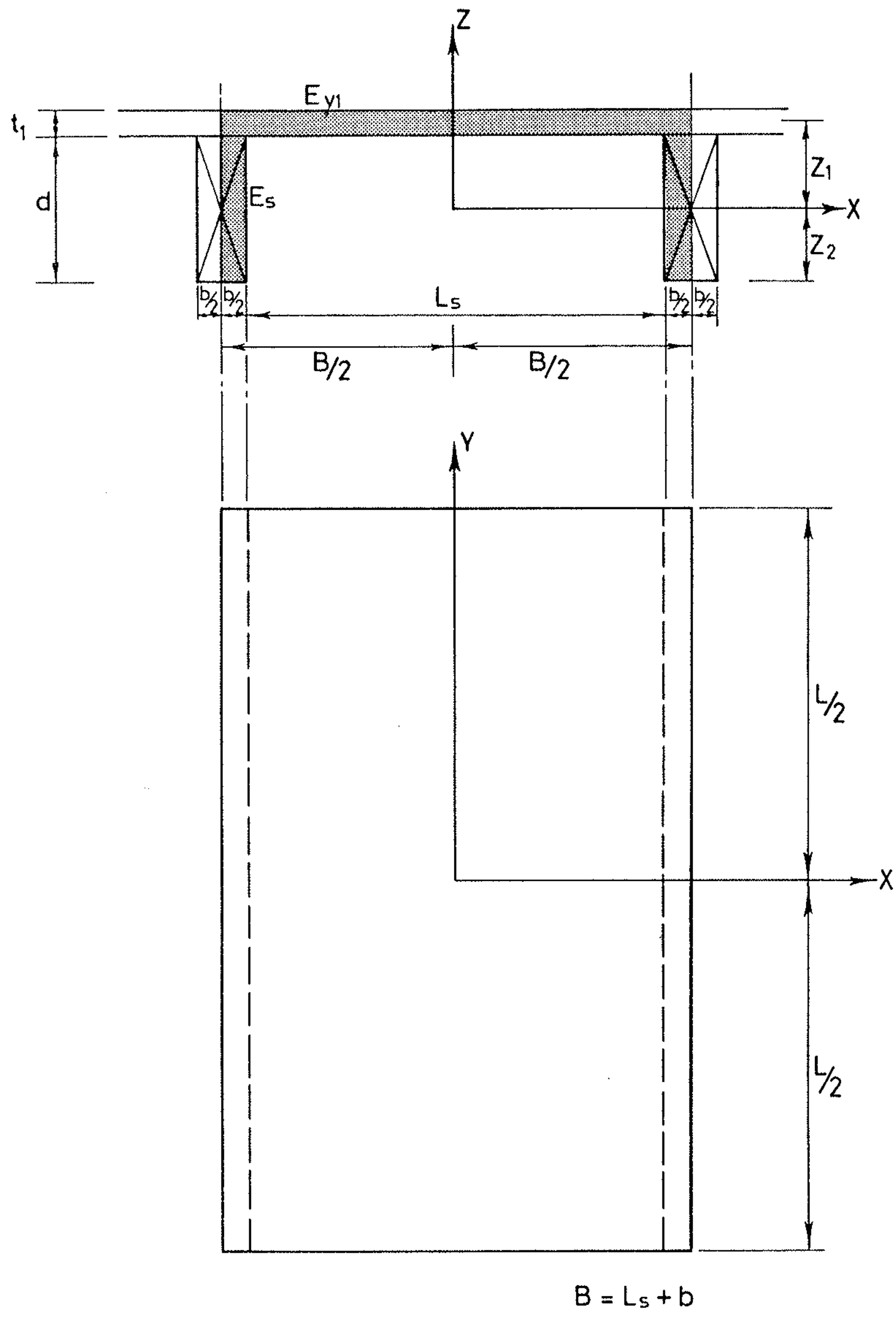
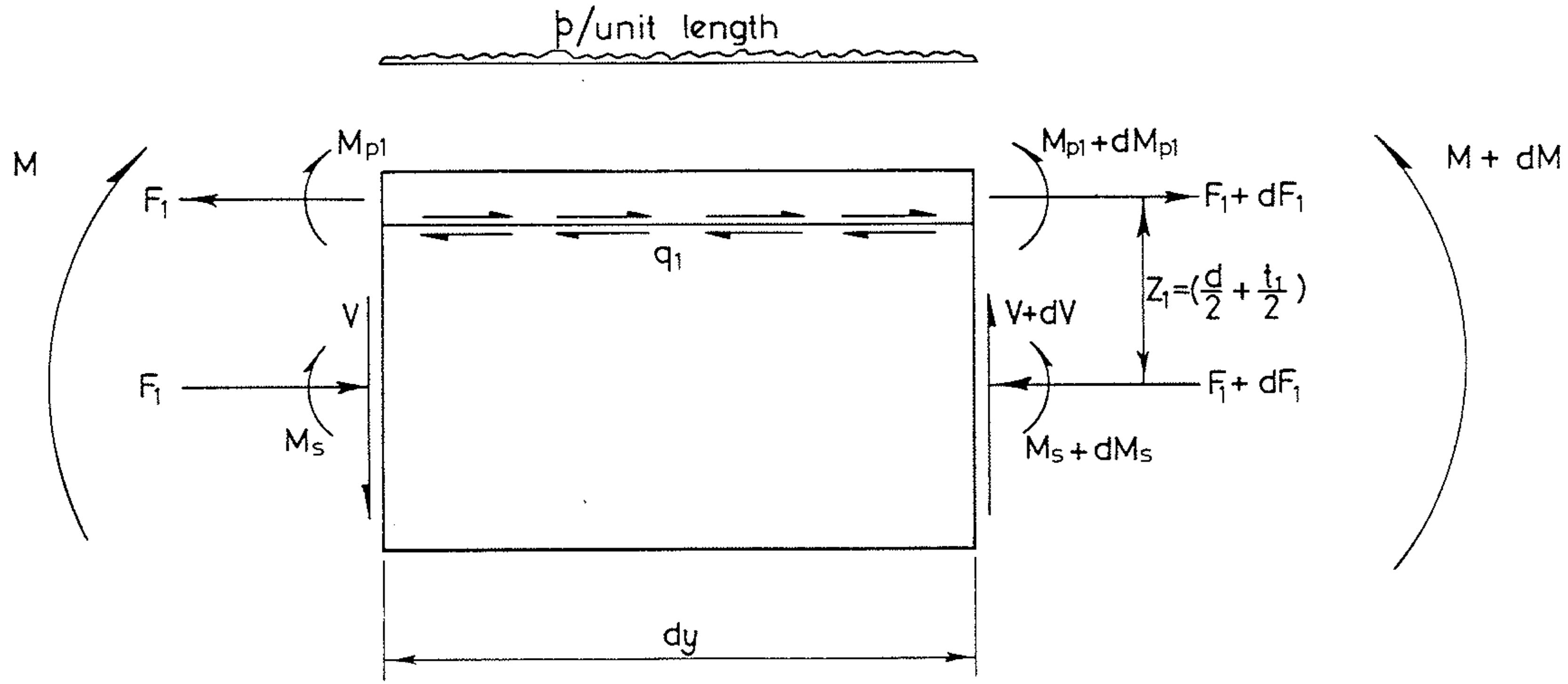


Fig. 8.



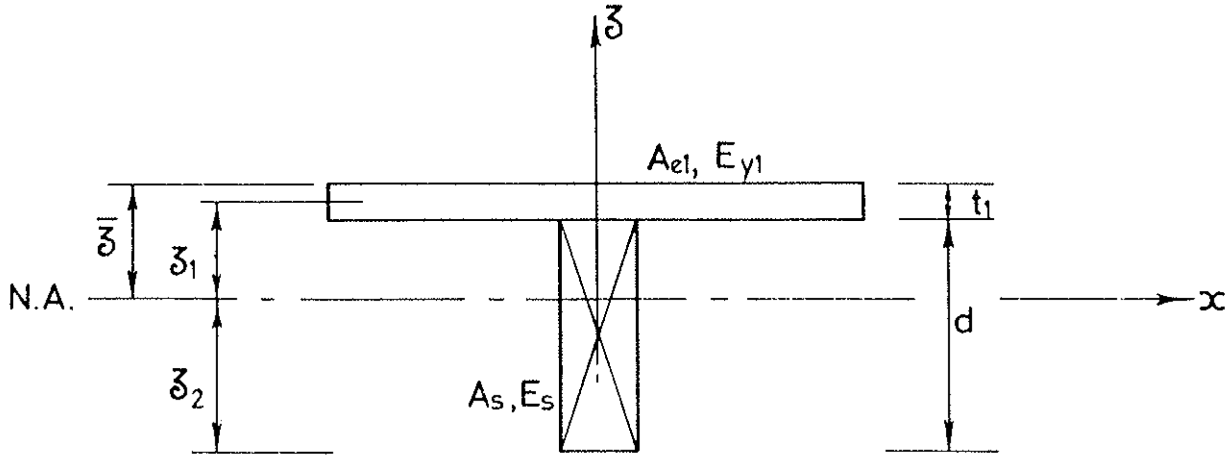
Repeating Unit for Single Skin Stressed Skin Panel with Multiple Stringers



$$M = M_{p1} + M_s - F_1 Z_1$$

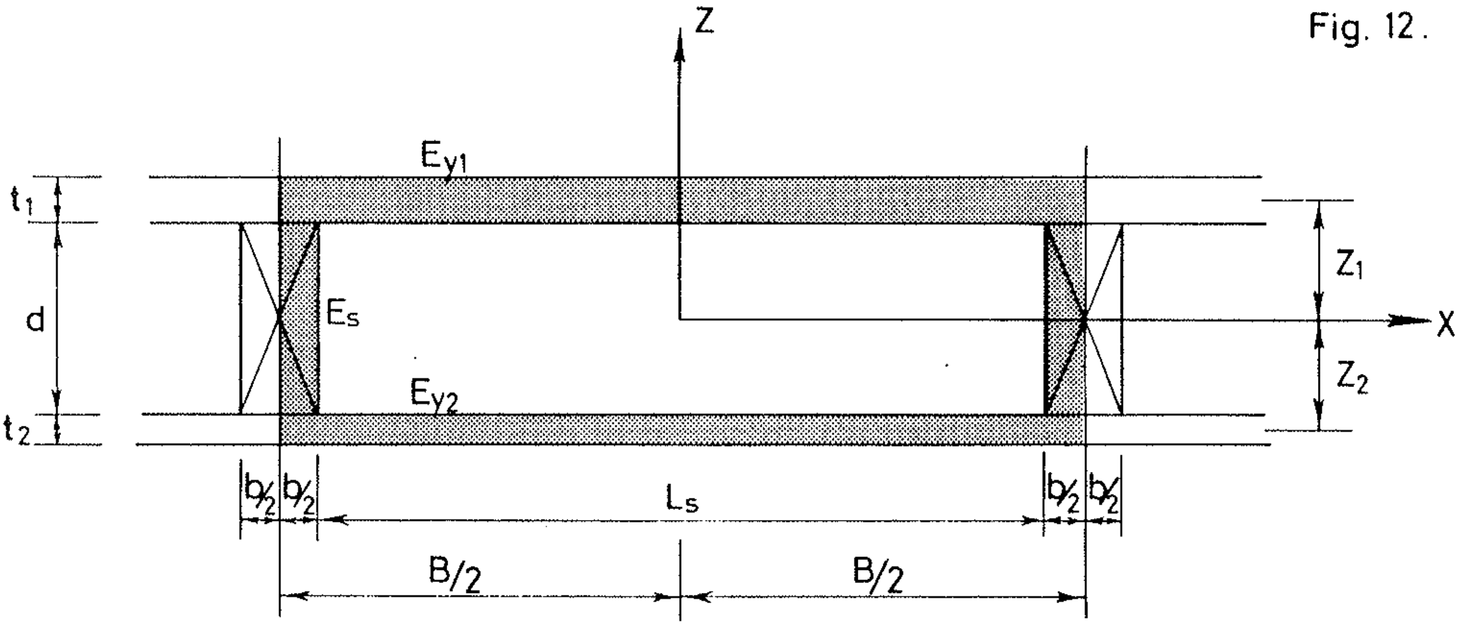
Fig. 10

Fig. 11.



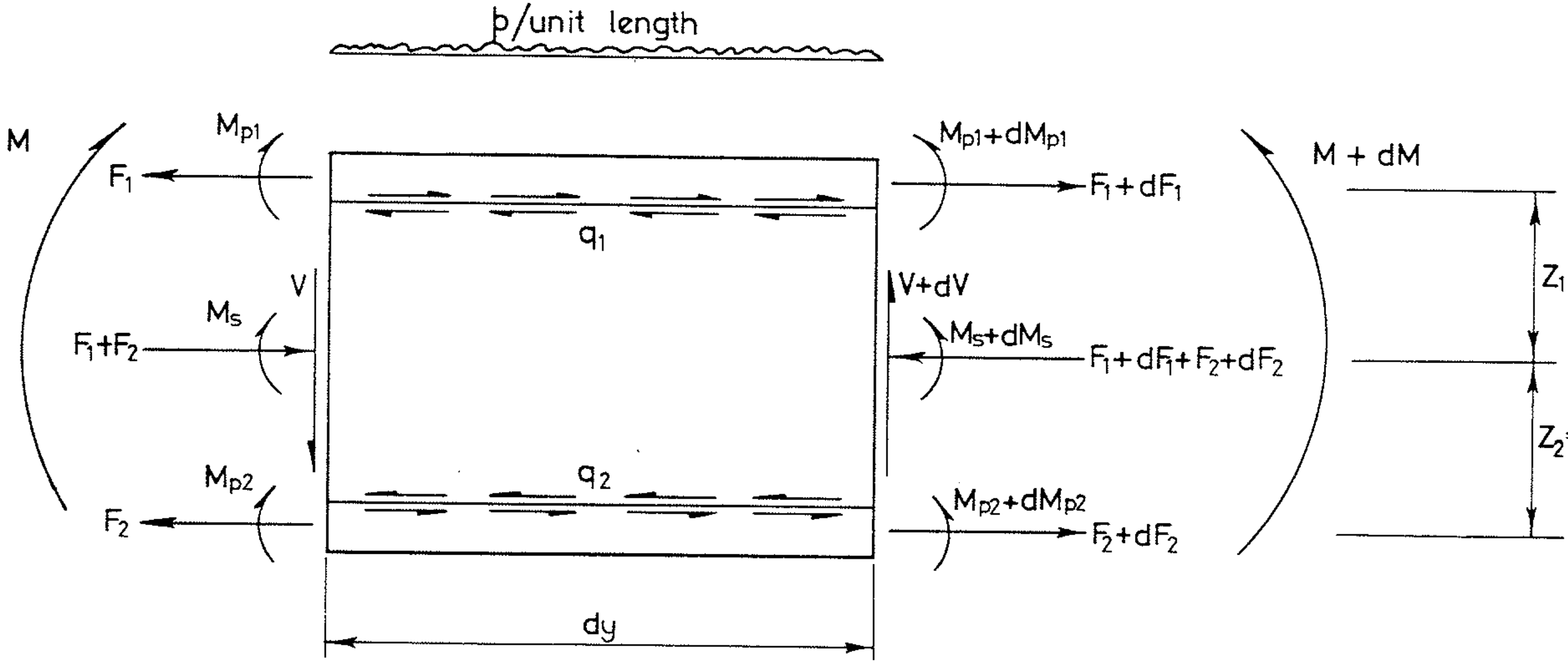
Equivalent T Beam for Single Skin Stressed Skin Panel with Multiple Stringers

Fig. 12.



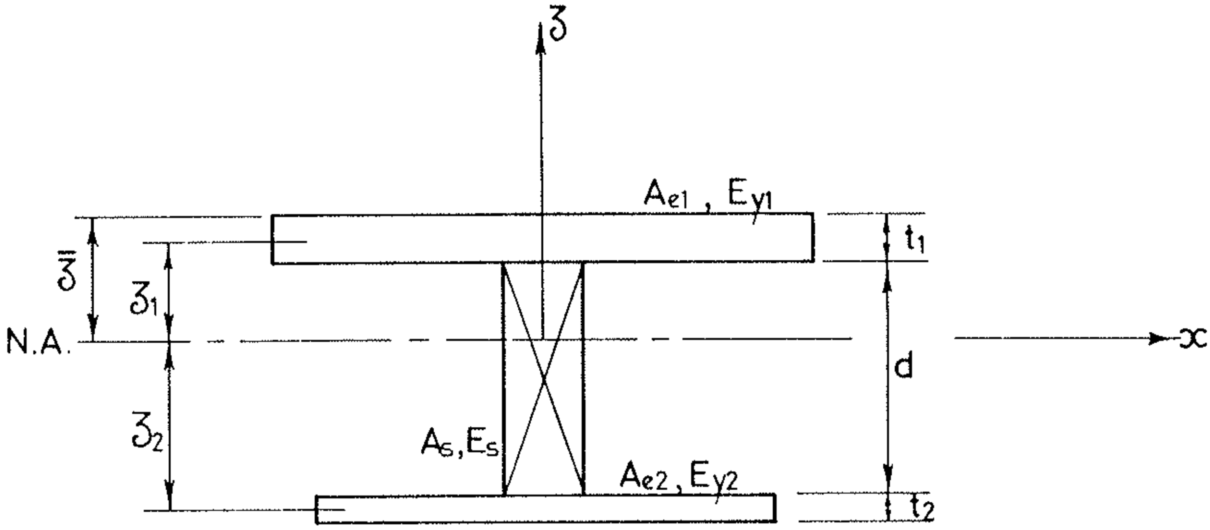
$B = L_s + b$

Repeating Unit for Double Skin Stressed Skin Panel with Multiple Stringers



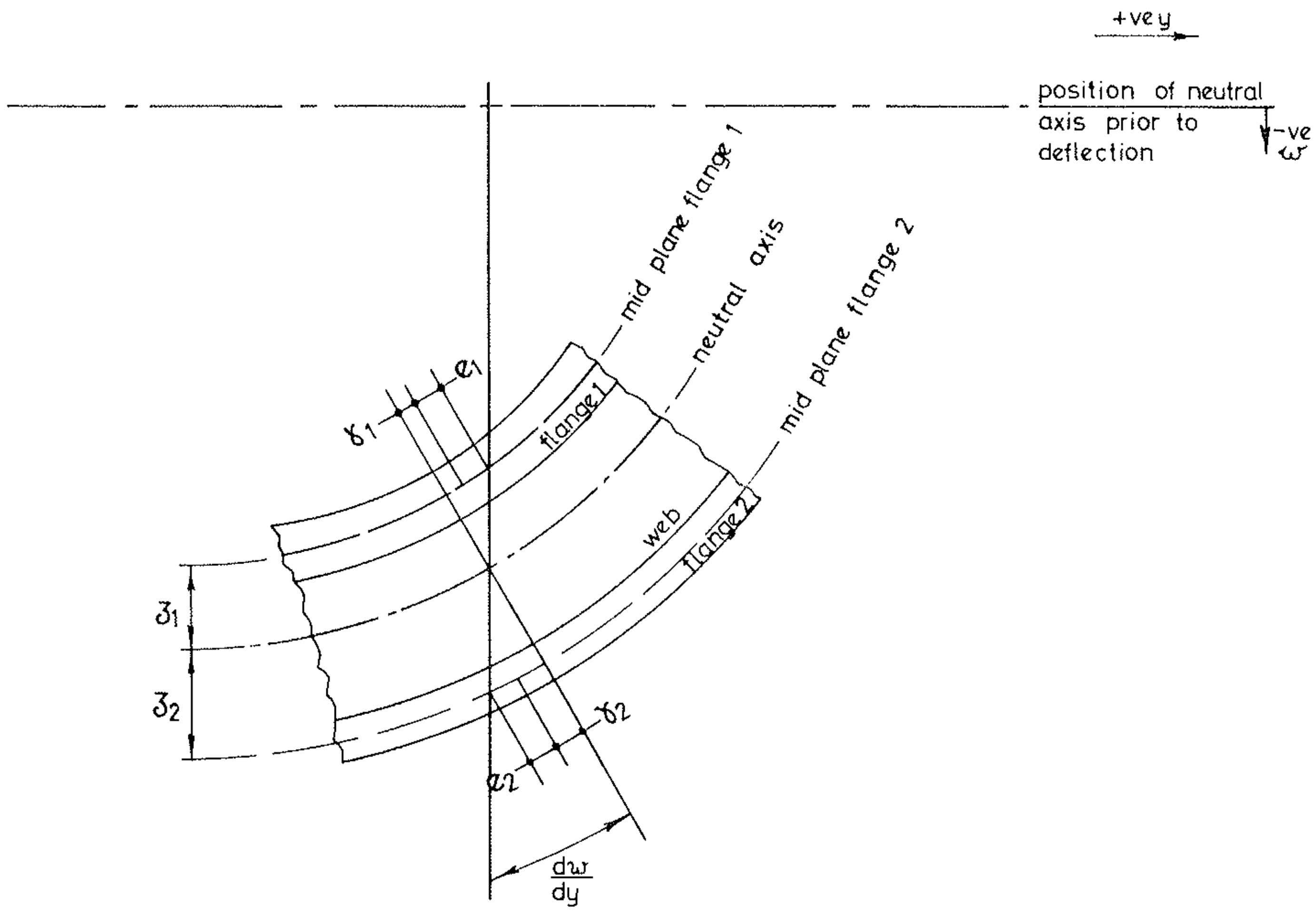
$$M = M_{p1} + M_{p2} + M_s - F_1 Z_1 - F_2 Z_2$$

Fig. 13.



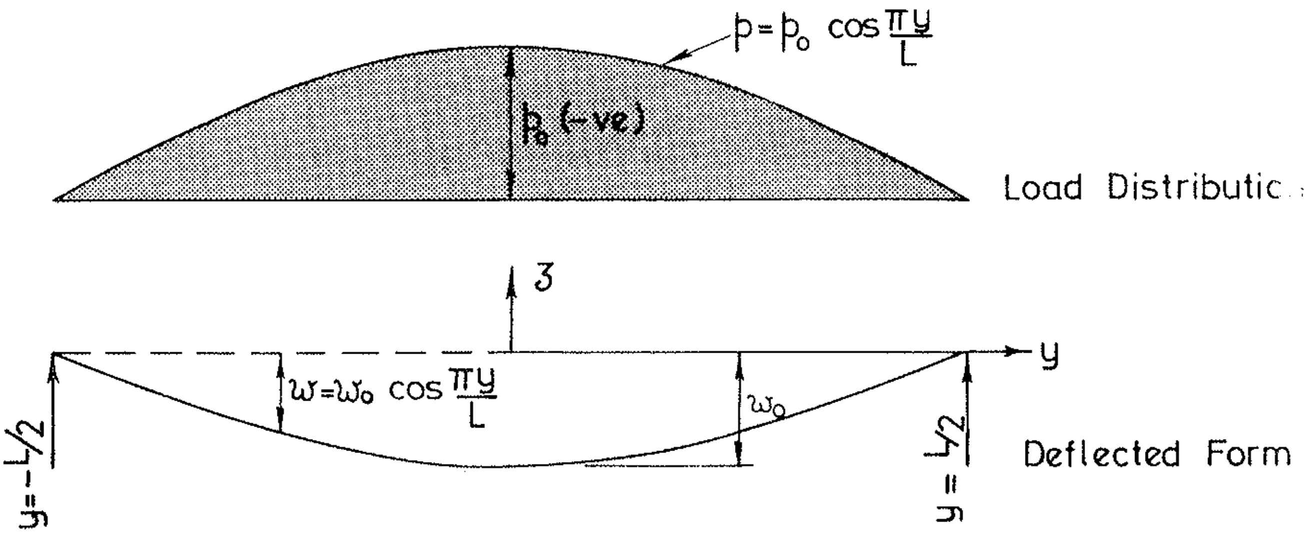
Equivalent I Beam for Double Skin Stressed Skin Panel with Multiple Stringers

Fig. 15



+ve y
position of neutral
axis prior to
deflection
-ve
w
mid plane flange 1
neutral axis
mid plane flange 2
e1
γ1
flange 1
web
flange 2
z1
z2
γ2
e2
dw/dy

Fig. 16.



p = p0 cos πy/L
p0 (-ve)
Load Distributic
z
y
w=w0 cos πy/L
w0
y=-L/2
y = L/2
Deflected Form

Fig. 17
Examples 1&2

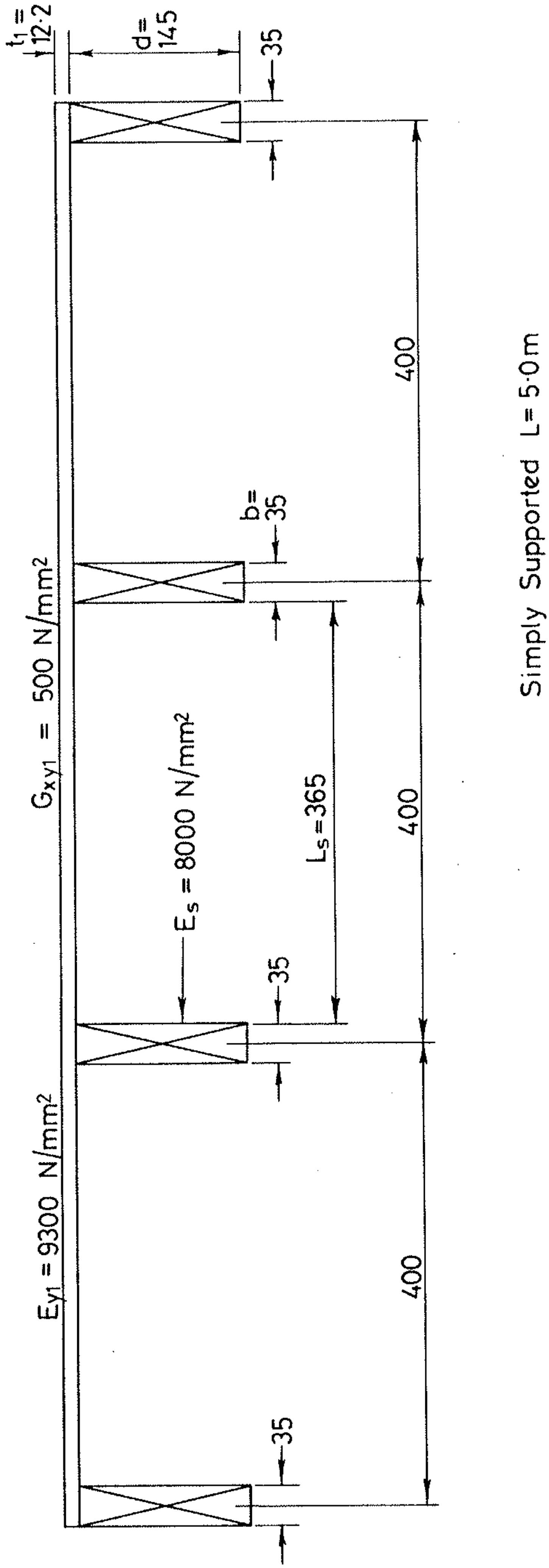
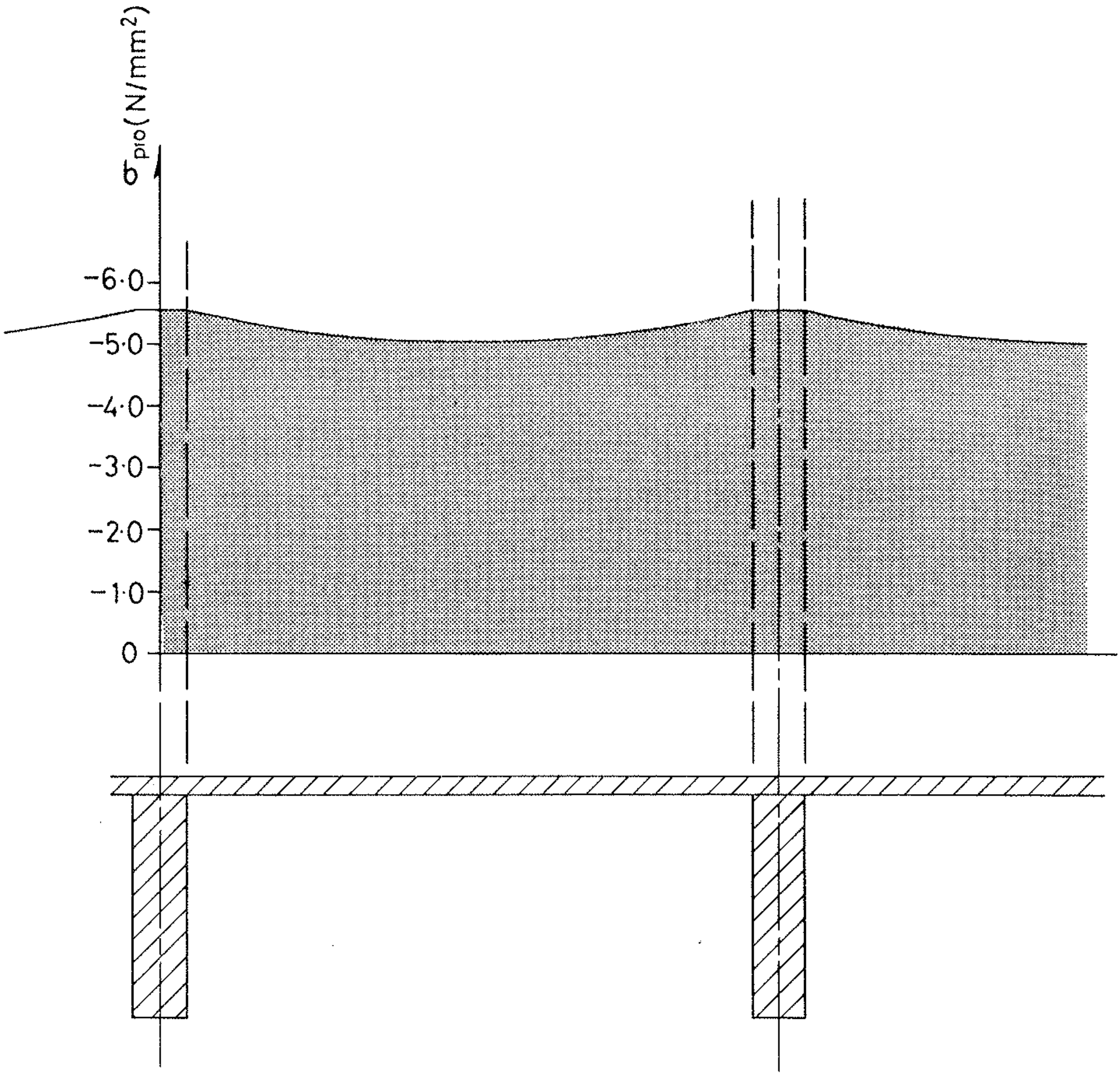
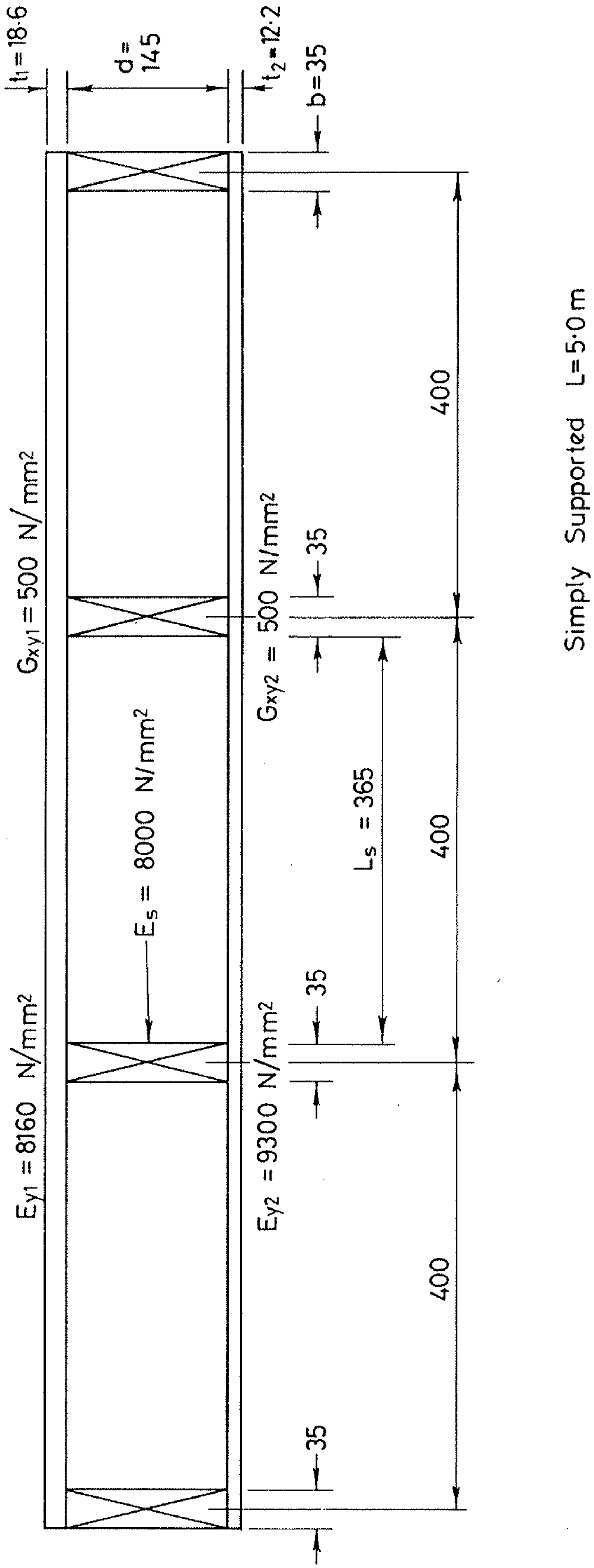


Fig . 18



Distribution of Normal Stress, at the Mid Plane of the Compression Skin of the Single Skin Stressed Skin Panel in Example 1, at $y = 0$

Fig . 19
Examples 3 & 4



APPENDIX I

The purpose of this appendix is to provide a quantitative comparison between (19A) and two alternative expressions for calculating the effective breadths at $y = 0$ of the plywood skins of a simply supported stressed skin panel with a uniformly distributed transverse load.

The expression for effective breadth (19A) is:

$$B_e = \frac{L}{\frac{4a}{3}\left(\frac{B}{L}\right) + \left(\frac{L}{B}\right)} \qquad \text{(19A)}$$

The expression in (19A) will be compared with the series solution for effective breadth (1.i), Tottenham {8}, and the more approximate solution for effective breadth (1.ii), Gubenko {5}.

$$B_e = \left[\frac{16(\lambda_1^2 - \lambda_2^2)}{\pi^2 L} \sum_{1,3,5} \frac{(-1)^{\frac{n-1}{2}}}{n^2\left(\lambda_1 \tanh \lambda_1 \frac{n\pi B}{2L} - \lambda_2 \tanh \lambda_2 \frac{n\pi B}{2L}\right)} \right]^{-1} \qquad \text{(1.i)}$$

where:

$$\lambda_1 = \sqrt{a + \sqrt{a^2 - b}}$$

$$\lambda_2 = \sqrt{a - \sqrt{a^2 - b}}$$

$$a = \frac{E_y}{2G_{xy}} - \nu_{xy}$$

$$b = \frac{E_y}{E_x}$$

$$B_e = \frac{2L \tanh \pi \sqrt{E_y/G_{xy}}\, B/2L}{\pi \sqrt{E_y/G_{xy}}} \qquad \text{(1.ii)}$$

The expressions in (1.i) and (1.ii) have been transformed so that their forms and notations are consistent with that used in this paper.

The following assumptions are common to the derivations of (19A), (1.i), and (1.ii):

a) the skins of a stressed skin panel act as orthotropic plates in a state of plane stress.

b) the normal force in each skin in the direction perpendicular to the stringers is zero, for all values of y, at each stringer.

c) if the transverse load is applied directly to the skin the transverse deformation of the skin is small.

The expression in (1.i) is representative of the series type of solution adopted by most investigators in the determination of the effective breadth of the skins of stressed skin panels with a uniformly distributed transverse load.

The expression in (1.ii) is equivalent to the first term of the series solution in (1.i) the simplifying assumptions having been made that λ_2, b, and ν_{xy} can be neglected. It is apparent therefore that (1.ii) represents an approximation to (1.i).

I.I Comparison of expressions for the effective breadths at $y = 0$ of plywood skins of a simply supported stressed skin panel with a uniformly distributed transverse load

For the purpose of comparing (19A),(1.i),and (1.ii) the relationship for each,between the ratios; effective breadth/breadth,(B_e/B), and breadth/effective span of panel,(B/L), will be examined.

Consider a typical plywood with the following elastic constants:

$$E_x = 4020\ N/mm^2$$
$$E_y = 7370\ N/mm^2$$
$$G_{xy} = 500\ N/mm^2$$
$$\nu_{xy} = 0{\cdot}074$$

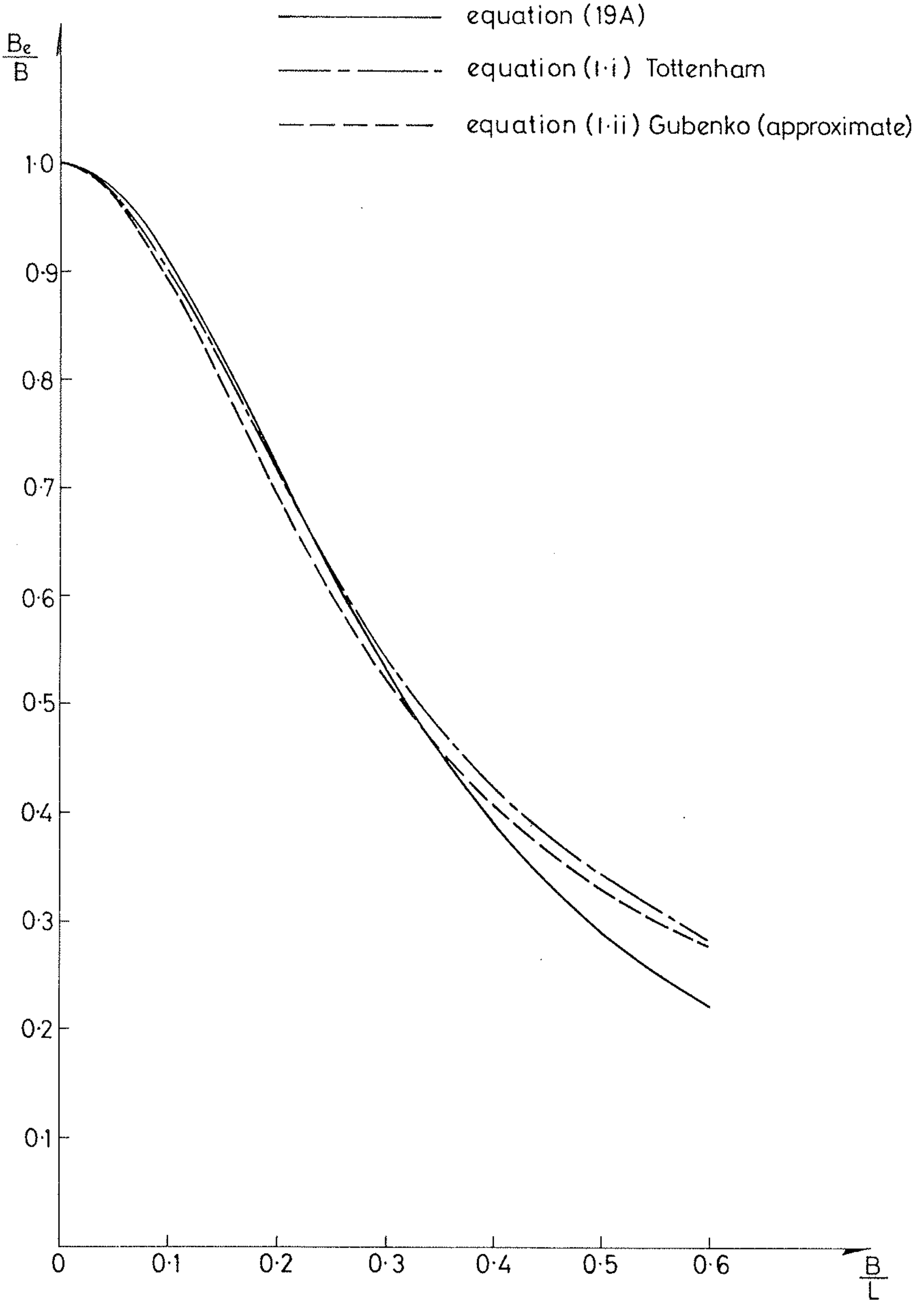
Fig.1.i gives a graphical comparison of the respective relationships between B_e/B and B/L for the above plywood for B_e calculated using the expressions in (19A),(1.i),and (1.ii).

Fig 1.i shows good agreement between all three expressions for effective breadth, (19A),(1.i),and (1.ii),for $B/L < 0{\cdot}3$,and for $B/L > 0{\cdot}3$ (19A) is conservative compared with (1.i) and (1.ii). For most practical stressed skin panel assemblies B/L is $< 0{\cdot}3$.

I.II Conclusions

The expression in (19A) is in good agreement with the series solution for effective breadth (1.i),Tottenham, and the more approximate solution (1.ii), Gubenko,when B/L is small,(<0.3 in example),and when B/L is relatively large, (>0.3 in example),(19A) is conservative compared with (1.i) and (1.ii).

Fig. 1·i



equation (19A)
equation (1·i) Tottenham
equation (1·ii) Gubenko (approximate)
$\frac{B_e}{B}$
1·0
0·9
0·8
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
0·1
0·2
0·3
0·4
0·5
0·6
$\frac{B}{L}$

APPENDIX II

The purpose of this appendix is to provide a quantitative comparison of the loading and displacement definitions for a beam simply supported at both ends, for a uniformly distributed transverse load, and for a sinusoidally distributed transverse load.

The following two assumptions are made:

1) The flexural rigidity of the beam, EI_B, is constant throughout the length of the beam.
2) The flexural rigidity of the beam is the same for both the uniformly distributed transverse load and the sinusoidally distributed transverse load.

The following two abbreviations will be used:

u.d.l. - uniformly distributed transverse load

s.d.l. - sinusoidally distributed transverse load.

II.I Comparison of Bending Moments due to the applied load for a beam simply supported at both ends for a u.d.l. and for a s.d.l.

$M_{(u.d.l.)}$ the bending moment at y due to the applied load for a u.d.l. is given by (33).

$$M_{(u.d.l.)} = -\frac{pL^2}{8} + \frac{py^2}{2}$$

$$= M_o + \frac{py^2}{2} \qquad \text{(2.i)}$$

where:

$$M_o = -\frac{pL^2}{8}$$

$M_{(s.d.l.)}$ the bending moment at y due to the applied load for a s.d.l. is given by (42)

$$M_{(s.d.l.)} = M_o \cos\frac{\pi y}{L} \qquad \text{(2.ii)}$$

For the purpose of comparison $M_{(u.d.l.)}$ and $M_{(s.d.l.)}$ can be expressed in the following forms:

$$\frac{M_{(u.d.l.)}}{M_o} = 1 - \frac{4y^2}{L^2} \qquad \text{(2.iii)}$$

and

$$\frac{M_{(s.d.l.)}}{M_o} = \cos\frac{\pi y}{L} \qquad \text{(2.iv)}$$

Fig.2.i gives a graphical comparison of (2.iii) and (2.iv) between $y = 0$ and $y = L/2$.

II.11 Comparison of transverse beam displacements at y, for a u.d.l. and for a s.d.l.

An expression for $w(u.d.l.)$, the transverse beam displacement at y for a u.d.l., can be derived in the following manner:

$$\frac{d^2 w(u.d.l.)}{dy^2} = \frac{M(u.d.l.)}{EI_B}$$

$$= \left(-\frac{pL^2}{8} + \frac{py^2}{2}\right)\frac{1}{EI_B}$$

therefore:

$$\frac{dw(u.d.l.)}{dy} = \left(-\frac{pL^2 y}{8} + \frac{py^3}{6}\right)\frac{1}{EI_B} + C_1$$

When $y = 0$, $\frac{dw(u.d.l.)}{dy} = 0$, therefore $C_1 = 0$.

$$w(u.d.l.) = \left(-\frac{pL^2 y^2}{16} + \frac{py^4}{24}\right)\frac{1}{EI_B} + C_2$$

When $y = \pm L/2$, $w = 0$, therefore $C_2 = \frac{5pL^4}{384 EI_B}$

thus:

$$w(u.d.l.) = \left(-\frac{pL^2 y^2}{16} + \frac{py^4}{24} + \frac{5pL^4}{384}\right)\frac{1}{EI_B} \qquad \text{(2.v)}$$

and

$$w_o(u.d.l.) = \frac{5pL^4}{384 EI_B} \qquad \text{(2.vi)}$$

Combining (2.v) and (2.vi) yields:

$$\frac{w(u.d.l.)}{w_o(u.d.l.)} = -\frac{24y^2}{5L^2} + \frac{16y^4}{5L^4} + 1 \qquad \text{(2.vii)}$$

$w(s.d.l.)$, the transverse beam displacement at y for a s.d.l., is given by (50).

$$w(s.d.l.) = -\frac{L^2 M_o}{\pi^2 EI_B}\cos\frac{\pi y}{L}$$

$$= \frac{pL^4}{8\pi^2 EI_B}\cos\frac{\pi y}{L} \qquad \text{(2.viii)}$$

therefore:

$$w_o(s.d.l.) = \frac{pL^4}{8\pi^2 EI_B} \qquad \text{(2.ix)}$$

Combining (2.viii) and (2.ix) yields:

$$\frac{w(s.d.l.)}{w_o(s.d.l.)} = \cos\frac{\pi y}{L} \qquad \text{(2.x)}$$

Equating (2.vi) and (2.ix) yields:

$$\frac{w_o(s.d.l.)}{w_o(u.d.l.)} = \frac{384}{5\times 8\times\pi^2} = 0{\cdot}9727 \qquad \text{(2.xi)}$$

Substituting (2.xi) in (2.x)

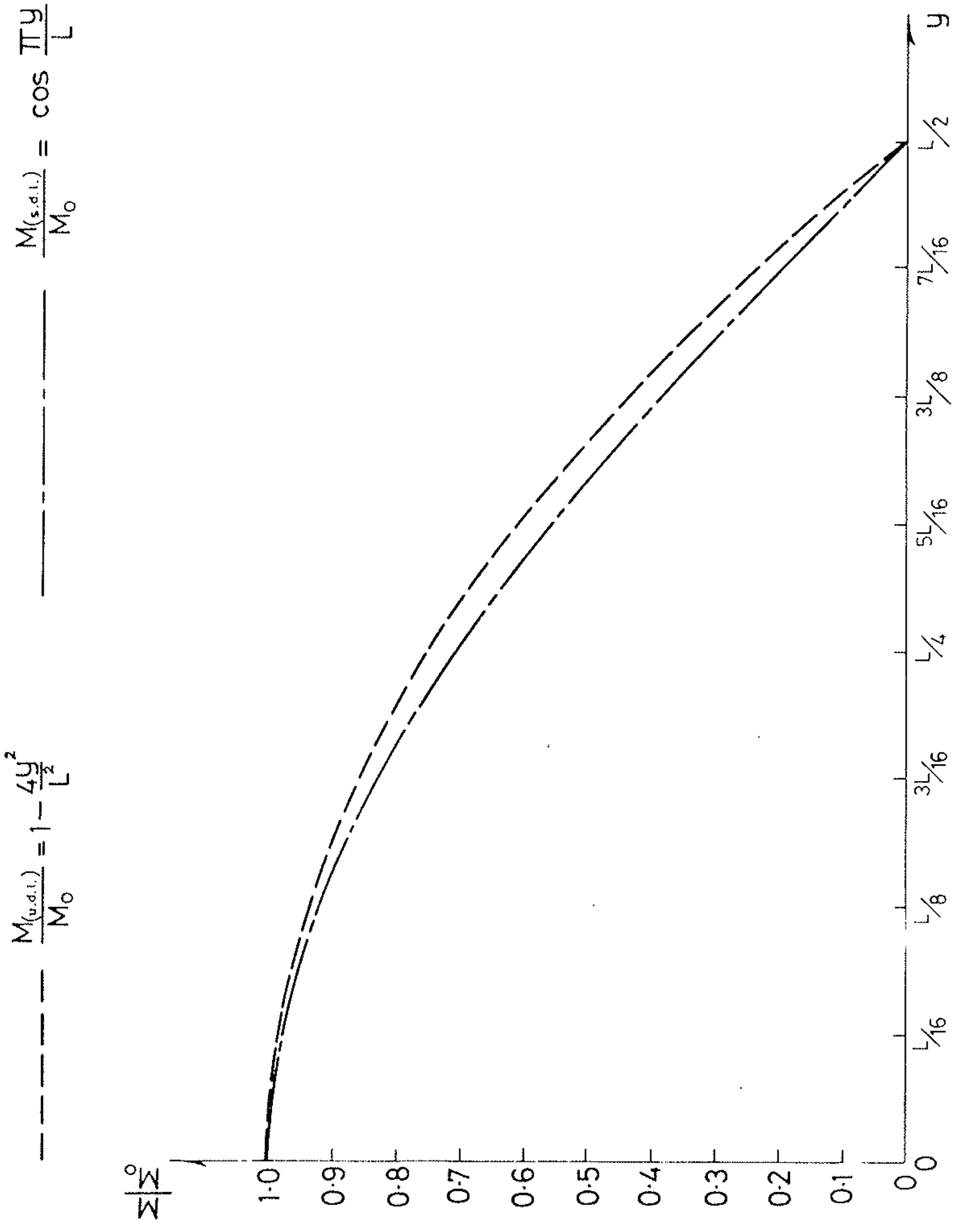
$$\frac{w(s.d.l.)}{w_o(u.d.l.)} = 0{\cdot}9727\cos\frac{\pi y}{L} \qquad \text{(2.xii)}$$

Fig.2.ii gives a graphical comparison of (2.vii) and (2.xii) between $y = 0$ and $y = L/2$.

11.111 Conclusions

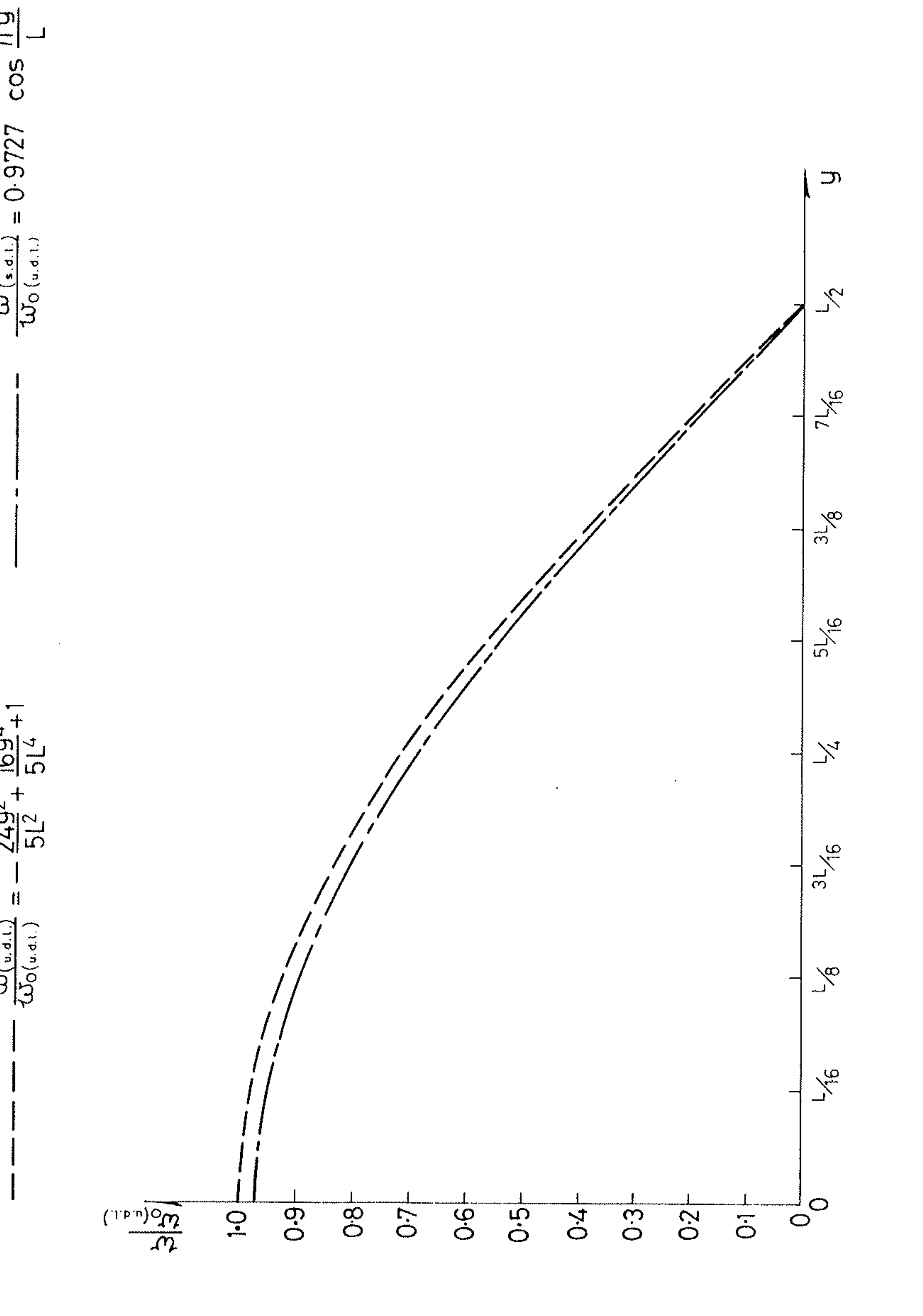
For most practical design purposes it is acceptable to approximate the above mentioned loading and displacement definitions for a beam simply supported at both ends which carries a uniformly distributed transverse load,by the corresponding loading and displacement definitions for a beam simply supported at both ends which carries a sinusoidally distributed transverse load.

Fig. 2·i



$\frac{M_{(u.d.l.)}}{M_o} = 1 - \frac{4y^2}{L^2}$
$\frac{M_{(s.d.l.)}}{M_o} = \cos \frac{\pi y}{L}$
$\frac{M}{M_o}$
1·0
0·9
0·8
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
0
L/16
L/8
3L/16
L/4
5L/16
3L/8
7L/16
L/2
y

Fig. 2·ii



$\frac{\omega_{(u.d.l.)}}{\omega_{0(u.d.l.)}} = -\frac{24y^2}{5L^2} + \frac{16y^4}{5L^4} + 1$
$\frac{\omega_{(s.d.l.)}}{\omega_{0(u.d.l.)}} = 0{\cdot}9727 \cos\frac{\pi y}{L}$
$\frac{\omega}{\omega_{0(u.d.l.)}}$
1·0
0·9
0·8
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
0
L/16
L/8
3L/16
L/4
5L/16
3L/8
7L/16
L/2
y

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

A COMPARISON OF PLYWOOD MODULUS OF RIGIDITY DETERMINED BY THE ASTM AND RILEM/CIB-3TT TEST METHODS

by

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VIENNA, AUSTRIA

MARCH 1979

ABSTRACT

Panel shear through thickness modulus of rigidity (G) was determined by testing specimens from ten Canadian softwood plywood 3/4 inch - 7 ply sheathing grade panels and from ten defect free Douglas fir 11/16 inch - 7 ply panels using the ASTM two rail (panel shear) and the RILEM/CIB-3TT (twisted plate) test methods.

Differences in G developed by these two methods ranged from 8% to 19% and were dependent on the type of plywood, and for the ASTM test on the face grain orientation to principal stress. These differences indicate that the interaction of grade, species, face grain to stress orientation, and possibly construction would make it difficult to develop one general modification factor to convert data from one test method to the other.

A COMPARISON OF PLYWOOD MODULUS OF RIGIDITY DETERMINDED BY THE ASTM AND RILEM/CIB-3TT TEST METHODS

1.0 BACKGROUND

During the development of the RILEM/CIB-3TT test standard, "Testing Methods for Plywood in Structural Grades for Use in Load Bearing Structures," differences of opinion developed on the choice of a suitable test method for determination of modulus of rigidity (G) in shear through thickness.

Individuals supporting the two rail shear test argued that the two rail test better simulates the in-service loading condition in the 0° and 90° directions to which the in-grade test data would apply (e.g., plywood web beams, shear walls, diaphragms, etc.).

Individuals supporting the twisted plate test argued that the plate test was simple and would result in a similar modulus of rigidity value.

2.0 OBJECTIVES

To compare the plywood modulus of rigidity determined by the ASTM and RILEM/CIB-3TT test methods for typical in-grade and defect free softwood plywood.

To determine (if required) if a modification factor could be developed which would permit modification of data determined using one test to the equivalent of what could be expected from the other test.

3.0 EXPERIMENTAL DESIGN

3.1 Replicates

Since the comparison of the test methods was to be indicative in nature, it was not necessary to obtain a large representative sample from several mills. Therefore it was decided to select ten sheathing grade panels and ten all Douglas fir defect free panels and test "matched" specimens obtained from these panels. Defect free panels were included as part of the program in case the within panel variability of the sheathing grade panels (caused mainly by the nonuniform defect distribution) overmasked any identifiable trends.

3.2 Plywood

In order to reflect the effect of the difference in stress distribution in the two methods of test on wood perpendicular to the principal direction of stress, it was deemed necessary to test plywood having a ratio of parallel to perpendicular to face grain wood approaching 1.00. Therefore 7 ply panels were selected for this indicative test program.

Sheathing grade Canadian softwood plywood (3/4 inch - 7 ply) manufactured to CSA O151-1974 was randomly sampled from two mills. Later, species identification determined that six of the sampled panels were made of Western Hemlock (Tsuga heterophylla) and four of the panels were of Western White Spruce (Picea glauca).

Defect free all Douglas fir (Pseudotsuga menziesii) plywood (11/16 inch - 7 ply) was custom made by one mill.

3.3 Grain Orientation

To reflect the interaction of the effects of grain orientation to principal stress, ASTM tests were conducted in the 0° and 90° orientation.

4.0 SPECIMENS AND TESTS

4.1 Specimen Preparation

Two 15 inch by 24 inch ASTM specimens and two 20 inch by 20 inch RILEM/CIB specimens were cut from each panel as shown in Figure 1.

4.2 Test Methods

ASTM D2719-76

"Standard Methods of Testing Plywood in Shear-Through-Thickness, Method C, Two Rail Test for Large Specimens."

This testing program deviated from the standard ASTM test method in that steel rails 3.5 inches wide and gauges 5.5 inches long were used. This was done as tests conducted during Phase II of the COFI Plywood 781 program indicated the use of re-usable steel rails combined with the 5.5 gauge achieved the same or better accuracy and consistancy than the wood rails specified in ASTM.

During the test two gauges were used - one on each side of the specimen. Two load distortion curves were simultaneously plotted by two x-y recorders (see Figure 2) and two G values were calculated from the measured test results. The specimen G value was then calculated as the average of these two results.

RILEM/CIB-3TT

"Testing Methods for Plywood in Structural Grades for Use in Load-Bearing Structures."

Each of the 20 inch by 20 inch square specimens were tested twice - the second test being conducted with a given diagonal in a position 90° to the position during the first test. A load deflection curve was plotted for each test (see Figure 3), and a corresponding G value was calculated for each of the two tests. The specimen G value was then calculated as an average of these two results.

4.3 Supplementary Tests

The moisture content of each specimen was determined on an oven dry basis using the procedure specified in ASTM D2016-65, Method A. The overall average moisture content of all CSP specimens was 5.9% and of all DFP specimens was 7.1%. Difference measured between the individual specimens of one matched group was a maximum of 0.2%.

The quality of glueline of each panel was found to be satisfactory by means of a knife test.

5.0 CALCULATIONS

The moduli of rigidity were calculated from deformation data below proportional limit as follows.

5.1 ASTM Tests

$$G = 0.5 \frac{P_g}{d} \frac{\ell}{Lt}$$

where

G = modulus of rigidity in the plane of plies (lb/in^2)

P_g/d = slope of force/deformation diagram (lb/in)

ℓ = gauge length (in)

L = length of shear area (in)

t = average thickness of shear area (in)

5.2 RILEM/CIB Tests

$$G = \frac{3a^2 \Delta F}{2t^3 \Delta w}$$

where

G = modulus of rigidity (lb/in^2)

a = distance from the centre of the specimen to the point where the deflection is measured (in)

ΔF = increment of load applied at each corner on the straight line portion of the load/deformation curve (lb)

t = thickness of specimen (in)

Δw = increment of deflection relative to the centre of the specimen corresponding to ΔF (in)

Note: Different notation is used in ASTM and RILEM/CIB formulas to reflect exactly the notation used in the respective standards.

5.3 Statistical Treatment

The analysis is based on comparison of arithmetical means and coefficients of correlation. These statistics were selected as being the most meaningful tools for obtaining approximate comparison and trends considering the relatively small number of replicates.

The statistics for the two variables ($x = G_{ASTM}$, $y = G_{RILEM/CIB}$) were calculated as follows.

Mean $$\overline{X} = \frac{1}{n} \sum_{i=1}^{n} xi$$

$$\overline{Y} = \frac{1}{n} \sum_{i=1}^{n} yi$$

STD Deviation $$\sigma x = \left(\frac{\sum_{i=1}^{n} (xi)^2 - n\overline{X}^2}{n - 1} \right)^{\frac{1}{2}}$$

$$\sigma y = \left(\frac{\sum_{i=1}^{n} (yi)^2 - n\overline{Y}^2}{n - 1} \right)^{\frac{1}{2}}$$

Coefficient of Correlation $$r_{xy} = \frac{\sigma xy}{\sigma x \sigma y}$$

6.0 TEST RESULTS

The test results for sheathing grade Canadian softwood plywood and for defect free Douglas fir plywood are presented in Tables 1 and 2 respectively. For each type of plywood the modulus of rigidity is listed for each specimen, for the panel average, and for the overall average. Ratios between the RILEM/CIB panel average values and the ASTM average, 0° and 90° values and the corresponding ratios for the overall averages are also listed. The linear correlation of each set of pairs is quantified by a coefficient of correlation.

7.0 DISCUSSION OF TEST RESULTS

7.1 Interaction of Species and Grade

Based on overall averages, the ratio of RILEM/CIB results to ASTM results are 1.10 and 1.16 for sheathing grade CSP and defect free DFP respectively. The difference between these ratios could be caused by the simultaneous interaction of several factors such as species (the difference between G_{TL}/G_{LT} ratios), natural growth and manufacturing characteristics.

7.2 Interaction of Grain Orientation

The ratio of RILEM/CIB average results to ASTM 0° and 90° average results are respectively 1.12 and 1.08 for sheathing grade CSP and 1.19 and 1.12 for defect free DFP. These ratio differences are due to the different 0° and 90° ASTM results.

The ratio of these differences are 1.04 (1.12/1.08) for sheathing grade CSP and 1.06 (1.19/1.12) for defect free DFP. The similarity of these ratios are as expected as the constructions tested were very similar.

It should be noted however that previously developed experimental data (COFI Report 105) indicate that the $G_{0°}/G_{90°}$ relationships change significantly with constructions having significantly different ratios of parallel to perpendicular to face grain wood.

7.3 Correlations

Coefficients of linear correlation between the two test methods, based on average panel results, are 0.788 for sheathing grade CSP and 0.729 for defect free DFP. The better correlation for in-grade than for defect free plywood is likely due to some very large differences for individual pairs of variables for defect free plywood and the overmasking effect of in-grade plywood.

The correlations when based on average RILEM/CIB results and the ASTM 0° and 90° results are good for the 0° orientation but are poor for the 90° orientation. This means that a statistically justified modification factor for conversion of data from one test to another cannot be developed for 90° ASTM values unless the factor is conservative so as to compensate for the poor correlation.

8.0 CONCLUSIONS

Based on the results of this indicative test program, the following can be concluded.

- The difference between the G values determined by the RILEM/CIB (twisted plate) method and ASTM (two rail) method is significant. This difference may vary significantly with grade and species.

- Linear correlations between average $G_{RILEM/CIB}$ and G_{ASTM} results and between average $G_{RILEM/CIB}$ and $G_{0°ASTM}$ are good, between average $G_{RILEM/CIB}$ and $G_{90°ASTM}$ are poor. This would permit development of a reasonably reliable modification factor to convert data from one test to the equivalent of what could be expected from data of the other test providing ASTM 0° orientation or the average of ASTM 0° and 90° is used. Since the interaction of grade and species appears to be significant, such a factor would have to be developed for a given grade and species of plywood.

- Examination of other data indicates that an interaction between the test method and the construction does exist. Therefore any modification factors may have to be developed either for a given type of construction or for a group of similar constructions.

- Any statistically quantified modification factors would require a larger number of replicates.

TABLE 1

MODULI OF RIGIDITY AS DETERMINED BY THE ASTM AND RILEM/CIB-3TT TEST METHODS -
3/4 INCH - 7 PLY SHEATHING GRADE CANADIAN SOFTWOOD PLYWOOD

Average Moisture Content: 5.9%

PANEL NUMBER	MODULUS OF RIGIDITY (psi)						RATIO (R) and COEFFICIENT OF CORRELATION (r)					
	RILEM/CIB-3TT METHOD			ASTM D2719-76 METHOD			(4)/(7)		(4)/(5)		(4)/(6)	
	SPECIMEN I	SPECIMEN II	AVERAGE $\frac{(2)+(3)}{2}$	0° TO FACE GRAIN	90° TO FACE GRAIN	AVERAGE $\frac{(5)+(6)}{2}$	R	r	R	r	R	r
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	75770	80476	78123	75642	74538	75090	1.04		1.03		1.05	
2	83566	85458	84512	76678	80748	78713	1.07		1.10		1.05	
3	83348	77858	80602	71805	70075	70940	1.14		1.12		1.15	
4	88202	91003	89602	86495	83332	84913	1.06		1.04		1.08	
5	84111	74116	79114	65530	64933	65231	1.21		1.21		1.22	
6	73546	73870	73708	64903	74724	69813	1.06		1.14		0.99	
7	80030	80932	80481	71369	79707	75538	1.06		1.13		1.01	
8	82331	83333	82832	77165	69437	73301	1.13		1.07		1.19	
9	74586	79585	77085	63074	68767	65920	1.17		1.22		1.12	
10	76490	81793	79149	68626	79675	74150	1.07		1.15		0.99	
MEAN	80198	80842	80521	72129	74594	73361	1.10	0.788	1.12	0.881	1.08	0.485

TABLE 2

MODULI OF RIGIDITY AS DETERMINED BY THE ASTM AND RILEM/CIB-3TT TEST METHODS - 11/16 INCH - 7 PLY DEFECT FREE ALL DOUGLAS FIR PLYWOOD

Average Moisture Content: 7.1%

PANEL NUMBER	MODULUS OF RIGIDITY (psi)						RATIO (R) and COEFFICIENT OF CORRELATION (r)					
	RILEM/CIB-3TT METHOD			ASTM D2719-76 METHOD			(4)/(7)		(4)/(5)		(4)/(6)	
	SPECIMEN I	SPECIMEN II	AVERAGE $\frac{(2)+(3)}{2}$	0° TO FACE GRAIN	90° TO FACE GRAIN	AVERAGE $\frac{(5)+(6)}{2}$	R	r	R	r	R	r
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	120265	113204	116734	99116	89562	94339	1.24		1.18		1.30	
2	107690	104336	106012	83305	91026	87166	1.22		1.27		1.16	
3	108279	101985	105132	97554	100710	99132	1.06		1.08		1.04	
4	99678	99195	99437	80254	90030	85142	1.17		1.24		1.10	
5	106760	104752	105756	83055	91862	87459	1.21		1.27		1.15	
6	109933	98639	104286	97419	104178	100799	1.03		1.07		1.00	
7	101129	90025	95577	87719	89822	88771	1.08		1.09		1.06	
8	96624	89097	92861	73732	81314	77523	1.20		1.26		1.14	
9	90033	94723	92378	74367	80553	77460	1.19		1.24		1.15	
10	96480	88653	92567	70752	81504	76128	1.22		1.31		1.14	
MEAN	103687	98461	101074	84727	90056	87392	1.16	0.729	1.19	0.792	1.12	0.574

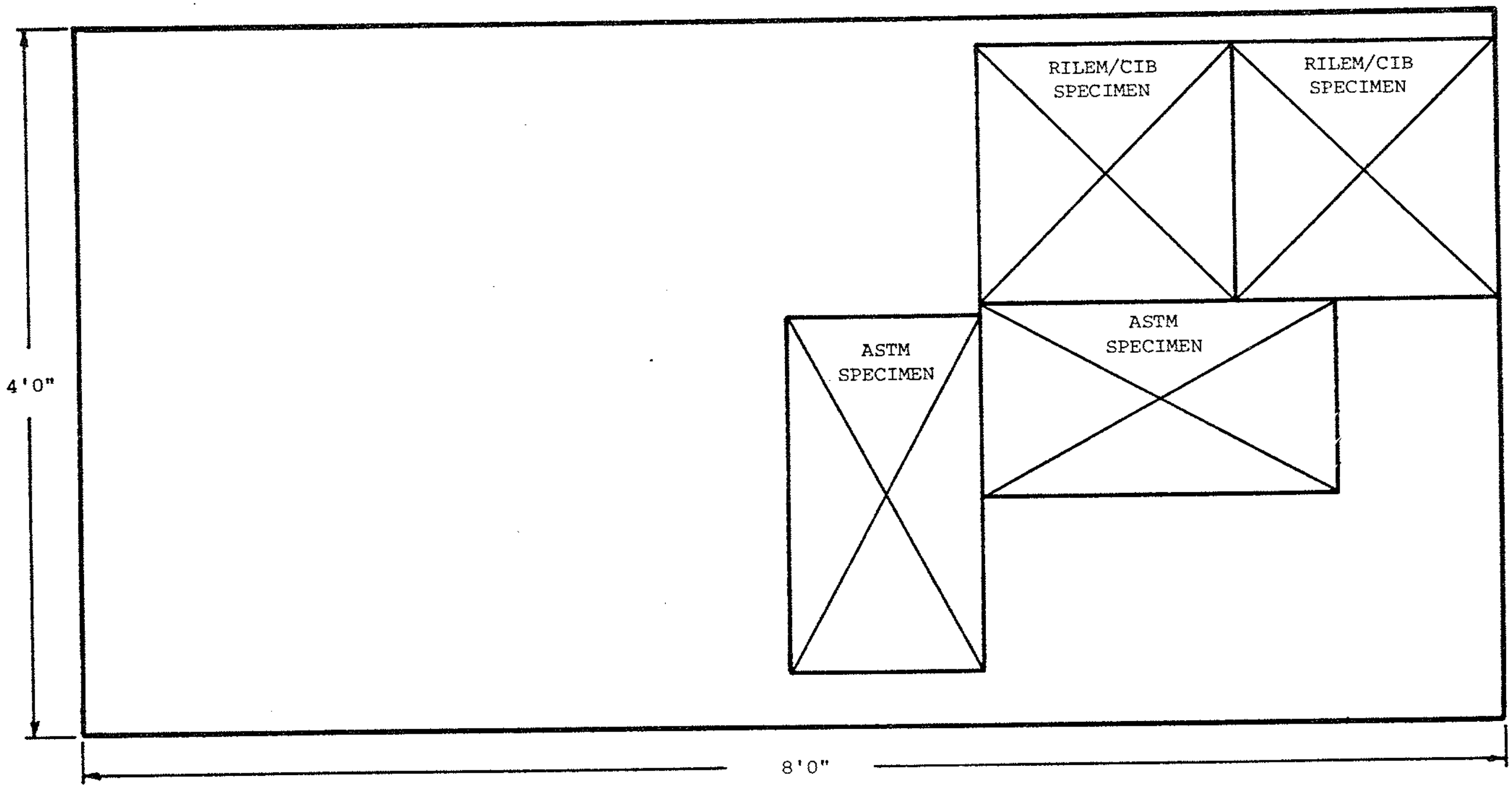

FIGURE 1

CUTTING PLAN

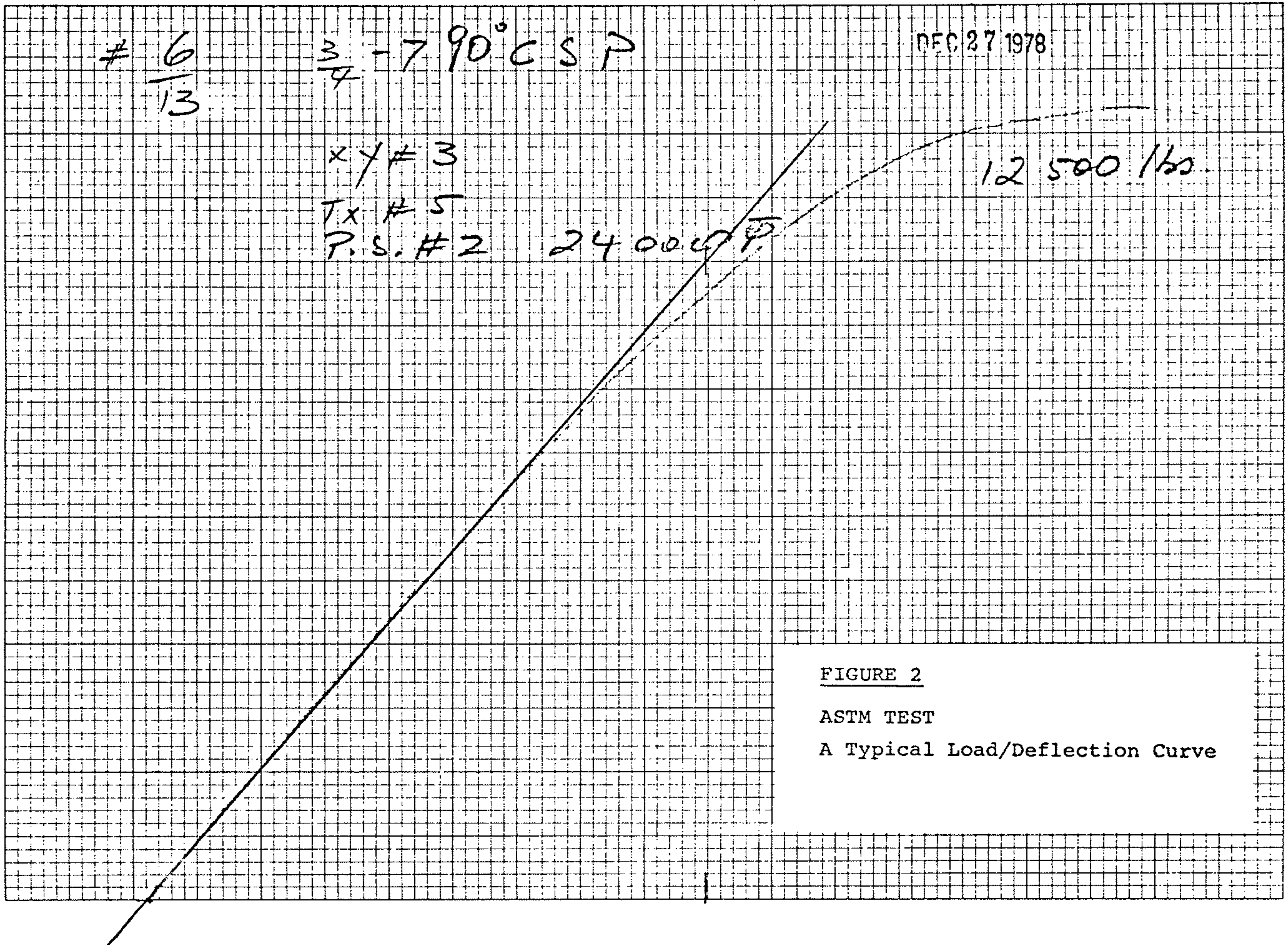

FIGURE 2

ASTM TEST

A Typical Load/Deflection Curve

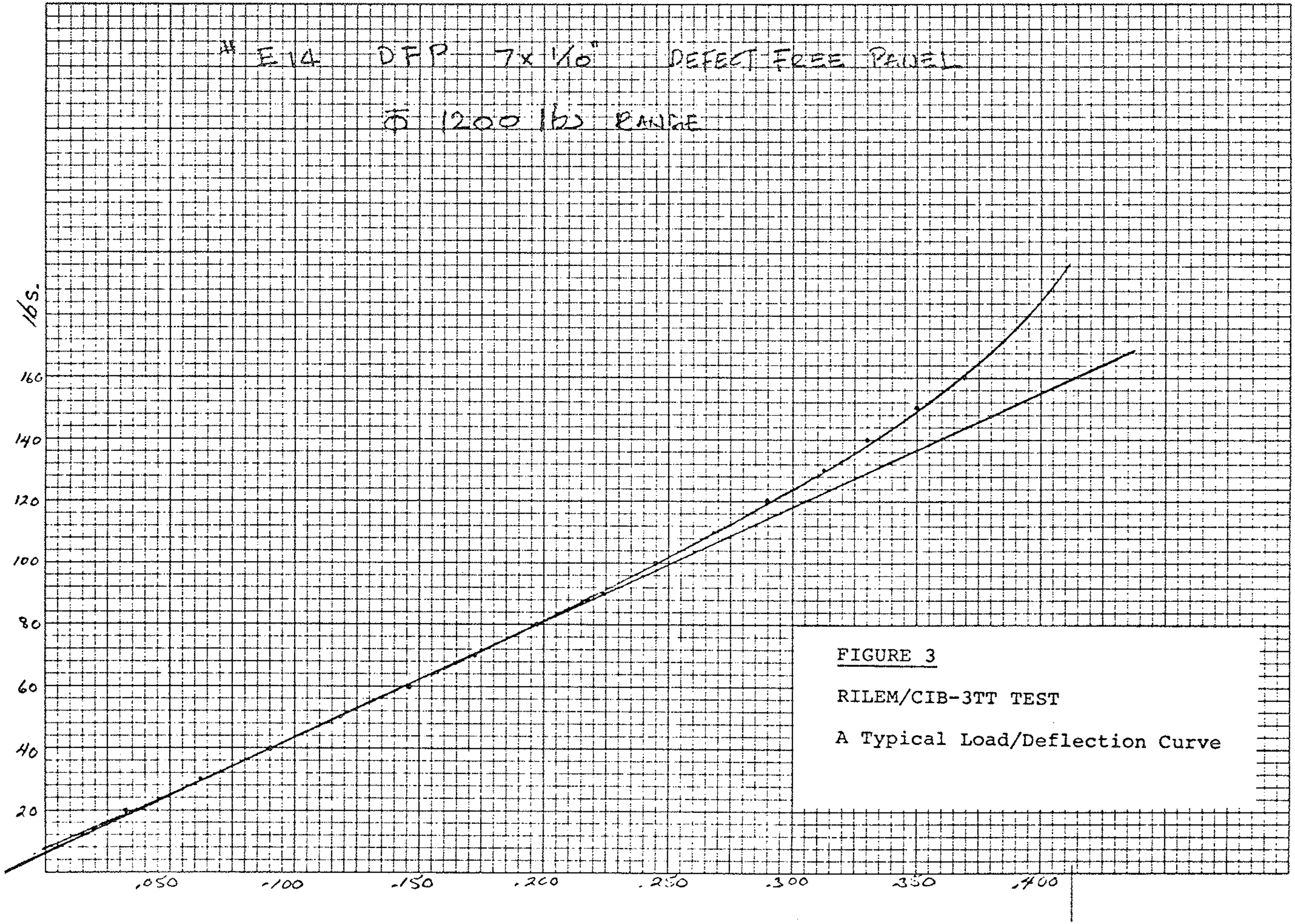

FIGURE 3

RILEM/CIB-3TT TEST

A Typical Load/Deflection Curve

CIB-W18/11-4-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SAMPLING OF PLYWOOD FOR TESTING STRENGTH

by
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MARCH 1979

SAMPLING OF STRUCTURAL PLYWOOD FOR TESTING STRENGTH

B Norén - Swedish Forest Products Research Laboratory, Stockholm

0 Introduction

At the meeting of CIB-W18 in Perth the author presented a second draft of "The Sampling of Plywood and the Derivation of Strength Values" (9-4-3). The meeting agreed that the sub-committee responsible for the plywood testing standard comprising Norén, Booth, Kuipers and Wilson, should be reconvened, together with Finnish and American representatives, to redraft the paper in a more precise format. This sub-committee has so far not done anything to the matter. Anyway, after a discussion with L G Booth and W T Curry 1979-03-02, the author has limited this third draft for a standard to the sampling subject.

1 Purpose

The sampling here described is used for testing plywood in order to establish characteristic strength and stiffness values for structural use.

2 Definition of the population

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

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

3 Sampling of panels

In a sample, the number of panels of each construction (grade, nominal thickness and lay-up) and manufacture shall be proportional to the percentage of volume produced. However, if proved by pilot testing or calculation (parallel-plies theory) that the characteristic strength of part of the population (panels of certain construction and/or from certain factories) by high probability is lower than the characteristic strength of the total population, then the final sampling can be limited to panels from that part (substitute population). In any case the number of panels in the sample must allow each strength property to be tested on specimens from at least 60 panels of each construction (thickness) and thereby from at least 15 panels from each factory.

4 Sampling of specimens from panels

4.1 The number of specimens and cutting schedule is given in ISO/TC 165 N, document 14E.

If a characteristic feature (such as a joint) occurs on a regular distance from the edges of the panels, the position of the cutting schedule relatively to the edges shall be changed at random from panel to panel.

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

CIB-W18/11-6-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

EVALUATION OF LUMBER PROPERTIES IN THE UNITED STATES

by
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VIENNA, AUSTRIA
MARCH 1979

Abstract

A research program at the U.S. Forest Products Laboratory will assess the performance of Light Frame structures. Particular emphasis is placed on analytical analysis of wall and floor performance. Structural Douglas-fir and southern pine lumber is being sampled and tested at lumber mill locations to provide data for the structural analysis. Testing to date is limited to a preliminary phase. Future sampling, testing, and analysis will be based on insights gained from the preliminary data and from supporting research on statistical sampling and analysis.

EVALUATION OF LUMBER PROPERTIES IN THE UNITED STATES

A Status Report

By

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and
J. H. HASKELL, Mathematical Statistician

Forest Products Laboratory,[1] Forest Service
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Introduction

The reliability of wood use in structures has withstood the test of inservice performance. Houses in the United States have been built basically by today's practices for nearly 100 years and others built by earlier practices are still in use. Wood systems other than houses are giving equally good service. Recent tests of full-scale floor and wall components and a full-scale house designed by presently accepted methods, lumber grades, and design stresses show that houses generally are overdesigned (6,11,12).[2] However, no analytical explanation for this performance has been completed, and both performance in use and laboratory evaluations of components and structures are contrasted with some recent tests of single pieces of full-size, stress-rated structural material which are showing some strength values to be less than published values for design use (8). This anomaly must be resolved if the efficient use of wood is to be continued.

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[1] Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

[2] Underlined numbers in parentheses refer to Literature Cited at the end of this report.

A comprehensive research program in light-frame construction was organized in 1977 by the Forest Products Laboratory with advice and counsel from representatives of the wood-producing industry, wood-using industry, government regulatory bodies, and universities. This program, a "Five-Year Action Plan for Light-Frame Construction Research"(7), covers total research needs in the light-frame construction, including performance factors such as structural, fire, thermal and moisture, acoustics, and durability. Figure 1 illustrates the overall approach to this research. Material property research was recognized as essential but was included at only a modest level. The assumption was that present lumber grades and grading systems were adequate in multiple-member systems. Without deviating from this basic assumption, steps have since been taken to strengthen the structural property research in the light-frame program. One motivation has been the current U.S. interpretations of tort liability.

The evolving concepts of tort liability in the United States pose serious problems for continued efficient use of wood in structures. When one facet of the material property-design-construction-end use sequence of building is found to be questionable, those responsibile must make changes to protect themselves from costly claims.

The current questions being asked about the reliability of lumber properties have been brought about by differing compounding factors. Pressures on wood resources in the post-World War II period led to

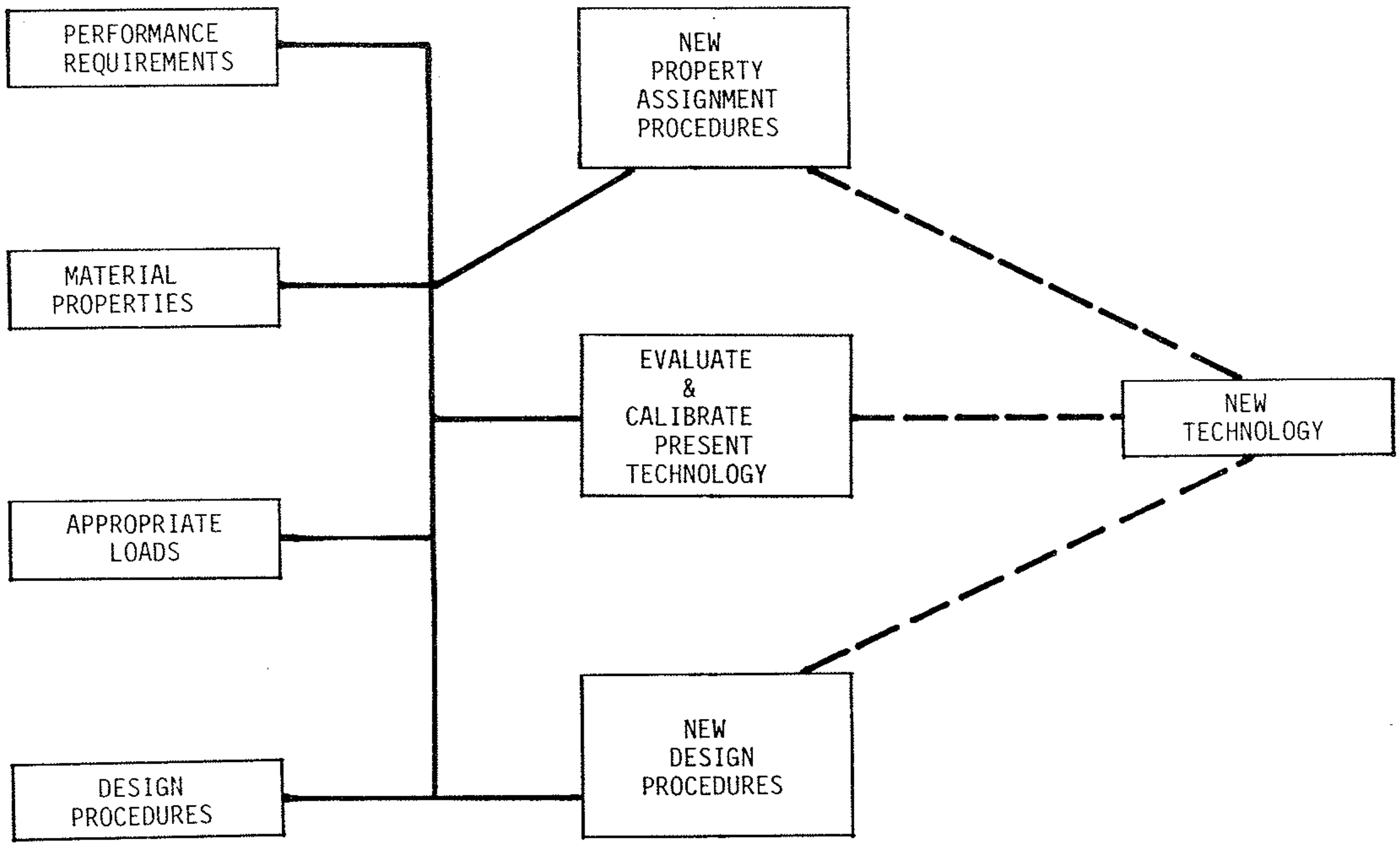

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Figure 1. Diagram of Forest Products Laboratory research on Light Frame structures.

expansion and extrapolation of earlier concepts of lumber grading and design application. Economic pressures now are forcing more refinement in designs. Advanced computer technology is influencing design decisions. These all demand more precise knowledge of material properties and interaction of design, material property, and end use.

Determination of the effects of such changes in practice is not simple. Material properties are only one link in the chain of events that make wood-frame systems perform so well--traditional design principles, loading assumption, and construction practices combine to produce structures that can be expected to last a lifetime or longer. What we have not described analytically is how these factors interact to produce long-lived structures.

The material property/engineering design emphasis (now commonly called the in-grade testing program) in the light-frame program is planned to answer three basic questions: (1) What are the properties of lumber furnished to the design/consumer? (2) What are the properties as developed by direct interpretation of American Society for Testing and Materials (ASTM) based grading procedures? and (3) How do structures perform analytically using these lumber properties? This program is a cooperative effort among the U.S. Forest Products Laboratory, major U.S. grading rules writing agencies, and major U.S. universities. Universities currently involved include the Universitv of Wisconsin, the State University of New York at Albany, Washington State University, and Colorado State University.

The purpose of this paper is to report the present goals and status of the major research on structural properties of lumber--the so-called "in-grade" testing plan for the United States.

The In-Grade Program

Objectives and Scope

The principal objective of the in-grade research is to develop a basic characterization of mechanical properties of lumber produced under the American Lumber Standard (ALS) (1). This characterization is to be suitable for analytical research to define the structural performance of light-frame buildings. The initial emphasis and research now under way will:

1. Characterize the bending strength and stiffness of the 2x8-inch No. 2 Joist and 2x4-inch STUD grades of Douglas-fir and southern pine determined to be "on grade" by agency inspectors. This will make available a structural characterization related directly to single ALS- and ASTM-based grades.

2. Characterize the bending strength and stiffness of lumber "as graded" by the mill. This sample may include for marketing purposes grades other than No. 2 and may include some lumber "off grade" as stamped. This then will allow a structural characterization related directly to lumber as it is currently produced, sold, and used under ALS.

3. Analytically model the performance of Douglas-fir and southern pine in conventional floor and wall systems.

Future research planning will consider other species, grades, and sizes.

Lumber Evaluation

Evaluation research is composed of four major activities: development and calibration of test equipment, development of a sampling plan, physical and mechanical testing, and analysis. These are outlined briefly as follows:

Testing Equipment.--The cooperating grading rules writing agencies developed portable, hydraulic machines to test lumber in bending. The agencies have accepted the responsibility for field calibration and maintenance of these machines.

The machines and practices differ from traditional laboratory devices and practices by (a) being operated at an approximate 17/1 span/depth ratio, (b) measuring deflection by loading head movement at the 1/3 point, (c) testing an edgewise orientation with the lumber supported horizontally, (d) measuring deflection between two prescribed load levels for E calculation, and (e) having the capability of very rapid loading. Since several of these features may influence modulus of rupture and modulus of elasticity values, a supplementary research effort is being carried out at Washington State University by Professor R. Pellerin to calibrate the field test devices relative to standard ASTM procedures. The calibration objectives are:

1. To determine the characteristic mechanical accuracy and precision of the field testing machine relative to standards appropriate for machine calibration.

2. To determine the relationship between the test results of the field machine versus standard ASTM D 198 and D 2915 test procedures with full-sized lumber (1,2).

There is a related concern that all field-test machines should be built and function on an equal performance basis. In 1979, there are already devices of at least four different designs being evaluated or used in North America. The calibration research will establish a basis for proposed ASTM standard specifications and calibration procedures for field-test machines.

Sampling.--The goal is to sample with a statistical design that will permit probabilistic statements about the production over the entire growth region. Figure 2 illustrates the mill locations for Douglas-fir and southern pine in the United States. Initial sampling emphasizes sufficiently intensive sampling to answer preliminary statistical questions: What are the important sources of variation? What are the magnitudes of variation?

The sampling plan format divides the species growth regions into geographic subregions based on general trends of topography and known timber growth characteristics. In this way, the general philosophy of homogeneity of lumber properties sampled within regions is approached. This desire is balanced by the need for a sufficient number of mills within each region to provide for adequate estimation of precision. Thus, in some cases, compromise has taken place in selection of regional boundaries to provide a balance of mills between regions.

The analytical basis of the sampling plan is finite sampling. This plan dictates the specification of sampling regions and selection sites (mills) and provides the basis of the probabilistic presentation of results. Details of this procedure are the subject of research at the State University of New York (SUNY)-Albany, sponsored by FPL.

Sampling of specimens is based on serial selection, that is, specimens will be taken in the order occurring in the bundle or stack.

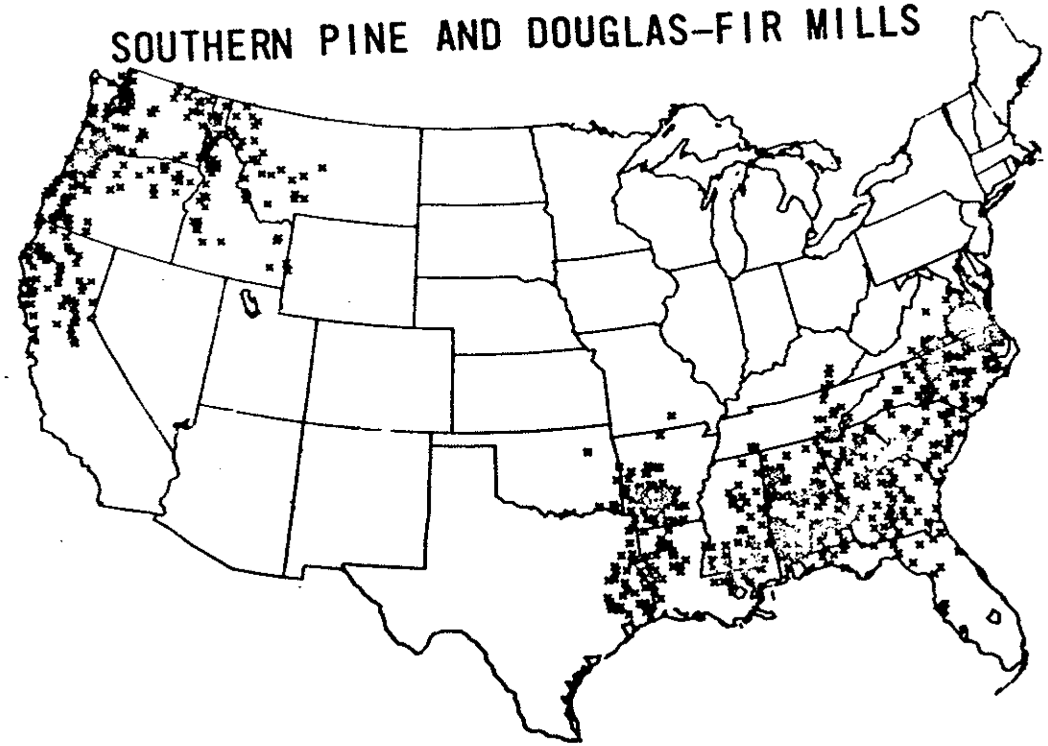

Figure 2. Lumber mill locations for Douglas-fir (left) and southern pine (right).

Serial lots contain 10 specimens. This procedure provides a close relationship to the order in which lumber is ultimately used. Bundles or stacks from which serial lots are selected are, in turn, selected in a systematic manner to assure reasonable continuity in the selection process in all regions. More detail is provided in the Appendix.

While data analysis is not complete, it is possible to note some of the sampling difficulties encountered to date. The procedure for selecting package units from which to select lots required much interpretation by the sampling teams due to variation in mill practice. The most serious problem was the lack of inventory--the industry often was selling and shipping the lumber as fast as it was produced. For example, some mills had insufficient inventory, yet were currently producing; thus, a portion of the sampling was deferred to the following day.

Other similar experiences likely will influence a change for subsequent samples. An effort must be made to simplify the process while maintaining a relative uniformity in format from mill to mill, without bias. One aspect that was found satisfactory was the "on-grade" and "as-graded" lot selection.

Physical and Mechanical Testing.--Data collected from each specimen included actual size, actual grade as verified by a grading supervisor, measurements of grade-controlling characteristics, and moisture content. The latter was taken with a resistance meter. A record also was kept of the type of dry kiln and of mill production volume for possible statistical weighting of results.

Mechanical test data included a deflection increment obtained at preset load levels and a maximum load. In order to provide additional deflection information desired for floor and wall failure models, a load-deflection plot was obtained for each specimen. Details are included in the Appendix.

Analysis.--The primary statistical analysis pertains to the population characteristics of 10-piece lots of lumber. The characteristic of most interest is some lower tail property of the distribution. Generally, the 5th percentile is the one used in discussion, but the analysis has the capability of estimating any tail property. The statistical confidence needed is, of course, a judgment. A typical desired statement might be "with a certain confidence, P percent of the average lot stiffness values exceed some design target E_p."

One objective of initial sampling is to develop insights on the question of confidence that can be obtained from actual sampling. Subsequent sampling then will be designed with a specific confidence level as a target. It is also desirable to make statistical inferences about piece properties. The proper statistical design is based on the statistical research being supported at the State University of New York (SUNY)-Albany and the University of Wisconsin.

The research at SUNY focuses on the development of statistical theory necessary to compute confidence intervals for finite population quantities and tolerance regions. Present theory does not allow adequate confidence statements for finite populations. The best method available to estimate the necessary parameters for finite populations will be judged by the confidence statements the method will allow. The research will not be complete this year.

The research collaboration with the University of Wisconsin is devoted to the Weibull distribution. The three-parameter Weibull family of statistical distributions is the most commonly employed parametric model for the strength properties of lumber. This investigation was primarily concerned with the derivation of a lower tolerance bound when the underlying strength distribution is known to belong to this Weibull family. The approach was based on the large sample properties of the maximum likelihood estimator, which therefore only produces an approximate tolerance bound.

By Monte Carlo simulation, the accuracy of the approximate tolerance bound based on the Weibull distribution and its coverage was examined. This bound was compared with its chief competitor, the nonparametric tolerance bound. Further, as part of the comparative study, tolerance bounds based on normal and lognormal parent distributions were considered. Finally, the properties of these procedures were considered under a few lognormal and normal parent populations.

The results indicate that for small samples (n=70) the approximate tolerance bound is not good in terms of the actual confidence obtained. Figure 3 shows that while 95 percent confidence is the target, in all cases only 85 percent confidence was obtained for the 5 percent lower tolerance region used. For larger samples the approximation will improve, but for the n=200 case investigated the actual confidence still did not meet the expected confidence. Generally, if it is necessary to have precise control over the actual confidence, only the nonparametric procedure will provide this assurance. Figure 3 illustrates that the nonparametric tolerance procedure is consistently closest to the target.

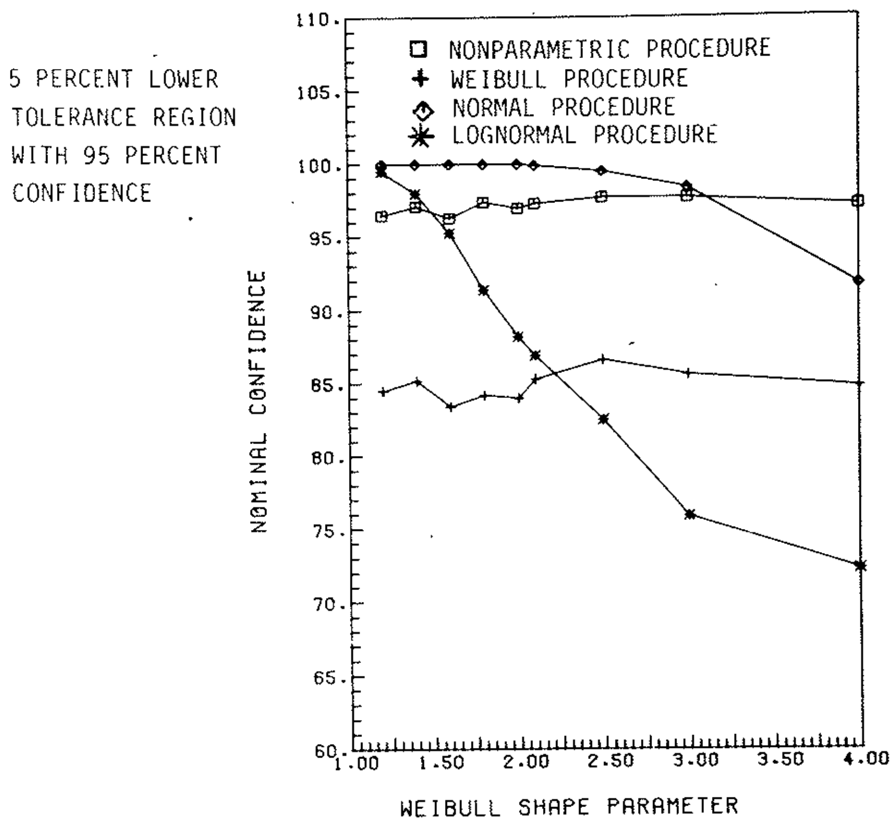

Figure 3. Comparison of procedures when the Weibull distribution is the underlying parent population (n=70). The expected confidence is 95 percent.

Further, if one assumes the incorrect parent population for estimating tolerance limits, Figure 2 also illustrates the results. In the example, the underlying distribution is a Weibull and the normal and lognormal tolerance limits are shown to be substantially in error.

The consequences of this research may be far reaching as it may influence reliability based design concepts as well as suggesting more use of nonparametric statistics.

Analytical Modeling of Structural Performance

Wall Performance.--To determine how walls perform under axial and bending loads, the cooperative study conducted with Oregon State University (OSU) will characterize wall performance using the properties determined from the lumber evaluation for STUD grade Douglas-fir and southern pine. The objective of the study will be to develop wall strength and stiffness distributions. Such distributions will be interpreted as representing acceptable walls and will serve as a guidelines for future comparisons using other lumber species. Professor A. Polensek is the principal investigator at OSU.

Floor Performance.--An analysis of the performance of floors will be made with Colorado State University (CSU) using the results of the survey of Douglas-fir and southern pine 2x8's. This will be based on current CSU research devoted to development of a floor failure model. Floor strength and stiffness distributions will serve as an acceptable guideline for future comparison using other grades and species. Professor D. Vanderbilt is the principal investigator at CSU.

Future Research

Planning for Part II of this program will be initiated in 1979. Current tentative plans suggest incorporation of tension and compression tests as well as bending. The statistical goals and procedures are not yet clear. The cost of breaking all specimens sampled, thus achieving information on the entire property distribution, is being weighed against other techniques such as proof loading. If proof loading is used, it is likely some portion of the entire distribution would be tested to gather sufficient information for reliability-based design.

Proof-loading procedures adopted would be influenced by a survey of current proof-loading research in the U.S. and Canada by Professor R. Johnson of the University of Wisconsin. Professor Johnson's paper is under preparation for publication. The following are highlights:

(1) An important aspect of proof-loading concerns the estimation of a specified lower percentile of the population. The goal here may be either a point estimate or a tolerance bound. Rather than use the sample 5th percentile as an estimate of the population value, Madsen and Nielson (9) use a method of smoothing in which a straight line is fitted to the two sample points below and the two above the 5th percentile. The estimated value of the population 5th percentile is read from the fitted line at the cumulative frequency of 0.05.

(2) The contributions of Warren and Glick treat the situation where the sample size n has already been selected and is fixed (5, 14). They suggest that, if only the i-th order statistic is needed to obtain the

tolerance bound, there is no need to break all n boards in the sample. Rather, one should proof-load with loads that vary according to the preceeding experimental outcome.

The Warren-Glick adaptive sequential procedure could well be implemented in laboratory situations where specimens are costly and those that do not fail can be used for other purposes. Some feel that the scheme may be too cumbersome to employ in on-site testing at a lumber mill. In may be difficult to change the proof-load too many times in this adaptive manner.

However, in the in-grade testing scheme, some adjusting must be done if not enough pieces are being broken. In its favor, the Warren-Glick scheme guarantees that the weakist i piece will be broken. The method used by Madsen cannot provide an absolute guarantee of this while maintaining one or two fixed settings for the proof-load.

(3) Warren also considers the situation where another quantity, correlated with strength, can be non-destructively tested (14). Called strategy A Extended, this modified sampling strategy tries to utilize the information obtained by ordering the specimens in the sample according to the covariate. For purposes of example, the modulus of elasticity E could be non-destructively measured on each piece in the sample and bending strength determined by destructive testing.

It must be noted that, although this method does reduce the expected number of pieces destroyed, it is also more difficult to implement. All specimens must first be loaded in an apparatus to provide the values of the covariate E. Only after all of these are available can the specimens be ordered for the destructive testing phase.

(4) Other procedures are available in the statistical literature, but they may be unacceptable due to the fact that they do not have a fixed sample size or even an upper bound on the sample size.

(5) Researchers interested in probabilistic modeling (Zahn 15) of structures need to know more than the lower tail. The degree and accuracy needed is not yet clear. It is important to be aware of some of the implications regarding probabilistic design if the design information originates from proof-loading schemes. Research on this issue appears limited to two recent theses. Using a simulation approach, DeBonis (4) presents a favorable comparison of predicted system strength with the actual value obtained by constructing and breaking a simple three or five-member system. From the results of Pellicane (10) who studied the adequacy of certain sampling schemes, it may be expected that proof-loading methods designed to determine the 5th percentile will not provide adequate information for probabilistic design. Some observations on strength must be taken under higher loading.

The survey results by Johnson suggest further options in proof-loading that should be explored. Perhaps more important, however, is the potential conflict between needs of reliability-based design and simple proof-loading methods. More research is needed.

Conclusion

Evaluation of properties of structural lumber has been initiated in the United States with a preliminary sampling of Douglas-fir and southern pine 2x4 STUD grade and 2x8 No.2 grade. Sampling of lumber by serial lots was successful. The procedure for mill and lot selection may require revision in future sampling in order to simplify the procedure. Data from the preliminary phase will be used to plan a more comprehensive sample. The latter will be input to wall and floor system analysis to calibrate conventional light-frame design in the United States.

Supporting research on statistical methods suggests non-parametric statistics may be required if confidence goals are to be met for near-minimum properties. Several promising proof-loading procedures have been reviewed, but a potential conflict between distributional information and near-minimum estimates must be faced.

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APPENDIX

Sampling

The sampling procedures to be used in this portion of the study are based on the fact that the population of mills is finite. For example, approximately 600 mills produce southern pine. For this reason, finite sampling techniques rather than the usual infinite techniques are applicable to this study. The procedure to be used is usually called stratified cluster sampling. The stratification is due to the fact that regional differences within a species are expected. The sampling units are lots, and each mill is a cluster of lots.

Regional Organization

The fact that growth region differences within a species may produce lumber with different properties requires that attempts be made to account for these differences by stratification in the sampling design. Thus, for each species, attempts are made to define distinct growth regions for which there may be property differences. Within these regions, more homogeniety is assumed than between regions. For southern pine, this resulted in six regions (Fig. A-1) and for Douglas-fir, twelve regions.

Green/Dry Selection

For the Douglas-fir species, an additional factor is two levels of moisture content production, green and dry. The extent to which this is a problem is unknown at this time. The decision was made to sample mostly green lumber, with the assumption that any regional effect found would be the same for dry lumber. A small sample of dry lumber may permit evaluation of this assumption.

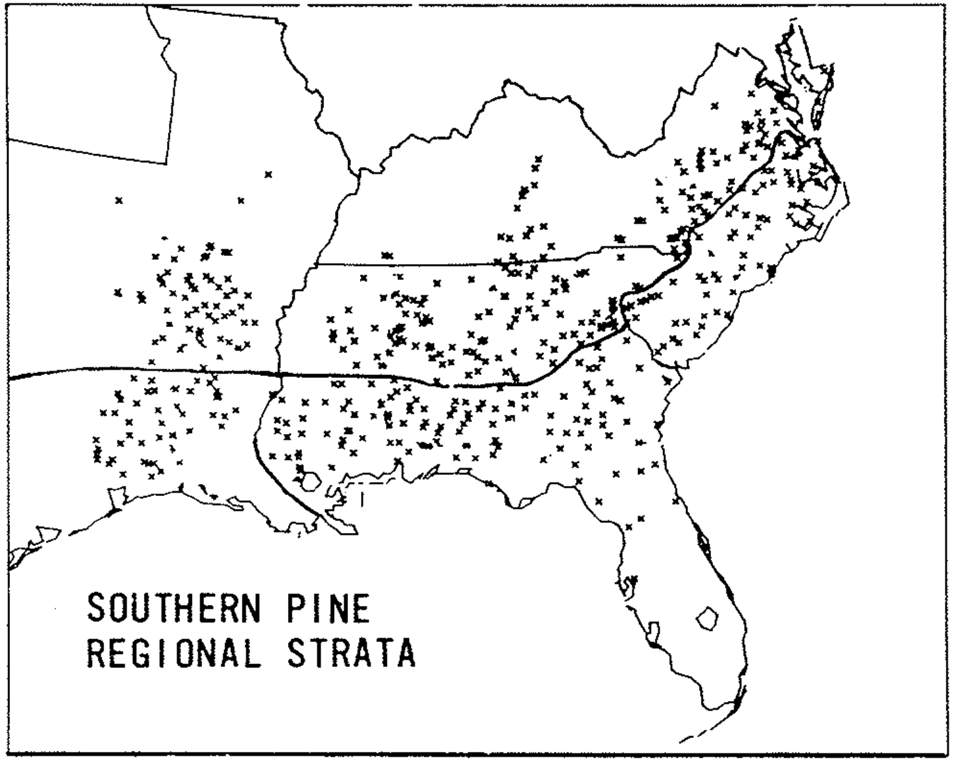

Figure A-1. Regional strata for Southern Pine.

Stud Production

The original intent of this study was to sample studs in the same proportion as the 2 by 8 material. With further discussions, it was determined that this was not feasible, since in most regions there are so few stud mills. Thus, initially, studs were sampled from mills which produced studs from veneer cores and those which produce from a more normal log supply. Comparison will determine whether these two types of producers have lumber with different properties.

Mill Selection

The mills within each region were partitioned into three classes based on volume production figures (3). Two mills in each region were chosen as sample sites in initial sampling--one mill from the high volume class for the region and one from either the middle or low volume class for the region.

Lot Selection

Within a sampled mill, 10-piece serial lots were selected for testing. Systematic lot selection was attempted in order to provide a better representation of the population. Random selection is ordinarily not feasible due to the quantities of material that would have to be moved to obtain a true random sample of lots.

Mechanical Test Procedures

Piece Orientation

Orientation procedures are designed to prevent bias in location of any strength-influencing defect in relation to the load application.

a. Wide face.--Pieces were placed in the test span with the grademark facing up and the grademarked end to the right of the operator facing the machine. In cases where pronounced crook was present, the piece was placed so the convex edge was in compression.

b. Length.--Pieces were placed in the testing machine so that the estimated maximum strength reducing characteristic was randomly located within the test span. If more than one defect present was judged equivalent, the defect nearest the center of the length was used to position the piece.

Loading Arrangements

a. Each specimen was tested in the condition found, and no specimen preparation was used for this test.

b. The specimens were tested with a total span-to-depth ratio as close as possible to 17 to 1 based on dry sizes.

c. The load was applied equally at the third points of the span.

d. Provisions were made to prevent lateral buckling in such a manner that friction forces were minimized.

e. The supports and the loadpoints were provided with a mechanism (such as rollers or pivots) to minimize the development of axial forces in the specimens.

f. The points where the forces were applied to the specimens were equipped with bearing plates large enough to prevent permanent crushing, but did not exceed 4 inches in length.

g. The accuracy of the measuring devices was verified at the beginning of the test and at least once for each 4 hours of testing using an accurate calibration device.

h. The stress was induced at a rate of approximately 16,000 pounds per square inch per minute. The rate of head travel varied by width.

i. The static load was calibrated to be within 1 percent at levels of a 500 pound load or higher.

j. Span length and location of loadpoints was within 1/4 inch.

Data Required

a. The modulus of elasticity was obtained by reading the difference in deflection at two preselected load levels of approximately 200 and 500 pounds.

b. Deflection measurements included a load-deflection plot to failure.

c. Load at failure was recorded.

CIB-W18/11-6-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STRESSES PERPENDICULAR TO GRAIN

by
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VIENNA, AUSTRIA

MARCH 1979

Prof. Dr.-Ing. K. Möhler, Universität Karlsruhe (TH)

Stresses Perpendicular to Grain

Comparing the curves given by Barret for Douglas-fir with test values received at testing laboratories at Karlsruhe and Stuttgart with European Whitewood the result is the following:

1. Using blocks, a good agreement could be noticed, whereat both the mean-values and the 5%-fractiles were insignificant lower for European Whitewood. As well the mean - as the 5%-values for blocks were considerably lower than for tapered or curved beams (fig. 1). On contrary the allowable tensile stresses perpendicular to grain for normal load and usual climate conditions are greater for big blocks than for tapered or curved beams (fig. 2). This seems to be a discrepancy to the test results obtained.

2. Listing the test results for tapered or curved beams into the diagram containing the mean- and 5%-curves given by Barrett a good agreement was shown for "big" beams, whereas for "smaller" beams the strength values obtained were lying deeper than expected by the curves (fig. 1). This starts the question if a k-factor as given for blocks wouldn't fit the test results better, i.e. constant factor until e.g. $V = 0{,}02\,m^3$. For greater volumes a diminishing factor in relation to the volume of the beams should be used also for European Whitewood.

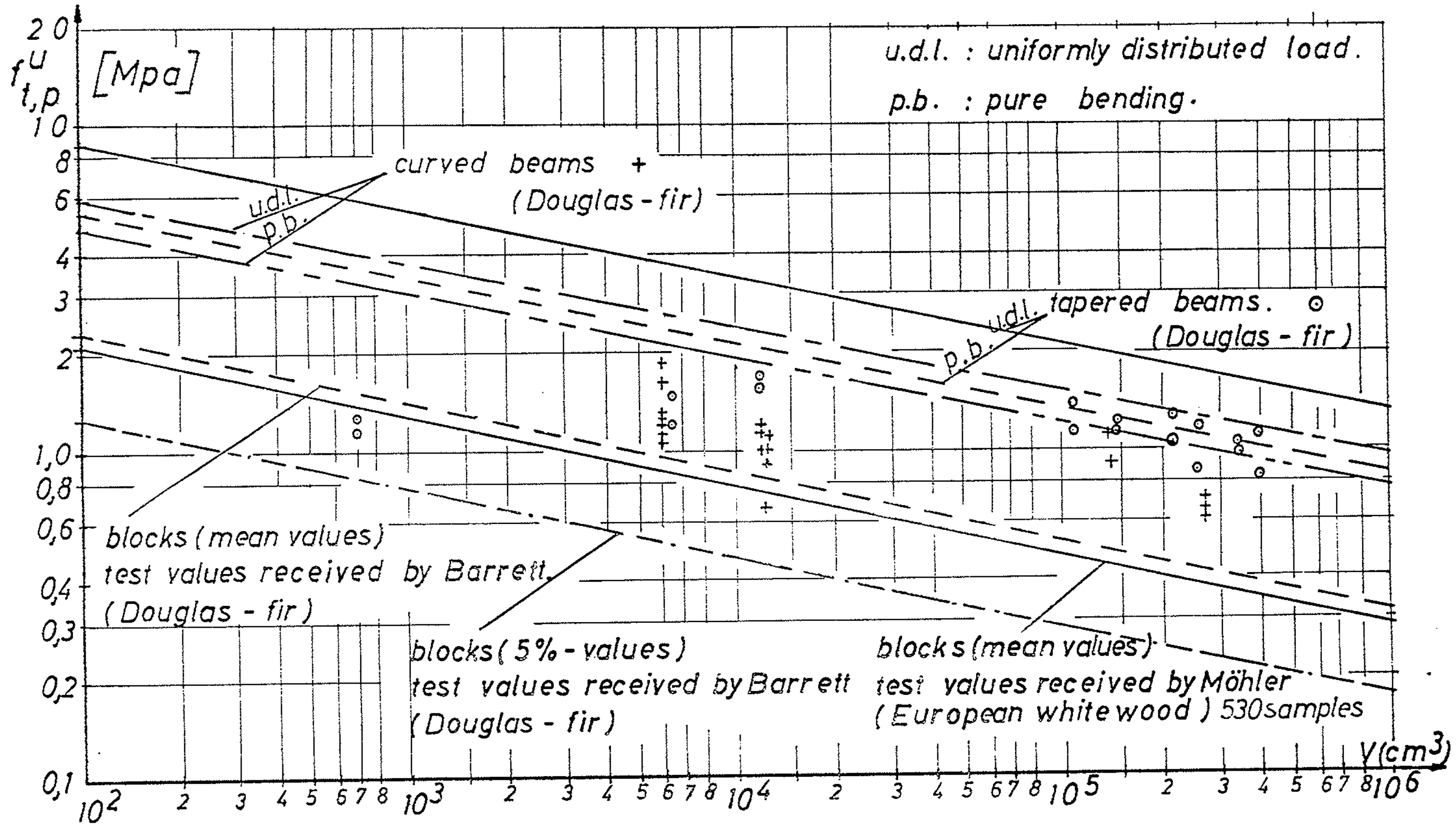


Fig. 1 Relation between the tensile strength perpendicular to grain $f^u_{t,p}$ and the volume V

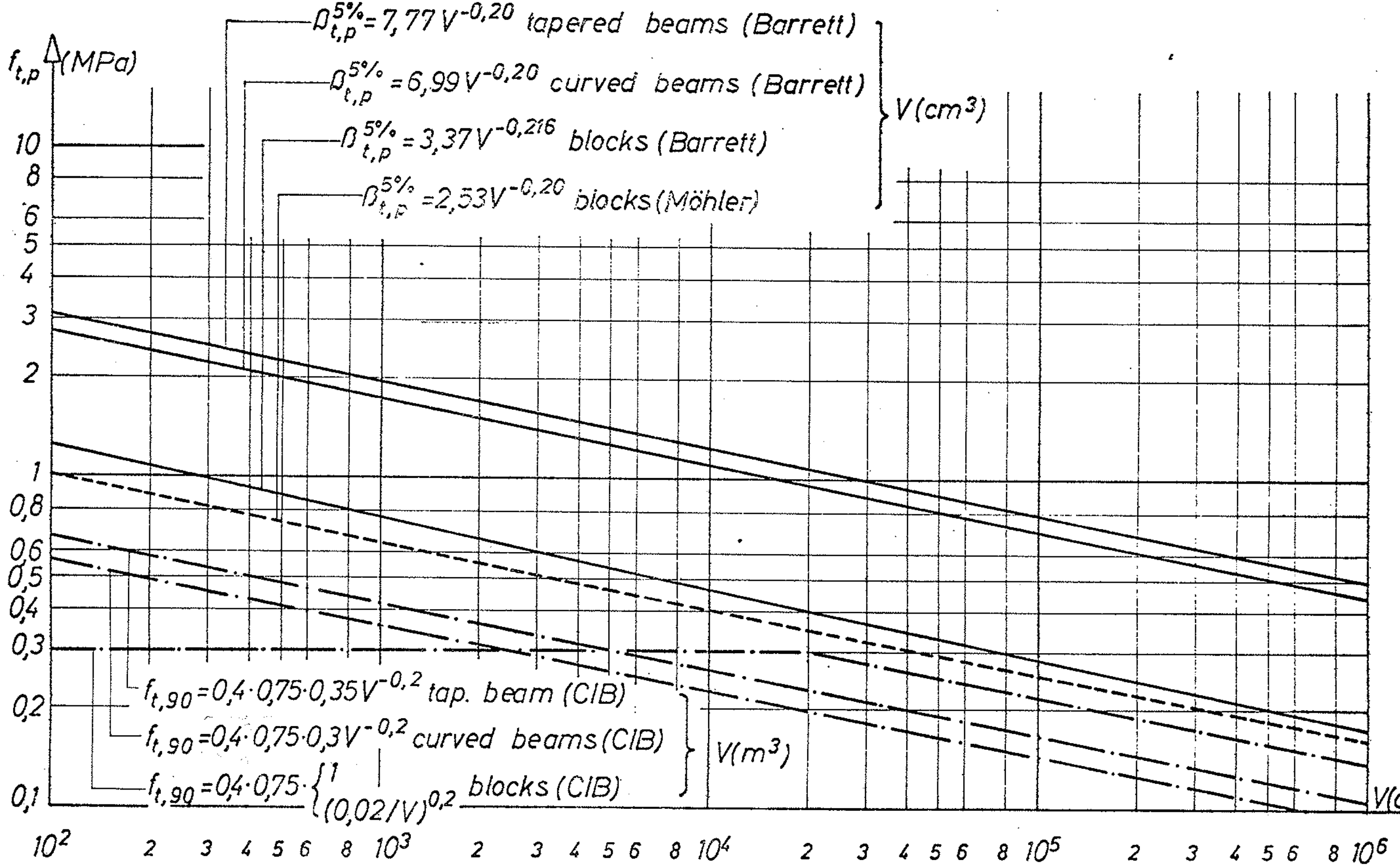


Fig.2 5% values and allowable values for tension stresses perpendicular to grain

CIB-W18/11-6-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CONSIDERATION OF COMBINED STRESSES FOR LUMBER AND GLUED-LAMINATED TIMBER

(Addition to Paper CIB-W18/9-6-4)

by
K Möhler
University of Karlsruhe
FEDERAL REPUBLIC OF GERMANY

VIENNA, AUSTRIA

MARCH 1979

Addition to Paper CIB W 18/9-6-4

"Consideration of Combined Stresses for Lumber and Glued Laminated Timber"

Finding large deviations partly between the curves given and the test values received, further investigations, only using glu-lam, were performed. As shown in figure 1 and 2 despite of the little number of additional tests a much better fitting of the test values by the calculated curves is given. Thus a good agreement between the given relation and the bearing capacity of tapered beams can be noticed.

Taking into account a savety factor of about 3 versus the mean values, the German Standard DIN 1052 will recommend the following formulas :

Slope on the compression face :

$$k_c = \frac{1}{14 \cdot \sqrt{\frac{1}{14^2} + \left(\frac{\tan\varphi}{2.4}\right)^2 + \left(\frac{\tan^2\varphi}{2}\right)^2}}$$

$$\cong \frac{1}{\sqrt{1 + 34\tan^2\varphi + 49\tan^4\varphi}}$$

Slope on the tension face:

$$k_t = \frac{1}{14 \cdot \sqrt{\frac{1}{14^2} + \left(\frac{\tan\varphi}{1.2}\right)^2 + \left(\frac{\tan^2\varphi}{0.25}\right)^2}}$$

$$\cong \frac{1}{\sqrt{1 + 136\tan^2\varphi + 3136\tan^4\varphi}}$$

In these formulas φ is the angle between the tapered face and the grain direction. The allowable compression or tension stresses at the tapered face of a beam are then given by the relation

$$f_{c,t}(\varphi) = f_{c,t}(\varphi = 0^\circ) \cdot k_{c,t} \, .$$

The $k_{c,t}$- values in relation to the angle φ are drawn in figure 3.

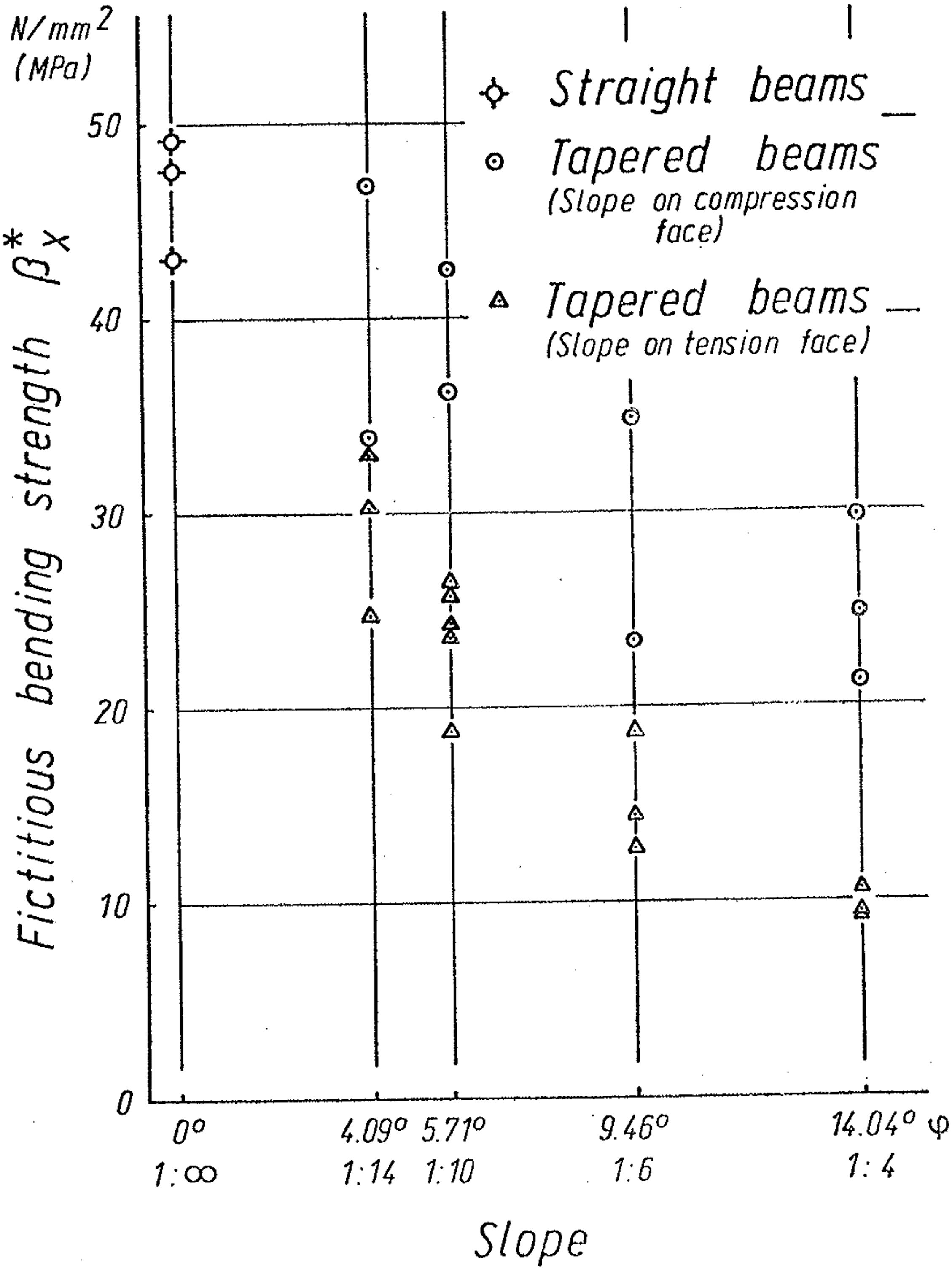


Fig.1 Fictitious Bending Strength at Various Slope Angles for Glulam

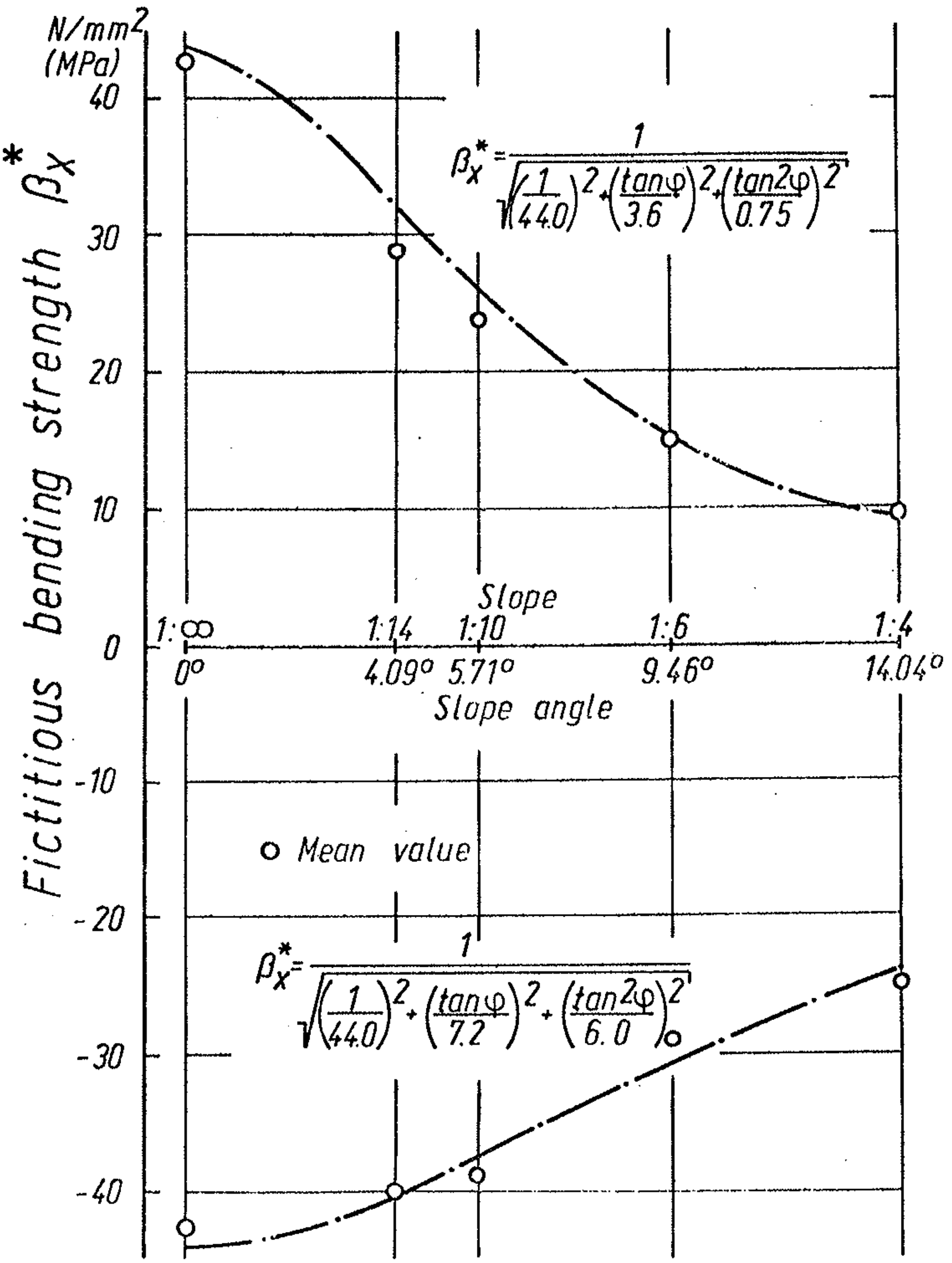


Fig.2 Fictitious Bending Strengths at Various Slope Angles for Glulam

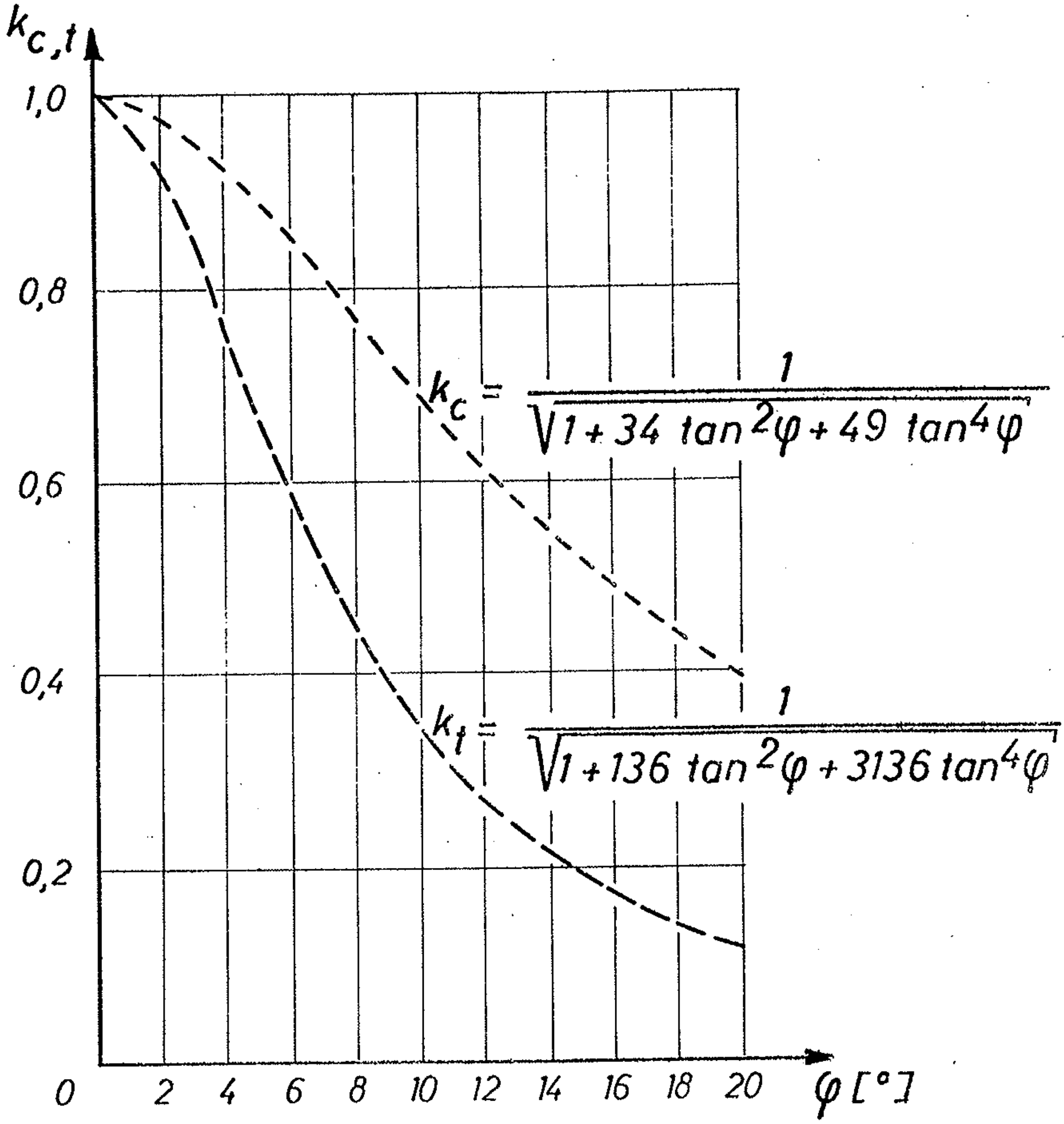


Fig. 3 $k_{c,t}$-values in relation to the slope-angle φ

CIB-W18/11-7-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

A DRAFT PROPOSAL FOR AN INTERNATIONAL STANDARD

ISO Document ISO/TC 165N 38E

VIENNA, AUSTRIA

MARCH 1979

ISO/TC 165 N **38E**

1979-01-22

(pages 1-17)

ISO

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION

TECHNICAL COMMITTEE 165

Timber Structures

Draft Proposal DP .... for an international standard:

TIMBER STRUCTURES

JOINTS

DETERMINATION OF STRENGTH AND DEFORMATION CHARACTERISTICS OF MECHANICAL FASTENERS

1. Draft, January 1979

HJL/AS/ep

CONTENTS

*Comments on misprints and linguistic comments on the french translation, ISO/TC 165 N 13F, are not mentioned.

0 INTRODUCTION

Developments in the field of load-bearing timber structures require that joints made with mechanical fasteners shall be tested to obtain information about their strength and deformation (slip) characteristics.

This Recommendation gives some general principles which should be followed in order to achieve comparability of results from investigations carried out in different laboratories. Standard rules for the determination of characteristic strengths and design loads for particular types of mechanical fasteners will be given in separate standards.

This international standard is based on Joint Recommendations from working commission W18 - Timber Structures of CIB* and committee 3TT - Timber Testing of RILEM**, who will also prepare the basis for the above-mentioned supplementary standards.

*International Council for Building Research Studies and Documentation.
**Reunion International des Laboratoires d'Essais et de Recherches sur les Materiaux et les Constructions.

0 INTRODUCTION

Developments in the field of load-bearing timber structures require that joints made with mechanical fasteners shall be tested to obtain information about their strength and deformation characteristics.

The present recommendations give some general principles which should be followed in order to achieve comparability of results from investigations carried out in different laboratories.

Standard rules on the determination of characteristic strengths or allowable loads for particular types of mechanical fastener will be given in CIB-RILEM Timber Standard 00 "Recommendations for the evaluation of results of tests on joints made with mechanical fasteners for use in load-bearing timber structures".

This International Standard is based on joint recommendations from Working Commission W18 - Timber Structures of CIB* and Committee 3-TT - Timber Testing of RILEM+.

Comments received:

Australia

It is difficult to make general comments without knowing how the test results so obtained are to be used in the derivation of working design loads. The proposed method does not appear to have any relation to the type of loading to which joints would be subjected to in practice, nor to give any more reliable or valuable information than the normal method of straightforward static loading to failure.

Canada

In our view, this document does not really give methods of determining strength and deformation characteristics of mechanical fasteners. It simply makes certain statements and references another standard. In addition, in the scope reference is made to further annexes which are in preparation. It is difficult, therefore, to comment on the document as a whole without being able to review the annexes which will likely contain methods of testing joints other than with punch metal plate fasteners.

1 SCOPE

This International Standard gives methods for the determination of the strength and deformation (slip) characteristics of joints made with mechanical fasteners.

Detailed procedures appropriate to joints made with specific fasteners will be given in related, separate International Standards.

It is recognised that for some special types of fasteners not covered by the annexes e.g. glu-lam rivets, modifications of the test procedure may be necessary.

1 SCOPE

This International Standard gives a method for the determination of the strength and deformation (slip) characteristics of joints made with mechanical fasteners.

Detailed procedures appropriate to joints made with specific fasteners are given in the annexes[1] to this International Standard.

Requirements of the materials of the joints, their geometry, workmanship, and storing depend on the purposes of the tests and are outside the scope of this standard.

Comments received:

Canada

The scope of the document should outline its contents in such a way as to prevent ambiguity. In our opinion, the proposed scope does not do this. In addition the scope includes an exclusion which we question. After all, the strength and deformation characteristics of mechanical fasteners depends, to a certain extent, on the materials and geometry of the joints.

France

Nous estimons que le dernier alinéa devrait soit être exclu de la norme, soit placé en note infra-paginale.

2 FIELD OF APPLICATION

This Recommendation is applicable to joints made with mechanical fasteners used in statically* loaded timber structures.

3 REFERENCES

*In case of time-dependent live loads with frequencies higher than about 0.4 of the natural frequency of the structure, dynamic effects must be expected.

2 FIELD OF APPLICATION

This International Standard is applicable to joints made with mechanical fasteners used in statically loaded timber structures.

NOTE - In case of time-dependent live loads with frequencies higher than 0.3 to 0.5 of the natural frequency of the structure, dynamic effects must be expected. In many cases, such as floors of ballrooms, gymnastic halls, etc, these effects have already been taken into account by the introduction of equivalent live loads in the loading standards and/or by stiffness requirements.

Comments received:

Canada

The note here refers to time-dependent live loads. It is not clear to us what effect that note is expected to have on the application and use of the tests covered in the document.

France

Nous souhaitons que la note soit exclue du texte de la norme, les effets dynamiques ne devant pas être prévus dans ce document.

4 SYMBOLS

F	applied load, in N
F_{max}	maximum load, in N
F_{est}	estimated maximum load, in N
k	slip modulus, in N/mm
v	joint slip, in mm

Subscripts for the joint slip, v, relate to load points in figure 2 and are defined in 8.

4 SYMBOLS

F_u	maximum load, in N
F_{est}	estimated maximum load, in N
k	slip modulus, in N/mm
v	joint slip, in mm

Subscripts for the joint slip, v, relate to load points on Figure 2 and are defined in Section 8.

Comments received:

The Netherlands

F_u should be F_{max}; see also ad 8.

F_{est} for estimated maximum load is not complete. Better, although longer, may be $F_{max,est}$.

5 CONDITIONING OF TEST SPECIMENS

Attention should be paid to the conditioning of the timber before the manufacture of the joint and also to the conditioning of the joints as a whole before testing.

The preconditioning should be conducted in such a way that a similar performance of the test joints and the joints in a structure can be guaranteed. It is important that the moisture content should be able to influence the strength properties of the timber and the occurrence of gaps etc, through shrinkage, in a realistic manner.

Detailed requirements for specimens made with specific types of fasteners are given in the relevant related standards*.

*Standards giving test methods for joints made with punched metal plate fasteners, nails and staples are in preparation.

5 CONDITIONING OF TEST SPECIMENS

Attention should be paid to the conditioning of the timber before the manufacture of the joint and also to the conditioning of the joints as a whole before testing.

The preconditioning should be conducted in such a way that a similar performance of the test joints and the joints in a structure can be guaranteed. It is important that the moisture content should be able to influence the strength properties of the timber in a realistic manner, and that the shrinkage should allow realistic gaps to appear in the joints.

Comments received:

Canada

The statement of this paragraph is too general and too ambiguous, open to different interpretations by different people.

The Netherlands

The proposal to condition test pieces before and after fabrication of the test joints according to the use of the structure seems justified in itself. Difficulties however arise:

- test results become again less comparable
- interpretation of the effect of moisture content becomes difficult
- the conditions of use of joints to be used in a broad range of structures are not known beforehand.

It is proposed that a paragraph should be added, giving one preferred method of conditioning that should be used in any case in all test programs.

United Kingdom

Conditioning of test specimens

We feel it would be useful to include a note on climatic conditions as in the RILEM and CIB-W18 drafts. The clauses in the annexes dealing with the conditioning of specimens with different types of fasteners could then be linked to these and in the future document "Evaluation of Data" modification factors could be introduced to convert this data from one climate to another.

6 FORM AND DIMENSIONS OF TEST SPECIMENS

The test joints shall be of such a realistic form and dimensions that the necessary information about the strength and deformation of joints in service can be obtained. Detailed information about the form and dimensions of the test specimens suitable for different types of mechanical fasteners are given in the relevant related standards*.

*See footnote to 5.

6 FORM AND DIMENSIONS OF TEST SPECIMENS

The test joints shall be of such a realistic form and dimensions that the necessary information about strength and deformation in actual service can be achieved.

Detailed information about the form and dimensions of the test specimens suitable for different types of mechanical fasteners together with the number of tests required is given in the relevant annexes.

Comments received:

Canada

The same comments as for 5 above.

The Netherlands

Can the first alinea be read to give an opening to the use of scale tests? This is not the intention.

A general information about the number of tests being sufficient to ensure a statistically justified evoluation should be added in the first alinea or in a seperate clause (cf. remark on page 2).

7 APPARATUS

In addition to equipment for measuring the geometry of the test specimens, moisture content etc, the following shall be available:

a) A testing machine able to apply and record load with an accuracy of ± 1 per cent or better;

b) Equipment to measure joint slip under load with an accuracy of ± 1 per cent or better, or for slips of less than 2 mm with an accuracy of ± 0.02 mm. The equipment shall ensure that eccentricities, twist etc. have no influence on the measurements*.

*Equipment that can continuously record load and slip is recommended; exceptionally slips may be measured at chosen load levels provided the measurements can be made without significantly influencing the continuity of the load application. A sufficient number of load levels must be chosen to ensure that the calculations (see 8.5) and the adjustments (see 8.6) can be made.

7 APPARATUS

In addition to equipment for measuring the geometry of the test specimens, moisture content, etc, the following shall be available:

a A testing machine able to apply and record the necessary load with an accuracy of ± 1 per cent.

b Equipment to measure the mutual displacements and geometry of the test specimens with an accuracy better than ± 1 per cent, or for displacements less than 2 mm to an accuracy of ± 0,02 mm. The equipment shall ensure that eccentricities, twist, etc have no influence on the measurements.

c Equipment that can continuously record coherent values of loads and displacements is recommended. Exceptionally, it may be accepted that the displacements are only measured at chosen load levels provided the measurements can be carried out without significantly influencing the continuity of the load application.

Comments received:

8 LOADING PROCEDURE

8.1 Estimation of maximum load

The estimated mean maximum load, F_{est}, for the type of joint to be tested shall be determined on the basis of experience, calculation or preliminary tests.

8.2 Application of load

The loading procedure shown in figure 1 should generally be followed.

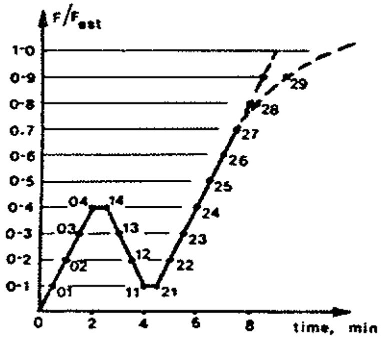


FIGURE 1 - Loading procedure

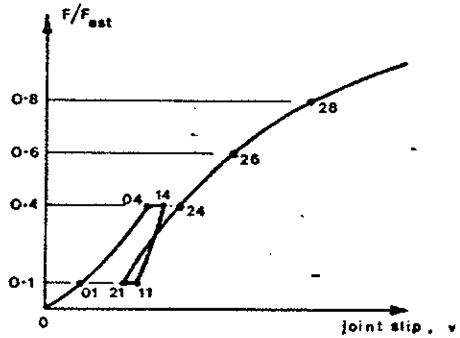


FIGURE 2 - Idealised load-deformation curve and measurements

For each test the load shall be increased and decreased at a rate corresponding to 0.2 F_{est} per min $\pm$ 25 per cent. The load shall be applied up to 0.4 F_{est} and maintained for 30 s. The load shall then be reduced to 0.1 F_{est} and maintained for 30 s. Thereafter the load shall be increased

8 LOADING PROCEDURE

8.1 Estimation of maximum load

The estimated maximum load F_{est} for the joint to be tested, shall be determined on the basis of experience, calculation or preliminary tests.

8.2 Application of the load

The loading procedure shown in Figure 1 shall be followed.

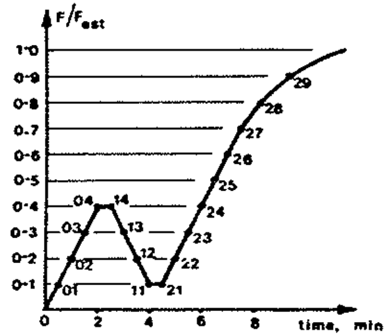


FIGURE 1 - Loading procedure

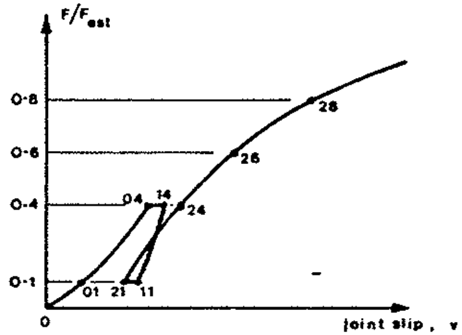


FIGURE 2 - Idealised load-deformation curve and measurements

The load shall be applied at a constant rate of 0.2 F_{est} per minute up to 0.4 F_{est}, which is to be maintained for 30 seconds. The load shall then be reduced at the same rate to 0.1 F_{est} which is also to be maintained for 30 seconds. The load shall be increased again at 0.2 F_{est} per minute to 0.7 F_{est}. Above 0.7 F_{est} a constant rate of deformation shall be maintained until the maximum load is reached in 3-5 minutes additional testing time (total testing time 10-15 minutes). If large deformations occur a gradual acceleration in loading rate is allowed to achieve the total test time.

The test may be stopped when the slip exceeds 15 mm even if the maximum load has not been reached.

until the maximum load or slip of 15 mm is reached. Above 0.7 F_{est} the rate of loading may be adjusted so that the maximum load or maximum slip is reached in 3-5 min additional testing time (total testing time about 10-15 min).

The test may be stopped when the maximum load is reached, or when the slip is 15 mm. For particular tests the preload cycle up to 0.4 F_{est} may be omitted with a corresponding adjustment to the total testing time*.

8.3 Measurement of slip

The slip measurements v_{01}, v_{04}, v_{14}, v_{11}, v_{21}, v_{24}, v_{26} and v_{28} shown in figure 2 shall be recorded for each test specimen. The slip at maximum load, F_{max}, shall also be recorded. When a load/slip diagram is not available measurements of slip should be taken at each 0.1 F_{est} increment of load (see figure 1).

8.4 Measurement of load

The maximum load reached before or at a slip of 15 mm shall be recorded as the maximum load, F_{max}, for each test specimen.

*The requirement that the load be maintained constant for 30 s at 0.4 and 0.1 F_{est} is to permit adequate time for the loading to be reversed, it is not intended to provide information on creep behaviour.

8.~~4~~3 Measurement of deformation

At each load-increment or decrement, the deformation shall be measured in such a way that the continuity of the loading procedure is maintained (see clause 7).

8.4 Maximum load and slip measurements

For each test the maximum load F_u and the slip measurements v_{04}, v_{14}, v_{24}, v_{26} and v_{28} shall be recorded (see Figure 2).

Comments received to 8.1 - 8.4:

Australia

We do not consider that the method of test, using a constant rate of load increase and load holding, would be very practicable for small research laboratories with simple types of testing machines. We consider that it would be more practical to use a constant rate of slip increase or cross-head movement such as to ensure reaching 80% of ultimate load within 3 to 6 minutes.

Even if the value of the proposed cyclic loading can be demonstrated, we cannot see the value of holding the load for 30 sec. at the designated values. Our experience has shown that at 40% of the ultimate strength of a nailed joint, the creep can be quite considerable, and it is difficult to see how a measure of the creep taking place in 30 sec. can be put to practical use in predicting long-term deformations at practical load levels which are typically around 25% of ultimate. Our more immediate concern is with the effect of long-duration loading on the ultimate joint strength. In the absence of information gained from properly designed long-duration tests, we consider the shape of the total load-deformation curve in short-duration loading to be very significant in indicating the potential "brittle" performance of the joint. In our approach to joint design, we consider a "brittle" performance to be potentially unsatisfactory whereas a joint showing reasonable "ductility" we would expect to give a reasonably satisfactory performance under long-duration loading.

Canada

The sources from which information contained in these paragraphs has been obtained should be provided to all members of TC 165. Is there any experimental evidence that this procedure is generally relevant to all types of mechanical fasteners?

The statement that "the test may be stopped when the slip exceeds 15 mm even if the maximum load has not been reached", is not consistant with a Limit State Design philisophy.

continued on p. 11

Continued from page 10

Comments received on 8.1 - 8.4:

France

7ème ligne, entre parenthèse lire : "(la durée totale de l'essai est en général de 10 à 15 min).

Le Comité Membre français pose la question de savoir si la valeur de 15 mn indiquée est bien celle qu'il faut lire ou s'il sagit d'une erreur dactylographique et que l'on devrait lire 1,5 mm.

A notre avis il semble que la valeur doit être de 1,5 mm et non de 15 mm.

The Netherlands

The range of 3-5 min. for the last period of the test cannot lead to a total range of 5 mm, in the duration of the test (according to the procedure 9.5 to 11.5 min.).

The last sentence on page 9 should be:
"The test may be stopped when the slip exceeds 15 mm even if the failure load has not been reached. The max.load reached before or at a slip of 15 mm will be considered to be the maximum load F_{max} of the test specimen".

United Kingdom

We still feel that this is too complicated and often inappropriate. If it is eventually accepted by the majority then we would at least like to see a revision as in the original CIB version of October 1975 (5th draft). For more simple cases, for example for the determination of the tension and shear strength of net-sections of fasteners, the load could be raised continuously from start to failure (see A6.3.2.2 and A6.3.4 of Annex A).

The requirement that the load shall be applied at a constant rate at least up to 0.7 F_{est} will be difficult to satisfy and may be impossible with some testing machines. Generally it is easier to control overhead movement rather than rate of loading so that it might be better to specify "at an approximately constant rate". Considering the uncertainties involved in the estimation of 0.2 F_{est} it seems unrealistic to restrict the type of testing machine used to one where rate of load application can be controlled.

8.5 Calculations

From the recorded measurements the following values shall be determined for each test:

1	Maximum load	F_{max}
2	Estimated maximum load	F_{est}
3	Initial slip	$v_i = v_{04}$
4	Modified initial slip	$v_{i,mod} = \frac{4}{3}(v_{04} - v_{01})$
5	Joint settlement*	$v_s = v_i - v_{i,mod}$
6	Elastic slip	$v_e = \frac{2}{3}(v_{14} + v_{24} - v_{11} - v_{21})$
7	Initial slip modulus	$k_i = 0.4\ F_{est}/v_i$
8	Slip modulus	$k_s = 0.4\ F_{est}/v_{i,mod}$
9	Slip at 0.6 F_{max}	$v_{0.6}$
10	Modified slip at 0.6 F_{max}	$v_{0.6\ mod} = v_{0.6} - v_{24} + v_{i,mod}$
11	Slip at 0.8 F_{max}	$v_{0.8}$
12	Modified slip at 0.8 F_{max}	$v_{0.8\ mod} = v_{0.8} - v_{24} + v_{i,mod}$

Note - The values calculated for 9 to 12 above relate to the actual value of F_{max} for each of the tests. When a continous load/slip diagram is available these values may be obtained directly at the required load level. When only readings of slip at increments of F_{est} are available then the values must be obtained by interpolation.

*It should be noted that many load/slip curves are initially convex upwards so that v_s will be negative.

8.5 Calculations

From the recorded measurements the following values shall be calculated and tabulated:

maximum load	F_u
initial displacement	$v_{int} = v_{04}$
reduced initial displacement	$v_{int,red} = \frac{4}{3}(v_{04} - v_{01})$
joint settlement*	$v_s = v_{int} - v_{int,red}$
elastic displacement	$v_e = \frac{2}{3}(v_{14} + v_{24} - v_{11} - v_{12})$
slip modulus	$k = 0.4\ F_{est}/v_{int,red}$
displacement at 60% load ($F/F_u = 0.6$)	v_{26}
reduced v_{60}	$v_{26,red} = v_{26} - v_{24} + v_{int,red}$
displacement at 80% load ($F/F_u = 0.8$)	v_{28}
reduced v_{80}	$v_{28,red} = v_{28} - v_{24} + v_{int,red}$

*Many load-slip curves may be initially convex upwards so that $v_{int,red}$ will be greater than v_{int} and v_s will be negative.

Comments received to 8.5:

Australia

As the proposed estimations of elastic displacement and modulus (joint stiffness) are to be calculated for one load level only, these parameters do not describe adequately for practical purposes the load-slip characteristics of a joint. We do not subscribe to the use of the words "elastic" and "modulus". Tests have shown that even after many cycles of loading, the load-slip relation for a nailed joint never really shows a true elastic behaviour. The so-called modulus bears no relation to the shape of the intervening load-slip curve, and as it relates to one load only, does not give sufficient information for the general design of joints for stiffness.

The idea of basing the test on load levels which are functions of an estimate of the ultimate strength seems to us to give rise to problems of variability. Our tests have shown that the coefficient of variation of the mean ultimate strength is about 15% but can be as high as 30% and the coefficient of variation of the slip at 40% of ultimate load is on the average about 30%. This means that the designated load levels of 10% and 40% of ultimate for extracting data could lie on widely different parts of the load-slip curve of individual specimens and thus give rise to a very high variability in the estimated mean values of the so-called elastic deformation and modulus. As it has been shown that

continued on p. 13

continued from p. 12

Comments received to 8.5:

the static loading curve can fairly accurately be measured in the case of the curvilinear relationship for nailed and similar types of joints, we consider it to be more expedient, for the sake of reducing variability, to extract data, at various levels of slip.

Canada

Before we would be prepared to support this document we would need considerable supporting information on the formulae and calculation methods specified in 8.5.

The Netherlands

$v_{int.red}$ may be greater than v_{int} "revised" or "modified" or "corrected" initial displacement might be better?

The subscript "int" for initial should be "i" ("int" is used for "internal" according to ISO 3898).

United Kingdom

In the equation for elastic displacement (V_e) V_{12} should be replaced by V_{21}.

8.6 Adjustment

If during the execution of the tests the mean value of the maximum load of the tests already carried out deviates by more than 20 per cent from the estimated value, F_{est}, then F_{est} should be adjusted correspondingly for subsequent tests. The values of maximum load already determined may be accepted without adjustment as part of the final results. In this case the values of slip and slipmoduli determined in 3 to 8 of 7.5 should be adjusted to correspond to the adjusted value of F_{est}.

8.6 Adjustment of F_{est}

If during the execution of the tests the mean value of the maximum load of the tests already executed deviates by more than 20% from the estimated value, F_{est}, then F_{est} should be adjusted correspondingly for successive tests.

The already determined values of the maximum load may be accepted without adjustment as part of the final results.

The displacement values v_{04} etc, shall be recalculated for each piece whose maximum load deviates by more than 20% from the revised F_{est}.

In all cases v_{60} and v_{80} shall correspond to the actual maximum load, F_u.

8.7 Maximum load

The maximum load shall be taken as the maximum load obtained for a displacement of not greater than 15 mm in the joint.

Comments received:

France

Charge maximale, lire :

"La charge maximale retenue ... pour une rupture inférieure à 1,5 mm dans le joint".

The Netherlands

We will produce some comments to these adjustments and after these have been discussed in RILEM-3TT-CIB (in oktober 1977) we will send ISO/TC 165 a proposal.

9 TEST REPORT

The test report shall include the following data:

a) species, density and relevant strength properties of the timber;
b) quality, strength properties and surface finish of the materials of the fasteners (including anti-corrosive protection);
c) dimensions of the joints and the size and number of fasteners;
d) conditioning of timber and test specimens before and after manufacture, moisture content of the timber at manufacture and at test, gaps between members, fissures etc;
e) the loading procedure used (by reference to this Recommendation) and a statement of any deviations;
f) individual test results and any relevant information regarding adjustments, mean values and standard deviations, and descriptions of the modes of failure.

9 TEST REPORT

The test report shall include the following data:

a **Species, density and relevant strength properties of the timber.**
b **Quality, strength properties and surface finish of the material of the fasteners (including anti-corrosive protection).**
c **Dimensioned drawings of the joints and the size and number of fasteners.**
d **Conditioning of timber and test specimens before and after manufacture, moisture content at the time of testing, gaps between members, fissures etc.**
e **The load procedure used (by reference to this Standard) and a statement of any deviations.**
f **Individual test results, including modes of failure, together with mean values and standard deviations.**

Comments received:

Australia

No mention is made in the draft of the method of obtaining mean values. Our test results have shown that for most types of fasteners, the frequency distribution of test values can be far from normal. We use a logarithmic transformation of data for calculating means and other statistics.

The Netherlands

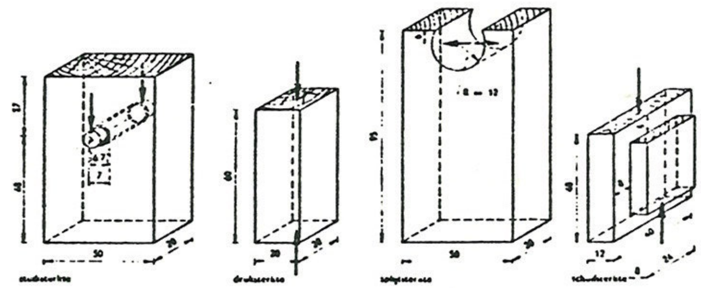
Which are the "relevant strength properties" of the timber and how are they determined. A suggestion has been added (see page 3).
It should be stated where the test pieces were fabricated in the laboratory or in a manifacturing plant, etc.

continued on p. 16

Continued from p. 15

Comments received:

Suggestion for the determination of relevant strength properties of the timber.



CIB-W18/11-10-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

TAPERED TIMBER BEAMS

by

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VIENNA, AUSTRIA
MARCH 1979

Summary.

There is proposed a method to calculate the stress distribution in tapered wood beams. The wood is assumed to be linear-elastic ortotropic.

The Norris interaction formula has been emploied to compare measured and predicted bending strengths of glulam and solid clear timber beams.

There was found a good agreement between measured and predicted bending strength of glulam beams.

Keywords.

Tapered beams, glulam, stress analysis, variable beam height, ortotropic beam.

Introduction.

In some cases it is appropriate that the height of a timber beam varies. In most cases one of the edges are parallel to the fibre direction. Because of this and because of that there only exist test results for this kind of beams this paper is limited to timber beams like those in figure 1. Meanwhile, there is no theoretical difficulties to describe beams where both the top and bottom edges incline with different angles to the fibre direction.

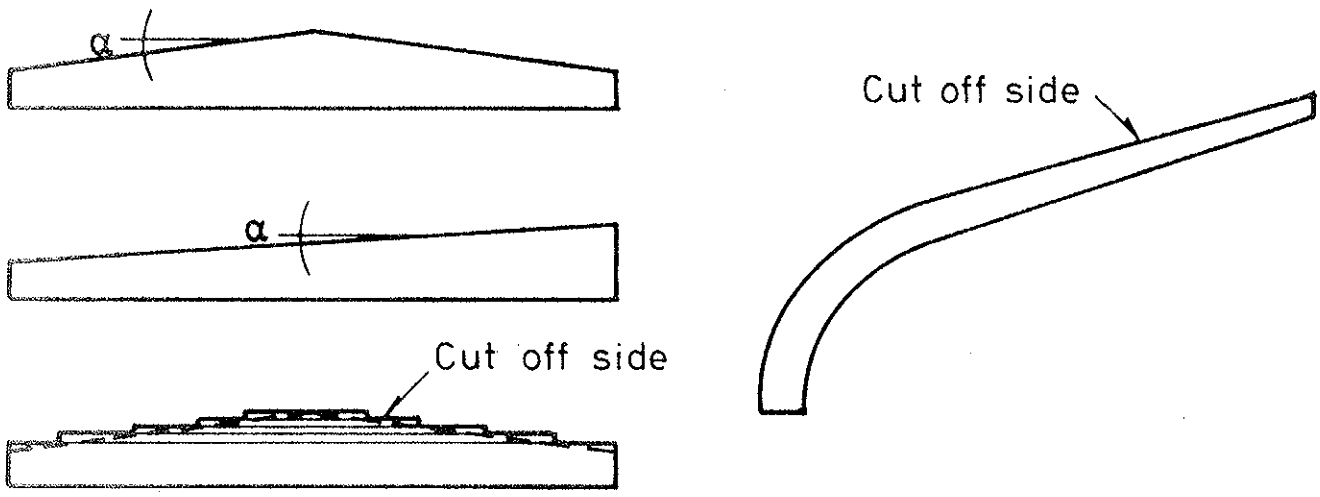


Figure 1. Timber beams with varying height.

This paper concentrates on the stress and strength analysis of that part of straight tapered beams, where there is a continuous slope of the edges.

Stress analysis

The beams are assumed to consist of an ortotropic linear elastic material with one of the principal axis parallel to the fibre direction. Among other things linear elasticity is assumed because of that strain gauge measurements on glulam beams reported in [Möhler and Hemmer, 1978] indicated linearity up to rupture, when the normal stress perpendicular to the fibre direction is a tension stress.

As shown on figure 2 there are several critical cross sections in a tapered beam, but this paper deals only with stress and strength analysis of cross section B , which lies not too close to either the end of the beam or the apex.

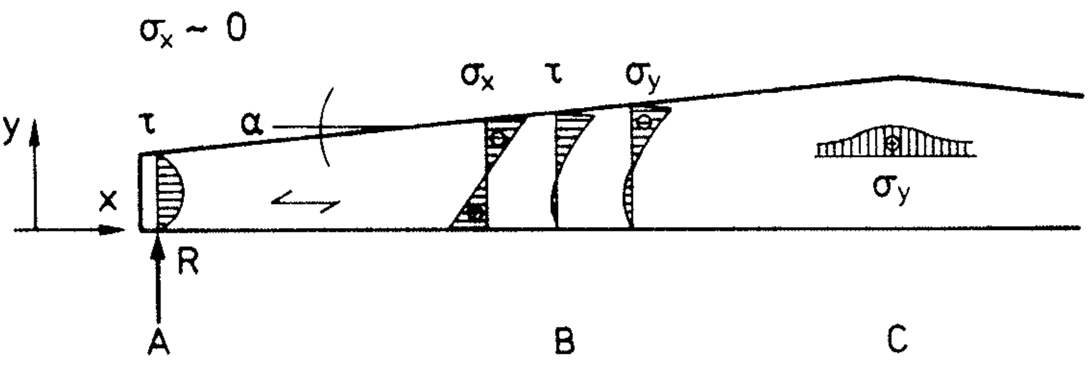


Figure 2. Stress distributions in a tapered beam at the critical cross sections.

It is possible to find an approximate analytical solution of the stress distribution at cross section B , when the moment is dominating for the stresses.

The stress distribution in the beam at cross section B in figure 3 is approximated with the stress distribution in the wedge, which is shaped by an extension of the beam. The force P at the apex of the wedge is determined so that the moments at cross section B are the same both in the beam and in the wedge.

$$M_{beam} = P \cdot (h_o/tg\alpha + x) \qquad (1)$$

The stress distribution in the wedge is found as described in [Lekhnitskii, 1968, chap. 20]. The calculations are given in appendix 1.

As shown in figure 3 the maximum stresses occur at top and bottom of a cross section. Since σ_r is a principal stress directed toward the apex of the wedge and the other principal stress σ_t is zero, the stress in the directions of the principal material axes can be written as

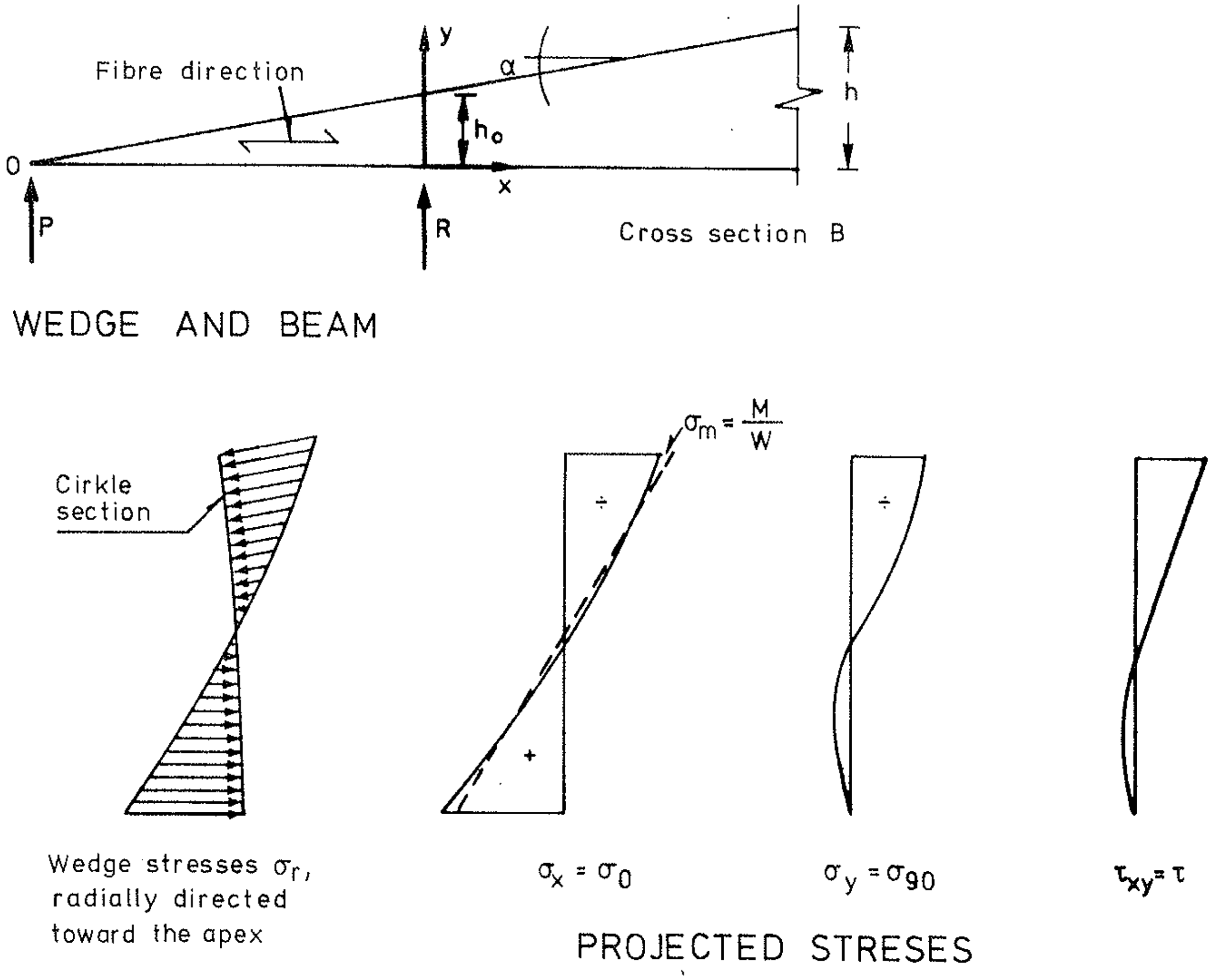


Figure 3. Stress distribution at cross section B found by means of the wedge solution.

$$\sigma_o = \sigma_x = \sigma_r(1 + \cos 2\theta)/2 \tag{2}$$

$$\sigma_{90} = \sigma_y = \sigma_r(1 - \cos 2\theta)/2 \tag{3}$$

$$\tau = \tau_{xy} = -\sigma_r \sin 2\theta/2 \tag{4}$$

$$\sigma_r = \sigma_m \cdot F_\alpha = 6 \cdot M \cdot F_\alpha/bh^2 \tag{5}$$

F_α : Factor given in appendix 1 figure 1.1.

where θ is the angle between σ_r and the fibre direction. Maximum stresses occur for $\theta = 0$ and $\theta = \alpha$.

It must here be emphasized, that it has been assumed that the beam is modelled as an ortotropic continuum. This means that it may be expected that the theory gives a good description of beams with small inhomogeneities (gluelam), while it is more questionable to apply the theory to beams with greater inhomogeneities (lumber).

Rupture criterion.

Several rupture criteria for wood are proposed, and they are normally only valid for clear wood. Two criteria have been investigated, namely those proposed by Stüssi and Norris. The criteria were applied to the test results in [Möhler and Hemmer, 1978].

It was found typical for Stüssi's criterion that for small values of the slope α the theoretical strength does not decrease as much as the measured strength, specially when tension perpendicular to the grain occurs. By comparison it was found, that Norris's criterion gave better agreement between the theoretical strength and the measured strength.

The comparison was carried out on the basis of stresses at rupture. In [Möhler and Hemmer, 1978] the simple bending stress $\sigma_m = M/W$ at rupture is given. The stresses σ_o, σ_{90}, τ at rupture can be found from the equations (2) - (5). The rupture criteria for a cross section may thus be written as in the following equations (6), (8), and (9).

Side parallel to fibres:

$$\sigma_{90} = 0, \ \tau = 0$$

$$\sigma_0 = F_\alpha \cdot \sigma_m < \begin{matrix} f_{c,0} \\ f_{t,0} \end{matrix} \qquad (6)$$

Cut off side, compression, $\sigma_r \leq 0$:

$$\left(\frac{\sigma_o}{f_{c,0}}\right)^2 + \left(\frac{\sigma_{90}}{f_{c,90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leq 1 \qquad (7)$$

or

$$|\sigma_r| = F_\alpha \cdot |\sigma_m| \leq 2/\sqrt{\left(\frac{1+\cos 2\theta}{f_{c,o}}\right)^2 + \left(\frac{1-\cos 2\theta}{f_{c,90}}\right)^2 + \left(\frac{\sin 2\theta}{f_v}\right)^2} \quad (8)$$

Cut off side, tension, $\sigma_r \geq 0$:

$$\sigma_r = F_\alpha \cdot \sigma_m \leq 2/\sqrt{\left(\frac{1+\cos 2\theta}{f_{t,o}}\right)^2 + \left(\frac{1-\cos 2\theta}{f_{t,90}}\right)^2 + \left(\frac{\sin 2\theta}{f_v}\right)^2} \quad (9)$$

Before the comparison can be carried out, one have to determine the principal strengths, $f_{c,o}$, $f_{t,o}$, $f_{t,90}$, $f_{c,90}$, f_v , which are assumed to be material values independent upon the combination of stresses.

For glulam the bending strength f_m and the compression strength $f_{c,o}$ were found by tests to 44 MPa and 44.7 MPa respectively. Since they are approximately equal $f_{c,o}$ and $f_{t,o}$ are put to f_m = 44 MPa irrespectively of the sign of σ_o . The three other strength parameters have been estimated, and relevant combinations of $f_{t,90}$, $f_{c,90}$, and f_v were investigated, see appendix 2, and the result of the best fit is given in figure 4. From appendix 2 it is further seen, that the mean value of the maximum stress at the side parallel to the fibres in all tests was smaller than the mean value of the bending strength f_m = 44 MPa

For solid timber beams following basic strengths were found by tests with small specimens: [Unit MPa]

$f_m = 82.5$ $f_{c,o} = 44.9$ $f_{t,o} = 70.8$

$f_{c,90} = 3.1$ $f_{t,90} = 2.7$ $f_v = 10.3$

Since the bending strengths both in the small clear specimens and in the solid timber beams with angles of slope less than 10^o are greater than the compression strength, there must occur a physical non-linearity in the wood. It is therefore questionable to use the stress distribution based on linearity, which yet has been done in this investigation.

Further the basic strengths determined from small clear specimens are not directly applicable to the solid tapered beams. There must occur a size effect and in some of the test specimens a notch-effect exists.

Because of the above mentioned problems several relevant combinations of the strengths parameters have been emploied in a comparison between experimentally measured and theoretically calculated strengths, see appendix 3. It appears from the table that in order to obtain a reasonable agreement between measured and calculated strengths one has to employ strengths parameters, which deviate from those measured on small specimens. In figure 4 an impression of goodness of fit is given.

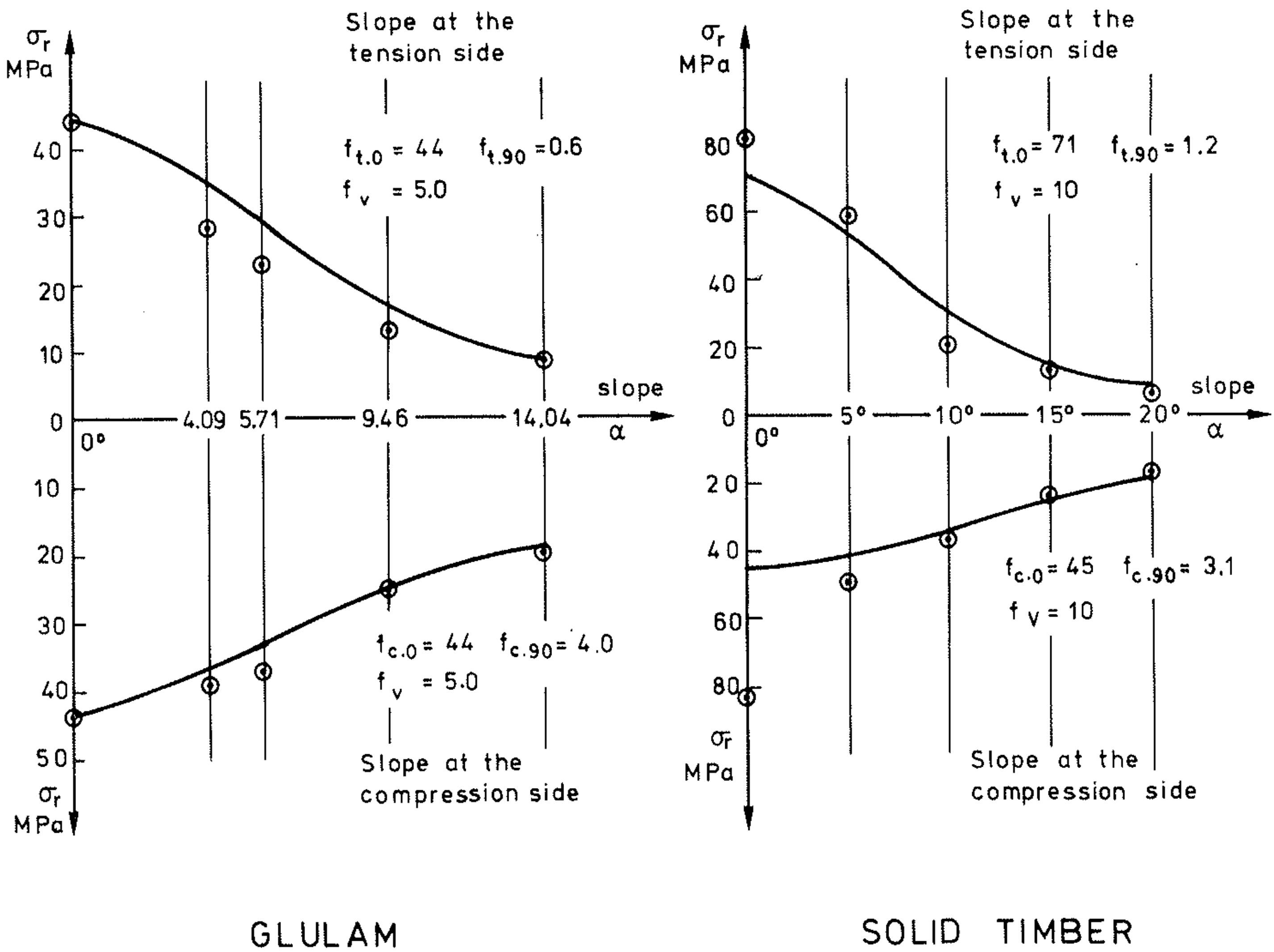


Figure 4. Measured and calculated strength of tapered beams. The strenght comparison is carried out on the basis of the radial stress σ_r.

Conclusions.

It has been demonstrated that the load capacity of tapered glulam beams can be predicted by a linear-elastic ortotropic stress analysis combined with a rupture critorion based on the maximum stresses at a point. It was found, that the Norris interaction formula gave the best prediction of the strength of glulam.

Apparently it is not possible to predict the strength of tapered solid beams on the same basis.

The test results with tapered glulam beams indicate that the Norris formula is not satisfactorily, since the basic strength parameters apparently are mutual dependent. It seems that "the shear strength is greater, when the stress perpendicular to the fibres is a compression stress than when it is a tension stress". This effect might be explained by assuming that rupture is trigered by cracks or flaws in the wood.

Literature.

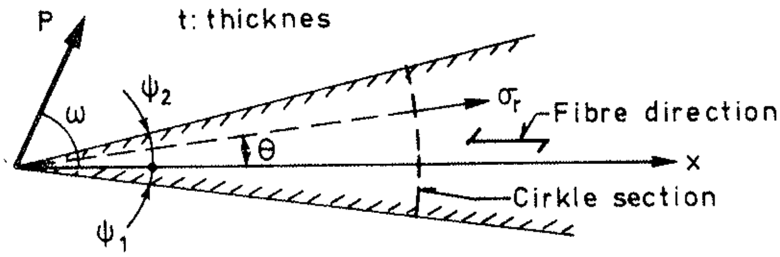
Möhler K. and Hemmer K., 1978. Paper presented at IUFRO Wood Engineering Meeting, August, 1978.

Lekhnitskii, S.G., 1968. Anisotropic plates. Gorden and Break Science Publishers.

Appendix 1.

STRESS DISTRIBUTION IN AN ORTOTROPIC WEDGE.

Basis:



Restrictions according to the paper:

$\psi_1 = \alpha =$ pitch

$\psi_1 = 0$

Material: Linear elastic, ortotropic

Constitutive equation:

$$\begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} = \begin{bmatrix} 1/E_1 & -\nu_{21}/E_2 & 0 \\ -\nu_{12}/E_1 & 1/E_2 & 0 \\ 0 & 0 & G \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \gamma_{xy} \end{bmatrix} \tag{1.1}$$

abbreviation: $\nu_{21}/E_2 = \nu_{12}/E_1 = \nu_1/E_1$

Results:

$$\sigma_\theta = 0$$

$$\tau_{r\theta} = 0$$

$$\sigma_r = \frac{1}{r}\frac{A\cos\theta + B\sin\theta}{L(\theta)} \tag{1.2}$$

where $L(\theta) = 1/E_\theta = \dfrac{\cos^4\theta}{E_1} + \left(\dfrac{1}{G} - \dfrac{2\nu_1}{E_1}\right)\sin^2\theta\cos^2\theta + \dfrac{\sin^4\theta}{E_2}$ (1.3)

and the terms A and B are defined by

$$A \cdot I_1 + B \cdot I_2 = -\frac{P}{t}\cos\omega \tag{1.4}$$

$$A \cdot I_2 + B \cdot I_3 = -\frac{P}{t}\sin\omega \tag{1.5}$$

$$I_1 = \frac{E_1}{\beta^2 - \delta^2}[\beta \operatorname{arctg}(\beta \operatorname{tg}\alpha) - \delta \operatorname{arctg}(\delta \operatorname{tg}\alpha)] \tag{1.6}$$

$$I_2 = \frac{E_1}{2(\beta^2 - \delta^2)} \ln[\frac{\cos^2\alpha + \beta^2\sin^2\alpha}{\cos^2\alpha + \delta^2\sin^2\alpha}] \tag{1.7}$$

$$I_3 = \frac{E_1}{\beta^2 - \delta^2} [- \frac{1}{\beta} \operatorname{arctg}(\beta \operatorname{tg}\alpha) + \frac{1}{\delta} \operatorname{arctg}(\delta \operatorname{tg}\alpha)] \tag{1.8}$$

where β and δ are defined by the roots of the characteristic equation. In this case where the complex parameters are purely imaginary and unequal one has

$$\mu_i = \beta \cdot i \qquad \mu_2 = \delta \cdot i \tag{1.9}$$

$$\mu^4 + (E_1/G - 2\nu_1)\mu^2 + E_1/E_2 = 0 \tag{1.10}$$

For conniferous wood in which the radial and tangential directions are not known (glulam) it is reasonable to employ

$$E_1/G - 2\nu_1 = 17$$

$$E_1/E_2 = 10$$

The sensitivity of the solution of this estimate is investigated at the end of this appendix.

The estimate gives

$$\beta = 1.1277, \qquad \delta = 3.9659 \tag{1.11}$$

This result must be inserted in the equations (1.4) - (1.8). Eq. (1.2) results in

$$\sigma_r = \frac{1}{r} \frac{A(\alpha)\cos\theta + B(\alpha)\sin\theta}{L(\theta)} \tag{1.12}$$

The maximum values of σ_r appear at the top and the bottom of the cross section, $\theta = \alpha$ or $\theta = 0$.

Equation (1.12) can be rewritten by means of the following rewriting

Moment in beam $= M = P \cdot (h_o/tg\ + x) \approx P \cdot r$

$A = A'(\alpha) \cdot P/tE_1$

$B = B'(\alpha) \cdot P/tE_1$

$\sigma_m = M/W = 6M/th^2$

$$\sigma_r = \frac{P}{t \cdot r} \frac{A'(\alpha)\cos\theta + B'(\alpha)\sin\theta}{\cos^4\theta + (E_1/G - 2\nu_1)\cos^2\theta\sin^2\theta + E_1/E_2\sin^4\theta}$$

$$= \frac{M}{t \cdot r(h_o/tg\alpha + x)} \cdot \overline{\qquad\qquad}$$

$$\approx \frac{M}{th^2} \cdot tg^2\alpha \cdot \frac{A'(\alpha)\cos\theta + B'(\alpha)\sin\theta}{\cos^4\theta + (E_1/G - 2\nu_1)\cos^2\theta\sin^2\theta + E_1/E_2\sin^4\theta}$$

$$= \sigma_m \cdot F_\alpha$$

where

$$F_\alpha = \sigma_r/\sigma_m = \frac{1}{6}\, tg^2\alpha \frac{A'(\alpha)\cos\ + B'(\alpha)\sin\theta}{\cos^4\theta + (E_1/G - 2\nu_1)\cos^2\theta\sin^2\theta + E_1/E_2\sin^4\theta}$$

Figure 1.1 shows the factor F_α for relevant slopes α.

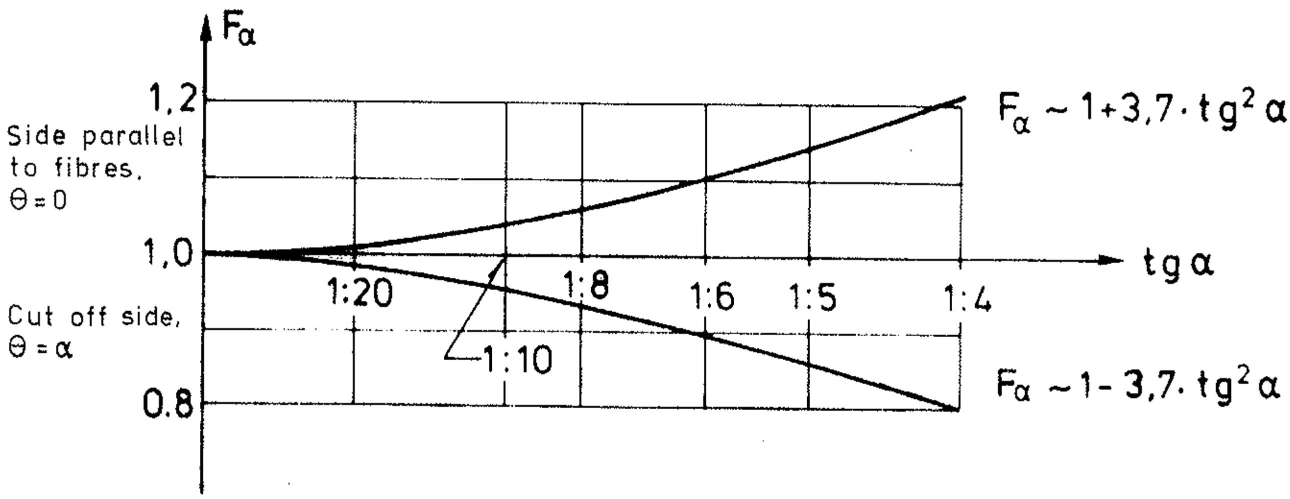


Figure 1.1. Factor $F_\alpha = \sigma_r/\sigma_m$

It has briefly been investigated how sensitive the stress distribution is to the stiffness parameters. As it can be seen from table 1.1, the changes in F_α are small for a slope of 1:10.

TABLE 1.1. F_α for $tg\alpha = 0.10$ and different relevant ratios between stiffness parameters.

$E_1/G - 2\nu_1$ E_1/E_2	17 20	14 20	17 30
$F_{\alpha,\theta=\alpha} = \sigma_{r,\theta=\alpha}/\sigma_m$	0,95	0,96	0,95
$F_{\alpha,\theta=0} = \sigma_{r,\theta=0}/\sigma_m$	1,04	1,06	1,03

Appendix 2. GLULAM BEAMS.

Measured and predicted rupture values of σ_r at the cut off side (inclined face).

				Compression side				Tension side						
Slope, α				4,09	5,71	9,46	14,04	4,09	5,71	9,46	14,04			
Strength parameters MPa				$\sigma_m \cdot F_\alpha = \sigma_r$, measured MPa										
$f_{c,o}$=				39	37	25	20	28	23	13	8	Sum of deviations squared, $(MPa)^2$		
$f_{t,o}$	$f_{c,90}$	$f_{t,90}$	f_v	Predicted values of σ_r MPa								Comp.	Tens.	Total
44	4	0,6	4	34,7	29,8	21,4	15,6	33,3	26,8	15,5	8,6	103	49	152
			5	37,4	33,3	25,1	18,6	35,7	39,2	16,7	9,0	18	112	130
			6	39,1	35,7	28,1	21,3	37,2	30,9	17,6	9,3	13	168	181
			7	40,3	37,5	30,6	23,5	38,2	32,0	18,1	9,4	46	213	259
44	5	0,7	4	34,7	29,8	21,5	15,7	33,7	27,5	16,6	9,5	101	68	169
			5	37,4	33,3	25,2	18,9	36,1	30,2	18,2	10,1	18	149	167
			6	39,1	35,7	28,3	21,6	37,7	32,0	19,2	10,5	15	216	231
			7	40,3	37,5	30,8	24,1	38,7	33,2	19,9	10,7	52	273	325
44	6	0,8	4	34,7	29,9	21,6	15,8	33,9	28,0	17,5	10,4	98	86	184
			5	37,4	33,3	25,3	19,0	36,4	30,8	19,3	11,1	27	343	360
			6	39,1	35,8	28,4	21,9	38,0	32,8	20,6	11,6	17	267	284
			7	40,3	37,6	30,9	24,4	39,1	34,1	21,5	12,0	56	325	381

Measured rupture values of σ_r at the side parallel to the fibres.

	4,09	5,71	9,46	14,04	4,09	5,72	9,46	14,04	
$\sigma_m \cdot F_\alpha$ MPa	41	41	32	30	30	25	17	12	

Appendix 3. SOLID TIMBER BEAMS.

Measured and predicted rupture values of σ_r at the cut off side (inclined face).

					Compression side				Tension side						
Slope, α					5	10	15	20	5	10	15	20			
Strength parameters MPa					$\sigma_m \cdot F_\alpha = \sigma_r$, measured, MPa								Sum of deviations squared, $(MPa)^2$		
					49	37	24	17	59	21	13	6			
$f_{c,o}$	$f_{t,0}$	$f_{c,90}$	$f_{t,90}$	f_v	Predicted values of σ_r MPa								Comp.	Tens.	Total
82,5	82,5	3,1	2,7	10,3	67,2	44,3	29,1	20,0	66,9	43,1	27,4	18,4	420	912	1332
82,5	82,5	3,1	2,0	10	66,5	43,6	28,6	19,7	65,3	39,0	23,1	14,8	378	543	921
45		3,1		10	42,0	34,3	25,6	18,8					62		
71			2,0	10					59,2	37,6	22,8	14,7		447	
71			1,2	10					56,7	30,0	26,0	9,7		109	
71			1,2	8					53,2	28,0	15,3	9,4		99	

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TESTS ON LAMINATED BEAMS FROM HARDBOARD UNDER SHORT AND LONG-TERM LOAD

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TESTS ON LAMINATED BEAMS FROM HARDBOARD UNDER SHORT AND LONG-TERM LOAD

1. Introduction

The tests provide for determination of strenght and behaviour of laminated beams of hardboard panels vertically glued under long-term load in bending in view of applying them for load-bearing structures of roof coverings.

Test program has been as follows:

1. Tests on beams in testing machine,
2. Tests on beams under short-term load,
3. Tests on beams under long-term cyclic load.

2. Inicial data

The beams to be tested have been made from hardboard panels having standard characteristics as follows /in brackets checking values are given/:

tensile strenght	$R_r \geq 200$ - /330/ kG/cm^2 = 33 MPa
compressive strenght	$R_c \geq 300$ - /766/ kG/cm2 = 76,6MPa
bending strenght	$R_g \geq 350$ - /351/ kG/cm2 = 35,1MPa
modulus of elasticity in bending	E_g = 30000 /46150/kG/cm^2= 4615 MPa
moisture content	W = /6,2/%

Dimensions of beams cross-section are given in figure 1. The beams have been glued with fenolic- formaldehyde adhesive AG until required size has been obtained.

For long-term tests one span beam was taken on two supports under loading of two forces concentrated in the centre of beam's span, the arrangement illustrates figure 1.

For calculation of long-term load ,premissible stress has been applied considering:

permissible bending stresses $k_g = 50$ kG/cm^2 = 5,0 MPa

modulus of elasticity $E_g = 3000$ kG/cm^2 = 3000 MPa

3. Test results

3.1. Tests on beams using testing machine

For test purpose a static arrangement was taken according to fig.1, where a= 40 cm and l = 150 cm. The tests have been carried out in 5- tons Armsler machine until failure, measuring deflection "u" and unit strain in cross-section ε in centre of beam span under changing load "P". Deflection has been measured with dial gauge having an accuracy of 0,01mm, and unit strain with an extensometer provided with tubular cap ,DDR type, having a reference line of 10 cm and an accuracy of 0,001mm.

The test covered 3 beams designated as IV-1-3.

Test results have been as follows:

a/ unit deformation at beam's height - an example is given in fig.2,

b/ load-deflection curves - figure 3,/see annex 1/,

c/ failure loads P_n - 850 /IV1/,885 /IV2/,860 /IV3/ kG,
   mean value of P_n - 865 kG = 8,65 kN,
   at 10-12% moisture content;
   corresponding stresses are:
   σ_g = 281, 292, 284 kG/cm2 ; mean value 285 kG/cm^2 = 28,5 MPa.

d/ deflection under near failure load u_n
   for IV-1 - 34,71mm under P = 800 kG
   for IV-2 - 35,12mm under P= 800 kG
   for IV-3 - 35,85mm under P = 850 kG

e/ fracture of beams, corresponding to brittle failure, illustrated in figure 4./see annex 1/.

3.2. Tests on beams under short-term load

For testing a static arrangement was taken according to figure 1, where a= 40 cm, l=220 cm. Two series of beams have been prepared, 3 beams in each, designated as follows: serie 1 - IV 19-21, serie 2- IV 22-24.

Test procedure has been as follows:

- the beams have been loaded to a limit established for individual serie and measurements of deflection have been taken in centre of beam span,
- the beams have been kept under load for 24 hours and subsequently

deflections have been measured,

- the loads have been removed and corresponding deflections have been measured,
- the beams have been kept without load for 48 hours and deflections have been measured after 24 and 48 hours.

During the test deflections of some beams have been continously recorded for 24 hours.

The tests have been carried out with the equipment as shown in fig.6 using concrete weights or bricks with an accuracy of loading of 1kG. The loads have been as follows:

for serie I P_{II} = 130 kG = 1,3 kN with respective stresses of
$1,4 \cdot k_g = 1,4 \cdot 50 = 70$ kG/cm^2 = 7,0 MPa

for serie II P_{III}= 186 kG = 1,84 kN with respective stresses of
$2,0 \cdot k_g = 2,0 \cdot 50 = 100$ kG/cm2 = 10 MPa

Deflections have been measured with deflectometer having measuring range of 5 cm and calibration of 0,05mm.

Deflections are given in table 1. In figure 5 load-deflection curve is shown after 24 hours of loading/see annex 1/.

Table 1

Mean deflection of beams of serie I and II under short-term load

Item	Deflection /mm/ immediately after loading	24h after loading	immediately after unloading	2h after unloading	24h after unloading	48h after unloading
IV 19-21	10,3	13,6	3,1	0,9	0,6	0,2
IV 22-24	15,8	19,9	4,0	1,6	0,7	0,5

After the tests described were completed, strenght of beams has been determined in Armsler testing machine and following results have been obtained with an arrangement shown in figure 1 and according to clause 3.1:

for serie I P_n = 1700 kG, 1700 kG , 1250 kG; mean value 1550kG=15,5kN
σ_n = 561 , 561, 412 kG/cm2 ; mean value =511kG/cm^2= 51,1MPa

for serie II P_n = 1800, 1200, 1300 kG, mean value 1433kG/cm^2=14,33kN
σ_n = 594, 396, 429 kG/cm^2; mean value 473 kG/cm^2= 47,3MPa

/at 4-6% of moisture content/.

3.3. Tests on beams under long-term constant load

The tests have been carried out with the equipment as shown in figure 6 using static arrangement as in figure 1, where a= 40cm, l= 220 cm. Three series of beams have been tested, 3 beams in each serie, under three different loads as follows:

serie I designated as IV-10-12 under load of

P_I = 93 kG /σ_g = 50 kG/cm^2/ = 930 N /5,0 MPa/

serie II designated as IV-13-15 under load of

P_{II}=130 kG /σ_g = 70 kG/cm^2/ =1300 N /7,0 MPa/

serie III designated as IV-16-18 under load of

P_{III}=186 kG /σ_g =100 kG/cm2/ = 1860 N /10,0 MPa/.

The beams have been placed in a heated, unconditioned room, situated in a cellar. The test provided for loading of beams and measurement of deflection in centre of their span during the time when load has been applied and after loading. Deflection has been measured with a deflectometer having measuring range of 5 cm and an accuracy of 0,05 mm. The test results are given in table 2 and in figure 7 as mean values for series I and II. As far as serie III was concerned exclusively results corresponding to a beam IV-18 are given providing that the tests for other two beams have not been continued / being stopped after approximately 300 days/

At the beginning the tests have been planned to be carried out until total failure. For the reasons independent from the autor the tests lasted out for 1300 days only without inducing failure.

Table 2

Deflections /in mm/ of beams from series I,II,III under long-term constant load

Loading time /days/	Deflection of beams /mm/			
	IV 10-12	IV 13-15	IV 16-18	IV 18
after loading	9,47	12,17	15,63	12,00
1	13,27	16,90	22,70	16,40
9	21,07	24,77	32,53	25,50
20	26,37	33,37	45,93	42,90
40	29,37	39,27	51,93	50,60
59	31,87	41,87	58,23	55,80
80	32,47	43,67	60,03	57,80
104	33,77	43,97	61,83	58,90
151	34,87	45,37	64,13	61,60
202	39,37	47,27	65,73	62,30
298	41,73	50,73	70,03	68,10

to be continued on page 5

Table 2

continued from pag

Loading time /days/	Deflection of beams /mm/ IV 10-12	IV 13-15	IV 16-18	IV 18
364	44,80	55,70	70,76	73,40
412	45,10	58,10	-	77,40
512	45,10	59,50	-	78,60
619	50,00	63,80	-	81,50
734	51,40	62,20	-	85,60
810	52,27	62,50	-	86,00
916	53,60	59,80	-	87,20
1001	54,50	60,90	-	88,50
1105	58,60	61,80	-	89,30
1184	59,30	62,40	-	90,00
1283	59,80	63,00	-	90,40

3.4. Tests on beams under long-term cyclic load

The tests have been carried out with the equipment as shown in figure 1, assuming a= 40 cm , l= 220 cm. Three series of beams have been prepared , 3 beams in each, loaded as in 3.3.

The corresponding designations have been as follows:

serie I - IV-1-3 P_I = 93 kp /σ = 50 kp/cm^2/

serie II - IV-4-6 P_{II}=130 kp /σ = 70 kp/cm^2/

serie III - IV-7-9 P_{III}=186 kp /σ = 100 kp/cm^2/.

The beams have been placed in a heated, unconditioned room. Afterwards the beams have been submitted to a cyclic changes of load during which deflections have been measured.

One cycle included:

a/ loading of beams according to serie with P_{I-III},

b/ permanence under load of beams above mentioned for 3 months /90 days/,

c/ total unloading of beams,

d/ permanence of beams without load for 3 months / 90 days/.

At the beginning the tests have been planned to be carried out until total failure. For the reasons independent from autor only four uncompleted cycles have been carried out, during which the beams have not failed.

The mean values of test results are given in figure 8 and in table 3.

Table 3

Deflection / in mm/ of beams from series I-III
under long-term cyclic load.
As a zero point starting -point before 1-st cycle have been taken.

Cycle	Specification	Deflection in mm for serie I	II	III
I	After loading	12,4	11,7	16,9
	After 97 days of loading	32,1	33,0	41,1
	After unloading	19,7	23,3	23,6
	After 107 days from unload.	12,8	15,7	16,4
2	After loading	24,1	27,9	36,5
	After 107 days of loading	31,8	34,6	47,1
	After unloading	21,0	27,4	31,1
	After 107 days from unload.	20,8	21,4	21,0
3	After loading	24,9	30,6	37,6
	After 111 days of loading	31,6	43,7	55,4
	After unloading	31,6	34,9	38,7
	After 112 days from unload.	22,7	29,5	28,3
	After loading	32,7	38,2	43,7
	After 21 days of loading	38,4	42,1	50,6

After the test under cyclic load have been completed, the beams have been tested in bending in Armsler testing machine according to an arrangement as shown in figure 1 and to data given in 3.1. The following mean values have been obtained:

serie I P_n = 1277 kp = 12,77kN σ_g = 421 kp/cm^2 = 42,1 MPa

serie II P_n = 1315 kp = 13,15kN σ_g = 434 kp/cm^2 = 43,4 MPa

serie III P_n = 1806 kp = 18,06kN σ_g = 596 kp/cm2 = 59,6 MPa

/at moisture content of 4-6 %/.

4. Interpretation of test results

4.1. Interpretation of test results for beams in testing machine

Deflection data for beams in dependence of loads applied and failure load values have been obtained. Modulus of elasticity changes during the tests described have been also interesting, so corresponding value has been calculated as follows:

$$E = \frac{P \cdot c}{2 \cdot 24\, f\, I} / 3 \cdot l^2 - 4\, c^2 / \qquad /1/$$

where: $l = 150$ cm

$c = 55$ cm

$I = 416{,}6\ cm^2$

Other values have been taken as given in figure 1. The corresponding values of "f" and "P" / deflection and load respectively/ attained during the test have been taken into account.

Calculation results for beam IV-2 are given in table 4. These results suggest that "E" changes with load. The decrement of modulus of elasticity corresponding to failure is 30% in accordance with table 4.So time-deformation - curve is not straight-line as shown in figure 2.

Table 4

Decrease of E value relating to short-term load

Load kG	Stress kG/cm^2	E kG/cm^2	Decrement of E %
30	10	48,113	0
50	16	46,736	3
100	33	46,030	5
150	50	45,345	6
200	66	43,656	10
300	99	43,120	11
400	132	41,657	14
500	165	40,392	16
600	198	38,719	20
700	231	36,776	24
800	264	34,706	28

For design purposes only a small portion of strenght is considered and corresponding stress value is of 50 kG/cm^2 /5 MPa/. From table 4 one can deduce that respective stress increment is small /approximately 6%/, so E value can be considered as constant at determined level.

Figure 3 suggests that neutral axis of beam is not in line with a geometrical one. This problem shall be investigated separately, for we are convinced, that deformation in compression is different from deformation in tension.

4.2. Interpretation of test results for beams under short-[illegible]

There are some interesting aspects in this test. First o[illegible] the curve corresponding to deflection immediately after loading of great importance, for tests of assesement of elements shall be as short as possible. For this reason relative deflection in % has been calculated considering deflection increment ΔU_t to deflection increment after 24 hours ΔU_{24} ratio, for beams described above.

The results are shown in table 5.

Deflections taken as mean values for both series after 24 hours of loading are given in figure 9 /see annex 1/.

Table 5

Relative deflection increment after time t to deflection increment after 24 hours ratio /$\Delta U_t : \Delta U_{24}$/

Item	Deflection in % after time t					
	1 hour	2 hours	4 hours	6 hours	12 hours	24 hours
IV 19-21	12	24	39	46	74	100
IV 22-24	12	21	39	51	73	100

Interesting is also a relative deflection increment to initial deflection ratio /immediately after loading/ U_{to}, the initial deflection being considered as an elastic one.

The relating data are given in table 6 as /$\Delta U_t : U_{to}$ ratio/ in %.

Table 6

Relative deflection increments after time t to initial deflection ratio /$\Delta U_t : U_{to}$/ in %

Item	Deflection in % after time t						
	to	1 hour	2 hours	4 hours	6 hours	12 hours	24 hours
IV 19-21	100	103	107,9	112,8	115,2	124,0	132
IV 22-24	100	103	105,8	110,7	113,6	119,5	127

From the above tables one can deduce that deflection increment after 6 hours constitutes approximately 50% of deflection after the first 24 hours and about 15% of elastic deflection. After 24 hours deflection increment is about 30% of elastic deflection.

Another aspect constitutes a form of deflection curve a
loading. In accordance with table 1 initial deflection U_{to} is
vered after unloading, while the deflection U_t imposed under ac
of load is not recovered. Relative value of such deflection to U_o
ratio is given in table 7.

Table 7

Relative deflection value in %
to deflection after unloading ratio / $U_t : U_o$/

Item	Deflection in % after time t from unloading			
	0 hours	2 hours	24 hours	48 hours
IV 19-21	100	29	19	6,5
IV 22-24	100	40	18	12,5

Relative deflection value to initial deflection U_{to} after time t-U ratio has been also calculated as given in table 8.

Table 8

Relative deflection value in %
to initial deflection U_{to} after t time from unloading ratio

Item	Deflection in % after time t from unloading			
	0 hours	2 hours	24 hours	48 hours
IV 19-21	100	0,9	0,6	0,2
IV 22-24	100	1,0	0,4	0,3

From tables 6 and 7 one can deduce that 60-70% of deflection induced by loading after time t / not the initial elastic one/ is recovered after 2 hours and after 24 hours 80% of deflection is recovered. After 48 hours only 6-10% of deflection is not recovered. Besides it is obvious that , so called, permanent deflection, which is not recovered after 24 and 48 hours from unloading is very small in comparison with initial deflection and constitutes 0,2 - 0,5 % of its value.

It also can be seen from table 6, that deflection after time t depends upon initial deflection and time of action of load. So deflection after 6 hours constitutes 15% of initial one and after 24 hours - 30% of it.

Obviously deflection value depends upon load value and upon bending stresses in the material, but this relation is included in initial deflection /elastic one/. The change of deflection can be expressed also as a function of coeficient of elasticity "E_t". The corresponding calculation have been made according to formula /1/, assuming $l = 220$ cm, $c = 90$ cm, $I = 416{,}6\ cm^4$, $P_{II} = 130$ kG /1,4 kN/. The results are given in table 9.

Table 9

Changes of coeficient E_t related with loading time.

t	kG/cm^2		%	
	E_{II}	E_{III}	E_{II}	E_{III}
after loading	64000	59800	100	100
1 hour	61600	57800	96	97
2 hours	59600	56400	93	94
4 hours	56800	54000	89	90
6 hours	55200	52600	86	88
12 hours	47000	50000	73	84
24 hours	44000	47000	69	78

Data from table 9 suggest that after 24 hours of loading, decrement of coeficient of elasticity E_t is about 30% of its initial value.

4.3. Interpretation of test results for beams under long term-load

Considering test results one can deduce easily that deflection increases with time. Maximum deflection increment after one day loading was observed. This value has been taken as a characteristic one due to a fact, that can be obtained with easiness when testing units or beam.

Table 10 presents deflection increment after time t to deflection after one day of loading ratio $\Delta U_t : \Delta U_1$.

Table 10

Deflection increment after time t
to deflection increment after one day of loading ratio $\Delta U_t : \Delta U_1$

Table 10

Deflections ratio	Loading time	Comparative data in % for series			
		I/IV10-12/	II/IV13-15/	III/IV16-18	[illegible]
$U_{364}: U_1$	1 year /364 days/	9,3	9,2	13,0	13,9
$U_{734}: U_1$	2 years /734 days/	11,0	10,6	-	16,7
$U_{1065}: U_1$	3 years /1065 days/	12,0	10,5	-	17,5

Other characteristic value constitutes elastic deflection U_{to} immediately after loading. Deflection increment in time U_t after 1 year, 2 and 3 years to elastic deflection U_{to} ratio is also interesting. Corresponding results are given in table 11.

Table 11

Deflection increment after time t to elastic deflection ratio $\Delta U_t : \Delta U_{to}$

Deflections ratio	Loading time	Comparative data in % for series			
		I/IV10-12	II/IV13-15	III/IV16-18	III/IV-18
$U_1 : U_{to}$	1 day	0,401	0,388	0,364	0,366
$U_{364}: U_{to}$	1 year	3,727	3,574	3,525	5,116
$U_{734} : U_{to}$	2 years	4,424	4,116	-	6,133
$U_{1065}: U_{to}$	3 years	4,804	4,059	-	6,408

Data from tables 10 and 11 suggest that deflection increment after one year of loading is 9-14 times of deflection after 1 day of loading. The respective deflection is 3,5 times of elastic one. After 2 years deflection increment is 11 times of deflection after one day of loading , the corresponding deflection being 4,5 times of elastic one. After 3 years period deflection increment is 12 times of deflection after 1 day of loading and deflection is 5 times of elastic deflection.

To know what shall be successive deflections in time is also important. For this very reason a deflection velocity V_t has been calculated in relation with time from the formula:

$$V_t = \frac{\Delta U_t}{t} \qquad /2/$$

where:

U_t - deflection increment in time t, mm

t - loading time, days

V_t values are given in table 12 and in figure 10 / see annex 1/.

Table 12

Mean values of V_t for beams serie I-III /in mm/ 24h/

Loading time /days/	V_t values for serie			
	I/IV10-11	II/IV13-15	III/IV16-18	IV-18
Aftér loading	0	0	0	0
1	3,80	4,73	7,04	4,40
9	1,28	1,40	1,81	1,50
20	0,84	1,06	1,51	1,54
40	0,49	0,67	0,90	0,96
59	0,37	0,50	0,72	0,74
80	0,28	0,39	0,55	0,57
104	0,22	0,30	0,44	0,45
151	0,16	0,21	0,32	0,32
202	0,14	0,17	0,24	0,24
298	0,10	0,12	0,179	0,18
364	0,096	0,12	0,100	0,16
412	0,086	0,11	-	0,15
512	0,069	0,092	-	0,13
619	0,065	0,081	-	0,11
734	0,056	0,068	-	0,10
810	0,052	0,062	-	0,091
916	0,048	0,052	-	0,082
1001	0,045	0,048	-	0,076
1105	0,044	0,044	-	0,070
1184	0,042	0,042	-	0,066
1283	0,039	0,039	-	0,061

Dotted curves in figure 10 represent mean values. To draw ... a bilogarithmic net has been taken and owing this the curves ... nearly straight-lined. The curves V_t - t after approximation have been converted into straight lines and corresponding formul... has been developed. In a bilogarithmic net the described relation is as follows:

$$\lg V_t = \Psi \lg t + \lg c \qquad /3/$$

where:

V_t - deflection velocity of beams , mm/24 h

Ψ - directivity factor of straight line in bilogarithmic net

t - loading time, days

c - velocity v after one days of loading.

As the terms of formula are determined by the curves in figure 10, deflection of beam can be calculated after arbitrary time using following formula:

$$U_t = c \cdot t^{1+\Psi} + U_{to} \qquad /4/$$

where:

U_{to} - initial elastic deflection

On basis of diagram 10 the following values of Ψ and c have been calculated /table 13/:

Table 13

Ψ and c values

Serie	Factor	
	Ψ	c
I/IV 10-12/	- 0,750	7,6
II /IV 13-15/	- 0,748	9,5
III/IV 16-18/	- 0,755	14,5

In accordance with table 13 the slopes of curves V_t in bilogarithmic net are almost the same for each serie , it means are independent from load value. So inclination of curves depends exclusively upon material properties and probably upon its viscoelasticity. Deflection velocity is determined by term c, e.g. by deflection velocity after one day of loading, which depends upon load value, it means, upon stresses.

Stress value may be determined relatively easily during th...
However it is necessary to take into account that the mode of determination of deflection of structures after loading time t is based upon tests carried out for 100 days / what allows to draw a diagram in a bilogarithmic net/ is only an approximate one. It shall be necessary to test major number of specimens in different climates, especially as far as humidity is concerned.

The tests undertaken prove an extreme susceptibility of material and dependance of material behaviour on humidity and temperature of air / steps on deflection curves/.

Let us interpret again c values from table 13. For serie I respective stress value has been of 50 kG/cm2, for serie II - 70 kG/cm2 and for serie III - 100 kG/cm2, so coeficient of stress increment has been: for serie I - 1,0 , for serie II - 1,4 , for serie III - 2,0. Comparing c values one can obtain : $\frac{9,5}{7,6} = 1,25$ /instead of 1,4/, $\frac{14,5}{7,6} = 1,94$ /instead of 2,0 /.

From above calculations one can deduce that c value shall be propor-tional to stress value. The differences are due to imprecision of drawing aproximate straight lines in figure 10 owing to scatter of results /steps/ and too reduced number of tests.

In accordance with it, one can try to determine stresses imposed in beams assuming pre-fixed deflection values in time, e.g. design beams on basis of limit states method taking into account their serviceability.

Deflections calculated from formula /4/ in relation with their serviceability are as follows: /table 14/

Table 14

Deflection of beams in time /mm/ calculated from formula /4/

Stress value kG/cm2	U_{to} mm	Deflection in mm after years			
		5	10	25	50
50	9,5	59,1	68,5	83,5	96,6
70	12,2	74,2	86,0	105,0	121,1

The tests results from table 14 suggest that def[illegible]on induce by stresses of 50 kG/cm2 are considerable and it is [illegible]sible t[illegible] take them into account in structural design. Besides stress valu[illegible] given in Polish Standard PN-73/B - 03150 for hardboard as 100 kG/cm2 /10 MPa/ is excedingly high and it is necessary to correct it.

From data interpreted before there is impossible to deduce what kind of relation exists between deflection velocity and span of beam at the same stress value. This problem shall be investigated in future.

Besides it is interesting what will be a coeficient of elasticity E_t value according to formula /1/ after replacing φ by φ_t , e.g. by deflection changing in time. The respective calculations are given in table 15.

Table 15

Coeficient of elasticity E_t values calculated from formula /1/ as a function of loading time, 10^3 kG/cm2

Loading time /days/	E_t values for beams IV 10-12	IV 13-15	IV-18	E_t mean values kG/cm2 10MPa	%
0	49,8	54,2	78,7	60,4	100
1	35,6	39,0	76,1	50,2	82,4
9	22,4	26,6	37,0	28,7	47,1
20	17,9	19,8	22,0	19,9	32,7
40	16,1	16,8	18,7	17,2	28,2
59	14,8	15,7	16,9	15,8	25,9
80	14,5	15,1	16,3	15,3	25,1
104	13,9	15,0	16,0	15,0	24,6
202	11,9	13,9	15,2	14,0	23,0
298	11,3	13,0	13,8	12,7	20,8
364	10,5	11,8	12,9	11,7	19,2
412	10,5	11,4	12,2	11,4	18,7
619	9,4	10,3	11,6	10,4	17,1
810	9,0	10,5	11,0	10,2	16,7
1001	8,7	10,8	10,6	10,0	16,4
1283	7,9	10,5	10,4	9,6	15,8

From the results given in table 15 one can deduce that after three years E_t value is 16% of initial value E_o.

4.4. Interpretation of test results for beams under long-term cyclic load

The first aspect constitutes a change of deflection value, so called elastic deflection, under cyclic load.
The respective values have been calculated on basis of table 3 and are given in table 16.

Table 16

Changes of deflection of beams immediately after loading and under cyclic load

Cycle	Deflection in mm and % for series					
	I		II		III	
	mm	%	mm	%	mm	%
1	12,4	100	11,7	100	16,9	100
2	11,3	91	12,2	104	20,1	119
3	14,1	114	9,2	79	16,6	98
4	10,0	81	8,7	74	15,4	91

The results given in table 16 suggest that deflection of beams under load decreases in dependance on cycle of loading , e.g. every load applied to a beam induce a deflection smaller than an anterior one. Too reduced number of cycles is not sufficient to define rules for changes taking place.

It would be recommendable to investigate the relation between deflection and number and level of load. Another interesting aspect constitutes the increment of permanent deflection / not to be recovered/ corresponding to each cycle. Respective results are given in table 17.

Table 17

Permanent deflection increment in mm and % for separate cycles

Cycle	Permanent deflection increment in separate cycles /mm/ for beams of serie					
	I		II		III	
	mm	%	mm	%	mm	%
1	+15,7	100	12,8	100	16,4	100
2	- 2,0	12	+5,7	44	+4,6	28
3	-11,9	76	+8,1	63	+7,3	44

From table 17 one can deduce that permanent deflection value [illegible]creases with each subsequent loading. This value depends probably upon loading of beams in every cycle. Due to a fact that test results have been greatly influenced by changing climate /changes of temperature and humidity of air/ such relation is not to be determined on the basis of them.

Another aspect analysed has been a variation of deflection velocity within separate cycles of loading. In figure 11 /see annex 1/ curves illustrating deflection velocity are shown, calculated from formula /2/ for separate cycles,

Proceding according to 4.3. approximate straight lines have been drawn for cycles 2 and 4 as shown in figure 11, what allowed to calculate of coeficients Ψ and c. However not for every curve illustrated in figure 11 has been possible to determine the coeficients above mentioned considering difficulty of applying an approximation method because deflection of beams have been affected also by air humidity and temperature.

Ψ and c values so determined are given in table 18.

Table 18

Coeficients Ψ and c calculated from formulae /3/ and /4/ for separate cycles

Cycle	Beam	Coeficients values	
		Ψ	c
1	IV 1-9	-0,795	-
2	IV 1-3	-0,705	1,5
	IV 4-6	-0,705	1,9
	IV 7-9	-0,705	2,7

According with table 18 the slope of approximate straight line in bilogarithmic scale is similar to one described in 4.3. The slope seems to be related with number of cycles and characteristic value of material. c value for separate cycle depends upon load applied and subsequent cycle. c value increases with number of cycles. Precise c values may be determined after more complete tests during which load values, beam spans and lot size shall be recorded.

Thus on the basis of tests undertaken one can deduce that deflection velocity depends upon load value and number of cycles of loading.

5. Conclusions

1. The relation above mentioned suggest that the tests on o[illegible] under long-term load are important and useful , especially [illegible] far as new structures from wood- based products are concern[illegible]
2. In accordance with the results obtained, is necessary to consider the time factor while designing structural units. The design shall include the serviceability time of structure until failure will occur, assuming correct safety margin.
3. There is a necessity to develop design methods based on serviceability time of structure until failure will take place.
4. As the test results obtained suggest the bending strenght of laminated elements from hardboard is independent from service period and variations of stress /relatively slow/, but deflection and deflection velocity depend upon load, loading time and changes of load applied.
5. Further investigation deserves determination of climate effect on deflection and deflection velocity considering loading time and changes of load applied.
6. It is purposeful to correct stresses established by Polish Standard PN-73/B_ 03150 "Wooden structures. Static calculations and design".
7. In accordance with the tests performed, vertical lamination contributes to increased strenght and elasticity of beams.

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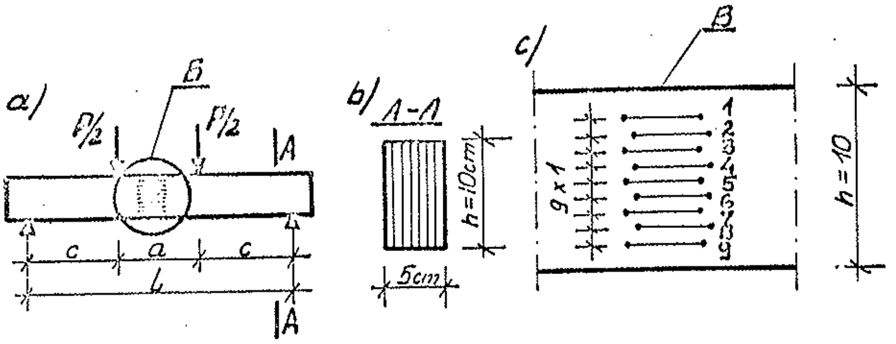

Fig. 1 Details of beams to be tested
a/ static arrangement
b/ cross - section
c/ measurement points ε.

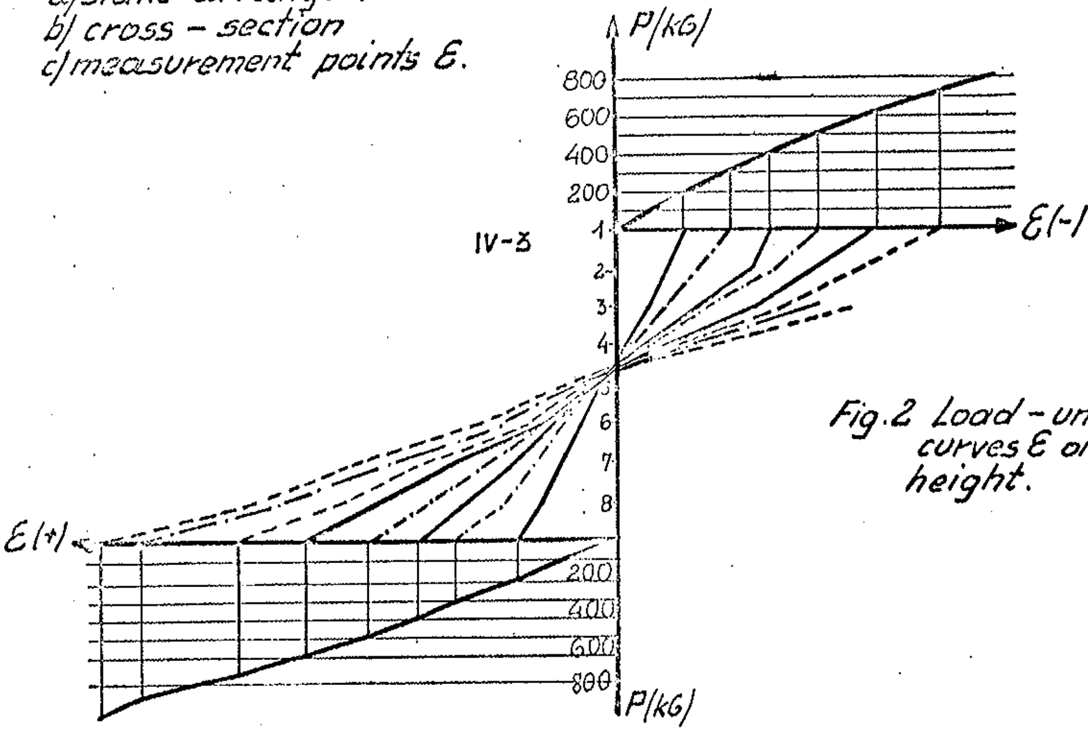

Fig. 2 Load - unit deformation curves ε on beam's height.

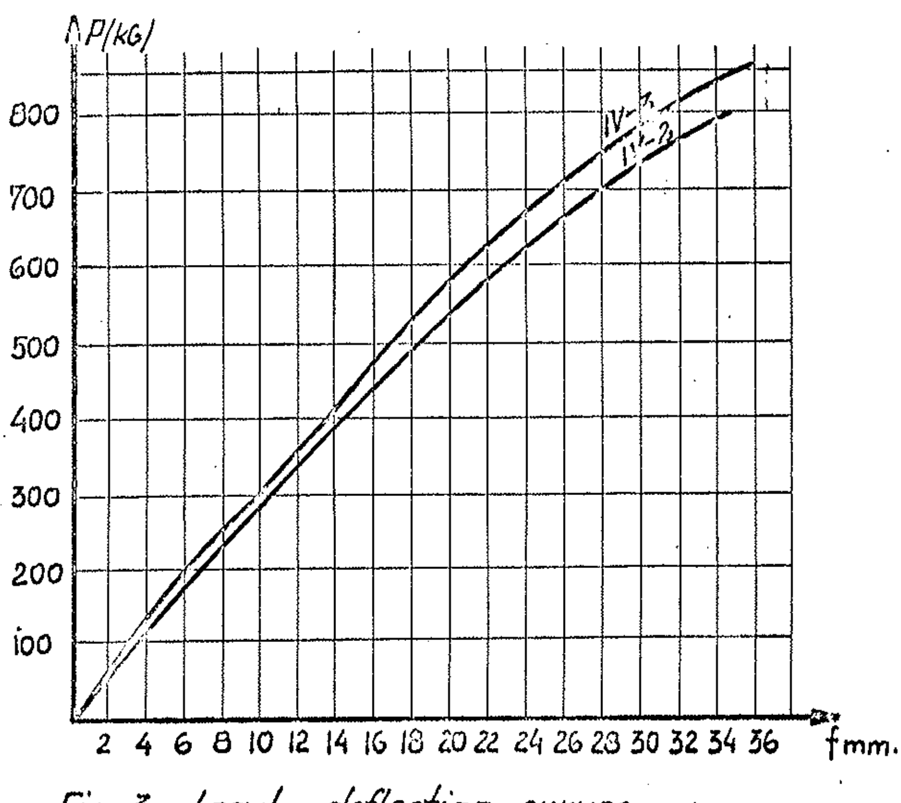

Fig. 3 Load - deflection curves.

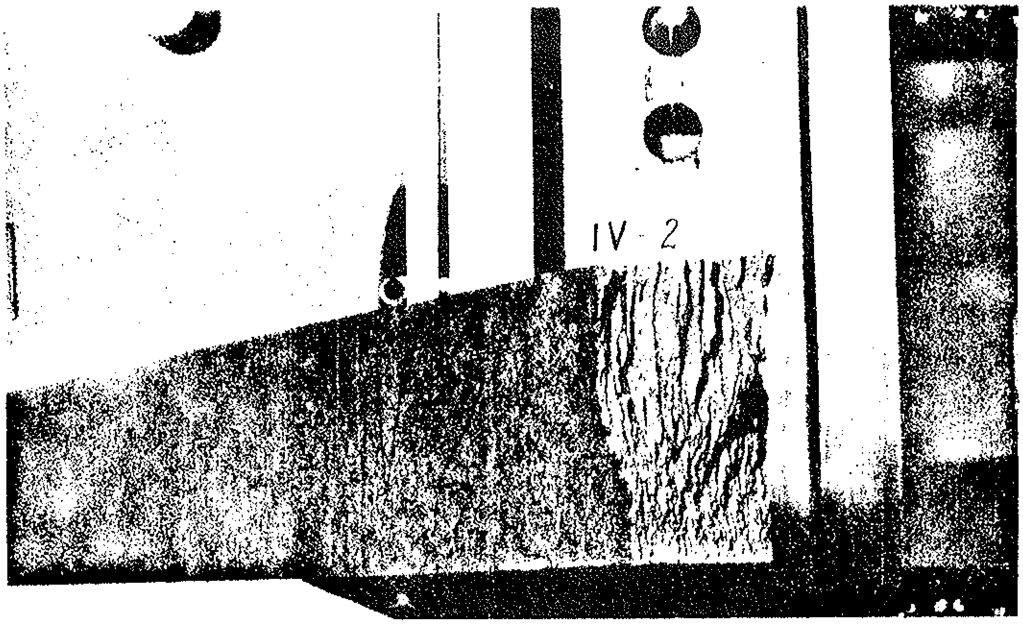

Fig. 4 Illustration of failure of beam IV-1

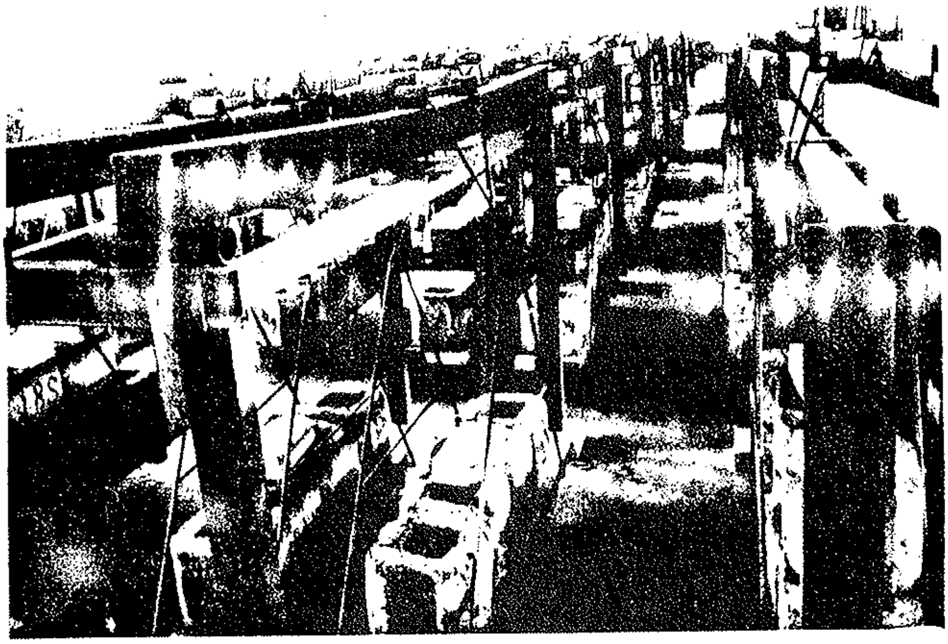

Fig. 6 Test on beams under long-term load.

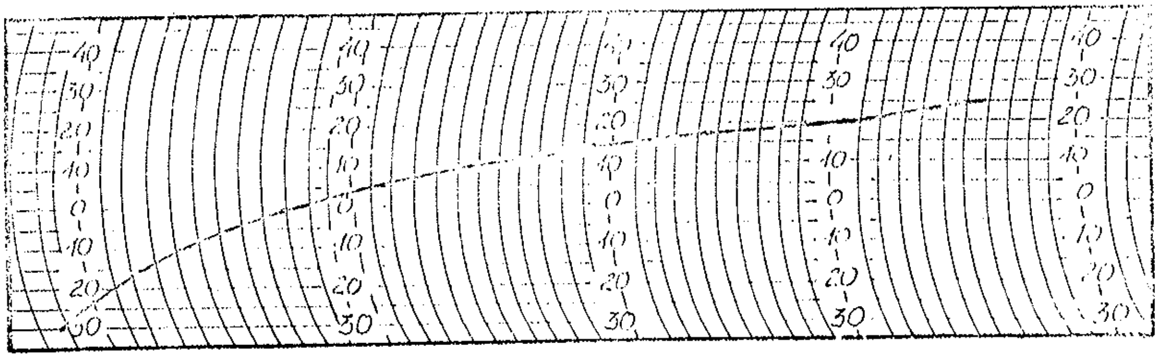

Fig. 5 Load - deflection curve for beam under long-term load after 24 hours of loading.

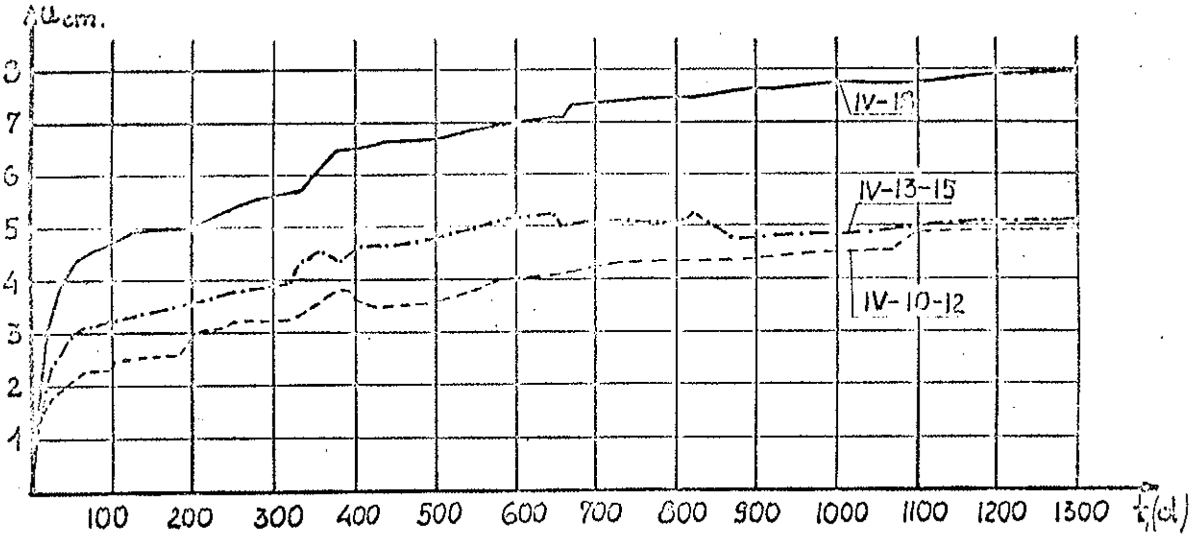

Fig. 7 Time - deflection curve (mean values).

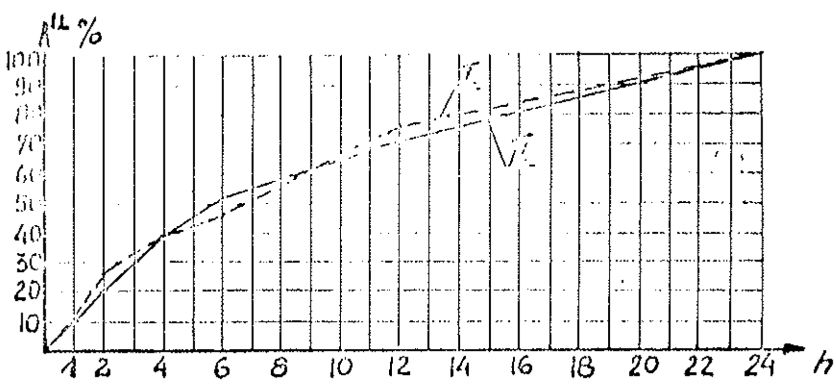

Fig. 9 Deflection curves during 24 hours of loading I - serie I, II - serie II.

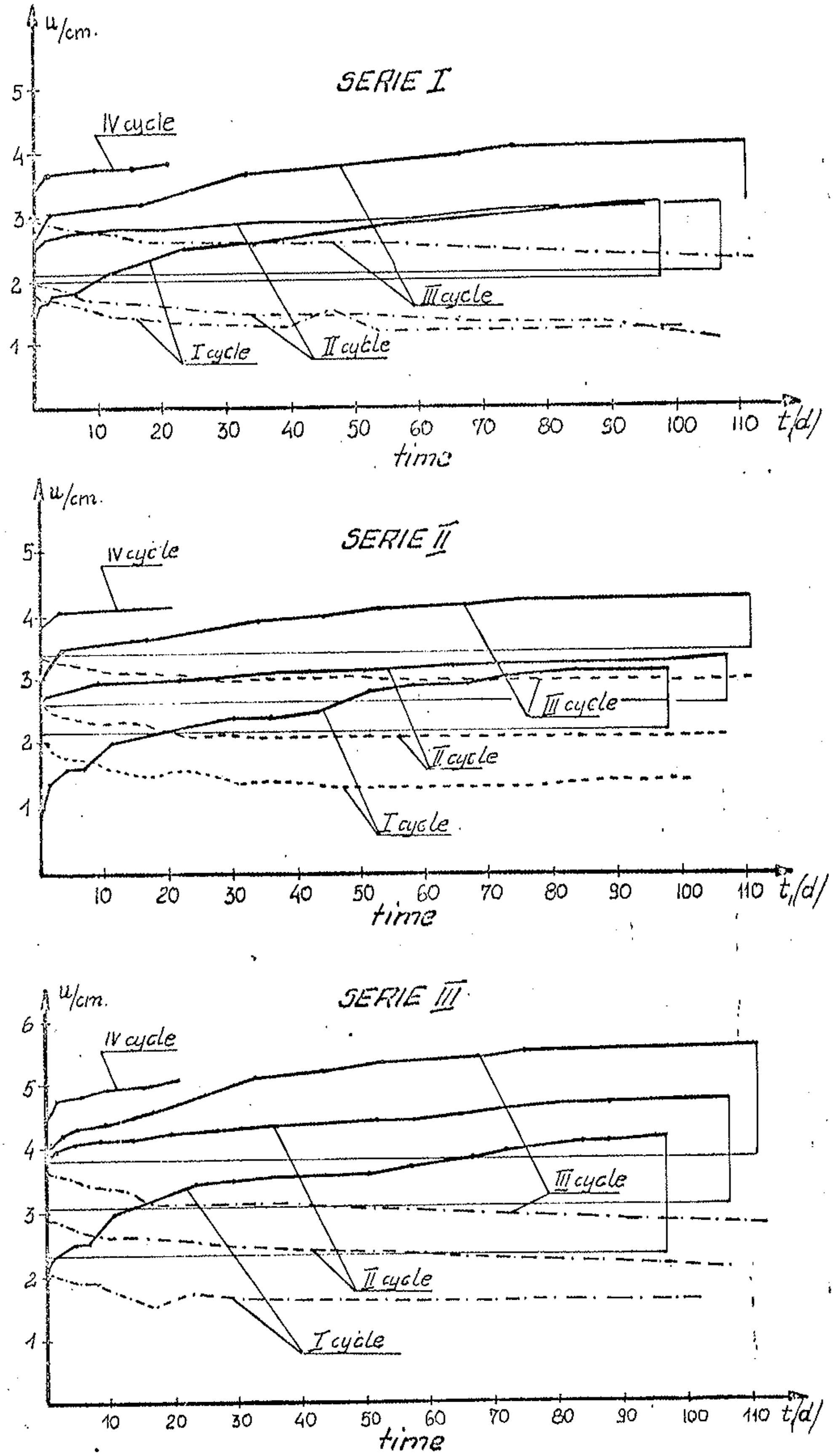

Fig. 8 Time-deflection curves under long-term cyclic load.

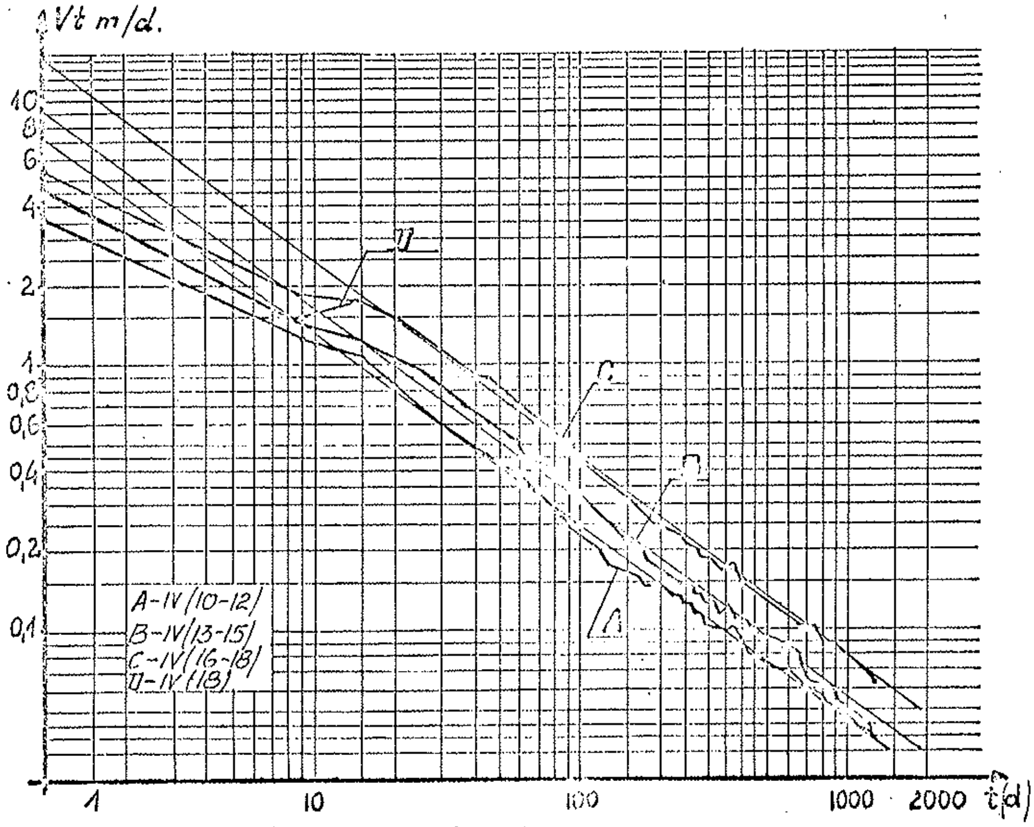

Fig.10 Vt – time curves.

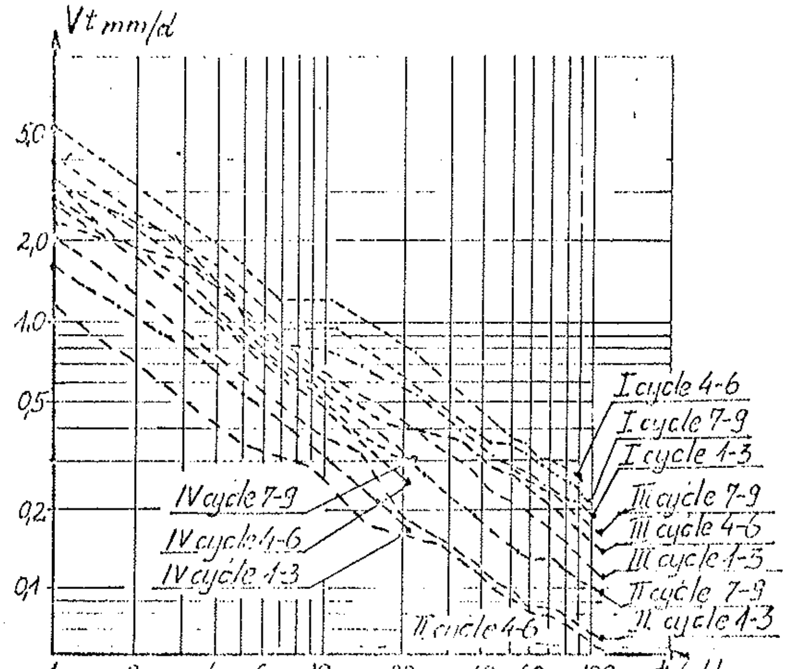

Fig. 11 Deflection velocity for beams in separate cycles.

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WORKING COMMISSION W18 - TIMBER STRUCTURES

DETERMINATION OF DEFORMATION OF SPECIAL DENSIFIED HARDBOARD UNDER LONG-TERM LOAD AND VARYING TEMPERATURE AND HUMIDITY CONDITIONS

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DETERMINATION OF DEFORMATION OF SPECIAL DENSIFIED HARDBOARD UNDER LONG-TERM LOAD AND VARYING TEMPERATURE AND HUMIDITY CONDITIONS

1. Introduction

Taking into account the growing demand for constructional materials in Poland, tests have been started in order to determine suitability of special densified hardboard for building structures.

The tests have been undertaken acknowledging physical and mechanical properties of special densified hardboard which proved to be a promissing material as far as strenght, influence of air humidity and homogeneous structure are concerned.

For complete assessment of special densified hardboard it has been necessary to know its behaviour under long-term load and in normal service conditions, it means under varying climate. The results given in this paper fill up the gap in this subject.

2. Scope

The tests have been carried out on 3,2 mm special densified hardboard manufactured by Zakłady Płyt Pilśniowych in Czarna Woda, Poland, which have been considered as a representative of all special densified hardboards produced in Poland, on the basis of their physical and mechanical properties determined before /5/.

Raw materials used for manufacture and basic data concerning manufacture are as follows:

a/ wood-based raw materials:

- edgings for pulping	circa 27 per cent
- slash	53 per cent
- solvent- extracted shavings	16 per cent
- scrap veneer and other waste materials	4 per cent

b/ improving materials

- wood rosin and slack emulsion - 4,2 kg/ton of sheet material /calculated in respect with dry matter of components/,
- aluminium sulphate 3,6 kg/100 m^2

- colorant /helion brown/ 0,1 kg/100m2
- oil preservative /mixture of linseedoil and talacyd D-50 in proportion of 3:1/ 95 kg/ ton of hardboard

c/ basic data concerning manufacture:

- moulding pressure 5,5 MPa
- moulding temperature 205°C
- oil preservative temperature circa 110°C
- oil treatment temperature circa 165°C

The tests on hardboard carried out by us as well as those carried out by technical staff of the producer proved its compliance with polish industry-wide standard BN-69/7122-11 "Fibreboard from wood".

3. Research program

The program has been based on principles and data from national and foreign literature , and was aproved by national scientific and research institutes interested in this theme.

Research program included:

1. Measurement of deformation of hardboard under long-term load and under cyclic changes of climate, with basic parameters as follows:
   a/ climate
   - temperature of 20°C and R.H. of 90%,
   - temperature of 60°C and R.H. of 60%.

   b/ load range: 15%, 20%, 25% and 30% of short-term strenght
   c/ type of long-term load:
   - in tension,
   - in bending

   d/ cycle of climatic changes: 15 days
   e/ loading time for specimens: 60 days.
2. Determination of short-term strenght of specimens
   a/ after conditioning / the strenght determined so constitutes a reference level for other tests/,
   b/ unloaded , e.g. after being under action of long-term load for 2, 4 and 6 months,
   c/ after being in different varying climates for 2,4 and 6 months without load.

4. Selection of samples for long-term tests

4.1. Specimens for measurement of deformation in tension

The design and dimensions of specimens for this test have been established after discussing the data related to specimens used for such tests in Poland and abroad considering besides the possibility of carrying them out. The form of specimens has been as the one recommended by FAO /4/ but with modified dimensions. The specimen is shown in figure 1 /see annex 1/.

4.2. Specimens for measurement of deflection in bending

For this test 150 mm long and 50 mm wide specimens have been taken , whose dimensions corresponded to those recommended by FAO /4/ and in accordance with polish standard /1/.

5. Testing equipment

5.1. Equipment for measurement of deformation in tension

The equipment /fig. 2 - see annex 1/ has been composed of two metal frames, the one being perpendicular to the other,jointed rigidly together.

In the vertical frame there have been 12 grips for specimens and in the horizontal frame have been 12 lever arms with leverage of 1 : 10 .

The specimens have been secured by grips, upper and lower ones. The lower grip has been connected with a lever arm by means of a connector having adjustable lenght.

At the edge of lever arm cast iron weights have been hanged.

The equipment has been placed in a climatic chamber in a way allowing testing of 48 specimens at the same time.

For measurement of elongation gauges having working range of 10mm and graduation of 0,01mm have been used. The gauges have been attached to the specimens /fig.3 - see annex 1/ by means of nailed plates driven into specimens.

5.2. Equipment for measurement of deflection in bending

The equipment has been composed of a rigid horizontal frame with legs so dead weights attached to the specimens were allowed to hang down /fig.4 - see annex 1/. To the frame two supports have been fixed, spaced at 100 mm, on which the specimens have been collocated.

The specimens placed on the supports have been loaded with cast iron weights hanged on a metal bar provided with a shackle, whose upper part formed a thrust having a radius of curvature required.

The deflection has been measured with a dial gauge having working range of 10mm and graduation of 0,01mm. The gauge has been additionally provided with special calibrated legs, allowing thus measurement of deflection exceeding its working range. The gauge leg crossing the hole in the beam and the hole in the shackle thrust /fig.5 - see annex 1/ has been rested on the centre of lenght and width of specimen, while the ring around the gauge leg /under gauge box/ rested on the beam, providing identical collocation of gauge during subsequent measurements.

5.4. Climatic chamber

A climatic chamber has been situated in the basement and adapted for this purpose. Outside the chamber two ventilators have been installed providing so a circulation of air in closed cycle.

The temperature inside the chamber has been mantained as required using contact thermometer for control of electric heaters. The relative humidity of air has been controlled by contact hygromenters connected with electric heaters in containers with water . The vapour produced by them increased air humidity. When a decrease of humidity has been needed the air has been impelled from the outside and electric heaters were switched on.

The chamber has been provided with two doors having a vestibule between them. In such a way the chamber has been protected against variation of temperature and humidity, when entering it.

The temperature inside the chamber of 60°C has been mantained with an accuracy of +2°C and humidity of 40% with an accuracy of +5%, while the temperature of 20°C has been mantained with an accuracy of -5°C and respective humidity of 90% with an accuracy of -5%.

6. Test procedure and results

6.1. Sampling

We have received 30 panels of hardboard, 6 of which have been selected for sampling. Out of each panel samples have been taken as follows:

- 36 samples for testing in tension,
- 36 samples for testing in bending.

While taking samples the following rules have been observed:

- the controls have been taken in accordance with polish industry-wide standard BN-69/7122-11/1,
- the samples have been taken in such a way that 50% of them have been situated across and 50% along the panel,
- the samples / except controls/ have not to be taken from the 15cm strip along the edges of panel,
- the samples used for one test have been representatives of all panels.

Considering the inconvenience of testing such a great amount of specimens in so small a chamber, reduced number of testing stations and long duration of a single test, each test has been carried out with 12 specimens.

6.2. Determination of short-term strenght

6.2.1. Standard procedure

Each of 6 panels underwent such a procedure in accordance with the standard /1/. It is necessary to mention that, according to the standard /1/ only one panel has to be controlled, but having in consideration a determination of precise properties, samples have been taken from every panel.

We considered also necessary to take the mean value of bending strenght as a reference level for specimens submitted to long-term tests.

The results of checking special densified hardboard /after conditioning in 20°C and 65% R.H./ concerning density, moisture uptake, swelling, moisture content and bending strenght are given in table 1.

The test results obtained have complained with the requirements of standard /1/.

6.2.2. Determination of tensile strenght

For determination of tensile strenght / not covered by the standards/ 5-ton Armsler machine has been used, the specimens being as shown in figure 1.

To this test 12 specimens have been submitted.

The tensile strenght has been calculated from the formula:

$$R_r = \frac{P}{b \cdot a} \text{ MPa}$$

where: $\bar{R}_r$ - tensile strenght, in MPa

b - width of specimen measured in reduced area, in cm

a - thickness of specimen, in cm.

The tensile strenght values obtained during the test on special densified hardboard pre-conditioned in 20°C and 65% R.H. ,are given in table 1.

Table 1

Mean short-term strenght values for special densified hardboard specimens

Item	Properties	Units	Results	Remarks
1	Density	kg/ m3	1030	1050 x/
2	Moisture uptake	per cent	15	
3	Swelling	per cent	9	
4	Moisture content	per cent	5	
5	Bending strenght	MPa	72,2	40,0 x/
6	Tensile strenght	MPa	47,5	25,0 x/

x/ requirements of PN-73/B-03150.

6.3. Long-term tests in tension

6.3.1. Procedure

The specimens have been conditioned until constant weight was obtained in 20°C and 65% R.H. climate, considered as a reference level Afterwards their working cross-sections have been measured /in the centre of reduced area/ which were taken for calculation of load. The load has been calculated considering, as it was said before, 15%, 20%, 25% and 30% short-term strenght /table 1/. The reference line 100 mm wide has been established, crossing the centre of working cross-section and on this line a dial gauge was fixed. The dial gauge has been fixed to the specimen by means of nailed plates driven into reference line. On the other reference line a stop has been fixed /in a similar way/ into which a leg of gauge was inserted. The specimen fixed in the equipment and prepared for the test shows figure 3 /see annex 1/.

Prior to test, the specimens have been pre-loaded /with small load/ to check the operation of a stopping lever arm gear. After taking zero reading, the specimen has been loaded very slowly and the next reading has been taken immediately after the loading was finished.

After finishing the loading, the equipment for I cycle of climate

/20°C , 90% R.H./ has been put into operation.

The readings of deformation have been taken after 12 hours, and subsequently after 1, 2, 3, 4 and 5 days and later every 5 days for a six month period. The tests have been run under cyclic changes of climate: for 15 days 20°C and 90% R.H. climate was mantained and for the next 15 days 60°C and 40% R.H.

To such variations of climate loaded specimens have been submitted also without measuring their deformation as well as non-loaded specimens. In such a case different strenght values have been measured to determine the influence of long-term load under cyclic changes of climate and separately the effect of cyclic changes of climate /without action of load/.

The test procedure has been as given in 6.2.2. using specimens pre-conditioned in 20°C and 65% R.H. climate.

6.3.2. Test results

The mean value of deformation of specimens in bending in relation with time and load level are given in table 2 and represented as a curve in figure 6 /see annex 1/.

The mean value of deformation in relation with time and cyclic changes of climate are given in table 3.

Table 2

Mean deformation value for special densified hardboard in tension in relation with time, load level and cyclic variations of climate

Climate t °C / R.H.%	Loading time days	Mean deformation in mm at load level of 15%	20%	25%	30%
1	2	3	4	5	6
	prior to loading	0,00	0,00	0,00	0,00
	immediately after loading	0,04	0,05	0,05	0,08
	0,5	0,17	0,19	0,13	0,16
	1	0,19	0,21	0,17	0,22
	2	0,23	0,24	0,19	0,25
20/90	3	0,26	0,26	0,20	0,28
	4	0,28	0,28	0,23	0,32
	5	0,30	0,31	0,28	0,41
	10	0,30	0,32	0,57	
	15	0,32	0,34	0,79	
	20	0,37	0,49	0,95	
60/40	25	0,40	0,52	1,05	
	30	0,41	0,53	1,27	

to be continued on page 8

Table 2

continued from page 7

1	2	3	4	5	6
20/90	35 40 45	0,45 0,50 0,54	0,61 0,77 0,81	1,39 1,52 1,66	
60/40	50 55 60	0,57 0,61 0,62	0,82 0,83 0,72	1,68 1,67 1,79	
20/90	65 70 75	0,64 0,66 0,68	0,70 0,77 0,95	1,82 1,83 1,86	
60/40	80 85 90	0,69 0,70 0,71	0,98 1,12 1,03	1,87 1,89	
20/90	95 100 105	0,74 0,76 0,81	1,00 1,09 1,16		
60/40	110 115 120	0,84 0,89 0,94	1,21 1,25 1,21		
20/90	125 130 135	0,98 1,00 1,02	1,20 1,18 1,22		
60/40	140 145 150	1,06 1,08 1,10	1,26 1,21 1,21		
20/90	155 160 165	1,13 1,17 1,20	1,22 1,24 1,28		
60/40	170 175 180	1,24 1,26 1,28	1,28 1,36 1,41		

Table 3

Mean tensile strenght of special densified hardboard in relation with loading time and changing climate

Load level in %	Tensile strenght in MPa during long-term loading for a period of			Remarks
	2 months	4 months	6 months	
0	41,1	39,5	39,8	
15	43,1	38,3	33,8	
20	-	36,3	33,8	
25	-	-	-	9 specimens have failed in 55days
30	-	-	-	all specimens have failed in 14 days

6.3.3. Interpretation of test results

Discussing the results from table 3 is possible to ascertain that strenght of specimens in changing climate and without loading for six months has decreased in circa 16%. The strenght of specimens under 15% load has been reduced in 30%.

The strenght of specimens under load for 2 months /at load level corresponding to 20% of short-term strenght/ has not been determined, but after 6 months under such load strenght decrement has been of 30%.

Under load corresponding to 25% and 30% of short-term strenght all specimens have failed before 2 months were over. The results are given in table 3.

The loads used for test have their corrsponding stress values which are as follows:

- at 15% of short-term strenght the respective stress is 7,1 MPa
- at 20% of short-term strenght the respective stress is 9,5 MPa
- at 25% of short-term strenght the respective stress is 11,9 MPa
- at 30% of short-term strenght the respective stress is 14,2 MPa

Analysing the deformation curves in figure 6 one can state that at stresses of 7,1 MPa and 9,5 MPa, their initial portion is curved but the next one has been practically straight-lined, what proves stabilization of deformation velocity.

On the basis of curves mentioned , formulae for straight-lines are given / using approximation method/ :

- for 7,1 MPa stress $y = 0{,}00583\ t + 0{,}25$
- for 9,5 MPa stress $y = 0{,}00555\ t + 0{,}50$.

The above equations represent the relation between deformation /elongation/ of panel in bending "y" in mm and time "t" in days of action of load. The equations for other values of stresses could not be calculated because to many specimens have failed after short time .

6.4. Long-term tests in bending

6.4.1. Test procedure

The specimens have been conditioned in 20°C and 65% R.H. until constant weight was obtained and then their working cross-sections have been measured, for which load value was calculated taking the short-term strenght /table 1/ for calculation of load. After the shackles have been weighed , a number of weights have been attached to the rods collocated on them /fig.4/. The specimens have been put on the supports, the span between them being of 100 mm and

and zero reading has been taken /without loading/. Afterwards the shackle with weights has been hanged on the specimen /in the centre of span between supports/ and successsive readings were taken. The method of measurement is shown in figure 5 /see annex 1/.

After readings and loading have been completed, the device maintaining inside the chamber a temperature of 20°C and 90% R.H. were put in operation. The readings of deformation have been taken after 1, 2, 3 ,4 and 5 days , and then every five days during six month period. In the run of the test the climate has been changed cyclically every 15 days /from 20°C ,90% R.H. to 60°C , 40% R.H./.

To the action of such a climate loaded specimens have been also submitted for which no reading of deformation has been taken, as well as non-loaded specimens.

Those specimens have been tested to check the effect of long-term load and cyclic changes of climate or changes of climate exclusively.

The tests have been carried out in accordance with the requirements of standard /1/ and on specimens pre-conditioned in 20°C and 65% R.H. having modified width as described in 4.2.

6.4.2. Test results

The mean deformation of specimens in bending in relation with load level and time is given in table 4 and illustrated in figure 7 as a curve.

The mean values of bending strenght in dependence on loading time and cyclic changes of climate are given in table 5.

Table 4

Mean deformation of special densified hardnoard in bending in dependendence on load level and cyclic changes of climate

Climate t °C / RH %	Loading time in days	Mean deformation in mm under load level of 15%	20%	25%	30%
1	2	3	4	5	6
	Prior to loading	0,00	0,00	0,00	0,00
	Immediately after loading	1,68	2,70	3,23	3,94
20/90	1	2,15	3,13	5,49	8,54
	2	2,19	3,29	-	-
	3	2,21	3,40	-	-
	4	-	-	6,36	10,09
	5	2,34	3,59	-	-
	8	-	-	6,85	11,31
	10	2,42	4,10	6,59	
	15	4,85	7,65	7,50	

To be continued on page 11

Table 4

continued from page 10

1	2	3	4	5	6
60/40	20	5,31	8,34	7,87	
	25	5,46	8,67	8,82	
	30	5,65	8,98	9,07	
20/90	35	5,76	9,19	9,52	
	40	5,77	9,32	10,75	
	45	5,86	9,51	10,98	
60/40	50	5,92	9,73		
	55	6,07	10,10		
	60	6,27	10,18		
20/90	65	6,36	10,32		
	70	6,39	10,47		
	75	6,42	10,57		
60/40	80	6,43	10,61		
	85	6,44	10,66		
	90	6,46	10,70		
20/90	95	6,48	10,74		
	100	6,50	10,77		
	105	6,53	10,80		
60/40	110	6,55	10,88		
	115	6,57	11,02		
	120	6,61	11,36		
20/90	125	6,73	11,64		
	130	6,79	11,99		
	135	7,13	12,51		
60/40	140	7,45	12,80		
	145	7,63	13,02		
	150	7,78	13,32		
20/90	155	7,84	13,71		
	160	7,88	13,83		
	165	7,93	14,31		
60/40	170	8,12	14,53		
	175	8,30	15,01		
	180	8,39	13,93		

Mean bending strenght of special densified hardboard in relation with loading time and cyclic changes of climate

Load level in %	Bending strenght in MPa under long-term load for a period of			Remarks
	2 months	4 months	6 months	
0	69,3	66,6	61,3	
15	64,0	58,9	56,5	
20	63,1	x/	x/	x/ specimens have been improper for testing
25	-	-	-	specimens have failed after approximately 1,5 month
30	-	-	-	specimens have failed after 2 days

6.4.3. Interpretation of test results

The specimens of special densified hardboard tested not under load but under cyclic changes of climate exclusively after 2, 4 and 6 months showed a streght decrement of 4%, 8% and 15 % respectively. Strenght decrement in case of specimens under 15% load level for 6 months has been of 10% in comparison with non-loaded specimens. Under 20% load level strenght decrement has been determined solely after 2 month , which was of 9%, because after 4 and 6 months the specimens have been greatly deformed.

The load levels applied in bending have corresponded to stresses as follows:

- to a 15% of short-term strenght corresponds a stress of 10,8 MPa
- to a 20% of short-term strenght corresponds a stress of 14,4 MPa
- to a 25% of short-term strenght corresponds a stress of 18,1 MPa
- to a 30% of short-term strenght corresponds a stress of 21,7 MPa

From the table 4 and figure 7 one can deduce that deformation depends on climate and time of its action, but always the load determine decively deformation value. 25% and 30% load levels have been to high for special densified hardboard, for all specimens failed after 1,5 month and after 8 days respectively.

On the basis of results obtained equations of straight lines /after approximation of curves from figure 7/ are given:

- for 10,8 MPa stress $y = 0,02\ t + 4,8$
- for 14,4 MPa stress $y = 0,0394\ t + 7,4$

what illustrates the relation between deflection "y" and time of action of determined stress.

6.4.4. General interpretation of test results

From the tests above described one can deduce that strenght of specimens submitted for 6 months to variable climate is reduced in 15% in tension and in bending.

At 15% load level strenght decrement has been of 30% in tension and 10% in bending in case of non-loaded specimens placed in cyclically varying climate. The specimens being under load corresponding to 20% of short-term strenght showed strenght decrement in 30% in tension , but in bending all specimens have failed and the respective decrement could not be determined.

Comparing the strenght of specimens after 6 months under load level of 15% and 20% of short-term strenght in variable climate /tables 3 and 5/ with short-term strenght from table 1, the conclusions are as follows:

- in tension - under load of 15% and 20% of short-term strenght the corresponding strenght decrement has been of 30%,
- in bending - under load of 15% of short-term strenght , the corresponding strenght decrement has been of 22% ,for under load of 20% of short-term strenght the specimens failed.

Load levels of 25% and 30% of short-term strenght are for special densified hardboard too excessive in tension and in bending, for the specimens have failed. So in consideration can be taken only 15% and 20% load levels with corresponding stresses of 10,8 MPa and 14,4 MPa in bending and 7,1 MPa and 9,5 MPa in tension.

The results obtained suggest that corresponding stresses for special densified hardboard in polish standard PN-73/B-03150 /table 13 and clause 3.10.2/ which for tension are of 60 x 0,5 = 30 kG/cm2= 3,0 MPa , and for bending of 100 x 0,5 = 50 kG/cm2 = 5,0 MPa ,should be corrected.

The deformations attained under 15% and 20% load level in tension have been practically equal, but under 25% load level deformation has been so large that specimens failed before 3 months were over.

Deformations in bending have been very unfavourable, e.g. under 20% load level after 6 months, deformation values were in 90% greater than those obtained under load level of 15%. It is necessary to mention that corresponding curves have been also very unfavourable.

Considering the growing use of special densified hardboard for wooden structures it is very important to calculate deflection of such structures taking into account the factor of time related with large deformations.

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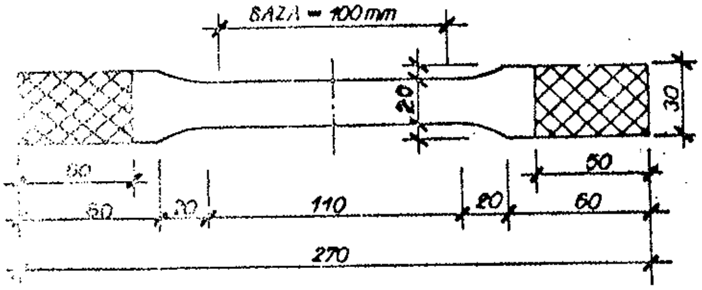


Fig.1 Details of specimen for measurement of deformation in tension.

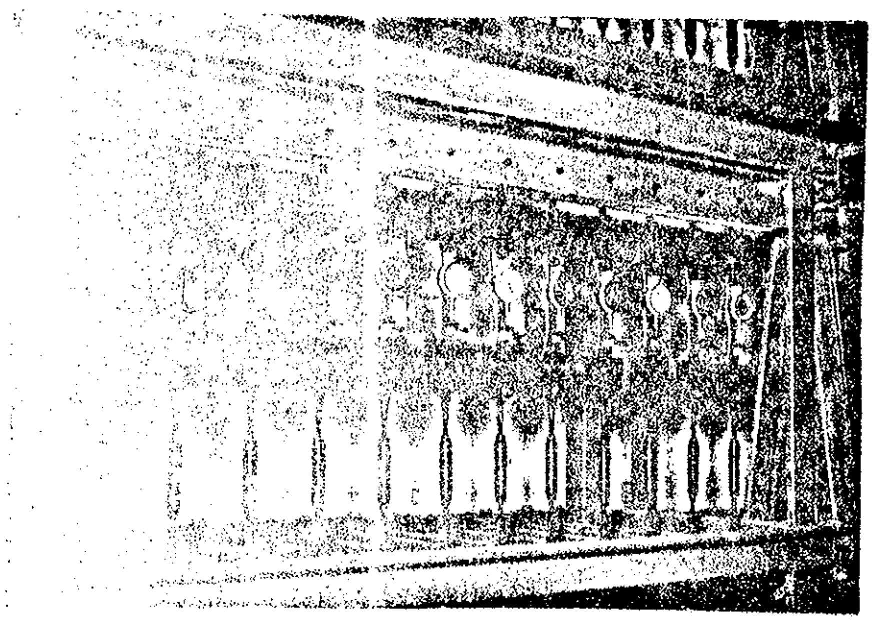


Fig. 2 Testing equipment for measurement of deformation in long-term tension.

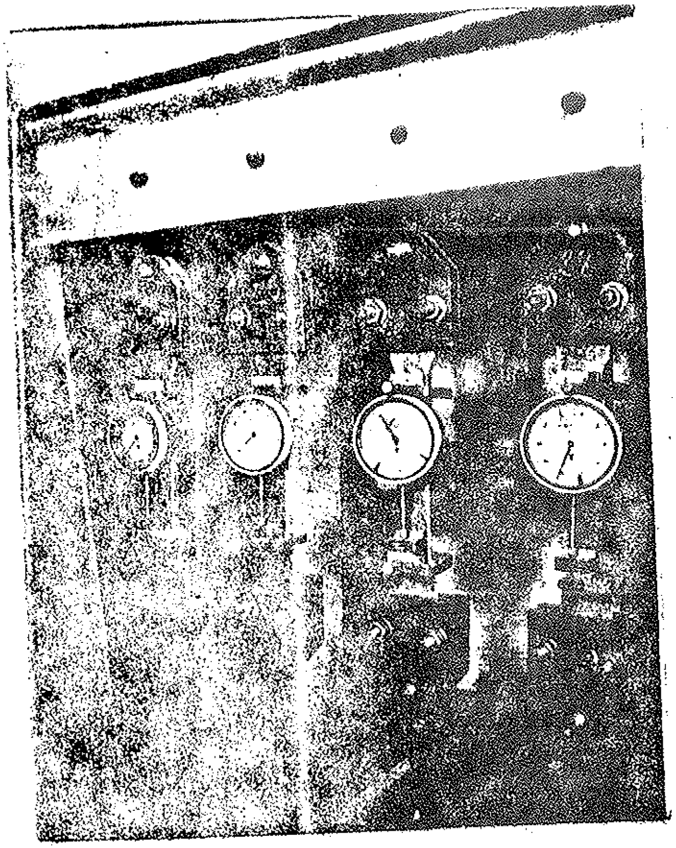


Fig.3 Specimens with gauges prepared for test.

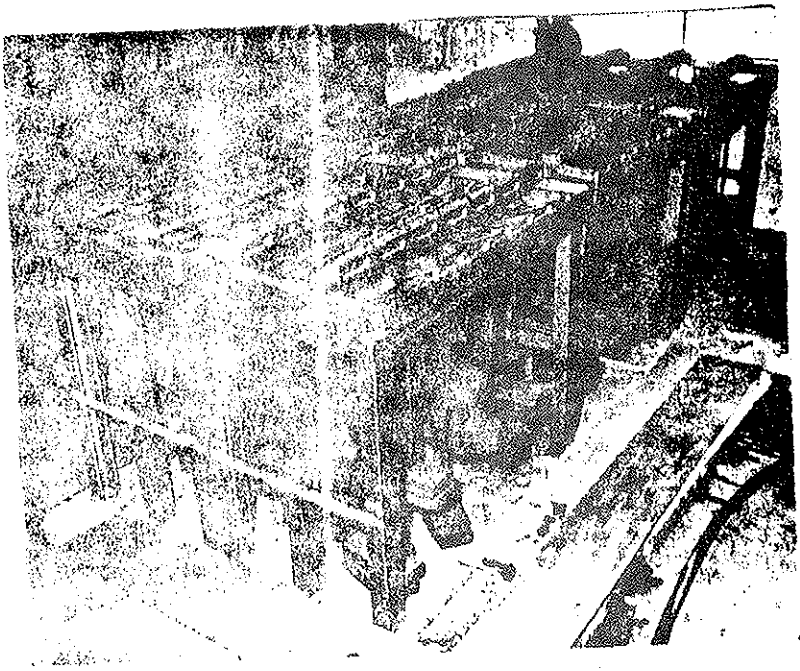


Fig.4 Equipment for measurement of deflection under long-term load in bending.

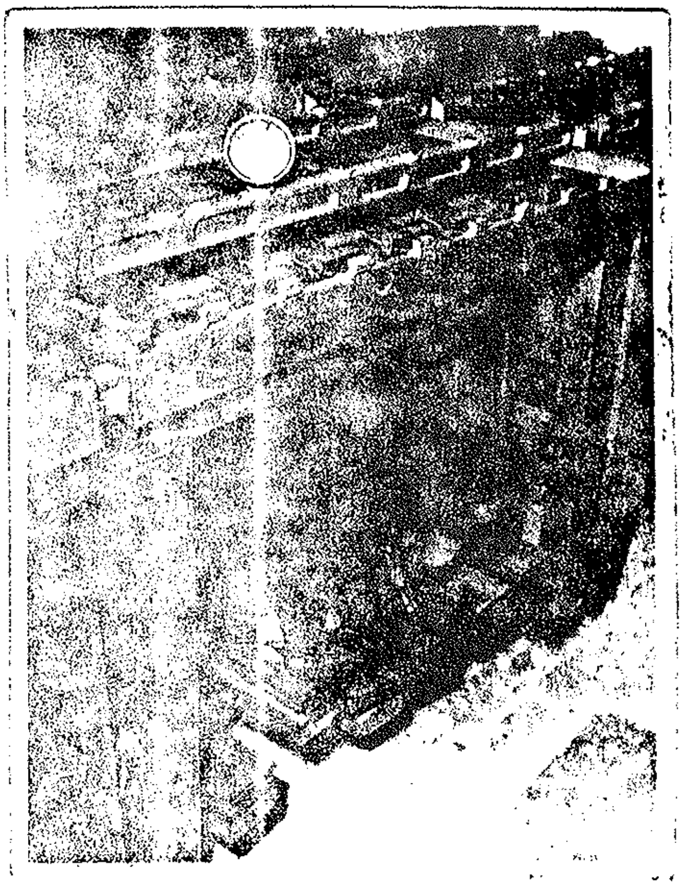
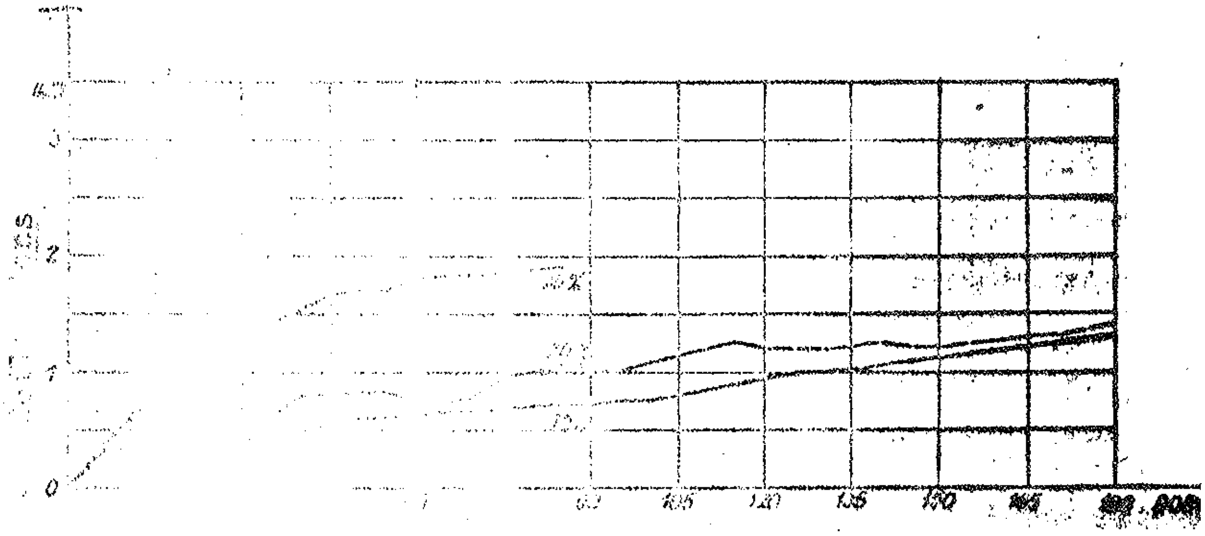
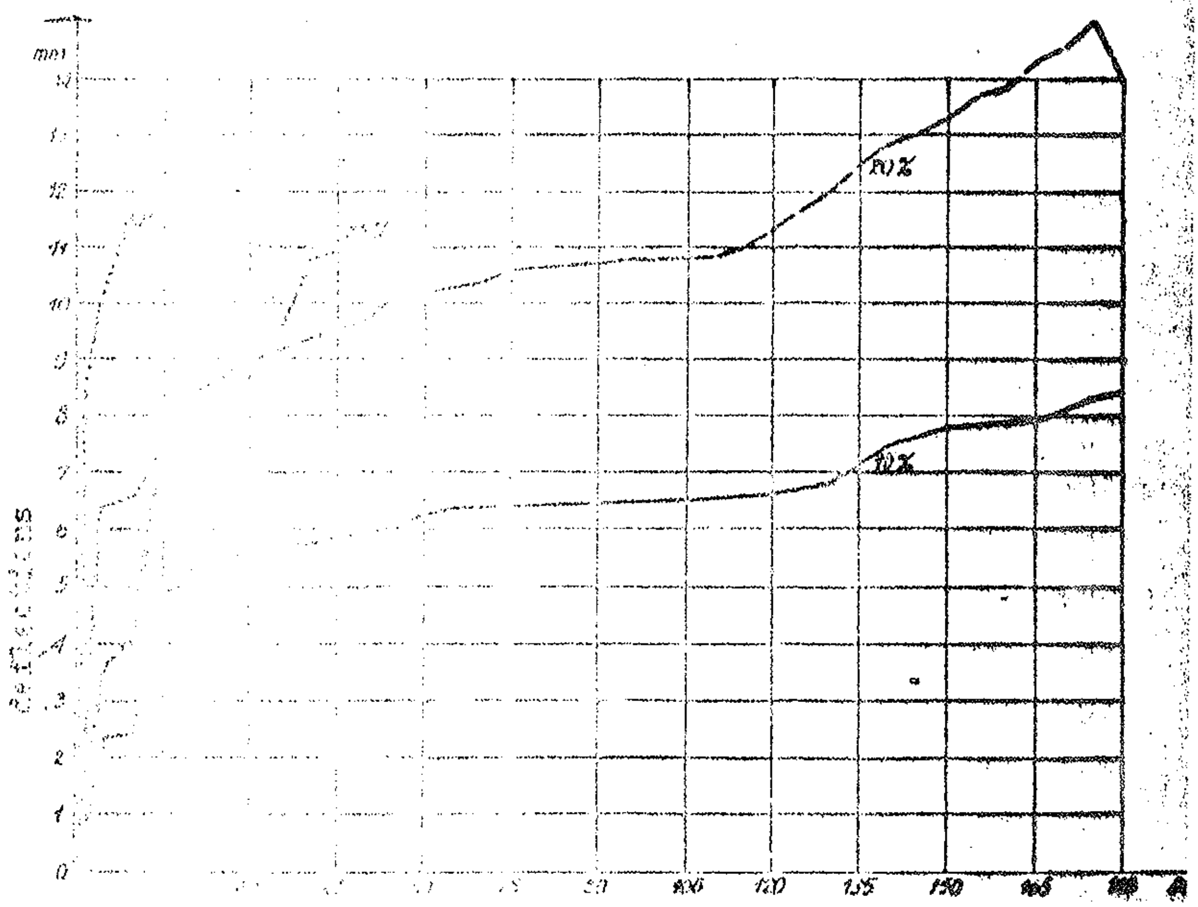


Fig.5 Method of measuring deflection of specimens.



d.

Fig.6 ... curves in tension for ... hardboard /BT/ under ... load levels.



d.

Fig.7 ... curves in bending for ... hardboard under different load

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DETERMINATION OF DEFORMATION OF HARDBOARD UNDER LONG-TERM LOAD IN CHANGING CLIMATE

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DETERMINATION OF DEFORMATION OF HARDBOARD UNDER LONG-TERM LOAD IN CHANGING CLIMATE

1. Introduction

Constantly growing interest in hardboard is due partly to a planned increase of its production in Poland and partly to a possibility of substitution of wood with a similar product having more homogeneous structure.

Recommending the application of hardboard for building constructions, structures made from wood and hardboard , such as roof coverings in form of laminated or nailed prefabricated units have been taken into account.

2. Scope

The tests have been carried out with 4mm hardboard manufactured by Zakłady Płyt Pilśnowych in Czarna Woda ,Poland, this being acknowledged as a representative of all national hardboards.

On the grounds of data received from the manufacturer, we describe here a characteristic properties of raw materials as well as basic data concerning manufacture of hardboard:

a/ wood-based raw materials

- edgings for pulping	circa	25 per cent
- slash		53 per cent
- solvent- extracted shavings		16 per cent
- scrap veneer and other waste materials		4 per cent

b/ improving materials

- wood rosin and slack wax emulsion - 4,2 kG/ton of sheet material / calculated in respect with dry matter of components/,

- aluminium sulphate	circa	3,6 kG/100m2
- colorant /helion brown/		0,1 kG/100m2

c/ Data concerning manufacture:

- moulding pressure	5,5 MPa
- moulding temperature	205°C
- oil treatment temperature	165°C

Tests on hardboard panels carried out by us as well as those carried out by the technical control staff of manufacturer proved their compliance with the polish industry-wide standard BN-69/7122 "Fibreboard from wood".

3. Research program

The program has covered procedures of determination of hardboard properties, important from view-point of design of different building units, primarily roof coverings from wood and hardboard.

So measurement of deformation of various specimens of panels under action of load in changing climate conditions became necessary. The most undesirable effect may take place under cyclic changes of climate.

The research program has been established on the basis of related literature and after consultations held with Instytut Techniki Budowlanej /Institute of Building Technology/, Politechnika Warszawska /Technical University of Warsaw/, Centralne Laboratorium Płyt Pilśniowych / Central Fibreboard Laboratory/ and Academies of Agriculture of Warsaw and Poznań.

Research program included:

1. Measurement of deformation of hardboard under cyclic changes of climate with basic parameters as follows:
   a/ climate
      - temperature 20°C and R.H. 90%
      - temperature 60°C and R.H. 60%
   b/ load range: 15%, 20%, 25% and 30% of short-term strenght
   c/ type of long-term load
      - in tension
      - in bending
   d/ cycle of climatic changes: 15 days
   e/ loading time for specimens : 60 days.

2. Determination of short-term strenght of specimens
   a/ after conditiong the strenght determined so,constitutes a reference level for other tests.
   b/ unloaded , e.g. after being under action of different long-term loads for 2, 4 and 6 months,
   c/ after being in different changing climates for 2, 4 and 6 months without load.

4. Selection of samples for long-term tests

4.1. Specimens for measurement of strain in tension

The design and dimension of specimens for the test have been determined after analysing data relating to specimens used for such tests in Poland and abroad , considering besides the possibility of carrying them out. The design of specimens has been as the one recommended by FAO /4/, but having modified dimensions.
The specimens is shown in figure 1 /see annex 1/.

4.2. Specimens for measurement of deflection in bending

For this test 150mm long and 50 mm wide specimens have been taken, having dimensions equal to those recommended by FAO /4/ and corresponding to the standandard /1/.

5. Testing equipment

5.1. Equipment for measurement of strain in tension

The equipment / illustrated in figure 2 - annex 1/ has been composed of two metal frames, the one being perpendicular to the other, jointed rigidly together.

In the vertical frame there have been 12 grips for specimens and in the horizontal one there have been 12 lever arms having leverage of 1:10, provided with grips situated on the connectors having ajustable lenght.

The specimens have been fixed to the grips.

Considering the relatively small surface of climatic chamber, the equipment described has been situated in two levels / figure 2/ allowing in this way testing of 48 specimens at the same time.

For measurement od elongation gauges having operating range of 10 mm and graduation of 0,01 mm have been used. The gauges have been fixed to the specimens with nailed plates , driven into specimens.

5.2. Equipment for measurement of deflection in bending

The equipment has been composed of a rigid horizontal frame with legs, so dead weights attached to the specimens were allowed to hang down / figure 4 - annex 1/. To the frame have been fixed two supports spaced at 100mm and having radius of curvature complying with the standard /1/. In the centre of this span a beam has been fixed above the frame with 6 holes for the leg of dial gauge for measurement od deformation of specimen in the centre of its lenght and width.

A specimen placed on the supports above the holes in the beam, has been loaded with dead-weights hanged on the metal bar provided with shackle, whose upper part formed a thrust having a radius of curvature required.

The deformation has been measured with dial gauge having working range of 10 mm and graduation of o,01 mm. The gauge has been aditionally provided with special calibrated legs , allowing thus measurement of deformation exceeding its working range. The gauge leg passing through the hole in the beam and the hole in the shackle thrust /figure 5 - annex 1/ has been rested on the centre of lenght and width of specimen, while the ring around the gauge leg /under gauge box/ rested on the beam providing identical collocation of gauge during subsequent measurements.

5.4. Climatic chamber

A climatic chamber has been $12m^2$ room situated in the basement and adapted for this purpose. Two ventilators situated outside the chamber with pipes provided air circulation inside the chamber in closed cycle.

The temperature inside the chamber has been mantained as required using contact thermometer for control of electric heaters.

The relative humidity of air has been controlled by contact hygrometers connected with electric heaters submerged in containers with water. The vapour produced by them increased air humidity. When a decrease of humidity has been needed the air has been impelled from outside and electric heaters of air have been switched on.

The chamber has been provided with two doors having a vestibule between them. In such a way the chamber has been protected against variation of temperature and humidity, when entering it.

The temperature of 60°C inside the chamber has mantained with an accuracy of 2°C and humidity of 40% with an accuracy of+5%, whilst the temperature of 20°C has been mantained with an accuracy of +5°C and humidity of 90% with an accuracy of -5%.

6. Test procedure and results

6.1. Sampling

We have received 30 panels of hardboard, 6 of which have been selected for sampling. Out of each panel samples have been taken as follows:

- 36 samples for testing in tension,
- 36 samples for testing in bending,
- 18 samples for testing in compression.

While taking samples the following rules have been observed:

- the controls have been taken in accordance with polish industry-wide obligatory standard BN-69/7122-11/1,
- the samples have been taken in such a way that 50% of them were situated across and 50% along the panel,
- the samples / except controls/ have not to be taken from the 15cm strip along the edges of a panel,
- the samples used for one test have been representatives of all panels.

Considering the inconvenience of testing such a great amount of specimens in so small a chamber, reduced number od testing stations and long duration of single test, each test has been carried out with 12 specimens, except long-term testing in compression where only 5 specimens have been used.

6.2. Determination of short-term strenght

6.2.1. Standard procedure

Each of six panels underwent such a procedure in accordance with the standard /1/. It is necessary to mention that, according to the standard /1/ only one panel has to be controlled, but having in consideration a determination of precise properties, samples have been taken from every panel.

The test procedure and calculations have been in accordance with the standard /1/. It has been decided to use a mean bending strenght as a reference level for long-term load of specimens.

The results of checking of hardboard / after conditioning in 20°C and 65% R.H./concerning density, moisture uptake, swelling, moisture content and bending strenght are shown in table 1.

The results obtained have complained with the requirements of standard /1/.

6.2.2. Additional tests

6.2.2.1. Determination of tensile strenght

For determination of tensile strenght /not covered by the standard mentioned/ 5-ton Armsler machine have been used , the specimens being as shown in figure 1 /see annex 1/.

To this test 12 specimens have been submitted.

The tensile strenght has been calculated from the formula:

$$R_r = \frac{P}{b \cdot a} \text{ MPa}$$

where: R_r - tensile strenght , in MPa
b - width of specimen measured in reduced area, in cm
a - thickness of specimens,in cm.

The tensile strenght values obtained during the test on hardboard conditioned in 20°C and 65% R.H. , are given in table 1.

6.2.2.2. Determination of compression strenght

For determination of compression strenght / not covered by polish standards/ 5-ton Armsler machine has been used for testing laminated specimens ,glued with resorcinol adhesives. The specimens having coross-section of 24x24 mm and span of 100 cm have been glued of 6 laminae, and afterwards conditioned in 20°C and 65% R.H. climate until constant weight has been obtained.
The compression strenght has been calculated from:

$$R_c = \frac{P}{b \cdot a} \text{ MPa}$$

where: R_c = compression strenght in MPa
b - width of specimen , in cm
c - thickness of specimen , in cm.

The mean compression strenght value ,R_c is given in table 1.

Table 1

Mean values of short-term strenght of hardboard specimens

Properties	Units	Results
Density	kg/m3	940
Moisture uptake	per cent	17
Swelling	per cent	11
Moisture content	per cent	7
Bending strenght	MPa	45,4
Tensile strenght	MPa	27,6
Compression strenght	MPa	26,3

6.3. Long-term tests in tension

6.3.1. Procedure

The specimens have been conditioned until constant weight was obtained in 20°C and 65% R.H. climate ,considered as a reference level.

Afterwards their working cross-sections have been measured /in the cetre of reduced area/,which were taken for calculation of load. The load has been calculated considering, as it was said before, 15%,20%, 25% and 30% short-term strenght /table 1/.The reference line 100mm wide has been established, crossing the centre of working cross-section and on this line a dial gauge was fixed. The dial gauge has been fixed to the specimen by means of nailed plates driven into reference line. On the other reference line a stop has been fixed /in a similar way/ into which a leg of gauge was inserted.

The specimen fixed in the equipment and prepared for test illustrates figure 3 /see annex 1/.

Prior to test the specimens have been pre-loaded /with small load/ to check the operation of a stopping lever arm gear. After taking zero reading, the specimen has been loaded very slowly and the next reading was taken immediately after the loading was finished.

After finishing the loading, the equipment for I cycle of climate /20°C, 90% R.H./ has been put in operation.

The readings of deformation have been taken after 12 hours and subsequently after 1, 2, 3, 4 and 5 days and then every 5 days for a six month period. The tests were run under cyclic changes of climate: for 15 days 20°C and 90% R.H. climate have been maintained and for the next 15 days 60°C and 40% R.H.

To such variations of climate loaded specimens have been submitted also without measuring their deformation as well as non-loaded specimens. In such case different strenght values have been measured to determine the influence of long-term loading under cyclic changes of climate /without action of load/.

The test procedure has been as given in 6.2.2.1. using specimens conditioned in 20°C and 65% R.H. climate.

6.3.2. <u>Results</u>

The mean value of deformation of specimens in bending in relation with time and load level are given in table 2 and illustrated as a curve in figure 6 /see annex 1/.

The mean value of deformation in relation with time and cyclic changes of climate are given in table 3.

Table 2

Mean deformation value for hardboard in relation with time, load level and changes of climate

Climate t °C / R.H.	Loading time days	Mean deformation in mm at load level of 15%	20%	25%	30%
20/90	prior to loading	0,00	0,00	0,00	0,00
	immediately after loading	0,01	0,01	0,09	0,28
	0,5	0,03	0,07	0,27	0,37
	1	0,05	0,09	0,28	0,42
	2	0,08	0,13	0,29	0,42
	3	0,10	0,17	0,31	0,44
	4	0,23	0,33	0,33	0,45
	5	0,34	0,36	0,50	0,46
	10	0,39	0,79	1,04	0,63
	15	0,57	0,83	1,07	1.26
60/40	20	0,63	0,87	1,10	[illegible]
	25	0,66	0,91	1,14	1,66
	30	0,72	1,03	1,22	1,97
20/90	35	0,73	1,12	1,29	2,01
	40	0,73	1,17	1,31	2,01
	45	0,69	1,13	1,32	1,87
60/40	50	0,71	1,09	1,24	1,88
	55	0,72	1,10	1,26	1,93
	60	0,95	1,17	1,31	2,00
20/90	65	1,02	1,18	1,32	2,12
	70	1,02	1,19	1,30	2,12
	75	1,01	1,15	1,30	2,09
60/40	80	1,02	1,14	1,28	2,11
	85	1,14	1,13	1,31	2,64
	90	1,15	1,24	1,34	2,59
20/90	95	1,10	1,26	1,38	2,57
	100	1,06	1,28	1,40	2,55
	105	1,06	1,25	1,37	2,56
60/40	110	1,10	1,22	1,33	2,57
	115	1,18	1,28	1,31	2,73
	120	1,19	1,19	1,35	2,75
20/90	125	1,11	1,27	1,38	2,82
	130	1,04	1,29	1,38	2,61
	135	1,05	1,25	1,39	2,44
60/40	140	1,27	1,30	1,35	2,64
	145	1,25	1,37	1,39	2,76
	150	1,33	1,39	1,44	2,77
20/60	155	1,35	1,41	1,43	2,77
	160	1,35	1,40	1,43	2,77
	165	1,34	1,39	1,44	2,77
60/40	170	1,27	1,34	1,45	2,78
	175	1,22	1,36	1,46	2,79
	180	1,29	1,47	1,61	2,79

Table 3

Mean tensile strenght in relation with loading time and changing climate

Load level in %	Tensile strenght in MPa during long-term loading for a period of 2 months	4 months	6 months	Remarks
0	24,8	24,1	24,6	
15	21,7	20,8	19,4	
20	22,2	21,9	18,6	
25	-	-	18,2	the mean of 6 samples,for the others have failed under long-term load
30	-	-	18,5	the mean of 5 samples for the others have failed under long-term load

6.3.3. Interpretation of test results

Discussing the results from table 3 is possible to ascertain that the strenght of specimens in changing climate and without loading for 6 months has decreased in circa 10%.

The strenght of specimens under 15% load has been reduced in 30% and those under 20% load in 33%.

In accordance with the results /given in the table 2 and illustrated in figure 6/ deformation depends on variation of temperature and ralative humidity of air as well as on the time of their action. During 6-month period the specimens under 20% load level have not failed ,anyhow those being under greater load levels partially failed / approximately in 50 %/.

Discussing load levels is also valuable give their respective stresses which are as follows:

- to a 15% load level corresponds 4,2 MPa stress,
- to a 20% load level corresponds 5,5 MPa stress,
- to a 25% load level corresponds 6,9 MPa stress,
- to a 30% load level corresponds 8,3 MPa stress.

Whilst analysing deformation curves /fig. 6/ one can ascertain with easiness, that the initial portion of curve is not straight-lined and the successive one is almost straight-lined, what proves proportional velocity of deformation increment.

On the basis of the curves mentioned, formulae for straight lines are given / using approximation method/ for stresses of 4,2 MPa and 5,5 MPa respectively:

-

- for 4,2 MPa stress $y= 0,00472\ t + 0,57$
- for 5,5 MPa stress $y= 0,00264\ t + 0,95$

The above equations illustre the relation between deformation /elongation/ of panels in tension "y" in percent and time/in days/ of action of determined stresses. It has been impossible to calculate equations for higher stresses, because to many specimens failed while tested.

6.4. Long-term tests in bending

6.4.1. Test procedure

The specimens have been conditioned until constant weight was obtained in 20°C and 65% R,H, and then their working cross-sections have been measured, for which load value was calculated taking short-term strenght /table 1/ as an initial strenght for all specimens.

After the shackles have been weighted, a proper number of weights were attached to the rods collocated on them /fig.4/. The specimens have been put on the supports, the span between them being of 100mm and zero reading was taken /without loading/. Afterwards the shackle with weights has been hanged on the specimen / in the centre of span between supports/ and the subsequent readings were taken.

The method of measurement is shown in figure 5 /see annex 1/.

After the readings and loading have been finished, the devices maintaining inside the chamber a temperature of 20°C and 90% R.H. were put in operation. The readings of deformation have been taken after 12 hours, 1,2, 3, 4 and 5 days , and then every 5 days during six month period.

In the run of test the climate has been changed cyclically every 15 days /from 20°C - 90% R.H. to 60°C and 40% R.H./.

To the action of such a climate loaded specimens have been also submitted for which no reading of deformation was taken, as well as non-loaded specimens. Those specimens have been tested to check the effect on them the long-term load and cyclic changes of climate or changes of climate exclusively.

The tests have been carried out in accordance with the requirements of standard /1/ and on specimens previously conditioned in 20° C and 65% R.H.

6.4.2. Test results

The mean deformations of specimens in bending in relation with load level and time are given in table 4 and illustrated as a curve in figure 7 /see annex 1/.
The mean values of bending strenght in dependence on loading time and cyclic changes of climate are given in table 5.

Table 4

Mean deformation of hardboard in bending
in dependence on load level and cyclic changes of climate

Climate t °C / RH %	Loading time in days	Mean deformation in mm under load levels 15%	20%	25%	30%
	Prior to loading	0,00	0,00	0,00	0,00
	Immediately after loading	0,89	1,16	1,99	2,20
	0,5	1,25	1,68	2,60	3,12
	1	1,41	1,85	3,23	4,08
	2	1,47	1,98	-	-
20/90	3	1,62	2,15	-	-
	4	2,09	2,34	-	-
	5	2,43	3,61	3,68	4,33
	10	2,85	4,44	4,14	6,28
	15	3,10	5,05	6,10	9,05
	20	4,12	6,59	6,42	9,32
60/40	25	4,27	6,84	6,84	9,78
	30	4,30	7,05	7,02	10,08
	35	4,25	7,08	7,12	10,17
20/90	40	4,25	7,17	7,19	10,44
	45	4,51	7,25	7,69	11,00
	50	4,57	7,40	7,95	11,42
60/40	55	4,60	7,43	8,23	11,87
	60	4,61	7,47	8,41	12,83
	65	4,60	7,46	8,68	12,60
20/90	70	4,58	7,46	8,92	13,19
	75	4,62	7,46	9,43	13,53
	80	4,69	7,47	9,93	13,80
60/40	85	4,71	7,52	10,40	14,06
	90	4,66	7,58	10,40	14,24
	95	4,60	7,63	10,64	14,07
20/90	100	4,57	7,66	10,93	14,05
	105	4,66	7,71	-	-
	110	4,90	7,79		
60/40	115	4,93	7,85		
	120	5,01	7,86		
	125	4,92	7,68		
20/90	130	4,88	7,64		
	135	4,99	7,65		
	140	5,10	7,73		
60/40	145	5,16	7,76		
	150	5,14	7,81		

to be continued on page 13

Table 4

continued from page 12

	155	5,13	7,82
20/90	160	5,14	7,84
	165	5,15	7,85
	170	5,15	7,87
60/40	175	5,27	7,89
	180	5,28	7,90

Table 5

Mean bending strenght in relation with time of loading and cyclic changes of climate

Load level in %	Bending strenght in MPa under long-term loading for a period of			Remarks
	2 months	4 months	6 months	
0	44,4	44,7	43,6	
15	41,3	40,5	39,3	
20	42,1	41,0	40,0	
25	no results have been obtained			2 specimens have failed after 10 days 4 specimens have failed after 85 days 6 specimens have failed after 100 days
30	no results have been obtained			all specimens have failed in the course of 10 -100 days

6.4.3. Interpretation of test results

The results corresponding to specimens tested in bending without loading under cyclic changes of climate exclusively, showed the strenght not different from the short-term one as given in table 1. The differences between the strenght of specimens tested under cyclic changes of climate for 2, 4 and 6 months are insignificant, for they are included in limit of error.

Comparing the results corresponding to the specimens tested in bending under cyclic changes of climate for six months with the ones corresponding to the specimens under load of 15% their strenght, 10% strenght increment has been observed. There have been no differences between 15% and 20% load levels. Due to a failure of specimens during the test the comparison between 25% and 30% load levels has been impossible.

From the results given in table 4 and from the curve in figure 7 one can deduce that deformation depends upon changes of climate and time of their influence, but always the load determine decisively the deformation value. During the tests carried out for 6 months at 15% and 20% load levels the specimens did not failed. Finally at 25% load level the specimens failed after 100 days and at 30% load level the first specimens failed after 10 days and all of them failed after 100 days.

To provide better information it is necessary to give stresses corresponding to load level:

- to a 15% load level corresponds 6,8 MPa stress,
- to a 20% load level corresponds 9,1 MPa stress,
- to a 25% load level corresponds 11,4 MPa stress,
- to a 30% load level corresponds 13,6 MPa stress.

Time-deformation functions / as shown in figure 7 - see annex 1/ in their initial portion are curves and in the next portion are practically straight-lined ,what proves stabilization of deformation velocity. From the curves one can deduce that deflection is in direct proportion with loading time.

For those results the equations of straight lines /after approximation of curves fiven in fig. 7/ are given for 6,8 MPa stress and 9,1 MPa stress respectively:

- for 6,8 MPa stress $y = 0{,}00722\ t + 4{,}1$
- for 9,1 MPa stress $y = 0{,}00611\ t + 7{,}0$

what illustrates the relation between deflection "y" and time of action of determined stress.

6.5. General interpretation of test results

From the tests above described one can deduce that the strenght of specimens under cyclic changes of climate is reduced in 10%. At 15% load level 30% strenght decrement has been observed in tension and 10% in bending. In bending under 20% load level the decrease of strenght could not be determined, due to excessive deformation /failure/ of specimens.

Comparing the standard strenghts which in accordance with Polish Standard PN-73/B-03150 /table 13/ for hardboard are as follows:
tensile strenght - 5,0 MPa bending strenght - 10,0 MPa
with the test results corresponding to determined levels, one can deduce that tensile strenght values are correct , whilst bending strenght differs.

At 20% load level, e.g. at 9,1 MPa the specimens in bending have been greatly deformed, but at level slightly lower than 11,4 MPa 50% of specimens have failed before six months have passed.

So it seems necessary to correct the standard value of bending strenght.

Considering the growing use of hardboard for wooden structures, it is necessary to calculate deflection of such structures taking into account the factor of time related with large deformations.

7. Conclusions

The conclusions drawn from the tests undertaken are as follows:

1. Strenght of hardboard submitted exclusively to the action of cyclic changes of climate showed only a small decrement during the time of their action.
2. The strenght under cyclic changes of climate and under different load levels has decreased considerably even at the lowest level applied, equal to 15% short-term strenght, at which 30% decrement occured in tension and 10% in bending.
3. According to the results obtained there is a necessity to correct standard strenght in PN-73/B-03150 /table 13/.
4. In design of wooden structures with hardboard is necessary to consider the factor of time in calculation of deflection.

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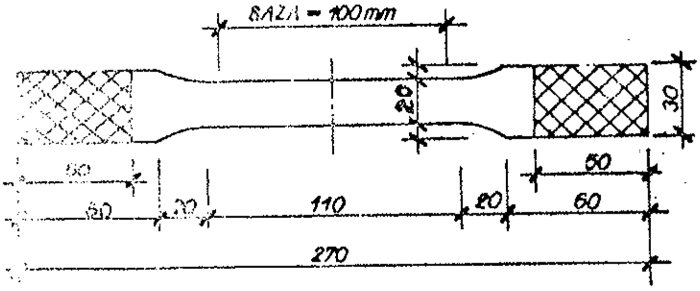


Fig. [illegible] Details of specimen for measurement of deformation in tension.

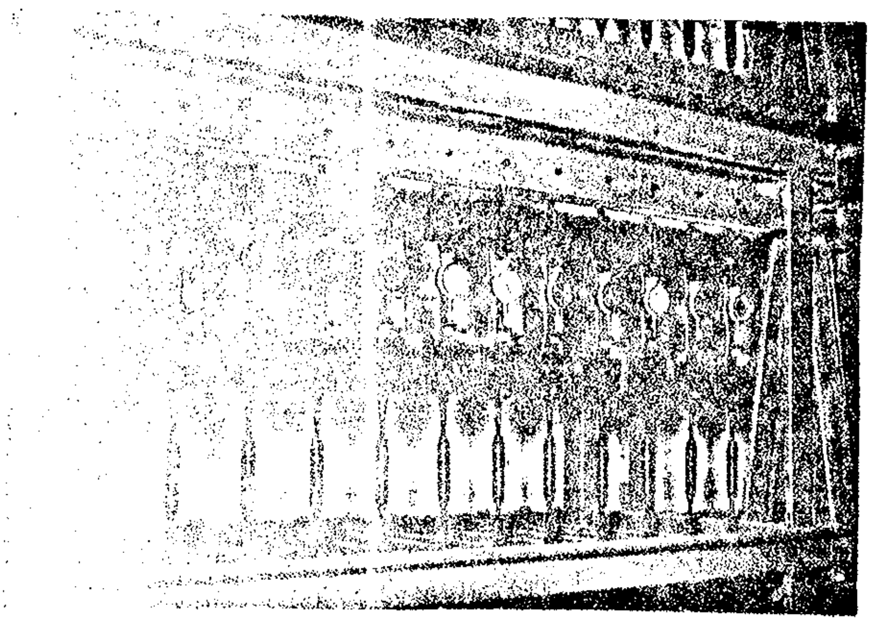


Fig. [illegible] Testing equipment for measurement of deformation in long-term tension.

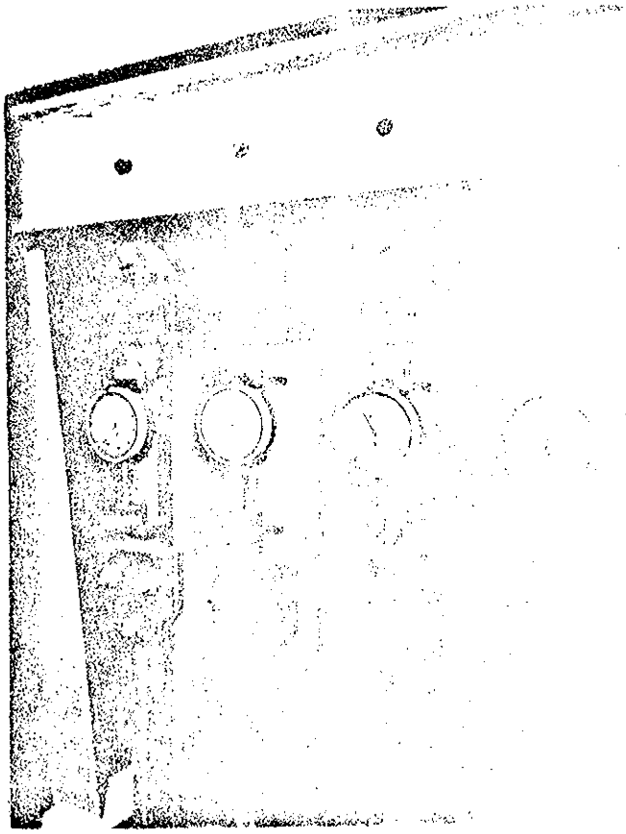


Fig.3 Specimens with gauges prepared for test.

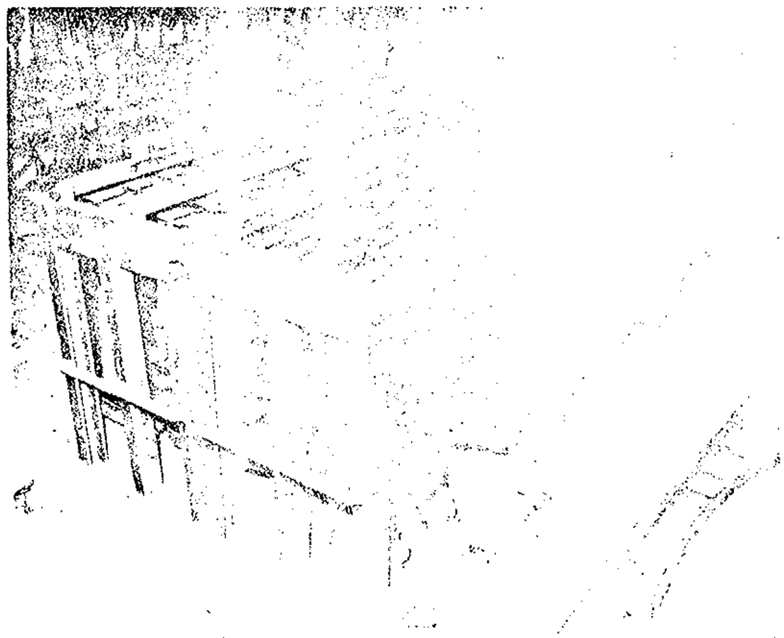


Fig.4 Equipment for measurement of deflection under long-term load in bending.

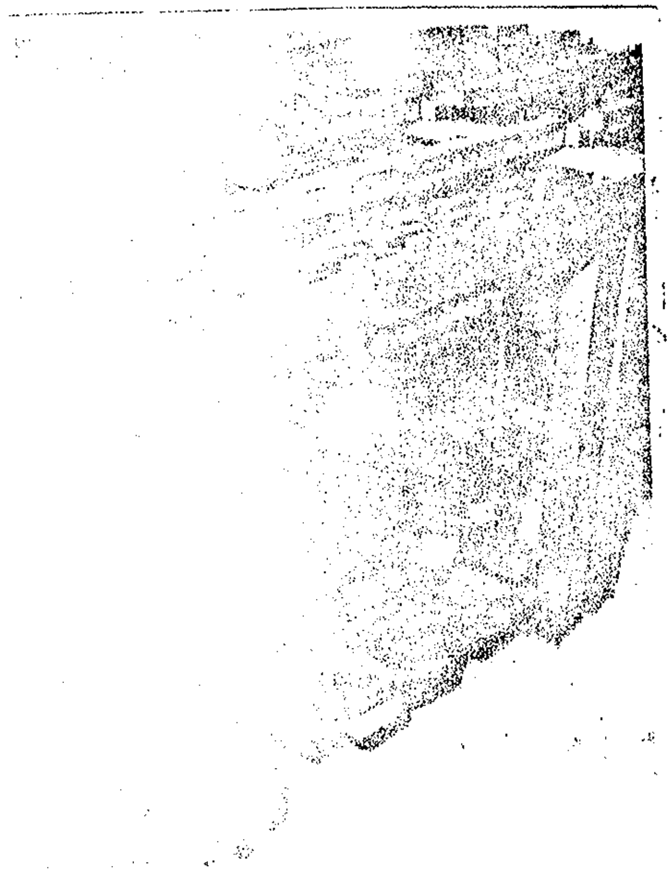
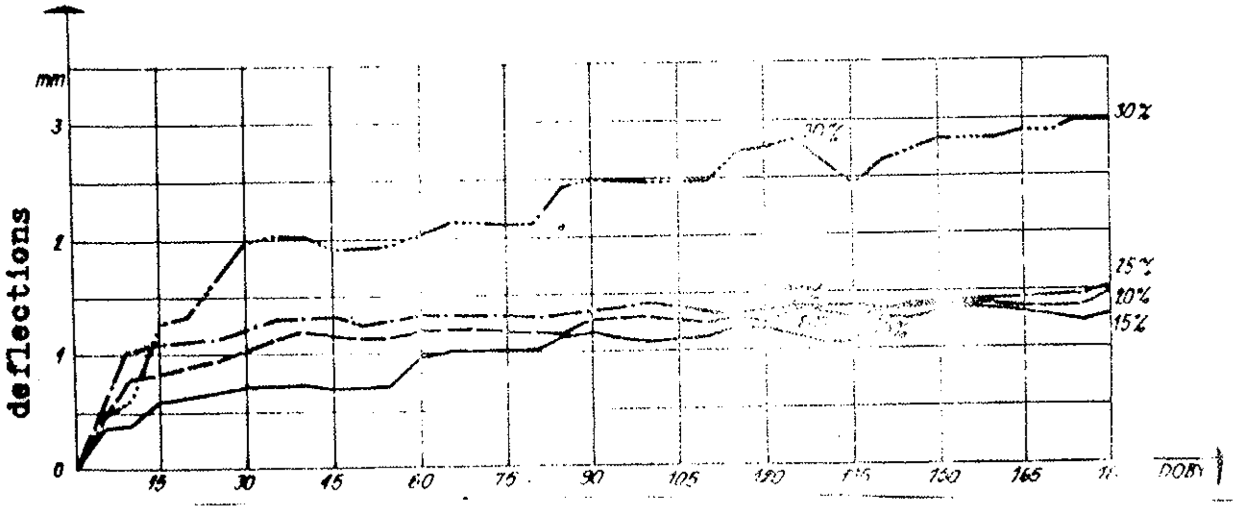
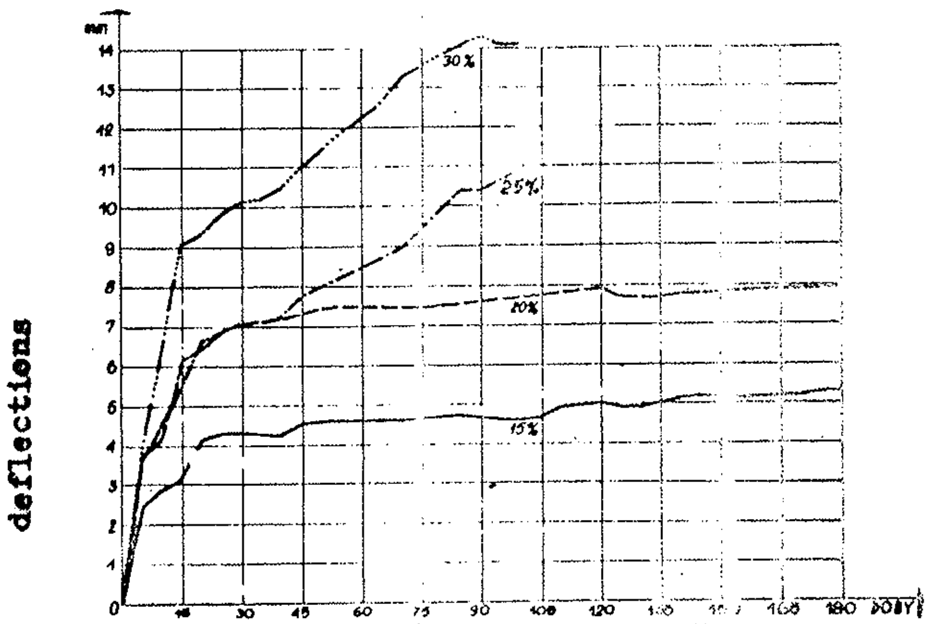


Fig. [illegible] Method of measuring deflection of specimens.



Time of load, d,

Fig.6 Time- deformation curves in tension for hardboard /T/.



Time of load, d,

Fig.7 Time- deflection curves in bending for hardboard under different load levels.

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WORKING COMMISSION W18 - TIMBER STRUCTURES

DESIGN OF METAL PLATE CONNECTED WOOD TRUSSES

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FOREWORD.

At the last meeting of CIB W 18 (Perth, Scotland June 1978) the second draft of the CIB Timber Code was discussed with respect to the section "Design of Timber Trusses". During the discussion of this section it was noted that the section "Trusses" was not sufficiently comprehensive to cover all necessary aspects in the design of timber trusses. I agreed to prepare some notes for discussion at the next meeting (Vienna, March 1979).

When working with truss design in various countries the immediate differences in the design rules do not seem significant, but a closer examination shows that the differences are in fact quite important.

They arise, however, from different ways of considering the same effect in the truss. The conclusion must be that most codes or recommendations could, with minor changes, form a special CIB Trussed Rafter Code, which in principle could be followed by all countries.

This paper is not a thorough investigation in differences in the national specifications, but only points out some of the significant differences.

INTRODUCTION.

The present design criteria for metal-plate-connected wood trusses in different countries have been developed and refined more or less according to the same principles during the last twenty years. During that period design methods have become more complex, and each country seems to have arrived at a different result, but basicly the same design philosophy is applied in the individual national design specifications. The present differences lie in the level of development reached rather than in the principles. It is correct to say that no national specification has reached a satisfactory level in the development of a truss design procedure. The level of sophistication is dependent on experience, available knowledge and analytical techniques. In general, the design of timber trusses according to different national design procedures results in quite different results assuming similar conditions for loads and material parameters.

This paper is meant as a registration of different national design rules applied to standard configuration timber trusses. The purpose is to reveal some of the variations, and it is hoped that, after discussion, it will result in a common procedure for a section in the CIB Timber Code, and a start to the necessary work on a separate Design Code for Trussed Rafters.

At present only a few special trussed rafter codes or recommendations exist. These are:

1. British Standard Instruction BS 5268: Part 3 THE STRUCTURAL USE OF TIMBER TRUSSED RAFTERS FOR ROOFS (Draft Oct 1977).

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.

In the following text these standards are referred to as BS 5268, CTB 111 and TPI 78.

BACKGROUND.

In general it is agreed that the design of even a simple timber truss is quite complicated if all the governing parameters for the real behaviour of the truss with respect to stiffness and strength are to be taken into account. Research, laboratory tests and field experience show that the present calculation methods, however complex they may be, cannot reproduce the real behaviour of the truss with respect to stiffness or strength. In short the methods do not give the right results.

The reason for this is the lack of relevant parameters for the calculations and the method of calculation which is based on the concept of the linear theory of elasticity which does not describe the limit state sufficiently accurately.

However, numerous existing trusses have been designed with very crude and simple methods and it has been proved that no serious incidences of failure due to design method alone have taken place.

The validity of the design is more a result of field experience and prototype testing than actual well-proven theoretical design calculations. In short, the design method has been calibrated to the experience.

This applies to the standard type of truss configurations w, ww--trusses, but today most countries are faced with a demand for individual trusses or other standards, such as the cantilever truss, where the existing procedures which were perfectly valid for w-trusses are either unsafe or grossly overdimensioned. Therefore there is a need for a general, accurate and efficient truss design method.

The establishment of such a design method requires the production of sufficient data to be used in a complex computerized structural analysis. Already developed computer programs from other fields of engineering are available to take any complexity of timber structure into account, but the lack of realistic strength and stiffness values for members and joints will at present make the use of advanced computer techniques in the design of timber trusses unrealistic.

Unfortunately the more complex calculation method often gives a too conservative design. The future research must therefore emphasize determination of the necessary strength and stiffness parameters to be used in the advanced computer programs. Furthermore investigation of the design criterion, the "interaction formula", must be carried out. This criterion applies reasonably well to an individual column but in a multicomponent structure conditions for columns are very different due to the interaction between the components.

REQUIREMENTS FOR A TRUSS DESIGN METHOD.

Experience and numerous prototype tests show that a fully developed and comprehensive design method for trussed rafters must take the following effects and aspects of the truss behaviour into consideration:

1. A timber truss is an indeterminate structure where interaction between components takes place dependent on the stiffness variation in the structural members.
2. An indeterminate structure is able to redistribute forces (stresses) because the ductility of timber (plasticity), enables the structure to carry load when parts of it are stressed beyond the yield stresses (ultimate stresses). The final collapse occurs when several parts of the structure have reached the ultimate stress level.
3. Agreement between the stress condition in the integrated structure and the stress used in the design formulas for the individual components (members and joints).
4. Agreement between the mathematical model used in the calculations and the actual physical structure.
5. Real eccentricities in the joints which means that the real load transfer point in joints must be considered. The forces (axial force and moment) in joints will naturally be transferred by wood to wood contact combined with forces in the plates (tension-compression forces and moments). This will in general cause considerably smaller eccentricities in joints compared to forces acting in theoretical centre lines.

6. Interaction between rigid or semi-rigid joints and structural members. Joint stiffness and plasticity have considerable influence on member forces and, for example, buckling length.
7. Variability of the stiffness and strength parameters of the structural components. In reality the structure consists of components with a variability of stiffness and strength scattered as random variables.The probability of the weakest strength being placed at the maximum stressed point must be taken into account.
8. Increased buckling stiffness relative to axial forces. In combined bending and compression the critical load increases according to the ratio between the axial stress and stresses induced by moments (the outermost fibers in the tension side are relaxed).

DESIGN PROCEDURES.

In the following sections some of the most significant design rules from different national design codes and recommendations are considered and commented upon. This description only considers ordinary standard configuration trusses; design of individual timber structures is not treated.

METHOD OF ANALYSIS.

At present only two methods of analysis exist: the pin-jointed and the frame analysis.

The most common method is the pin-jointed analysis. The structure is modelled as a statically determinate structure assuming the members to be pin-connected. Axial forces in truss members are determined explicitly from an equilibrium equation. Distributed loads on top and bottom chords are equivalent to concentrated loads applied at panel joints. For manual calculation and the computer analysis this is a very easy operation performed at low cost and with high accuracy. The simple mathematical model is shown in fig. 1.

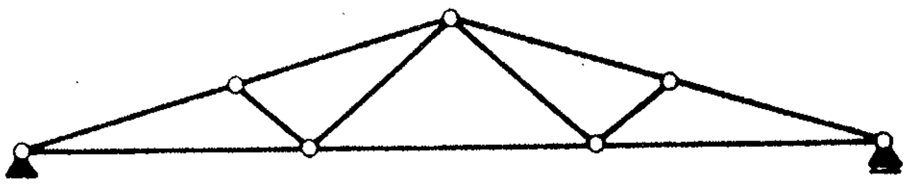


Fig. 1. Pin-joint model.

The moments are determined by moment coefficients depending on the type of loading and the number of panels normally based on an elastic solution for continuous beams over a fixed support.

This method is used in most countries except Norway where the trusses are considered as indeterminate structures. This method considers a mathematical model of the truss with the real stiffness variation of the members (constant along each member). The partly rigid joints are modelled as small beam elements with stiffness and length defined according to the physical model. Models in Norway only exist for w and ww trusses. The two models are shown in fig. 2.

Forces and deflections are determined by solving a great number of linear equations which is expensive and time-consuming on a computer and impossible by manual calculation. The models can be simplified, and investigation shows that the accuracy of the calculations is maintained.

In the following, the analysis as a non determinate structure is referred to as a frame analysis. The frame analysis is based on the theory of elasticity and produces a solution where the forces are in equilibrium. This type of solution will normally be an upper bound solution and thus be very conservative. Furthermore the moment distribution is very dependent on the mathematical model.

In modelling the truss the centre lines are normally used for the theoretical calculations. For the determination of axial forces in the pin-jointed analysis or the frame analysis the centre line model

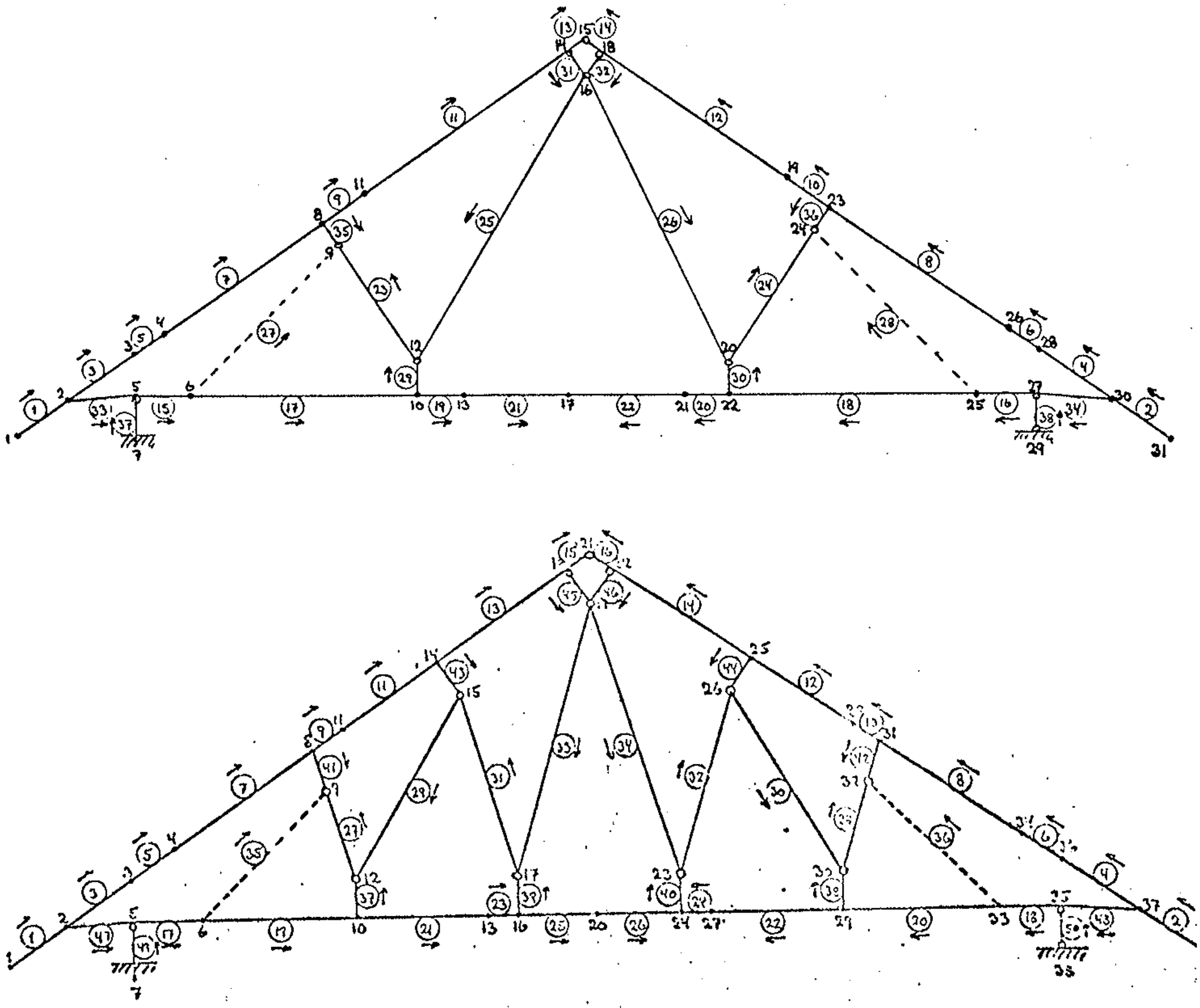


Fig. 2. Statically indeterminate models of
W and WW-trusses.

gives accurate results. For determination of moments in the pin-joint analysis the centre line model gives minor inaccuracies because the moment coefficients are fixed and only vary with the load and panel length, but for the frame analysis the real force path through the structure is very decisive especially in the joints. Therefore the real force transfer points must be considered in the frame analysis, and not the theoretical centre lines in which the resultant force is only situated in a long straight member.

In the frame analysis the moments are determined through the solution of the equilibrium equations. For the pin-joint analysis these are determined as described in the following section.

MOMENT ANALYSIS.

The critical top and bottom chord moments are determined by moment coefficients. As a general example a table from CTB 111 is shown in fig. 3.

Nombre de travées		0		1		2		3	
Réactions sous charge répartie R = Kpl	2	0,375		1,25					
	3	0,4		1,1					
	4	0,393		1,143		0,928			
	5	0,395		1,132		0,974			
	6	0,394		1,135		0,962		1,019	
	7	0,394		1,134		0,965		1,007	
Moment sous charge répartie M = Kpl²	2		+ 0,0703	− 0,1250					
	3		+ 0,0800	− 0,1	+ 0,0250				
	4		+ 0,0772	− 0,1071	+ 0,0364	− 0,0714			
	5		+ 0,0779	− 0,1053	+ 0,0332	− 0,0789	+ 0,0461		
	6		+ 0,0777	− 0,1058	+ 0,0340	− 0,0769	+ 0,0433	− 0,0865	
	7		+ 0,0778	− 0,1053	+ 0,0338	− 0,775	+ 0,0440	− 0,845	+ 0,0405
Déformation sous charge répartie $f = \frac{Kpl^4}{Ei}$	2		0,0052						
	3		0,00675		0,00052				
	4		0,0063		0,0019				
	5		0,0064		0,0015		0,0032		
	6		0,0064		0,0016		0,0028		
	7		0,0064		0,0016		0,0029		0,0024

Fig. 3. Coefficients for the pin-joint analysis (CTB 111).

Here the coefficients do not vary with respect to the slope of the truss. A theoretical frame analysis, however, gives a variation.

The TPI 78 gives a method for introducing moment coefficients where the coefficients are a function of the slope. Fig. 4 gives the necessary information where Q is defined from the equation $M = \frac{W(QL)^2}{8}$ and determined from the two tables.

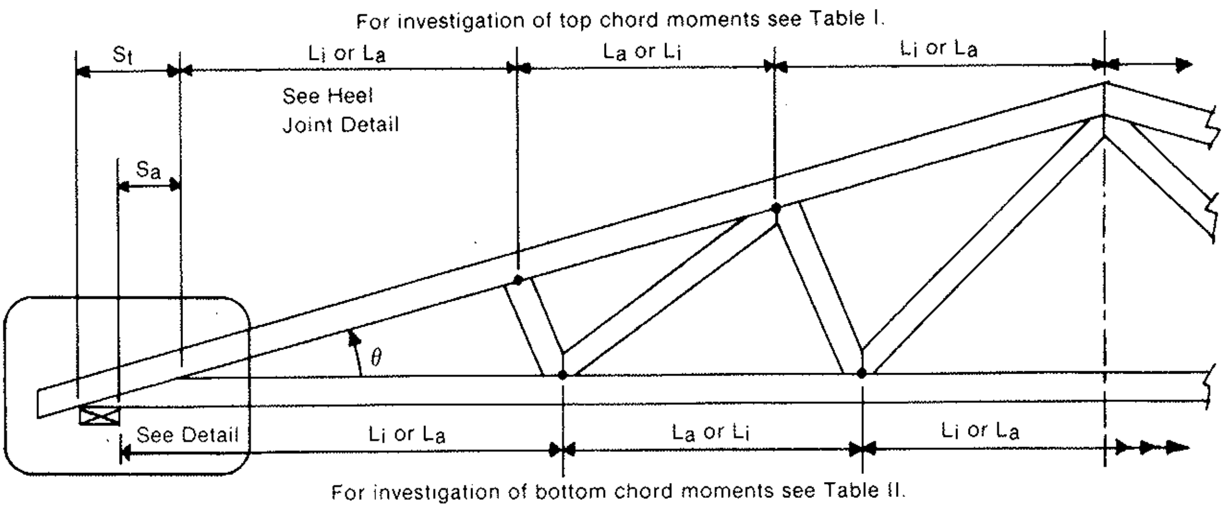


TABLE I

Panel Point Moment		Mid-Panel Moment	
Q	L	Q	L
One Panel Not Applicable	One Panel Not Applicable	One Panel 0.90	One Panel[3] $L_i + S_a$
Two Panels 0.90	Two or Three Panels[1] Largest of: $0.9\,L_i$, or $\frac{(L_i + L_a)}{2}$, or $0.9\,L_a$	Two Panels[2] $0.58(\cot\theta)^{0.23}$	Two or Three Panels[3,4] Largest of: $0.9(L_i + cS_a)$, or $\frac{(L_i + L_a)}{2} + cS_a$, or $0.9(L_a + cS_a)$
Three Panels 0.85		Three Panels[2] $0.53(\cot\theta)^{0.36}$	

[1] If S_t exceeds 24 inches, add excess to end (heel) panel L_i or L_a (see Figure 2).

[2] $Q = \alpha(\cot\theta)^{\beta}$ but shall not be less than 0.74; α and β are constants derived from PPSA analysis.

[3] $S_a = S_t - B$ but not less than zero. cS_a shall be added only to the length of the end (heel) panel.

[4] $c = 0.5$ for two panels; $c = 0.33$ for three panels; if neither L_i nor L_a are end (heel) panel lengths, then $cS_a = 0$.

TABLE II

Q	L
One Panel 1.0	One Panel L_i
Two or More Panels 1.0	Two or More Panels L is largest of: $0.9\,L_i$, or $\frac{(L_i + L_a)}{2}$, or $0.9\,L_a$

Fig. 4. Moment coefficients and panel length.

This rule is probably very reasonable and very easily applicable on the computer. The coefficients are calibrated from a frame analysis assuming particular mathematical models. Here it must be mentioned that the real moment distribution after redistribution of forces is considerably different from any elastic solutions.

BUCKLING ANALYSIS

The formulas used in the member design are quite similar in most national codes. Fig. 5 shows the formula for the buckling analysis.

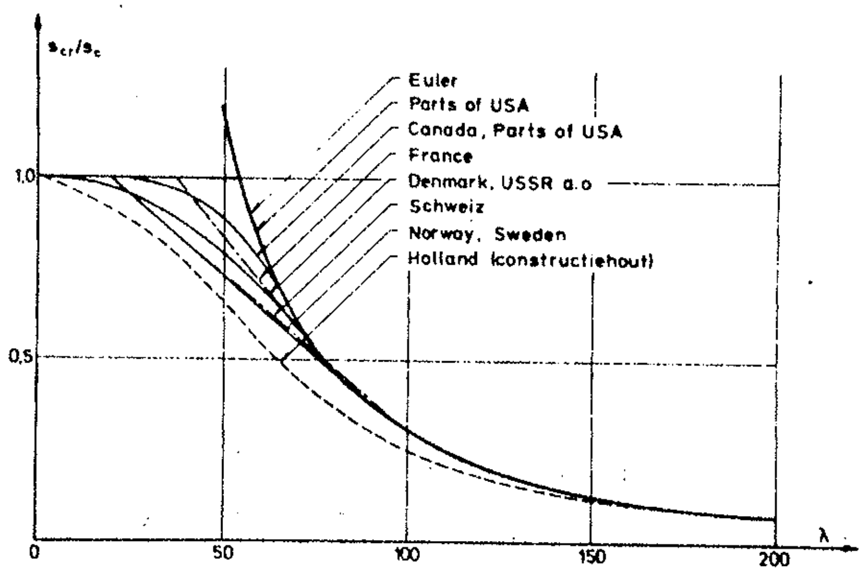


Fig. 5 National buckling formulas.

Despite the use of approximately the same buckling formula the effective buckling length is defined varying from the distance between panel point to 0.6 times this distance. In TPI78 the effective buckling length is taken from the table shown in fig. 4 where the effective length is $L' = QL \sec\Theta$.

In order to take the redistribution of forces into account, especially over panel points where peak moments arise and are often decisive in the interaction formula, a reduction of 20% is allowed in, for example, Sweden. This is counter-balanced by adding 20% to the bay moments. In many cases this effect is already taken into account in the recommended moment coefficients. In TPI78 a buckling stiffness factor C_T may be used to account for the increased chord buckling stiffness relative to axial forces. With certain limitations this applies to the buckling equations:

1. $11 < L'/d < K,$

$$K = 0.671\sqrt{\frac{C_T E}{F_c}}$$

$$F'_c = F_c[1 - \frac{1}{3}(\frac{L'/d}{K})^4]$$

2. $L'/d \geq K,$

$$F'_c = \frac{0.30 C_T E}{(L'/d)^2}$$

where in normal cases $C_T = 1+0.002L'$ but may not exeed 1.096.

BS 5268 gives the following formulas for calculation of reduction in intermediate supports

Case	Bending moments
One panel	$BM_B = + \frac{12\ E\ I\ \Delta}{S^2}$
Two panels	$BM_B = + \frac{EI}{S^2}(32.42\ \Delta_1 - 21.58\ \Delta_2)$ $BM_C = + \frac{EI}{S^2}(32.42\ \Delta_2 - 21.58\ \Delta_1)$
Three panels	$BM_B = + \frac{EI}{S^2}(58.28\ \Delta_1 - 41.14\ \Delta_2 + 10.29\ \Delta_3)$ $BM_C = + \frac{EI}{S^2}(-41.14\Delta_1 + 68.57\ \Delta_2 + 41.14\ \Delta_3)$ $BM_D = + \frac{EI}{S^2}(10.29\ \Delta_1 - 41.14\ \Delta_2 + 58.28\ \Delta_3)$

where

E is the mean Young's modulus (in N/mm^2);

I is the moment of area (in mm^4);

Δ is the relative deflection at node point (in mm);

S is the overall length of member (in mm).

JOINT DESIGN

Joint design rules in the different national specifications differ only in principle with respect to assimilation of moments. Most plate rules disregard calculation of nail- and plate stresses arising from moments. An exception is the support joint where a general reduction of plate and nail values is introduced. In BS 5268 the total number of effective nails (N) required for each side of the joint is calculated from the formula:

$$N = \frac{1}{V}(F + \frac{5M}{D})$$

where

F is the direct force in member x 2.5 (2.5 ultimate load factor);

M is the bending moment in member (P1/18 for point load only); as bending moment is for point load only no load factor is applied;

D is the depth of splice plate;

V is the permissible load per nail x 2.25 (2.25 ultimate load factor).

In CTB 11 the reduction scheme is:

Pente en degrés (α)	Coefficient de réduction
$\alpha \leqslant 15°$	0,85
$15° < \alpha \leqslant 18°$	0,80
$18° < \alpha \leqslant 22°$	0,75
$22° < \alpha \leqslant 25°$	0,70
$\alpha > 25°$	0,65

In most German plate permissions (Zulassung) the reduction scheme is as shown below:

Dachneigung	Abminderungsfaktor η
$\leqq 15^\circ$	0,85
$15^\circ \leqq 18^\circ$	0,80
$18^\circ \leqq 22^\circ$	0,75
$22^\circ \leqq 25^\circ$	0,70
$> 25^\circ$	0,65

In Denmark and Norway the stresses from moments are checked from the formula

$$\tau_m = \frac{M}{J_p} \cdot r_{max}$$

where J_p is the polar moment of inertia from each plate part and r_{max} is the maximum distance from the centre of gravity to the outermost placed nail for the plate part considered.

The latter rule has caused enormous trouble in the plate dimensioning and has never been proved accurate.

It should be noticed that BS 5268 is the first code to allow plasticity in the joint design.

For most national design procedures (except Norway) it is not recognized in the design of the members that the joints are designed to transfer moments.

DEFLECTION ANALYSIS

The traditional deflection analysis based on the pin-joint model where only axial strain is taken into account will only produce an unrealistic deflection. The slip in the joint can easily be included and CTB 111 recommend the following expression:

$$f = \frac{\Sigma\Delta e + \Sigma g}{tg\,\alpha}$$

where f is the deflection in a point, Δe is the elongation of a member, g is the slip in a joint and α is the roof slope.

BS 5268 recommends calculating the deflection Δ by the formula

$$\Delta = \Sigma\frac{FKL}{AE} + \Sigma Kx$$

where

- F is the force in the member (in N);
- K is the force in the member due to a unit load applied at the node point whose deflection is to be calculated (in N);
- L is the length of the member (in mm);

A is the cross-sectional area of the member (in mm^2);

E is the mean Young's modulus (in N/mm^2);

X is the slip of the nail or tooth at full design load (in mm).

None of the formulas consider the contribution to the deflection from the moments and the rigidity of the joints. The calculated deflection will be fictitious and can only be calculated realistically by a frame analysis.

PROTOTYPE TESTING

Load test of full size trusses is accepted as a method for the design of structural performance. Prototype-testing has proved to be a very efficient way to obtain an optimum structure meeting the functional requirements with respect to stiffness and factor of safety under service. From an economic point of view the method obviously has limited possibilities.

In the evaluation of test procedures in different specifications the national safety system influences the required factor of safety, but by comparing the factor of safety for similar load conditions huge differences appear.

For strength tests BS 5268 requires a factor of safety on full design load of 2.5 for one truss. For a number of trusses the load factors are given as in this table:

Number of units	Load factor
2	2.30
3	2.15
4	2.05
5 and more	2.00

In CTB 111 three trusses must be tested and the factor of safety for full design load is required to be $\geqq$ 2.5 for the mean value and $\geqq$ 2.2 for the minimum value TPI 78 requires a factor of safety corresponding to dead load plus 2.5 times the design live load.

In Sweden the factor of safety for five tests will approximately be 3.0 for a normal ratio between the dead and live load.

CONCLUSION

There are two different demands to be met in a CIB timber code as well as in most national design specifications for truss design.

A. A simplified efficient design procedure which produces designs for ordinary standard truss configurations, which can be satisfactorily compared with those produced by using sophisticated, computerized analytical methods.

B. A sophisticated computerized method which produces accurate design for any timber truss, taking all significant effects from the truss behaviour into account.

The sophisticated method should be used to generate limiting span tables for standard configurations. These tables may be available in a convenient computer system, where the information is stored direct or generated by interpolation routines. In this way costly computer runs are only performed once.

The simplified analysis could be programmed and used in a computerized system, but this will normally be an expensive calculation when used repeatedly without using the result again.

Design of timber trusses involves such a tremendous amount of rules and information that it is suggested that the design specifications for trusses be placed in a separate CIB-TIMBER TRUSSES CODE.

CIB-W18/11-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB STRUCTURAL TIMBER DESIGN CODE

(Third Draft)

VIENNA, AUSTRIA

MARCH 1979

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

WORKING GROUP W18
TIMBER STRUCTURES

CIB
STRUCTURAL TIMBER DESIGN CODE

Third draft, September 1978

FOREWORD

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

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

A second draft of the code was prepared by G. Booth, H. J. Larsen and J. R. Tory. This was discussed, except for Chapter 6, Mechanical Fasteners, at the CIB-W18-meeting in June 1978 and it was agreed that a third draft should be sent out for comments, among others from the members of ISO/TC 165-Timber Structures.

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

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

Related Documents

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

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

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

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

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

LIST OF CONTENTS

The background for the CIB-Structural Timber Design Code is technical papers prepared for and discussed at meetings in CIB-W18 Timber Structures.

The papers are identified by CIB-W18 and a number a-b-c:

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

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

b denotes the subject by the following numerical classification:

1.	Limit State Design	12.	Laminated Members
2.	Timber Columns	13.	Particle and Fibre Building Boards
3.	Symbols	14.	Trussed Rafters
4.	Plywood	15.	Structural Stability
5.	Stress Grading	100.	CIB Timber Code
6.	Stresses for Solid Timber	101.	Loading Codes
7.	Timber Joints and Fasteners	102.	Structural Design Codes
8.	Load Sharing	103.	International Standards Organisation
9.	Duration of Load	104.	Joint Committee on Structural Safety
10.	Timber Beams	105.	CIB Programme, Policy and Meetings
11.	Environmental Conditions	106.	International Union of Forestry Research Organisations

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

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

A current list of CIB-W18 technical papers is given as an Annex to this code.

SECRETARY'S NOTE

To avoid unnecessary duplication, the list of technical papers that formed an Annex to this document has not been reproduced. An up-dated list may be found earlier in these proceedings.

1. INTRODUCTION

1.1 Scope

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

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

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

Deviations from the requirements of this code and the use of materials and methods of design or construction of wood structures not covered by this code are permitted when the validity is substantiated by analytical and engineering principles or reliable test data, or both.

1.2 Conditions for the validity of this document

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

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

1.3 Units

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

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

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

Table 1.3 Units for structural timber design

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

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

1.4 Notations

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

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

The following general terms and symbols are used. Symbols which are not explained here are defined when used. Attention is drawn to the special notations used in Annex 7A - B - C.

Main symbols

A	Area
E	Modulus of elasticity
F	Force
G	Shear modulus
I	Second moment of area (moment of inertia)
M	Moment, unless otherwise stated Bending moment
N	Axial force
V	Shear force Volume
a	Distance
b	Width
d	Diameter Side length for square nails
e	Eccentricity
f	Strength
h	Depth of beam
k	Factor, always with a subscript
ℓ	Span Length
r	Radius
t	Thickness
x y z	Coordinates
α	Angle
β	Factor
η	Factor
κ	Factor
λ	Slenderness ratio
μ	Ratio
ν	Poisson's ratio
ρ	Specific gravity
σ	Normal stress
τ	Shear stress
φ	Ratio

Subscripts

apex	At apex
axial	Axial
bearing	Bearing
bolt	Bolt
buck	Buckling
c	Compression
col	Column
con	Connector
crit	Critical
curv	Curvature
depth	Depth
E	Euler
e	Effective
f	Flange
head	Head (nail)
i	Inner
inst	Instability
m	Bending
max	Maximum
mean	Mean value
min	Minimum
nail	Nail
o	Outer
size	Size
t	Tension
tang	Tangency
thread	Threaded
tor	Torsion
v	Shear
w	Web
x	Related to the x-direction
y	Related to the y-direction Yield

Numbers 1, 2, . . . are used. The following have a special meaning:

0	In the fibre direction, parallel to grain
90	Perpendicular to the fibre direction, perpendicular to grain

1.5 Definitions (will be prepared at completion of the work)

The climate class grading is based on CIB-W18/5-11-1 with small changes as motivated in 6-11-1.

The load-duration grading is based on CIB-W18/3-9-1 and especially 7-9-1.

2. BASIC ASSUMPTIONS

2.1 Characteristic values and mean values

2.1.1 Characteristic values

The characteristic strength and stiffness values given in this code for timber and wood-based materials are defined as lower 5-percentile values (i.e. 95% of all possible test results exceed the characteristic value) directly applicable to a very short-term load condition (3 to 5 mins.) at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02. Where the characteristic values are estimated from a limited number of tests the estimate shall be made with a confidence level of 0.75.

The characteristic strength values are related also to

- a section depth of 200 mm for the bending strength of solid timber,
- a section depth of 300 mm for the bending strength of glued laminated timber,
- a volume of 0.02 m^3 for the tensile strength perpendicular to grain.

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

2.1.2 Mean values

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

2.2 Climate classes

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

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

Climate class 1

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

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

Climate class 2

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

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

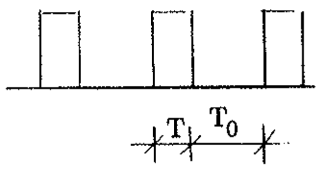
Climate class 3

All other climatic conditions.

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

2.3 Load-duration classes

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



The load-duration class for a given load (specified by its time spectrum) depends on the properties of the material or the whole structure. For the intermittent load shown the effective loading time is equal to T if T_0 is long as compared to the recovery time for the material, and equal to the accumulated loading time if there either is no recovery or if T_0 is comparatively short.

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

Table 2.3a Load-duration classes

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

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

Table 2.3b Examples of action classifications

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

The standard strength classes are introduced in CIB-W18/9-6-1.

REQUIREMENTS FOR MATERIALS

4.0 General

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

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

4.1 Solid structural timber

4.1.0 General

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

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

Strength and stiffness parameters shall be determined by standardized short-term tests in accordance with ISO/TC 165: Timber structures - Timber in structural sizes - Determination of some physical and mechanical properties. The test specimens shall contain a grade-determining defect - preferably knots - in the zone with maximum force or bending moment.

4.1.1 Standard strength classes

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

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

Table 4.1.1 Standard strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

standard strength class		SC15	SC19	SC24	SC30
bending	f_m	15	19	24	30
bending	$E_{0,mean}$	6000	7200	8500	10000
tension	$f_{t,0}$	6	9	16	20

: It is emphasized that the introduction of standard strength classes does not prevent the introduction of other grades
: with for example higher values for $E_{0,mean}/f_m$ and $f_{t,0}/f_m$.
: European redwood/whitewood graded according to UN/ECE Recommended standard for stress grading of coniferous
: sawn timber (UN/ECE TIM/WP.3/AC3/B-Annex I) can be assumed to meet the demands of the following standard
: grades: ECE6 : SC19 - ECE8 : SC24 - ECE10 : SC30.
: Annex 4 contains a survey of which national grades can be assumed to satisfy the requirements of the different
: standard grades. (Under preparation).

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

The requirements apply to the glulam, not to the laminae. The additional tensile strength requirement for glulam made from more than one species is necessary to ensure a normal property-profile. The introduction of standard glulam strength classes does not prevent the introduction of other grades, e.g. grades with a higher $f_m/f_{t,0}$-ratio by using high-strength wood in the uttermost laminae.

4.2 Finger jointed structural timber

4.2.0 General

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

Strength and stiffness parameters shall be determined according to section 4.1.0 coupled with the rules in $E_{0,mean}$ and the characteristic values for f_m and $f_{t,0}$ are not less than given in table 4.1.1.

4.2.1 Standard structural classes

Finger jointed structural timber can be referred to one of the standard strength classes stated in 4.1.1 if $E_{0,mean}$ and the characteristic values are not less than given in table 4.1.1.

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

4.3 Glued laminated timber

4.3.0 General

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

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

4.3.1 Standard glulam strength classes

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

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

Table . .1 Standard glulam strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

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

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

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

4.4 Wood-based sheet materials

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

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

For particle board and fibre board:

4.5 Glue

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

4.6 Mechanical fasteners

Refer to chapter 6.

4.7 Steel parts

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

Table 4.7 Minimum protection against corrosion

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

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

The modification factors correspond to the traditionally used reduction factors of $9/16 \sim 0.6$ for long-term load and 0.85 for exterior conditions. The reductions are probably less, especially for the low grades.

5. DESIGN OF BASIC MEMBERS

5.1 Solid timber members

5.1.0 Characteristic values

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

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

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

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

Table 5.1.0 b Modification factors *Provisional*

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

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

Values in parantheses apply to tension perpendicular to grain.

5.1.1 Tension members, compression members, beams and columns

5.1.1.0 General

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

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

See footnote on page 4.1.

In cases where the influence of the size can be disregarded the conditions (5.1.1.1 a) and (5.1.1.1 b) can be generalized to an arbitrary angle, α, between stress and grain direction, viz.:

$$\sigma_t \leqslant \frac{1}{\sqrt{\left(\frac{\cos^2\alpha}{f_{t,0}}\right)^2 + \left(\frac{\sin^2\alpha}{f_{t,90}}\right)^2 + \left(\frac{\sin\alpha\cos\alpha}{f_v}\right)^2}}$$

cf. formula (5.1.1.6 a).

The introduction of size factor for the two directions of practical interest has been found more important than having a general formula.

The values for $k_{size,90}$ are based on papers by J. D. Barrett (Wood and Fiber, Vol. 6, No. 2, 1974 and Canadian Journal of Civil Engineering, Vol. 2, 1975).

Together with the Hankinson formula this formula has been used for many years in many codes for designing traditional timber structures. No need has been felt for replacing it with a more sophisticated and more restrictive expression based on formula (5.1.1.6 a). The formula has been chosen in preference to the Hankinson formula due to its simplicity.

$k_{bearing}$ is based on a comparison of the rules in a number of codes, cf. CIB-W18/5-10-1, and on the work of G. Backsell (Swedish Institute for Building research, Report 12/66 1966).

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

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary for nails and screws with a diameter of 5 mm or less.

5.1.1.1 Tension

The stresses shall satisfy the following conditions:

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

for tension parallel to the grain direction, and

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

for tension perpendicular to the grain, and

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

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

: Recommendations on the size factor $k_{size,0}$ will be produced.

5.1.1.2 Compression without column effect

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

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

cf. fig. 5.1.1.2 a.

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

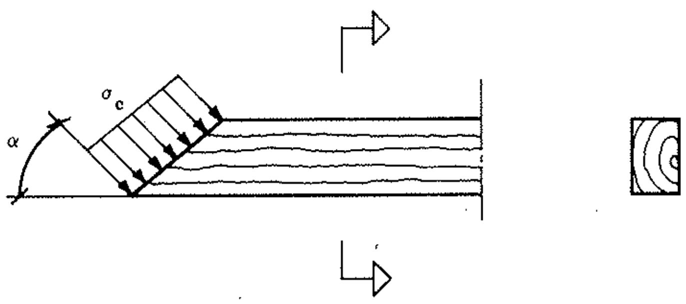


Fig. 5.1.1.2 a

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

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

A depth effect can be counteracted by stricter knot limitations for the bigger sizes. This may be the reason for the different traditions concerning depth factor, and it explains why it is impossible to give a general expression in this code.

The limiting depth of 200 mm has been chosen on the basis of tradition in the countries where such a factor has been used.

At present the depth dependence for the ECE-grades is investigated. A provisional value of $\kappa = 1/9$ has been suggested.

Reference is also made to the survey in CIB-W18/5-10-1.

The rules concerning lateral instability are based on the work of Hooley & Madsen (ASCE Journal of the Structural Division, Vol. 70 (1964) ST 3) as described in CIB-W18/5-10-1.

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

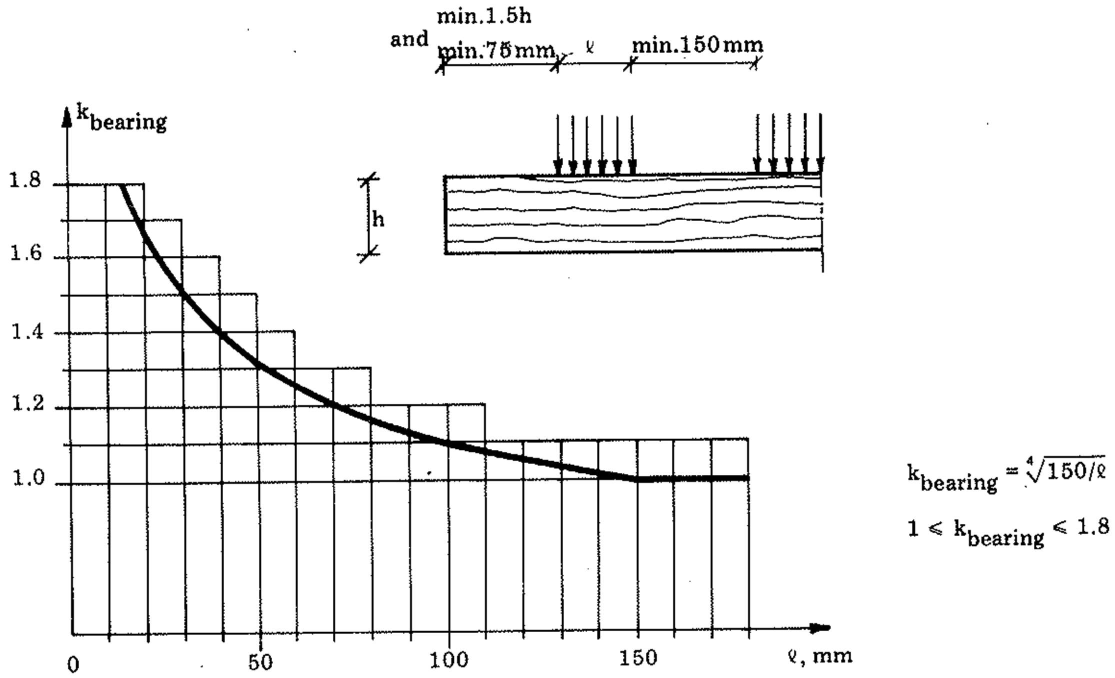


Fig. 5.1.1.2 b

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

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

5.1.1.3 Bending

The bending stresses, σ_m, calculated according to the theory of elasticity shall satisfy

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

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

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

The value of κ depends on among other things the wood species and the grading rules. Recommendations will be produced.

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

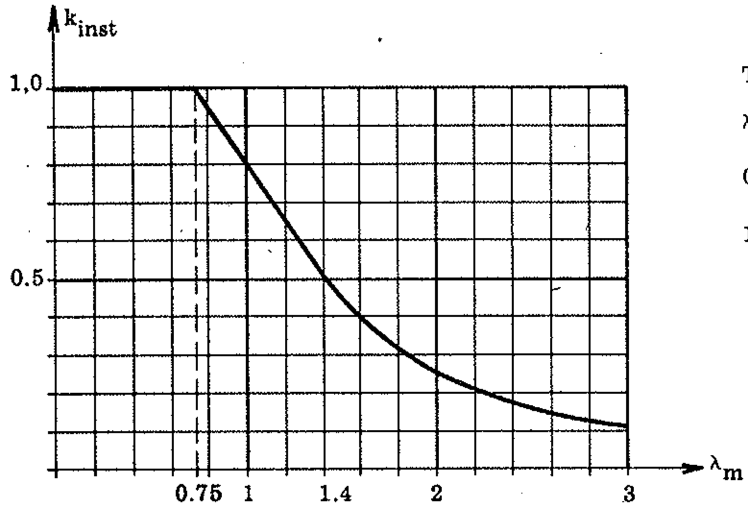
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

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

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

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

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



The curve corresponds to

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

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

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m^2$

Fig. 5.1.1.3

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

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

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

The free length is determined as follows:

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

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

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

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

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

The reduction of the load near the supports is discussed in CIB-W18/9-10-1.

5.1.1.4 Shear

The shear stresses, τ, calculated according to the theory of elasticity shall satisfy the following condition

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

For beams with bearing in the bottom side and load on the top, loads placed nearer than the beam depth from the support may be disregarded in calculation of the shear force.

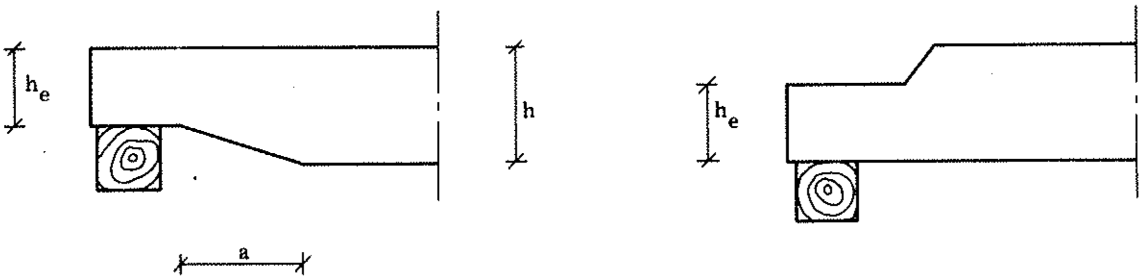


Fig. 5.1.1.4

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

$$\tau \leqslant \left(\frac{h_e}{h} + \frac{a}{3h}\right) f_v \qquad (5.1.1.4\ b)$$

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

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

5.1.1.5 Torsion

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

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

5.1.1.6 Combined stresses

General

At present no general theory of rupture exists, but only empirical or semi-empirical expressions for the most important practical cases, some of which are given below.

Plane stress

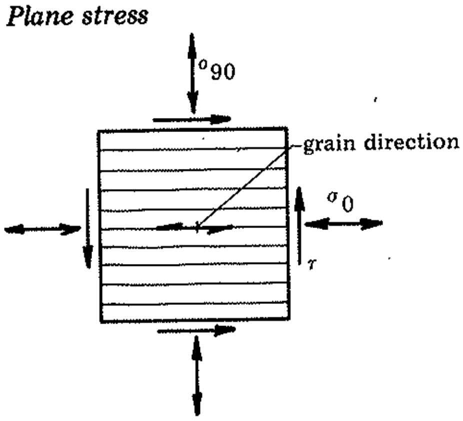
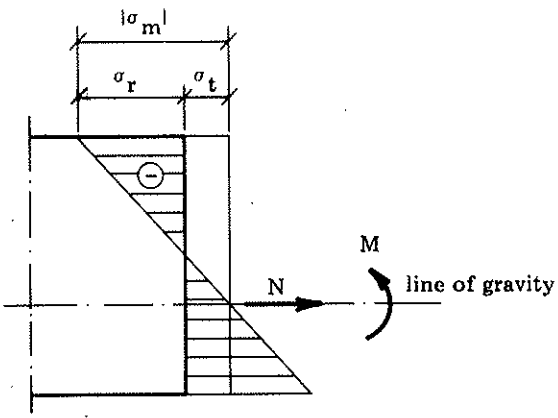


Fig. 5.1.1.6

The interaction formula is empirical and suggested by Norris, cf. Forest Products Laboratory, Madison, Report 1816, 1962. A more complicated formula

$$\left(\frac{\sigma_0}{f_0}\right)^2 - \frac{\sigma_0 \sigma_{90}}{f_0 f_{90}} + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leqslant 1$$

given in the report mentioned is not supported by the tests described in CIB-W18/9-6-4.



Formula (5.1.1.6c) (and (5.1.1.6e)) ensure that the resultant stress, σ_r, does not exceed f_m, which could be the case for unsymmetrical cross-sections if only (5.1.1.6b) (or (5.1.1.6d)) was used, since

$$\frac{\sigma_r}{f_m} = \frac{|\sigma_m|}{f_m} - \frac{\sigma_t}{f_m} = 1 + \frac{\sigma_t}{f_{t,0}} - \frac{\sigma_t}{f_m} \geqslant 1$$

for $f_{t,0} < f_m$.
See also CIB-W18/4-10-2.

(5.1.1.6f) is based on a paper by Möhler & Hemmer (Holz als Roh- und Werkstoff 35(1977)473-478).

The background for the column design is given in CIB-W18/2-2-1. The code format has emerged from discussion in CIB-W18 and the work in connection with a revision of the British Code of Practice for The Structural Use of Timber.

The stresses shown in fig. 5.1.1.6 should - unless otherwise stated (see e.g. 5.1.1.2a) - satisfy the following condition:

$$\left(\frac{\sigma_0}{f_0}\right)^2 + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leqslant 1 \qquad (5.1.1.6\ a)$$

f_0 and f_{90} are chosen according to the sign of the σ_0 and σ_{90}, respectively. If σ_0 is a bending stress then $f_0 = f_m$.

Tension and bending
Only the case with tension in the grain direction is considered.

The stresses should satisfy the following condition

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

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

$$|\sigma_m| - \sigma_t \leqslant f_m \qquad (5.1.1.6\ c)$$

Compression and bending without column effect
Only the case with compression in the grain direction is considered.

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

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

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

$$\sigma_m + \sigma_c \leqslant f_m \qquad (5.1.1.6\ e)$$

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

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

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

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

These conditions can be assumed satisfied if the stresses satisfy the following condition:

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

σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_{col} and k_E are factors depending on the slenderness ratio λ, the material parameters and the initial curvature.

The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

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

where r_{core} is the core radius.

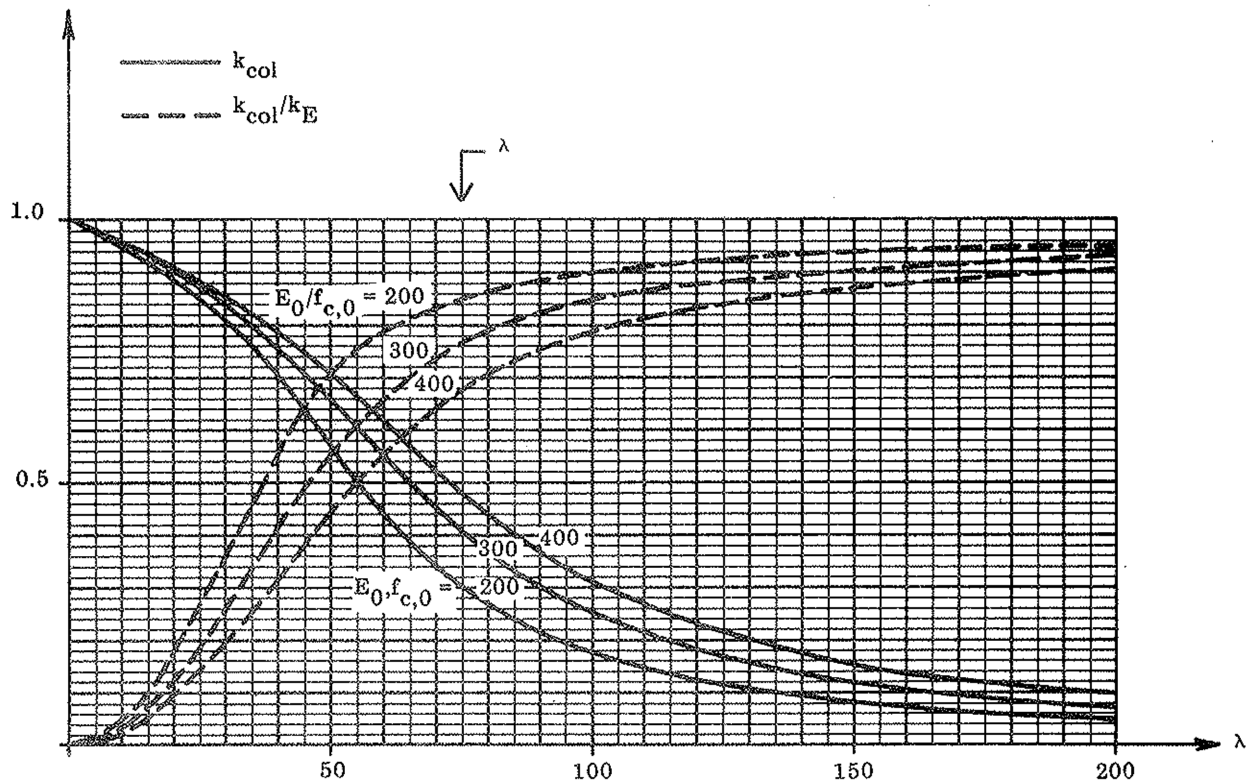
$$k_E = \frac{\sigma_E}{f_{c,0}} = \frac{\pi^2 E_0}{f_{c,0}\lambda^2} \qquad (5.1.1.7\ c)$$

where E_0 is the characteristic value of modulus of elasticity

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

σ_E is the Euler stress.

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



: Fig. 5.1.1.7

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

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

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

Table 5.1.1.7 Relative effective length of compression members

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

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

5.1.2 Cambered beams

Relevant parts of section 5.2.2 may be applied.

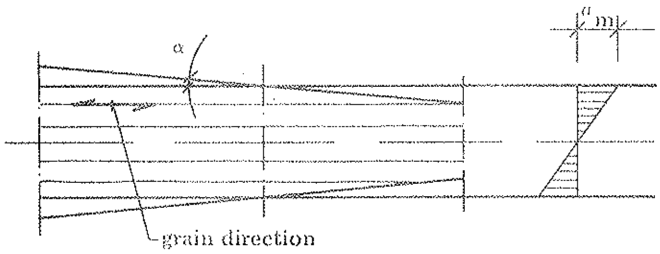


Fig. 5.1.2

Where there is an angle α between the grain direction and the top or bottom of the beam the bending stresses, σ_m, calculated as usual should in accordance with formula (5.1.1.6a) satisfy the following conditions:

Tension side:

$$\frac{\sigma_m}{f_m} \leqslant \frac{1}{\sqrt{1 + \left(\frac{f_m}{f_v} \tan\alpha\right)^2 + \left(\frac{f_m}{f_{t,90}} \tan^2\alpha\right)^2}} \qquad (5.1.2a)$$

Compression side:

$$\frac{|\sigma_m|}{f_m} \leqslant \frac{1}{\sqrt{1 + \left(\frac{f_m}{f_v} \tan\alpha\right)^2 + \left(\frac{f_m}{f_{c,90}} \tan^2\alpha\right)^2}} \qquad (5.1.2b)$$

5.2 Glued laminated members

5.2.0 Characteristic strength and stiffness values

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

Table 5.2.0 Characteristic values and mean elastic moduli, in MPa *Provisional*

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

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

See footnote on page 4.2.

The influence of notches has been found much more severe for glulam than for solid timber, and in some countries notching of glulam is not allowed. The expression har been proposed by Möhler, see CIB-W18/9-6-4.

The rules are in principle the same as in the Canadian Standard CSA-086/1977, but slightly simplified.

5.2.1 Straight beams and columns

Section 5.1.1 for solid timber applies except that formula (5.1.1.3b) should be replaced by

$$k_{depth} = \begin{cases} 1 & \text{for} \quad h \leqslant 300 \text{ mm} \\ (\frac{300}{h})^{\kappa} & \text{for} \quad h > 300 \text{ mm} \end{cases} \qquad (5.2.1a)$$

and formula (5.1.1.4b) by

$$\tau \leqslant [1 - 2.8\,\frac{h - h_e}{h}(1 - \frac{a}{14(h - h_e)})]f_v \qquad (5.2.1b)$$

and notches with $h_e < 0.75h$ are not allowed.

5.2.2 Cambered beams

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

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

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

with

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

that the tensile stresses perpendicularly to the grain satisfy the condition 5.1.1.1 b, i.e.

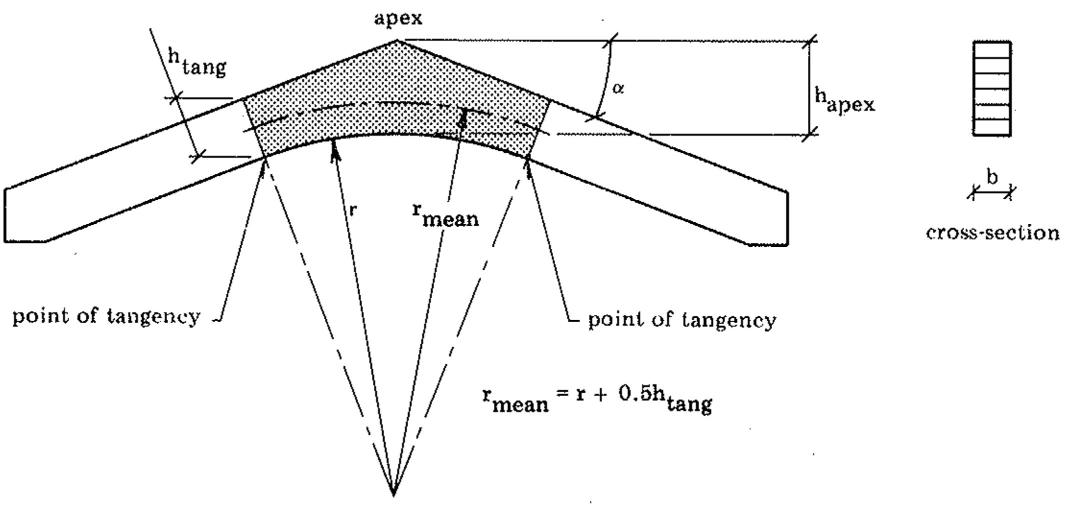


Fig. 5.2.2 a Double tapered curved beam

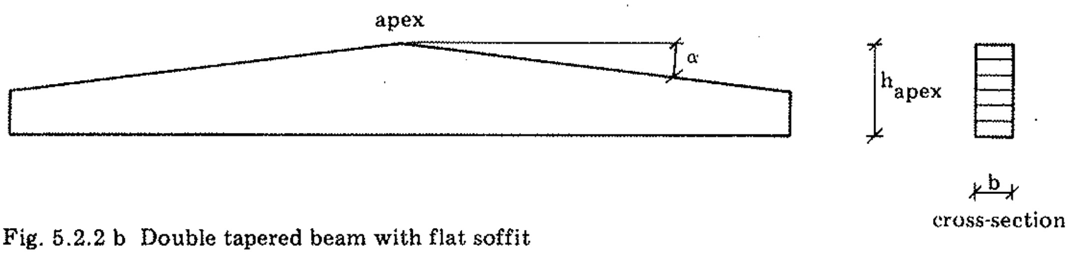


Fig. 5.2.2 b Double tapered beam with flat soffit

The background for the calculation of radial tensile stresses etc. is given by Foschi & Fox (See e.g. ASCE, Journal of the Structural Division, Vol. 76(1970) ST10.

The formula and the diagrams are based on papers by H. Blumer (among others Holzbau 6-8/1975) and Möhler & Blumer (Berichte aus der Bauforschung, Berlin 1974, Nr. 92).

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

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

Where there is an angle between the grain direction and the top or bottom the bending stress should satisfy the conditions in section 5.1.2.

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

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

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

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

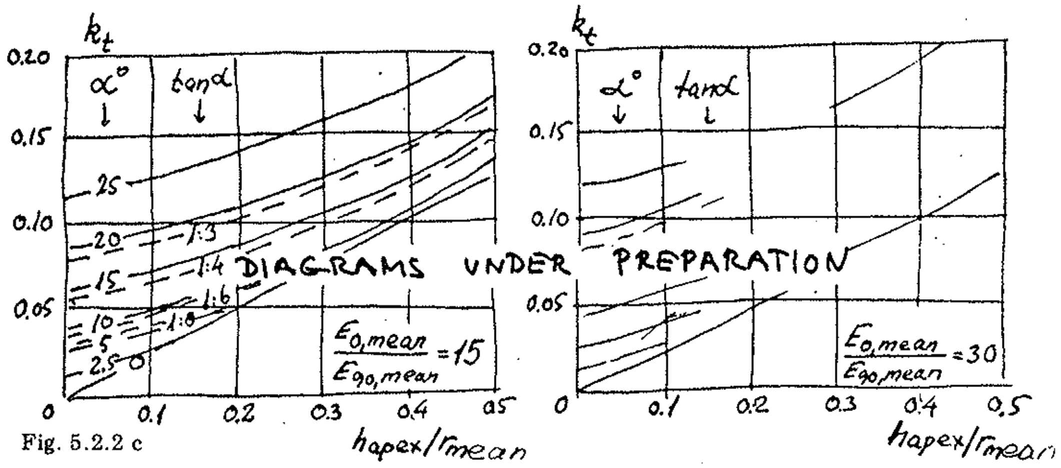


Fig. 5.2.2 c

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

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

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

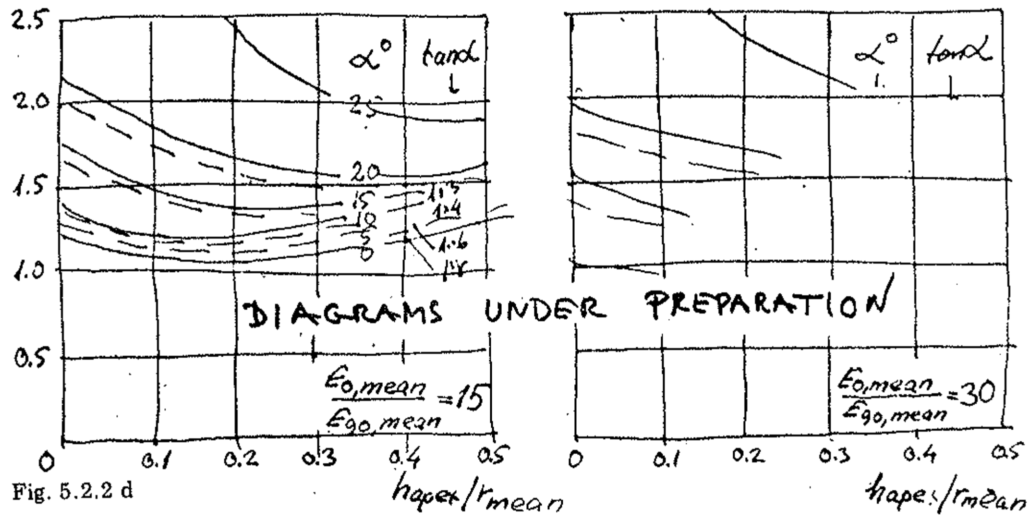


Fig. 5.2.2 d

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

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

The formula is a simplification of the Wilson- and Hudson-formulas, see CIB-W18/5-10-1.

The values are suggested by K. Möhler, cf. CIB-W18/5-10-1.

5.2.3 Curved beams

This section applies to curve~~s~~d beams with constant, rectangular cross-section, see fig. 5.2.3 a.

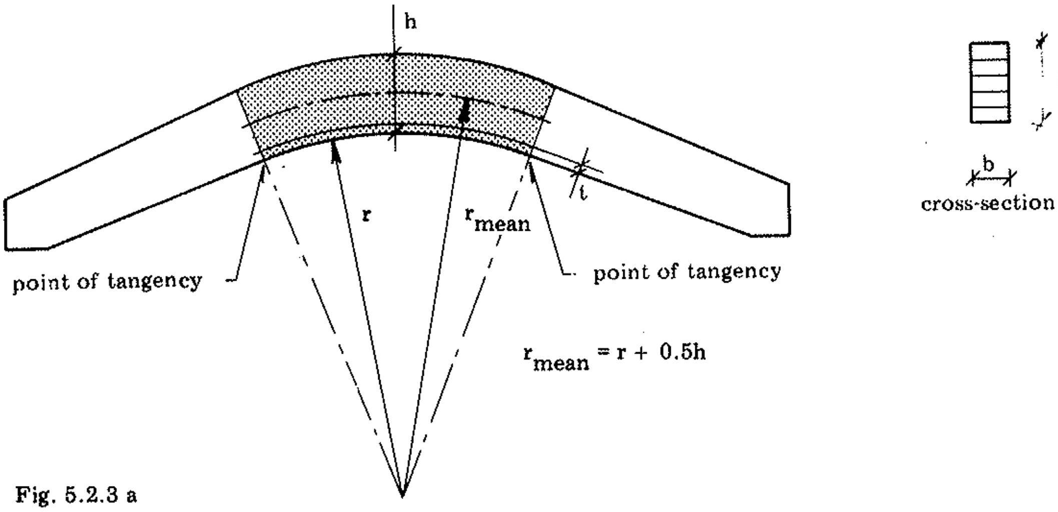


Fig. 5.2.3 a

Reduction of strength

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

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

$$k_{curv} = 0.76 + 0.001\frac{r}{t} \tag{5.2.3 a}$$

Distribution of bending stresses

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

: The bending stresses in the innermost fibre can be calculated as

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

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

$$\sigma_{mo} = \frac{6M}{bh^2} \tag{5.2.3 c}$$

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

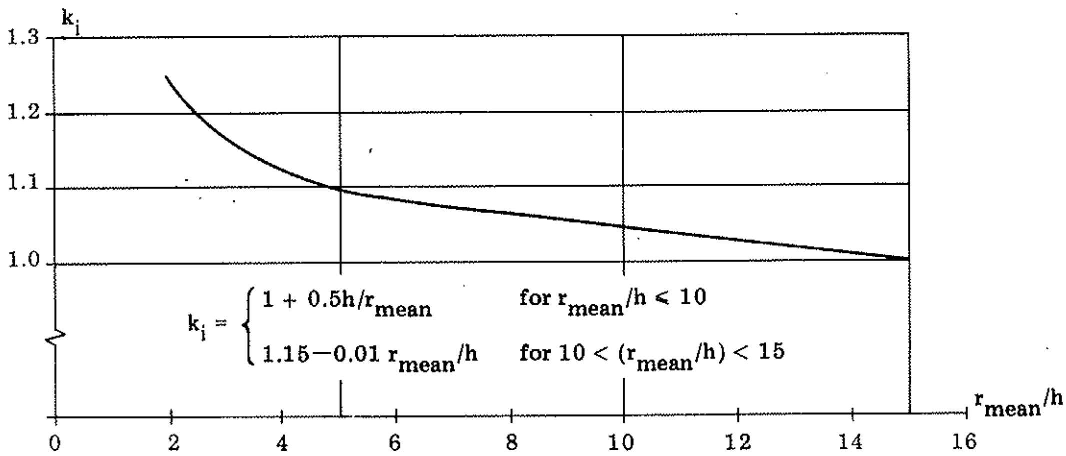


Fig. 5.2.3 b

A simplified version of the rules in Canadian Standard CSA-086-1977.

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

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

where

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

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

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

:

: $$\sigma_t = \frac{1.5\ M}{r_{mean} bh} \qquad (5.2.3\ f)$$

The inclusion of limitations on $\sigma_{m,wc}$ and $\sigma_{m,wt}$ is discussed. According to established practice in USA and Canada they are disregarded in designing plywood beams. If the stresses are calculated according to the theory of elasticity they will for most plywood types be decisive.

7. DESIGN OF COMPONENTS AND SPECIAL STRUCTURES

7.1 Glued components

7.1.1 Thin-webbed beams

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

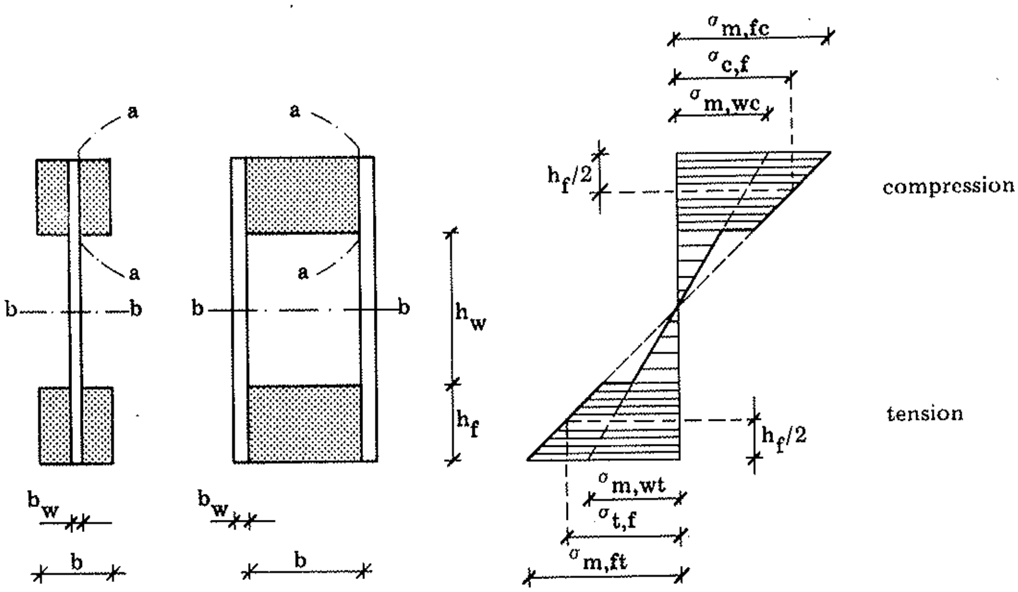


Fig. 7.1.1 a

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

$$|\sigma_{m,fc}| \leq f_m \qquad (7.1.1\ a)$$

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

$$\sigma_{t,f} \leq f_{t,0} \qquad (7.1.1\ c)$$

$$\sigma_{m,ft} \leq f_m \qquad (7.1.1\ d)$$

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

For box beams an investigation of the lateral instability may be omitted if (7.1.1 a) is replaced by

$$|\sigma_{m,fc}| \leq k_{inst} f_m \qquad (7.1.1\ e)$$

where k_{inst} is determined according to section 5.1.1.3. This is on the safe side.

These rules are based on calculations according to the general method given below.

The method given is based on the theory of elasticity, see e.g. Halasz & Cziesielski (Berichte aus der Bauforschung, Heft 47) and CIB-W18/6-4-3.

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

It must be shown that the webs do not buckle.

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

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

a buckling investigation is not necessary.

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

Table 7.1.1.2

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

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

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

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

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

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

σ_{crit} can be determined as

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

where $k_{buck,\sigma}$ for a number of cases is given in fig. 7.1.1 c and fig. 7.1.1 d.

τ_{crit} can be determined as

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

where $k_{buck,\tau}$ for pure shear is given in fig. 7.1.1 e.

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

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

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

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

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

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

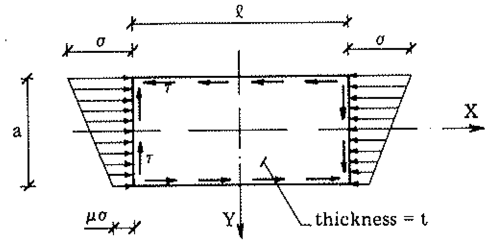


Fig. 7.1.1 b

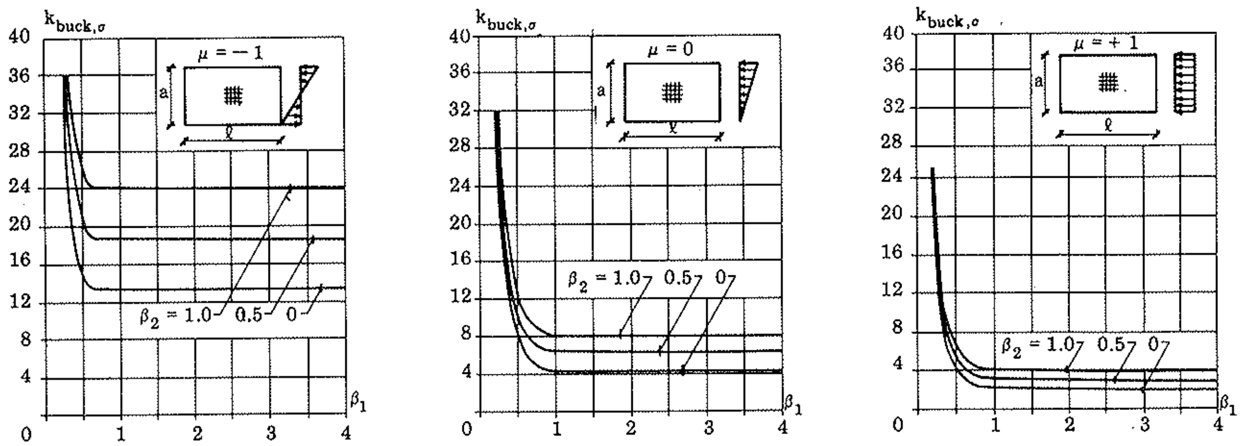


Fig. 7.1.1 c

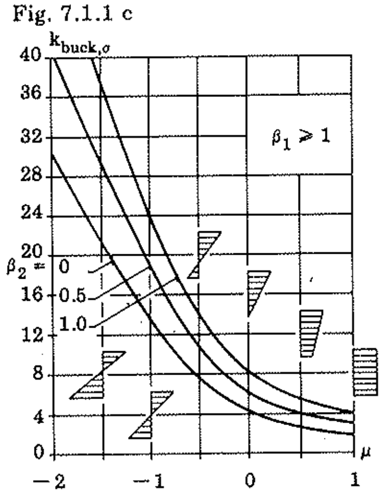


Fig. 7.1.1 d

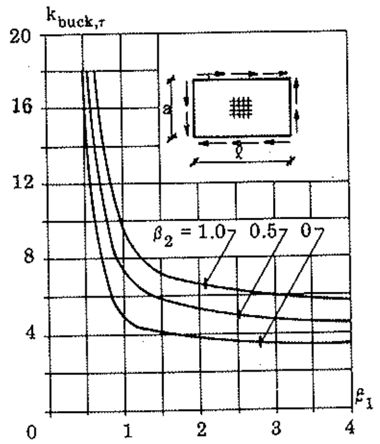


Fig. 7.1.1 e

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

The simple method has been shown by Booth to be satisfactory (IUFRO-Section 41, Madison, 1971). The effective width for uniform load for plywood corresponds approximately to $b_{f,e} = 0.15\ell$ (Möhler a.o. in Holz als Roh- und Werkstoff Nr. 21, 1963) but has been reduced to take into account the effect of uneven load distribution.

The limit b_{max} has been calculated according to the method given above.

The design of spaced columns and mechanically jointed components is discussed in CIB-W18/3-2-1.

7.1.2 Thin-flanged beams (stiffened plates)

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

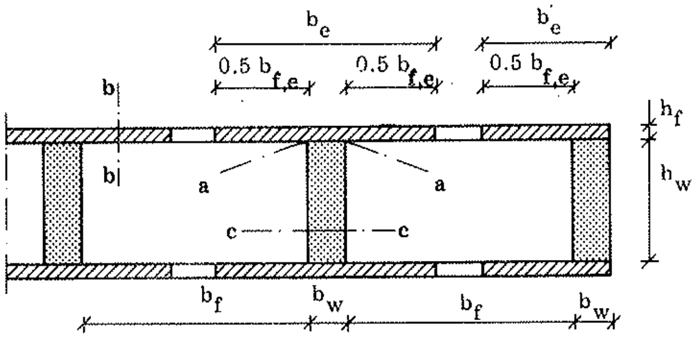


Fig. 7.1.2

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

$$b_e = b_{f,e} + b_w \qquad (7.1.2\ a)$$

or

$$b_e = 0.5\ b_{f,e} + b_w \qquad (7.1.2\ b)$$

respectively.

The free effective width, $b_{f,e}$ ($\leqslant b_f$), is given in table 7.1.2.

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

Table 7.1.2

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

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

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

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

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

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

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

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

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

The following guidelines were given in CIB-Timber Code, second draft:

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

A sub-group was formed at the CIB-W18-meeting in Perth with the task of discussing these rules, especially with regard to the use of nail-plates.

7.2 Mechanically jointed components

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

In addition the recommendations of sections 5 and 7.1 apply.

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

Table 7.2

Fastener	Slip modulus (N/mm)
Round nails with $d < 5$ mm☆	$0.002\ E_0 d$
Round nails with $d > 5$ mm☆	$0.1\ E_0$
Bolts with pressed-in connectors	$1.3\ E_0$

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

☆ For square nails 15% higher values are allowed.

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

7.3 Trusses

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

As an alternative a simplified calculation after the following guidelines is permitted:
(under preparation)

8. CONSTRUCTION

8.0 General

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

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

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

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

8.1 Materials

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

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

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

The limitations on bow in most national stress grading rules are inadequate for the selection of material for columns and beams where lateral instability may occur. Particular attention should therefore be paid to the straightness of columns; e.g. limiting bow to about 1/300 of the length, and to beams where lateral instability may occur; e.g. limiting bow to about 1/200 of the length. It may also be necessary to introduce more stringent limits on other particular members, e.g. twist for torsional members.

8.2 Machining

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

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

8.3 Joints (Preliminary. Final version to be prepared at completion of section 6)

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

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

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

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

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

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

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

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

Connectors should fit tightly in the grooves.

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

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

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

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

b. The lead hole for the threaded portion shall have a diameter determined by the characteristic density of the species or species group and by the length and diameter of the screw.

: Recommendations on lead hole diameters will be provided.

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

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

8.4 Assembly

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

8.5 Transportation and erection

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

8.6 Surface treatment

Under preparation. Will contain recommendations on painting, staining, etc.

ANNEX 7A
MECHANICALLY JOINTED BEAMS AND COLUMNS WITH I-, T- OR BOX CROSS-SECTIONS

1. SCOPE

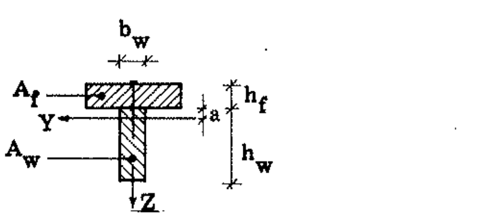
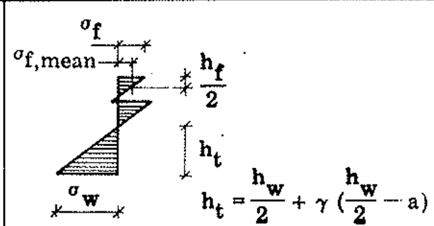
Beams or columns with cross-sections as shown in fig. 1 are dealt with. The individual members are connected to each other by nails, bolts with toothed metal plate connectors or similar non-rigid fasteners.

A method is given to determine stresses, deflections and load on the fasteners of beams and the load-carrying capacity of columns, including the necessary requirements to the fasteners.

2. NOTATIONS

Reference is made to fig. 1. In all cases the Z-axis is a symmetry axis. For cross-sections of type 1 the Y-axis is a gravity axis, while for type 2 and 3 it is a symmetry axis.

For beams bending about the Y-axis is assumed.

<table>
<tr><th>Type No.</th><th>Cross-section</th><th>A_r</th><th>Stress distribution</th></tr>
<tr><td>1</td><td>
</td><td>$\frac{A_f A_w}{A_f + A_w}$</td><td>
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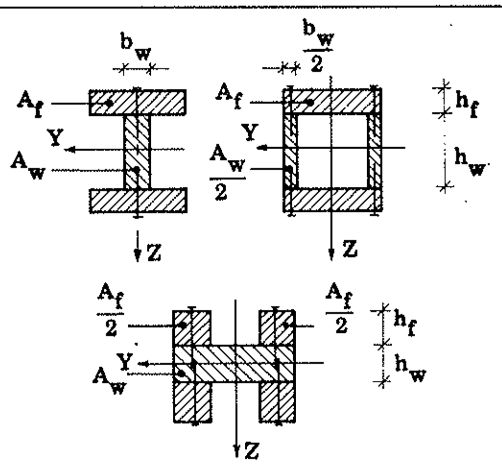
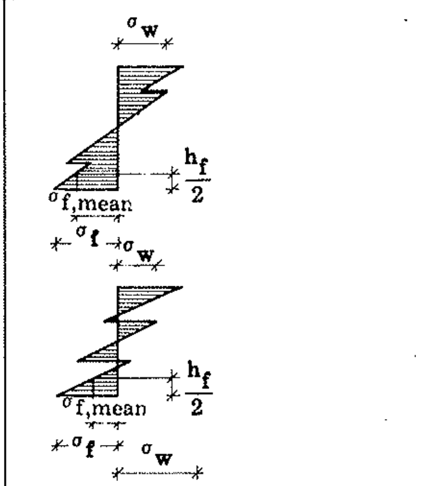
<tr><td>2</td><td>
</td><td rowspan="2">A_f</td><td>
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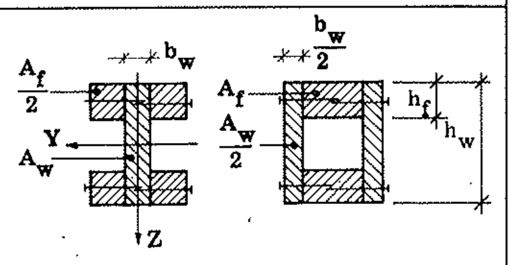
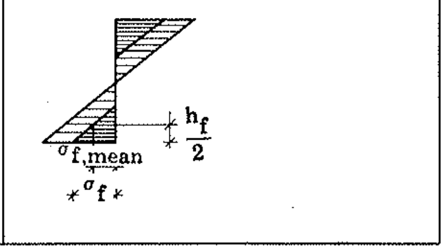
<tr><td>3</td><td>
</td><td>
</td></tr>
</table>

Fig. 1

A	Area	
	A_{tot}	Total area
	A_f	Flange area
	A_w	Web area
	A_r	See fig. 1
E	Modulus of elasticity	
F	Load on one fastener	
G	Shear modulus	
I	Moment of inertia (second moment of area)	
	I_{tot}	Total value calculated around the geometric gravity axis (Y-axis)
	I_{own}	Sum of moments of inertia for the individual parts around own gravity axis
	I_e	Effective moment of inertia, see formula (2)
K	Slip modulus, see section 3	
M	Bending moment (about Y-axis)	
P	Axial load on column	
	P_{crit}	Critical value
Q	Shear force (in the direction of the Z-axis)	
S	Static moment (first moment of area)	
	S_f	Static moment of flange about Y-axis
a	Distance of gravity axis from flange	
b	Width	
	b_w	Thickness of web
f	Strength	
	$f_{c,0}$	Compressive strength
h	Depth	
	h_f	Depth of flange
	h_w	Depth of web
	h_t	Depth of tension zone in web
k_{col}	Column factor	
ℓ	Span of beam or free length of column	
s	Spacing of fasteners (1/s is number of fasteners per unit length). If s is varying evenly in the longitudinal direction according to the shear force between s_{min} and $s_{max} \leqslant 4s_{min}$ an effective value of s equal to $s = 0.75\ s_{min} + 0.25\ s_{max}$ may be used.	
u	Slip between jointed members	
γ	Effectiveness factor, see section 4	
λ	Slenderness ratio	
	λ_e	Effective slenderness ratio
σ	Axial stresses	
	σ_f	σ in outermost fibre of flange
	$\sigma_{f,mean}$	σ in the middle of the flange
	σ_w	σ in outermost fibre of web
τ	Shear stress	

3. ASSUMPTIONS

The member is assumed either loaded in the Z-direction giving moments about the Y-axis, or with an axial load acting in the gravity axis.

The case where both bending moment and axial force are acting (beam columns) at the same time is only treated in a special case, cf. section 7.

The conditions are assumed linear-elastic and the following relation between the load on a fastener and the slip is assumed to apply.

$$F = Ku \tag{1}$$

All members are assumed to have the same modulus of elasticity, but the expressions may be extended to apply also to cross-sections where the cross-section members have different properties by transforming of the cross-section sizes in relation to their stiffness after the usual methods.

4. EFFECTIVE MOMENT OF INERTIA

The effective moment of inertia is determined by

$$I_e = I_{own} + \gamma(I_{tot} - I_{own}) \tag{2}$$

where

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} \frac{E}{K} s} \tag{3}$$

For beams with thin webs of plywood, particle boards or fibre boards, however,

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} (\frac{E}{K} s + \frac{E}{2Gb_w})} \tag{4}$$

5. BEAMS

5.1. Calculation of stresses

Fig. 1 is referred to.

The stresses in cross-sections of type 1 are calculated from the following expressions:

$$|\sigma_w| = \frac{M}{I_e} h_t \tag{5}$$

$$|\sigma_{f,mean}| = \frac{M}{I_e} \gamma (\frac{h_f}{2} + a) \tag{6}$$

$$|\sigma_f| = |\sigma_{f,mean}| + \frac{M}{I_e} \frac{h_f}{2} \tag{7}$$

and in cross-sections of types 2 and 3 from:

$$|\sigma_w| = \frac{M}{I_e} \frac{h_w}{2} \tag{8}$$

$$|\sigma_{f,mean}| = \frac{M}{I_e} \gamma \frac{h_w + h_f}{2} \tag{9}$$

$$|\sigma_f| = |\sigma_{f,mean}| + \frac{M}{I_e} \frac{h_f}{2} \tag{10}$$

5.2. Calculation of maximum shear stresses

For cross-sections of type 1 the maximum shear stresses occur where the stresses in the web are zero and can be calculated from

$$\max\tau = \frac{Qh_t^2}{2I_e} \tag{11}$$

For cross-sections of types 2 and 3 the maximum shear stresses occur in the middle of the web and can be calculated from

$$\max\tau = \frac{Q}{I_e b_w}\left(\gamma A_f \frac{h_f + h_w}{2} + \frac{1}{8} A_w h_w\right) \tag{12}$$

5.3. Calculation of load on fasteners

The load per fastener can be determined from

$$F = \gamma \frac{QS_f}{I_e} s \tag{13}$$

5.4. Deflections

The deflections from the moment are calculated as usual applying the effective moment of inertia I_e.

6. CONCENTRICALLY LOADED COLUMNS

6.1. Load-carrying capacity

The load-carrying capacity corresponding to deflection along the Z-axis can be determined as

$$P_{crit} = k_{col} f_{c,0} A_{tot} \tag{14}$$

The columnn factor k_{col} is determined as for a corresponding column with rigid joints between the dross-section members, but the effective slenderness ratio:

$$\lambda_e = \ell \sqrt{\frac{A_{tot}}{I_e}} \tag{15}$$

is used.

For the T-cross-section and the I-cross-section, type 2, the load-carrying capacity for deflection in the Y-direction is found as the sum of the load-carrying capacity of the individual members, i.e. the stiffening effect that the members might have on each other is not taken into account.

6.2. Load on fasteners

The load on the fasteners can be calculated by eq. (13), assuming

$$Q = \begin{cases} \frac{P}{60}\frac{1}{k_{col}} & \text{for } 60 \leqslant \lambda_e \\ \frac{\lambda_e}{60}\frac{P}{60}\frac{1}{k_{col}} & \text{for } 30 \leqslant \lambda_e \leqslant 60 \\ \frac{P}{120}\frac{1}{k_{col}} & \text{for } \lambda_e \leqslant 30 \end{cases} \tag{16}$$

7. COMBINED LOADS

In cases where small moments resulting from e.g. own weight are acting apart from axial load, the usual interaction formulas can be used for the stresses determined above.

8. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up timber columns. CIB-W18/3-2-1.

ANNEX 7B
SPACED COLUMNS WITH NAILED OR GLUED PACKS OR BATTENS

1. SCOPE

Columns as shown in fig. 1 are dealt with, i.e. columns with two or in certain cases three or four identical shafts jointed with packs or battens. The joints may be either nailed or glued or bolted with toothed metal plate connectors. Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio, whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

It is assumed that the construction rules given in section 3 are observed, and that the joints are designed for forces as stated in section 6.

Only concentrically loaded columns are dealt with.

2. NOTATIONS

Reference is made to fig. 1.

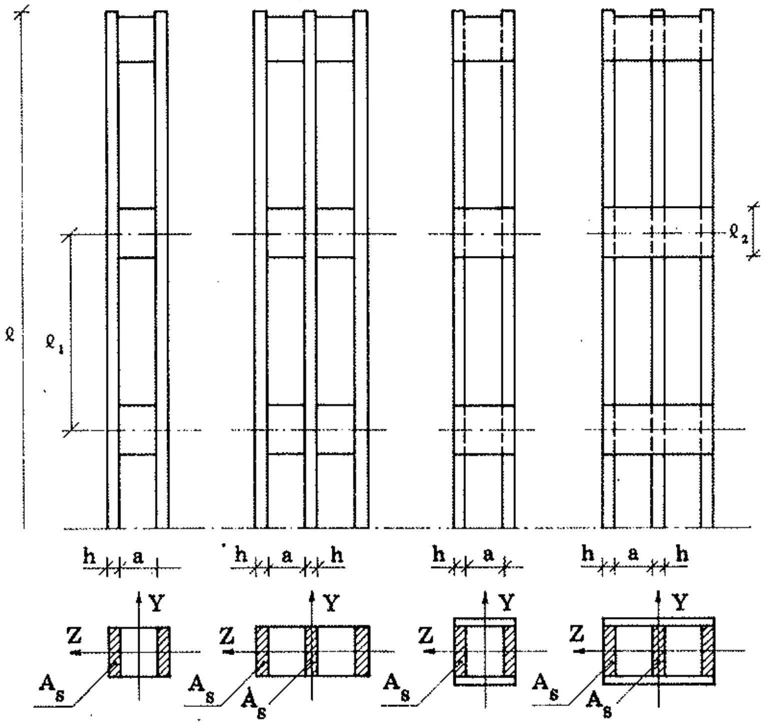


Fig. 1.

A	Area	
	A_s	Area of one shaft
	$A_{tot} = nA_s$	Total area
I	Moment of inertia (second moment of area)	
	I_s	I for one shaft about own gravity axis
	I_{tot}	Total moment of inertia about Y-axis
P	Column load	
	P_{crit}	Load-carrying capacity
Q	Shear force	

T_0 See fig. 3

a Distance

- a Free distance between shafts
- $a_1 = a+h$ See fig. 3

$f_{c,0}$ Compressive strength parallel to the grain

k_{col} Column factor

ℓ length

- ℓ Free length of column
- ℓ_1 Distance between midpoints of packs or battens
- ℓ_2 Length of packs or battens

n Number of shafts

λ Slenderness ratio

- $\lambda = \ell \sqrt{\frac{A_{tot}}{I_{tot}}}$ Slenderness ratio for a solid column with the same cross-section
- $\lambda_1 = \ell_1 \sqrt{\frac{A_s}{I_s}}$ Slenderness ratio for the shafts
- λ_e Effective slenderness ratio

η Factor, see table 2.

3. ASSUMPTIONS

- The cross-section is composed of 2, 3 or 4 identical shafts.
- The Y- and Z-axes are symmetry axes.
- The number of free fields are at least 3, i.e. the shafts are at least jointed in the ends and in the third points.
- The free distance between the shafts is not greater than 5 times the lamella thickness ($a/h \leqslant 5$).
- The joints and packs and battens are designed for a shear force Q as stated in section 5.
- The length of the packs should satisfy the condition $\ell_2/a \geqslant 1.5$.
- For nailed joints there should in each section be at least 4 nails or 2 bolts with metal plate connectors. For nailed joints at the ends apply that there should be at least 4 nails in a row in the longitudinal direction of the column.
- Battens should be made of structural plywood and their length satisfy the condition $\ell_2/a \geqslant 2$.
- The columns are solely subjected to concentrical axial loads.

4. LOAD-CARRYING CAPACITY

For deflection in the Y-direction the load-carrying capacity can be determined as the sum of the load-carrying capacity of the individual members.

For deflection in the Z-direction the load-carrying capacity are determined as

$$P_{crit} = k_{col} f_{c,0} A_{tot} \tag{1}$$

The column factor k_{col} is determined as for solid columns, but the usual slenderness ratio

$$\lambda = \ell \sqrt{\frac{A_{tot}}{I_{tot}}} \tag{2}$$

is replaced by the effective slenderness ratio

$$\lambda_e = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \qquad (3)$$

η is given in table 2.

Table 2: η

	Packs			Battens	
	glued	nailed	bolted*	glued	nailed
Long-term loading	1	4	3.5	2	6
Short-term loading	1	3	2.5	2	4.5

* with toothed metal plates.

5. SHEAR FORCES

The load on the fasteners and battens or packs can be calculated as stated in fig. 3, assuming

$$Q = \begin{cases} \dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 60 \leqslant \lambda_e \\ \dfrac{\lambda_e}{60}\dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 30 \leqslant \lambda_e \leqslant 60 \\ \dfrac{P}{120}\dfrac{1}{k_{col}} & \text{for } \lambda_e \leqslant 30 \end{cases}$$

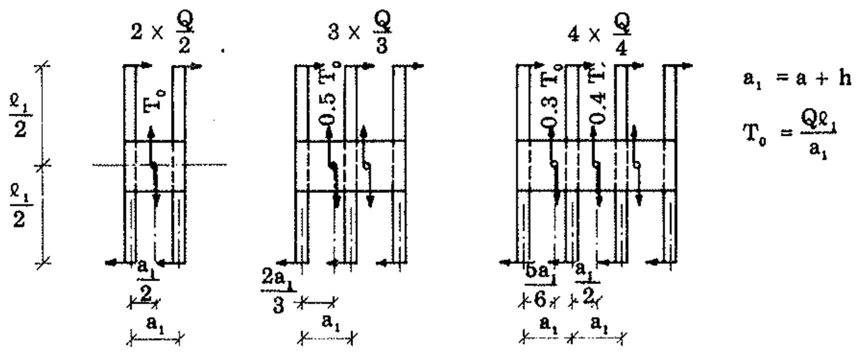


Fig. 2

6. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up columns. CIB-W18/3-2-1.

ANNEX 7C
LATTICE COLUMNS WITH GLUED OR NAILED JOINTS

1. SCOPE

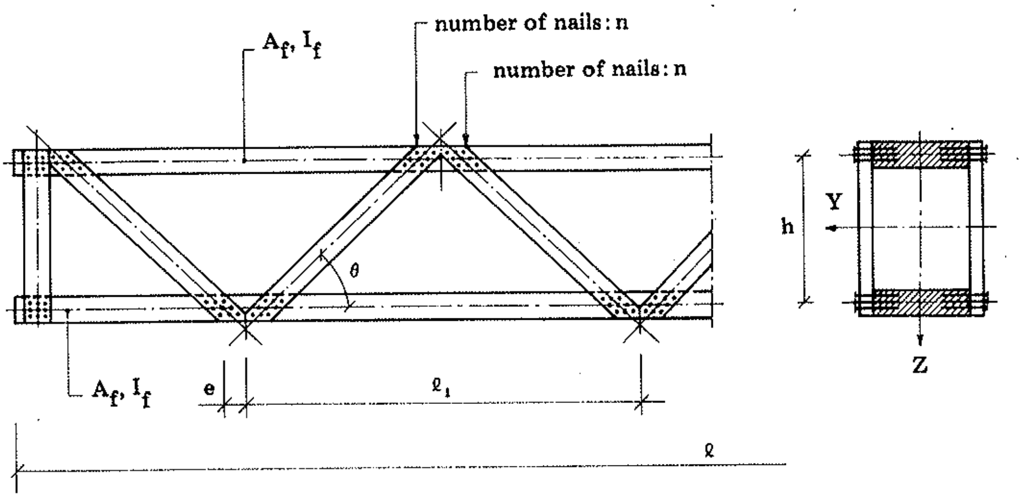
Lattice columns with N- or V-lattice and with glued or nailed joints are dealt with.

Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

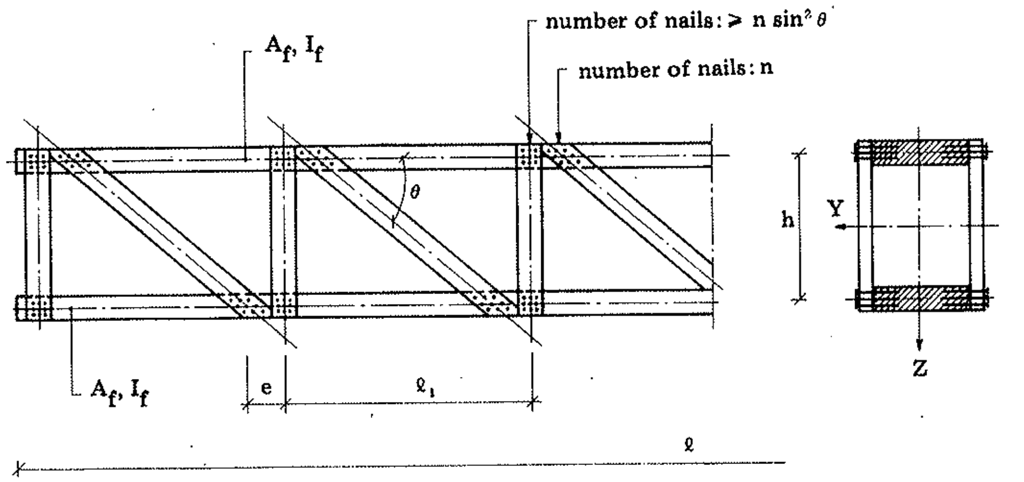
It is assumed that the joints are designed for forces as stated in sections 3 and 5.

2. NOTATIONS

Reference is made to fig. 1.



V-truss



N-truss

Fig. 1

A Area

- A_f Area of one flange

I Moment of inertia (second moment of area)

- I_f I for one flange about own axis
- I_{tot} Total value: $I_{tot} = 2I_f + \frac{1}{2} A_f h^2$

K Slip modulus for one nail, i.e. the force per nail that will cause a slip of 1

P Axial load on column

Q Shear force

e Eccentricity

$f_{c,0}$ Compression strength

h Depth of column (flange centre distance)

i_f Radius of gyration ($=\sqrt{I_f/A_f}$)

k_{col} Column factor

ℓ_1 Joint distance

n Number of nails per diagonal in a joint. If the diagonal consists of two or more pieces, n is the sum of nails, not the number of nails per shear face

θ Angle between a flange and a diagonal

λ Slenderness ratio

- λ_e Effective slenderness ratio
- λ_f Slenderness ratio of a flange (ℓ_1/i_f)

μ Parameter, see section 4.

3. ASSUMPTIONS

The structure is assumed to be symmetric about the Y- and Z-axes of the cross-section. However, the lattice of the two sides are allowed to be staggered the length $\ell_1/2$.

There should be at least 3 fields, i.e. $\ell >$ about $3\ell_1$.

The column should be designed for a shear force Q, as stated in section 5, i.e. the diagonals and joints should be designed for $Q/\sin\theta$.

In nailed structures at least 4 nails per shear should be used in each diagonal in each nodal point. At each end bracings should be used.

The slenderness ratio for the individual flange corresponding to the length ℓ_1 must not exceed 60, i.e.

$$\lambda_f \leqslant 60$$

Besides, it assumed that no local rupture is occurring in the flanges corresponding to the column length λ_1.

4. LOAD-CARRYING CAPACITY

The load-carrying capacity corresponding to deflection in the Y-direction is equal to the sum of the load-carrying capacity of the flanges for deflection in this direction.

For deflection in the Z-direction the load-carrying capacity is assumed to be equal to

$$P_{crit} = 2k_{col}f_{c,0}A_f$$

where the column factor k_{col} is determined as for a corresponding column of solid timber, but instead of the geometrical slenderness ratio of the column

$$\lambda = \ell\sqrt{\frac{2A_f}{I_{tot}}}$$

the effective slenderness ratio

$$\lambda_e = \lambda\sqrt{1+\mu}$$

is used, where μ is determined as stated below:

Glued V-truss:

$$\mu = 4\left(\frac{e}{i_f}\right)^2\left(\frac{h}{\ell}\right)^2$$

λ_e is not to be taken less than 1.05 λ.

Glued N-truss:

$$\mu = \left(\frac{e}{i_f}\right)^2\left(\frac{h}{\ell}\right)^2$$

λ_e is not to be taken less than 1.05 λ.

Nailed V-truss:

$$\mu = 25\frac{hEA_f}{\ell^2 nK\sin 2\theta}$$

Nailed N-truss:

$$\mu = 50\frac{hEA_f}{\ell^2 nK\sin 2\theta}$$

5. SHEAR FORCES

The column should be designed for a shear force Q given by

$$Q = \begin{cases} \dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 60 \leqslant \lambda_e \\ \dfrac{\lambda_e}{60}\dfrac{P}{60}\dfrac{1}{k_{col}} & \text{for } 30 \leqslant \lambda_e \leqslant 60 \\ \dfrac{P}{120}\dfrac{1}{k_{col}} & \text{for } \lambda_e \leqslant 30 \end{cases}$$

6. REFERENCES

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up columns. CIB-W18/3-2-1.

CIB-W18/11-100-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

Comments received on the CIB Code:

a) U Saarelainen - Finland

b) Y M Ivanov - Union of Soviet Socialist Republics

c) R H Leicester - Australia

d) W Nożyński - Poland

e) W R A Meyer - The Netherlands (List of Contents)

f) P Beckman; R Marsh - United Kingdom

g) W R A Meyer - The Netherlands (Centrically and Eccentrically Loaded Construction Elements)

h) A Vronwenvelder - The Netherlands (Lateral Buckling)

VIENNA, AUSTRIA

MARCH 1979

CIB-W18/11-100-2a

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

by

U Saarelainen
Technical Research Centre of Finland
FINLAND

VIENNA, AUSTRIA

MARCH 1979

From: U Saarelainen,
Technical Research Centre of Finland. Jan 1979

The national timber code group has studied the CIB draft and has put through me the following comments:

2.1.1. A volume of 0,02 m^3 seems to be surprisingly high.

4.3.1. A definition of the extreme eighths is deviating from the current practice (extreme sixths). The reason for change is unknown. Further, the recommend use of structural timber instead of stress graded laminae in glulams seems to a new feature, of which we have not got any experience. The current Finnish practice makes some clear distinctions between laminae grading and stress grading of structural timber.

4.7. The referred climate class 0 has not been defined in section 2.2.

5.1.0.a The applied ratio $E/G = 12$ seems to be very low compared to previous practice of $E/G = 20$.

5.1.0.b In the modification of deformation calculations it is assumed that only after 10 years the deformations will increase by 40 %. The studies have, however, shown that deflections may increase by 100 % even during the first two years of loading.

5.1.1.7 Table

The correct fixing of a wooden post restrained at one end both in position and direction and free at the other end, is merely theoretical. A value of l_c/l between 2.1 and 2.5 is to be recommended.

CIB-W18/11-100-2b

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

by

Y M Ivanov
Central Research Institute for Building Structures
UNION OF SOVIET SOCIALIST REPUBLICS

VIENNA, AUSTRIA
MARCH 1979

SOME QUESTIONS AND PRELIMINARY COMMENTS
ON THE THIRD DRAFT OF THE CIB W-18
STRUCTURAL TIMBER DESIGN CODE

from

Yu M Ivanov
Central Research Institute for Building Structures
Gosstroi
Union of Soviet Socialist Republics

The third draft of the CIB W-18 Structural Timber Design Code is an interesting attempt to explain for a designer succession of the transition from strength indices of real material - structural wood - to design allowable stresses for calculation of the members of wood structures. Such an approach must be welcome having made it easy to practical use at engineering calculations. In order to assist as far as possible to accomplish given text as well as to clear up its separate points we give below some questions and notes which are preliminary since given draft of the Code is not completed and in several important states of principle significance is not supported by necessary experimental basing data.

a the first question: by what reliability factor during service life of structures the design allowable stresses (Table 5.1.0a; 5.1.0b etc) proposed are ensured by means of suggested "standard strength classes" of structural wood and what are concrete limitations of qualitative defects of wood in these classes?

b the second question connected with the first: on what experimental investigations (tests up to failure of structural members of natural size at main stress states, for example in tension parallel to the grain) are based the short-time strength values for suggested "standard strength classes" of wood (Table 4.1.1 and 4.3.1)?

c how is ensured the transition from deflection of board through its plane by machine stress grading to the characteristic strength given in Table 5.1.0a?

d in 2.1.1 it should be pointed, besides temperature and relative air humidity, the equilibrium moisture content of wood which is reached only as a result of prolonged stay of structure at conditions mentioned.

e to 2.2. "The climate classes" are in fact the service conditions since classes 1,2 and 3 given in 2.2 can be in the same climate zone. It is necessary to add to given in 2.2 three classes of service conditions the following service conditions of wood structures: (1) under roof but in outside atmosphere; (2) in contact with earth; (3) concrete forms and scaffolding ought to be separated from the members which are permanently in water, coefficients in Table 5.1.0b having been corrected accordingly and transferred here.

The "marine works" are in conditions of permanent moistening (partly in water), so it might be hardly agreed with decrease of absolute values of strength of wood (Table 5.1.0b) in state of saturation by (1-0,5/0,6)100=(1-0,835). 100=16,5 per cent in relation to strength of wood at equilibrium moisture content 12 per cent. The difference of strength of wood mentioned reaches in fact 30 per cent. The matter of fact is that with rising of moisture content the absolute magnitude of the strength of wood decreases but the relative decrease of the latter is not changed what ought to be taken into consideration at determination of correcting coefficients for design stresses.

f in Table 2.3.a and 2.3.b - "load-duration classes" uninterrupted load action ("permanent" load) equals in fact to periodical one ("normal" load) since for both is proposed the same coefficient of decrease of allowable stress 0,6 (Table 5.1.0b). Such an equality it will rather be difficult to base since, as is known, a periodical action of load is determined by a sum of duration its applications during the service life t' of structure what is of course less than t'. From this standpoint the statement between pages 2.1 and 2.2 is applicable only to modulus of elasticity but not to characteristic strength since the load-duration effect has a cumulative nature.

The range of "normal" class from 3 weeks to 10 years (and practically - to 50 years), during which the wood structure can decrease its bearing capacity for 15 per cent, is too big, bringing in a marked uncertainty. It might be proposed: (1) I class ("permanent") to attribute to uninterrupted action of whole design load, for example earth and water pressure, during service life of structure, ie 25-50 years, with coefficient 0,5; (2) second class ("normal") to attribute to periodical action of design load which sum of duration of its applications during service life of structure reaches several years (up to 10 years), with coefficient 0,6. According to this the other coefficients for the rest classes ought to be corrected.

g Tables similar to 5.1.0a and 5.1.0b but for glued laminated members as well as allowable stresses for timber logs are absent.

h in 1.1. is desirable to mention as the base of the Code also the practice of construction and service of wood structures.

Besides of the questions of principle some questions are raised concerning easiness of use as well as the language of the code.

i it is desirable to retain the symbols used in mechanics, for stress σ, τ; for example: $[\sigma]$, $[\tau]$ instead of f; $\bar{\sigma}$, $\bar{\tau}$ for allowable stress etc.

j it is desirable to unite the Table 2.3.a and b and to place here the number coefficients from Table 5.1.0b.

k 5.1.1. and later: it is desirable to include in the right side of all the inequalities necessary calculation coefficients for stress what will simplify the calculation work of designer.

l it is desirable to include in the code a Table with design stresses for members of the more frequently used timber and glued laminated structures and also to give examples of calculation of allowable stress for other type of wood structures. This will allow to avoid mistakes at practical application of the code to design of wood structures in different cases of service.

25 1 79

CIB-W18/11-100-2c

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

by

R H Leicester
Commonwealth Scientific and Industrial Research Organization
AUSTRALIA

VIENNA, AUSTRIA

MARCH 1979

CSIRO

DIVISION OF BUILDING RESEARCH

P.O. BOX 56, HIGHETT, VIC. 3190. (GRAHAM ROAD) TELEPHONE 95 0333. TELEX AA 33766 TELEGRAMS BUILDRESEARCH MELBOURNE

RHL:VM

Professor H.J. Larsen,
Institute of Building Technology
and Structural Engineering,
Aalborg University Centre,
Danmarkagade 19,
DK-9000,
Aalborg,
DENMARK.

26th February, 1979

Dear Professor Larsen,

CIB STRUCTURAL TIMBER DESIGN CODE

The CSIRO Division of Building Research has been requested by the Standards Association of Australia to provide comment on the above document. Enclosed is a copy of these comments. We have deemed it appropriate at this time to limit comment in general only to matters relating to the format of the code.

Our main comment is that in its current format the CIB draft code would not be either useful or useable to Australia. The range of structural situations covered by the CIB draft code does not include all the basic ones considered in AS:1720-1975, the Australian Timber Engineering Design Code. More importantly however, the CIB draft code does not cater for the range of species utilised in Australia. The Australian code AS:1720-1975 considers the timber in terms of property group classifications. It is of interest to point out that we have published, or will soon publish group classifications for some 500 Australian species, 100 South American species, 700 African species, and 700 South-East Asian species. Using these classifications, structural design with all these species may be carried out through the use of the Australian code. However this will not be possible with the CIB draft code.

In order to provide some constructive comment, I am including with this letter some notes on five aspects of format in the Australian code AS:1720-1975 related to topics which we feel are inadequately covered by the CIB draft code. These topics are the following -

1. Strength classifications.
2. Design Rules for green timber.
3. Laminated timber.
4. Buckling strength.
5. Fracture strength.

...../2

I would be grateful if you could circulate a copy of these Notes, the enclosed Comments to the Standards Association of Australia and this letter to other members of the drafting Committee for the CIB code.

I am also sending you under separate cover four copies of the Australian Timber Engineering Code AS:1720-1975. Could you please give these to members of the drafting Committee who may not have access to this Code? It will help to place some of the enclosed Comments and Notes in their appropriate context. For the same reason I am also forwarding to you under separate cover some relevant publications, reprints and lecture notes. These include the Australian Standard AS:1649-1974, a Standard for the Determination of Basic Working Loads for Metal Fasteners for Timber.

I regret writing so much critical comment, and find even less joy in promoting a local code (AS:1720-1975) as one having a preferable format; however I trust that you will accept that these comments are offered in good faith and with the best of intentions. The first draft of the Australian Code was drawn up relatively recently, i.e. after 1970, and so for this undertaking we had access to most of the other current existing Timber Engineering Codes to use as a basis for the Australian Code. In addition we had a background of some 35 years experience in developing a strength classification system for designing with a great range of species, including both softwoods and hardwoods, and green and dry timbers.

Since its inception AS:1720-1975 has been under continual review and this year a major revision will be made to take into account the findings of recent research to assess the effectiveness of the Code and also to see if it is possible to modify the format to satisfy criticisms (particularly with respect to complexity) received from the structural engineers who use the code. As it is hoped that this will be the last major change in format for some time it is planned to include a commentary with the next edition of the code.

There are obviously many more matters we would like to discuss but unfortunately there is a limit to the quantity of written communication that it is feasible to attempt; and alas the tyranny of distance prevents our attendance at the W-18 discussions. However, from all here at the CSIRO Division of Building Research, we send you our best wishes for success in the future deliberation of the W-18 Committee.

Yours faithfully,

R.H. Leicester

Dr. R.H. Leicester
Officer-in-charge
Structures Section.

Copy sent to Mr. J.R. Tory, Technical Secretary CIB-W18

COMMENT ON
CIB STRUCTURAL TIMBER DESIGN CODE THIRD DRAFT, SEPT. 1978

Submitted by -
R.H. Leicester,
(CSIRO, Division of
Building Research.
Melbourne, Australia.
1979)

GENERAL COMMENT No.1.

Although the code is intended to be an International Standard, because of the format in which it is written it will not be useable in countries that utilise a wide range of species. Examples of such countries are Australia, and the countries of South America, Africa and South-East Asia.

An example of an alternative format which would be suitable for an international code is the Australian Timber Engineering Code, AS:1720-1975. Here the properties of timber (including their performance with metal connectors) have been classified in groups across the full practical range of timber that is utilised. Either published or shortly to be published are the strength groupings of 500 Australian species, 100 South American species, 700 African species and 700 South-East Asian species. With this data engineering design for all these species can be carried out using AS:1720-1975. This will not be possible with the proposed CIB design code.

GENERAL COMMENT No.2.

The code does not appear to mention whether the timber is initially green or air-dry. Both of these types of timber are used extensively in Australia. For comparable material (i.e. with the same strength group, stress grade and density) the performance of the green timber is significantly inferior to that of the dry timber with respect to creep, creep buckling and fastener strength.

GENERAL COMMENT No.3.

Several design rules for common basic structural elements that have been written into the Australian Timber Engineering Code AS:1720-1975 in response to requests from structural engineers, are not considered in the CIB draft code. These include the following -

(i) Design rules for a member (such as the top chord of a truss) that can buckle laterally due to beam instability and also buckle about both major and minor axes due to column instability. Also the code contains no rule for the design of a beam-tie for the case of a beam sufficiently slender that it can buckle laterally.

(ii) Rules for the design of buckling restraints.

(iii) Design rules for the case of point loads acting on a grid system.

(iv) Design rules for the case of a point load acting on the flange of a plywood web beam which does not have vertical stiffeners attached to the web.

(v) Load sharing effects of multiple member systems.

(vi) Rules for the acceptance of timber structures based on prototype or proof testing procedures.

(vii) Rules for the design of pole structures.

CLAUSE 2.2

The intent of the choice of climate classes is not clear. For an international Standard the choice of climate classes should cover the full practical range of climates encountered, ranging from the cold Northern European climates through the hot and dry tropics, to hot and humid equatorial conditions.

CLAUSE 4.1.0

Strength grading should be permitted by any proven method. For example in Australia a promising method under study is the grading of timber through proof testing every stick in bending. This method would not be permitted by Clause 4.1.0.

CLAUSE 4.1.1

For all methods of grading, it should be necessary to ensure that the chosen characteristic strengths satisfy Table 4.1.1. The implied effectiveness of visual grading is not in keeping with the fact that hundreds of species are utilised, and tension strength measurements have been made on less than a dozen.

A strong objection is raised against the limited range of strength classes in Table 4.1.1. This is quite inadequate to cover the range of timbers used in Australia, and the potential utilisation of timbers in the countries of South America, Africa and South-East Asia.

One minor point that needs stating in the specification of characteristic strength of beams is the definition of the strength of a beam. For example,is it the bending strength at the worst defect with the defect edge stressed in tension, or is it the strength of the beam tested with the defect located at a random location and edge? If the latter, then is the beam tested through centre point loading, third point loading or uniform bending moment? Different answers for characteristic strength are obtained for these four cases.

CLAUSE 4.3.1

The strength classes in Table 4.3.1 are too limited in range and concept to be useable in Australia and many other countries. In Australia dozens of species have been used,often with more than one species in a member, and often with butt-joints in a portion of the laminations. The design procedure used in the Australian code AS:1720-1975, is to consider laminated timber essentially as solid timber with slight modifications due to laminating effects.

CLAUSE 4.4

Either in this clause, or else where in the code there should be a table providing a set of design stresses for plywood of different strength classes.

CLAUSE 5.1.0

Table 5.1.0a has obviously been drawn up with certain specific species in mind, and therefore is not suitable for countries utilising a wide range of timber species. For example, since bending strength is a function of both clear material strength and defects whereas compression strength perpendicular to the grain is a function only of clear material strength, then it is possible to obtain lumber of two species which have different clear material strength and hence different compression strength perpendicular to the grain, yet have the same bending strength because the defect sizes in the two populations of lumber are different. Thus Table 5.1.0a is not suitable for an international code that covers a wide range of species.

The modification factors for deformation calculations given in Table 5.1.0b do not mirror observations made in Australia (where some experimental observations have been made over a period of 10 years in practical, rather than laboratory environments).

The deformation of timber under stress is due not only to time dependent effects, but also to changes in moisture content. For most practical situations in Australia, the "creep" component due to changes in moisture content is the larger one. One result is that timber initially green and allowed to dry creeps considerably more than timber that is initially dry. For example it has been found that initially green timber, placed in an unheated shed in Melbourne (i.e. probably climate Class 2) creeps more than 200-percent, i.e. the appropriate modification factor for deformation should be 0.33, rather then the value of 0.6 shown in Table 5.1.0b.

It is not clear whether the modification factors in Table 5.1.0b contain load factors to cover uncertainty (i.e. "factor of safety"), or whether another code is to provide load factors to be used with characteristic values. In either case these should take into consideration that there is a great range of variability to be found in structural timbers. For example, in Australia we have commercially used timbers species which have an in-grade coefficient of variation for bending strength of less than 20-percent, and others with a coefficient of variation greater than 40-percent. Obviously different load factors need to be provided for these two cases.

CLAUSE 5.1.1.3

The recommendation for buckling strength does not take into consideration the effects of creep buckling when long duration loads are applied. This can be very significant for timber that is initially green.

CLAUSE 5.1.1.4

Contrary to the comment in this clause, the shear (and bending) strength of notched beams has been well studied (at least in Australia) and using the concepts of conventional fracture mechanics, fracture strength can be predicted with good accuracy. A list of some relevant Australian references on this topic have been forwarded to Professor H.J Larsen. One important characteristic of fracture that is predicted by fracture mechanics and verified experimentally is that there is a significant effect of size on fracture strength, i.e. the larger a member the lower the nominal stress level at fracture. This size effect is not found in equation 5.1.1.4b, which is therefore conceptually wrong for the case when a notch is placed in tension and fails through fracture. Also there is no need to limit the notch depth when the design strength is derived through fracture mechanics.

Finally, our experience to date on a limited number of species of timber is that very little increase in strength is obtained through rounding notch roots.

CLAUSE 5.1.1.7

The column formula appears unnecessarily complex for a code, particularly in view of the limited data that is available for the every few species that have been tested in structural sizes. A column is a simpler element than a beam, and yet the proposed design formula is more complex. Also the column formula appears to ignore the effects of creep buckling on strength.

For parctical purposes Table 5.1.1.7 should also contain effective lengths for compression members (a) supported by flat ends and (b) supported by two bolts at each end.

CLAUSE 5.1.2

The title should read 'Tapered Beams'.

CLAUSE 5.2.0

The same comments apply here as were made for Clause 5.1.0.

CLAUSE 5.2.1

The same comments apply here as for Clause 5.1.1.4. The apparent increased severity for notches in glulam beam is probably a manifestation of the size effect that has been conceptually omitted, i.e. beams tested to derive equation (5.2.1b) were larger than those tested to obtain (5.1.1.4b).

CLAUSE 7.1.1

For the case of slender webs, the Australian code AS:1720-1975 coincides with the procedure outlined in the CIB document whereby the ultimate load is taken to be the elastic critical load. However experiments to verify this during the past year have shown this approach to be far too conservative, often by as much as a factor of 2 or 3. This is due largely to membrane action effects.

CLAUSE 7.2

The slip of nails in non linear even at very low loads and it is doubtful whether the concept of a linear slip modulus leads to realistic deformation predictions. For bolts some allowance should be made for the inital slip as a load is applied.

There appears to be no consideration of creep effects. In the Australian design code AS:1720-1975 the deformation that occurs with permanent loads is taken to be ten times that of short term loads in the case of initially green timber, and five times for the case of initially dry timber.

*** **** ***

NOTE 1

BASIC DESIGN STRESSES AND DESIGN LOADS IN AS:1720-1975
(THE AUSTRALIAN TIMBER ENGINEERING CODE)

Submitted by -
R.H. Leicester,
(CSIRO, Melbourne
Australia, 1979)

In order to make provision for structural design with the wide range of species that are utilised in Australia (and most countries around the world) it is necessary to simplify the design procedure by grouping timbers into a limited set of classes. The following are the groups used in AS:1720-1975, the Australian Timber Engineering Code:

(i) Strength Group (S1-S7 and SD1-SD8) - classification according to the strength properties of small clear pieces of timber.

(ii) Stress Grades (F2-F34) - classification according to the properties of structurally graded timber.

(iii) Joint Groups (J1-J4) - classification according to density.

As an example of the above, the two following Tables from AS:1720-1975 show the classifications for a limited number of timbers used in Australia. In publications either published or being printed by CSIRO, strength group classifications are given for roughly 500 species utilised in Australia, 100 utilised in South America, and 700 utilised in Africa. In addition the strength group properties of some 700 South-East Asian species are currently under preparation.

TABLE 1.6

RELATIONSHIP OF VISUAL STRUCTURAL GRADES TO STRESS GRADES FOR GREEN AND SEASONED MATERIAL OF SOME COMMON TIMBERS

(For a more comprehensive list of structural timbers, see Appendix B)

Species	Australian Standard	Strength Group		Stress Grade									
		Green	Seasoned	F4	F5	F7	F8	F11	F14	F17	F22	F27	F34
				Visual Grade									
AUSTRALIAN GROWN TIMBERS													
Unidentified hardwoods from —													
New South Wales — Highlands	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
Elsewhere	O82 and O83	S3						Bldg	Std	Sel.			
			SD3							Bldg	Std	Sel.	
Queensland	O82 and O83	S3						Bldg	Std	Sel.			
			SD3							Bldg	Std	Sel.	
South Australia	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
Tasmania	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
Victoria	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
Western Australia	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
Unknown	O82 and O83	S4					Bldg	Std	Sel.				
			SD4						Bldg	Std	Sel.		
UNIDENTIFIED SOFTWOODS													
Pinus spp.			SD7		Std Bldg	Sel. Bldg	Std Eng	Sel. Eng					
Imported		S6		Merch.	Sel. Merch.	Std Eng	Sel. Eng						
			SD7		Merch.	Sel. Merch.	Std Eng	Sel. Eng					
ash, alpine	O83	S4					Bldg	Std	Sel.				
			SD3							Bldg	Std	Sel.	
ash, mountain	O83	S4					Bldg	Std	Sel.				
			SD3							Bldg	Std	Sel.	
ash, silvertop	O83	S3						Bldg	Std	Sel.			
			SD3							Bldg	Std	Sel.	
blackbutt	O82	S2							Bldg	Std	Sel.		
			SD2								Bldg	Std	Sel.
fir, Douglas	O106	S5			Merch.	Sel. Merch.	Std Eng	Sel. Eng					
			SD5				Merch.	Sel. Merch.	Std Eng	Sel. Eng			
gum, red, river	O81	S5				Bldg	Std	Sel.					
			SD6				Bldg	Std	Sel.				
gum, spotted	O82	S2							Bldg	Std	Sel.		
			SD2								Bldg	Std	Sel.
jarrah	1483	S4						Str 2	Str 1				
			SD4							Str 2	Str 1		
karri	1483	S3							Str 2	Str 1			
			SD2									Str 2	Str 1
messmate	O82	S3						Bldg	Std	Sel.			
			SD3							Bldg	Std	Sel.	
pine, cypress, white	1648	S6		Bldg	Std	Sel.							
			SD7		Bldg	Std	Sel.						
pine, hoop	O107	S6			Bldg	Std	Sel.						
			SD5					Bldg	Std	Sel.			
pine, radiata	1490		SD7		Std Bldg	Sel. Bldg	Std Eng	Sel. Eng					
tallowwood	O82	S2							Bldg	Std	Sel.		
			SD2								Bldg	Std	Sel.

Legend. Visual grades defined in Australian Standards are:

Bldg	Building Grade	Std Eng	Standard Engineering Grade
Std	Standard Grade	Sel. Eng	Select Engineering Grade
Sel.	Select Grade	Str 1	Structural Grade 1
Merch.	Merchantable Grade	Str 2	Structural Grade 2
Sel. Merch.	Select Merchantable Grade	Std Bldg	Standard Building Grade
Eng	Engineering Grade	Sel. Bldg	Select Building Grade

NOTE: As each set of grading rules is converted to metric units the reference number of the standard will be changed. For example, AS 1490 is the metric version of AS O78.

TABLE 4.1.1
CLASSIFICATION OF TIMBERS FOR USE IN JOINT DESIGN

Group	Timber		
J1	ash, Crow's	gum, grey	mahogany, red
	ash, hickory	gum, Maiden's	mahogany, white
	bloodwood, brown	gum, red, forest	messmate, Gympie
	box, black	gum, salmon	penda, red
	box, grey, coast	gum, sugar	tallowwood
	box, grey	gum, yellow	tuart
	box, red	hardwood, Johnstone River	wandoo
	box, white	ironbark, grey	woollybutt
	box, yellow	ironbark, red, narrow-leaved	balau
	carbeen		chengal
	gum, blue, southern		
J2	ash, silvertop	gum, red, river	stringybark, white
	blackbutt	gum, scribbly	stringybark, yellow
	blackbutt, Western Australian	gum, spotted	turpentine
	bloodwood, red	karri	yertchuk
	bloodwood, yellow	mahogany, southern	balau, red
	box, brush	marri	kapur
	box, white-topped	oak, tulip, brown	kempas
	gum, blue, Sydney	satinay	keruing
	gum, grey, mountain	stringybark, red	kwila (merbau)
	unidentified hardwoods from Western Australia		
	unidentified hardwoods from New South Wales, other than from the highlands of that State		
J3	alder, brown	gum, rose	pine, cypress, northern
	ash, alpine	gum, shining	pine, cypress, white
	ash, mountain	gum, swamp	satinash, grey
	ash, silver, northern	jarrah	stringybark, brown
	ash, silver, southern	maple, scented	tea-tree, broad-leaved
	brownbarrel	messmate	lumbayau (mengkulang)
	candlebark	oak, tulip, red	mersawa
	gum, manna	peppermints (various)	ramin
	gum, mountain	pine, celery-top	sepetir
	unidentified hardwoods from New South Wales—highlands		
	unidentified hardwoods from Victoria		
	unidentified hardwoods from Tasmania		
	unidentified hardwoods from South Australia		
	unidentified hardwoods from Queensland		
	unidentified hardwoods from unknown origin		
J4	alder, rose	oak, silky, northern	walnut, yellow
	ash, white	pine, bunya	meranti, bakau
	fir, Douglas	pine, hoop	meranti, dark-red
	hemlock, western commercial*	pine, loblolly	meranti, light-red
	maple, Queensland	pine, radiata	meranti, white
		pine, slash	meranti, yellow

* As present marketed is mixed with a small percentage of amabilis fir.

The following are examples of design values related to the classification groups:-

TABLE 2.2.2

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

Strength Group		Compression perpendicular to grain F'_p	Shear at joints details F'_{sj}
Green	Seasoned		
	SD1	10·4	4·15
	SD2	9·0	3·45
	SD3	7·8	2·95
S1	SD4	6·6	2·45
S2	SD5	5·2	2·05
S3	SD6	4·1	1·70
S4	SD7	3·3	1·45
S5	SD8	2·6	1·25
S6		2·1	1·05
S7		1·7	0·86

TABLE 6.2

CORRESPONDENCE BETWEEN STRENGTH GROUP AND STRESS GRADE FOR ROUND TIMBERS GRADED TO AS O117

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

NOTE: The equivalence expressed in Table 6.2 is based on the assumption that poles or logs are from mature trees. This equivalence does not necessarily apply for poles taken from immature trees of all species (see Rule 1.6.4).

TABLE 2.2.1

BASIC WORKING STRESSES AND MODULUS OF ELASTICITY (MPa) FOR STRUCTURAL TIMBER

Stress Grade	Type of stress				Modulus of elasticity
	Bending F'_b	Tension parallel to grain F'_t	Shear in beams F'_s	Compression parallel to grain F'_c	E*
F34	34·5	27·5	2·45	26·0	21500
F27	27·5	22·0	2·05	20·5	18500
F22	22·0	17·0	1·70	16·5	16000
F17	17·0	14·0	1·45	13·0	14000
F14	14·0	11·0	1·25	10·5	12500
F11	11·0	8·6	1·05	8·3	10500
F8	8·6	6·9	0·86	6·6	9100
F7	6·9	5·5	0·72	5·2	7900
F5	5·5	4·3	0·62	4·1	6900
F4	4·3	3·4	0·52	3·3	6100
F3	3·4	2·8	0·43	2·6	5200
F2	2·8	2·2	0·36	2·1	4500

* Modulus of rigidity $G = \frac{E}{15}$

TABLE 5.2

BASIC WORKING STRESSES AND ELASTIC CONSTANTS (MPa) FOR STRUCTURAL PLYWOOD AT 12 percent MOISTURE CONTENT

Stress grade	Type of stress					Modulus of elasticity	Modulus of rigidity
	Bending F'_b	Tension in plane of sheet F'_t	Shear F'_s	Compression in plane of sheet F'_c	Compression normal to plane of sheet F'_p	E	G
F8	8·6	6·9	1·58	6·5	2·8	9100	455
F11	11·0	8·6	1·79	8·3	3·4	10500	537
F14	14·0	11·0	2·07	10·3	4·3	12500	620

(Comment: In the revision of AS:1720 this Table will be expanded to cover a total of 8 stress grades from F7-F34).

TABLE 4.2.1.1

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

Timber group	Basic lateral load per nail (N)						
	Nail diameter (mm)						
	2·5	2·8	3·15	3·75	4·5	5·0	5·6
J1	320	360	455	630	845	1020	1240
J2	270	320	390	530	720	865	1050
J3	230	265	330	450	605	730	885
J4	175	190	230	315	420	510	620

TABLE 4.5.2

BASIC WITHDRAWAL LOADS FOR COACH SCREWS IN GREEN TIMBER

Timber group	Withdrawal load (N/mm penetration of threaded portion) for coach screws of shank diameter (mm)					
	6	8	10	12	16	20
J1	59	67	75	83	96	108
J2	47	53	61	67	77	87
J3	33	39	45	49	57	65
J4	26	28	31	33	39	45

TABLE 4.6.2

BASIC WORKING LOADS FOR A SINGLE SPLIT-RING CONNECTOR IN GREEN TIMBER

1	2	3	4	5	6	7	8	9	10	11
Timber group	Internal diameter of ring mm	Minimum nominal thickness of timber mm		Basic load (N) per connector in single shear at angle of load to grain of —						
		Connectors in one face only	Connectors opposite in two faces	0°	15°	30°	45°	60°	75°	90°
J1	64	—	38	11 900	11 300	9680	8 320	7 320	6 720	6 540
		25	50	17 800	17 000	14 800	12 600	11 000	10 100	9 790
	102	—	50	22 900	22 200	20 500	18 700	17 100	16 000	15 800
		38	76	31 200	30 300	27 900	25 400	23 100	21 800	21 400
J2	64	—	38	9 520	9 010	7 790	6 630	5 700	5 450	5 030
		25	50	14 200	13 400	11 600	9 880	8 540	7 830	7 560
	102	—	50	19 700	19 000	16 900	14 700	13 100	12 100	11 800
		38	76	26 700	25 700	22 900	20 000	17 800	16 500	16 600
J3	64	—	38	8 900	8 300	6 920	5 630	4 760	4 250	4 140
		25	50	13 400	12 500	10 400	8 500	7 160	6 450	6 230
	102	—	50	18 000	16 900	14 500	11 900	10 100	9 120	8 900
J4	64	25	38	6 360	6 230	5 780	5 250	4 830	4 560	4 500
		—	50	8 720	8 540	7 900	7 230	6 680	6 340	6 180
		38	64	9 520	9 170	8 580	7 830	7 210	6 900	6 760
	102	—	50	13 900	13 400	12 500	11 300	10 400	9 790	9 660
		38	—	14 900	14 500	13 400	12 200	11 100	10 500	10 300
		50	76	18 400	17 800	16 600	15100	13 900	13 000	12 800

General Comment -

The above tables have been given as examples to illustrate the range of properties that must be covered by group classification. The method for assigning a specific timber to the classification groups is not critical and in Australia many methods are in use. For example in Australia the appropriate stress grade for a stick of timber may be derived through a visual, mechanical or proof grading procedures. Similarly for strength group classifications there are several methods possible, depending on the quantity and quality of data available. At one extreme there is a method that relies on a knowledge of mechanical strength properties of clear timber measured from many trees. At the other extreme there is a method suitable for situations in which only density measurements are available, and these from very few trees (less than five). Obviously the latter method will in general lead to very conservative strength grouping classifications.

NOTE 2

DESIGN RULES FOR GREEN TIMBER IN AS:1720-1975
THE AUSTRALIAN TIMBER ENGINEERING CODE.

Submitted by -
R.H. Leicester,
(CSIRO, Melbourne
Australia, 1979)

The CIB structural design code does not appear to consider the design of structures fabricated with green timber. In Australia, construction with green timber is commonly encountered and in some of our States it is the primary form in which timber is used in house construction. Further more it is not feasible to dry large sizes of solid timber prior to their use in construction.

In the Australian Standard AS:1720-1975 there are numerous rules that reflect the necessary differences in designing with dry and green timber. The following examples illustrate some of these differences.

(i) There is a difference in the strength groups and stress grades allocated.

TABLE 1.6

RELATIONSHIP OF VISUAL STRUCTURAL GRADES TO STRESS GRADES FOR GREEN AND SEASONED MATERIAL OF SOME COMMON TIMBERS
(For a more comprehensive list of structural timbers, see Appendix B)

Species	Australian Standard	Strength Group		Stress Grade									
		Green	Seasoned	F4	F5	F7	F8	F11	F14	F17	F22	F27	F34
				Visual Grade									
ash, alpine	O83	S4					Bldg	Std	Sel.				
			SD3							Bldg	Std	Sel.	
ash, mountain	O83	S4					Bldg	Std	Sel.				
			SD3							Bldg	Std	Sel.	
ash, silvertop	O83	S3						Bldg	Std	Sel.			
			SD3							Bldg	Std	Sel.	
blackbutt	O82	S2							Bldg	Std	Sel.		
			SD2								Bldg	Std	Sel.

(ii) There is an allowance related to member size to account for the effects of partial seasoning of green timber members.

TABLE 2.4.2

PARTIAL SEASONING FACTOR

Least dimension of member	38 mm or less	50 mm	75 mm	100 mm or more
Value of K_4	1·15	1·10	1·05	1·00

(iii) There is a difference on long term deflection of solid timber members.

TABLE 2.4.1.2

DURATION OF LOAD FACTOR FOR DEFLECTION

Duration of load	Average initial moisture content	Multiplying factor	
		Bending, compression and shear K_2	Tension K_3
Long duration*	above 25%	3	1·5
Long duration*	below 15%	2	1
Short duration†	any	1	1

* Long duration loading refers to a load duration of 12 months or greater.
† Short duration loading refers to a duration of 2 weeks or less.
NOTE: Creep factors for intermediate durations of 2 weeks to 1 year, and for initial moisture contents of 15 to 25 percent may be obtained by linear interpolation.

(iv) There is a difference on long term slip of joints fabricated with metal connectors.

TABLE H2

DURATION OF LOAD FACTOR K_{24}

Duration of load	Factor K_{24}			
	Nails		Bolts, split-rings and shear plates	
	Unseasoned members	Seasoned members	Unseasoned members	Seasoned members
More than 6 months	10	5	4	3
2 weeks – 6 months	3	2	2	2
5 min – 2 weeks	1·5	1·5	1·5	1·5
less than 5 min	1	1	1	1

(v) There is a difference in the design strength of metal connectors.

4.2 NAILED JOINTS — HAND MADE WITH COMMON WIRE NAILS.

4.2.1 Lateral Loads.

4.2.1.2 *Permissible loads.* Permissible loads for laterally loaded nailed joints shall be obtained by modifying the basic loads given in Table 4.2.1.1 by the following factors, as appropriate to the service conditions.

(a) *Moisture conditions.* An increase of 35 percent is permitted in the basic load for timber seasoned to below 15 percent moisture.

(vi) There is a difference in creep buckling strength.

TABLE 2.4.8

MATERIAL CONSTANT ρ FOR USE IN EVALUATING STABILITY FACTORS FOR BEAMS AND COLUMNS

Stress Grade	Basic stress MPa		Material constant ρ															
	In bending	In compression parallel to grain	Class B — Straightness										Class A — Straightness				
			Dry, where r =					Green, where r =					Dry, where r =				
	F'_b	F'_c	0	0·25	0·5	0·75	1	0	0·25	0·5	0·75	1	0	0·25	0·5	0·75	1
F34	34·5	26·0	1·23	1·25	1·18	1·14	1·22	1·35	1·36	1·25	1·17	1·12	1·14	1·16	1·10	1·08	1·07
F27	27·5	20·5	1·20	1·22	1·15	1·11	1·09	1·32	1·32	1·21	1·14	1·09	1·11	1·13	1·07	1·05	1·04
F22	22·0	16·5	1·16	1·18	1·12	1·08	1·05	1·27	1·28	1·18	1·10	1·05	1·07	1·09	1·03	1·01	1·00
F17	17·0	13·0	1·12	1·14	1·08	1·04	1·02	1·23	1·23	1·14	1·06	1·02	1·02	1·04	0·99	0·97	0·96
F14	14·0	10·5	1·07	1·09	1·04	1·00	0·98	1·18	1·19	1·10	1·02	0·98	0·98	1·00	0·95	0·92	0·91
F11	11·0	8·3	1·05	1·07	1·01	0·97	0·95	1·15	1·15	1·07	1·00	0·95	0·95	0·97	0·92	0·89	0·88
F8	8·6	6·6	1·02	1·04	0·99	0·95	0·93	1·12	1·13	1·04	0·97	0·93	0·93	0·94	0·89	0·87	0·86
F7	6·9	5·2	0·99	1·00	0·96	0·92	0·90	1·08	1·09	1·01	0·95	0·90	0·89	0·91	0·86	0·84	0·82
F5	5·5	4·1	0·96	0·98	0·93	0·90	0·88	1·05	1·06	0·99	0·92	0·88	0·86	0·88	0·84	0·81	0·79
F4	4·3	3·3	0·93	0·95	0·90	0·87	0·85	1·02	1·02	0·95	0·89	0·85	0·83	0·85	0·81	0·76	0·73
F3	3·4	2·6	0·91	0·93	0·89	0·86	0·83	0·99	1·00	0·93	0·88	0·83	0·81	0·82	0·77	0·72	0·70
F2	2·8	2·1	0·89	0·90	0·87	0·84	0·82	0·97	0·97	0·91	0·86	0·82	0·78	0·80	0·73	0·69	0·66

NOTES:

1. The definitions for Class A and Class B straightness are given in Appendix G.
2. Sawn timber, complying with the relevant grading rules, may be taken as Class B and glued-laminated timber may be taken as Class A.
3. r = (temporary load)/(total load), where the term 'temporary load' in this context refers to loads that act for a duration of less than 12 months.

NOTE 3

DESIGN OF LAMINATED TIMBER MEMBERS IN AS:1720-1975
THE AUSTRALIAN TIMBER ENGINEERING CODE.

Submitted by -
R.H. Leicester,
(CSIRO, Division of
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Melbourne, Australia
1979)

In Australia laminated timber has been fabricated with dozens of species, and concievably several hundred more will be used in the future. The complexity of having to design for multiple species is increased by the fact that frequently beams are made with a mixture of species, and that some of the laminations may contain butt-joints or finger joints therein.

In order to provide a fairly simple design procedure in AS:1720-1975, laminated timber is treated essentially as solid timber, with a few modification factors to take into account the minor effects of the laminating. The following is a brief outline of these modifications:

(i) Stiffness - Each lamination is accorded the appropriate modulus of elasticity for the stress grade of timber used, i.e. it is assumed that the stiffness of a lamination is not influenced by the laminating process.

(ii) Strength - The code AS:1720-1975 allows for two methods whereby the strength of a beam is increased through laminating. The first is a load sharing effect. For example in a beam the bottom 25-percent of laminations are assumed to take part in a load sharing operation and an increase in design stress is permitted according to the following table.

TABLE 2.4.5.1

PARALLEL SUPPORT FACTOR

Effective number of elements carrying common load	Factor K_8
1	1·00
2	1·14
3	1·20
4	1·24
5	1·26
6	1·28
7	1·30
8	1·31
9	1·32
10 or more	1·33

Another method whereby an increase in strength arises in laminating is due to the local reinforcement effect. In AS:1720-1975 this reinforcement is taken to be applicable to species having (a) low slopes of grain and (b) grading defects that are localised phenomena. The following Table shows the effect of local reinforcement according to AS:1720-1975.

TABLE 7.3.2

SPECIAL LAMINATION FACTOR

Lamination thickness mm	Factor K_{20} for special grades*			
	L1	L2	L3	L4
50	1·00	1·10	1·15	1·20
40	1·00	1·10	1·20	1·25
30	1·00	1·10	1·20	1·30
25	1·00	1·15	1·25	1·35
20	1·00	1·15	1·25	1·40
15	1·00	1·15	1·30	1·45
10	1·00	1·20	1·35	1·55
5	1·00	1·25	1·45	1·70

* Special lamination grades correspond to the following grades as defined in AS O84, AS O106, AS O107, and AS 1490, except that slope of grain is not to exceed 1 in 16:

L1 special grade = select engineering grade (radiata pine and Douglas fir) or select grade (hoop pine)

L2 special grade = standard engineering grade (radiata pine and Douglas fir) or standard grade (hoop pine)

L3 special grade = select building grade (radiata pine), or select merchantable grade (Douglas fir)

L4 special grade = standard building grade (radiata pine)

NOTE: for thicker laminations use factor for 50 mm thickness.

The local reinforcement effect has been derived through experimental measurement and the concepts of fracture mechanics. It is to be considered an alternative and not an addition to the load sharing effect.

NOTE 4

DESIGN FOR BUCKLING STRENGTH IN AS:1720-1975 THE AUSTRALIAN TIMBER ENGINEERING CODE

Submitted by -
R.H. Leicester,
(CSIRO, Division of Building Research, Melbourne, Australia, 1979)

The basic problem in choosing a suitable format for buckling strength is that not only is there a large number of stress grades of timber utilised (in AS:1720-1975 there are twelve), but also on infinite variety of structures may be built. The following describes the format adopted in AS:1720-1975 in order to reduce the design rules to manageable proportions.

First a stability factor K is defined by

$$\sigma_a = K\,\sigma_{stable} \tag{1}$$

where σ_a is the allowable design stress and σ_{stable} is the value of σ_a if the structure were completely restrained against buckling. The effects of buckling are then normalised through defining a slenderness coefficient S as follows

$$S = \alpha\sqrt{\sigma_{stable}/(\sigma_{cr}/3)} \tag{2}$$

where σ_{cr} denotes the critical elastic buckling load, and α is an arbitrarily chosen constant. (In AS:1720-1975 α has been chosen so that for pin-ended rectangular columns $S = L/D$, the length to depth ratio)

Through the use of equations (1) and (2), any relationship between σ_a, σ_{stable} and σ_{cr} may be expressed as a dimensionless equation relating K and S.

For example the equations -

$$\sigma_a = \sigma_{stable} \tag{3}$$

$$\sigma_a = \sigma_{cr}/3 \tag{4}$$

can be written -

$$K = 1 \tag{5}$$

$$K = \alpha/S \tag{6}$$

These equations are illustrated in the following Fig.1.

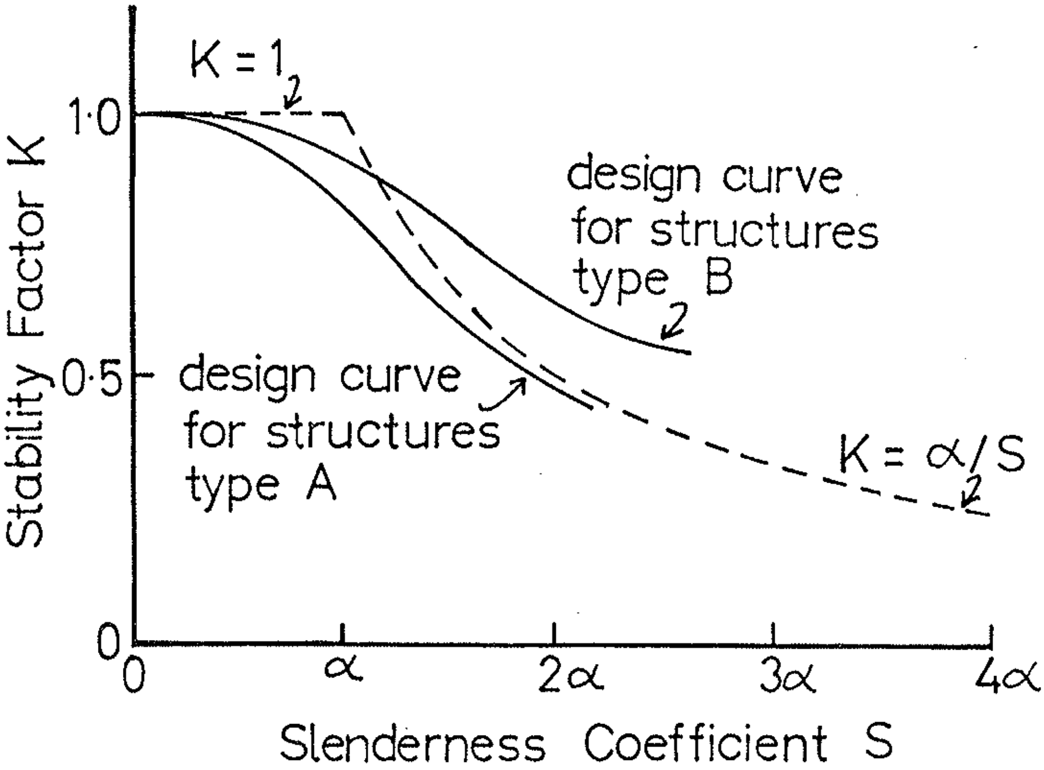


Fig.1.Relationship between stability factor K and slenderness co-efficient S.

In AS;1720-1975, a single K-S relationship is used to define the buckling strength of all timber structures constructed from the same type of timber and having the same duration of load. In the revision of AS:1720-1975, there will be three K-S relationships, one for each of the following classes of structure,

(i) columns
(ii) beams
(iii) plywood web diaphrams

This reflects the findings of recent research.

COMMENT No. 1.

From the above it follows that with the format used in AS:1720-1975, any type of slender timber structure may be designed provided an elastic critical stress σ_{cr} is first computed from conventional mechanics theory. This value of σ_{cr} is then used to derive S which in turn leads to K and hence the allowable stress σ_a. In AS:1720-1975, formulae for computing slenderness coefficients S in this way are given for the following -

(i) beams
(ii) columns
(iii) plywood webs
(iv) spaced columns
(v) arches

The formulae cover a great variety of structures, including cases where the intermediate restraints against buckling are not rigid.

An example of the wide range of structures considered is illustrated in Appendix E of AS:1720-1975, attached to the end of this Note. Appendix E gives formulae for the slenderness coefficients of beams of various types.

COMMENT No. 2.

In AS:1720-1975, the effects of long duration loads are considered not only in terms of strength, but also in terms of creep buckling. The following references contain relevant discussion on this topic:

1. Leicester, R.H. - A Rheological Model For Mechano-sorptive Deflections Of Beams. Wood Science and Technology. Vol.5, 1971 pp.211-220

2. Leicester, R.H. - Lateral Deflections Of Timber Beam-Columns During Drying. Wood Science and Technology. Vol.5, 1971 pp 221-231.

3. Leicester, R.H. - Contemporary Concepts For Structural Timber Codes. Seminar On Timber Design and Construction for the 70's, University of Auckland, New Zealand, Aug.1972.

4. Leicester, R.H. - Buckling Strength of Slender Timber Members. Lecture 10, CSIRO Seminar on Timber Engineering, 1973.

The following graph, taken from Reference 3 cited above, compares the measured effect of duration of load on column strength, with the corresponding design rules in some timber engineering codes.

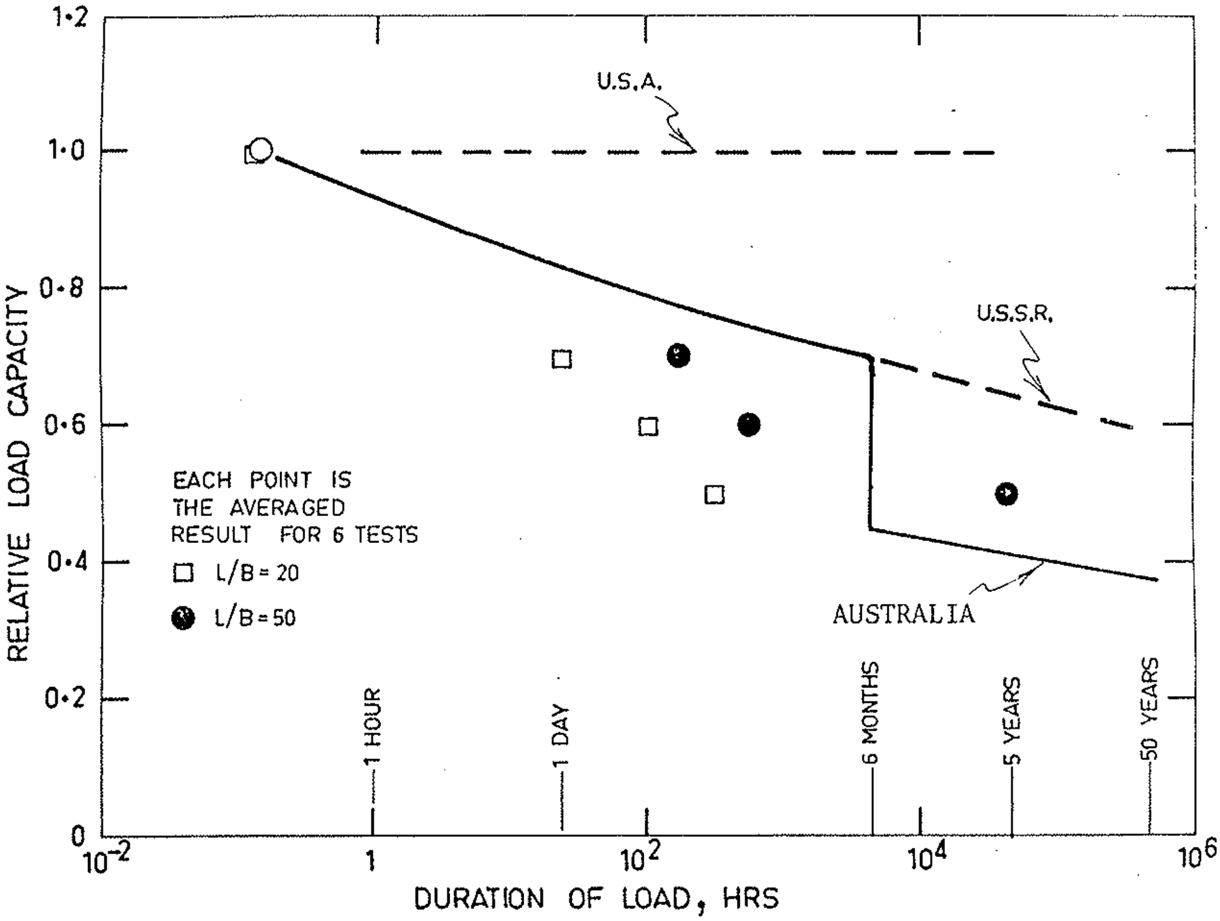


FIG 4. EFFECT OF LOAD DURATION ON COLUMN STRENGTH (L/B = 20 - 50)

NOTE - In the following the term 'Euler moment' should be read as 'critical moment'.

APPENDIX E

SLENDERNESS COEFFICIENTS FOR BEAMS

E1 GENERAL. To evaluate the stability factor K_{12} referred to in Rule 2.4.8, the slenderness coefficient S_1 of a beam shall be defined by—

$$S_1 = \sqrt{\left[\frac{1 \cdot 1(EI)_x}{M_E y}\right]} \quad \ldots \quad \ldots \quad \ldots \quad (E1)$$

where $(EI)_x$ is the stiffness in bending about the XX axis, y the distance from the neutral axis to the extreme fibre, and M_E the Euler buckling moment of the beam.

NOTE: In some odd cases, the evaluation of the above formula for a solid beam of rectangular section, can lead to a value of S_1 greater than given by the formula in Rule 3.2.3. In such a case, the value as given by Rule 3.2.3 may be used for obtaining K_{12}.

The evaluation of the slenderness coefficient requires a knowledge of M_E, the Euler buckling moment. Values of the Euler moment for particular structural situations can be obtained from standard texts on structural analysis. However, as an aid to design, some values of the Euler moment are presented in the following sections.

E2 END-SUPPORTED BEAMS.

E2.1 General. The following recommendations are applicable to end-supported beams of bisymmetrical cross-section for which the contribution of warping stiffness to the buckling strength may be neglected.

The ends of supports are assumed to be effectively restrained against twisting. This condition will be satisfied if the supports possess a torsional stiffness in excess of $20(GJ)/L$, where GJ is the torsional of the beam and L is its length.

A useful reference for information on more general sections, including the effects of warping stiffness, is the following:

NETHERCOT, D.A. and ROCKEY, K.C., 'A Unified Approach to the Elastic Lateral Buckling of Beams' *The Structural Engineer*, Vol. 49, No 7, July 1971, pp. 321–330. (For *erratum* see Vol. 51, No 4, April 1973, pp. 138–139.)

E.2.2 Beams with Intermediate Buckling Restraints. The Euler value of the maximum moment between two buckling restraints may be taken as:

$$M_E = C_5 \frac{1}{L_{ay}}\left[\frac{(EI)_y \times (GJ)}{\alpha}\right]^{1/2} \quad \ldots \quad \ldots \quad (E2)$$

where

$(EI)_x$, $(EI)_y$ = effective stiffness for bending about the major and minor axes respectively

(GJ) = effective torsional stiffness.

$$\alpha = 1 - \frac{(EI)_y}{(EI)_x}$$

C_5 = constant obtained from Table E1

L_{ay} = distance between buckling restraints.

For rectangular sections of solid wood, a conservative approximation to the value of slenderness coefficient obtained from formulas (E1) and (E2) is—

$$S_1 = \sqrt{\left(\frac{4{\cdot}8 D L_{ay}}{B^2 C_5}\right)} \quad .. \quad .. \quad .. \quad \text{(E3)}$$

E2.3 Beams with No Intermediate Buckling Restraints. For this case the Euler value of maximum moment may be taken as:

$$M_E = C_5 \frac{1}{L_{ay}}\left[\frac{(EI)_y \times (GJ)}{\alpha}\right]^{1/2} \left[1 - C_6\left(\frac{h}{L_{ay}}\right)\sqrt{\left(\frac{(EI)_y}{(GJ)\times\alpha}\right)}\right] \quad .. \quad \text{(E4)}$$

where

$(EI)_x$, $(EI)_y$, (GJ), α have the meanings defined in Paragraph E2.2 and

h = height above centroid of the point of load application

C_5, C_6 = constants obtained from Table E2

L_{ay} = span of beam.

For beams loaded only by end moments, formula (E4) may be used with $C_6 = h = 0$ and the coefficient C_5 taken from Table E1.

For rectangular cross-sections of solid wood, a conservative approximation of the value of slenderness coefficient obtained from formulas (E1) and (E4) is:

$$S_1 = \sqrt{\left[\frac{4{\cdot}8\,\frac{DL_{ay}}{B^2}}{C_5\left(1 - 2{\cdot}4 C_6 \frac{h}{L_{ay}}\right)}\right]} \quad .. \quad .. \quad \text{(E5)}$$

Formulas (E3) and (E5) are good approximations for $B \leqslant 0{\cdot}5D$.

E3 CONTINUOUSLY RESTRAINED BEAMS. For beams of bisymmetrical cross-section, continuously restrained against lateral displacement at a distance y_o below the neutral axis (see Fig. E1), the Euler moment M_E may be taken as—

$$M_E = \frac{(EI)_y\left(\frac{D^2}{4} + y_o^2\right)\left(\frac{\pi}{L_{ay}}\right)^2 + (GJ)}{\left(2y_o + \frac{8h}{\pi^2}\right)} \quad .. \quad \text{(E6)}$$

NOTE: In Table E1, the values of the coefficients C_5 and C_6 apply to beams with lateral restraints only at their end joints. However, these coefficients may be used for any other beam load system that has a similar shape of bending moment diagram between points of lateral restraint.

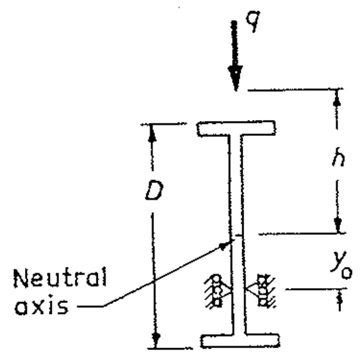
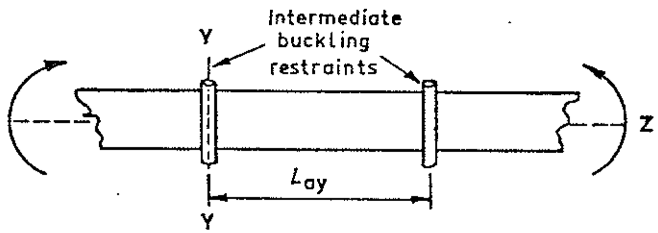


Fig. E1. CONTINUOUSLY RESTRAINED BEAM

TABLE E1

COEFFICIENTS FOR SLENDERNESS FACTOR OF BISYMMETRICAL BEAMS WITH INTERMEDIATE BUCKLING RESTRAINTS

Moment parameter β (see diagram below)	Slenderness factor C_5	
	Free restraint condition*	Fixed restraint condition*
1·0	3·1	6·3
0·5	4·1	8·2
0·0	5·5	11·1
−0·5	7·3	14·0
−1·0	8·0	14·0



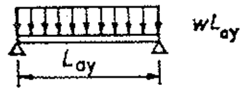
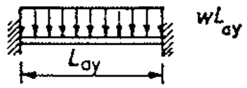
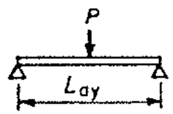
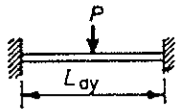
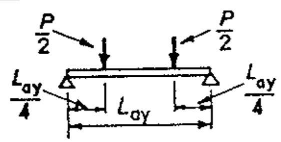
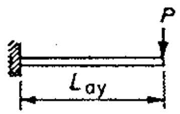
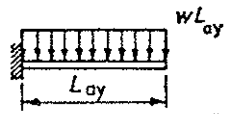
(a) SIDE ELEVATION OF BEAM

(b) DIAGRAM OF BENDING MOMENT BETWEEN BUCKLING RESTRAINTS

* The buckling restraints must prevent rotation of the beam about the ZZ axis. The terms 'free' and 'fixed' restraint condition refer to the possibility for rotation of the beam about the YY axis at the restraint locations.

TABLE E2

COEFFICIENTS FOR SLENDERNESS FACTORS OF BISYMMETRICAL BEAMS WITH NO INTERMEDIATE BUCKLING RESTRAINTS

Loading	Bending moment M	Condition of end restraint against rotation about YY axis*	Slenderness factors C_5	C_6
		Free	3·6	1·4
		Fixed	6·1	1·8
		Free	4·1	4·9
		Fixed	5·4	5·2
		Free	4·2	1·7
		Fixed	6·7	2·6
		Free	5·3	4·5
		Fixed	6·5	5·3
		Free	3·3	1·3
		Fixed	—	—
		Fixed	4·0	2·0
		Fixed	6·4	2·0

* See diagram in Table E1 (free ends of cantilevers excepted).

APPENDIX D

LATERAL AND TORSIONAL BUCKLING RESTRAINTS

The following method may be used for a design of slender beams having equally spaced buckling restraints. The restraint systems considered are either lateral or torsional ones as shown in Fig. D1, where the restraint stiffnesses K_A and K_B are defined as follows:

$$F_A = K_A \Delta_A \quad \cdots \quad (D1)$$

$$T_B = K_B \theta_B \quad \cdots \quad (D2)$$

where F_A and T_B are the restraint force and torque respectively that occur when the point of attachment of the restraint to the beam undergoes a displacement Δ_A and rotation θ_B. It is assumed that the end of beams are effectively restrained against torsional rotation (see Section E2.1).

Notation to be used in the design formulas are defined as follows:

K_{36} = 1·0 when loads are live loads only

= 1·5 when loads are deads load only and timber is initially dry

= 2·0 when loads are dead loads only and timber is initially green

NOTE: Values of K_{36} for other conditions may be obtained by linear interpolation.

K_{37} = 1·0 for sawn timber members

= 0·4 for laminated and other carefully fabricated timber members

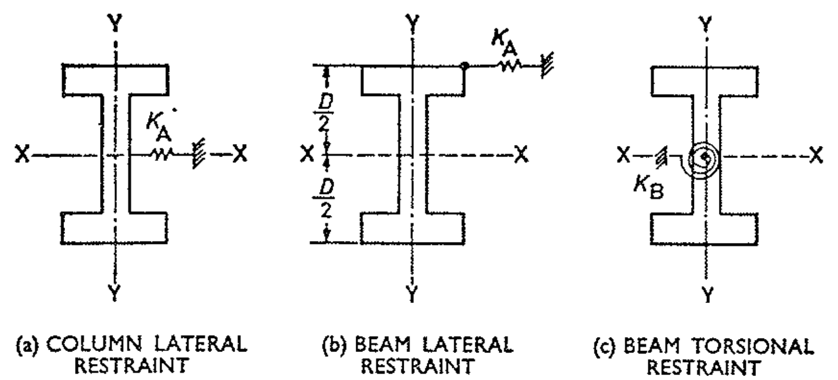


Fig. D1. INTERMEDIATE RESTRAINTS

K_{38} = lesser of $\frac{m+1}{2}$ and 5

m = number of members supported by each restraint system

n = number of equally spaced intermediate restraints

S_{max} = slenderness coefficient if there are no restraints

S_{min} = slenderness coefficient if the restraints are effectively rigid.

D2 COLUMNS.

D2.1 Load Capacity. In computing the load capacity of a column of length L with n intermediate lateral restraints as shown in Fig. D1 (a), the slenderness coefficient S_3 may be taken as —

$$S_3 = \frac{S_{max}}{(\alpha_1)^{1/4}} \quad \cdots \quad (D3)$$

but not less than S_{min} and not more than S_{max}, and where —

$$\alpha_1 = \frac{(n+1)\, K_A S_{max}^2 L}{2AE} K_{38} \quad \cdots \quad (D4)$$

D2.2 Force on Lateral Restraints. The design force F_A on the lateral restraints may be taken to be given by —

$$F_A = \frac{0{\cdot}1 P_a}{(n+1)} K_{36} K_{37} K_{38} \quad \cdots \quad (D5)$$

where P_a is the applied axial load.

D3 BEAM WITH LATERAL RESTRAINTS.

D3.1 Load Capacity. In computing the load capacity of a beam of length L with n intermediate lateral restraints as shown in Fig. D1 (b), the slenderness coefficient S_1 may be taken as —

$$S_1 = \frac{S_{max}}{(\alpha_2)^{1/6}} \quad \cdots \quad (D6)$$

but not less than S_{min} and not more than S_{max}, where —

$$\alpha_2 = \frac{(n+1)\, K_A DLS_{max}^2}{EZ_x} K_{38} \quad \cdots \quad (D7)$$

D3.2 Force on Lateral Restraints. The design force F_A, on each lateral restraint may be taken to be given by —

$$F_A = \frac{0{\cdot}2M_a}{D(n+1)} K_{36}\, K_{37}\, K_{38}, \text{ for members of rectangular section and for box beams}$$

$$F_A = \frac{0{\cdot}1M_a}{D(n+1)} K_{36}\, K_{37}\, K_{38}, \text{ for I-beams} \quad \ldots \quad \text{(D8)}$$

where M_a is the applied bending moment.

D4 BEAM WITH TORSONIAL RESTRAINTS.

D4.1 Load Capacity. In computing the load capacity of a beam of length L with n intermediate torsional restraints as shown in Fig. D1 (c), the slenderness coefficient S_1 may be taken as—

$$S_1 = \frac{S_{max}}{(1+\alpha_3)^{1/4}} \quad \ldots \quad \text{(D9)}$$

but not less than S_{min} and not greater than S_{max}, and where—

$$\alpha_3 = \frac{(n+1)\, I_y\, K_B\, S_{max}^4}{Z_x^2\, L\, E} K_{38} \quad \ldots \quad \text{(D10)}$$

D4.2 Torque on Torsional Restraints. The design torque T_B on each restraint may be taken to be given by—

$$T_B = \frac{0{\cdot}4M_a}{(n+1)} K_{36}\, K_{37}\, K_{38}, \text{ for members of rectangular section and for box beams}$$

$$T_B = \frac{0{\cdot}15M_a}{(n+1)} K_{36}\, K_{37}\, K_{38}, \text{ for I-beams} \quad \ldots \quad \text{(D11)}$$

where M_a is the applied bending moment.

NOTE 5

DESIGN AGAINST FRACTURE IN AS:1720-1975 THE AUSTRALIAN TIMBER ENGINEERING CODE.

Submitted by -
R.H. Leicester
(CSIRO, Melbourne
Australia, 1979)

The code AS:1720-1975 makes use of the concepts of fracture mechanics to derive design recommendations for situations where failure can occur through a fracture mode. Fracture mechanics is most directly applicable to members containing sharp cracks or notches. In these cases it provides a very accurate predictor of the failure load. In AS:1720-1975, the following applications of fracture mechanics are used:-

(i) Strength of notched beams (Clause 3.2.6)

(ii) Strength of glued lap joints (Clause 4.10.3.2)

(iii) Strength of butt-joints (gaps) in laminated timber (Clause 7.4.2)

Publications in which the above are discussed are given at the end of this note. One aspect of particular importance is formulating design rules related to fracture is that there is a predicted size effect on strength, i.e. the larger a member the smaller the nominal stress to cause fracture. An example of this for the case of notched beams is shown in the following figure taken from Ref. 5.

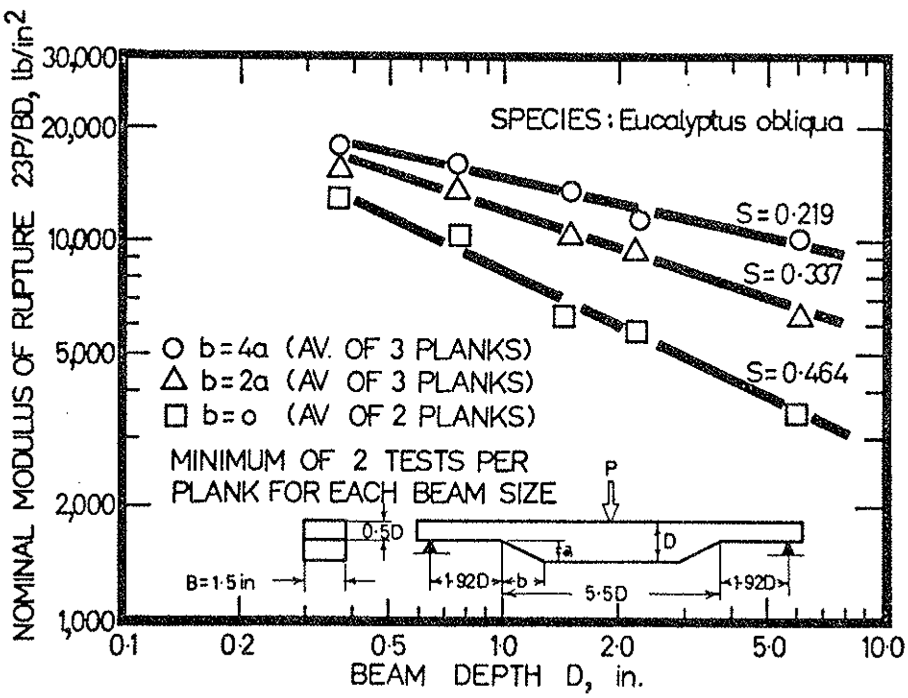


Fig. 9.--Effect of size on bending strength of notched timber beams.

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1. Leicester, R.H. and Bunker, P.C. - Fracture At Butt Joints In Laminated Pine. Forest Products Journal. Vol. 19, No.2, Feb. 1969, pp 59-60

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CIB-W18/11-100-2d

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

by

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VIENNA, AUSTRIA

MARCH 1979

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Remarks
relating to CIB - Structural Timber Design Code -
Third draft - September 1978

ad.2.1.1. When determining characteristic propierties of timber is necessary to take into account its moisture content. We suggest as a reference level $15 \pm 2\%$ of moisture content.
We propose besides to consider 25-30% of moisture content /complete saturation/ as a reference level for other characteristic propierties.

ad.2.2. We suggest the following modification:
- for climate class 1 : characteristic temperature for the climate shall be $23 \pm 3^{\circ}C$ and air humidity shall be up to 60%. The air humidity may be exceeded in 20% for a period no longer than one week.
- for climate class 2 : characteristic temperature shall be $23 \pm 3^{\circ}$ and air humidity shall be up to 80%. The air humidity can be exceeded for a period no longer than one week.

We propose to reduce the service temperature of structures from wood-based materials to $+55^{\circ}C$.

ad. 2.3. In our opinion classes of load duration shall he established as 1-5.

ad.4.1.1. We suggest to supress class SC15 and add to the class SC36 the following data: $f_m = 36$; $E_o$mean = 11 000; $f_{t,o}=24$.
Timber classified within SC15 has so poor characteristic values, that solely very unimportant structures may be made of it, but not building structures for general use.

ad.4.3.1. We propose for class SCL38 $f_m = 38$ not 37,5. The difference is not so great one, but classification based upon bending will be more clear.

ad.4.7. In table 4.7. class 0 shall be supressed.

ad.5.1.0. We consider G_{mean} from table 5.1.0.a. to high.
Actually we have not test results related with /in Poland such tests are not undertaken as yet/, but E:G ratio is circa 20 , while the given in table 5.1.0.a is 12,5.

- We propose to determine the factors from table 5.1.0.b. The factors mentioned are necessary for the formulae in the next part of the code.

ad.5.1.1. - Safety factor is not considered in the formulae. We suggest to introduce respective symbol whose value shall be included after calculations are finished. Theright side of the formulae should not include characteristic values solely.

- We propose to introduce a general factor covering a group of other factors /to be included in national codes/ such as service conditions, technical level of manufacture, particular type of building and so on.
- Clause 5.1.1.9. mentioned in 5.1.1.2 is omitted in the text received by us.

ad.5.1.1.7 - There are no conditions nor foundations to accept "e" /5.1.1.7./ for calculation of η/etha/.

- The curves as in 5.1.1.7. were drawn for $f_{c,o}/f_m$ =0,96. In our opinion the curves mentioned are not necessary for they shall be drawn for every national conditions separately / in Poland $f_{c,o}/f_m$ = 0,67/.

ad.5.2.2 - For calculation of k_t and k_m we suggest formulae instead of diagrams. Such formulae shall be prepared for separate national codes in relation with $E_o$mean/E_{90} mean ratio.

ad.7.1.1. - There is a mistake in the table number. Should be 7.1.1. instead of 7.1.1.2.

- Symbol "t" is omitted.

ad.7.1.3. - We suggest to refer to Annex 7A.

- In Annex 7B in calculation of battens the influence of bending moment was not taken into consideration. /Paper 8-102-1 clause 4.9.8. PN-73/B-03150/.

ad.7.2. - There is a mistake in table 7.2. We suppose that there should be $0{,}02E_{od}$ instead of $0{,}002\ E_{od}$.

CIB-W18/11-100-2e

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE
(List of Contents)

by

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THE NETHERLANDS

VIENNA, AUSTRIA

MARCH 1979

78/32
cie 3510103

WORKING GROUP W 18
TIMBER STRUCTURES

CIB TIMBER CODE
CRITICISM OF THIRD DRAFT - sept 1978
PART A - LIST OF CONTENTS

NNI Netherlands Rijswijk
P - Member ISO/TC 165 Timber structures

W.R.A. Meyer november 1978

List of Contents

Introduction

Within the frame work of preparing a new Dutch Timber Code, succeeding NEN 3852 "Regulations for the calculation of building structures. Timber structures", the sub-committee "TGB-Hout" sent in December 1977 a List of contents (nr. 77/55) to the secretariat of CIB-W18-Timber Structures.
This List of contents differs rather much from document CIB-W18/8-100-1, List of contents, Bruxelles-October 1977.
To discuss the comments, received on the CIB-W18 documents the Code Drafting Sub-Committee held a meeting at Prinses Risborough U.K. at 13-14 March 1978. In the meeting appeared, that the List of Contents (77/55) did not have been distributed between the members, through that a good discussion was not possible.
After that the following remarks have been made by the secretariat:

"Jan Kuipers (Technische Hogeschool, Delft) and W.R.A. Meyer (TNO)

A complete draft of a list of contents is given.
The draft has a very systematic structure, however, its deviation from all existing timber codes and the previous CIB-drafts is so strong that it was not thought possible to take it into consideration at the present time. This applies to both form and volume (among others there is a section on loads) and to the importance applied indirectly to the individual subjects through the subdivision. If the draft is maintained it is suggested to discuss it at the next CIB-W18 meeting".

Comments on the remarks of the secretariat of CIB-W 18

1 - "deviation from all existing timber codes"
This is in principle not an argument. Every time when codes change, it is difficult to set free the reliable "own" Code.

2 - "deviation from the previous CIB-drafts"
It is known that, with a view to harmonising the construction codes and regulatons of the members countries of the European Communities, the Council for Mutual Economic Assistance and the Nordic Building Regulation Committee, in conjunction with the Economic Commission for Europe, of the United Nations Organisation and the International Standardisation Organisation, the so called Unified Rules will be established.
At this moment the first two volumes of this group of codes of practice are in fact being published:

a. Volume I "Common Unified Rules for different Types of Construction and Material"
b. Volume II "CEB/FIP Model Code for Concrete Structures".

All referred previous CIB-drafts differ much from the approach, formulated in the volumes I and II. Draft 77/55 conforms well to Volume II. Timber structures will be Volume VI.

3 - "form and volume"
As already mentioned in the Preface of 77/55, the choosen framework of the code, is conformed to the user, the timber designer; all information, data, procedures etc. must be arranged and must be available in accordance with the sequence of calculation of the designer.
Moreover the numbering of the different parts has so been choosen that there is good accordance between correspondin

chapters to make it easier the user.
For "Nails" for instance, the following data are given:

201.3.1 Material specifications
204.3.1 Characteristic strength
205.3.1 Deformations for different load duration classes
206.3.1 Design value with stress-strain diagram
601.3.1 Regulations in relation to the design and workmanship.
701.3.1 Principles for basic test methods.

Starting point is, in any chapter, there are only references to previous chapters.

4 - "section on loads"
About loads there must be references to local codes. Regulations, especially for timber structures, must be mentioned, for example: "load sharing of concentrated floor loads": see 203.1.1.

To show the lay-out of a possible new CIB-Timber-Code, the whole contents of the First Draft, 76.04.20 and Third Draft, 78.01.06, has been divided over the corresponding chapters in accordance with 77/55. At the meeting in Princess Risborough the (concept) resulting report has been distributed.
At the CIB-W18-meeting in June 1978 at Perth, a more extensive draft has been delivered consisting of:

- List of Contents : see page 12
- CIB-Timber-Code : see page 21
- Calculation example : see page 37

The example shows the systematic structure and the comfort for the practising engineer.

As already mentioned in the Preface of 77/55 this concept has the benefit lacks in knowlegde and insight will be faster recognized; moreover the boundary conditions of future research will be clearly formulated. The choice is still free between a detailed code and a brief code with only basic information, completed with documentations concerning special applications and/or subjects.

To explain the train of thought followed in our previous reports, in this report only the numbering, in accordance with 77/55, will be given for the Chapters 2, 4 and 5.

Correspondence ISO/TC165 - CIB/W18 - subcommittee 35101003

As mentioned in the Introduction of this report, subcomm. 3510103 of P Member of ISO/TC 165 (NNI, Rijswijk, Netherlands) has choosen a frame work of the code, conformed to the user, the timber designer; all information, data, procedures etc. must be arranged and must be available in accordance with the sequence of calculation of the designer

Canada agrees with this approach.
References:

- Report ad hoc meeting on the organization of TC 165, held Tuesday 28 September 1976 in Copenhagen.
  ISO/TC 165 N7E 1976-11-04 page 7.
  "Canada found that drafting committees should be set up inside the technical committee. In CIB W 18 there might be too much weight on "research and science".
  Canada would therefore like to have the work of W 18 edited in an ISO group which would include the views of the "users" of the resulting standard".
- Remarks on the report, dated 1977-01-31 ISO/TC 165 N11 page 1.
  Canadian Comments on ISO/TC 165 N8E. Canada wishes to reiterate our statement made at ad hoc meeting of 1976-09-28.
  "We have serious reservations about entrusting the drafting work for TC 165 to CIB-W 18 because:
  (1. not appropriate)
  2. Members of CIB-W 18 are largely drawn from research establishments and universities.
     Practicing design or consulting engineers do not appear to be represented. There should be a greater input from users of the Code which can, we think, only be achieved by a properly balanced committee".

On the TC 165 meeting held 22 and 23 September 1977 in London, the decision has been taken Canada should take an active part in the drafting work of CIB W 18. Reference. ISO/TC 165 N 27 E page 3 item 8.1.

On the same meeting -see item 8.2- the appointment has been made "to send an outline of content of the code to the members of TC 165, who are invited to comment on the content.
If necessary, the secretariat may however first convene a meeting of the committee to discuss the comments".

As mentioned in the Introduction of this report, the sub-committee "TGB-hout" 3510103 of the NNI, sent in December 1977 a list of Contents to the Secretariat of CIB-W 18 - Timber Structures.

The subcommittee are very surprised about the following letter of the Secretariat of ISO/TC 165 1978-2-17 CL - 14 E page 2:
"The secretariat has received the proposal accepted by CIB W 18 (Timber Structures) for a list of contents for a CIB Timber Code. This we enclose as document ISO/TC 165 N 33 E".

It is the expectation of the sub-committee a real discussion about the List of Contents is still possible.

Short description of list of contents

Chapter 1 : General regulations
All sorts of regulations must be formulated exactly to start the design of timber structures. The necessary information speaks for themselves, no further comment.

Chapter 2 : Assumptions for the calculation.
The first information, necessary for the designer, is a survey of the available materials.
In 201 four groups are specified: primatic wood elements, sheet materials, mechanical fasteners and other material.
For each group are given definitions, shape and dimensions, corresponding grading rules etc.
As in many cases the material behaviour is depending on the moisture content, in 202 the moisture classes will be defined its consequences.
For the magnitude of the characteristic loading, acting the structure, has to be referred to the Local Codes. In this code only information will be given about load sharing for concentrated loads and load duration classes. The characteristic strength (204) referred to the moisture content (202) and the duration of load (203) is the next information necessary for the designer.
Characteristic values are given, corresponding with the material specification of 201.

In the same way, the deformations are given in 205. As in some codes calculations will be carried out based on design values of strength and loading in stead of characteristic values, addition information will be given in 206 en 207 resp. At last information will be given about scheme of the structure (208), construction parts and constructions (209) and theoretical span (210).
In fact all boundary conditions of the structure are defined now.

Chapter 3 : Calculation internal force distribution and ultimate load-carrying capacity of the structure.
After general information about theories (301), empirical methods (302), experimental investigations (303) accurate data will be given about beams (304), plates (305) and columns (306).
For many types of structures, formula's are giving to calculate the internal force distribution and the ultimate load-carrying capacity of the different cross sections.
The necessary basic information for this is given by the shape and dimensions of the structure and by information given in 201, 204 and 205.

Chapter 4 : Judgement of the structure.
For each of the limit states the margin between the actual characteristic load and the ultimate load acting on the structure, will be given in principle in local codes.
All information is now available to calculate the design values of the internal forces in the cross-section

Chapter 5 : Design of basic members and components
To compare the design values of the stresses with the design values of the strenght (206), formula's are given to calculate these stresses corresponding with axial force, bending moment, shear force etc.
Like in chapter 3, information will be given for beams (504), plates (505) and columns (506).

Chapter 6 : Regulations in relation to the design and workmanship.
This chapter contains conditions and information about minimum allowable timber thickness for roofs, maximum allowable distances between beams for floors, maximum allowable moisture content of timber for structures to be glued etc.

Chapter 7 : Materials
Principles for basic test methods are given. References to national or international codes can be given as well.

Chapter 8 : Workmanship
The necessary information speaks for themselves.

LIST OF CONTENTS

CHAPTER 1: GENERAL REGULATIONS

CHAPTER 2: ASSUMPTIONS FOR THE CALCULATION

202 Moisture classes
- 202.0 General
- 202.1 Service requirements
- 202.2 Effect on strength
- 202.3 Effect on geometrical properties

203 Characteristic loading
- 203.0 General
- 203.1 Live loads
  - 203.1.1 Load sharing for concentrated floor loads
  - 203.1.2 Load sharing for concentrated roof loads
- 203.2 Load duration classes

204 Characteristic strength

205 Deformations for different load durations classes

206 Design value for the strength with corresponding stress-strain diagram (deformability)

207 Design value for the loading

208 Scheme of the structure
- 208.0 General
- 208.1 Effective flange width of beams, loaded by bending moment
- 208.2 Effective flange width of beams, loaded by bending moment and compression
- 208.3 Effective (plate) width of plates, loaded by concentrated loads

209 Construction parts and constructions

210 Theoretical span

CHAPTER 3: CALCULATION INTERNAL FORCE DISTRIBUTION AND ULTIMATE LOAD-CARRYING CAPACITY OF THE STRUCTURE

301 Theories
- 301.0 General
- 301.1 Theory of linear elasticity
  - 301.1.1 Basic principles and assumptions
- 301.2 Theory of non-linear elasticity
  - 301.2.1 Basic principles and assumptions
- 301.3 Theory of plasticity
  - 301.3.1 Basic principles and assumptions

302 Empirical method

303 Experimental investigations

304 Beams
- 304.0 General
- 304.1 Prismatic solid members
  - 304.1.0 General
  - 304.1.1 Coëfficient for beams over more than two supports loaded by equal divided loading to calculate bending moments and shear forces.
  - 304.1.2 Lateral buckling
- 304.2 Composed solid timber (I,T or Box Cross section)
  - 304.2.0 General
  - 304.2.1 $[EI]_{eff.}$, $|EA|_{eff}$, $|GI_t|_{eff}$
    - 304.2.1.0 General
    - 304.2.1.1 Cross section type 1 | for | 1 nails
      - 2 " " " 2 | | 2 screws
      - etc. etc. | | 3 bolts
      - 4 connector
      - 5 nail plates
      - 6 glue

304.2.2 Bending moments and shear forces
304.2.3 Lateral buckling (instability)
304.3 Spaced beams with packs or battens
304.4 Lattice beams
304.5 Thin-webbed beams
304.6 Thin-flanged beams (stiffened plates)
305 Plates
306 Columns
306.0 General
306.1 Prismatic solid members
306.1.0 General
306.1.1 Slenderness ratio λ, effective slenderness ratio λ_e
306.1.1.1 Boundary conditions
306.1.1.2 Effective length l_e
306.1.2 Initial and second order excentricity
306.1.3 Axial force, shear forces and bending moments
306.1.4 Lateral buckling (instability)
306.2 Composed solid timber (I,T or Box crosssection)
306.2.0 General
306.2.1 $[EI]_{eff}$, $[EA]_{eff}$, $[GI_t]_{eff}$
306.2.1.0 General
306.2.1.1 Cross secion type 1 | for 1 nails
2 2 | 2 screws
etc. etc. | 3 bolts
4 connectors
5 nail plates
6 glue
306.2.2 Slenderness ratio λ, effective slenderness ratio λ_e
306.2.3 Initial and second order excentricity
306.2.4 Axial force, shear forces and bending moments
306.2.5 Lateral buckling (instability)
306.3 Spaced columns with packs or battens.

CHAPTER 4: JUDGEMENT OF THE STRUCTURE

CHAPTER 5: DESIGN OF BASIC MEMBERS AND COMPONENTS

504.1.1.0 General
504.1.1.1 Tension
504.1.1.2 Compression
504.1.1.3 Bending
504.1.1.4 Shear
504.1.1.5 Tension and bending
504.1.1.6 Compression and bending
504.1.1.7 Tursion
504.1.2 Deformation (deflection)
504.1.3 Durability
504.1.4 Fire resistance
504.1.5 Vibration
504.2 Composed solid timber
504.3 Spaced beams with packs or battens
504.4 Lattice beams
504.5 Thin-webbed beams
504.6 Thin-flanged beams

505 Plates
505.0 General
505.1 Plywood
505.2 Particle boards
505.3 Fibre board

506 Columns
506.0 General
506.1 Prismatic solid members
506.2 Composed solid timber
506.3 Spaced columns with packs or battens
506.4 Lattice columns

507 Arches, portals and frames

508 Trusses

CHAPTER 6: REGULATIONS IN RELATION TO THE DESIGN AND WORKMANSHIP

601 Materials
- 601.0 General
- 601.1 Prismatic wood elements
  - 601.1.0 General
  - 601.1.1 Solid timber
    - 601.1.1.1 Floor boarding
    - 601.1.1.2 Roof boarding
  - 601.1.2 End-glued timber
  - 601.1.3 Glued laminated timber
- 601.2 Sheet materials
  - 601.2.0 General
  - 601.2.1 Plywood
  - 601.2.2 Particle board
  - 601.2.3 Fibre board
- 601.3 Mechanical fasteners
  - 601.3.0 General
  - 601.3.1 Nails
  - 601.3.2 Screws
  - 601.3.3 Bolts and dowels
  - 601.3.4 Connectors
  - 601.3.5 Nail plates
- 601.4 Other materials
  - 601.4.0 General
  - 601.4.1 Glue

602 Supports

603 Stability

604 Beams

605 Plates

606 Columns

607 Arches, portals and frames

608 Trusses

CHAPTER 7: MATERIALS

701 Principles for basic testmethods;
Characteristic strengths and design values
Stress-strain relations
701.0 General
701.1 Prismatic wood elements
701.2 Sheet materials
701.3 Mechanical fasheners
701.4 Other materials

CHAPTER 8: WORKMANSHIP

801 General
802 Fabrication
803 Transport, storage, erection
804 Treatments
805 Inspection

CIB Timber Code

Chapter 2

201.0 General

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)

701.1 Prismatic wood elements
701.2 Sheet materials

Characteristic strength and stiffness values directly applicable to a very short-term load condition (3 to 5 mins.) at a temperature of 23 ± 3°C and relative humidity of 0.60 ± 0.02.

201.0 General

Where the characteristic values are estimated from a limited number of tests the estimate shall be made with a confidence level of 0.75.

701.1.1 Solid timber

The characteristic strength values are related also to

- a section depth of 200 mm for the bending strength of solid timber,

701.1.3 Glued laminated timber

The characteristic strength values are related to

- a section depth of 300 mm for the bending strength of glued laminated timber,

701.1.1 Solid timber

The characteristic strength values are related to

- a volume of 0.02 m³ for the tensile strength perpendicular to grain.

201.0 General

The characteristic specific gravity for a species or species group is defined as the lower 5-percentile value

701.0 General

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

201.0 General

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

202.0 General

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

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

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

Climate class 3
All other climatic conditions.

203.2 Load duration classes

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

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

Table 2.3a Load-duration classes

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

Chapter 4

204.0 General
205.0 General

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

401.3.3 Limit state in relation to durability

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

201.1.0 General

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

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

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

701.1.0 General

The test specimens shall contain a grade-determining defect - preferably knots - in the zone with maximum force or bending moment.

201.1.1 Solid timber

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

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

Table 4.1.1 Standard strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

standard strength class		SC15	SC19	SC24	SC30
bending	f_m	15	19	24	30
bending	$E_{0,mean}$	6000	7200	8500	10000
tension	$f_{t,0}$	6	9	16	20

601.1.2 End-glued timber

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

201.1.2 End-glued timber

Strength and stiffness parameters shall be determined according to section 4.1.0 coupled with the rules ir $E_{0,mean}$ and the characteristic values for f_m and $f_{t,0}$ are not less than given in table 4.1.1.

Finger jointed structural timber can be referred to one of the standard strength classes stated in 4.1.1 if $E_{0,mean}$ and the characteristic values are not less than given in table 4.1.1.

601.1.3 Glued laminated timber

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

201.1.3 Glued laminated timber

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

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

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

Table . .1 Standard glulam strength classes. Characteristic strengths and mean modulus of elasticity, in MPa

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

701.2 Sheet materials

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

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

For particle board and fibre board:

601.4.1 Glue

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

601.3.0 General

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

Table 4.7 Minimum protection against corrosion

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

Chapter 5

204.1.1 Solid timber.

Characteristic values for the standard strength classes defined in section ~~4.1.1~~ 201.1.1 are given in table 5.1.0 a. For the load-duration classes and climate classes defined in sections ~~2.2~~ 205.6 and ~~2.3~~ 202.0 the factors in table 5.1.0 should be applied.

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

		SC15	SC19	SC24	SC3(
characteristic values (for strength calculations)					
bending	f_m	15	19	24	30
tension parallel to grain	$f_{t,0}$	6	9	16	20
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	14	18	23	28
compression perpendicular to grain	$f_{c,90}$	6	7	7	7
shear*	f_v	2.5	3	3	3
modulus of elasticity	E_0	4200	5400	6900	8000

205.1.1 Solid timber

mean values (for deformation calculations)					
modulus of elasticity, parallel	$E_{0,mean}$	6000	7200	8500	1000
modulus of elasticity, perpendicular	$E_{90,mean}$	250	300	350	400
shear modulus	G_{mean}	500	600	700	800

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

204.1.0 General

Table 5.1.0 b Modification factors *Provision*

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

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

Values in parantheses apply to tension perpendicular to grain.

504.0 General

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

210 Span

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

601.0 General

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

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary for nails and screws with a diameter of 5 mm or less.

504.1.1.1 Tension

The stresses shall satisfy the following conditions:

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

for tension parallel to the grain direction, and

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

for tension perpendicular to the grain, and

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

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

504.1.1.2 Compression

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

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

cf. fig. 5.1.1.2 a.

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

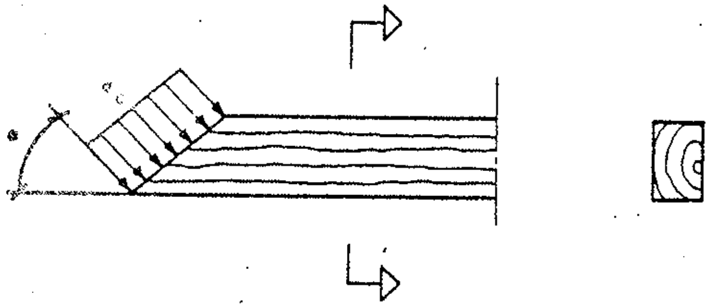
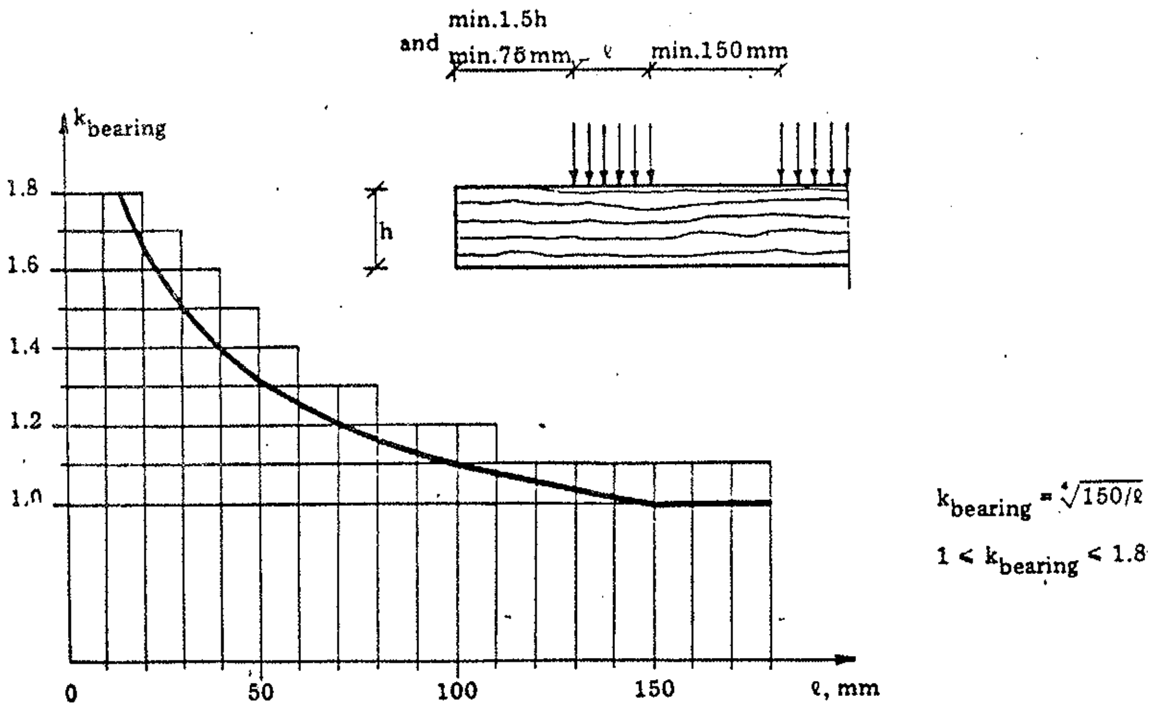


Fig. 5.1.1.2 a

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

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

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



504.1.2 Deformation

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

504.1.1.3 Bending

The bending stresses, σ_m, calculated according to the theory of elasticity shall satisfy

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

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

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

The value of κ depends on among other things the wood species and the grading rules. Recommendations will be produced.

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

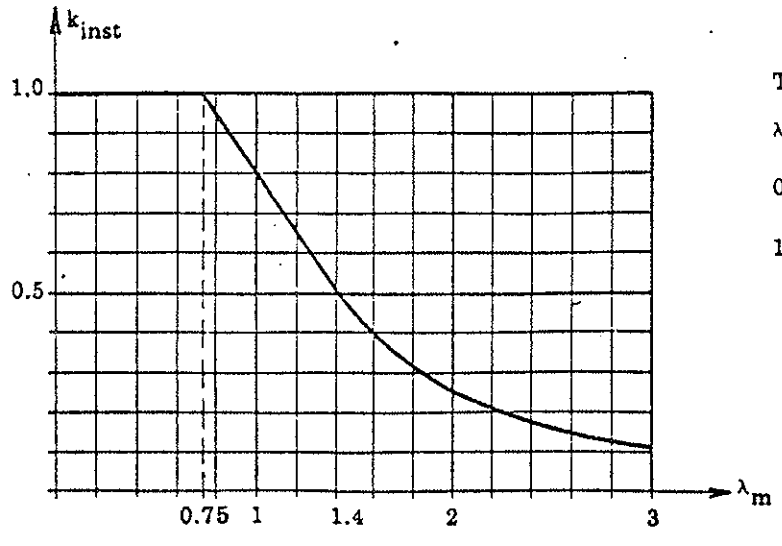
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

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

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

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

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



The curve corresponds to

$\lambda_m < 0.75$	;	$k_{inst} = 1$
$0.75 < \lambda_m < 1.4$	;	$k_{inst} = 1.56 - 0.75\,\lambda_m$
$1.4 < \lambda_m$	;	$k_{inst} = 1/\lambda_m^2$

Fig. 5.1.1.3

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

304.1.2 Lateral buckling

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

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

The free length is determined as follows:

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

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

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

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

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

504.1.1.4 Shear

The shear stresses, τ, calculated according to the theory of elasticity shall satisfy the following condi

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

For beams with bearing in the bottom side and load on the top, loads placed nearer than the beam de from the support may be disregarded in calculation of the shear force.

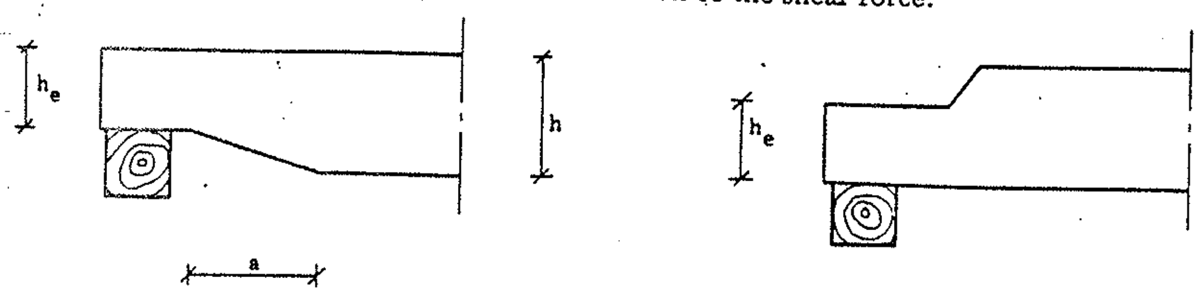


Fig. 5.1.1.4

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

$$\tau \leq \left(\frac{h_e}{h} + \frac{a}{3h}\right) f_v \qquad (5.1.1.4\ b)$$

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

504.1.1.7 Torsion

5.1.1.5 Torsion

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

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

504.1.1.8 Combined stresses

At present no general theory of rupture exists, but only empirical or semi-empirical expressions for the most important practical cases, some of which are given below.

Plane stress

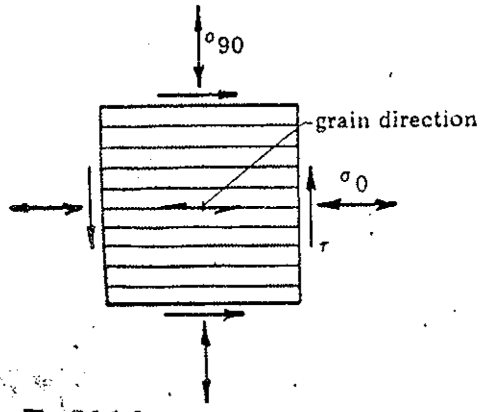


Fig. 5.1.1.6

The stresses shown in fig. 5.1.1.6 should - unless otherwise stated (see e.g. 5.1.1.2a) - satisfy the following condition:

$$\left(\frac{\sigma_0}{f_0}\right)^2 + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leq 1 \qquad (5.1.1.6\ a)$$

f_0 and f_{90} are chosen according to the sign of the σ_0 and σ_{90}, respectively. If σ_0 is a bending stress then $f_0 = f_m$.

a: Tension and bending

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

The stresses should satisfy the following condition

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

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

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

b: Compression and bending

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

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

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

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

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

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

c: Torsion and shear

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

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

5.6.0 General

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

5.6.1.1.1 Compression and bending

These conditions can be assumed satisfied if the stresses satisfy the following condition:

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

σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_{col} and k_E are factors depending on the slenderness ratio λ, the material parameters and the initial curvature.

The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

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

where r_{core} is the core radius.

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

where E_0 is the characteristic value of modulus of elasticity

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

σ_E is the Euler stress.

306.1.1 Slenderness ratio λ, effective slenderness ratio λ_e

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

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

Table 5.1.1.7 Relative effective length of compression members

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

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

504.1.1.3 Bending

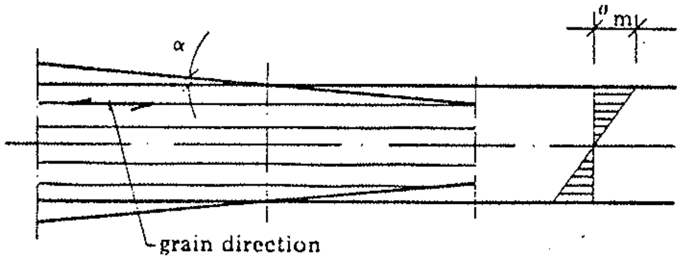


Fig. 5.1.2

Where there is an angle α between the grain direction and the top or bottom of the beam the bending stresses, σ_m, calculated as usual should in accordance with formula (5.1.1.6a) satisfy the following conditions:

Tension side:

$$\frac{\sigma_m}{f_m} \leqslant \frac{1}{\sqrt{1 + \left(\frac{f_m}{f_v}\tan\alpha\right)^2 + \left(\frac{f_m}{f_{t,90}}\tan^2\alpha\right)^2}} \tag{5.1.2a}$$

Compression side:

$$\frac{|\sigma_m|}{f_m} \leqslant \frac{1}{\sqrt{1 + \left(\frac{f_m}{f_v}\tan\alpha\right)^2 + \left(\frac{f_m}{f_{c,90}}\tan^2\alpha\right)^2}} \tag{5.1.2b}$$

204.1.3 Glued laminated timber

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

Table 5.2.0 Characteristic values and mean elastic moduli, in MPa *Provisional*

		SCL30	SCL38	SCL47
Characteristic values (for strength calculations)				
bending	f_m	30	38	47
tension parallel to grain	$f_{t,0}$	20	25	30
tension perpendicular to grain	$f_{t,90}$	0.75	0.75	0.75
compression parallel to grain	$f_{c,0}$	28	36	45
compression perpendicular to grain	$f_{c,90}$	7	7	7
shear*	f_v	3	3	3
modulus of elasticity	E_0	8000	9600	9600

205.1.3 Glued laminated timber

Mean values (for deformation calculation)				
modulus of elasticity parallel to grain	$E_{0,mean}$	10000	12000	12000
modulus of elasticity perpendicular to grain	$E_{90,mean}$	400	500	500
shear modulus	G_{mean}	800	1000	1000

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

504.2.1. Ultimate limit state : see 504.1.1

504.2.1.3 Bending : Straight beams

Section 5.1.1 for solid timber applies except that formula (5.1.1.3b) should be replaced by

$$k_{depth} = \begin{cases} 1 & \text{for } h \leq 300 \text{ mm} \\ \left(\frac{300}{h}\right)^{\kappa} & \text{for } h > 300 \text{ mm} \end{cases} \qquad (5.2.1a)$$

504.2.1.4 Shear : Straight beams

and formula (5.1.1.4b) by

$$\tau \leq [1 - 2.8 \frac{h - h_e}{h}(1 - \frac{a}{14(h - h_e)})]f_v \qquad (5.2.1b)$$

and notches with $h_e < 0.75h$ are not allowed.

50 4.2.1.1 Tension : Cambered beams.

5.2.2 Cambered beams

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

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

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.2.2 a}$$

with

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

that the tensile stresses perpendicularly to the grain satisfy the condition 5.1.1.1 b, i.e.

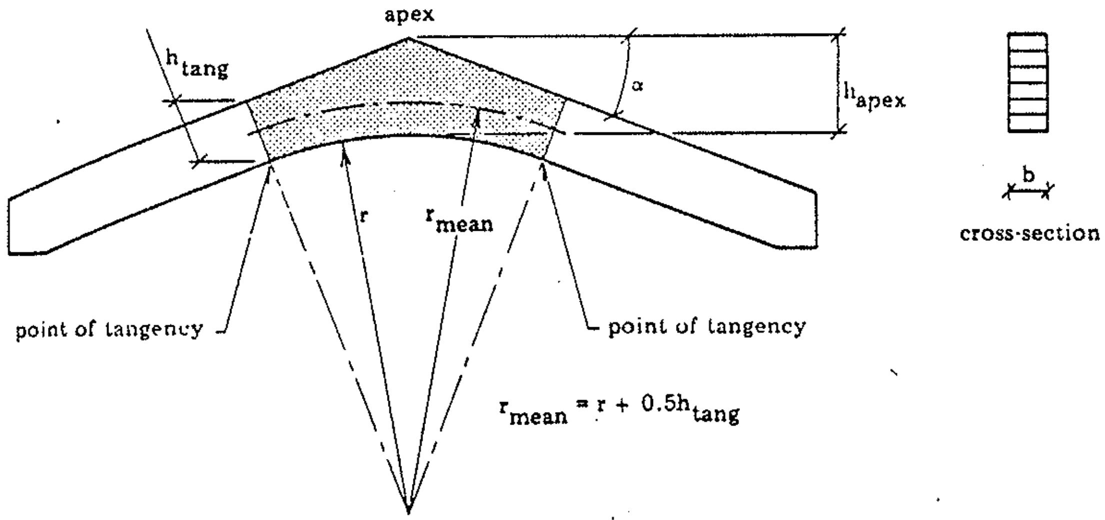


Fig. 5.2.2 a Double tapered curved beam

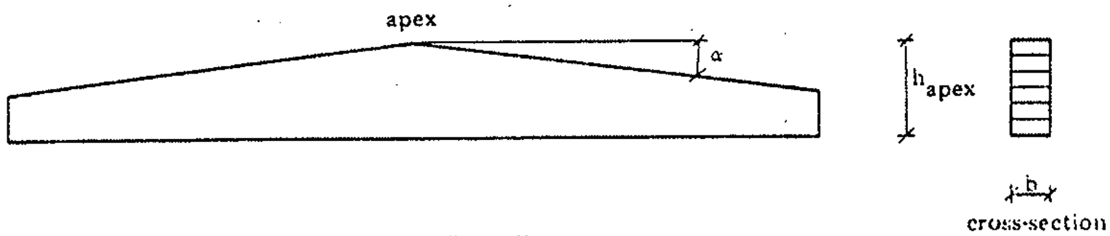


Fig. 5.2.2 b Double tapered beam with flat soffit

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

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

504.2.1.3 Bending : Cambered beams

Where there is an angle between the grain direction and the top or bottom the bending stress should satisfy the conditions in section 5.1.2.

504.2.0 General : Curved beams:

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

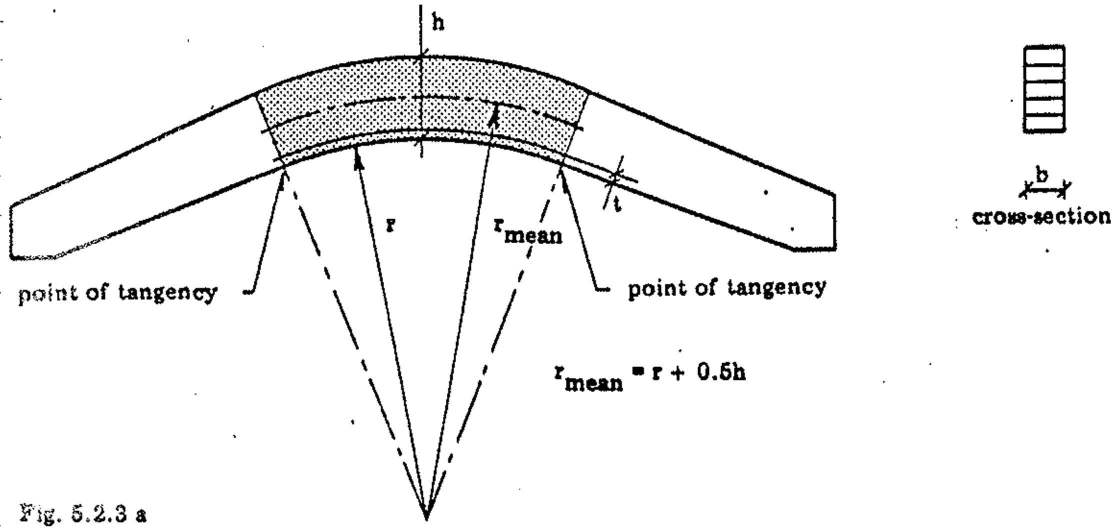


Fig. 5.2.3 a

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

504.2.1.3 Bending : Curved beams.

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.

504.2.1.1 Tension : Curved beams.

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

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

where

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

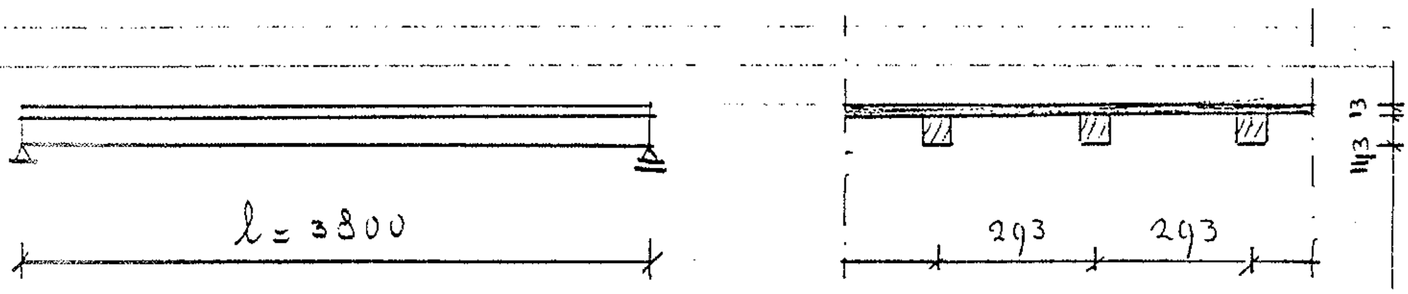
V is the volume of the curved part of the beam (corresponding to the shaded area in fig. 5.2.3 a).

Calculation example

Calculation example

To check the sequence of the Timber-Code, the following calculation exemple has been made.

Scheme of the structure:



1 General regulations

108 Units : mm, N

2 Assumptions for the calculation

201 Material specifications

201.1.1 Ribs: prismatic solid timber 30×143 mm^2
construction timber

201.2.1 Upper flange: Canadian Oregon Pine Plywood
$t = 13$ mm

201.4.1 Glue

202 Moisture classes

202.0 Climate class 1

203 Characteristic loading

203.0 Permanent loading : 500 N/m^2

Live loading : 1500 N/m^2

Live load, concentrated over an area of

$0.5 \times 0.5\ m^2$: 3000 N

203.1 Load sharing concentrated load over

$\frac{500}{293} + 1 = 2.7$ ribs

Concentrated load, acting one rib is thus:

$\frac{3000}{2.7} = 1110\ N$

204 Characteristic strength

204.1.1 Solid timber : Ribs

$f_{\sigma d} = 15\ N/mm^2$

$f_{\sigma t} = 15$ "

$f_{\tau} = 1.5$ "

204.2.1 Sheet material : Plywood

$f_{\sigma d //} = 7\ N/mm^2$

$f_{d \perp} = 4.5$ "

$f_{\tau p} = 2.5$ "

204.4.1 Glue

$f_{\tau_r} = 0.5\ N/mm^2$

205 Deformations

205.1.1 Solid timber : Ribs

$E = 11000\ N/mm^2$

$G = 500$ "

205.2.1 Sheet material : Plywood

$E_{d//} = 7000\ N/mm^2$

$E_{b_1//} = 8000$ "

$E_{b_1\perp} = 2500$ "

$G_{t//} = 750$ "

205.4.1 Glue

Displacement modulus K is $13\ N/mm^2$

206/207

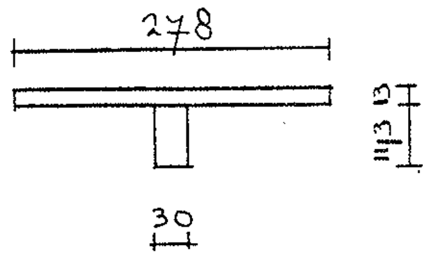
208 Scheme of the structure

208.1 Effective flange width b_e :

$$\frac{\pi}{2} \cdot \frac{b}{l} \cdot \sqrt{\frac{E_{d//}}{G_{t//}}} = 0.37$$

by graph follows: $\frac{b_e}{b} = 0.95$

$b_e = 278$ mm.



209

210 Theoretical span

$l = 3800$ mm

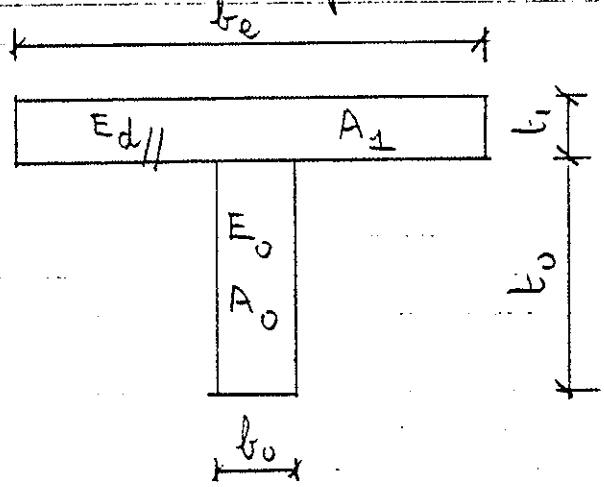
3 Calculation internal force distribution and ultimate load-carrying capacity of the structure.

301/302/303

304 Beams

304.6 Thin-flanged beams

304.6.1.1 Cross section type 1



304.6.1.1.6 Glue

Corresponding axial stiffness follows from:

$$[EA]_v = \frac{E_0 . A_0 \quad E_{d//} \quad A_1}{E_0 . A_0 + E_{d//} \quad A_1} = 16.5 \times 10^6 \text{ N}$$

Corresponding bending stiffness follows from:

$$p = \frac{\pi^2}{l^2} \cdot \frac{1}{b . K_v} \cdot [EA]_v = 0.03$$

the co-operation factor follows from:

$$\gamma = \frac{1}{1+p} = 0.97$$

The effective bending stiffness follows from:

$$[EJ]_w = \Sigma [EJ]_e + \gamma . t^2 [EA]_v = 161 . 10^9 \text{ N/mm}^2$$

4 Judgement of the structure

401 Limit states of the structure

401.3.1 Ultimate limit state

Permanent loading: $q_{dp} = 1.5 \times 500 = 750\ N/m^2$
$= 750 \times 10^{-6}\ N/mm^2$

Live loading: $q_{dq} = 1.5 \times 1500 = 2250\ N/m^2$
$= 2250 \times 10^{-6}\ N/mm^2$

$F_{dq} = 1.5 \times 1110 = 1665\ N$

401.3.2 Limit state in relation to deflection

Permanent loading: $q_{dp} = 500\ N/m^2$
$= 500 \times 10^{-6}\ N/mm^2$

Live loading : $q_{dq} = 1500\ N/m^2$
$= 1500 \times 10^{-6}\ N/mm^2$

$F_{dq} = 1110\ N$

Maximum deformation of the thin-flanged beam : $0.003 \times l = 11.4\ mm$
Maximum deformation of the plywood between the ribs: $\frac{1}{600} \times b = 0.49\ mm$

5 Design of basic members and components

501/502/503

504 Beams

504.6 Thin-flanged beams

504.6.1 Ultimate limit state

- Ribs :

$$M_{d(p+q)} = \frac{1}{8} \times (750 + 2250) \times 10^{-6} \times 293 \times 3800^2 = 158.7 \times 10^4 \text{ N.mm}$$

$$M_{dF} = \frac{1}{4} \times 1665 \times 3800 = 158.1 \times 10^4 \text{ N.mm}$$

$$D_{d(p+q)} = \frac{1}{2} \times (750 + 2250) \times 10^{-6} \times 293 \times 3800 = 1670 \text{ N}$$

$$D_{dF} = \frac{1}{2} \times 1665 = 833 \text{ N}$$

$$\sigma_{dd} = \frac{M_{d(p+q)}}{[EI]_w} \left[z \cdot \frac{[EA]_v}{E_0 A_0} - \frac{t_0}{2} \right] = 4.25 \text{ N/mm}^2 < f_{\sigma d} (=15$$

$$\sigma_{dt} = 11.25 \text{ N/mm}^2 < f_{\sigma t} = 15 \text{ N/mm}^2$$

$$\tau_d = \frac{D_{d(p+q)}}{2[EI]_w} \left[\left(\frac{\gamma \cdot z \cdot [EA]_v}{E_0 A_0} \right)^2 + \left(\frac{t_0}{2} \right)^2 \right] = 0.33 \text{ N/mm}^2 < f_{\tau} (=1.5$$

- Plywood :

$$M_{d(p+q)} = \frac{1}{8} \times (750 + 2250) \times 10^{-6} \times 293^2 = 32.2 \text{ Nmm/m}$$

$$\sigma_{d\,d//} = \frac{M_{d(p+q)}}{[EJ]_w} \cdot \chi \cdot \frac{[EA]_v}{A_1} = 1.57\ N/mm^2 < f_{\sigma d//}\ (=7)$$

$$\sigma_{d\,d\perp} = \frac{M_{d(p+q)}}{W} = \frac{32.2}{\frac{1}{6} \cdot 1 \cdot 13^2} = 1.14\ N/mm^2 < f_{\sigma\perp}\ (=4.5)$$

$$\tau_{dp} = \frac{D_{d(p+q)} \cdot \gamma \cdot \chi \cdot [EA]_v}{[EJ]_w} \frac{(b_e - b_0)}{2A_1} = 0.45\ N/mm^2 < f_{\tau p}\ (=2.5)$$

$$\sigma_{dp} = \frac{\pi^2\, t_1^2}{6 \cdot b_0^2} \sqrt{E b_{1//} \cdot E b_{1\perp}} = 16.1\ N/mm^2$$

- Glue

$$\tau_{d_n} = \frac{D_{d(p+q)} \cdot \gamma \cdot \chi\, [EA]_v}{b_{01} \cdot [EJ]_w} = 0.26\ N/mm^2 < f_{\tau_n}\ (=0.5)$$

504.6.2 Deformation

- Beam :

$$M_{d(p+q)} = 105.8 \times 10^4\ N.\ mm$$

$$M_{d\,F} = 105.8 \times 10^4\ N.\ mm$$

$$\delta_{(p+q)} = 0.104\ \frac{M_{d(p+q)} \cdot l^2}{[EJ]_w} = 9.9\ mm$$

CIB-W18/11-100-2f

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

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VIENNA, AUSTRIA

MARCH 1979

COMMENTS ON CIB STRUCTURAL TIMBER DESIGN CODE (THIRD DRAFT)

P Beckman and R Marsh

Ove Arup & Partners
United Kingdom

March 1979

GENERAL

It is most unfortunate that Section 3: BASIC DESIGN RULES is not included in this draft. This omission makes it practically impossible to make meaningful comments on the document as a whole. In some of the later sections stresses are compared with "strengths", but it is not clear whether the stresses are calculated from "characteristic loads", "factured characteristic loads" or "statutory loads". Similarly there is no indication of whether the "strengths" in the formulae are the "characteristic" or "design strengths" ($= \frac{1}{\gamma_m}$ x "characteristic"). In fact, there is no mention of whether a "partial safety factor" format is envisaged, or not.

NOTATIONS

The use of capital "V" for both shear force and volume is confusing. The subscript "inst" for "instability" could lead to confusion with "instantaneous" as applied to loadings and hence, possibly to forces in struts.

The subscript 'm' for bending is not immediately clear. If "buck is permitted for buckling, why not "bend" or "b" for bending? (Germ: "biegung", Scand: "bøjning").

2.1.1 CHARACTERISTIC VALUES

Presumbly the proportion of structural timber which is machine-graded is increasing. Machine-grading tests stiffness in bending about the minor axis. It therefore seems a complication to define the characteristic strength as related to a depth of 200 mm, eg bending about the major axis.

2.2 CLIMATE CLASSES

The definitions and the examples appear at variance with each other: "structures in outer walls in permanently heated buildings are quite likely to experience temperatures below 20°C; a reasonable range would seem to be 15°-25°C or 20°C ± 5°C NOT 23° ± 3°C. Similarly the examples quoted in Climate class 2 will during winter months experience temperatures down to say, -5°C, which would make for a range of 10° ± 15°C. (What effect does temperature variations between -5° and 25°C have on mechanical properties anyway?)

TABLES 2.3a and 2.3b

Very few grandstands are loaded for more than 4 out of every 24 hours, on a regular basis. It therefore seems too onerous to classify their load as "normal" ie $10^3 - 10^5$ hrs/50 years, as there must be some recovery during the 20 "unloaded" hours every day, and also because normal grandstand loadings include an allowance for dynamic effects, which are of very short duration.

REQUIREMENTS FOR MATERIALS

4.0 General

The first paragraph is too vague and all-embracing, suggesting a testing regime for each individual structure, however standard the material and the use might be.

4.1.1 Standard Strength Classes

This concept will be useful, subject to the grades corresponding to what can be obtained in practice from commercially available timber. For the most commonly available species there should be some "deemed-to-satisfy" provisions to eliminate the need for tensile tests on machine stress-rated timber.

Table 4.7

What is Climate Class 0? Clause 2.2 has classes 1,2 and 3 only.

Table 5.1.0a

The ratio between the strengths parallel to and perpendicular to the grain might be different for hardwoods and softwoods and even between species. Should this not be mentioned and possibly different tables given.

Table 5.1.0b

These modification factors appear NOT to be $\gamma_{Material}$ -factors, but only factors dealing with environment and load duration. This should be stated and a reference made to the clause (presummably in section 3) dealing with safety factors.

5.1.1.7

Where is the slenderness ratio λ defined? Is it equal to

$$\frac{\text{effective length}}{\text{least cross-sectional dimension}} \quad \text{or} \quad \frac{\text{effective length}}{\text{radius of gyration}} \quad ?$$

Table 7.1.2

It is, off hand, difficult to see why particleboard or fibreboard should be subject to less shear lag and hence qualify for a $b_{f,e}/l$ which is twice that for plywood.

Table 7.2

According to this the slip modulus for a 4.99 mm nail is 0.0098 E_o and for a 5.01 mm nail it is 0.1 E_o : there appears to be an error of a factor of 10 somewhere.

8.3 Joints

Is a washer diamter equal to 3 bolt diameters always adequate?

A washer thickness of one tenth of the diameter seems rather an onerous requirement.

General

The "disclosure" of the origin of the code provisions on the left hand pages is to be highly commended. It should be extended to cover all the formulae etc. It should be strongly recommended to all international and national code committees (BSI please note!)

CIB-W18/11-100-2g

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE

(Centrically and Eccentrically Loaded Construction Elements)

by

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MARCH 1979

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REPORT

Nr. B-79-146/62.4.2100
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CIB TIMBER CODE

CRITICISM OF THIRD DRAFT - sept. 1978

part B - centrically and eccentrically
loaded construction elements

Ir. W.R.A. Meyer

January 1979

CONTENTS

1 INTRODUCTION

The fundamental concepts of the safety of structures have advanced considerably in the course of the last few decades. To ensure an adequate and appropriate treatment of safety and serviceability, whatever the material or type of construction, a common basis has been given for setting up codes for the design and construction of buildings and civil engineering structures in Volume I of the Internation System of Unified Standard Codes of Practice. This volume I "Common unified rules for different types of construction and material" has been completed in November 1976 by the united efforts of the Inter-Association Joint Committee for Structural Safety, co-ordinated and led by CEB.
The aim of design is the achievement of acceptable probabilities that the structure being designed will not become unfit for the use for which it is intended.
A structure, or part of a structure is considered unfit for use, when it exceeds a particular state, called a limit state, beyond which it infringes one of the criteria governing its performance or use.
The following limit states have been defined:

a. the ultimate limit states, which are those corresponding to the maximum load-carrying capacity (ULS)
b. the serviceability limit states, which are related to the criteria governing normal use and durability (SLS).

In the Limit State design process, a calculation model should be established resulting in an equation which has on one side the probable resistance of the material, and on the other one the effect of the factored probable design loads of the structural environment.
This paper will pay attention to the resistance side of the equation for centrically and eccentrically loaded

columns, based on the "real" behaviour of the material. It will be clear that the pure bending case is included in this general approach.
The calculation method, described in Structural Timber Design Code of Working group W18, Timber Structures of CIB [1], for tension/compression and bending with/without column effect, has to my opinion two principle objections.

a. The relation between stress and strain is for tensile and compression not a constant value (see chapter II). The relation between axial force N and bending moment M is therefore not a straight line:

$$\frac{\sigma_{t(c)}}{f_{t(c)}} + \frac{\sigma_m}{f_m} \neq 1$$

(see chapter VII) and the calculation of a bending stress as M/W (W is the moment of resistance) gives no more than a reference value.
Due to bending and axial forces, only tensile and compression stresses will arise. The magnitude of the axial force and the bending moment and their relative position, define which strength, the compressive or the tensile strength, will be reached at first.

b. As deformations and thus second order effects influence the load carrying capacity, the problem will not be limited to a cross section calculation; by instability the magnitude of N (or M) will be limited, before the fracture strenght of the material (compression or tension) will be reached (see chapter V).

The approach described in this paper, is resulting in graphs giving the relation between the first order eccentricity and the corresponding ultimate compressive force for different values of the slenderness ℓ/h.

In the following chapters the calculation method will be explained and illustrated. The results are directly suitable for the designer. For two different stress-strain diagrams, the corresponding N - M relation has been calculated and compared with the CIB method [1, chapter 5 pages 6 and 7]. In relation to the here proposed method, the CIB results are sometimes unsafe (10 to 30%) and sometimes too conservative (40 to 60%).
By experimental research and by calculation of the results the here described method must be verified of course.

II MATERIALS

As mentioned in the introduction in this paper, the stress-strain relationship for compression and tensile is the basis for the calculation of the load carrying capacity.
The shape and numerical values of this relation will be influenced by many factors.
Baumann [2] and Kollmann [3] showed the reduction in strenght and stiffness when the direction of the acting force makes an angle with the direction of the wood fibres.
Graf [4] and other research-workers found a long-term strength of about 50 to 60% of the short-term strength.
The percentage of moisture has a very great influence on strength and stiffness [5, p. 59, 66], [6], [7, p. 320 - 324]
Numerical values for the stress-strain relations are given in [5, p. 49, 56, 57], [7], [8], [9, p. 83], [10, p. 786, 787, 791].
The influence of creep and relaxation [11], [12] can directly introduced in the diagrams.
The here choosen diagram of the stress-strain relation, as given in fig. 1, is simple and gives a rather good approximation of the real behaviour of timber.
The stress-strain relation for tensile is about linear-elastic resulting in a straight line in the graph.
For compression, on the other hand, the relation is

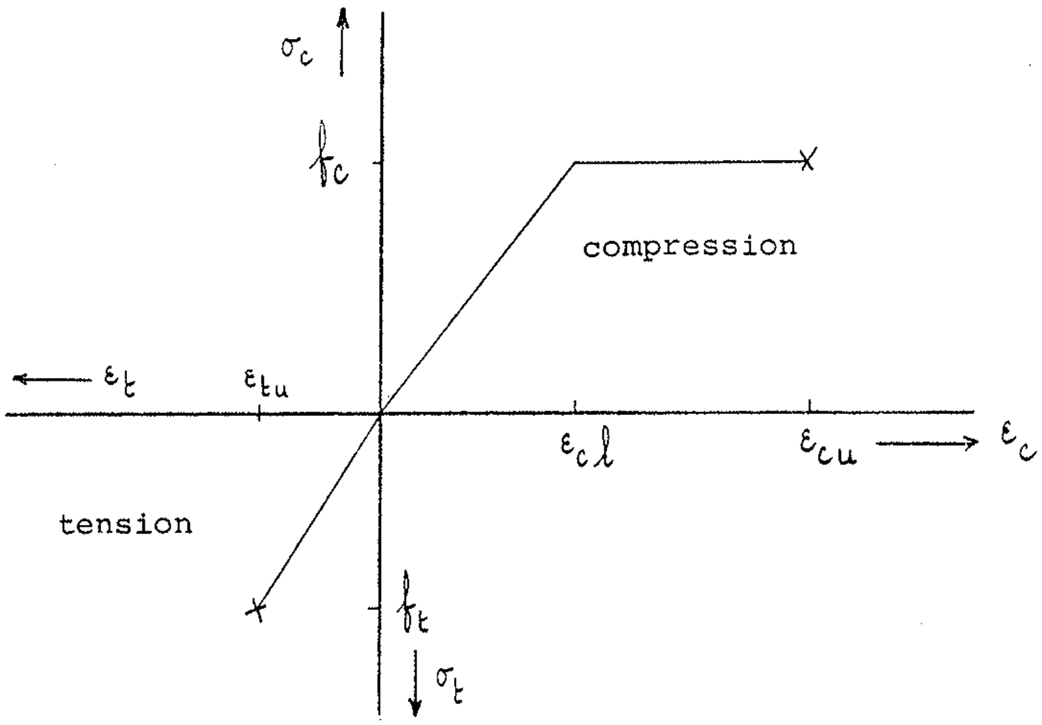


Fig. 1
Diagram stress-strain relation of timber

about linear-elastic until stresses of about 40 to 50 percent of the strength. Above these values, the deformation is non-linear, partly visco elastic, partly plastic.
A rather good approximation of these phenomena will be represented by a bi-linear stress-strain relationship (see fig. 1).
Values for f_c and f_t should correspond with some acceptable probability of occurence, e.g. 1 or 5 percent values usually will be choosen. Calculations are mostly based on the normal distribution; Pierce [13] showed however that the Weibull distribution is sometimes necessary to avoid the possibility of having negative 5th or 1st percentile values.
The calculation of the internal force distribution in construction elements, loaded by bending and axial force, will be based on the $\sigma - \varepsilon$ - relation of the material. The deformations and thus the stiffness is of great importance for the load-carrying capacity of the structure; these deformations will be governed by the stiffness properties of the construction element as a whole.

The load carrying capacity will be depend on the local strength in a decisive cross-section and by the mean stiffness of the whole construction element.
In the stress-strain relation, the strenght of the timber must be related to a lower boundary value.
As can be concluded from the fore going, the E-modulus is less critical and some value between the mean and e.g. 5 percent lower boundary can be taken.

III BEHAVIOUR OF CENTRICALLY AND ECCENTRICALLY LOADED CONSTRUCTION ELEMENT

Möhler [14] showed by tests, that the imperfections of a construction element, loaded by compression, can be taken into account by introducing an initial eccentricity for an assumed straight construction element.
This initial eccentricity e_o is assumed to be constant over the length of the element.

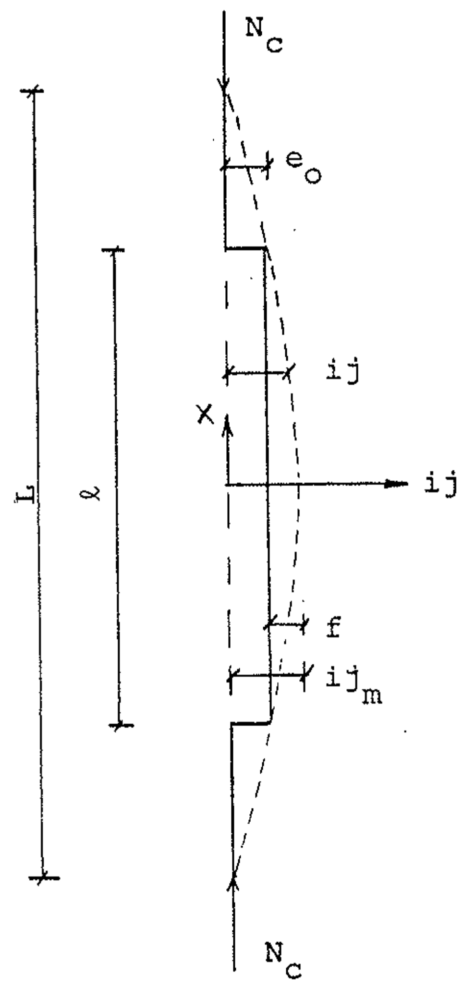


Fig. 2
Deformation of an eccentrically loaded column with constant initial eccentricity along the axis [15].

In [15] the influence is calculated of the deformations of a construction element on the internal force distribution for an eccentrically loaded column; see fig. 2.
A load N_c is applied to this column with an eccentricity e_o.
Owing to this the column is assumed to have a resulting displacement curve which is part of a sine wave; the amplitude is ij_m (x = 0) and the half-wave length is L. The length of the column is ℓ. The deformation ij follows from:

$$ij = ij_m \cos \frac{\pi x}{L} = (e_o + f) . \cos \frac{\pi x}{L} \qquad ...(1)$$

For x = 0 follows:

$$\frac{d^2 ij}{dx^2} = -ij_m . \frac{\pi^2}{L^2} = -(e_o + f) . \left(\frac{\pi \upsilon}{\ell}\right)^2 \qquad ...(2)$$

where in:

$$\upsilon = \frac{\ell}{L} \qquad ...(3)$$

In each cross section of the column the curvature κ follows from:

$$\kappa = \frac{1}{\rho} = \frac{d^2 ij}{dx^2} = - \frac{\varepsilon_1 - \varepsilon_2}{h} \qquad ...(4)$$

where in:

ε_1 = strain in least compressed or most pulled fibre
ε_2 = strain in most compressed fibre ...(5)

From (2) and (4) follows:

$$\frac{ij_m}{h} = \frac{e_o + f}{h} = \frac{(\varepsilon_1 - \varepsilon_2) . \lambda^2}{\pi^2 . \upsilon^2} \qquad ...(6)$$

where:

$$\lambda = \frac{\ell}{h} \qquad \ldots(7)$$

For $x = \frac{1}{2}\ell$ in (1), follows from $ij = e_o$:

$$e_o = ij_m \cdot \cos\frac{\pi\upsilon}{2} = (e_o + f) \cdot \cos\frac{\pi\upsilon}{2} = (e_o + f)\Theta \qquad \ldots(8)$$

where $\Theta = \cos\frac{\pi\upsilon}{2}$ ...(9)

$$f = e_o \left(\frac{1 - \Theta}{\Theta}\right) \qquad \ldots(10)$$

Substituting (10) in (6) gives

$$\frac{f}{h} = \frac{(e_1 - \varepsilon_2)\,.\,\lambda^2}{\frac{\pi^2\,\upsilon^2}{1 - \Theta}} \quad \text{while} \quad \frac{e_o}{h} = \frac{(\varepsilon_1 - \varepsilon_2)\,.\,\lambda^2}{\frac{\pi^2\,\upsilon^2}{\Theta}} \qquad \ldots(11)$$

Consider two special cases:

- For $\upsilon = 1.0$, thus for $\ell = L$, $e_o \to o$, $\Theta \to o$

  From (11) follows:

$$\frac{ij_m}{h} = \frac{f}{h} = \frac{(\varepsilon_1 - \varepsilon_2)\,.\,\lambda^2}{\pi^2} \approx \frac{(\varepsilon_1 - \varepsilon_2)\,.\,\lambda^2}{10} = 0.1\kappa h \left(\frac{\ell}{h}\right)^2 \qquad \ldots(12^a)$$

- For $\upsilon \to o$, thus for $\ell << L$, $e_o \to \infty$

  From (11) follows:

$$\frac{f}{h} = \frac{(\varepsilon_1 - \varepsilon_2)\,\lambda^2}{8} = 0.125\;\kappa\;h\;\left(\frac{\ell}{h}\right)^2 \qquad \ldots(12^b)$$

This last loading case with $\upsilon \to o$ and $e_o \to \infty$ (bending without axial force) gives the same expression for the deformation as the deformation of a column, only loaded by a constant bending moment M over its whole length ℓ:

$$\frac{f}{h} = \frac{m \cdot \ell^2}{8EI\ h} = -\frac{d^2ij}{dx^2} \cdot \frac{\ell^2}{8h} = \frac{(\varepsilon_1 - \varepsilon_2)}{8} \left(\frac{\ell}{h}\right)^2$$
$$= 0.125\kappa \cdot h \cdot \left(\frac{\ell}{h}\right)^2$$

The calculation of these two extreme loading cases, $\upsilon = 1.0$ and $\upsilon \to o$, shows that the ratio between de values of deformation f for a centrically loaded column and a column, only loaded by a bending moment amounts from 0.1 to 0.125 (8 to 10).

Lateron reference will be made to these values. In fig. 3 the schematic relation between N_c, ε_1 and ε_2 has been given.

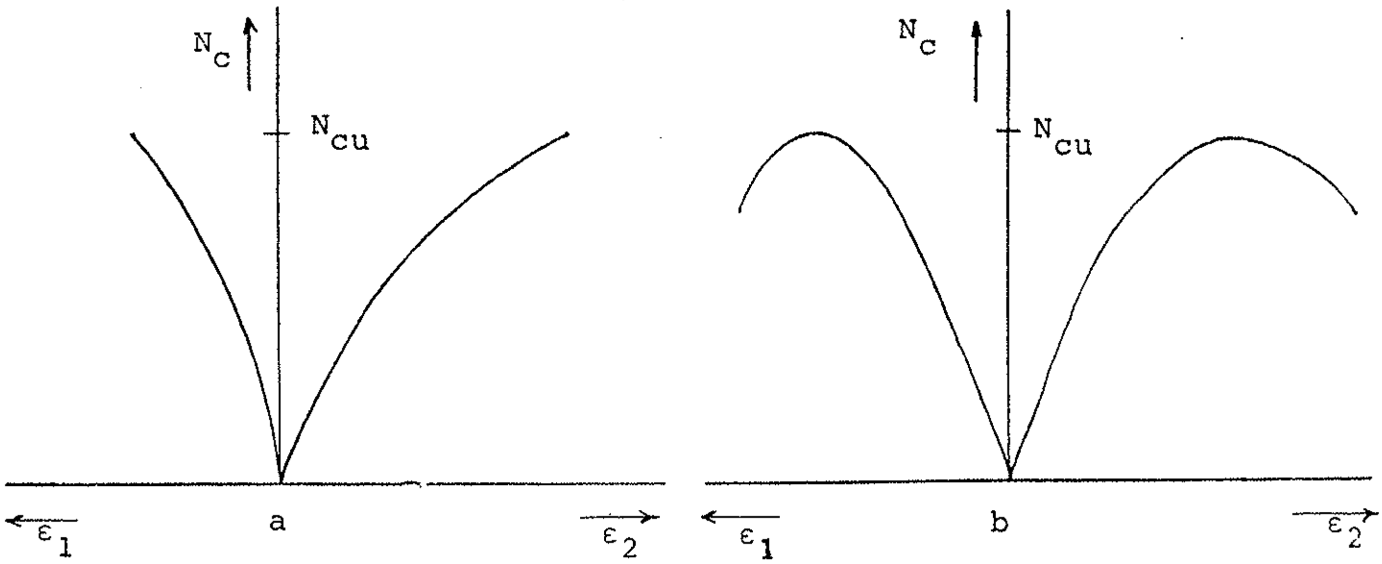


Fig. 3
Schematic relation between N_c, ε_1 and ε_2

The two possibilities are given.
In the cross section half-way the column of fig. 2 (x = o), the ultimate axial load N_{cu} will be limited by reaching the

maximum shortening ε_{cu} in the most compressed fibre or bij reaching the maximum extension ε_{tu} in the most pulled fibre of the cross section; see fig. 3^a. The collapse of the construction part will be introduced by reaching the strength of the material.
In fig. 3b the ultimate axial load N_{cu} will not be fixed when ε_1 or ε_2 reaches the values ε_{tu} and ε_{cu} resp. At the moment the axial force N_c is maximum, the strength of the material has not been reached. In this case instability governs the maximum carrying capacity.
The collapse criterium, strength or instability, is i.a. dependent on length of the construction element, initial eccentricity e_o, slenderness, boundary conditions and bending stiffness (stress strain diagram).
Bij means of calculations, the relation between ultimate axial force N_{cu} as function of the initial eccentricity e_o, can be fixed for a slenderness $\frac{\ell}{h}$ equal to 0, 10, 20 etc, resp. see fig. 4.

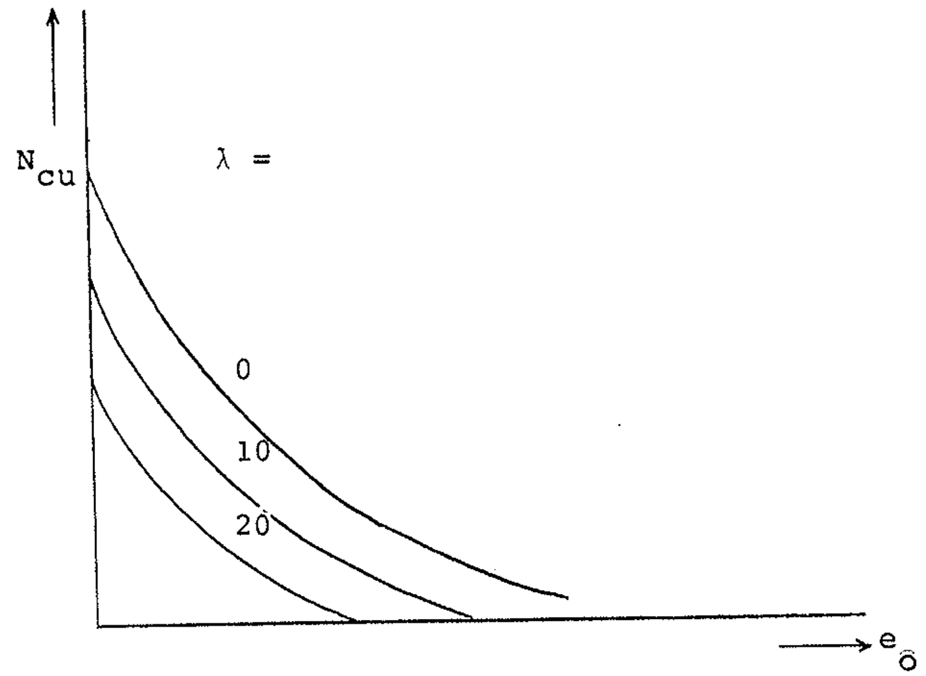


Fig. 4
Relation between N_{cu} and e_o for several values of λ.

IV CALCULATION METHOD

In this chapter a procedure will be discussed to calculate the load carrying capacity of columns loaded by an axial force at first, with a constant initial eccentricity and secundly with a linear declining eccentricity see fig. 5.

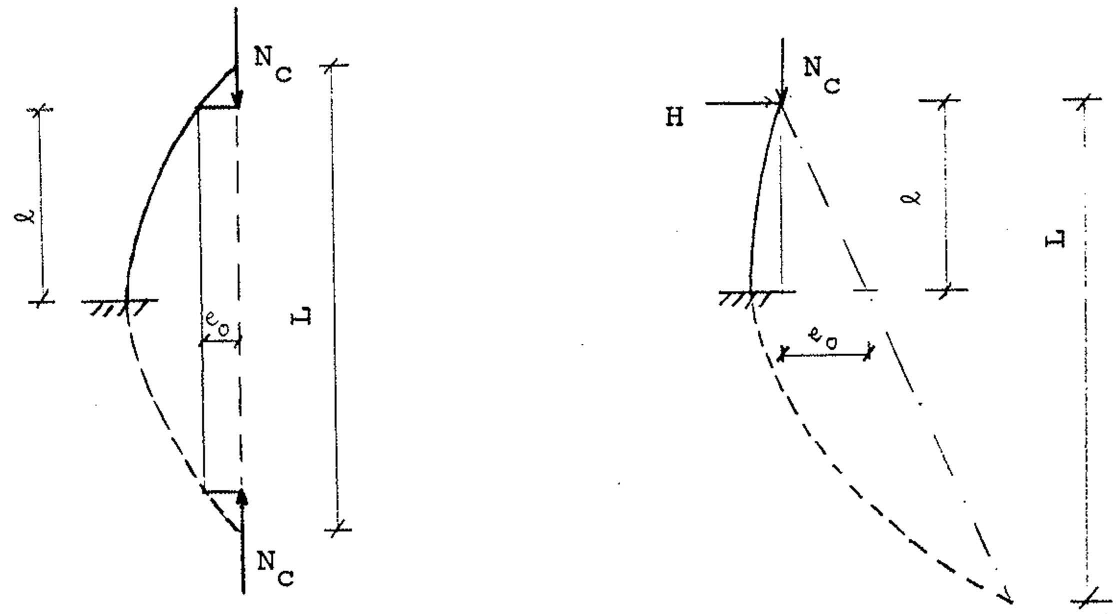


Fig. 5
Column champed in at the foot and free movable at the top with two different initial eccentricities.

From this difference in loading, the corresponding deformations will be different too: the value L in fig. 5a for a given value of e_o will not be equal to L in fig. 5b for the same value of e_o at the foot of the column. The calculation, however, will not be based on the exact sinus-wave corresponding with the linear elasticity, but on a

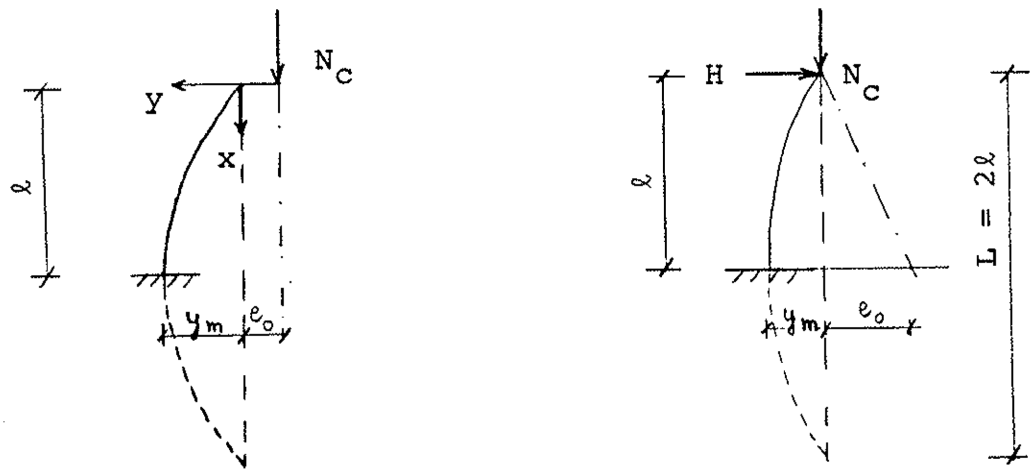


Fig. 6

Value L will be fixed equal to 2ℓ

This value L (= 2ℓ) is now independent of the magnitude of the initial eccentricity e_o and the course of the initial eccentricity over the length of the column.
Based on equation (12) the magnitude of the total deformation as function of the curvature, at the foot of the column can be formulated as:

$$f_m = 0.4 \kappa_m \cdot \ell^2 \qquad \ldots(13)$$

This relation between deformation f and curvature κ is for the column with an eccentricity as given in fig. 5a an approach at the safe side. For the column with a constant initial eccentricity, the approach is a little unsafe; the deviation is dependent on the loading type. For centrically loading columns there is no deviation at all.
With increase of the eccentricity and decrease of the axial force, the deviation increases to a maximum of 25 percent, at pure bending; equation (12b).

For this laoding case the deformation don't supply second order bending moments, sothat the measure of unsafety remains between reasonalbe limits. It is therefore justified to declare that a calculation based on $f_m = 0.4\ \kappa_m\ .\ \ell^2$ gives acceptable results, sothat for the sake of simplicity the same calculation approach will be maintained for different loading types.
In this way a complicated column analysis has been reduced to a simple crosssection calculation.

V EXAMPLES

As an illustration of the proposed procedure, the following example has been worked out; see fig. 7.

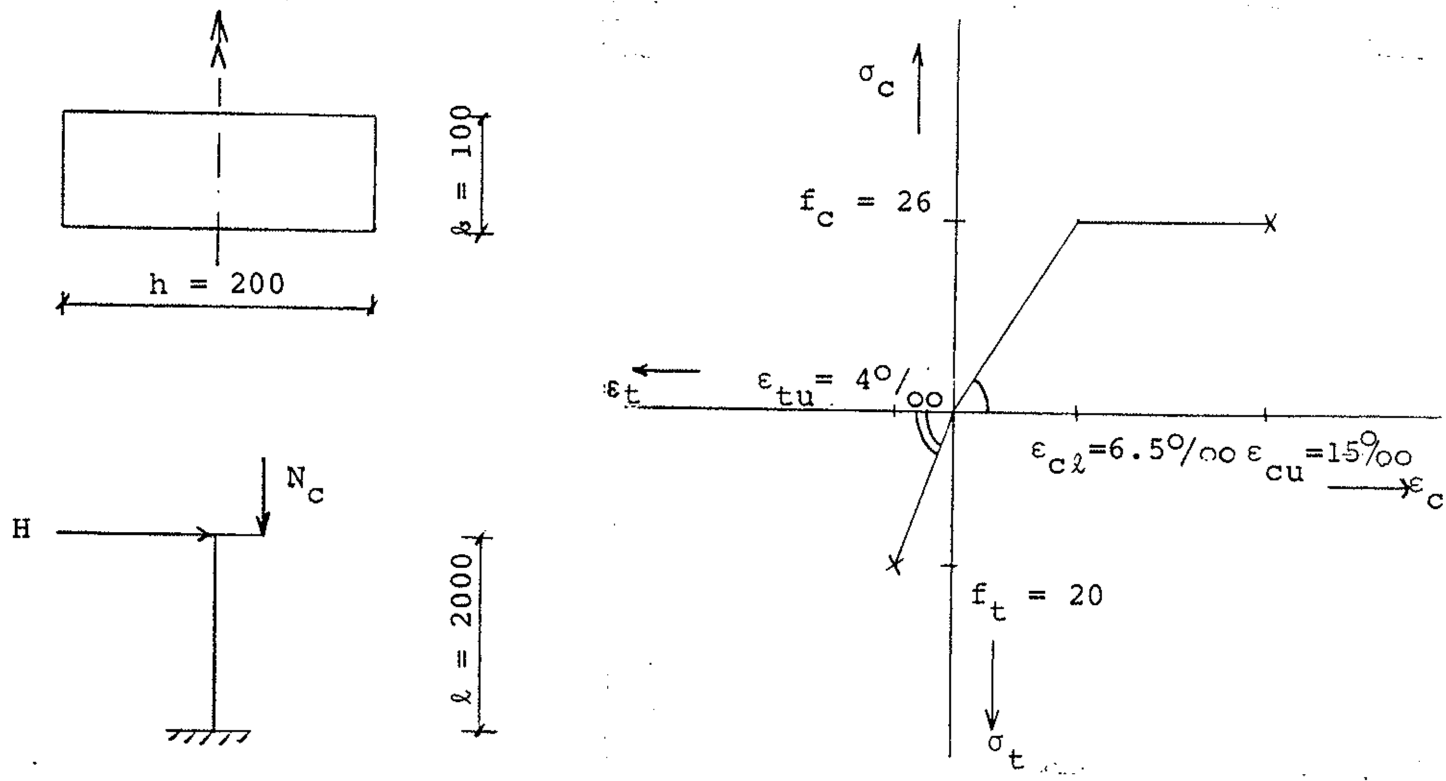


fig. 7
Calculation example of a column loaded by a horizontal and vertical force with corresponding σ - ε diagram

The column has been clamped in at the foot. At the top are acting a horizontal force H_d (= 2500 N) and an eccentrically vertical force N_c (= 0.104 . 10^6 N). The stress-strain diagram is linear for tensile and bi-linear for compression. This diagram is relative arbitrary.
The relation between bending moment, axial force and curvature will be governed by the so-called M-N-κ diagram. In fig. 8 this diagram has been given.

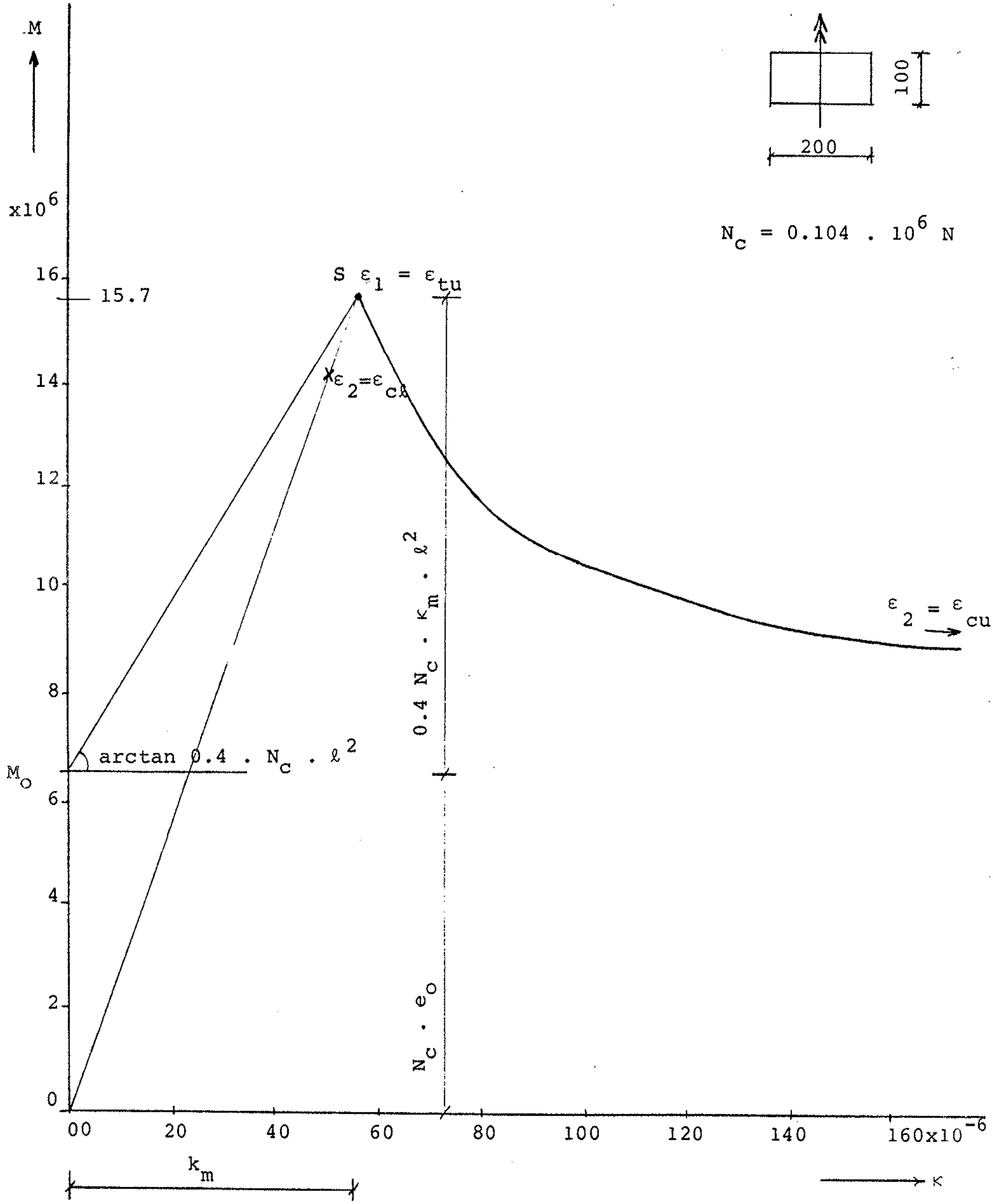


Fig. 12

M-N-κ-diagram for $N_c = 0.104 \times 10^6$ N

In this diagram some characteristic points have been indicated

- $\varepsilon_2 = \varepsilon_{cl}$ The "yield" compressive stress has been reached
- $\varepsilon_1 = \varepsilon_{tu}$ The tensile strength has been reached. At this point, the column will break over part of the cross section. This point gives the (maximum) design value of M (= 15.7 . 10^6N/mm)
- $\varepsilon_2 = \varepsilon_{cu}$ The maximum strain in the compressive zone has been reached

Design of the column:

The initial eccentricity u_o of N_c i.r.t. the axis will be in accordance with the Dutch Timber Code NEN 3852:

$$u_o = (0.10 + \frac{\lambda}{200}) . \frac{W}{A} = (0.10 + \frac{2 \times 2000}{0.289 \times 200 \times 200}) \frac{1}{6} \times 200 =$$

$$= 14.9 \text{ mm}$$

$$e_o = u_o + \frac{M}{N_c} = u_o + \frac{H_d \times \ell}{N_c} = 14.9 + \frac{2500 \times 2000}{0.104 \times 10^6} = 63 \text{ mm}$$

$$e_t = e_o + 0.4\ \kappa . \ell^2$$

$$M_t = N_c . e_t = N_c . e_o + 0.4\ N_c . \kappa_m . \ell^2$$

The straight line showing the equation for M_t, cut the M-N-κ diagram in print S, resulting in

$$M_t = 15.3 \times 10^6 \ [\text{N.mm}]$$
$$\kappa = 53 \times 10^{-6} \ [\text{mm}^{-1}]$$

This combination of H_d and N_c is just acceptable.

A second example will be worked out for a column, only loaded by an axial force with initial eccentricity e_o:

$$N_c = 0.416 \cdot 10^6 \text{ N}$$
$$H_d = 0$$
$$e_o = u_o = 14.9 \text{ mm}$$
$$M_o = N_c \cdot e_o = 0.416 \cdot 10^6 \cdot 14.9 = 6.2 \cdot 10^6 \text{ Nmm}$$

The line making an angle arctan $0.4 \cdot N_c \cdot \ell^2$ (see fig. 9a) don't cut the M-N-κ diagram.
The column can't resist the force N_c with eccentricity u_o.
The calculation for ℓ = 700 mm instead of ℓ = 2000 mm, will illustrate the possibility of instability.

$$N_c = 0.146 \cdot 10^6 \text{ N}$$
$$H_d = 0$$
$$e_o = u_o = \left(0.10 + \frac{2 \times 700}{0.289 \cdot 200 \cdot 200}\right) \cdot \frac{1}{6} \cdot 200 = 7.4 \text{ mm}$$
$$M_o = N_c \cdot e_o = 0.416 \cdot 10^6 \cdot 7.4 = 3.1 \cdot 10^6 \text{ Nmm.}$$

The line making an angle arctan $0.4 \cdot N_c \cdot \ell^2$ cut the M-N-κ diagram in two points: S and L. S represents a stable equilibrium, L an unstable one.

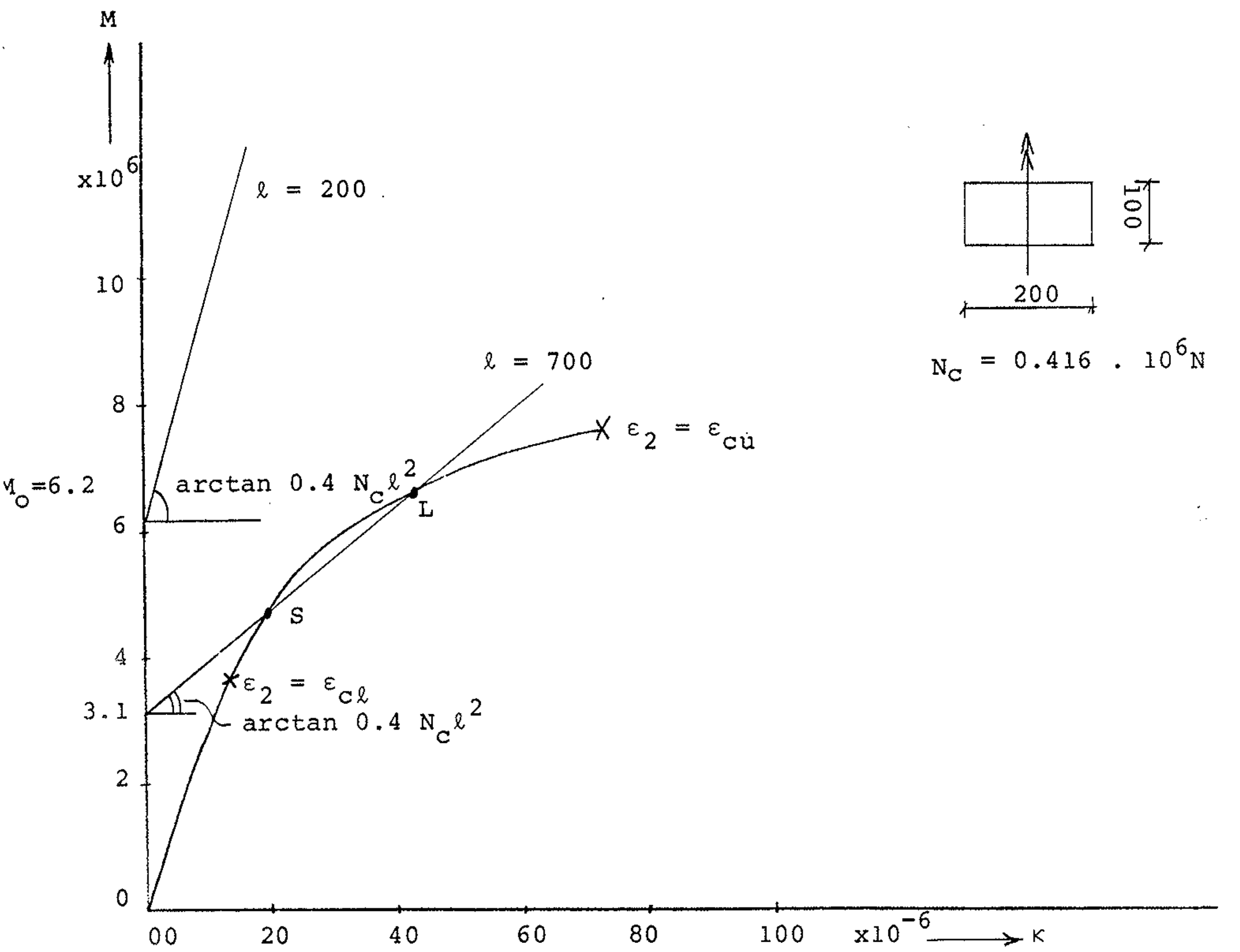


fig. 9a

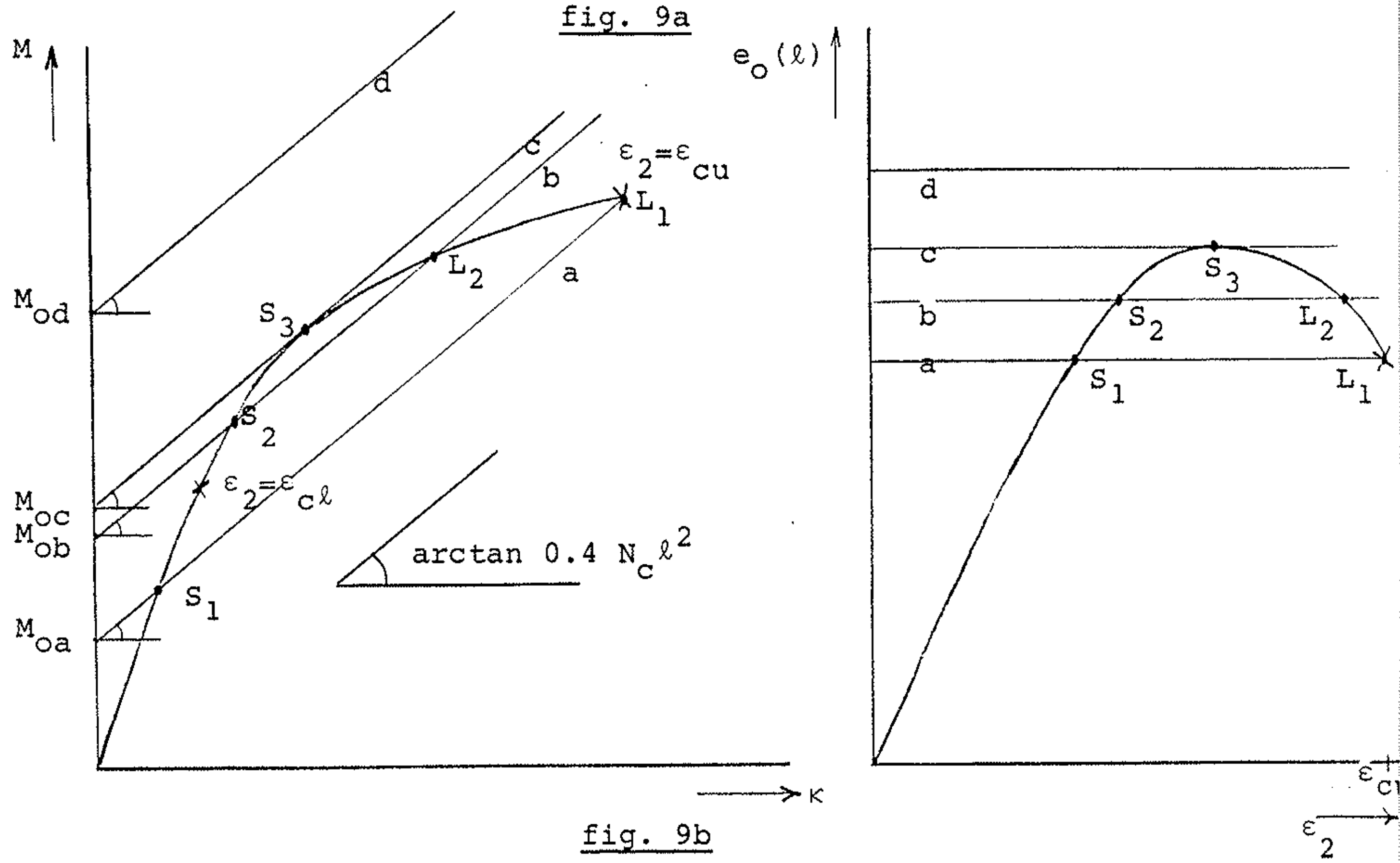


fig. 9b

Fig. 9

M-N-κ diagram and relation between ε_2 and e_o and ℓ resp.

In fig. 9b four lines have been drawn in the M-N-κ diagram, making a constant angle of arctan $0.4 \, . \, N_c \, . \, \ell^2$ with the horizontal axis; N_c and ℓ are constant values. Each straight line gives a corresponding bending moment M_o equal to $N_c \, . \, e_o$.

As for each point of the M-N-κ diagram the value ε_2 is known, the relation between e_o and ε_2 can be found.

The diagram shows that, the initial eccentricity e_o will reach a maximum value with constant values for N_c and ℓ, while ε_2 is less than ε_{cu}.

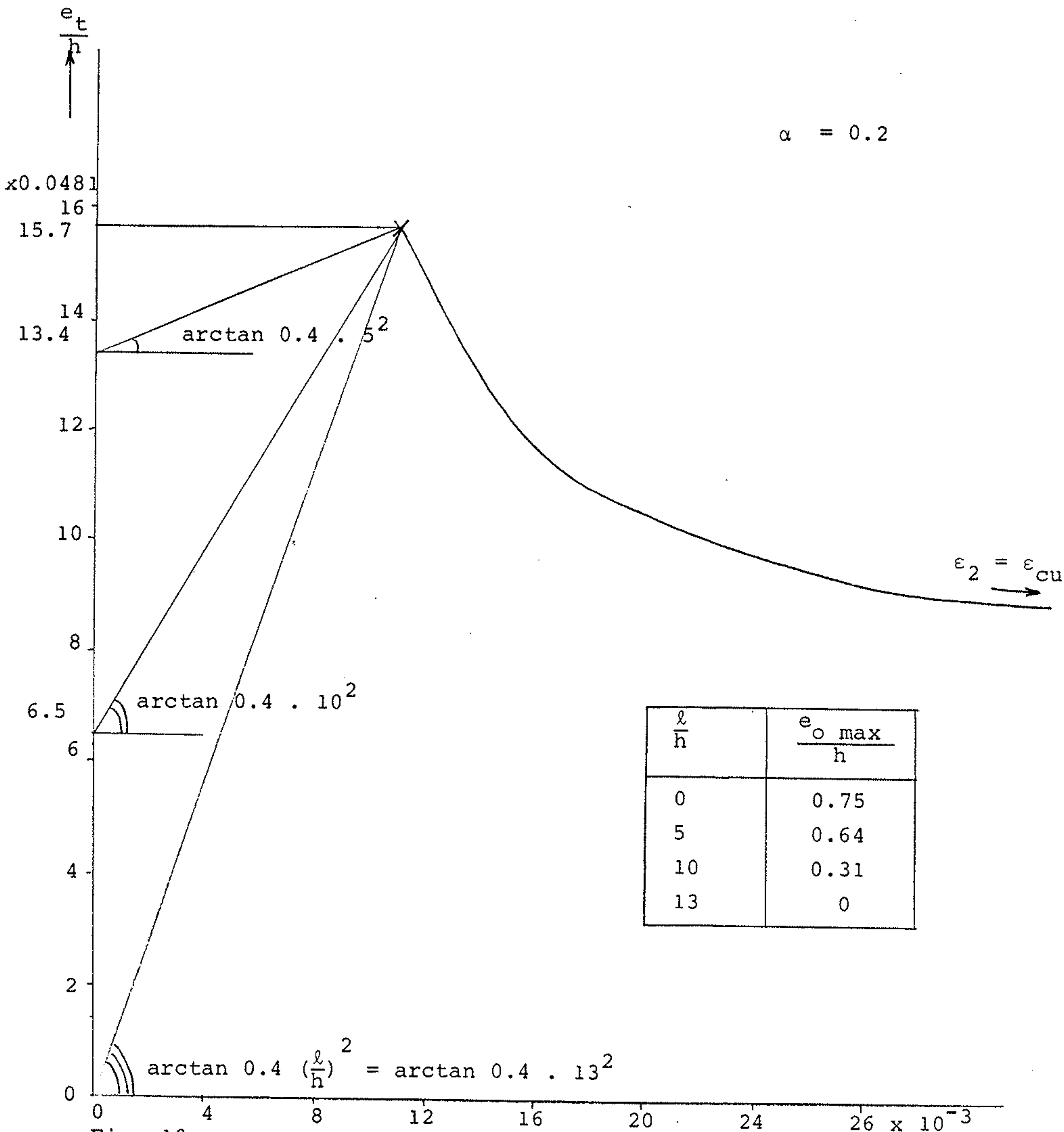
The procedure can be repeated with constant values for N_c and e_o; then the bending moment M_o is constant too. For different values of ℓ, and therefore of arctan $0.4 \, . \, N_c \, . \, \ell^2$, the relation between ℓ and ε_2 can be calculated.

Again, the maximum value of ℓ will be reached, while ε_2 is less than ε_{cu}.

VI RELATION BETWEEN e_o AND N_c

The background of the proposed method has now been discussed. Next the procedure will be given to determine the relation between the ultimate axial force N_c and the initial eccentricity e_o for different values of the slenderness $\frac{\ell}{h}$.
As a first step, the relation between M and κ, given by the M-N-κ diagram, will be rewritten to dimension less parameter $\frac{e_t}{h}$ and κ . h by dividing the bending moment by N_c . h and by multiplying the curvature with h resp. fig. 10 shows the resulting $\frac{e_t}{h}$ - N - κ . h diagram. The axial force will be expressed in the dimensionless parameter α:

$$\alpha = \frac{N_c}{N_{cu}} = \frac{N_c}{A \cdot f_c}$$



$\frac{\ell}{h}$	$\frac{e_{o\,max}}{h}$
0	0.75
5	0.64
10	0.31
13	0

Fig. 10

$\frac{e_t}{h}$ - N-κ.h diagram $\longrightarrow$ κ.h

determination of $\frac{e_{o\,max}}{h}$ for different values of $\frac{\ell}{h}$

From:

$$e_t = e_o + 0.4\ \kappa_m\ .\ \ell^2$$

follows:

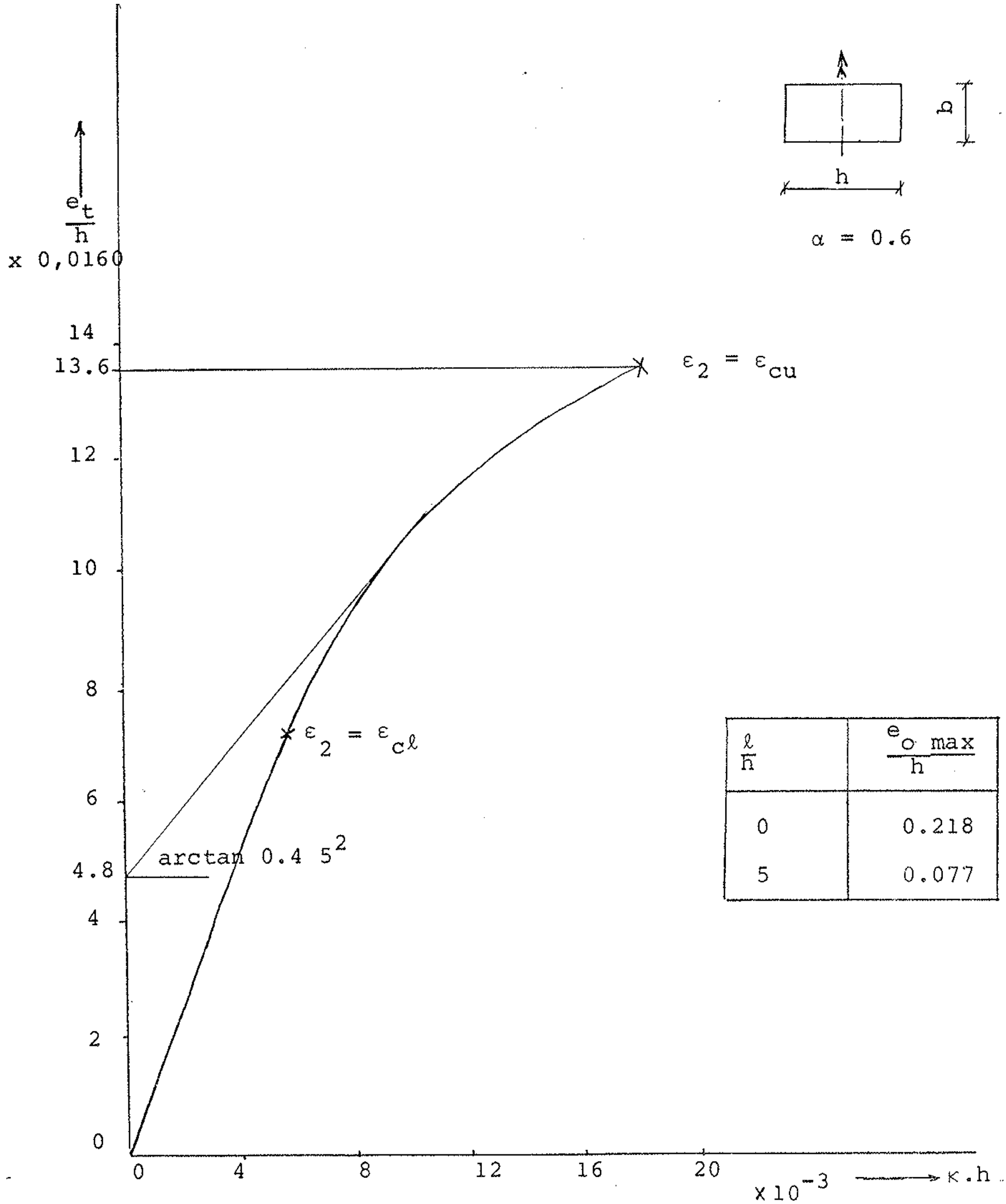
$$\frac{e_t}{h} = \frac{e_o}{h} + 0.4\ \kappa_m\ .\ h\ \left(\frac{\ell}{h}\right)^2$$

For values of $\frac{\ell}{h}$ equal to 0, 5, 10 etc, the corresponding maximum values of $\frac{e_o}{h}$ can be calculated.
Starting from the point, positioned on the $\frac{e_t}{h}$ - N - κ . h diagram with $\varepsilon_1 = \varepsilon_{tu}$ (maximum value for $\frac{e_t}{h}$), the maximum value for $\frac{e_{o\ max}}{h}$ follows directly by means of a horizontal line resulting in: $\frac{e_t}{h}$ = 15.7 . 0.0481 = 0.75 (0.0481 is a scale factor).
For $\frac{\ell}{h}$ = 10 results a value for $\frac{e_{o\ max}}{h}$ equal to 65. 0.0841 = 0.312 by means of a line from the same point as above with an angle arctan 0.4 . 10^2. The maximum value for $\frac{\ell}{h}$ follows from the angle of the $\frac{e_t}{h}$ -N-κ.h-diagram in the origin: $\frac{\ell}{h}$ = 13.
The corresponding eccentricty is zero: thus centrically loading:
By Euler's formula $P_E = \frac{\pi^2\ EI}{\ell_k^2}$

with $EI = \frac{M}{k} = \frac{15.6 \cdot 10^6}{54.5 \cdot 10^{-6}}$ follows:

$$P_E = \frac{\pi^2 \cdot 15.6 \cdot 10^6}{(2.13 \cdot 200)^2 \cdot 54.4 \cdot 10^{-6}} = 0.104 \cdot 10^6\ N$$

This result corresponds with the value N_c (= 0.104 · 10^6)
In fig. 11 the same procedure is shown for N = N_d = 0.312 · 10^6 N (α = 0.6)



$\frac{\ell}{h}$	$\frac{e_{o\,max}}{h}$
0	0.218
5	0.077

fig. 11
$\frac{e_t}{h}$ -N-κ.h diagram for α = 0.6
determination of $\frac{e_{o\,max}}{h}$ for different values of $\frac{\ell}{h}$

For each value of α this procedure can be repeated resulting in corresponding values for $\frac{e_o}{h}$ for different values of $\frac{\ell}{h}$ resp.
In fig. 15 these results are given in a graph.

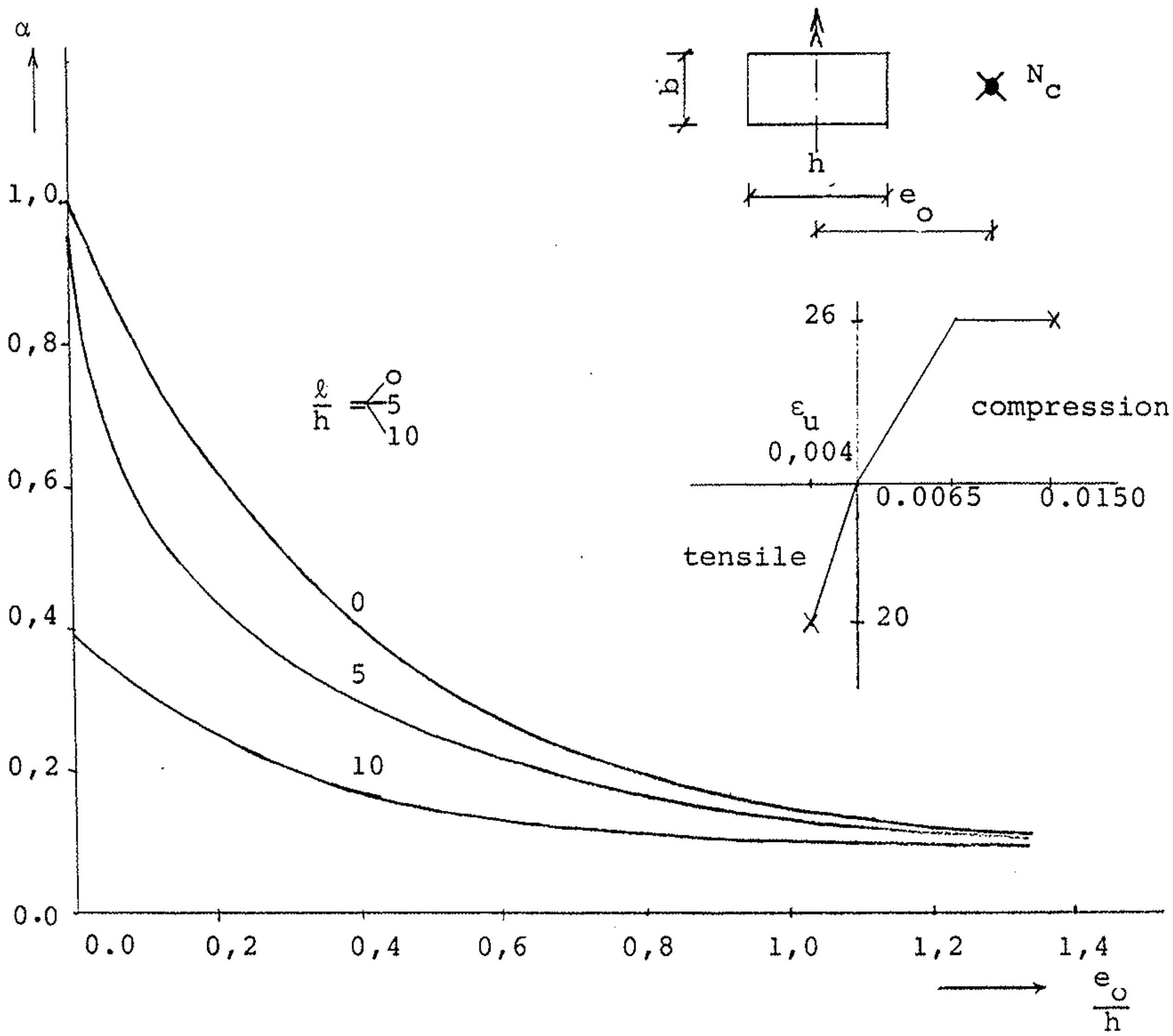


Fig. 15
Relation between $\frac{e_o}{h}$ and ultimate axial force for different values of $\frac{\ell}{h}$

By this way the relation between axial force, bending moment and shape and dimensions of the column, are been given by the three dimensionless parameters α, $\frac{e_o}{h}$ and $\frac{\ell}{h}$ resp.

VII COMPARISON TNO AND CIB RESULTS

For two different stress-strain diagrams the procedure has been carried out. The diagrams and corresponding results are given in fig. 16 and fig. 17.
The first diagram (fig. 16) is the same one as used in fig. 7. (f_c = 26 N/mm^2, f_t = 20 N/mm^2). Fig. 16^a and 16^b give in principle the same information. However, in fig. 16^a α is plotted against $M/(f_c.A.h)$ while in fig. 16^b α is shown as a function of e_o/h ($M/(f_c Ah) = \alpha \cdot e_o/h$)
As in CIB approach a value for the bending strength f_m is used, it is necessary to introduce such a value to compare the TNO and CIB results. From fig. 16^a it follows that $M/(f_c\ A\ h) = 0.12$ for $\alpha = 0$ (no axial force) and $\frac{\ell}{h} = 0$.
For a rectangular cross-section, with dimensions b and h, f_m follows from:

$$f_m = \frac{M}{f_c.A.h} \cdot f_c.b.h\ h \frac{6}{b\ h^2} = 6\ f_c \cdot \frac{M}{f_c.A.h} =$$
$$6.26 \cdot 0.12 = 18.8\ \text{N/mm}^2$$

This value f_m is less than f_c and f_t (26 and 20 N/mm^2 resp.).

In the CIB draft [1] however, f_m is greater than f_c and f_t: see table 5.10 a "Characteristic values and mean elastic moduli". For SC 30 for instance is f_m, f_t and f_c 30, 20 and 28 N/mm^2 resp. In analogy with these values, two values for f_m will be assumed: f_m = 18.8 N/mm^2, based on TNO approach and f_m = 30 N/mm^2, based on CIB draft.
In fig. 16 two groups of lines are drawn for $\frac{\ell}{h} = 0$ and 5 resp. Each group consists of three lines; one according to TNO's approach and two according to CIB's approach with f_m = 18.8 and 30 N/mm^2 resp.

The differences between corresponding lines are great. In general, the CIB approach gives too conservative results in relation to TNO approach.
For small axial forces (α values less than say 0.2) then TNO ultimate bending moments are less than the CIB values.
As already mentioned in the introduction the relation between axial force and bending moments for $\frac{\ell}{h} = 0$ is according to TNO's opinion, not a straight line.
The second stress-strain diagram used in fig. 17 is quite different from that one used in fig. 16: f_t is now much greater than f_c (f_t = 84 N/mm^2 and f_c = 44 N/mm^2).
The value of the bending strength f_m follows from:

$$f_m = 6 \cdot f_c \cdot \frac{M}{f_c . A . h} = 6 \cdot 44 \cdot 0.277 = 73.2 \text{ N/mm}^2$$

is positioned between the values for f_t and f_c.
In analogy with CIB approach a second value for f_m is assumed equal to 100 N/mm^2.
In fig. 17 all results are shown.
For $\frac{\ell}{h} = 0$ the differences between TNO and CIB with f_m = 73.2 N/mm^2, are small but great with f_m = 100 N/mm^2.
For $\frac{\ell}{h} = 10$ both CIB results are lying at the unsafe site compared with the TNO results. In fig. 17, however, the influence has been drawn between the consequences of formula 12a and 12b (see chapter III)

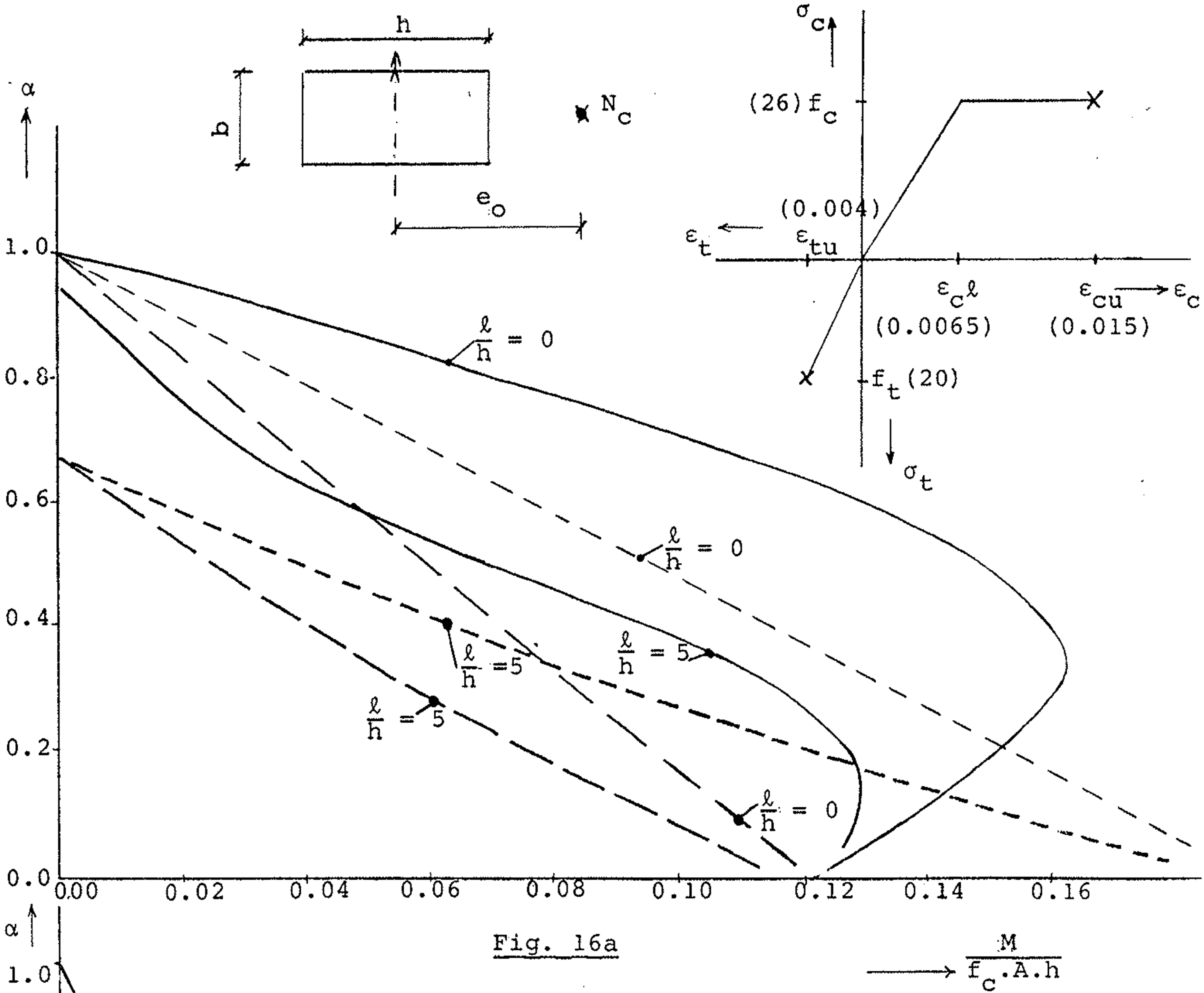
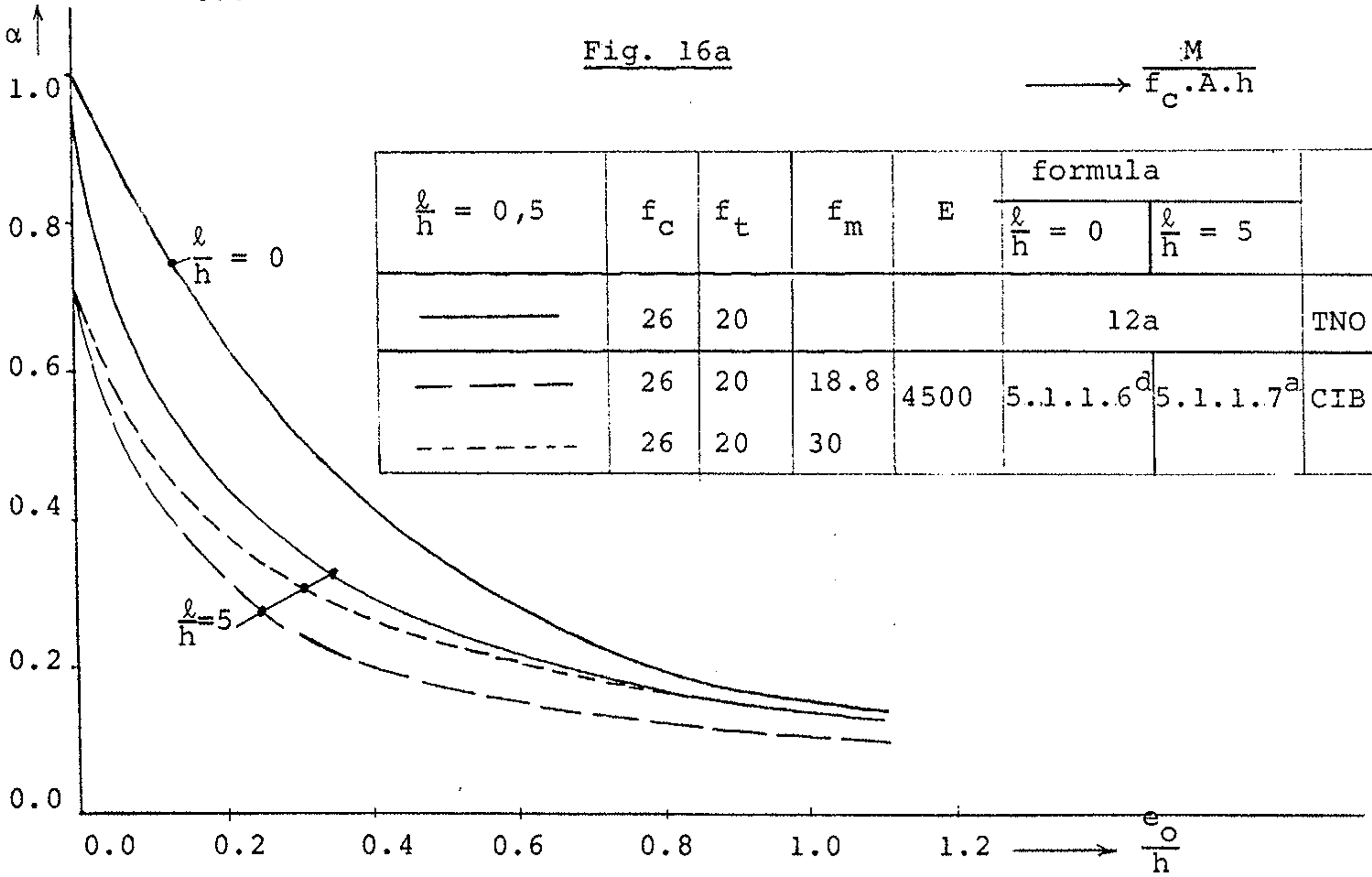


Fig. 16a



$\frac{\ell}{h}$ = 0,5	f_c	f_t	f_m	E	formula $\frac{\ell}{h}$ = 0	formula $\frac{\ell}{h}$ = 5	
———————	26	20			12a		TNO
— — —	26	20	18.8	4500	5.1.1.6[d]	5.1.1.7[a]	CIB
- - - - - - -	26	20	30				

Fig. 16b

Fig. 16

Relation between M, N and e_o for different values of $\frac{\ell}{h}$

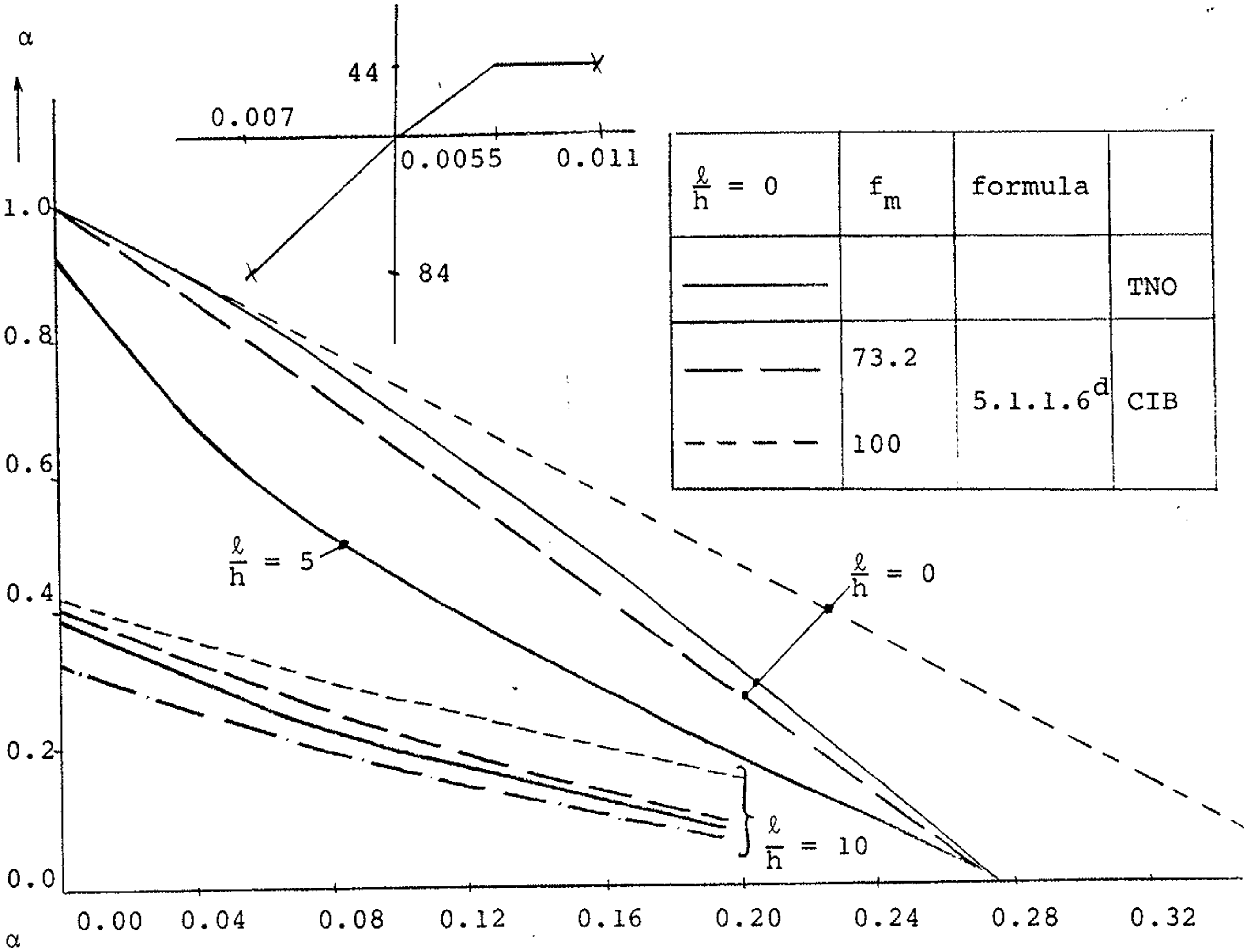


Fig. 17a

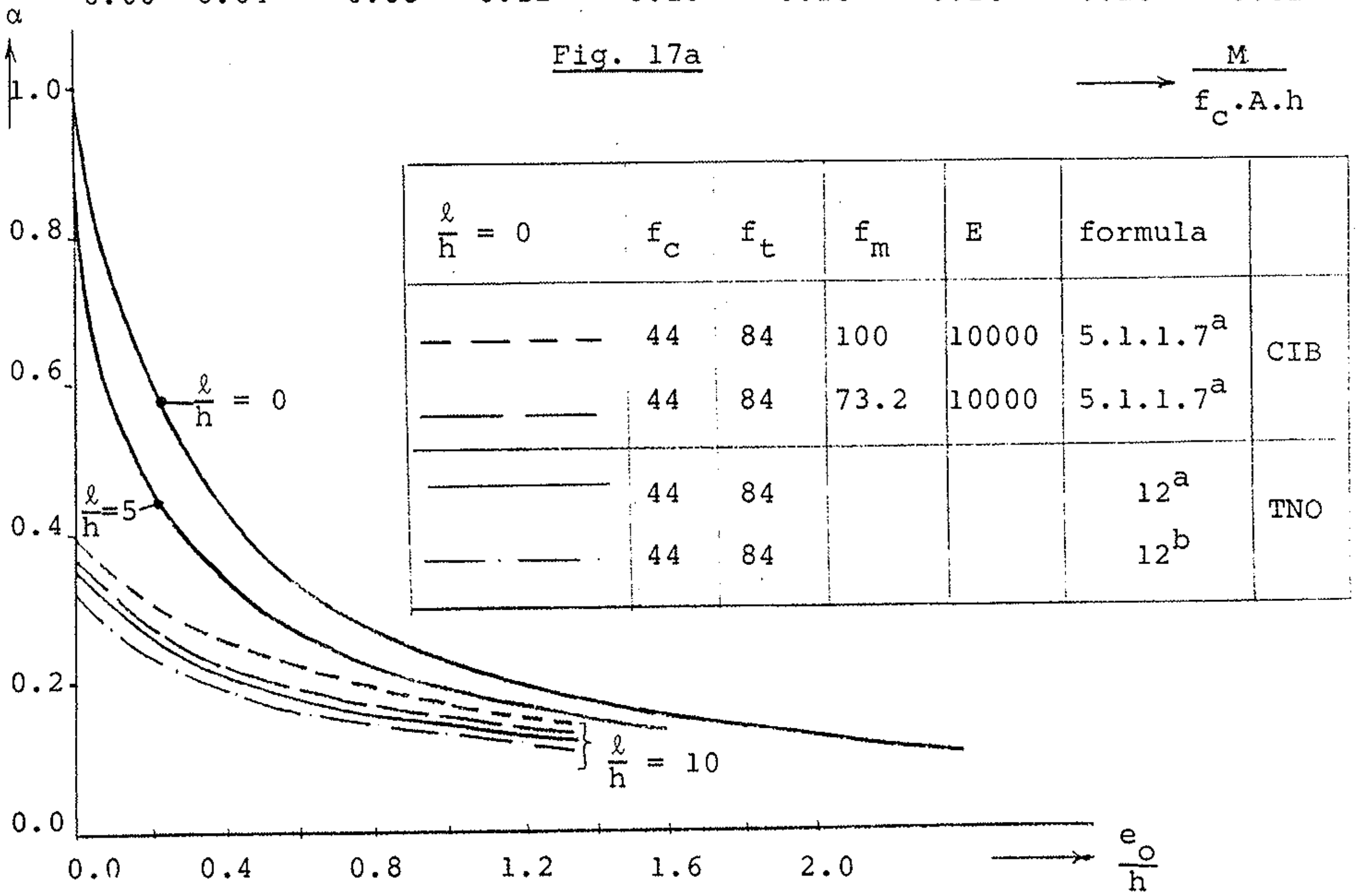


Fig. 17b

Fig. 17

Relation between M, N and e_o for different values of $\frac{\ell}{h}$

$$\frac{f}{h} = 0.1\ \kappa\ .\ h\ \left(\frac{\ell}{h}\right)^2 \qquad \ldots(12a)$$

$$\frac{f}{h} = 0.125\ \kappa\ .\ h\ \left(\frac{\ell}{h}\right)^2 \qquad \ldots(12b)$$

As the deformations, calculated by (12b) are greater than by (12a), the N-M- relation will be reduced, of course. The corresponding results are given in fig. 17.too.

VIII SUMMARY AND CONCLUSIONS

In the Ultimate Limit State design, it is necessary to calculate the maximum load-carrying capacity. Especially for construction elements, loaded by axial force and bending moments, geometrical and fysical non-linearities are very important. As the stress-strain relation for timber is not-linear, an accurate calculation of the ultimate load-carrying capacity is "only" possible by a computer.

By a suitable choise of the deflection curve of the concerned construction element, the calculation procedure can be simplified.
For a non-linear elastic material, the load carrying capacity can be limited by the compression or the tensile strength of the material, but also by instability.
In this paper a method has been evolved, to determine the relation between the design value of the axial force and the initial eccentricity, as function of $\frac{\ell}{h}$, starting from the $\sigma - \varepsilon$ - diagram of the timber used.
Directly the results can be used by the designer. By comparing TNO and CIB results, the CIB results are sometimes unsafe (10 to 30%) and sometimes too conservative (40 to 60%).
In this field still many experiments must be done, in order to obtain the correct $\sigma - \varepsilon$ - diagrams. Furthermore it should be advisable to check the computer method by direct experiments on columns.

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CIB-W18/11-100-2h

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENT ON THE CIB CODE
(Lateral Buckling)

by

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VIENNA, AUSTRIA

MARCH 1979

INSTITUTE TNO FOR BUILDING MATERIALS AND BUILDING STRUCTURES

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REPORT

Nr. B-79-148/62.4.2100
March 1979

CIB TIMBER CODE

CRITICISM OF THIRD DRAFT (sept 1978)

part C lateral buckling

Ir. A. Vrouwenvelder March 1979

member of SHR-OC15

Research commitee for timber stability problems

Proposal for modification of article 5.1.1.3

It is proposed to replace formula (5.1.1.2.d) from the CIB draft (appendix 1) by

$$\lambda_m = \sqrt{\frac{1}{\pi} \frac{\ell_e h}{b^2} \frac{f_m}{E_o} \sqrt{\frac{E_{o\ mean}}{G_{mean}}}} \qquad (5.1.1.3\ d)$$

Accordingly tabel 5.1.1.3 has to be replaced by:

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

Type of beam and load	ℓ_e/ℓ
Simply supported, uniform load or equal end moment	1.00
Simply supported, concentrated load at centre	0.85
Cantilever, uniform load	0.60
Cantilever, concentrated end load	0.85
Cantilever, end moment	1.00
The values apply to loads acting in the gravity axis. For downwards acting loads ℓ_e is increased by 2.25 h for loads on the top side and reduced by 0.75 h for loads on the bottom side.	

Comment

The proposed modification does not alter the technical contents of article 5.1.1.3 but only the way it is presented. In fact the modification only concerns the definition of the effective length ℓ_e.

Both CIB formula (5.1.1.3d) as the above proposed modification are based upon the idea that loading type and support conditions can be handled by adjustment of the length-parameter. In the CIB approach however there is no elementary standard case where the effective length is equal to the fysical length. This now is considered as an unlucky choise which does not serve the logic and the recognizability of the formula. Therefore a new formula is proposed where $\ell_e = \ell$ corresponds to the elementary case of a simply supported beam loaded by two equal end moments (figure 1, for the proof see appendix 2).

Another argument in favour of the proposed modification is that this apporach is more consistent with the column stability approach of section 5.1.1.7. The algebraic factor π which is very common in elastic stability analysis, is also present in formula (5.1.1.7c) ($k_e = \pi^2 E_o / f_{c,o} \lambda^2$) and the situation $\ell_c = \ell$ is preserved for the elementary Euler case.

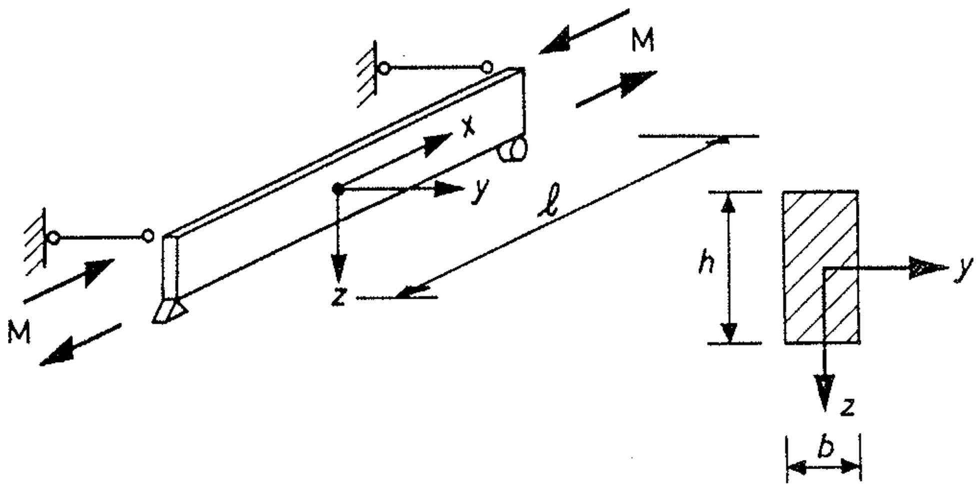


Fig. 1 Elementary case for lateral buckling.

Appendix I

CIB article 5.1.1.3

5.1.1.3 Bending

The bending stresses, σ_m, calculated according to the theory of elasticity shall satisfy

$$\sigma_m \leqslant k_{depth} k_{inst} f_m \tag{5.1.1.3 a}$$

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

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

The value of κ depends on among other things the wood species and the grading rules. Recommendations will be produced.

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

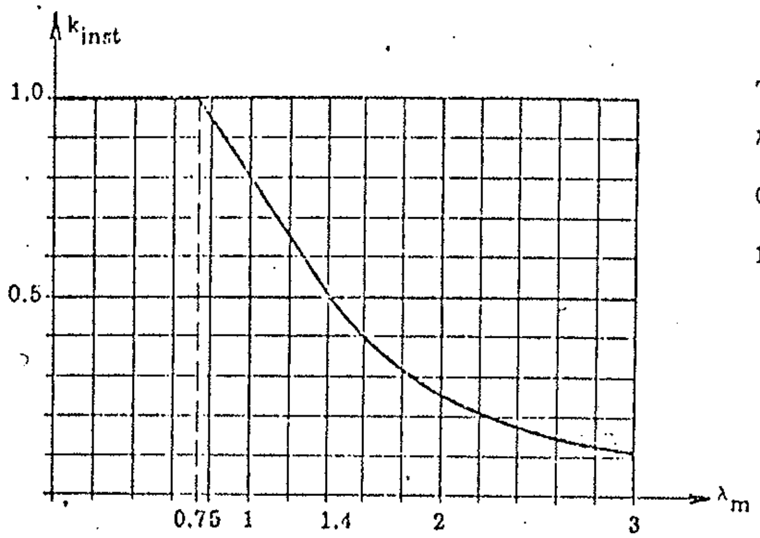
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

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

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

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

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



The curve corresponds to

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

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

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m^2$

Fig. 5.1.1.3

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

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

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

The free length is determined as follows:

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

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

Table 5.1.1.3 Relative effective beam length ℓ_e/ℓ

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

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

Appendix 2 Derivation of the proposed formula

The definition of the slenderness parameter λ_m is given by (see 5.4.1.3c):

$$\lambda_m = \sqrt{f_m / \sigma_{m,crit}} \qquad \ldots(1)$$

In here f_m is the bending strength and $\sigma_{m,crit}$ is the critical stress for laterel buckling based on the elastic second order theory.
If one considers the rectangular beam as indicated in figure 1, supported in such a way that displacements and axial rotation are prevented and loaded by two equal end moments, than the classic result for the critical load is given by:

$$M_{crit} = \pi \frac{\sqrt{EI_{yy} \cdot GI_t}}{\ell} \qquad \ldots(2)$$

where EI_{yy} is the bending stiffness for the weak axis, GI_t is the torisonal rigidity and ℓ is the length of the beam.
If W represents the section modulus the critical stress can be written as:

$$\sigma_{m,crit} = \frac{M_{crit}}{W} \qquad \ldots(3)$$

For the rectangular cross section holds:

$$I_{yy} = \frac{1}{12} h b^3 \qquad I_t = \frac{1}{3} h b^3 \qquad W = \frac{1}{6} bh^2 \qquad \ldots(4)$$

Combining (1) to (4) leads to:

$$\lambda_m = \sqrt{\frac{1}{\pi} \frac{\ell h}{b^2} \frac{f_m}{E} \sqrt{\frac{E}{G}}} \qquad \ldots(5)$$

Apart from the indication as to what values should be taken for E and E/G this formula corresponds exactly to formula (5.1.1.3 d-new) for $\ell = \ell_e$

CIB-W18/11-100-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB STRUCTURAL TIMBER DESIGN CODE

CHAPTER 3

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MARCH 1979

3. BASIC DESIGN RULES

3.0 General

Structures should be designed in such a way that there is a prescribed safety against the limit states described below being exceeded.

Furthermore, they should be designed in such a way that when exposed to fire they have adequate load-carrying capacity and integrity for a certain amount of time, out of regard for evacuation, limitation of flame spread and protection of firemen.

The main structure should normally be designed in such a way that it should not subsequently be damaged to an extent disproportionate to the extent of the original incident. This requirement may be achieved by

a) designing the structure in such a way that if any single load-bearing member becomes incapable of carrying load this will not cause collapse of the whole structure or any significant part of it, or

b) where necessary, ensuring (by design or by protective measures) that no essential load-bearing member can be made ineffective as a result of an accident.

3.1 Limit states

A structure, or part of a structure, is considered to have become unfit for its intended purpose when it reaches a particular state, called a limit state, in which one of the criteria relating to its load-bearing capacity or its conditions of service is infringed.

Limit states are classified into ultimate limit states and serviceability limit states.

3.1.1 Ultimate limit states

Ultimate limit states correspond to the maximum load-carrying capacity or to complete unserviceability.

: Ultimate limit states may for example correspond to:
: - loss of static equilibrium of the structure, or part of the structure, considered as a rigid body (overturning),
: - rupture of critical sections of the structure due to exceeding of the material strength (in some cases reduced by repeated loading) or by deformations,
: - loss of stability (due to among other things buckling),
: - unlimited slip of the whole structure or mutually between parts of it.

3.1.2 Serviceability limit states

Serviceability limit states are related to the criteria governing normal use.

: Serviceability limit states may for example correspond to:
: - deformations which affect the efficient use of a structure or the appearance of structural or non-structural elements,
: - excessive vibrations producing discomfort or affecting non-structural elements or equipment (especially if resonance occurs),
: - local damage (including cracking) which reduces the durability of a structure or affects the efficiency or appearance of structural or non-structural elements,
: - local buckling of thin plates (for example in thin webs or flanges) without rupture.

3.2 Actions and their combinations

3.2.1 Actions

It is assumed that actions are classified according to ISO . . . (DP 6116, under preparation) and that the following values are given for the individual actions

- characteristic value F_k
- combination value $\psi_0 F_k$

and furthermore, that the necessary information is given to assign the actions to one of the climate classes given in section 2.2.

: It is assumed that the load values are given in separate documents, for example in the form of common load regulations or ISO standards.

: An action is an entity of
: - an assembly of concentrated or distributed forces acting on the structure (direct actions), or
: - the effect of imposed or constrained deformations in the structure (indirect actions)
: and due to the same cause.

: Actions are classified according to their variation in time as
: - permanent actions
: - variable actions
: - accidental actions

: The characteristic value of permanent actions from self-weight of the structure and weight of superstructure, etc. may be calculated from the intended values of the geometrical parameters and the mean unit weight of the material.

: For variable action the characteristic value is defined as the value which has a prescribed probability of not being exceeded in one year. When characteristic values cannot be determined from statistical data, as for example for actions from special equipment, the corresponding values may be estimated on the basis of available information. In some cases minimum characteristic values are prescribed by the competent public authority.

: For the combination value the factor ψ_0 takes account of the reduced probability of simultaneously exceeding the design values of several actions, as compared with the probability of the design value of a single action being exceeded.

: For accidental actions the characteristic value is normally prescribed by the competent public authority.

3.2.2 Combinations of actions

In the ultimate limit states the following two types of combinations should be applied:

- ordinary combinations
- accidental combinations.

In the serviceability limit states the combinations of actions should be chosen with regard to the purpose of the actual calculation.

3.2.2.1 Ordinary combinations

Permanent action + one variable action with its characteristic value + variable actions with their combination values.

3.2.2.2 Accidental combinations

Permanent action + one accidental action + variable actions with their combination values.

3.3 Verification of reliability

The verification of the reliability should be made according to the method of partial coefficients.

In this method

- actions are expressed by design values F_d according to 3.3.1,
- strength parameters are expressed by design values f_d according to 3.3.2. Other relevant properties (e.g. modulus of elasticity in connection with instability design) are treated in a similar way,
- geometrical parameters are expressed by design values a_d according to 3.3.3.

: If the general condition for the actual limit state not being exceeded is expressed as

$$\theta(F, f, a, \mu, C) > 0 \qquad (3.3a)$$

: the design criteria will be

$$\theta(F_d, f_d, a_d, \mu_d, C) > 0 \qquad (3.3b)$$

: where

: F represents actions,
: f represents material properties,
: a represents geometrical parameters,
: μ are quantities covering the uncertainties of the calculation model,
: C are constants including preselected design constraints,
: $\theta()$ represents the limit state function, and
: subscript d denotes design value.

: The special problems in connection with soil mechanics are not treated in this code.

3.3.1 Design values of actions

The design value shall be obtained from the characteristic value or the combination value by multiplication by a partial coefficient γ_f:

$$F_d = \gamma_f F_k \tag{3.3.1a}$$

or

$$F_d = \gamma_f \psi_0 F_k \tag{3.3.1b}$$

The values of γ_f are prescribed by the relevant public authority.

: In CEB-volume II, concrete, the values for γ_f given in table 3.3.1a have been proposed.

: *Table 3.3.1a CEB-proposal*

action combination	action	γ_f	unfavourable effect	favourable effect
ordinary	permanent	γ_g	1.35	1.0
	variable	γ_q	1.5	not to be taken into account
accidental	permanent	γ_g	1.1	0.9
	variable	γ_q	1.0	not to be taken into account
	accidental	γ_a	1.0	not to be taken into account
serviceability	permanent	γ_g	1.0	
	variable	γ_q	1.0	

: In the NKB-proposal the values for γ_f given in table 3.3.1b have been proposed.

: *Table 3.3.1b NKB-proposal*

action combination	action	γ_f	unfavourable effect	favourable effect
ordinary	permanent	γ_g	1.0	0.8*)
	variable	γ_q	1.3	not to be taken into account
accidental	permanent	γ_g	1.0	1.0
	variable	γ_q	1.3	not to be taken into account
	accidental	γ_a	1.0	not to be taken into account
serviceability	permanent	γ_g	1.0	
	variable	γ_q	1.0	

: *) The factor should be applied to the total permanent action, not to part of it.

: The design load combinations according to section 3.2.2 are thus for ordinary combinations

: $\gamma_{g,unf}F_{g,unf} + \gamma_{g,fav}F_{g,fav} + \gamma_q(F_{q,1} + \psi_{0,2}F_{q,2} + \psi_{0,3}F_{q,3} \cdots)$ (3.3.1c)

:

: and for accidental combinations

:

: $\gamma_{g,unf}F_{g,unf} + \gamma_{g,fav}F_{g,fav} + \gamma_a F_a + \gamma_q(\psi_{0,1}F_{q,1} + \psi_{0,2}F_{q,2} + \psi_{0,3}F_{q,3} \cdots)$ (3.3.1d)

:

:

3.3.2 Design values of strength parameters

The design values shall be obtained from the characteristic values modified according to climate class and load-duration class by division by a partial coefficient γ_m:

$$\gamma_m = 1.0 \qquad (3.3.2a)$$

for serviceability limit states and

$$\gamma_m = \gamma_{m1}\gamma_{n1} \qquad (3.3.2b)$$

for ultimate limit states.

The partial coefficient γ_{n1} depends on the possible consequences of failure according to table 3.3.2a.

Table 3.3.2a The partial coefficient γ_{n1}

failure class	consequences	γ_{n1}
less serious	risk to life negligible and economic consequences small or negligible	0.9
serious	risk to life exists and/or economic consequences considerable	1.0
very serious	risk to life great and/or economic consequences very great	1.1

: The failure class for a given type of structures is normally prescribed by the competent public authority, e.g. in
: the form of a list of examples as given below. The authority may decide to refer all structures to the class serious.

: *Less serious:*

: - Buildings in which there are persons only occasionally, for example warehouses and sheds, if the buildings have no more than two stories and moderate spans.
: - Partition walls.
: - Lintels.
: - Roof and wall sheathing.

: *Serious:*

: - Buildings in which there are persons only occasionally, for example warehouses, if the buildings have more than two stories, or in one- or two-storey buildings with large spans (hall structures).
: - Buildings in which there are frequently many persons, for example dwelling houses, offices or factories, if the buildings have no more than two stories and moderate spans.
: - Building scaffolds and concrete forms.
: - Exterior walls.
: - Staircases.
: - Railings.

: *Very serious:*

: - Buildings in which there are frequently many persons, for example dwelling houses, offices, theatres, sports halls or factories, if the buildings have two or more stories or large spans (hall structures).
: - Grandstands.

: A structure is to be referred to a higher class than stated, if, in special cases, stricter requirements are made with regard to the risk of personal injury or the significance of the structure alltogether as compared to what is normal for the actual type of structure.

: For structures where the surroundings are decisive for the class consideration should be given to future buildings, if any, in relation to the life time of the structure.

: For structures under erection, the class to which the structure is to be referred must be decided upon in each individual case.

: In CEB-volume II, concrete, the values given in table 3.3.2b have been proposed. For concrete the characteristic value is defined as lower 10-percentile value, whereas 5-percentiles are used for timber.

Table 3.3.2b CEB-proposal for γ_{m1}

action combination	concrete	steel
ordinary	1.5	1.15
accidental	1.3	1.0

: According to the NKB-proposal the following values apply to timber and wood-products:

$\gamma_{m1} = 1.40$ for structures or structural members produced in a factory under special control arrangements
and
for structures or structural members where the characteristic values have been found by testing

$\gamma_{m1} = 1.50$ in all other cases

: The value assumes a coefficient of variation not deviating from what is usual in timber structures, i.e. a coefficient of variation between app. 0.15 and 0.25.

: According to the NKB-proposal accidental action combinations are considered only for the failure class »very serious».

3.3.3 Design values of geometrical parameters

In most cases the geometrical parameters may be assumed as specified in the design. When the size of a geometrical parameter may have a significant effect on the structural behaviour or the resistance of the structure the design values a_d should be obtained from the characteristic value a as

$$a_d = a + \Delta a \tag{3.3.3a}$$

or

$$a_d = a - \Delta a \tag{3.3.3b}$$

where Δa takes account of the importance of variations in a and the given tolerance limits for a.

3.4 Analysis

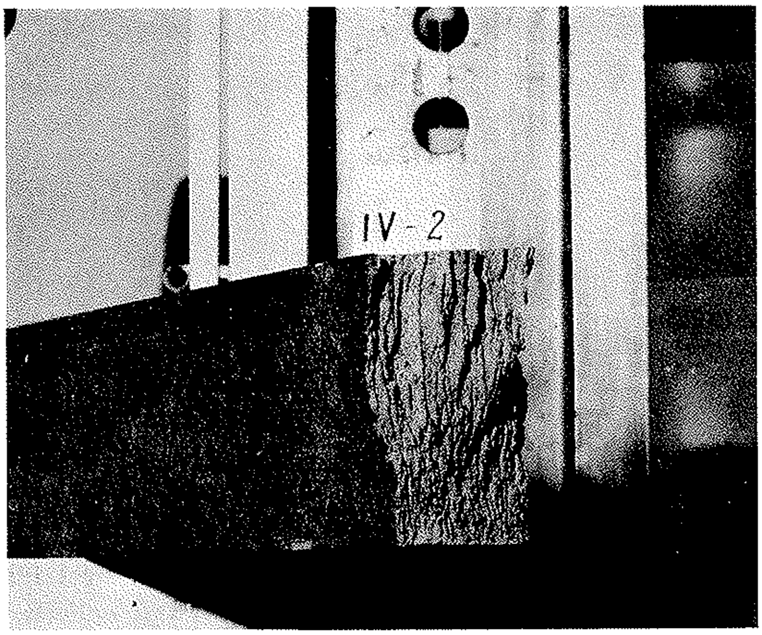
For the ultimate limit states linear, non-linear and plastic theories may be applied according to the structural response of the structure or structural member to the actions. The characteristic values in section 5 are, however, derived from the test loads by the theory of linear elasticity and this theory should therefore also be used in the design of the members.

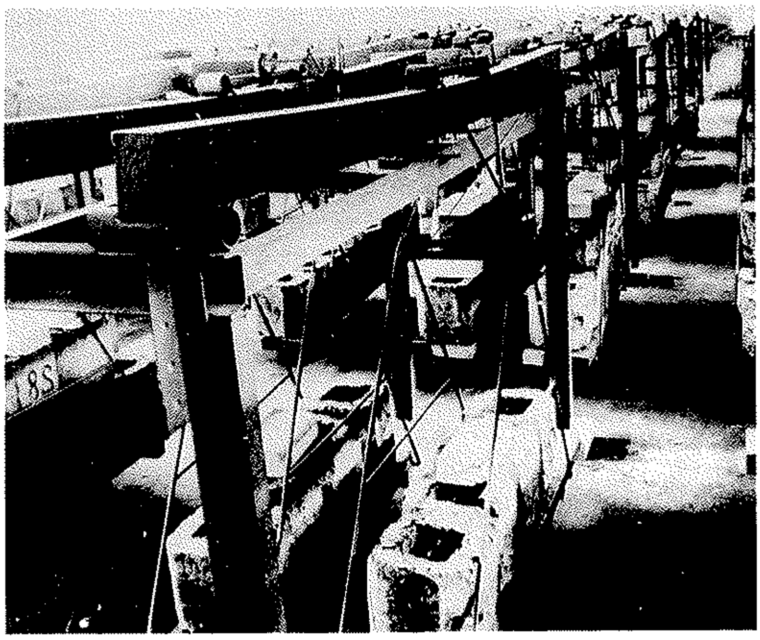
: It does not involve the requirement that the stress resultants in, for example, a lattice structure must be calculated under the assumption of linear-elastic behaviour but only that investigation of the individual members/cross-sections from the stress resultants found must be in accordance with the theory of elasticity, provided the strength values and simplified design methods used in the code are used directly.

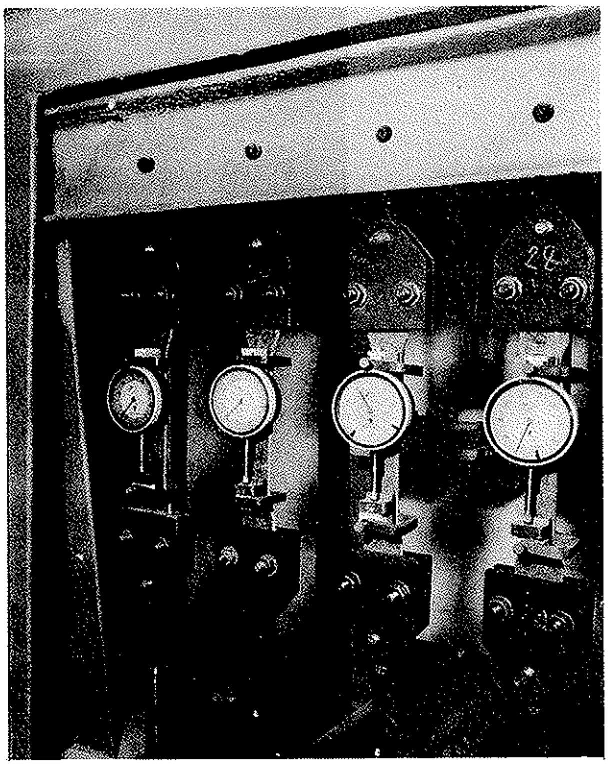
For the serviceability limit states linear methods of analysis will usually be appropriate.

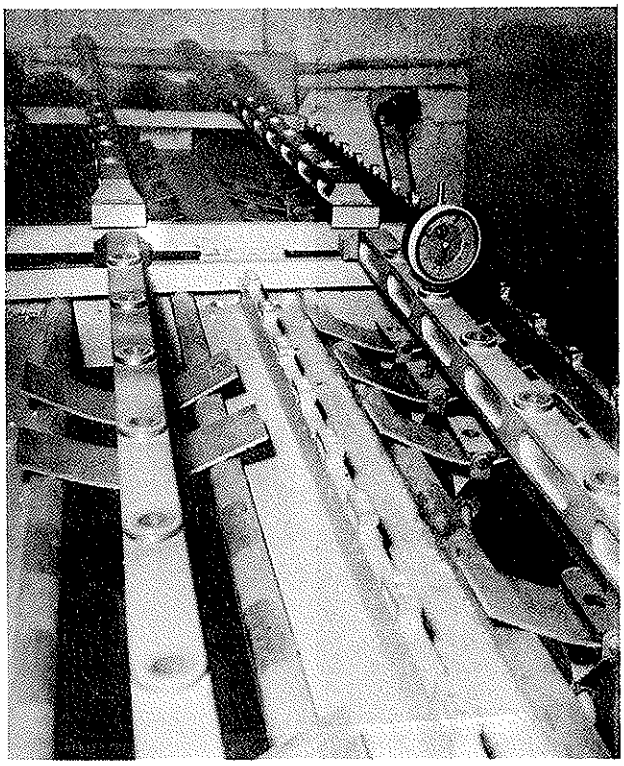
In the calculation of distribution of forces in statically indeterminate structures consideration should be given to unfavourable slip in joints etc.

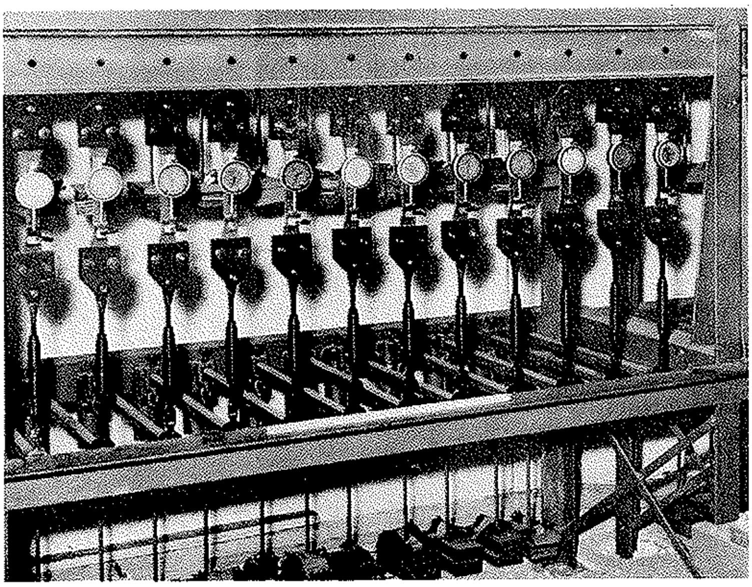
Plates relevant to CIB W18 Papers 1, 2 and 3

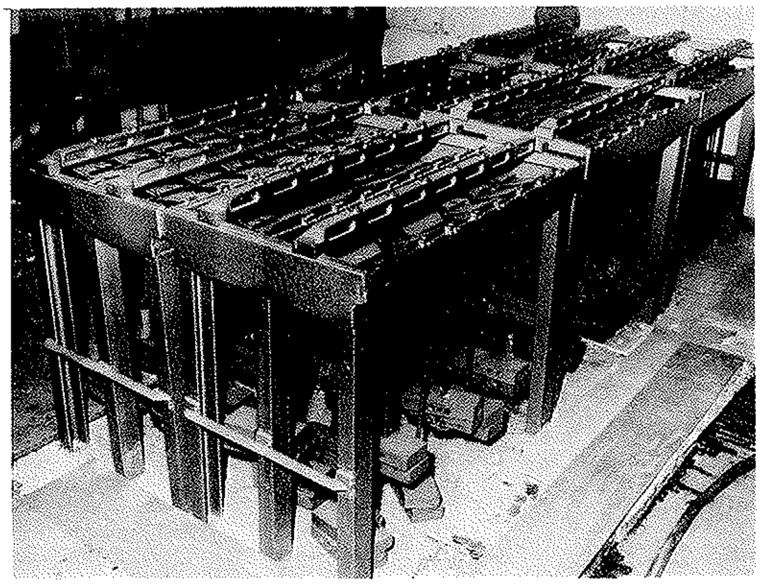












CIB-W18/ 11-102-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

EUROCODES

by

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VIENNA, AUSTRIA
MARCH 1979

The EEC has decided that common codes, Eurocodes (EC), are to be drawn up for the construction area.

7 Eurocodes have been planned so far, namely:

EC1	General rules, including safety regulations
EC2	Concrete
EC3	Steel
EC4	Composite structures (concrete-steel)
EC5	Timber
EC6	Masonry
EC7	Foundation

It is assumed that they will come into force as optional standards, meaning that the users can choose whether they will use Eurocodes or the national codes of the country in which the structure is to be built. For the authorities, however, there is no option: The member countries must accept the rules, at least for public building.

Working groups and drafting groups are set up for each EC. The working group for EC1 will also function as coordinating group for EC2 - 7, cf. the organization plan.

Each working group has up to 3 representatives of each country and no more than 2 for each of the relevant international technical organizations. For EC1 these will probably be JCSS, CEB, CECM, CEI-BOIS and FIEC.

The working groups set up drafting groups with no more than 2 members per country and a total of two for the organizations. One of the national representatives should come from the working group the other should be a specialist.

The working groups will be assisted by a secretariat drawing on the relevant experts; for EC1 it is Dr. Rowe, the English editor of CEB, Volume I.

The work with EC1 has now started while EC2 and EC3 will wait for about half a year. EC5 will probably be actual in the spring of 1980.

The time limit in the EEC-commission is 20 months for elaboration of the technical sections of the proposals and then 9 months for »unofficial» consultations in the member countries. Then the proposal is handed over to the Council of Ministers who will be responsible for the official consultations.

A controversial issue is the implementation : It is the object of the commission to try to implement EC1 in conformity with §100 (unanimity) with the authority to implement the others after §155 (majority).

Concerning EC1 it should be noted that it is only the intention to prescribe the general principles for the safety determination and that it is up to the individual countries to determine the safety level, i.e. the partial coefficients (perhaps only in the form of scale factors to fixed sets of coefficients).

Especially on the cooperation CIB-W18 / CEI-BOIS

On application to the Council of Ministers the CEI-BOIS have established themselves as representatives of the timber area and have prevented CIB from participating.

To attain this members of CEI-BOIS have stated that they would be able to prepare a number of code proposals within a short time. Subjected to pressure the chairman demanded at a meeting in November 1978 that CEI-BOIS before the end of the year should submit a list of contents. CEI-BOIS have not been able to do this and at a meeting in March 1979 they were not even able to give the titles. It is the general opinion of practically all the member countries that CIB alone will be able to deliver a usable foundation for the work.

On this background the continued resistance of CEI-BOIS against establishing a reasonable cooperation with CIB-W18 is obscure, and if not they draw us into the work, we should in my opinion inform CEI-BOIS and EEC that CIB's proposal will not be at disposal as a working basis.

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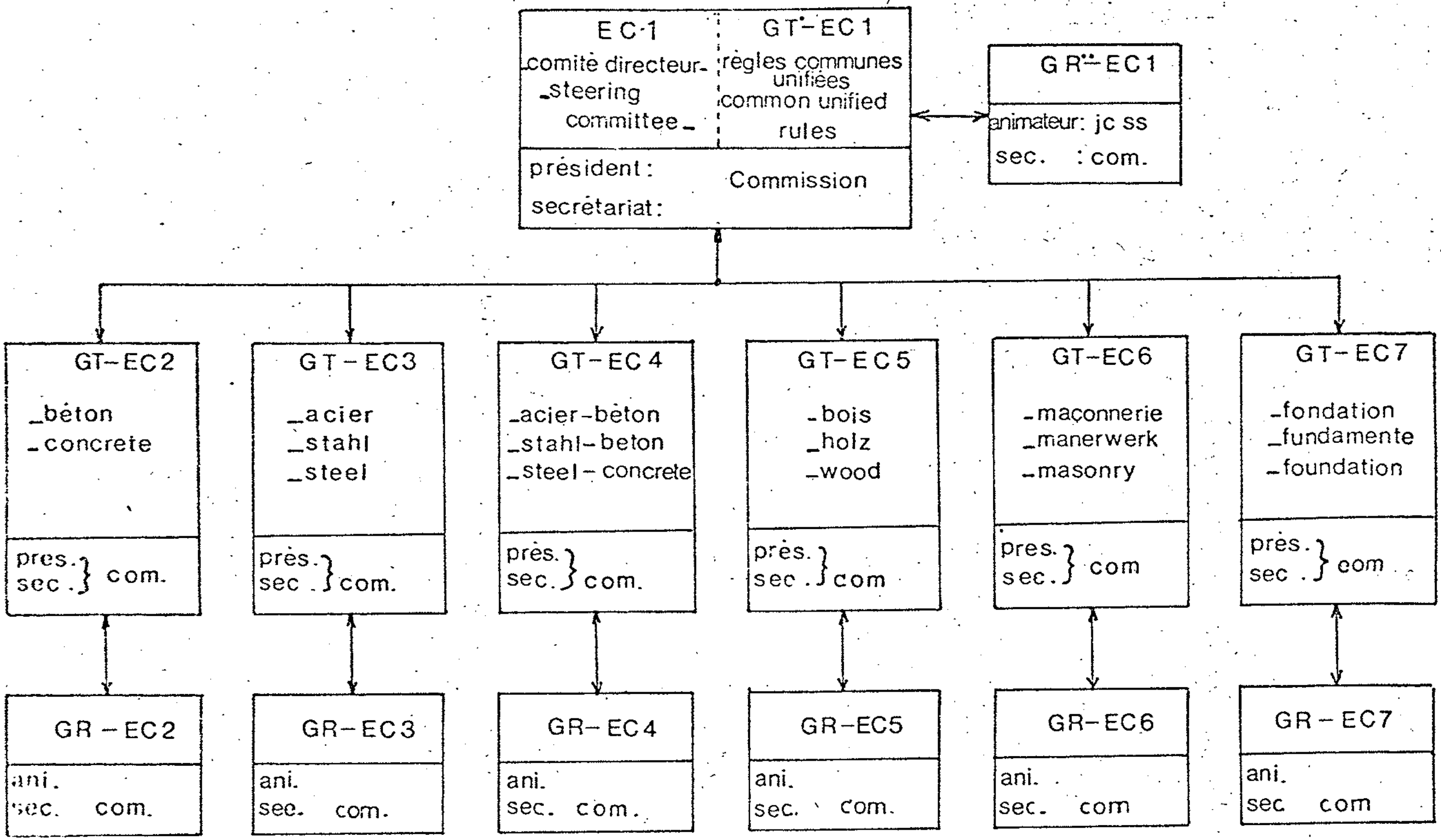
« EUROCODES »
construction



GT = WG = AB = Groupe de travail = Working group = Arbeitsgruppe
GR = DG = RG = Groupe de rédaction = Drafting group = Redaktionsgruppe
Animateur = animator . Präsident = Président