(4) Möhler

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

MEETING OF WORKING COMMISSION W18 - TIMBER STRUCTURES

TECHNISCHE HOGESCHOOL

DELFT, HOLLAND

20-21 JUNE 1974

3. Sitzung

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1 LIST OF DELEGATES

BELGIUM	
P Sonnemans	Institut Belge du Bois, Brussells
CANADA	
C R Wilson	Council of Forest Industries of BC, Vancouver
D E Kennedy	Eastern Forest Products Laboratory, Ottawa
DENMARK	
M Johnasen	Building Research Institute, Copenhagen
H J Larsen	Danmarks Ingeniorakademi, Aalborg
A R Egerup	Technical University, Copenhagen
H Riberholt	Technical University, Copenhagen
ENGLAND	
L G Booth	Imperial College, London
H J Burgess	TRADA, High Wycombe
E Levin	TRADA, High Wycombe
W T Curry	BRE, Princes Risborough
(1)A P Mayo	BRE, Princes Risborough
(2)J G Sunley	BRE, Princes Risborough
R Marsh	Arup Associates, London
FRANCE	
P Crubile	Centre Technique du Bois, Paris
GERMANY	
H Kolb	Otto-Graf Institut, Stuttgart
K Möhler	Technische Hochschule, Karlsruhe
HOLLAND	
D Korfker	Technische Hogeschool, Delft
J Kuipers	Technische Hogeschool, Delft
H Ploos van Amstel	Technische Hogeschool, Delft
P Vermeyden	Technische Hogeschool, Delft
K Griffioen	Houtinstituut TNO, Delft
A van der Velden	Houtinstituut TNO, Delft
E J Heidema	Houtvoorlichtingsinstituut, Amsterdam
NORWAY	
O Brynildsen	Norsk Treteknisk Institute, Oslo
SWEDEN	
B Norén	Svensak Traforskningsinstitutet, Stockholm
SOUTH AFRICA	
J A Simon	Timber Research Unit, CSIRO, Pretoria

(1) Secretary CIB-W18

(2) Co-ordinator CIB-W18 and Chairman for Meeting

2 CHAIRMAN'S ADDRESS

MR SUNLEY, as Co-ordinator of CIB-W18 and Chairman of the meeting, welcomed the delegates to this, the third meeting of the reconstituted Timber Structures Group. He reminded delegates of the terms of reference under which the group operated, which are as follows:

> "To study and highlight the major differences between the relevant national design codes and standards and suggest ways in which the future development of these codes and standards might take place in order to minimise or eliminate these differences."

He went on to say that in view of the large agenda for the two day meeting he thought that future meetings should be for three days with the work split into two sections. The first two days would be taken up with the presentation and discussion of technical papers on specific topics and the third day would be devoted to the discussion and drafting of codes and standards.

Finally he outlined the programme for the two day meeting and this was approved by the delegates.

3 CORRESPONDENCE

MR MAYO, Secretary CIB-W18, reported that two papers had been received from Russia on "Long Term Carrying Capacity of Timber Structures from Short Term Testing" by Yu M Ivanou. However, as one was in Russian and the other in German it was proposed that they be held over until the next meeting. This was agreed and Professor Mohler undertook to report back on the contents of the German paper and Dr Noren and Dr Kuipers the Russian one.

Mr Sunley said that he had had correspondence with the CIB Secretariat regarding membership of CIB-W18 and had been informed that whilst any members of CIB could take part in the activities of the group, leading timber engineers who were not members of CIB could be invited to participate in the work if they had a valuable contribution to make.

MR SUNLEY also informed the group that the Sixth CIB Congress was being held in Budapest on 2-10 October 1974 and he would attend. However, if any other member also wanted to attend they could get further information from him.

Finally the delegates were informed that a joint UN/ECE Seminar on Massed Produced Timber Components was being arranged in Oslo during June 1975 and CIB-W18 had been invited to participate. MR SUNLEY asked if any delegate wished to represent the group at the seminar and write a short article on the work of CIB-W18. However, no-one felt they could undertake this at the present time.

4 CEB PROPOSALS FOR AN INTERNATIONAL SYSTEM FOR STRUCTURES

MR SUNLEY introduced this item by briefly describing some of the groups concerned with the drafting of European and international codes and standards. These were the International Standards Organisation (ISO) which is comprised of Technical Committees dealing with a wide range of subjects. The committee dealing with structural design is TC 98 "Bases for Design of Structures" and this is divided into a number of sub-committees dealing with terminology and symbols, safety of structures, loads and forces, and deformations. In addition other Technical Committees deal with materials ie, TC 139 - plywood, TC 55 - timber, TC 71 - concrete. A second organisation is the Economic Commission for Europe (ECE) which is concerned with the production of European codes. This organisation has links with the Comité Européen du Béton (CEB) which has proposed an "International System of Unified Standard Codes of Practice for Structures" to cover all building materials. Volume I in this system deals with structural design methods in general and Volumes II-IV deal with different materials and specific methods of design related to those materials and the general principles laid down in Volume I. MR SUNLEY said that it had been proposed by CEB that CIB-W18 should undertake to draft Volume V which deals with structural timber design. He then asked the delegates for their comments on the following:

i Should CIB-W18 undertake to draft codes for organisations such as ISO and CEB?
ii To whom should such drafts be submitted?

MR KENNEDY said he would not like CIB-W18 to undertake code drafting work and he did not think this came within the terms of reference of the group. Furthermore he was concerned that the work of CIB-W18 appeared to be towards European codes and standards, which may not be acceptable in USA and Canada and it should be remembered that CIB-W18 is an international group. MR SUNLEY agreed that at present the emphasis was on European codes but he considered that the production of a European code for structural timber was the first step towards an international code and for this reason it was important that USA and Canada, together with other countries outside Europe should participate in the work so that their views could be fully considered at an early stage.

PROFESSOR LARSEN said that CIB-W18 already did 80% of the work required in drafting codes and therefore the group should complete the work by drafting the relevant sections of a code or technical standard. However, he pointed out that such codes would only be advisory whereas building regulations, which were usually dealt with by governments, were mandatory and he did not think that CIB-W18 should get involved with this sort of work.

PROFESSOR MOHLER said that government officials were included on the committees within CEB and as this was not so with CIB-W18 he was concerned that any draft codes submitted to CEB for inclusion in their unified system of structural codes, would not carry sufficient weight and may be substantially amended.

DR KUIPERS said that CIB-W18 drafts should be submitted to ISO, which does include government representatives, in addition to submitting them to CEB.

DR BOOTH pointed out that within ISO there are a number of Technical Committees dealing with timber and timber based products, ie TC 55 - solid timber, TC 89 - Fibreboard, TC 139 - plywood, TC 151 - particle board, and it is not clear which, if any, of these could adequately deal with draft codes submitted by CIB-W18. MR SUNLEY agreed with this and said he thought there may be a case for setting up a new ISO/TC to deal specifically with the structural use of timber and related materials.

MR CURRY said that there was no doubt that an attempt would be made to draft a structural code for timber and if CIB-W18 did not do it then some other organisation, possibly with interests not confined solely to timber, would do the work. Alternatively CEB may be forced to set up the rules in their proposed unified system of structural codes without giving adequate consideration to timber. In either event the timber industry would complain that their interests were not sufficiently represented and for these reasons MR CURRY thought that CIB-W18 was the most suitable organisation to draft a structural code of timber.

In conclusion MR SUNLEY said that it appeared that the majority of the delegates were in favour of CIB-W18 drafting a code on structural timber design and when this was completed it could then be decided which would be the best method of getting it introduced and adopted, ie either through ISO or CEB, or both.

5 ISO DRAFT - "METHOD FOR PREPARATION OF STANDARDS CONCERNING THE SAFETY OF STRUCTURES"

Prior to the meeting the ISO draft document ISO/DIS 3250 was circulated to members of CIB-W18 with a request for written comments. Nine members replied and subsequently these comments were discussed by the delegates at the meeting. It was agreed that more suitable documents on this matter already existed and these were not in agreement with the ISO draft. These alternative documents were the CEB proposals for an "International System of Unified Standard Codes of Practice for Structures" and the CIB-W18 "Framework for the Production of an International Code of Practice for the Structural Use of Timber".

Finally it was agreed to issue a statement for submission to ISO/TC 98. This statement was as follows:

> ISO/DIS 3250 - "Method for the Preparation of Standards Concerning the Safety of Structures"
>
> At a meeting of CIB-W18 in June 1974 the above document was discussed.
>
> The delegates agreed that the draft, ISO/DIS 3250, was not a very helpful document and could even be harmful since it could conflict with the proposed formats being used by other organisations in drafting codes. In this we specifically refer to the format proposed by the Comité Européen de Béton (CEB). For this reason we would not like to see this draft published.

Generally the delegates at the meeting were unable to agree whether or not such a document should contain both permissible stress methods of design and limit state methods. However, they did not like the method of preventing this in Clause 2.01.

With regard to detailed comments, it was felt that:

i Clause 3 should be in a load standard covering all materials and not in a standard related to a particular material.

ii Clause 7 was unnecessary.

iii Clauses 3,4,5 and 6 need to be clarified to make sure that it is clearly understood.

- a) How a structure is to be analysed
- b) Whether or not the stresses and forces are in agreement with permissible stress levels.
- c) What are the necessary design details to be considered.

It was agreed that Mr Mayo should send the above statement to the Secretary ISO/TC 98 following the meeting.

6 ECE UNIFIED SYSTEM FOR STRESS GRADING

MR SUNLEY introduced the topic with a brief outline of the first ad hoc meeting of stress grading experts organised jointly by a combined ECE/FAO joint committee on timber which took place in Geneva in October 1973. Following this meeting a drafting sub-committee was appointed to draft a set of stress grading rules for Europe to put before the full committee in October 1974. Mr Curry and M Crubile, both members of CIB-W18, were appointed to this drafting sub-committee.

MR CURRY said that the sub-committee had been criticised for allowing insufficient time for discussion. However, as there was a need for urgency in producing a draft, time was limited and it was decided not to deal with the assigning of stress values to the proposed grades and the sub-committee limited itself to grading limits and the related yields. Two visual grades EC1 and EC2 were proposed and these were based on the knot area ratio (KAR) method of grading recently published in the British Standard BS 4978:1973 "Specification for Timber Grades for Structural Use". However, it was expected that the stresses assigned to the European grades would be higher than those proposed for the British grades and also it was probable that different limits would apply for the European grades when a margin condition existed. Under the British grading rules the yields on a parcel of Swedish fifths would be on average 84% Special Structural (SS), 10% General Structural (GS) and 6% reject. However, if the proposed changes for the margin condition in the European grades were adopted then the reject portion would increase by about 1%.

MR HEIDEMA said that at a recent Softwood Exporters/Importers Conference delegates were not satisfied with the BS 4978 - GS grade and Holland had introduced a KAR margin condition of 2/5.

At this point MR SUNLEY recommended that the meeting should not get deeply involved with the details of the European grades but he would be interested to hear general comments on the proposals.

DR NOREN said that the grading rules should be arranged so that stresses could be given for a particular grade without reference to the species of the timber.

MR CURRY said that this would only be possible if different grading rules were made for each species otherwise the stresses assigned to a grade would be significantly lower for many species than if a distinction was made between the species.

MR SUNLEY said he thought that the grouping of species and stresses should be left to the structural code and not dealt with in a grading code.

PROFESSOR LARSEN said that whilst he was in agreement that there should be a unified set of stress grading rules he thought each country should be allowed to assign the stresses to the grades and provision should also be made for additional grades to be implemented by each country.

DR KUIPERS agreed with this but suggested that the grades should be higher than the BS 4978 grades so that they would match the stresses used in Holland.

MR CURRY said that the Canadian delegation had asked for lower grades and the committee recognised that this could be done by each country if required.

MR KENNEDY asked what influence Canadian opinions had on ECE in Geneva and he was informed that Mr W Townsley of Canada was chairman of the ad hoc meeting on grading rules. MR KENNEDY went on to say that in Canada the timber producers set the grading rules and later other timber experts assigned the stresses to whatever grades were produced.

In conclusion MR SUNLEY said that members were generally agreed on the need for a unified set of stress grading rules and he proposed that one of the subjects for the next meeting of the group should be the derivation of stresses from tests on different grades of timber. He said that this would involve establishing a consistent set of test methods and agreeing a standard method for the derivation of stresses from the test results.

7 STANDARD METHODS OF TESTING FOR PLYWOOD PROPERTIES

DR KUIPERS introduced his paper "Standard Methods of Testing for the Determination of Mechanical Properties of Plywood" and asked for comments and discussion.

DR NORÉN thought that the paper should include a section on the techniques of sampling for test specimens and these techniques should link the tests on small clear samples with those on full size panels. This in turn would determine to a large extent the methods to be used for the derivation of stresses from the test results. He thought that the work of establishing suitable sampling techniques could be carried out by CIB-W18.

PROFESSOR LARSEN suggested that testing methods should also be available for different sizes of panel related to the end use. In addition tests were also needed for medium size panels. DR KUIPERS replied that this had been considered and in his opinion there was a size of test specimen beyond which, size had no significant effect on the test results. He suggested that the maximum size of test specimen should be 250 mm. DR NORÉN agreed with Dr Kuipers but thought that a maximum size of 300 mm was more suitable. DR WILSON did not agree with this view and said that full size panel tests had been developed to take account of defects within a panel. He was not convinced that an average of the test results obtained from tests on a number of samples from one panel would give the same results as tests on the complete panel.

DR BOOTH said that tests on small clear samples would be useful for assessing the effects of various treatments on plywood such as glue bond strength, preservatives and fire retardants. He agreed that generally test samples of 250 mm or 300 mm would be adequate for the determination of the strength properties of plywood although there may be some special cases where tests on full size panels would be desirable. DR KUIPERS said that one solution to this problem would be to have a standard set of test samples of various sizes linked to end use and then the appropriate size could be tested depending on the particular end use at the time.

MR SUNLEY said that although it was generally accepted that test samples for normal dry conditions should be conditioned to $(20\pm3)^{\circ}C$ and a relative humidity of $(65\pm3)\%$ (Page 24) he thought that the high humidity test conditions of $(20\pm3)^{\circ}C$ and a relative humidity of $(85\pm3)\%$ (Page 25) were a new proposal and had not yet been generally accepted. He also asked Dr Kuipers to clarify his recommendations regarding "rate of testing" (Page 25). DR KUIPERS explained that his recommendation was that a standard time should be set to take the test sample up to the ultimate load and this time should be within the range 2-5 minutes. PROFESSOR LARSEN suggested that a better alternative was to specify a constant rate of strain such that the test sample reached the ultimate load not less than 2 minutes or more than 5 minutes after the start of the test. This was generally agreed by the delegates although PROFESSOR LARSEN pointed out that with some testing machines there may be practical difficulties in maintaining a constant rate of strain.

Commenting on the panel shear tests (Page 26) DR KUIPERS said that in the USA work had been done to compare the two sizes of test samples used and the two rail test (see pages 18-19) and it was concluded from this work that the two rail test was best.

On the rolling shear test (page 26) DR KUIPERS said that he thought the current ASTM test method (page 21) was dangerous to personnel and a better method was required. MR RIBERHOLT said that the method proposed by Dr Kuipers (page 26) would give rise to tension stresses perpendicular to the grain of some of the plies. After further discussion it was agreed that the British Standard test (see page 20) was probably the best or alternatively some form of compression test with the plywood test sample glued to softwood members to form a sandwich.

DR BOOTH said that he thought there was a need to devise a standard test to assess the bending strength and stiffness in the plane of the panel (page 27) DR KUIPERS generally agreed with this.

DR NORÉN commented that a knowledge of the effect of long term loading on plywood was required. He thought that this could be achieved by either devising a standard test to be included in a plywood testing standard or by including plywood as a further variant in any work on long term loading. DR KUIPERS suggested there was a need for a method to predict long term performance from short term tests. PROFESSOR SONNEMANS said that maybe the type of glue used in the manufacture of plywood would have an important effect on long term performance and as an example he mentioned the known tendency of some phenolic glues to creep under prolonged loading.

MR SUNLEY said he considered that the implications in the last sentence of the paper (page 27) relating to other board materials were important. He suggested that it might be desirable if the test methods devised for plywoods could be applied to other board materials although he agreed this may be difficult. DR BOOTH thought this was a question of priorities and he preferred to confine the tests to plywood initially with the possibility of modifying them later when the structural use of other board materials showed signs of increasing. DR NORÉN said that because the important properties may not always be the same ones for different board materials it may be uneconomic and undesirable to have a standard set of test methods for all board materials.

MR SUNLEY asked the delegates for their comments on the next stage to be taken with this work. DR KUIPERS suggested that, after amendment to take account of the points raised in discussion, the paper should be submitted to RILEM who after comment would submit it to ISO as a proposal for standard methods of test for plywood. DR BOOTH said that there were already ISO groups working on small clear tests and the specification of plywood and ISO/TC 139 - plywood, may think that they should be setting up standard test methods for plywood without realising the work which CIB-W18 has already done. In addition the British Standards Institution held the view that the quality control of plywood should be linked with the testing methods. DR BOOTH went on to suggest that as the inclusion of some large size panel tests was being considered, it would be desirable if the RILEM committee could include somebody from North America where these tests were developed, such as Dr Wilson, DR WILSON agreed with this and said that it was important to have a consistant approach involving both small clear tests and large size panel tests. DR KUIPERS said that the RILEM 3TT Committee had four main members and they had good links with North America, however, there were also four or five corresponding members and this type of membership would be open to Dr Wilson. Summing up MR SUNLEY proposed that Dr Wilson and Mr Kennedy be asked to send their comments on the proposed test methods to DR KUIPERS who would submit the paper and comments to RILEM with the request that after adding their comments RILEM should submit it to ISO. This was agreed by the delegates.

8 STRENGTH AND STIFFNESS OF PLYWOOD

In the absence of the author C K A Stieda, MR KENNEDY introduced the paper "Bending Strength and Stiffness of Multiple Species Plywood". He said that the paper assumes that data already exists on the strength properties of the individual plies making up the plywood and suggests a method for using this data to obtain design stresses for the plywood. Both the parallel plies and the full cross section approaches had been considered.

PROFESSOR LARSEN said that the basic theory in the paper was not new and there was no practical experimental evidence to show that it worked. However, there were other theories (B Norén 1954) in existence which had been shown to be capable of predicting strength properties. DR BOOTH agreed with Professor Larsen and suggested a fresh approach to the problem should be made.

DR WILSON said that he was involved in a research programme with the Canadian Forest Products Laboratories which had the objective of relating plywoods of different species to a standard plywood, ie Douglas fir, by means of bending tests. This work would ultimately lead to the development of a method for predicting the strength properties of different plywoods. He agreed to keep CIB-W18 informed on this work although he thought that there may be some difficulty about releasing the information but this would be decided by the Eastern and Western Forest Products Laboratories.

DR BOOTH said that he considered that predictive theories could be divided into three groups - linear elastic, elasto plastic and plate theory. He suggested that the best theory might be an elasto plastic one along the lines first proposed by Dr Norén in 1954. However, there would be a problem in establishing the various parameters related to the plies.

MR CURRY said there was still work to be done in looking at the suitability of all the predictive methods at present in existence, but that it still might be necessary to resort to the kind of approach suggested earlier by Dr Wilson where different plywoods were related back to some base plywood by means of small scale tests.

MR KENNEDY said that although it might be difficult to develop a suitable method for predicting the strength properties of plywood he thought it was necessary to have such a method if only as a kind of emergency provision.

Finally it was proposed by MR SUNLEY that Dr Booth be requested to write a paper on this subject for the next meeting of the group. This was agreed and Dr Booth accepted the proposal.

9 SYMBOLS FOR STRUCTURAL TIMBER DESIGN

DR NORÉN and DR KUIPERS introduced their joint paper "Symbols for Structural Timber Design" which was then discussed at length by the delegates. DR NORÉN also informed the meeting that only a few days earlier the ISO/TC98-SC 1 "Terminology and Symbols" group had held a meeting in Paris where symbols for use in structural design had been discussed. However, as yet no details of this meeting were available but a further meeting of the group had been called for October 1974 in Stockholm. DR NORÉN proposed that he and DR KUIPERS should amend the CIB-W18 paper to take account of the comments of the delegates at the present meeting and also the proposals discussed at the recent ISO meeting. The amended list of symbols should then be submitted to ISO/TC 98-SC 1, by the secretary of CIB-W18, for discussion at the October meeting of

ISO/TC 98-SC 1. This was agreed by the delegates.

It was proposed by MR SUNLEY, and agreed, that all future CIB-W18 papers should be written with symbols in accordance with the amended list to be produced by DR NORÉN and DR KUIPERS. He also suggested that members of CIB-W18 should encourage the use of the CIB-W18 symbols generally in their respective countries and this would be helped if they were published. Dr BOOTH agreed with this and suggested that the CIB-W18 list of symbols should be circulated to all IUFRO timber engineers. This was agreed. PROFESSOR MOHLER said that although he agreed that the CIB-W18 symbols should be used in the work of the group he felt there would be difficulties.in introducing them in Germany as there had recently been a new draft of DIN 1018 on symbols.

10 BUILT-UP TIMBER COLUMNS

PROFESSOR LARSEN introduced his paper "The Design of Built-up Timber Columns" which is a follow up paper to the one he submitted to the previous CIB-W18 meeting in Copenhagen 1973, on the design of solid timber columns.

PROFESSOR MOHLER said that since Professor Larsen had written his paper the results of further experimental work, in Germany, on built-up columns, had been published which may be useful to the work of CIB-W18. He went on to say that under the present DIN standards bolts were forbidden in built-up columns except glued constructions, because of problems of relaxation due to shrinkage of the timber. A further requirement in Germany was that the slenderness ratio of the central section of a lattice column should be less than that at the ends. PROFESSOR LARSEN agreed that bolts should be forbidden in built-up columns because even if the shrinkage was insignificant some slip always occurred even if split ring connectors were used. With regard to the variation in slenderness ratio along the length of the column PROFESSOR LARSEN considered this to be the normal method of design. However, PROFESSOR MOHLER did not think that this was the case. DR KUIPERS questioned the ban on the use of bolts with connectors because he maintained that the split rings would take up any movement and remain tight.

MR BURGESS raised the problem of designing columns to take lateral loading and asked how this could be taken into account. PROFESSOR MOHLER said the German Standard DIN 1052 expressly forbids this type of loading on columns except for the normal wind loading on the faces of the column. PROFESSOR LARSEN said that he did not know of any standards or codes where this type of loading was discussed and an engineer would have to use his experience and take full responsibility for any designs he produced with this type of loading. PROFESSOR LARSEN thought that it was a dangerous situation and should be avoided whenever possible.

DR KUIPERS asked what percentage of the short term test load could be considered as a safe long term load. He explained that the Dutch code had a series of factors related to the duration of loading and the slenderness ratio and the British code also had a similar series. He suggested that more efficient designs may be obtained for columns which do not carry the design load continuously, by taking the periods of partial loading into account. PROFESSOR LARSEN disagreed with this saying that during the periods of full loading there would be slip in the spacer connections and a gradual increase in deflection and bending leading to collapse if the columns were not designed for the maximum load. Furthermore he maintained that as a general principle structures were designed to carry the maximum load whether it be continuous or not.

PROFESSOR MOHLER questioned the relationship in section 5.2.1 (page 33) between the stiffness and diameter of the nails and the thickness of the spacers (ie, distance between the main elements of the column) - $^k/_d = {}^{450}/_\alpha$. He said that from his own research work the constant 450 was too large. DR KUIPERS and DR VERMEYDEN both agreed with Professor Mohler and said that from their own experiences a smaller constant would be more suitable.

In conclusion MR SUNLEY said that at the next meeting of the group he hoped the position regarding the use of bolted connections and split rings could be resolved which would then allow the present paper on built-up columns to be combined with an earlier paper by PROFESSOR LARSEN on solid columns to form the CIB method for the design of timber columns. When this was achieved he asked delegates to try and introduce the method into their respective countries.

11 LONG TERM LOADING

DR NORÉN presented his paper "Definitions of Long Term Loading for the Code of Practice" and asked the delegates for their comments. He also said that the proposed Nordic code of practice on timber structures would include the method he had developed in his paper.

MR SUNLEY said that Dr Norén had based his approach on a rheological model and it would be helpful if a second approach could be made based on experimentation.

Delegates went on to discuss the work of Mr B Madsen, on long term loading, at the USDA Forest Products Laboratory, Madison, USA and concluded that his work tended to show that the effect of long term loading was more marked on high grade timber than on low grade material but more significant was the level of applied stress.

MR KENNEDY described the recently initiated work on long term loading at the Eastern Forest Products Laboratory, Canada where a series of timber beams are to be fully loaded for a period, representing the winter snow loads. Following this period the "snow" loads will be removed and the recovery measured during the following "summer" period after which the cycle will be repeated. PROFESSOR MOHLER commented that this type of cyclic loading tended to cause failure at a lower load than would be achieved if the loading was continuous to failure.

MR SUNLEY suggested that a survey of the ways in which timber engineers dealt with long term loading at present would be useful. However, DR NOREN said that it was already known that all the current methods were based on the "Madison curve". MR KENNEDY commented that this curve really referred to constant loading in constant conditions which was different from the practical situation and the main problem was how to relate these ideal conditions to the practical situation.

DR NORÉN suggested that the next step was to go to the material side and put the results of experimental work into the theory to establish what modifications, if any, are necessary to the theory. This was agreed but due to the work which had already been undertaken for the next meeting of the group it was not possible for any of the delegates to take this extra work on at present. MR SUNLEY therefore concluded the discussion by asking delegates to consider Dr Noren's paper at greater length and to submit to the next meeting any articles or statements which they thought relevant to the work.

12 LOAD SHARING

MR LEVIN introduced his paper "Load Sharing - An Investigation on the State of Research and Development of Design Criteria" by saying that it was intended to be a catalogue of present information, principles, work completed and work in progress.

He pointed out that he had circularised members of the group for the information on the subject prior to writing the paper but had received few replies. He repeated his request for information so that he could write a follow up paper for the next meeting saying that he was particularly interested in information relating to experimental testing. MR LEVIN said that the factors on which load sharing depended were the natural variability of timber, which was already well documented, and the transfer mechanism by which the loads were transferred. MR SUNLEY said that he considered the problem could be divided into two categories. The first was the type of load sharing which occurred when a number of similar members connected by a stiff diaphram, such as floor joists and boarding, supported a uniformly distributed load. In this case the natural variability in the stiffness of the joists caused the weak joists to shed load to the stronger ones, through the boarding. The second case was where the same type of structure was subjected to a concentrated load, when the boarding distributed this load to a number of joists in the vicinity of the load. In this case the natural variability in the stiffness of the joists was less significant with regard to the degree of load sharing than in the UDL case.

In conclusion MR SUNLEY proposed that Mr Levin should write a paper for the next meeting of the group defining the different types of load sharing which can occur, together with an explanation of how different codes and standards take account of load sharing in design. Such a paper might include an outline of the background to the various methods in the codes and a discussion of any research work being carried out at the present time. This was agreed by the delegates and Mr Levin undertook to produce a paper along the lines described.

13 FUTURE PROGRAMME OF WORK

In opening the discussion on future programme of work of W18 MR SUNLEY said that the group had now become well established with an active membership of about twenty five and in a number of the areas the work had reached the stage when firm proposals and recommendations could be submitted to other international organisations or published as a CIB-W18 recommendation. Because of the mounting volume of work it was now necessary to extend the next meeting of the group to three days, during which time, it had been agreed that the following subjects would be discussed.

i) Solid and spaced timber columns - The paper by B Johanson on "Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns" which had been held over from the present meeting due to a lack of time was already on the table and Professor Larsen had undertaken to write a concluding paper to follow his previous two papers on the subject. At the next meeting it was hoped to agree a CIB-W18 method for the design of solid and spaced timber columns.

ii) Plywood - Dr Booth had undertaken to write a paper on predicting the strength properties of plywood from a knowledge of the lay up of the board and the properties of the individual plies. Dr Kuipers had agreed to amend his paper on testing methods for plywood, on the receipt of written comments from Mr Kennedy and Dr Wilson. The amended paper would then be submitted to RILEM for comment and Dr Kuipers would report back to the next meeting.

iii) Long term loading - Articles and statements to be submitted by the delegates for discussion at the next meeting.

iv) Load sharing - Mr Levin had agreed to write a further paper on load sharing describing the various types which can exist, outlining the methods used to take it into account in design and describing any current research being undertaken on the subject.

v) Testing of joints - As a new area of work for the group Dr Kuipers agreed to write a paper on the methods of testing and the interpretation of the test results.

vi) Derivation of stresses - Mr Curry agreed to write a paper on the methods used to derive the design stress at present in use in different countries with the intention that this would lead to the development of a standard method for the derivation of stresses from test results.

vii) Design of beams - A paper by Professor Larsen on the design of beams subjected to normal force and axial loading, which had been held over from a previous meeting, would provide a suitable introduction to this second new area of work for the group.

The above subjects would be discussed during the first two days of the next meeting and the third day would be devoted to work on codes and standards. This would include.

i) A discussion on the progress and developments in the CEB proposals for a unified structural code.

ii) A discussion on the Nodic proposals for a structural code.

iii) The current revision of the British code CP 112.

iv) Amendments to the CIB-W18 format for an international code on timber structures which was proposed at the previous meeting of the group in Copenhagen 1973.

In conclusion MR SUNLEY informed the meeting that M Crubilé had kindly offered to act as host to the next meeting which could take place at the Centre Technique du Bois, Paris. This was gratefully accepted by the delegates and the date for the next meeting was fixed for 19-21 February 1975. Finally on behalf of all the delegates MR SUNLEY thanked Dr Kuipers for organising the present meeting and for the warm hospitality offered to the delegates by him and his colleagues.

14 PAPERS PRESENTED AT THE MEETING

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

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

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

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

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

Paper 6 The Design of Built-up Timber Columns - H J Larsen.

Paper 7 Definitions of Long Term Loading for the Code of Practice - B Norén.

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

PAPER 1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

INTERNATIONAL SYSTEM OF UNIFIED STANDARD CODES OF PRACTICE FOR STRUCTURES

by

COMITÉ EUROPÉEN DU BETON

DELFT - JUNE 1974

COMITÉ EUROPÉEN DU BÉTON

CEB

7/05/1974

Ref. YS/SP - 423/74 E

Proposal for an
INTERNATIONAL SYSTEM
OF UNIFIED STANDARD CODES OF PRACTICE FOR STRUCTURES

established by the EUROPEAN COMMITTEE OF CONCRETE (CEB) and the INTERNATIONAL FEDERATION FOR PRESTRESSING (FIP)
in collaboration with the EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK (CECM),
the INTERNATIONAL COUNCIL FOR BUILDING RESEARCH, STUDIES AND DOCUMENTATION (CIB),
the INTERNATIONAL UNION OF TESTING AND RESEARCH LABORATORIES (RILEM)
and other organizations of Civil Engineering.

VOLUME I - COMMON UNIFIED RULES FOR DIFFERENT TYPES OF CONSTRUCTION AND MATERIAL

VOLUME II - MODEL CODE FOR CONCRETE STRUCTURES

VOLUME III - MODEL CODE FOR STEEL STRUCTURES

VOLUME IV - MODEL CODE FOR COMPOSITE STEEL CONCRETE STRUCTURES

VOLUME V - ... Model codes for other structures and materials

VI

etc...

Volume I

COMMON UNIFIED RULES
FOR DIFFERENT TYPES OF CONSTRUCTION AND MATERIAL

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
A - REQUIREMENTS OF SAFETY AND SERVICEABILITY				
1 - GENERAL RULES	VI	J.C.S.S.(1)	III(5)	TC.98/SC.2
Appendix :				
Requirements for fire resistance	-	CIB-W.14	III	?
2 - LIMIT STATE DESIGN (semi-probabilism = level 1)..................	VI	J.C.S.S.	III	TC.98/SC.2
Appendix :				
Limit state design (probabilism = level 2)	VI	J.C.S.S.	III	TC.98/SC.2
3 - GUIDELINES FOR ACTIONS	VI	J.C.S.S.	III	TC.98/SC.2
Appendices :				
Loads in habitation buildings	-	CIB-W.23	III	TC.98/SC.3
Loads in public buildings (schools, hospitals, etc..)	-	CIB-W.23	III	TC.98/SC.3
Loads in industrial buildings	-	CIB-W.23	III	TC.98/SC.3
Loads on road bridges	-	IABSE ? (2)	III	?
Loads on railroad bridges	-	UIC (3)	?	?
Climatic actions (snow, wind)	-	J.C.S.S.	III	TC.98/SC.3
Seismic actions	-	E.Eng.(4)	?	?
etc...				

(1) J.C.S.S. = Joint Committee "Structural Safety" (CEB-CECM-CIB-FIP-IABSE)
(2) I.A.B.S.E.= International Association for Bridge and Structural Engineering
(3) U.I.C. = Union Internationale des Chemins de fer : International Railroad Union
(4) E.Eng. = International Association for Earthquake Engineering
(5) III = Department of Industrial and Technological Affairs

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
B - REQUIREMENTS FOR MATERIALS AND COMPONENTS				
4 - GUIDELINES FOR MATERIALS	VI	RILEM	III/XI (4)	(different)
Appendices :				
4.1 - Agrément and control of reinforcement steel	II	RILEM	III/XI	TC.17 ?
4.2 - Agrément and control of prestressing steel and anchorages of tendons	FIP-Steel	RILEM	III/XI	TC.17 ?
4.3 - Agrément and control of concrete cast in situ	CEB-CIB-	NBF (1)	III/XI	TC.71/SC.3
4.4 - Agrément and control of ready-mix concrete	FIP-RILEM	NBF	III/XI	TC.71/SC.3
4.5 - Agrément and control of cement	-	RILEM	III/XI	TC.74
4.6 - Agrément and control of adjuvants	-	RILEM	III/XI	TC.74
4.7 - Agrément and control f aggregates	-	RILEM	III/XI	?
5 - GUIDELINES FOR COMPONENTS	XIII	UEAtc(2)	III/XI	TC.59
Appendices : "ad libitum" (including composite steel concrete components)				
C - NOTATIONS AND UNITS				
6 - BASIC NOTATIONS	VII	CECM ACI (3)	-	TC.98/SC.1
Appendix : Terminology (trilingual phraseological glossary of important terms)	VII	CECM	III	TC.98/SC.1
7 - UNITS	VII	CECM	-	TC.12

(1) NBF = Nordic Concrete Federation
(2) UEAtc = European Union for the Agrément of Construction Techniques
(3) ACI = American Concrete Institute
(4) XI = Department of Internal Exchanges

Volume II

MODEL CODE
FOR CONCRETE STRUCTURES

(Main text + Comments on the opposite page)

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
1 - GENERAL REQUIREMENTS				
1.1 - SCOPE	I	-	III	TC.71/SC.2
1.2 - GENERAL DESIGN REQUIREMENTS, SAFETY, SERVICEABILITY AND DURABILITY	VI XIII	-	III	TC.71/SC.2
1.3 - SPECIFIC NOTATIONS	VII	ACI	III	TC.71/SC.2
1.4 - UNITS	VII	-	-	-
2 - GENERAL DATA				
2.1 - DATA FOR CONCRETE .... (e.g. σ-ε diagram, shrinkage, creep,...)	IVc XV	-	III	TC.71/SC.2
for the special case of lightweight concrete	XII			
2.2 - DATA FOR REINFORCEMENT STEEL .... (e.g. σ-ε diagram, relaxation, ...)	II	-	III	TC.71/SC.2
for the special case of prestressing steel	FIP-Steel			
2.3 - DATA FOR PRESTRESSING TENDONS .... (e.g. friction, anchorages, ...)	FIP-Steel FIP-Anchorages	-	III	TC.71/SC.2
2.4 - DATA FOR ACTIONS	VI XIII	-	III	TC.71/SC.2
for the special case of losses of prestress	FIP-Steel XV			
2.5 - DATA FOR EXECUTION TOLERANCES TO BE TAKEN INTO ACCOUNT FOR DESIGN	VI XIII	-	III	TC.71/SC.2

3 - STRUCTURAL DESIGN AND DETAILING

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
General statement for this chapter : a) Every subject, when applicable, deals with the four classes on the (complete) level 1 and, by appropriate indexing it will be possible to identify the material relevant to each class.	-	-	-	-
b) Special rules for a simplified level 1 will be given in appendix .				
c) Detailing rules to assure fire resistance will be added to the text, especially in the sections 3.4 and 3.5	FIP-Fire	-	III	TC.71/SC.2
d) In some cases, it may be necessary to provide additional explanatory clauses, e.g. for : lightweight concrete, precast élements, prestressed concrete,....	-	-	-	-
3.1 - DESIGN METHODS				
3.1.1 - Limit states (classes)	IVa	-	III	TC.71/SC.2
3.1.2 - Design through calculation	I	-	III	TC.71/SC.2
3.1.3 - Design by testing	I	NBF	III	TC.71/SC.2
3.1.4 - Safety (partial safety factors)	VI	-	III	TC.71/SC.2
3.2 - DETERMINATION OF ACTION EFFECTS				
3.2.1 - Linear members	XI	-	III	TC.71/SC.2
3.2.2 - Plane structures	X	-	III	TC.71/SC.2
3.2.3 - etc....	-	-	III	TC.71/SC.2
3.2.4 - etc....	-	-	III	TC.71/SC.2

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
3.3 - CHECKING OF LIMIT STATES				
3.3.1 - Limit states of equilibrium	VI etc...	-	III	TC.71/SC.2
3.3.2 - Limit states of bond and anchorage	II FIP-Steel	-	III	TC.71/SC.2
3.3.3 - Ultimate limit states of resistance :				
1 - under normal action effects .... (definition of sections, strain diagrams, analysis, ...)	III	-	III	TC.71/SC.2
2 - under tangential action effects .... 2.1 - shear 2.2 - torsion	V	-	III	TC.71/SC.2
3.3.4 - Ultimate limit states reached by buckling	VIII	-	III	TC.71/SC.2
3.3.5 - Limit states of cracking :				
1 - under normal action effects .... (definition of sections, ...) 1.1 - limit state of decompression 1.2 - limit state of crack formation 1.3 - limit state of cracking control	IVa	-	III	TC.71/SC.2
2 - under tangential action effects .... 2.1 - shear 2.2 - torsion	IVa V	-	III	TC.71/SC.2
3.3.6 - Limit states of deformation :				
1 - bending deflections	IVb X	-	III	TC.71/SC.2
2 - torsional deformations	IVb V	-	III	TC.71/SC.2

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
3.4 -DETAILING OF REINFORCEMENT (GENERAL RULES) (based on the Manuals concerning Reinforcement)				
3.4.1 - Reinforced concrete .... 3.4.2 - Prestressed concrete (tendons, anchorages)	II FIP-Steel FIP-Anchorages	-	III	TC.71/SC.2
3.5 -DESIGN OF STRUCTURAL ELEMENTS : (including special rules for detailing of reinforcement) e.g. : - beams (e.g. bearing zones) - columns (e.g. binding) - frames - floors - slabs (e.g. punching shear) - shells - deep beams - brackets - footings - concrete hinges - specially shaped pieces etc...	I II V X etc..	-	III	TC.71/SC.2

4 - **PRACTICAL CONSTRUCTION, WORKMANSHIP AND SUPERVISION**

4.1 - REQUIREMENTS FOR MATERIALS
- 4.1.1 - Concrete
  - 4.1.1.1 Composition
  - 4.1.1.2 Characteristics
- 4.1.2 - Reinforcement
- 4.1.3 - Prestressing tendons

4.2 - WORKMANSHIP
- 4.2.1 - Classification of supervision
- 4.2.2 - Classification of competence

4.3 - FORMWORK
- 4.3.1 - Construction
- 4.3.2 - Stripping

4.4 - REINFORCEMENT
- 4.4.1 - Properties, classification
- 4.4.2 - Cutting and bending
- 4.4.3 - Fixing
- 4.4.4 - Welding
- 4.4.5 - Special requirements for prestressing tendons

4.5 - CONCRETE
- 4.5.1 - Constituent Materials
- 4.5.2 - Mixing
- 4.5.3 - Transportation
- 4.5.4 - Properties
  - 4.5.4.1 Fresh concrete, classification of workability
  - 4.5.4.2 Hardened concrete classification of strength
- 4.5.5 - Casting (and steaming, etc..)
- 4.5.6 - Curing

BODIES CONCERNED		
CEB, FIP and others	EEC	ISO
In close cooperation with the FIP Commission "Practical Construction"	III	TC.71/SC.3

4.6 - <u>SPECIAL REQUIREMENTS FOR PRESTRESSED CONCRETE</u>
- 4.6.1 - Pretensioning
- 4.6.2 - Posttensioning
- 4.6.3 - Grouting

4.7 - <u>SPECIAL REQUIREMENTS FOR PRECAST CONCRETE UNITS</u>
- 4.7.1 - Transportation
- 4.7.2 - Erection
- 4.7.3 - Jointing

4.8 - <u>DURABILITY REQUIREMENTS</u>
(plan and contents to be worked out)

4.9 - <u>SUPERVISION</u>
- 4.9.1 - Internal control
  - 4.9.1.1 Formwork
  - 4.9.1.2 Reinforcement
  - 4.9.1.3 Concrete
  - 4.9.1.4 Prestressing
  - 4.9.1.5 Inserts
  - 4.9.1.6 Structures
- 4.9.2 - Acceptance inspection
  - 4.9.2.1 Reinforcement
  - 4.9.2.2 Concrete
  - 4.9.2.3 Structures
- 4.9.3 - Observation of structural behaviour

5 - MAINTENANCE AND REPAIR

....
....   (to be produced)
....

BODIES CONCERNED		
CEB-FIP and Others	EEC	ISO
cp. note of page 8	III	TC.71/SC.3
NBF RILEM	III	TC.71/SC.3

6 - USE OF PRECAST COMPONENTS

(with references to chapters 1 to 5)

6.1 - Scope :
(special problems of connections)

6.2 - Precast component +cast in situ concrete
- 6.2.1 - Beam + slab
  - precast beam + cast in situ slab
  - precast slab + cast in situ beam
- 6.2.2 - Slab + slab
- 6.2.3 - Cast in situ column + panel (vertical joint)
- 6.2.4 - Cast in situ foundations + column or panel

6.3 - Precast concrete construction
- 6.3.1 - Segments of beams or arches
- 6.3.2 - Beam + beam
  - coaxial
  - non-coaxial
- 6.3.3 - Column + column
- 6.3.4 - Column + beam
  - no concrete core
  - with concrete core
- 6.3.5 - Linear member + plate component
  - footing + column
  - beam + slab
  - beam + panel
  - column + panel (vertical joint)
  - column + slab
- 6.3.6 - Plate components
  - stripes of plates
    - one-way floors
    - two-way slabs
  - footing + panel
  - panel + panel slab +slab
  - panel + slab
- 6.3.7 - Box units
- 6.3.8 - Precast shells and shell components

BODIES CONCERNED			
CEB-FIP	Others	EEC	ISO
XIII FIP-Prefabr.	-	III	?

	BODIES CONCERNED			
	CEB-FIP	Others	EEC	ISO
APPENDICES TO VOLUME II				
a) Rules for the simplified level 1	I	-	III	TC.71/SC.2
b) Fire resistance of concrete structures	FIP-Fire	-	III	TC.71/SC.2
c) etc...				

PAPER 2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

METHOD FOR THE PREPARATION OF STANDARDS CONCERNING THE SAFETY OF STRUCTURES - ISO/DIS 3250

by

INTERNATIONAL STANDARDS ORGANISATION - ISO/TC 98

DELFT - JUNE 1974

DRAFT INTERNATIONAL STANDARD ISO/DIS 3250

Method for the preparation of Standards concerning the safety of structures

1 SCOPE AND FIELD OF APPLICATION

This International Standard defines the method of drafting of International Standards concerning the safety of structures. These Standards should indicate the rules for applying the limit state theory to the design of structures, taking into account, if possible, a probabilistic or semi-probabilistic concept of load parameters and of the properties of materials.

As, however, these concepts are not generally used by some ISO Member Bodies, it is advisable to treat the matter in these International Standards in such a way as to allow the use of traditional concepts in national standards as an alternative for a transitional period.

Research is always ahead of the current practice on which standards are based. This implies :

- that the recommendations should take into consideration the current methods of structural analysis and should stimulate the development of these methods, according to the results of current research,
- that the recommendations cannot deal with premature decisions on controversial problems.

In order to compensate for this conservative attitude, it is important to have a quick procedure for drafting and approving Standards as well as for their regular revision to take account of the results of research and of technical achievements.

2 LIST OF CLAUSES FOR AN INTERNATIONAL STANDARD

The text of a Standard should correspond to items listed below, with the possibility of introducing any changes, according to the properties of the structures and the materials concerned and the particular conditions.

Clause 0 Introduction

Scope of the International Standard, including the types of structures and materials. Differences in relation to existing rules. The reasons for the Standard or for its revision.

01 Indication of the design methods

	Deterministic methods	Probabilistic methods
Permissible stresses methods		
Limit state methods		

02 Field of application – Definition

Clause 1 Basic principles

The concise description of hypotheses and principles of the limit state method as well as of the semi-probabilistic method. The approach to the determination of the safety parameters, according to the types of structures and materials.

The estimation of applicability of traditional and deterministic methods. The list of basic concepts, definitions and symbols.

1.1 Reference to International Standard ISO 2394.

1.2 Reference to the International Standards concerning materials

1.3 Reference to the International Standards concerning loads

1.4 Reference to the International Standard concerning terminology and notations.

Clause 2 Characteristic values and design values for the strength of materials

The methods of assessment of characteristic and design values of materials properties. Definitions of coefficients introduced. Explanation of particular properties connected with the materials covered by the Standard. Methods and principles of determining the numerical values of coefficients.

Clause 3 Characteristic values and design values of actions (loads and imposed deformations)

The method of assessment of characteristic and design values of loads and imposed deformations. Definitions of coefficients introduced. Explanation of particular problems connected with the actions. Methods and principles of determining the numerical values of coefficients.

Clause 4 Determination of internal forces

The analysis of those characteristics of various types of structures which are important for the appropriate limit states. Hypotheses for calculating the internal forces. Particular specifications related to various structures and materials. Applicability of model tests.

Clause 5 Verification of sections and of structural elements

Detailed design indications concerning various ultimate limit states and serviceability limit states. Determination of internal forces in sections and elements for the verification of their safety.

Clause 6 Correlation between the design, the calculation and the structural detailing

Particular items concerning structural detailing related to safety.

Clause 7 Presentation of calculation and design details

The form of presenting the calculation and design details; the form of presenting the results of tests on models and on structures.

The concise descriptions in this Standard represent the general intentions of ISO/TC 98 and relate to existing documents and experiences. However, for each International Standard, the clauses should be arranged according to the needs of the object of the Standard.

EXPLANATORY REPORT FOR ISO/DIS 3250

ISO/TC 98

Secretariat Poland

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION · МЕЖДУНАРОДНАЯ ОРГАНИЗАЦИЯ ПО СТАНДАРТИЗАЦИИ · ORGANISATION INTERNATIONALE DE NORMALISATION

ISO/DIS 3250 - Method for the preparation of standards concerning the safety of structures

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The work on the document in question was undertaken in June 1972 by Poland. The document was distributed in August 1972 (reference number 98/2 N 27) to all the members of SC 2 "Safety of structures" and TC 98 for comments and consideration at the Paris meeting.

The document was thoroughly discussed by the SC 2. The results of the discussion have been introduced to the new version of this document (reference number 98/2 N 33) which has been adopted unanimously (see resolution No. 11 of SC 2).

After discussion at the plenary meeting of TC 98, the document has been adopted unanimously - by 9 votes of P-member bodies of TC 98 present at the meeting - see resolution No. 38 of TC 98.

Both versions of the document (English and French) have been verified form the standpoint of languages by the AFNOR and the BSI.

PBK

PAPER 3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STANDARD METHODS OF TESTING FOR THE DETERMINATION OF MECHANICAL PROPERTIES OF PLYWOOD

by

J KUIPERS

STEVIN LABORATORIES

TECHNICAL UNIVERSITY DELFT

NETHERLANDS

DELFT - JUNE 1974

Contents

I Introduction
II Bases of information
III General information
IV Tension tests
V Compression tests
VI Bending tests
VII Panel shear tests
VIII Rolling shear tests
IX Other mechanical properties
X Conclusions and remarks

Standard methods of testing for the determination of mechanical properties of plywood.

I Introduction

This report tries to give information about the methods used in different countries for the evaluation of mechanical properties of plywood. Togather this information cooperation was asked and given by the members of CIB - W 18, for which cooperation I am most grateful. Despite this international collaboration this outline probably will not be complete. I hope that readers of this report will be so kind to inform me about other relevant methods which should be held in mind during the preparation of recommendations for testing methods in this field.

Testing methods are only summarised here in such a way that - after my opinion - the most important characteristics have been mentioned. Especially the ASTM gives much more detailed information. If in future recommendations will be worked out, these detailed informations should be used also and incorporated into the recommendations.

II Bases of information

Country	
Australia	Test methods in use are not incorporated in an Australian standard. Information available comes from CSIRO.
Canada	Relevant standards are:

CSA	0115 - 1974	(?)	Hardwood plywood revision expected
CSA	0121 - 1973		Douglasfir plywood
CSA	0151 - 1974	(?)	Canadian softwood plywood
CSA	0152 - 1964		Performance of construction plywood
CSA	0153 - 1963		Poplar plywood

CSA 0152 requires testing for strength and deflection of plywood to be used for roof sheathing or sub--flooring; the other ones attempt to assess the quality of the glue bond in terms of wood failure.

Strength values are determined following ASTM standards with a few exceptions.

Denmark	Plywood is not tested in Denmark.
England	Methods of test for clear plywood are given in BS 4512 : 1969
Germany	Relevant standards are:

DIN 68705: "Sperrholz Baufurnierplatten, Gütebedingungen; Januar 1968.
Requirements for the construction, glue, glue bond and the bending strength

DIN 53255: shear test for glue bond

DIN 52371: bending strength test

Netherlands	Relevant standards:	
	NEN 3278	Plywood; classification, dimensions and tests
	NEN 3519	Plywood; determination of mechanical properties
	Other information:	
	TGH '71	Tabellen en grafieken voor houtconstructies; Houtvoorlichtingsinstituut, Postbus 4225, Amsterdam.

Sweden

No standards of the Swedish Standards Associations SIS.

Relevant information is given in:

. Recommendations for specification of strength and stiffness values for wood based panel boards. NKB - public No 1974; translation into English see Report from IUFRO Section V meeting 1973.

. Recommendations for strength and rigidity of flooring and roof sheating. NKB Public No 1974; transl. CIB/W 18 march 1973.

. Norén B: Swedish pine plywood. Strength and working stresses; Byggforskningen Handlingar nr. 45, 1964.

United States of America

. ASTM D 805-72 Testing veneer, plywood and other glued veneer constructions

. ASTM D 906-64 (1970) Test for strength properties of adhesives in plywood type construction in shear by tension loading.

. ASTM D 1038-52 (1970) Definition of terms relating to veneer and plywood.

. ASTM D 2718-71 Testing plywood in rolling (shear in plane of plies)

. ASTM D 2719-71 Testing plywood in shear through-the-thickness

. ASTM D 3043-72 Testing plywood in flexure

. ASTM D 3044-72 Test for shear modulus of plywood.

III General information

Australia

Canada

England

. specimens shall in no case be taken from less than 50 mm from the edges of the board
. test specimens shall be selected from as wide a range of the material as possible; from a number of boards and from random positions within these boards.
. to determine the average strength values for a particular species or type of construction, not less than 30 acceptable tests to measure each property shall be made.
. prior to machining and testing all specimens shall be conditioned to constant mass *) and moisture content in a room with RV 65 ± 2% and T = 20 ± 3 °C.

Germany

Netherlands

NEN 3519 states that for different uses a method is given, e.g. for the determination , the verification and the control of mechanical properties, as well as the calculation of allowable stresses. A terminology has been

*) constant mass is considered to be reached when two successive weighing operations carried out at an interval of 24 hours, do not differ by more than 0,1% of the mass of the test piece.

given for different amounts of plywood, e.g. a description of a plywood "type", a "make" of plywood, a "category" etc. is introduced. Furthermore a list of symbols is included. With respect to sampling general guiding principles have been given, belonging to the several categories of amounts of plywood, while a cutting plan gives information about how to cut the necessary test specimens from a panel. Panels and /or test specimens shall be conditioned until constant weight in a room with T = (20 $\pm$ 2)$^{\circ}$C and Rel. Hum = (65 $\pm$ 2)%. Along with the determination of the mechanical properties some non-mechanical properties must be tested, e.g. with respect to dimensions, quality of the glue, etc. More details are given in the Appendix.

Sweden

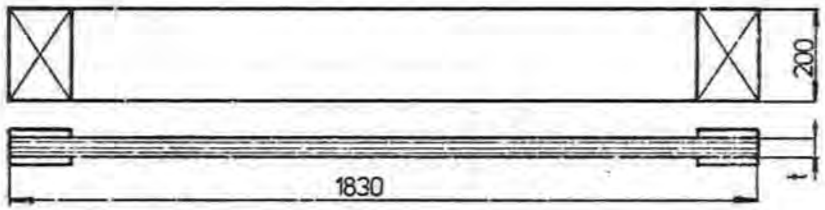
The two recommandations mentioned in I do not give information about sampling or testing methods. SIFI (Svenska Träforsknings Inst.) gives rules for external control of plywood in structural elements. The control consists of prototype control and continuous quality control on strength,
values and the variation therein as well as with respect to other properties. The control covers plywood, other laminated panels of related type, including laminated wood. Testing is done at a moisture content of about 12%, at 22% or in wet state.
The test specimens for strength values shall be taken from the weakest parts of the panels. From the test results characteristic strength values are calculated. These values have to be determined on the basis of prototype testing; the continuous control may then be limited to thickness, panel construction, glue line quality and strength in bending and tension.

United States of America

ASTM D 805-72 gives methods of testing veneer plywood and other glued veneer constructions and is the most general standard in the ASTM-series mentioned in chapter II. With respect to the selection of test specimens no general rules are given, but it is recommended that a sufficient number of tests should be made to permit statistical treatment of the data. The distance of the specimens to the edge of a panel should be at least twice the panels thickness. Specimens are conditioned to constant weight at a Rel. Humidity of $(65 \pm 1)\%$ and temperature of $(20 \pm 3)^{o}C$. Some general remarks about the content of test reports are made.

IV Tension tests

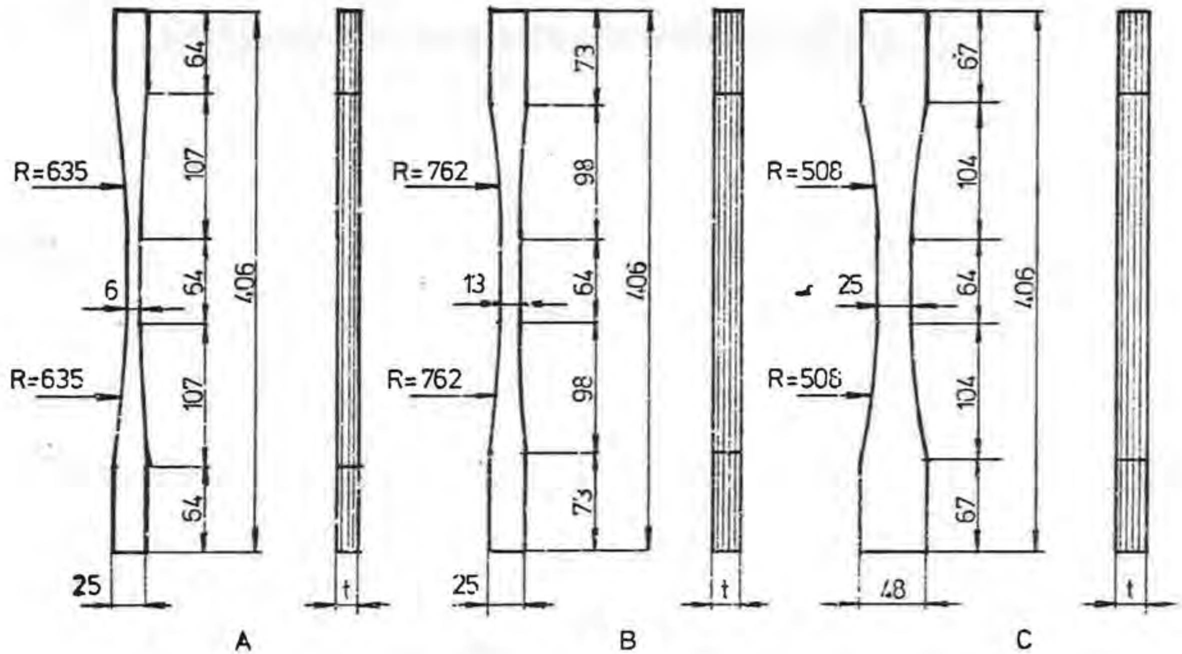
Australia



Canada

ASTM procedure
Also frequently used:
200 x 1200 mm
or 250 x 1200 mm

England

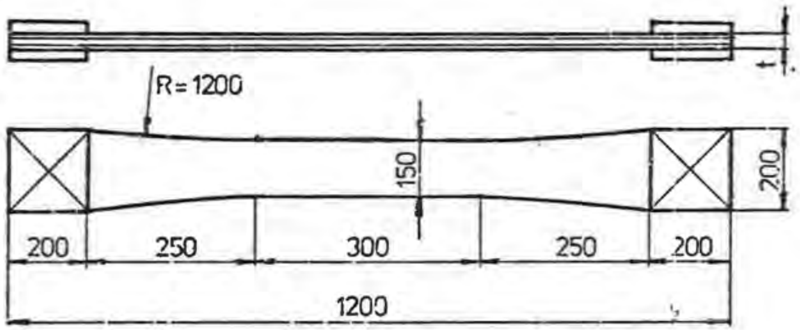


Test specimen A for material over 6 mm thick, type B for material 6 mm or less in thickness. Test specimen C shall be used if the grain of the individual plies makes angles other than 0 or 90° with the length of the specimen. In that case type C shall be used regardless of the thickness. Rate of deformation: movable head of testing machine 0.015 mm/sec.;specimen held in wedge type grips.

Load extension curves eventually based upon 12 or better 15 and more readings; a modulus of elasticity is determined from the straigth line portion of the curve.

Germany

Netherlands



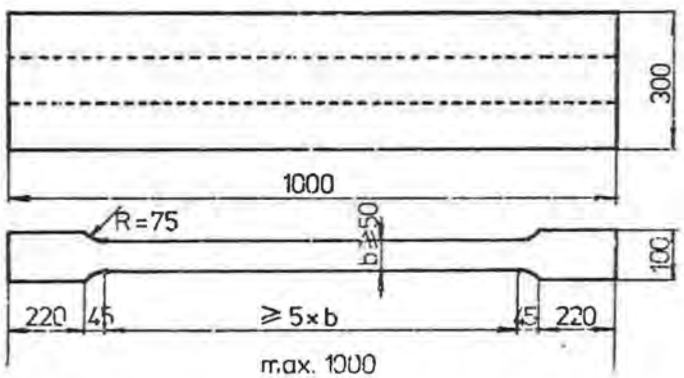
2 specimens // grain top veneers
2 specimens grain top veneers
see appendi

$$E_t = \frac{F'l}{bt\Delta l} \; ; \; F' = \text{ca } 0{,}4 \; F_{max}$$

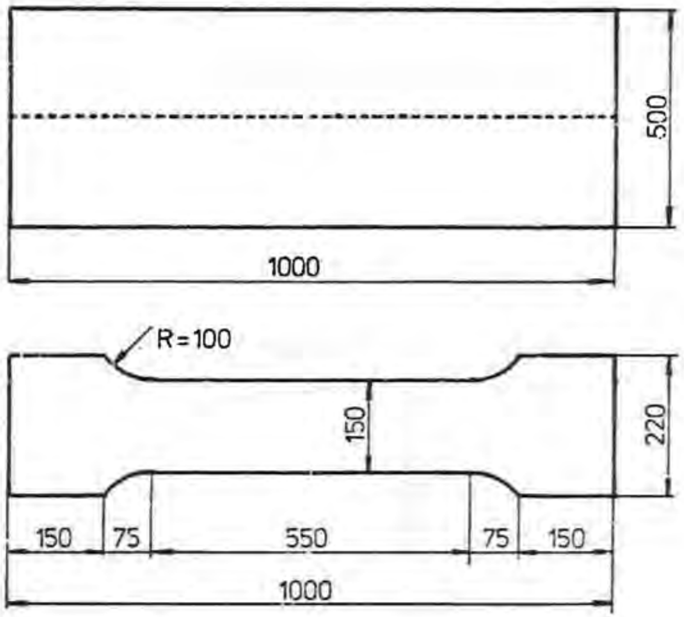
Δl = deformation over l

l = gauge length

Sweden



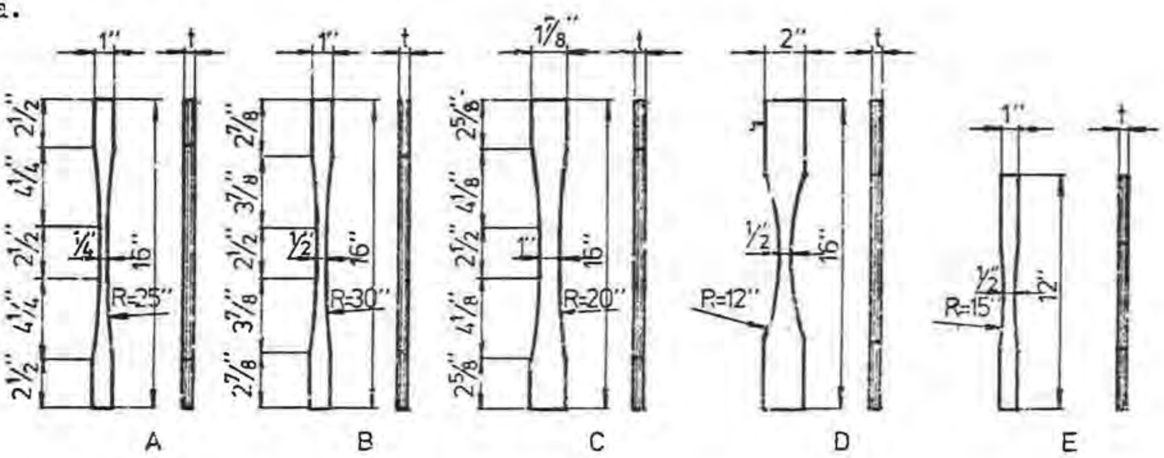
An orginal sample 300 x 1000 mm selected (as far as the tension // grain direction) with knot rows in the zone which shall be tested to rupture is split up into 3 specimens 100 x 1000 mm.



Test specimen, stipulated by Statens Planverk for approval testing; also accepted by NKB.
Test specimen used at STFI (Swedish Forest Products Res. Lab).

Small specimens according to ASTM, BS or DIN will serve the purpose for homogenous plywood types such as birch or beech plywood.

United States of America.



Dimensions of test specimens A,B and C are the same as in England.
Types D and E may be used if only the tensile strength (and not E) is required.
Type D: plywood thickness $> \frac{1}{4}$";
Type E: plywood thickness $\leq \frac{1}{4}$"
Rate of deformation: 0,89 mm/min

V Compression tests

Australia

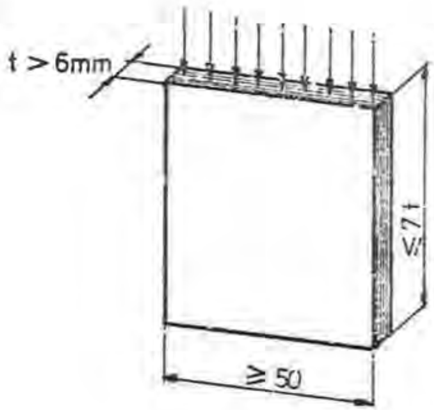
Compression tests not carried out; little need for them is seen.

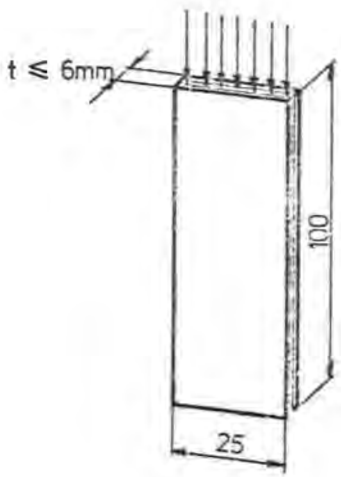
Canada

ASTM-procedure

Council of Forest Ind. often uses a frame that provides restraint against rotation along all four edges of a rectangular specimen of 150 x 450 mm.

England

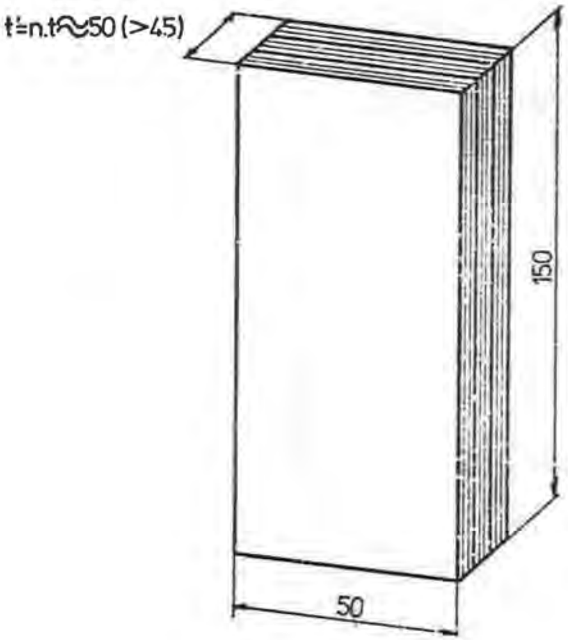




Test specimens 6 mm or less in thickness shall be supported laterally to prevent buckling. A description of the apparatus is included in the standard. Rate of deformation: 5.10^{-5} mm per mm of length of the specimen per second with a variation of ± 25%. Load-deformation curves may be made; a modulus of elasticity is determined from the straight line portion of the curve.

Germany

Netherlands



. A test specimen is made by gluing n pieces of plywood together until a total thickness of about 50 mm is reached.

. Per panel 2 specimens are tested parallel to the grain of the top veneers and 2 in a direction perpendicular to that.

. The compressive strength is calculated over the total surface of the specimen. If more specimens are tested the standard deviation of the compressive strength shall be calculated as $s = s'\sqrt{n}$, where s' is the st. deviation calculated directly from the test results.

Sweden

U.S.A. as given by the U.K; the lateral support may be omitted if only maximum compressive strength is to be evaluated.

VI Bending tests

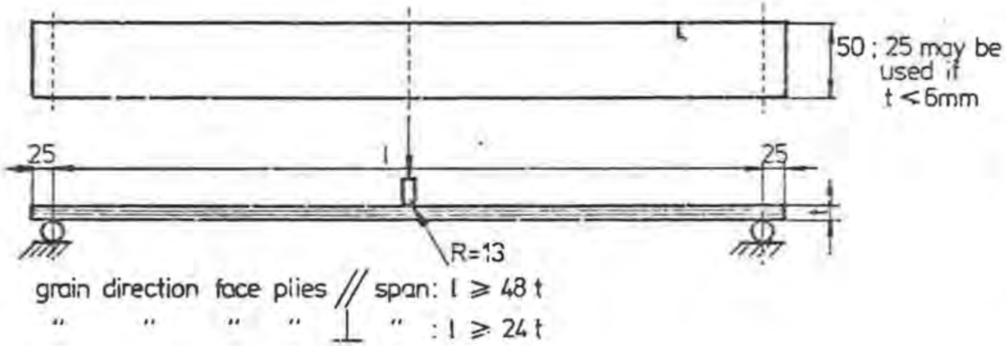
Australia

ASTM is followed, however test specimen is 9" wide.

Canada

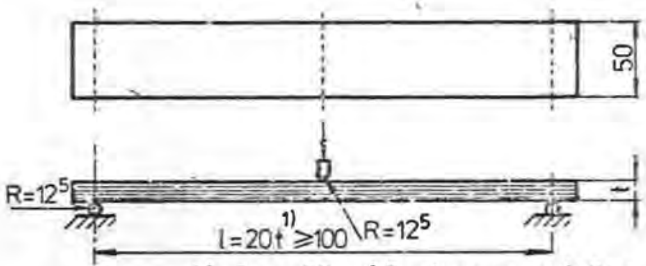
ASTM is followed

England



Rate of deformation chosen so as to give rate of strain in outer fibres of 5.10^{-5} mm/mm per second;
Modulus of elasticity calculated from deflection measurements using the straight line portion of the load-deflection curve.

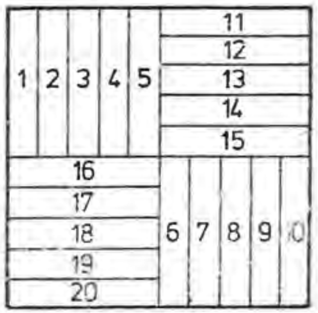
Germany



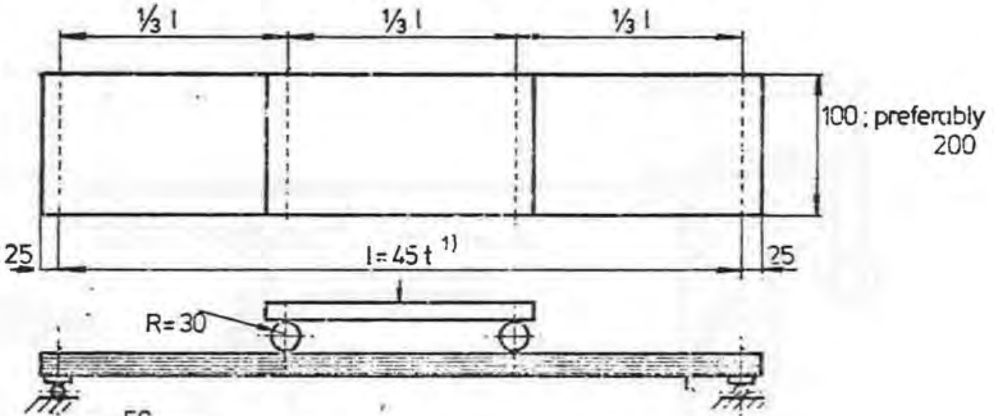
1) l = 30t if also a modulus of elasticity must be determined.

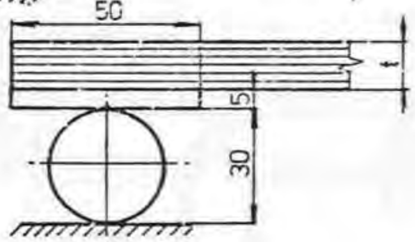
Conditioning: Rel. Hum. (65 ± 3) %
temp. (20 ± 2) °C.

Deflection is measured 6 times until a load of 1/3 F_{max} is reached.
F_{max} must be reached in (1,5 ± 0,5) min.
Cutting scheme:



Netherlands

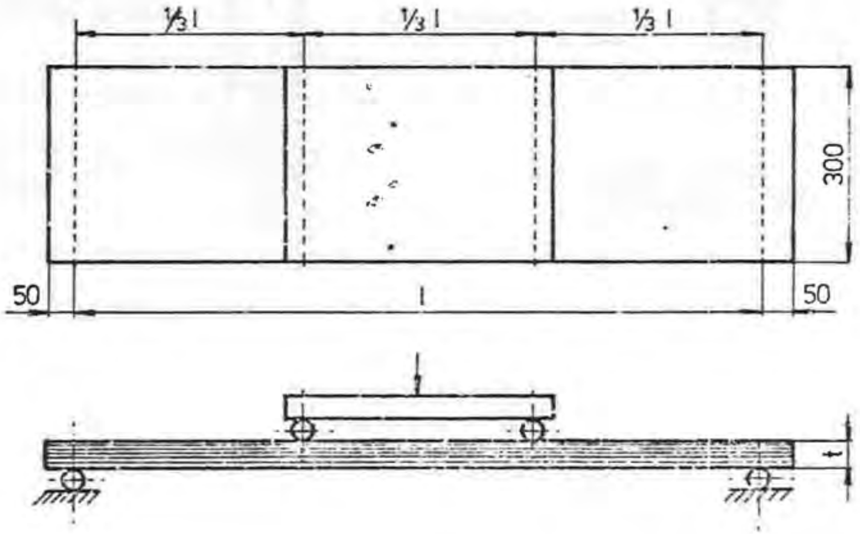




1) l = 45 t' if faceplies are ⊥ span;
t' = plywoodthickness − thickness faceplies.

Modulus of elasticity E_b is determined from the deflection at a load of about 40% of the maximium load F.
Rate of loading: max. load must be reached in 2 to 5 minutes.

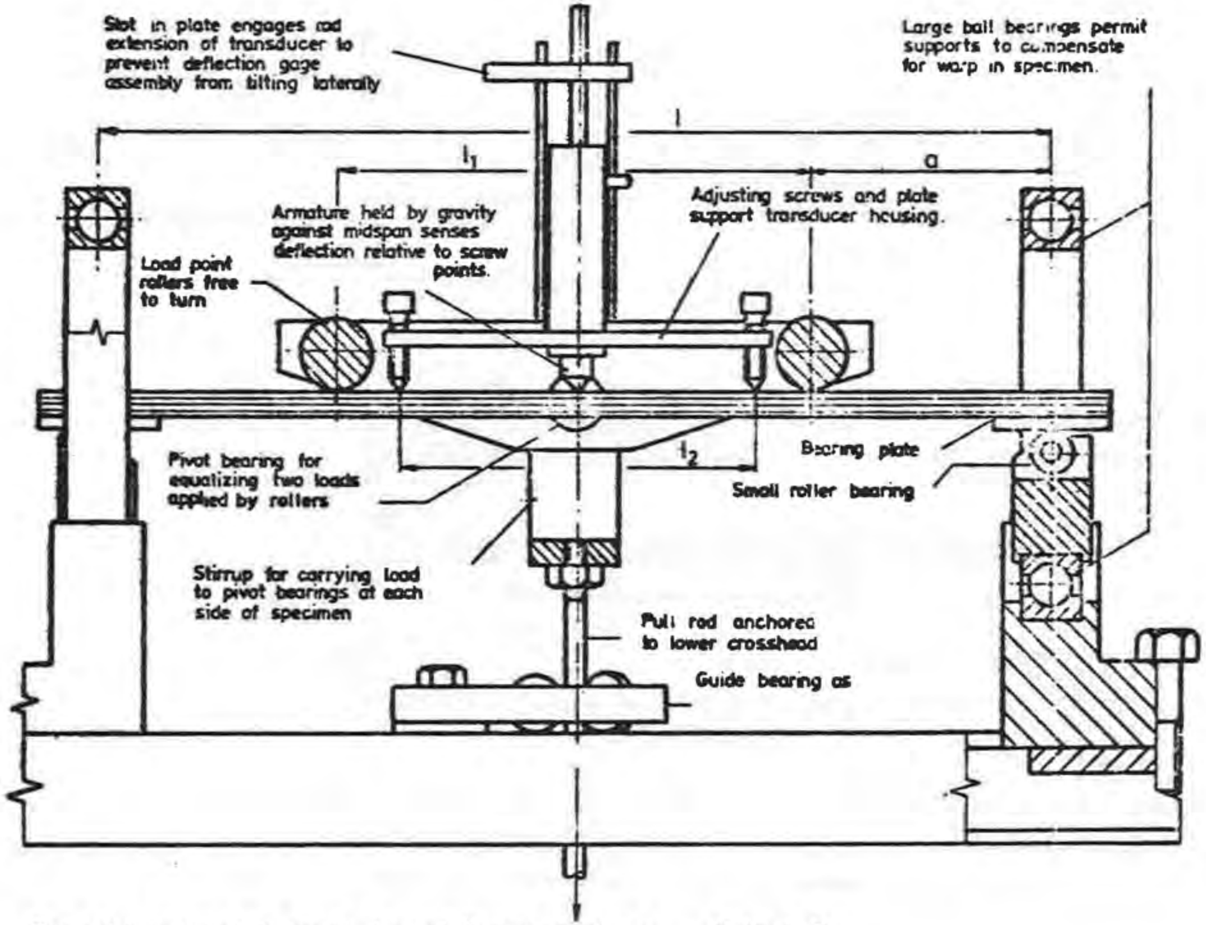
Sweden



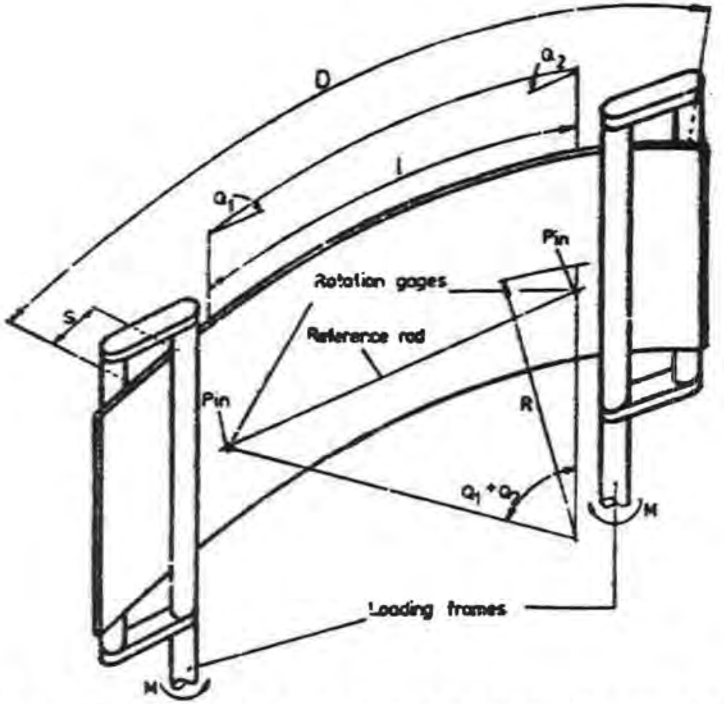
number of plies with grain direction // bending direction	span l
1	300
2	450
≥ 3	660

t	deflection speed mm/min I	II
7-10	13	17
10-14	18	13
14-18	14	18
18-22	11	14
>22	9	11

USA-methods B and C.



Two point load test for small plywood flexure specimens. Method B



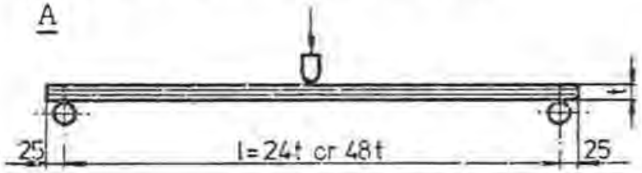
Pure bending test of plywood panel showing angular rotation gages and loading frames.

Method C

Weakest section, eg. with knot row, placed in region with maximum moment.

U.S.A. Three methods are described:

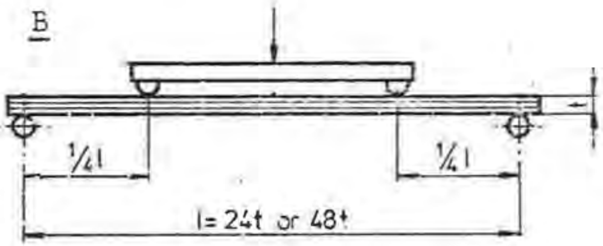
A



l = 48 t if face plies // span
l = 24 t if face plies ⊥ span
width of specimen 25 mm if t < 6 mm
and 50 mm if t ≥ 6 mm

. Strain rate in outer fibers 0.10^{-6} mm/min per second
. EI is calculated from the deflections
. center point flexure test for small, simply supported specimens

B



same test specimen as in A but using two points loading and a more elaborous equipment with special reaction bearings.

. Method B may be used to evaluate certain features such as core gaps and veneer joints where effects are readily projected to full panels.
. two point flexure test for small specimens.

C Specimens shall be of a size comparable to that of the material in use, frequently consisting of the

. Method C is ideally suited for evaluating effects of knots, areas of sloping entire panel. Width 610 mm and in no case less than 300 mm.

grain, etc.

. Loading frames apply pure bending moments to the panel; they are therefore free to move toward or away from each other.

. detailed information about test procedure, measurements, calculations have been given.

VII Panel shear.

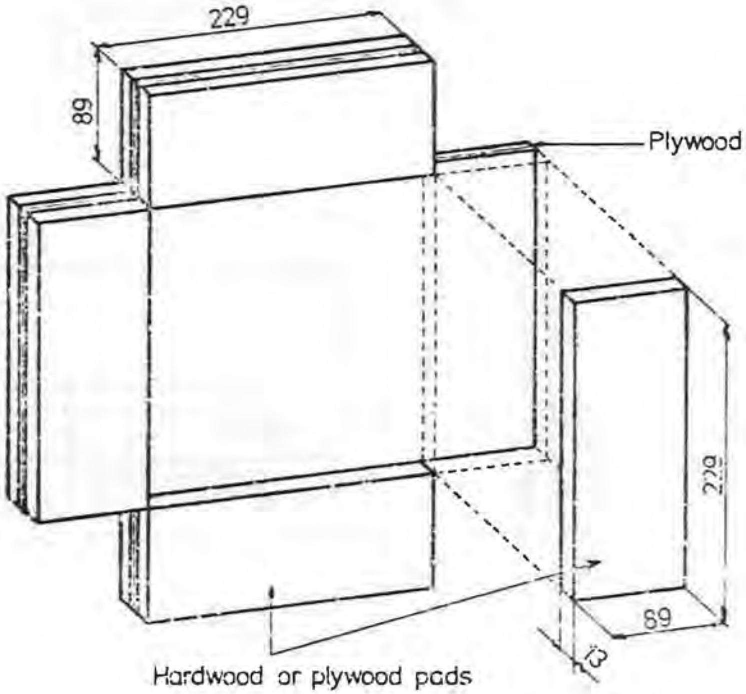
Australia

A 9" square panel is loaded in compression along a diagonal. Instead of rigid steel bars along the edges (as in the orginal de Havilland test) the specimen is loaded through well-fitting steel pins with shaped brass packing pieces between them. This gives the specimen greater scope to move the way it wants. Incidentally the edges of the specimen are reinforced by a 1"; wide veneer glued to the panel.

Canada

ASTM is followed

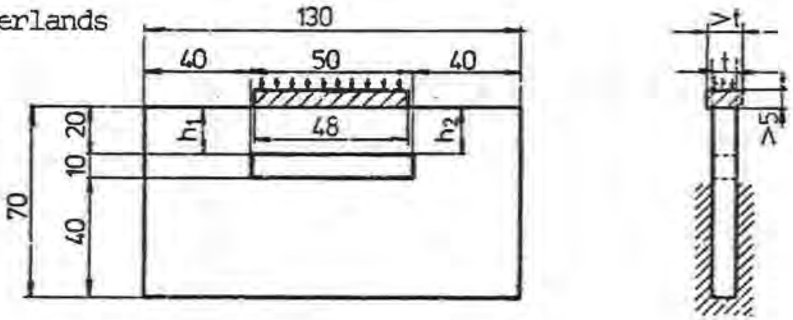
England



Load application, by means of steel plates fastened along the reinforcing pads, along the diagonal of the test specimen.
Loading in compression.

Germany

Netherlands



- Per panel two specimens with face plies // and two with face plies ⊥ direction of loading.
- Rate of loading: max. load must be reached in 2 to 5 minutes.

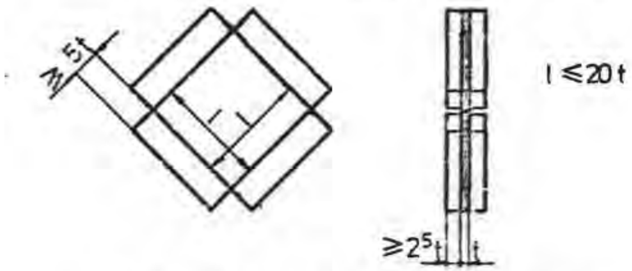
Test specimen also used in Norwegian research.

Sweden

U.S.A.

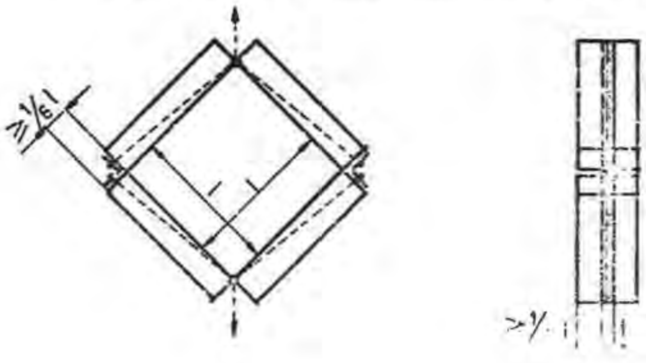
Three test methods:

A Panel shear test for small test specimens.



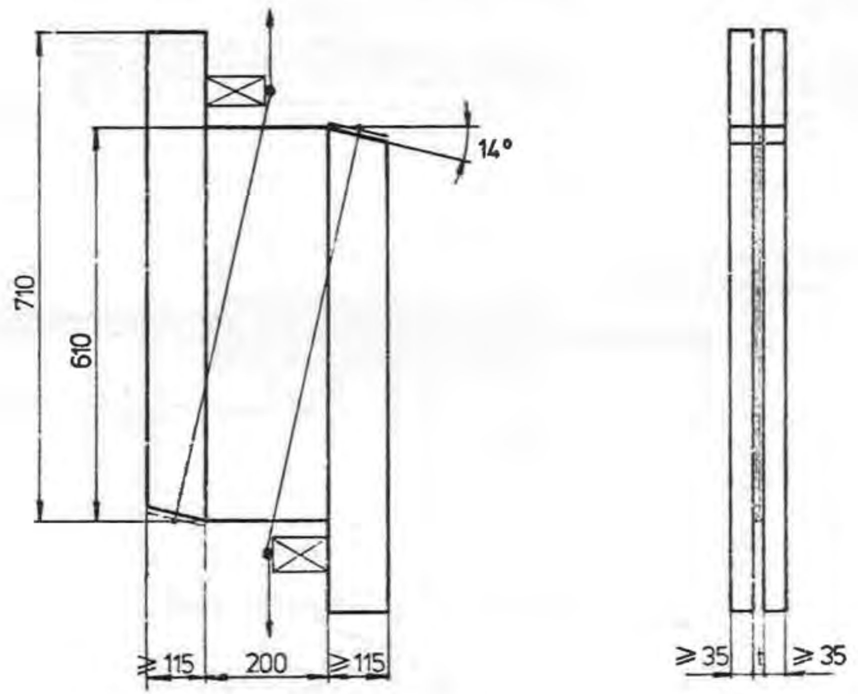
- Load is applied by a roller bracket assembly to the glued-on blocks. Loading diagonally in compression.
- Rate of deformation in vertical direction 4.10^{-5} mm/min/sec.
- Modulus of rigidity G is calculated from deformation in vertical direction.

B Panel shear test for large specimens.



- Load is applied by a system of pins and yokes; loading diagonally in tension.

C Two rail method for larger specimens.

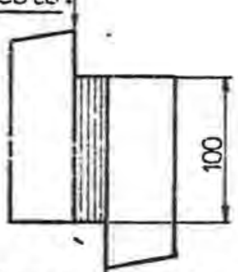


Load is applied by a system of pins and yokes in tension. Center part of specimen is subjected to a nearly constant shear stress. Crosshead motion of 0,1 ± 0,025 cm/min produces shear strain rate of 4.10^{-5} mm/min per sec.

Shear strength and modulus of rigidity G can be calculated.

VIII Rolling shear tests.

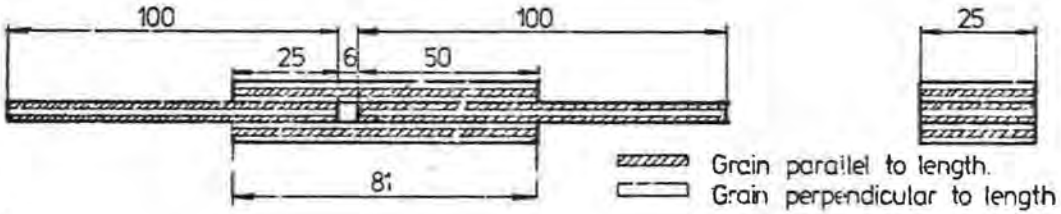
Australia



test specimen 50 x 100 mm.

Canada

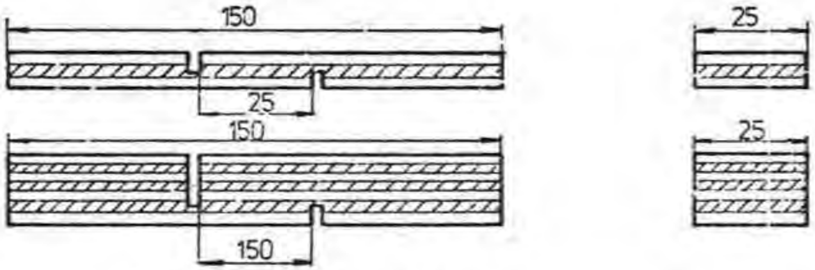
England



Rate of loading: 0,01 mm/sec of the movable head of the testing machine.

Germany

Netherlands



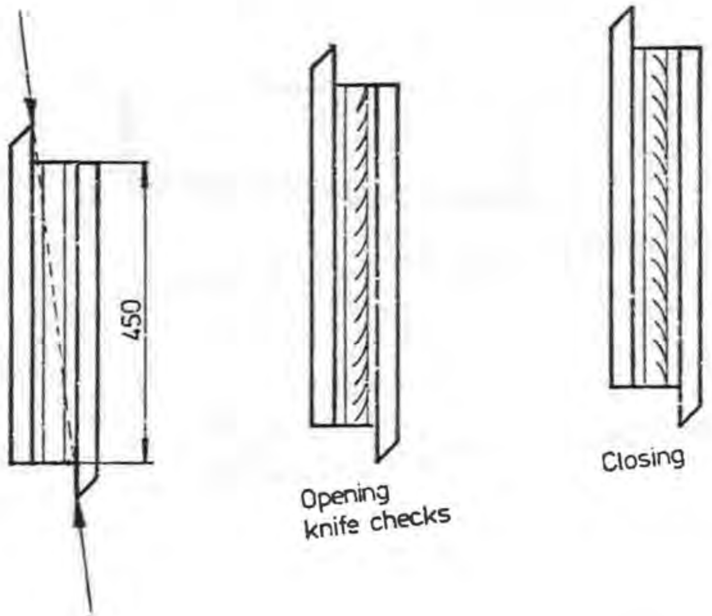
width of specimen 25 mm;

5 specimens are made from square pieces of 150 x 150 mm.

Failure must be reached within 1 to 2 min.

Sweden

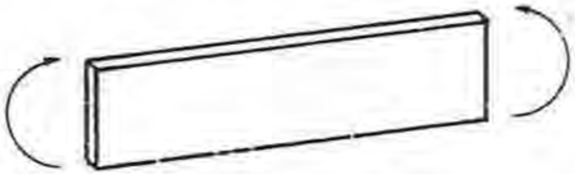
U.S.A.



- Specimen is bonded to steel plates.
- Dimensions 150 x 450 mm or larger. For uniform material smaller specimens of 4t x 12t are allowed (t = plywood thickness)
- Both test with opening the knife checks and closing these checks must be done.
- Rate of deformation calculated from a formula and dependent on the thickness of the plies in both directions as well as of the shear modulus.
- test demands cautious operating.

IX Other mechanical properties

Australia	Plate shear : according to ASTM, but 9" square	measured value of G seems to be sensitive to specimen size.
Canada		
England	Plate shear : according to ASTM	
Germany		
Netherlands	a) compressive strength ⊥ plies according to ASTM D 143	For calculations the allowable stress of the weakest kind of wood in the plywood can be used.
	b) bending strength and stiffness in plane of panel	

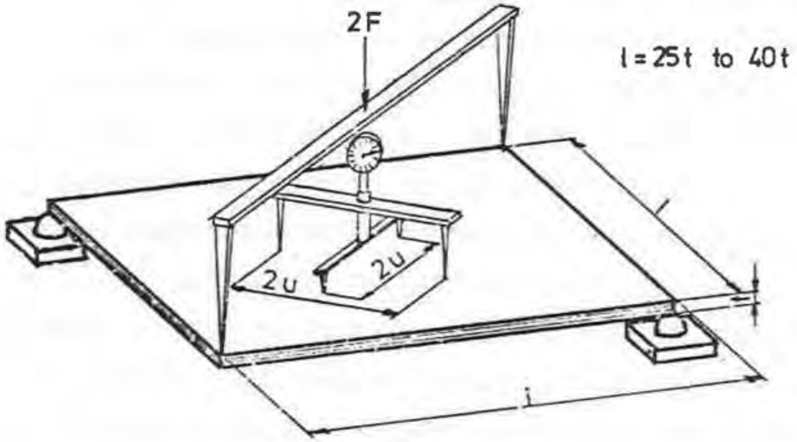


No test method; bending stress in tension may be taken aequivalent to tensile strength, bending stress in compression equal to compressive strength.

c) plate shear : according to ASTM

Sweden

U.S.A. Plate shear;



dial readings give twice the average deflections relative to the panel centre

.Conditioning:

Rel Hum $(65 \pm 1)\%$

$(20 \pm 3)^{\circ}C$.

Deflection rate of movable cross head 0,012 L per minute

.Modulus of elasticity G is calculated as:

$$G = \frac{3\ u^2 F}{2\ wt^3}$$

2F = total load

t = thickness

w = deflection relative to center of panel

2u = distance between supports.

X Conclusions and remarks.

The information gathered in this report should help to bring into existance an international accepted set of recommendations for the mechanical testing of plywood, with the aim to reach a position where the figures about strength properties in different countries and from different materials can be compared. To reach such a state it seems necessary to make first clear that there are at least two different subjects we have to deal with:

1) quality control and

2) trustworthy property-values to rely upon when permissible stresses etc. are to be determined.

With respect to quality control we seem to have to accept that the better plywood qualities are used for nonload bearing purposes. The use of lower grade veneers in cheap enough plywood to fullfill the more risk-bearing functions in loadbearing structures demand for better quality control. At the moment, however, changes in kind of wood species and quality are made without informing the structural users. In my opinion the structural engineer shouldbe supplied with a limited number of plywood types for the special use in load bearing structures each of them being of standardisedconstruction, sothat the engineer can trust this material. The strength values must not necessarily be the highest possible; it should be very welcome if the variation in strength figures were not too high; may be some minimum values could be made obligatory.
For the quality control of such material a relative simple set of tests could be set up.

With respect to the determination of strength figures it seems to be possible to set up a recommendation. Some ideas there about are mentioned in the following.

a) Conditioning.

Most standards describe conditioning of the test specimens. It could be agreed upon one or two conditions:

1) structures in normal dry conditions : conditioning at Rel Hum $(65 \pm 3)\%$
Temp $(20 \pm 3)^{\circ}C$

2) structures in condition with high humidity : conditioning at Rel Hum $(85 \pm 3)\%$
Temp $(20 \pm 3)^{\circ}C$

In following this recommendation it should be understood that it is desirable to maintain the Rel Hum and Temp. as nearly constant as possible at a certain value within the ranges.
End of the conditioning is supposed to be reached if the weight mass of the test specimen in 24 hours has not more changed than 0,1 %.

b) Rate of testing.

The rate of testing is given in different manners:rate of loading, rate of deformation and total time until failure. Perhaps it could be agreed upon the desirability of a constant rate of deformation such as to reach failure within a time of 2 to 5 min.

c) Tension tests.

For the determination of strength values as a basis for permissible stresses it seems that in most countries large test specimens are in use. Dimensions of about 200 x 1200 mm are frequently used with a decreased width of 150 mm.
For homogenous material (but how to decide there upon?) smaller specimens can be used, for instance specimen C of England and ASTM.

d) Compression tests.

A test specimen like the Dutch one seems to give good results and is relatively simple.

e) Bending tests

The dimensions of test specimens vary somewhat more than those for the tension test. It seems however possible to choose a width of 200 or 250 mm and to take a span of 45t or 48t[1], together with a two points loading. Here also a smaller test specimen could be taken for homogenous material, with the same difficulty as with the tension test.

Such a small specimen perhaps also could be used for quality control purposes.

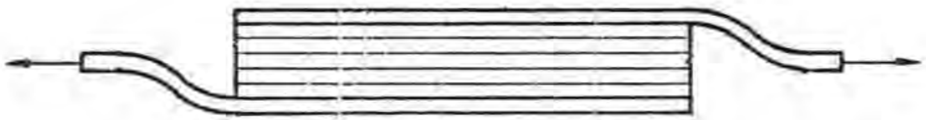
Of course also tests on whole panels can be done; I wonder if this must be standardised.

f) Panel shear.

A choice must be made between the Dutch (small) specimen, the square specimens and the two-rail method. For the determination of strength values one of the larger test specimens seems to be preferable; on the other hand if it is desirable that the different test specimens can be taken from one panel the greater dimensions will cause difficulties.

g) Rolling shear

Test specimens of Australia and ASTM are essentially the same; a slight transformation could change the compressive test to a tensile test with less danger for the operator. Dimensions of the test specimen could be chosen 50 x 100 mm to or more.



1) If faceplies are $\perp$ span direction t' = (total thickness - thickness face veneers) could be used; may be also a minimum span must be prescribed.

h) <u>Plate shear for determination of G</u>

ASTM method is followed in most countries and seems acceptable.

i) <u>Bending strength and stiffness in plane of panel</u>

Only the Dutch code gives some information, no test method.

If the work in this direction should be pursued, appointments must be made about the procedure to be followed with respect to CEN, ISO, RILEM, etc. It must also be decided if in these recommendations long duration tests should be incorporated.

At last it should be kept in mind that there are developments in the structural use of other board materials.

Delft, March 1974

J. Kuipers.

APPENDIX

NEN 3519 provides a standardized method to:

- determine the mechanical properties of a type of plywood or category of plywood by type or category testing;
- verify published mechanical properties of a plywood type or category by a verifying inspection;
- control by means of a verifying inspection, the indicated mechanical properties of a lot of plywood delivered or intended for delivery;
- derive, from the results of verification, allowable stresses and moduli of elasticity to be used in design calculations.

For a definition of the various concepts and symbols see chapter 1.3.

1.3 Terminology and symbols

plywood type : Plywood of a clearly described composition, notably with regard to thickness, number and species of veneer and durability class of the glue line.

make of plywood: A type of plywood which is manufactured by one manufacturer or by more than one manufacturer under common control.

plywood category: A group of several types of plywood and/or makes which, for instance, on the basis of their similar mechanical properties, are treated as one type.

trade mark: The name of the factory or the brand name under which plywood is put on the market. (Remark: Various types may be put on the market under a certain trade mark or brand name.)

lot : This may mean - dependent on the purpose of the testing:

- the total output or import of one type or category.
- a part (for instance, intended for delivery or already delivered) of the output or the import of one type or group.

investigation as to type: The investigation as to unknown properties of the total output or import of a plywood type.

investigiation as to category: The investigation as to unknown properties of a plywood category.

verifying investigation: The verification of the total output or import of a plywood type or category with the intention of verifying the accuracy of quoted properties.

verification check: The checking of a part (for instance, intended for delivery or already delivered) of the output of one plywood type or category with the intention of verifying the quoted properties.

2. Testing

2.1 General

2.1.1 Sampling

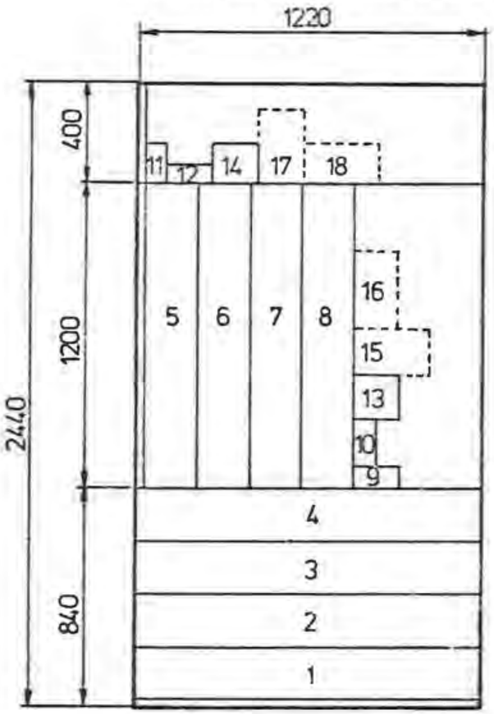
The testing must be carried out on a representative number of panels from the lot to be investigated. The number of panels to be sampled and their dispersion in the lot depends on the purpose of the testing and cannot be indicated exactly.

A guiding principle should be:

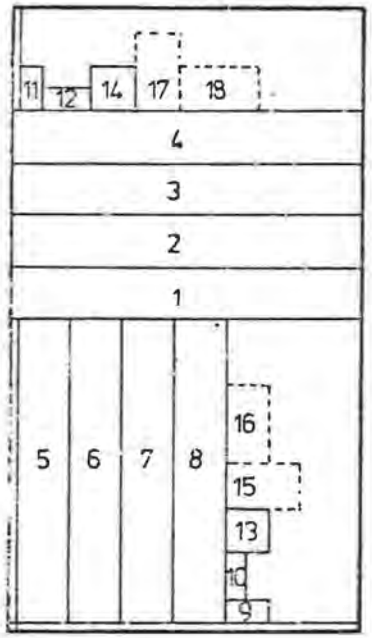
- for category or type testing at least ten panels per type and per make must be sampled. The dates of manufacture of the panels must be spread out over as long a period as feasible;
- for a verifying investigation at least five panels per type and per make have to be sampled. The dates of manufacture of the panels must be spread out over as long a period as feasible;
- for a verification check at least three panels have to be sampled if the lot consists of 100 panels or less. At least five panels must be taken if the lot is larger. The panels must be taken from quite separate parts of the lot.

2.1.3 Cutting Plans

Cutting plans for panels with nominal dimensions of 244 x 122 cm appear in Appendix 1. One half of the number of panels is to be cut in accordance with plan A, the other half in accordance with Plan B. Similar plans must be used for panels of other dimensions. The purposes of the different test pieces are indicated in Appendix 1. The test specimens used to determine the durability of the glue line are to be taken from the remaining pieces of panel. In case the requirements for sampling stated in NEN 3278 are in conflict, deviation from NEN 3278 is allowed.



Cutting scheme A



Cutting scheme B

Bending test	: 1,3,5,7.
Tension test	: 2,4,6,8.
Compression test	: 15,16,17,18.
Panel shear test	: 9,10,11,12.
Rolling shear test	: 13,14.

PAPER 4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

BENDING STRENGTH AND STIFFNESS OF MULTIPLE SPECIES PLYWOOD

by

CKA STIEDA

WESTERN FOREST PRODUCTS LABORATORY

VANCOUVER, CANADA

DELFT - JUNE 1974

BENDING STRENGTH AND STIFFNESS OF MULTIPLE SPECIES PLYWOOD

C.K.A. Stieda

1. Introduction

Assessment of the safe load carrying capacity of plywood requires an understanding of its ultimate load capacity. Plywood strength properties in common with those of other wood based products exhibit variabilities sufficiently large to have to be considered in the assignment of design strength properties. For the purpose of this paper it is immaterial by what method these design properties will be derived. Such methods could include derivation of design stresses using some judiciously chosen constant factor of safety or a more elaborate limit state design procedure. The method might include provision for effects due to changing moisture contents or load durations. In every case the design properties will have to be derived in some predetermined way from the appropriate strength distributions.

In the case of plywood manufactured from only one species it is feasible to obtain estimates of the strength distributions by testing a sufficiently large number of representative specimens. To assure reasonably reliable estimates, even for a single species, it is often necessary to test a large number of specimens, perhaps 50 or more for a single property. If two or more species are to be mixed, involving potentially several grades of veneer for each species, physical testing of all possible veneer combinations becomes excessively time consuming and expensive. It is therefore desirable to have available an analytical method of estimating the expected strength of plywood combining veneers of different species.

This paper proposes one possible method of deriving the properties of such multiple species plywood. The method follows well recognized engineering principles* and is based on a consideration of the effect of the elastic properties of different veneer species on the properties of the plywood manufactured from these veneers. To illustrate the principle of the method flexural properties are considered here. However the method is equally applicable for the derivation of other strength properties, and the required equations can readily be derived. Following the presentation of the necessary equations a number of numerical coefficients will be presented and the calculation of basic veneer properties will be outlined.

2. Basic Assumptions

For the purpose of this paper the following assumptions will be made:

1. Veneer properties are orthotropic and homogeneous throughout the thickness of the veneer.
2. Restraining effects due to cross-banding of veneers are negligible, i.e., the effect of Poisson's ratio will not be considered.
3. Originally plane sections remain plane during bending.
4. All veneer deformations are linearly elastic.
5. Thicknesses of individual veneers and veneer species are symmetrical with respect to the centre veneer.

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*Hoff, N.J. The strength of laminates and sandwich structural elements. In "Engineering Laminated" ed. by A.G.H. Dietz. Wiley. 1949.

6. In calculating bending properties for the case where the outer plies are stressed in a direction perpendicular to their grain orientation the outer veneer on the tension side will be neglected.

Both the "full cross-sectional" approach to design and the "parallel-plies only" approach will be considered.

3. Full Cross-Sectional Design Approach

3.1 Stiffness Relations

The "full cross-sectional" design approach assumes that the plywood is homogeneous throughout its thickness t and that it has an apparent modulus of elasticity E'. The effective bending stiffness K of this plywood can be calculated from the dimensions t_i (Fig. 1) and the elastic moduli E_i of the individual veneers and their location.

$$K = E'I = \sum_i (E_i I_i) \tag{1}$$

where for a strip of unit width

$$I = t^3/12$$

$$I_i = t_i^3/12 + t_i(z - t_i/2)^2$$

z = distance from centroid to outer face of veneer (Fig. 1)

It is evident that the apparent modulus of elasticity E' will be a function of the number of veneers and their individual moduli of elasticity. The bending stiffness K can be redefined in terms of a reference modulus of elasticity, E, and a coefficient k_1

$$K = k_1 EI \tag{2}$$

The coefficient k_1 expresses the influence of the stiffness and location of individual veneers. If the outer veneers are stressed parallel to their grain direction this coefficient is given by

$$k_1 = 12.0\,[2.0(I_a/t^3)(E_a/E) + \underline{2.0(I_b/t^3)(E_b/E)} +$$

$$\underline{2.0(I_d/t^3)(E_d/E)} + (I_c/t^3)(E_c/E)] \tag{3}$$

Where I_a, I_b etc. are the second moments of area of the individual veneers with respect to the centroid of the total thickness, i.e. the mid-height of the section (Fig. 1). E_a, E_b etc. are the corresponding moduli of elasticity of individual veneers. For plywood of 3-ply construction only the terms involving I_a and I_c are to be used for the calculation of k_1.

For 5- and 7-ply the first underlined term and for 7-ply also the second underlined term in Eq. 3 is to be added to calculate k_1.

If the outer veneer is stressed perpendicular to the grain it cannot be assumed any more that the centroid of the effective cross section is located at mid-height. Because of the low tensile strength of veneer in the tangential direction of the wood it will be assumed that the outer veneer on the tension side is not effective. The location (z_c) of the centroid of the remaining section is then determined from the requirement of static equilibrium of all forces in the plane of the plywood (Fig. 2). This will result in the following expression for z_c.

$$z_c/t = \frac{(t_a/t)^2(E_a/E) + (t_c/t)(E_c/E) + 2\ R}{2.0\ \{(t_a/t)(E_a/E) + (t_c/t)(E_c/E) + 2\ R\}} \quad (4)$$

The term R is zero for plywood of 3-ply construction. For 7-ply construction $R = (t_b/t)(E_b/E) + (t_d/t)(E_d/E)$, and for 5-ply construction the term in R containing t_d/t is deleted.

The stiffness coefficient k_1 can now be expressed in terms of the second moments of areas of individual veneers about the centroid given by z_c. The individual second moments of area for 3 to 7-ply construction are

$$\begin{aligned}
I_{ac} &= t_a^3/12 + t_a\ (z_c - t_a/2)^2 \\
I_{bc} &= t_b^3/12 + t_b\ (z_c - t_a - t_b/2)^2 \\
I_{dc} &= t_d^3/12 + t_d\ (z_c - t_a - t_b - t_d/2)^2 \qquad (5) \\
I_c &= t_c^3/12 + t_c\ (z_c - t_a - t_b - t_d - t_c/2)^2 \\
I_{dt} &= t_d^3/12 + t_d\ (z_c - t_a - t_b - 1.5\ t_d - t_c)^2 \\
I_{bt} &= t_b^3/12 + t_b\ (z_c - t_a - 1.5\ t_b - 2.0\ t_d - t_c)^2
\end{aligned}$$

The stiffness coefficient k_1 for bending perpendicular to the face grain can now be calculated from the previously stated condition that the effective stiffness E'I be equal to k_1EI.

$$k_1 = 12.0\ \{(E_a/E)(I_{ac}/t^3) + (E_c/E)(I_c/t)^3 + S\} \quad (6)$$

For 3-ply construction the term S is zero. For 5- and 7-ply construction $S = (E_b/E)(I_{bc}/t^3 + I_{bt}/t^3) + (E_d/E)(I_{dc}/t^3 + I_{dt}/t^3)$

with the second term containing E_d/E deleted for 5-ply construction.

3.2 Moment Capacity

The moment carrying capacity of a plywood section can be calculated from the normal stresses f distributed over the section A.

$$M = {}_{A}\int z dA \qquad (7)$$

where f = stress at a distance z from the neutral axis and A is the cross-sectional area.

It will be assumed that during bending originally plane sections will remain plane. It will also be assumed that stresses do not exceed the elastic limit of the wood. For each individual lamination the stress f_i will therefore be given by

$$f_i = \varepsilon_i E_i$$

The moment capacity M (Fig. 3) for a section of unit width then becomes

$$M = (\varepsilon^*/z^*)(E_a \, {}_{ta}\int z^2 dz + E_b \, {}_{tb}\int z^2 dz + \ldots) \qquad (8)$$

where ε^* is the normal tensile strain on the outside of some judiciously chosen reference ply at a distance z^* from the neutral plane. The integrals in Eq. 8, of course, are the second moments of area of the individual veneers. The moment for a 7-ply construction can therefore be written:

$$M = (\varepsilon^*/z^*)[E_a(I_{ac} + I_{at}) + E_b(I_{bc} + I_{bt}) + E_d(I_{dc} + I_{dt}) + E_c I_c] \qquad (9)$$

For 3- and 5-ply construction the redundant terms of $E_i I_i$ in Eq. 9 have to be deleted. Introducing a reference tensile stress $f^* = \varepsilon^* E$ and expressing the total stiffness in Eq. 9 as k_1 EI (Eq. 2) the moment will become

$$M = k_1 f^* I/z^* = k_3 f^* S$$

where $S = \frac{t^2}{6}$ is the section modulus of a rectangular section of thickness t and unit width, and

$$k_3 = k_1 t/(2z^*).$$

For plywood where the face veneers are stressed parallel to the grain the reference properties f^* and E should be those of the face veneers. The distance z^* for that orientation will be t/2 and k_3 will be simply equal to k_1.

For an orientation of plywood producing bending stresses perpendicular to the grain of the outer fibres z^* should be calculated by neglecting the veneer thickness t_a on the tension side. The critical stress f^* then equals the maximum stress in veneer t_b on the tension side while $z^* = t - z_c - t_a$.

If the moment capacity is calculated on the basis of the parallel to grain strength of veneer t_b it may also be necessary to check the compressive stress perpendicular to the grain in the outer ply on the compression side. From Fig. 2 it is readily seen that for perpendicular orientation of plywood this stress is given by

$$f_{ac} = f_{bt}\,(E_a/E_b)(t - t_a - z^*)/z^* \tag{12}$$

4. "Parallel Plies" Design Approach

4.1 Plywood Stiffness

An approximation of the stiffness and strength of plywood can be obtained by considering only those veneers that are stressed in a direction parallel to the grain. The degree of approximation obtained by this approach can be estimated by considering again the contribution of all plies. The total bending stiffness here is given by

$$K = \sum_i (E_i I_i) = k_2 E I' \tag{13}$$

where I' is the second moment of area of all plies oriented parallel to the bending stresses and k_2 is a stiffness coefficient corresponding to a reference modulus of elasticity E. For plies having their grain oriented parallel to that of the face plies the notation I_1 will be used for I', for plies oriented at 90 degrees to the face plies $I' = I_2$. Since the total stiffness $\Sigma E_i I_i$ is also given by Eq. 2 the stiffness coefficient k_2 can be calculated simply as

$$k_2 = k_1 I/I' \tag{14}$$

If the outer plies are stressed parallel to their fibres $I' = I_1$ can be readily obtained from the appropriate values of I_a, I_d or I_c for the particular plywood construction being considered. For perpendicular stressing of outer fibres $I' = I_2$ will be equal to the sum of the appropriate values of I_{bc}, I_c and I_{bt} (Eq. 5). It should be noted that for parallel stressing of outer plies the centroid of the section is located at mid-height (t/2), while for perpendicular stressing z_c is given by Eq. 4.

4.2 Moment Capacity

For the "parallel plies" approach to design Eq. 10 for the moment capacity will be replaced by

$$M = k_2 f^* I'/z^* = k_4 f^* S' \tag{15}$$

where S' is a nominal section modulus to be defined below.

For plywood stressed parallel to the outer fibres the reference stress f^* will be the maximum tensile stress in the outer veneer. The corresponding distance from the neutral axis to the outer fibres is $z^* = t/2$. If the corresponding section modulus S' is defined as $I'/(t/2)$ the coefficient k_4 will be equal to k_2 for orientation of outer veneers parallel to the bending stresses.

If the plywood is rotated by 90 degrees so that the outer plies are stressed in bending perpendicular to the grain, the reference tensile stress f^* should be that in the veneer immediately below the face with a corresponding value of $z^* = t - z_c - t_a$. The nominal section modulus S' can then be defined as $S'^c = I'/(t/2 - t_a)$, and the corresponding coefficient k_4 will be

$$k_4 = k_2(t/2 - t_a)/(t - z_c - t_a) \qquad (16)$$

With definitions of S' for the two orientations of plywood considered here the moment capacity of plywood can be calculated from the strength of the reference veneer, t_a or t_b, and the appropriate factor k_4 for the particular elastic properties and plywood construction.

5. Stiffness Coefficients k_1, k_2 and Stress Coefficients k_3, k_4

5.1 Full Cross Sectional Design Approach

Stiffness Coefficients

Stiffness and stress coefficients for a range of veneer thickness ratios and relative moduli of elasticity are given in Tables 1 to 6. Tables 1 to 4 are calculated for plywood manufactured from a single species. The reference modulus for both plywood orientations therefore is $E = E_L$ where E_L is the modulus of elasticity in the longitudinal direction of the veneer. The ratios E_a/E for loading parallel to the grain of the outer plies and E_b/E for perpendicular loading are therefore both unity. It will be recalled that the stiffness coefficient k_1 (Eq. 2) indicates the fraction of the bending stiffness that is available if the stiffness of plywood is calculated from the second moment of area of the full cross section and some reference modulus of elasticity E. The coefficient k_1 can also be viewed as a factor indicating how much the reference modulus of elasticity E has to be reduced to obtain an effective modulus of elasticity E' that can be used together with the second moment of area I for the full cross section (Eq. 1)

In Table 1 it has been assumed as indicated earlier that the reference modulus of elasticity is equal to the modulus of elasticity of the face plies for parallel-to-grain loading and to that of the adjacent plies for perpendicular-to-grain loading. It becomes immediately apparent that for three-ply construction nearly the total stiffness EI is available for parallel-to-grain loading. For 5-ply construction the lower stiffness of the cross-plies t_b becomes effective reducing the available stiffness to as low as 52 percent of EI for plywood with thin face veneers. For 7-ply construction the reductions are similar. In all cases the effect of changes in the magnitude of the modulus of elasticity of the cross-plies (t_b) is small compared to the effect of changes in relative thickness of veneers.

As would be expected the reductions in stiffness are much more drastic, if the plywood is stressed perpendicular to the grain of the face veneers. With thin outer veneers 5-ply construction shows a reduction in stiffness of about 50 percent. But if all veneers are of the same thickness the effective stiffness can be as low as 23 percent of EI. From Table 1 it is apparent that, while it is possible to design plywood lay-ups for 3- and 7-ply construction which will have about the same stiffness parallel and perpendicular to the outer grain orientation ($k_1 = 0.5$), this cannot be done for 3-ply construction.

Stress Coefficients

The stress coefficient k_3 (Eq. 10) indicates the reduction in the moment carrying capacity of a plywood section compared to a solid section when both are stressed to the same tensile stress f^*. As indicated earlier for moments producing tensile stresses parallel to the grain of the outer plies the stress coefficient k_3 is identical to k_1 and f^* is equal to the maximum tensile stress in the outer plies. If the applied moment stresses the outer plies in a tangential direction the critical tensile stress f^* will be that in the cross ply. This ply will be stressed in the longitudinal direction of its grain and the stress coefficient is given by k_3 in Table 3. Assuming linear stress-strain behaviour up to failure coefficients k_1 or k_3 therefore also allow calculation of the ultimate load carrying capacity of a plywood section when stressed to its ultimate capacity in tension in the longitudinal direction of the veneer. For the range of modular ratios E_s/E considered in Tables 1 and 3, k_1 and k_3 are changing little for a given plywood lay-up. However, changes in the veneer ratios have a more pronounced effect on these stress coefficients.

The lowest stress factor k_3 and therefore also the largest reduction in moment carrying capacity is found for 3-ply construction stressed perpendicular to the grain of the outer veneer. For 3-ply panels with approximately equal thickness for all 3 veneers, the moment carrying capacity can be as low as 12 to 20 percent of the capacity of a solid section stressed to the same ultimate stress. But even a 5-ply panel with relatively thick face veneer could be reduced in strength to 22 percent of that of a solid section. Plywood constructed of 5 plies of equal thickness should be able to carry about 80 percent of the ultimate moment of a solid section, when stressed parallel to the grain of the face plies and about 40 percent when stressed perpendicular to the face grain. It should be noted that the relative moment carrying capacity of plywood parallel and perpendicular to the grain of the outer plies ($M_1/M_2 = k_1/k_3$) is not constant but depends on the plywood construction.

5.2 Parallel-Ply Design Approach

Stiffness Coefficients

Since the modulus of elasticity in the tangential direction of wood is small compared to that in the longitudinal direction, it is often found convenient to ignore the contribution of plies stressed perpendicular to the grain in the calculation of the stiffness of plywood. The stiffness factor k_2 gives an indication of the increase in actual stiffness over that obtained by considering only plies oriented parallel to the applied bending stresses. The stiffness factors in Table 2 again assume that the reference modulus of elasticity E is equal to that of the face plies for a grain orientation in the face parallel to the bending stresses and that it is equal to that of the cross plies for a perpendicular orientation of face plies.

For parallel orientation of the grain in the face plies the increases in stiffness range from 0 to 1 percent for thick outer veneers in plywood of 3-ply construction to 17 percent for thin outer veneers of 7-ply construction. Changes in the modulus of elasticity of the cross plies (t_b) have a more pronounced effect on k_2 than changes in veneer thickness.

For grain orientation in the plywood surface perpendicular to the bending stresses the changes in stiffness can be considerable.

The most dramatic increases in nominal bending stiffness EI' are found for three-ply construction. For equal thicknesses of all three veneers linear interpolation of the k_2 values in Table 2 indicates an actual stiffness about 60 percent higher than the nominal stiffness, if the ratio of the two moduli of elasticity is $E_a/E = 0.05$. As the difference between the two moduli E_a and E decreases the value of k_2 increases greatly. According to Table 2 for 3-ply construction the value of k_2 is sensitive to changes both in veneer ratio and modulus of elasticity ratio.

For 5-ply construction, even with a low E_a/E ratio of 0.05, the increase in stiffness over the nominal stiffness EI', based on parallel plies only, can be as high as 23 percent for thick outer veneers. For larger E_a/E ratios the increase can go well over 50%. The value of k_2 however is not as sensitive to changes in the veneer ratio or the moduli of elasticity as for 3-ply construction.

For 7-ply construction the increase in stiffness is in the order of 3 to 10 percent for a modular ratio of $E_a/E = 0.05$, but could be as high as 50 percent for $E_a/E = 0.20$ and a thick outer veneer.

The reference modulus E for all tables is that of the outer plies in the longitudinal direction $E = E_L$. As long as plywood is manufactured from only one species the ratio E_a/E for parallel loading and the ratio E_b/E for perpendicular loading will both be equal to unity. It should be noted here that for

3-ply construction E_b is identical to E_c. If however, plies "a" and "b" are of different species, then the modulus of elasticity E_b in the longitudinal direction of veneer "b" will differ from E_a in the longitudinal direction of veneer "a" and for loading perpendicular to the grain direction of the outer fibres the ratio E_b/E will not be unity any more. The resulting effect on the stiffness coefficient k_2 is illustrated on four examples in Table 5. For all plywood constructions variations in E_b/E produce considerable changes in the stiffness coefficient. The effect is less for 3-ply construction than for the other 3 examples. In 5- and 7-ply construction the change in k_2 appears to be almost directly proportional to the change in E_b/E. The effect of variations in E_c/E for the examples in Table 5 were also calculated but, as expected, were found to be negligible.

Stress Coefficients

If only parallel plies are considered in the calculation of the section properties I' and S' the actual moment carrying capacity for a given tensile stress f^* will be larger than the product f^*S'. Table 2 indicates that the increases range from zero for most 3-ply constructions to about 17 percent for 7-ply construction with thick and stiff cross-bands, if the grain of the outer veneer is stressed in the longitudinal direction. Rotating the plywood 90 degrees to the applied moments (Fig. 6b) produces somewhat larger increases for 5- and 7-ply construction (Table 4). In 3-ply construction, however, the increases in moment-carrying capacity above those indicated by the section modulus S' become quite formidable. For 3 veneers of equal thickness the increase ranges from about 50 percent for $E_a/E = 0.05$ to more than 100 percent for $E_a/E = 0.20$. For relatively thick outer veneers these increases become even larger. Table 4 indicates that for 3-ply construction the stress coefficient k_4 is very sensitive both to changes in the veneer thickness ratios and to changes in the elastic modular ratio E_a/E.

If for perpendicular loading the reference modulus E is not equal to E_b (5- or 7-ply construction) or E_c (3-ply construction) - this could occur if the plywood were manufactured from two different species - Table 6 indicates considerable differences in k_4. In other words the moment carrying capacity of plywood of any construction is greatly affected by E_L and E_T of both face and cross plies. On the basis of the four examples given in Table 6 it appears that 3-ply construction is slightly less affected than both 5- and 7-ply construction. Similarly to the stiffness coefficient k_2, the bending stress coefficient k_4 also varies in almost direct proportion to the ratio E_c/E for 3-ply and E_b/E for 5- and 7-ply construction.

6. Bending Strength and Stiffness of Individual Veneers

6.1 Veneer Stiffness

A number of equations have been developed above that will allow the calculation of plywood properties of sheets manufactured from one or more species. The basic data required for these calculations are the thickness, the bending strength and the stiffness of individual veneers in their longitudinal and tangential directions. Strength and stiffness of veneers are required for their assembled state, i.e. after being glued and pressed. It is therefore not possible to test individual sheets of veneer and it becomes necessary to calculate veneer properties indirectly.

To compute veneer properties two models will be considered. In the first model it will be assumed that both face veneers contribute to the stiffness of plywood when the outer veneer is stressed perpendicular to its grain direction (Fig. 5). The second model will correspond to the previously made assumption that the face veneer, when stressed in tension perpendicular to the grain, will not contribute to the plywood stiffness. It will be assumed that bending tests are performed with plywood made from one species and grade of veneer. The bending stiffness of this plywood is therefore determined by the moduli of elasticity in the longitudinal (E_L) and the tangential (E_T) directions of the veneer. Since bending stiffnesses can be measured for two different orientations of the plywood -- face plies parallel or perpendicular to the bending stress -- two equations can be set up, each containing the two unknown moduli of elasticity. The solution of these two equations in terms of the observed bending stiffnesses will provide the required values of E_L and E_T.

<u>Model 1</u>. Let K_1 and K_2 be the observed bending stiffnesses for the parallel and perpendicular orientation of the face plies. The stiffnesses are then determined by the following two equations.

$$K_1 = I_1E_L + I_2E_T$$

$$K_2 = I_1E_T + I_2E_L \tag{17}$$

For 7-ply construction (Fig. 5a) the second moments of area for parallel and perpendicular plies are $I_1 = 2(I_a + I_d)$ and $I_2 = (2I_b + I_c)$. For other constructions the appropriate values of I_i have to be chosen. The second moments of area of individual veneers here are circulated for the centroid located at $z_c = 0.5t$. Solving these two linear equations in the usual manner leads to the following expressions for the moduli of elasticity.

$$E_L = (I_1K_1 - I_2K_2)/(I_1^2 - I_2^2) \tag{18}$$

$$E_T = (I_1K_2 - I_2K_1)/(I_1^2 - I_2^2)$$

<u>Model 2</u>. Again the observed stiffnesses are K_1 and K_2. Assuming the face ply stressed in tension perpendicular to the grain to be ineffective (Fig. 5b), the appropriate second moment of area will differ from that for model 1. The stiffnesses are now given by

$$K_1 = I_1E_L + I_2E_T$$

$$K_2 = I_3E_T + I_4E_L \qquad (19)$$

where, for a 7-ply construction, $I_3 = I_{ac} + I_{dc} + I_{dt}$ and $I_4 = I_{bc} + I_{bt} + I_c$. The second moments of area I_{ac} etc. are to be calculated with Eq. 5 using the proper value of z_c. For other plywood constructions the corresponding second moments of areas have to be summed to obtain I_3 and I_4. From Eq. 19 the following two moduli of elasticity are obtained.

$$E_L = (I_3K_1 - I_2K_2)/(I_1I_3 - I_2I_4)$$

$$E_T = (I_1K_2 - I_4K_1)/(I_1I_3 - I_2I_4) \qquad (20)$$

To calculate I_1 and I_3 for model 2 it is necessary first to assume some value for z_c, since z_c is a function of the elastic moduli E_L and E_T (Eq. 4). Having solved Eq. 20 for E_L and E_T a new value of z_c can be computed leading to a revised estimate of E_L and E_T. The process can be repeated until the required degree of precision for the elastic moduli has been reached.

6.2 Veneer Strength

In the following discussion it will be assumed that the bending strength of plywood is determined by the tensile strength of veneer. If plywood bending tests are performed with the face grain oriented parallel and perpendicular to the plane of the applied bending moments M_1 or M_2 (Fig. 6) then two sets of data will be available to determine two independent estimates of the tensile strength of the veneers parallel to the grain. If the grade of the face plies is different from that of the inner plies, but the species is the same for all veneers, the two sets of data could still be used to calculate the tensile strength of each grade separately.

To calculate the required elastic moduli either model 1 or model 2 could be employed. The moment capacity for each of the two orientations is given by Eq. 10. If the outer plies are stressed parallel to their grain, the moment arm z^* will be equal to $t/2$. The corresponding reference modulus E is the modulus of elasticity of the face ply, E_L. The parallel-to-grain tensile strength of the outer veneer therefore will be

$$f^* = f_{at} = 0.5t\ M_1/(I_1 + I_2E_T/E_L) \qquad (21)$$

Similarly the parallel-to-grain tensile strength of the veneer adjacent to the face ply can be calculated from tests on plywood where the face grain is oriented perpendicular to the plane of the moments M_2.

$$f^* = f_{bt} = (t - z_c - t_a)M_2/(I_2 + I_1E_T/E_L) \qquad (22)$$

If model 1 is used, $z_c = 0.5t$. For model 2 the position of the neutral axis given by z_c has to be calculated with Eq. 4 for the particular elastic properties of the test panels. Second moments of area I_1 and I_2 are replaced by the corresponding second moments of area I_3 and I_4, calculated for the appropriate value of z_c.

7. Summary

It is proposed that bending strength and stiffness of multiple species plywood be calculated from certain basic veneer properties. Equations are given showing the relationship between geometrical and elastic properties of the veneer and those of the finished plywood. A method for calculating basic veneer properties from the strength and stiffness of single species plywood is given. It is suggested that the method developed here in some detail for bending can be used equally well for the derivation of other properties of multiple species plywood.

TABLE 1

Stiffness Coefficient k_1

Plywood Construction	Veneer Thickness Ratio				Grain Orientation in Plywood Surface: Parallel to Bending Stresses, $E_a/E = 1.0$, E_b/E				Grain Orientation in Plywood Surface: Perpendicular to Bending Stresses, $E_b/E = 1.0$, E_a/E			
	t_a/t	t_b/t	t_d/t	t_c/t	0.05	0.10	0.15	0.20	0.05	0.10	0.15	0.20
3-ply	0.20			0.60	0.80	0.81	0.82	0.83	0.24	0.25	0.27	0.29
$E_c/E \equiv E_b/E$	0.30			0.40	0.94	0.94	0.95	0.95	0.09	0.11	0.13	0.15
	0.32			0.36	0.96	0.96	0.96	0.96	0.07	0.09	0.11	0.13
	0.34			0.32	0.97	0.97	0.97	0.97	0.06	0.08	0.10	0.11
	0.36			0.28	0.98	0.98	0.98	0.98	0.05	0.07	0.09	0.10
	0.38			0.24	0.99	0.99	0.99	0.99	0.04	0.06	0.08	0.09
	0.40			0.20	0.99	0.99	0.99	0.99	0.03	0.05	0.07	0.08
5-ply	0.10	0.30		0.20	0.52	0.55	0.57	0.60	0.52	0.53	0.54	0.55
$E_c/E = E_a/E$	0.20	0.10		0.40	0.86	0.86	0.87	0.88	0.17	0.20	0.21	0.23
	0.20	0.20		0.20	0.80	0.81	0.82	0.83	0.23	0.25	0.26	0.28
7-ply	0.10	0.10	0.20	0.20	0.71	0.73	0.74	0.76	0.33	0.35	0.37	0.39
$E_d/E = E_a/E$	0.10	0.20	0.10	0.20	0.57	0.59	0.61	0.64	0.47	0.49	0.50	0.52
$E_c/E = E_b/E$	0.10	0.20	0.15	0.10	0.57	0.60	0.62	0.64	0.46	0.48	0.49	0.51
	0.20	0.10	0.10	0.20	0.85	0.86	0.86	0.87	0.18	0.20	0.22	0.24

TABLE 2

Stiffness Coefficient k_2

Plywood Construction	Veneer Thickness Ratio				Grain Orientation in Plywood Surface: Parallel to Bending Stresses $E_a/E = 1.0$, E_b/E				Perpendicular to Bending Stresses $E_b/E = 1.0$, E_a/E			
	t_a/t	t_b/t	t_d/t	t_c/t	0.05	0.10	0.15	0.20	0.05	0.10	0.15	0.20
3-ply $E_c/E = E_b/E$	0.20			0.60	1.01	1.03	1.04	1.06	1.09	1.18	1.26	1.34
	0.25			0.50					1.17	1.33	1.49	1.64
	0.30			0.40	1.00	1.01	1.01	1.01	1.35	1.68	1.99	2.28
	0.32			0.36	1.00	1.01	1.01	1.01	1.49	1.94	2.37	2.76
	0.34			0.32	1.00	1.00	1.01	1.01	1.70	2.35	2.93	3.48
	0.36			0.28	1.00	1.00	1.00	1.00	2.05	3.00	3.85	4.63
	0.38			0.24	1.00	1.00	1.00	1.00	2.67	4.13	5.44	6.61
	0.40			0.20	1.00	1.00	1.00	1.00	3.86	6.30	8.43	10.31
5-ply $E_c/E = E_a/E$	0.10	0.30		0.20	1.05	1.10	1.15	1.20	1.03	1.05	1.07	1.10
	0.10	0.35		0.10	1.05	1.10	1.16	1.21	1.02	1.05	1.07	1.09
	0.15	0.15		0.40	1.02	1.04	1.06	1.08	1.07	1.14	1.20	1.26
	0.15	0.20		0.30	1.02	1.05	1.07	1.09	1.06	1.11	1.16	1.21
	0.15	0.25		0.20	1.03	1.05	1.08	1.10	1.05	1.10	1.15	1.19
	0.15	0.30		0.10	1.03	1.05	1.08	1.10	1.05	1.09	1.14	1.18
	0.20	0.10		0.40	1.01	1.02	1.03	1.04	1.15	1.28	1.41	1.54
	0.20	0.20		0.20	1.01	1.02	1.04	1.05	1.09	1.18	1.27	1.35
	0.20	0.25		0.10	1.01	1.03	1.04	1.06	1.09	1.18	1.26	1.34
	0.25	0.10		0.30	1.01	1.01	1.02	1.02	1.23	1.43	1.63	1.81
	0.25	0.15		0.20	1.01	1.01	1.02	1.03	1.18	1.36	1.52	1.67
	0.25	0.20		0.10	1.01	1.01	1.02	1.03	1.17	1.33	1.49	1.64

TABLE 2 (continued)

Stiffness Coefficient k_2

Plywood Construction	Veneer Thickness Ratio				Grain Orientation in Plywood Surface: Parallel to Bending Stresses $E_a/E = 1.0$, E_b/E				Grain Orientation in Plywood Surface: Perpendicular to Bending Stresses $E_b/E = 1.0$, E_a/E			
	t_a/t	t_b/t	t_d/t	t_c/t	0.05	0.10	0.15	0.20	0.05	0.10	0.15	0.20
	0.10	0.10	0.15	0.30	1.02	1.05	1.07	1.10	1.07	1.13	1.20	1.26
	0.10	0.10	0.20	0.20	1.02	1.04	1.07	1.09	1.07	1.15	1.22	1.29
	0.10	0.15	0.10	0.30	1.04	1.07	1.11	1.14	1.04	1.08	1.12	1.16
7-ply	0.10	0.15	0.15	0.20	1.03	1.07	1.10	1.13	1.05	1.09	1.14	1.18
$E_d/E = E_a/E$	0.10	0.20	0.10	0.20	1.04	1.08	1.13	1.17	1.03	1.07	1.10	1.13
$E_c/E = E_b/E$	0.10	0.20	0.15	0.10	1.04	1.08	1.12	1.16	1.03	1.07	1.10	1.13
	0.15	0.10	0.10	0.30	1.02	1.03	1.05	1.07	1.09	1.17	1.25	1.33
	0.15	0.15	0.10	0.20	1.02	1.04	1.06	1.08	1.07	1.13	1.19	1.26
	0.15	0.20	0.10	0.10	1.02	1.05	1.07	1.09	1.06	1.11	1.16	1.21
	0.20	0.10	0.10	0.20	1.01	1.02	1.03	1.04	1.14	1.27	1.40	1.52
	0.20	0.15	0.10	0.10	1.01	1.02	1.04	1.05	1.11	1.21	1.31	1.41

TABLE 3

Bending Strength Coefficient k_3
for Bending Stresses Perpendicular to the Grain
of the Surface Veneers and $E_b/E = 1.0$

Plywood Construction	Veneer Thickness Ratio				E_a/E			
	t_a/t	t_b/t	t_d/t	t_c/t	0.05	0.10	0.15	0.20
3-ply	0.20			0.60	0.38	0.41	0.43	0.45
	0.25			0.50	0.28	0.31	0.34	0.36
$E_b/E = E_c/E$	0.30			0.40	0.20	0.24	0.27	0.30
	0.32			0.36	0.18	0.22	0.25	0.28
	0.34			0.32	0.16	0.20	0.23	0.26
	0.36			0.28	0.14	0.19	0.22	0.25
	0.38			0.24	0.13	0.18	0.21	0.24
	0.40			0.20	0.12	0.17	0.20	0.22
5-ply	0.10	0.30		0.20	0.64	0.65	0.66	0.67
	0.10	0.35		0.10	0.64	0.66	0.67	0.68
$E_c/E = E_a/E)$	0.15	0.15		0.40	0.42	0.43	0.45	0.46
	0.15	0.20		0.30	0.47	0.48	0.50	0.51
	0.15	0.25		0.20	0.49	0.51	0.52	0.54
	0.15	0.30		0.10	0.50	0.52	0.53	0.55
	0.20	0.10		0.40	0.27	0.29	0.31	0.33
	0.20	0.20		0.20	0.37	0.39	0.41	0.42
	0.20	0.25		0.10	0.38	0.40	0.42	0.44
	0.25	0.10		0.30	0.22	0.25	0.27	0.29
	0.25	0.15		0.20	0.26	0.29	0.31	0.33
	0.25	0.20		0.10	0.28	0.31	0.33	0.35
7-ply	0.10	0.10	0.15	0.30	0.43	0.45	0.47	0.49
	0.10	0.10	0.20	0.20	0.40	0.43	0.45	0.47
$E_d/E = E_a/E$	0.10	0.15	0.10	0.30	0.53	0.55	0.57	0.58
	0.10	0.15	0.15	0.20	0.51	0.53	0.54	0.56
$E_c/E = E_b/E$	0.10	0.20	0.10	0.20	0.58	0.60	0.61	0.62
	0.10	0.20	0.15	0.10	0.57	0.59	0.60	0.61
	0.15	0.10	0.10	0.30	0.37	0.40	0.42	0.44
	0.15	0.15	0.10	0.20	0.43	0.45	0.47	0.48
	0.15	0.20	0.10	0.10	0.47	0.49	0.50	0.52
	0.20	0.10	0.10	0.20	0.29	0.32	0.34	0.37
	0.20	0.15	0.10	0.10	0.34	0.36	0.38	0.40

TABLE 4

Bending Stress Coefficient k_4 for Bending Stresses Perpendicular to the Grain of the Surface Veneers and $E_b/E = 1.0$

Plywood Construction	Veneer Thickness Ratio				E_a/E			
	t_a/t	t_b/t	t_d/t	t_c/t	0.05	0.10	0.15	0.20
3-ply	0.20			0.60	1.07	1.13	1.18	1.24
$E_b/E = E_c/E$	0.25			0.50	1.13	1.24	1.35	1.44
	0.30			0.40	1.27	1.50	1.69	1.86
	0.32			0.36	1.38	1.68	1.94	2.15
	0.34			0.32	1.54	1.96	2.29	2.55
	0.36			0.28	1.80	2.38	2.81	3.16
	0.38			0.24	2.24	3.06	3.64	4.08
	0.40			0.20	3.03	4.20	4.98	5.55
5-ply	0.10	0.30		0.20	1.02	1.03	1.05	1.06
$E_c/E = E_a/E$	0.10	0.35		0.10	1.02	1.03	1.05	1.06
	0.15	0.15		0.40	1.04	1.08	1.12	1.16
	0.15	0.20		0.30	1.03	1.07	1.10	1.13
	0.15	0.25		0.20	1.03	1.06	1.09	1.12
	0.15	0.30		0.10	1.03	1.06	1.09	1.12
	0.20	0.10		0.40	1.08	1.16	1.24	1.32
	0.20	0.20		0.20	1.06	1.12	1.17	1.22
	0.20	0.25		0.10	1.06	1.12	1.17	1.22
	0.25	0.10		0.30	1.13	1.25	1.36	1.46
	0.25	0.15		0.20	1.12	1.22	1.32	1.40
	0.25	0.20		0.10	1.12	1.23	1.32	1.41
7-ply	0.10	0.10	0.15	0.30	1.06	1.11	1.16	1.22
$E_d/E = E_a/E$	0.10	0.10	0.20	0.20	1.06	1.12	1.18	1.24
$E_c/E = E_b/E$	0.10	0.15	0.10	0.30	1.03	1.06	1.09	1.12
	0.10	0.15	0.15	0.20	1.03	1.07	1.10	1.14
	0.10	0.20	0.10	0.20	1.02	1.05	1.07	1.09
	0.10	0.20	0.15	0.10	1.02	1.05	1.07	1.09
	0.15	0.10	0.10	0.30	1.07	1.13	1.19	1.25
	0.15	0.15	0.10	0.20	1.05	1.09	1.14	1.18
	0.15	0.20	0.10	0.10	1.04	1.07	1.11	1.14
	0.20	0.10	0.10	0.20	1.10	1.20	1.29	1.37
	0.20	0.15	0.10	0.10	1.07	1.14	1.21	1.27

TABLE 5

Effect of Variations in Relative Veneer Moduli of Elasticity on Stiffness Coefficient k_2

Grain Orientation in Plywood Surface												
			Parallel to Bending Stresses						Perpendicular to Bending Stresses			
			E_b/E						E_a/E			
Plywood Construction	E_a/E		0.05	0.10	0.15	0.20	E_c/E		0.05	0.10	0.15	0.20
3-ply												
$t_a/t = 0.3$	1.5						1.5		1.86	2.20	2.53	2.84
$t_c/t = 0.4$	1.0		1.00	1.01	1.01	1.01	1.0		1.35	1.68	1.99	2.28
	0.5						0.5		0.84	1.14	1.41	1.64
5-ply	E_a/E	E_c/E					E_b/E	E_c/E				
$t_a/t = 0.1$	1.5	1.0					1.5	0.10	1.53	1.55	1.57	1.60
$t_b/t = 0.3$	1.0	1.0	1.05	1.10	1.15	1.20	1.0	0.10	1.03	1.05	1.07	1.10
$t_c/t = 0.2$	0.5	1.0					0.5	0.10	0.53	0.55	0.57	0.59
$t_a/t = 0.2$	1.5	1.0					1.5	0.10	1.60	1.69	1.77	1.86
$t_b/t = 0.2$	1.0	1.0	1.01	1.03	1.04	1.05	1.0	0.10	1.10	1.18	1.27	1.35
$t_c/t = 0.2$	0.5	1.0					0.5	0.10	0.59	0.68	0.75	0.82
7-ply	E_a/E	E_d/E					E_b/E	E_d/E				
$t_a/t = 0.10$	1.5	1.0					1.5	0.10	1.54	1.57	1.59	1.62
$t_b/t = 0.20$	1.0	1.0	1.04	1.08	1.13	1.17	1.0	0.10	1.04	1.07	1.09	1.12
$t_d/t = 0.10$	0.5	1.0					0.5	0.10	0.54	0.56	0.59	0.61
$t_c/d = 0.20$												
$E_c/E = E_b/E$												

TABLE 6

Effect of Variations in Relative Veneer Moduli of Elasticity on Bending Stress Coefficient k_4

Plywood Construction			Grain Orientation in Plywood Surface Perpendicular to Bending Stresses			
			E_a/E			
3-ply	E_c/E		0.05	0.10	0.15	0.20
$t_a/t = 0.3$	1.5		1.78	2.03	2.25	2.45
$t_c/t = 0.4$	1.0		1.27	1.50	1.69	1.86
	0.5		0.75	0.93	1.07	1.17
5-ply	E_b/E	E_c/E				
$t_a/t = 0.1$	1.5	0.10	1.52	1.53	1.55	1.56
$t_b/t = 0.3$	1.0	0.10	1.02	1.03	1.05	1.06
$t_c/t = 0.2$	0.5	0.10	0.52	0.53	0.54	0.56
$t_a/t = 0.2$	1.5	0.10	1.56	1.62	1.67	1.72
$t_b/t = 0.2$	1.0	0.10	1.06	1.12	1.17	1.21
$t_c/t = 0.2$	0.5	0.10	0.56	0.61	0.65	0.68
7-ply	E_b/E	E_d/E				
$t_a/t = 0.10$	1.5	0.10	1.53	1.55	1.56	1.58
$t_b/t = 0.20$	1.0	0.10	1.03	1.05	1.06	1.08
$t_d/t = 0.10$	0.5	0.10	0.53	0.55	0.56	0.58
$t_c/d = 0.20$						
$E_c/E = E_b/E$						

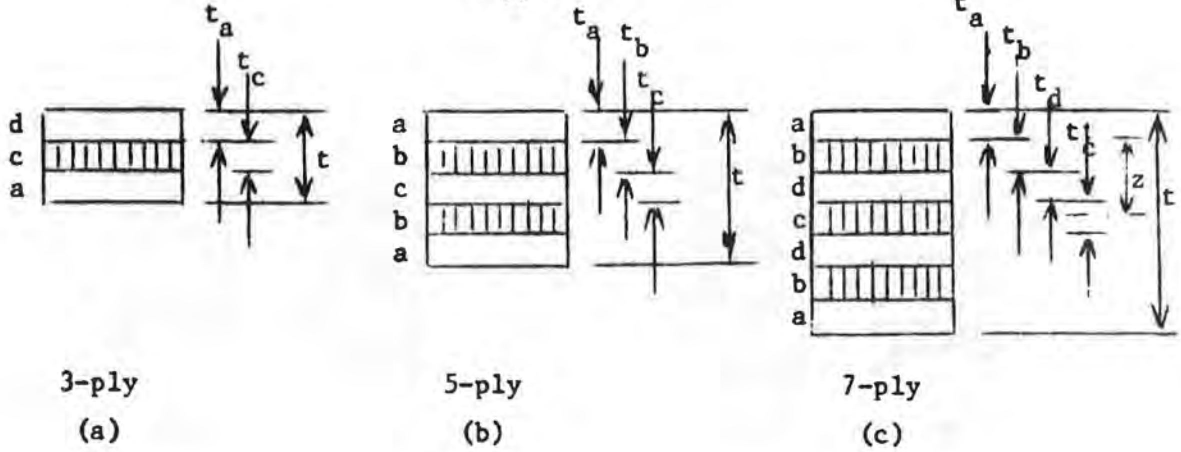

Fig. 1. Plywood Construction -- Typical Sections

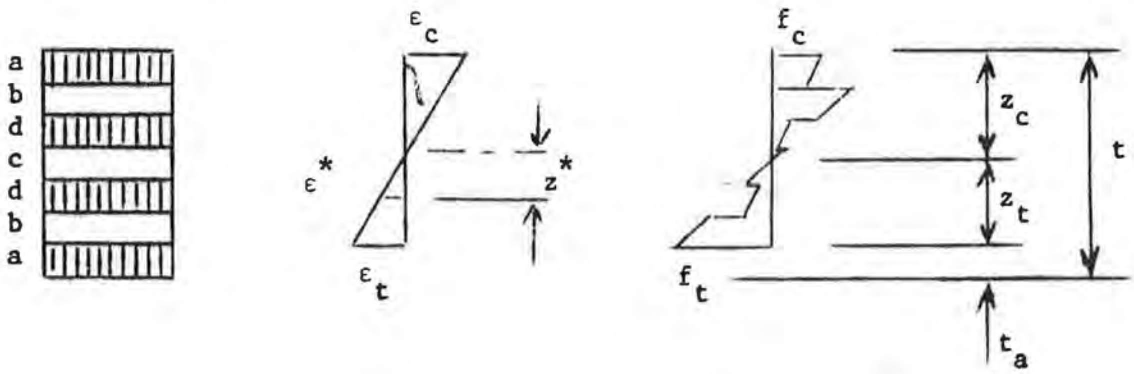

Fig. 2. Bending Stress Distribution -- Perpendicular to Grain Loading

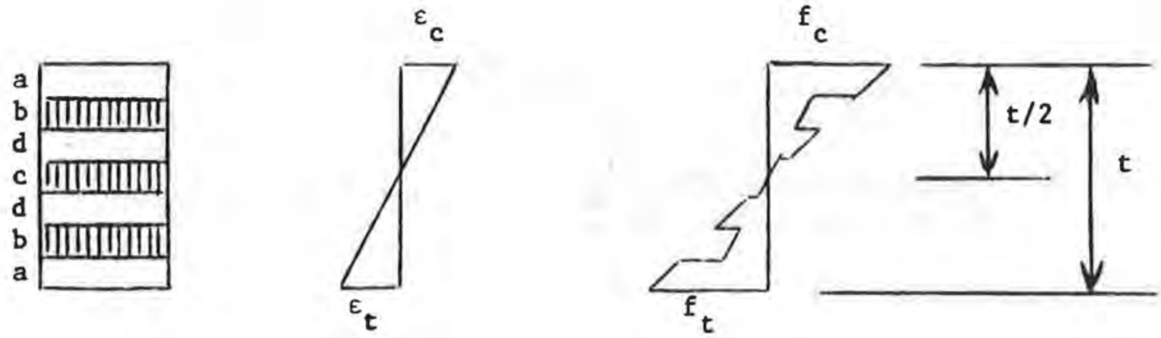

Fig. 3. Bending Stress Distribution -- Parallel to Grain Loading

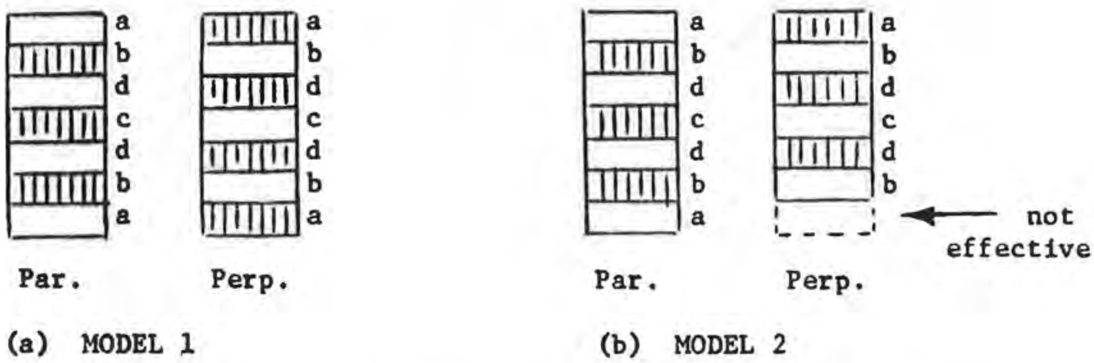

Fig. 5. Direction of Bending Stresses Relative to Grain Orientation in Face Veneer.

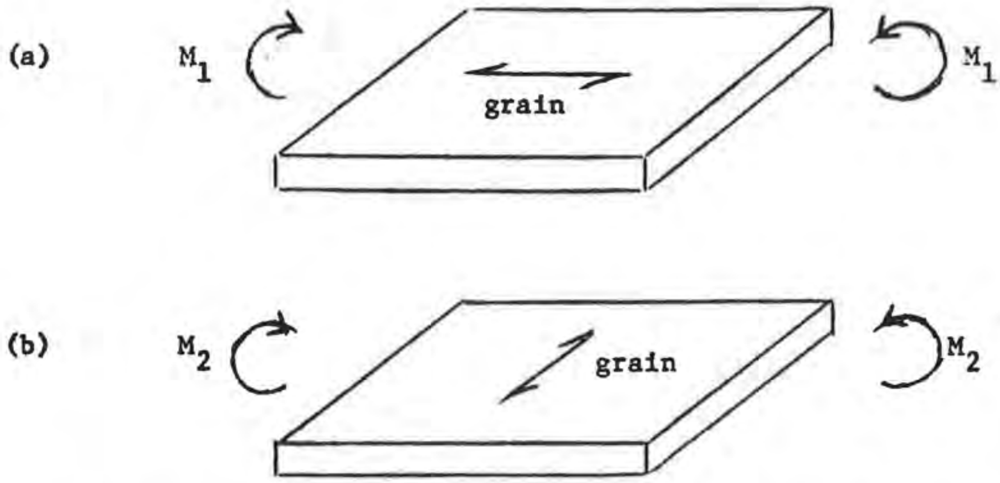

Fig. 6. Moments Producing Stresses Parallel (a) or perpendicular (b) to Grain of Face Veneer.

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SYMBOLS FOR STRUCTURAL TIMBER DESIGN

by

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SYMBOLS FOR TIMBER STRUCTURE DESIGN

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General

In introducing standard symbols for the design of timber structures the ISO recommendations should be followed both in principle and for general applications. Examples of such applications are: forces, stresses, deformations, and geometry. Thus there is little reason for W18 to undertake an extensive comparison of the symbols used in the codes of practice of different countries.

In the application list for wood structures, to be presented to ISO by W18, it would be appropriate to repeat certain general symbols. However, the supplementary symbols considered adequate in the standards for timber structures should be emphasised.

Present standards and proposals for standards

In the Dutch regulations for timber structures, NEN 3852, and in the Norwegian standard, NS 3470, symbols are being used extensively. Contrary to this, in BS CP 112:1971, symbols were used only to a small extent.

Appendix 1 (Appendix 1 contains the Dutch and Norwegian lists. Symbols without subscripts complying with ISO have been encircled.)

The subscripts used in NEN 3852 are based on the Dutch language and only in some cases do they comply with the ISO subscripts, which are based on the English language. The Norwegian symbols also have subscripts deviating from ISO.

Naturally, one has to leave considerable freedom for the use of non-standardised symbols and subscripts, however it should be possible to agree on common symbols and subscripts for the most frequent quantities. Randomly chosen subscripts should not interfere with the ISO-subscripts and consequently, it may be necessary to use two-letter subscripts sometimes.

Subscripts should be used to distinguish between external and internal diameters. Also B and H, as used in the Norwegian standard, should be replaced by b and h.

The ISO symbols Q (q) for live load (intensity of live load) and P (p) for total load is contrary to previous Swedish and Norwegian practice, and to the Dutch Standard. However, if it is possible to design in other materials using the ISO interpretation of p and q, it should also be possible for timber. Therefore it is not recommended that p should stand for permanent, as in the Dutch Standard. Permanent load is equivalent to dead load for which ISO recommends the symbol G together with g for intensity of dead load.

Finally, in the Norwegian standard, slip (in a joint) is denoted by δ. However, Greek lower case letters should be reserved for non-dimensional quantities. The letter, a, used for distance and displacement in the ISO proposals should be acceptable, although this may mean one has to refrain from using this symbol as a geometric measure of the structure. In addition it is difficult to accept the NKB proposal to use the letter a for acceleration and distance simultaneously. The acceleration should reasonably be $\ddot{a} = d^2a/dt^2$.

In appendix 1, the symbols in parentheses are those which are not likely to be acceptable to ISO. For instance, this applies to the letter f for deflection. The use of a Roman lower case letter for deflection is in itself in agreement with the ISO main principle, and this symbol seems fairly common for deflection. However, the ISO document says: a = deflection, and f = strength. But the use of other letters such as y, z or u is not forbidden.

Another symbol which is used in both the Dutch and the Norwegian standard is t for thickness. It is not quite clear if ISO wants it or not for thickness.

Further symbols worthy of comment are those for moisture content and density. In wood-technological literature both u and r are well established symbols. However, the moisture content ratio being non-dimensional, according to ISO should be denoted by a Greek letter. ISO (NKB) propose ω for moisture content (mass of water per mass of solid) and D for density. Most likely these particular ISO symbols will be argued by wood technologists, but there seems to be no real obstacle for introducing ω and D in the Timber Structure Standards.

Further symbols, not marked in the appendix, with the reservation for subscripts, are in compliance with the ISO principles. They are also of a nature that they need not of necessity be identical in different national standards.

REFERENCE: Symbols in building regulations
ISO/TC 98/SC 1 (WG 1-4) 46. January 1973

NOTE:

Following discussion between the authors further points could be made: The β (β^2 is the ratio of effective moments of inertia for the extreme cases of joints which cannot take shear stress to those that can take shear stress without slip) may not be necessary as a standard denotation. Also it may not be very useful to introduce E_ϕ.

The letter κ (kappa) should actually be avoided due to risk of misunderstanding. The letters θ (theta) and ϕ (phi) for rotation and creep are lower case letters, <u>not</u> capitals.

SYMBOLS FOR USE IN THE DESIGN OF TIMBER STRUCTURES

GEOMETRIC QUANTITIES	SYMBOL		UNIT	
Cross section quantities	Ordinary	Printer	Basic	Multiple
Width	b	B	m	mm
Depth, height	h	H		
Depth of compression zone	h_c, c	HJC, C		
Depth of tension zone	h_t, t	HJT, T		
Thickness	h	H		
Thickness	a	A		
Co-ordinate	y	Y		
Co-ordinate	z	Z		
Co-ordinate, polar	r	R		
Radius	r	R		
Radius of gyration	i	I		
Diameter	d	D		
Area	A	AA	m^2	mm^2
Area of flange	A_f	AAJF		
Area of web	A_w	AAJW		
Volume	V	VV	m^3	mm^3
First moment of area	S	SS	m^3	mm^3
Second moment of area	I	II	m^4	mm^4
Moment of resistance	W	WW	m^3	mm^3
Composed section quantities				
Number of members or layers	n	N		
No of member	i	I		
Contribution of member No i to total height	a_i	AJI		
area	A_i	AAJI		
second moment	I_i	IIJI		
Second moment of area of member No i (based on own centre)	I_{0i}	IIJOI		
Coeff of disintegration $\sqrt{\Sigma I_{0i}/I}$	β	BE		
Effective second moment of area	I_e	IIJE		

GEOMETRIC QUANTITIES	SYMBOL		UNIT	
Other geometric quantities	Ordinary	Printer	Basic	Multiple
Length, span	l	L	m	mm
Co-ordinate	x	X		
Critical (buckling) length	l_c	LJC		
Spacing (distance centre to centre)	s	S		
Edge distance (d centre to edge)	c	C		
Excentricity	e	E		
Knot diameter ratio	κ_d	KAJD		
Knot area ratio	κ	KA		
DIRECTIONS (Subscripts)				
Angle between force and fibre direction	α	AL		
Parallel (to fibre direction)	$\|\|$ or par	PAR		
Perpendicular (to fibre direction)	$\perp$ or tra	TRA		
Parallel to fibre direction (longitudinal)	l	L		
Radial to annual rings	r	R		
Tangential to annual rings	t	T		
For shear stress (τ)				
Direction r, in plane perpendicular to l	lr	LR		
Direction z, in plane perpendicular to x	XZ	XZ		
etcetera				
For shear deformation (γ) and rigidity moduli (G)				
Shear caused by τ_{lr}	lr	LR		
etcetera				

Special applications

Equal symbols or x, y, z can be used for laminated timber or panels:

Laminated wood	Plywood and similar prod	Particle- and fibreboard	SYMBOL Ordinary	SYMBOL Printer	UNIT Basic	UNIT Multiple
Fibre direction of laminae	Fibre direction of face veneer	Machine- or longitudinal direction	l	L		
			x	X		
Perpendicular to fibre direction of laminae, parallel to plane of joints	Perp to fibre direction of face veneer, parallel to glue lines	Perp to machine- or longitudinal direction, parallel to face veneer	t	T		
			y	Y		
Perpendicular to plane of joints	Perp to face (through thickness)	Perp to machine- or longitudinal direction	r	R		
			z	Z		

FORCES AND STRESSES	SYMBOL Ordinary	SYMBOL Printer	UNIT Basic	UNIT Multiple
Force in general	F	FF	N	kN, MN
Normal force, axial force	N	NN		
Axial force in member also	S	SS		
Shear force	V	VV		
Moment	M	MM		
Torque	T	TT		
Reaction force	R	RR		
Horizontal reaction force	H	HH		
Vertical reaction force	V	VV		
Reaction or force in general, parallel to X-axis	X	XX		
Reaction or force in general, parallel to Y-axis	Y	YY		
Reaction or force in general, parallel to Z-axis	Z	ZZ		

	SYMBOL Ordinary	SYMBOL Printer	UNIT Basic	UNIT Multiple
For intensity (force per unit length or area) use lower case letters. See also stresses:				
Normal stress	σ	SI	$Pa=N/m^2$	$MPa=N/mm^2$
Compressive stress	σ_c	SIJC		
Tensile stress	σ_t	SIJT		
Bending stress (M/W)	σ_b	SIJB		
Shear stress	τ	TA		
STRAIN, DEFORMATION AND DISPLACEMENT				
Strain (incl compressive strain)	ε	EP		
Displacement, deflection	a	A	m	mm
Displacement deflection	y	Y		
Rotation	θ	TH		
Shear	γ	GA		
<u>Rheology quantities (subscripts)</u>				
Creep	c	C		
Recovery	r	R		
Permanent (irrecoverable)	ir	IR		
Creep coefficient (function)	ϕ	PH		
Relaxation coefficient (function)	ψ	PS		
<u>Moduli and coefficients</u>				
Elasticity modulus	E	EE	$Pa=N/m^2$	$MPa=N/mm^2$ $GPa=kN/mm^2$
Fictive elasticity modulus $\frac{E}{1+\phi}$	E_ϕ	EEJPH		
Rigidity (shear modulus)	G	GG		
Displacement modulus	K	KK		
Slip modulus	k	K	N/m	N/mm
Poisson's ratio	ν	NU		
Viscocity for pure flow	η	ET	Ns/m^2	

	SYMBOL		UNIT	
	Ordinary	Printer	Basic	Multiple
PHYSICAL AND MECHANICAL QUANTITIES				
Temperature	T	TT	^{o}C, K	
Relative humidity	RH	RRHH		
Moisture content ratio	ω	OM		
M.c.r, mass of water to mass of solid	ω	OM		
M.c.r, mass of water to mass of water + solid	ω'	OM'		
Density	D	DD	kg/m^3	
Density, weight at ω, volume at ω	D_{ω}	DDJOM		
Density, weight at $\omega = 0$, volume at ω	D_0	DDJO		
Density, weight at $\omega = 0$, volume at $\omega = 0$	D_{00}	DDJOO		
Coefficient of swelling	α	AL		
Coefficient of shrinkage	β	BE		
Weight	G	GG		
Strength, general	f	F		
Strength in compression	f_c	FJC	N/m^2	N/mm^2
Strength in tension	f_t	FJT		
Strength in shear	f_v	FJV		
SPECIAL SUBSCRIPTS				
Characteristic value of force (load), strength or deformation	k	K		
Mean value	m	M		
Design value	d	D		
Ultimate value	u	U		
Yield value	y	Y		
Admissible (permissible) value	adm or a	ADM or A		
Critical value	crit	CRIT		
Critical (length)	c	C		

		blz.
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Lijst van symbolen

A	doorsnede
A_a	doorsnede van een aangesloten staafdeel of som van de doorsneden van de aan één zijde van de materiaalvrije as aangesloten staafdelen (bij samengestelde staven)
ΣA	doorsnede gehele staaf (bij samengestelde staven)
$\cancel{D}$	dwarskracht
	grootste afmeting van een ringdeuvel, kramplaat of sluitplaat (bij verbindingen)
D_M	dwarskracht veroorzaakt door het buigend moment
D_N	dwarskracht veroorzaakt door de normaalkracht bij uitbuiging
E	elasticiteitsmodulus
E	elasticiteitsmodulus evenwijdig aan de vezelrichting
$E_{\perp}$	elasticiteitsmodulus loodrecht op de vezelrichting
F	kracht of belasting
F_k	basisbelasting per kramplaat
F_p	permanente belasting
$\bar{F}$	toelaatbare belasting
$\bar{F}_{\parallel}$	toelaatbare belasting op uittrekking (evenwijdig aan de as van draadnagel resp. houtschroef)
$\lvert F \rvert$	absolute waarde van F
G	afschuivingsmodulus
$G_{\parallel}$	afschuivingsmodulus bij afschuiving t.g.v. schuifspanningen in een vlak evenwijdig aan de vezelrichting
$G_{\perp}$	afschuivingsmodulus bij afschuiving t.g.v. schuifspanningen in een vlak loodrecht op de vezelrichting
I	traagheidsmoment
I_a	traagheidsmoment van een staafdeel om zijn eigen as (bij samengestelde staven)
I_m	traagheidsmoment van de gehele staaf t.o.v. de materiaalas
I_{th}	theoretisch traagheidsmoment van de gehele staaf
I_w	werkzaam traagheidsmoment
L	afschuivende kracht per staaflengte (bij samengestelde staven)
M	buigend moment

Q	totale belasting
Q_h	totale zijdelingse belasting
$\cancel{R}$	kromtestraal van de as van een constructiedeel
R_1	kromtestraal van een plank in een constructiedeel
S	statisch moment
S_a	statisch moment van het aangesloten deel (bij samengestelde staven)
W	weerstandsmoment
a	afstand hart op hart van vloerbalken gemiddelde afstand hart op hart van de in één rij geschoven gedachte verbindingsmiddelen (bij samengestelde staven)
b	breedte grondbreedte van vingerlas
c	coëfficiënt afstand van de punten waarin een constructiedeel zijdelings wordt gesteund
d	middellijn
d_b	middellijn bout
d_g	middellijn gat
d_n	middellijn draadnagel
$d_{//}$, $d_{\perp}$	afstand hart op hart van twee verbindingsmiddelen in één staaf gemeten resp. evenwijdig aan en loodrecht op de staafas.
e	afstand van zwaartepunt staafdeel tot materiaalvrije as (bij samengestelde staven) eindafstand (bij verbindingen)
e_b	afstand tot het belaste staafeinde
e_o	afstand tot het onbelaste staafeinde
(f)	doorbuiging, pijl van een boog, factor
f_s	doorbuiging veroorzaakt door dwarskrachtvervorming
f_∞	doorbuiging in de eindtoestand
h	hoogte
i	traagheidsstraal
i_h	traagheidsstraal van drukflens bij uitknikken uit het vlak van de ligger (bij vollewand-constructies)
i_y	kleinste bij de kniklengte l_{k_y} behorende traagheidsstraal van de gehele doorsnede (bij samengestelde staven)
i_1	kleinste bij de kniklengte l_{k_1} behorende traagheidsstraal van een staafdeel (bij samengestelde staven)
j	aantal in de krachtrichting achter elkaar geplaatste verbindingsmiddelen
k	verschuivingsmodulus (kracht per verschuivingsafstand)
l	lengte, overspanning
l_h	hechtlengte
l_k	kniklengte
l_{ky}	kniklengte van de gehele staaf bij beschouwing van knik loodrecht op de Y-as (bij samengestelde staven)
l_{k_1}	kniklengte van een staafdeel bij beschouwing van knik loodrecht op de Y-as (bij samengestelde staven)
l_u	uitkragende lengte (bij een kraagligger)
m	aantal staafdelen waaruit een samengestelde staaf bestaat
m_1, m_2, ...	modificatiefactoren
n	aantal
$\cancel{p}$	permanente verdeelde belasting
q	veranderlijke verdeelde belasting
r	kernstraal randafstand (bij verbindingen)
r_b	afstand tot de belaste rand
r_o	afstand tot de onbelaste rand

s	lengte van een boogdeel
	steek (bij vingerlassen)
t	dikte
	diepte van tand, hiel of hieltand (bij verbindingen)
t_a	kleinste per snede aan te sluiten houtdikte
t_k	kleinste aan te sluiten houtdikte
t_{kt}	kleinste aan te sluiten triplexdikte
t_m	dikte van tweezijdig aangesloten hout (middenhout)
t_z	dikte van éénzijdig aangesloten hout (zijhout)
u	totale uitbuiging van een staaf in belaste toestand
u_o	toevallige uitbuiging van een staaf in onbelaste toestand
z_u	afstand van de materiaalvrije as tot de uiterste vezel van het lijf (bij samengestelde staven)
α	lasthoek (vergelijk 5.1.7)
β	hoek
γ	samenwerkingsfactor (d.i. de factor die de mate van samenwerking van de delen van een samengestelde staaf in rekening brengt)
	hielhoek (bij hielverbindingen)
η	gedeelte van de totale op een constructiedeel te rekenen belasting dat permanent werkzaam is: $\eta = p/(p + q)$
λ	slankheid
λ_w	werkzame slankheid
λ_x	slankheid van de gehele staaf betrokken op de X-as
λ_y	$= l_{ky}/i_y$, theoretische slankheid van de gehele staaf, betrokken op de Y-as (bij samengestelde staven)
λ_1	$= l_{k1}/i_1$, slankheid van een staafdeel, betrokken op zijn eigen as evenwijdig aan de Y-as van de gehele samengestelde staaf
σ	spanning
σ_b	buigspanning
σ_d	drukspanning evenwijdig aan de vezelrichting
$\sigma_{d\perp}$	drukspanning loodrecht op de vezelrichting
$\sigma_{d\alpha}$	drukspanning in een richting onder een hoek α met de vezelrichting
σ_p	berekende spanning ten gevolge van de permanente belasting
σ_q	berekende spanning ten gevolge van de veranderlijke belasting
σ_t	trekspanning
σ_w	berekende spanning ten gevolge van de windbelasting
σ_λ	drukspanning bij slankheid λ
$\bar{\sigma}$	toelaatbare spanning
$\sigma_\perp$	trekspanning loodrecht op de lijmvoeg (bij gelamineerde constructies)
τ	schuifspanning
$\bar{\tau}$	toelaatbare schuifspanning
ϕ	reductiefactor voor een geconcentreerde vloerbelasting
ϕ'	reductiefactor voor een geconcentreerde dakbelasting
ψ	coëfficiënt

Romdensitet – vekt i tørr tilstand dividert med målt volum ved et bestemt fuktighetsinnhold.

Rulleskjærspenning – skjærspenning i finérets plan tvers på et enkeltfinérs fiberretning.

Stift – i denne standard er med stift ment firkantet (skarpkantet) og rund tradstift.

Tre – i denne standard er med tre ment kvalitetssortert trevirke av gran og furu med fastlagte styrke- og elastisitetsegenskaper. Trevirket kan være fingerskjøtt og trykkimpregnert.

1.5 Symboler

A	tverrsnittsareal
A_d	delstavers tverrsnittsareal
A_{net}	netto tverrsnittsareal
~~B~~	bredde
E	elastisitetsmodul, tøyningsmodul
F	kraft, last, kapasitet
F_r	omregnet bruddlast
G	skjærmodul, tyngde
~~H~~	høyde
I	arealtreghetsmoment, (aksialt arealmoment)
I_e	effektivt arealtreghetsmoment
I_p	polart arealtreghetsmoment, (polart arealmoment)
L	lengde, spennvidde
L_k	beregningsmessig knekklengde
M	bøyningsmoment
N	normalkraft (nyttelast)
S	statisk moment av areal
V	skjærkraft
W	motstandsmoment
a	avstand
b	bredde
b_e	effektiv bredde
d	diameter, tverrmål
e	eksentrisitet, avstand
f	friksjonskoeffisient, nedbøyning
g	egenlast
h	høyde
i	treghetsradius
+ k	forbindelsesmidlers stivhet
k_τ	faktor ved beregning av skjærspenning ved bjelkeinnsnitt
k_λ	faktor ved knekningsberegning
l	lengde, spenvidde
l_k	knekklengde
m	fugeantall
n	antall
p_0	aksialpåkjenning i spikerplater for vinkel v = 0
p_v	aksialpåkjenning i spikerplater for vinkel v

p	fordelt nyttelast
q	fordelt totallast
r	radius (romdensitet)
s	standardavvik (skjærspenning i spikerplater)
t	(lamelltykkelse, trevirketykkelse) tid, temperatur
u	fuktighetsinnhold
v	vinkel
y	nedboyning
α	vinkel
β	forholdstall, vinkel
γ	vinkel
γ_1	koeffisient for sortering
γ_2	koeffisient for utforelsesklasse
γ_3	koeffisient for beregningskontroll
γ_4	koeffisient for bruddkonsekvens
γ_f	lastkoeffisient
γ_m	materialkoeffisient
$\cancel{\gamma}$	glidning, forskyvning
ε	toyning
λ	slankhet
λ_e	effektiv slankhet
λ_v	bjelkeslankhet ved vippingsberegning
σ	aksialspenning, normalspenning
σ_b	boyningsspenning
σ_d	dimensjonerende materialfasthet
σ_F	flytegrense for stål
σ_k	trykkspenning ved knekningsberegning
σ_{kar}	karakteristisk fasthet
σ_p	prismestyrke for tre
σ_s	strekkspenning
$\sigma_{s\parallel}$	strekkspenning parallelt med fibrene
$\sigma_{s\perp}$	strekkspenning tvers på fibrene
$\cancel{\sigma_t}$	trykkspenning
$\sigma_{t\parallel}$	trykkspenning parallelt med fibrene
$\sigma_{t\perp}$	trykkspenning tvers på fibrene
σ_α	trykkspenning for skrå trykkflater
τ	skjærspenning
$\tau_\parallel$	skjærspenning parallelt med fibrene
$\tau_\perp$	skjærspenning tvers på fibrene (som regel betegner dette rulleskjærspenning)
τ_0	skjærpåkjenning mellom spikerplate og tre for v = 0
τ_v	skjærpåkjenning mellom spikerplate og tre for vinkel v
τ_M	skjærpåkjenning mellom spikerplate og tre pga. moment M

1.6 Enheter

I disse regler er SI-enheter innført.

Multipler av 10^{3n} bør prefereres f.eks. MN, kN, N.

Tabell 1.6 Oversikt over endel enheter

Størrelse	Enhet		
	Benevning		Multipler
lengde	meter	m	mm
areal	kvadratmeter	m^2	mm^2
kraft, (last)	newton	N	kN
moment	newtonmeter	Nm	kNm
spenning, elastisitets-modul	pascal = newton pr. kvadratmeter	1 Pa = 1 N/m^2	1 MPa = 1 N/mm^2 1 kPa = 1 kN/m^2
kraft pr. lengde	newton pr. meter	N/m	kN/m
kraft pr. areal	pascal = newton pr. kvadratmeter	1 Pa = 1 N/m^2	1 kPa = 1 kN/m^2

Omregning fra tekniske enheter til SI-enheter

For kraft:

$$1 \text{ kp} = 9{,}806\,65 \text{ N} \approx 10 \text{ N}$$

For spenning:

$$1 \text{ kp/cm}^2 = 0{,}098\,066\,5 \text{ MPa} \approx 0{,}1 \text{ MPa} = 0{,}1 \text{ N/mm}^2$$

For øvrig skal Norsk Standard følges.

Det vises til

Norges Standardiseringsforbunds hefte: P53, Tekniske og fysiske størrelser og enheter – SI, mai 1972.

NKB-skrift nr. 16, november 1970, Måttenheter enligt SI

PAPER 6
3-2-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 TIMBER STRUCTURES

THE DESIGN OF BUILT-UP TIMBER COLUMNS

by

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DELFT - JUNE 1974

THE DESIGN OF BUILT-UP TIMBER COLUMNS.

* * *

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1. SUMMARY

The introduction in Section 2 reviews the general basis for the computation of built-up structures where the individual components are connected by semi-rigid connections.

In Section 3 the load-carrying capacity of long perfect columns is determined. It is found that this can be expressed by the Euler formula by using a reduced effective moment of inertia, I_e. It is suggested that the load-carrying capacity be generally based on the usual expression, but with the slenderness ratio determined from I_e and not from the total moment of inertia I. The justification for this is discussed.

Section 4 gives expressions for the effective moments of inertia for various types of columns: continuously jointed columns, spaced columns with glued, nailed or bolted packs, spaced columns with glued or nailed battens (Vierendeel columns) and glued or nailed lattice columns.

Section 5 describes a number of tests with the said types of columns, partly to assess the applicability of the proposed theoretical expressions and partly to determine the rigidity of the connections.

In Section 6 various approximations are mentioned and assessed, and Section 7 gives a brief proposal for design rules.

Section 8 consists of a summary of the literature to which direct reference is made in the text (indicated by [ ]), and Section 9 gives the generally used symbols. Symbols which are only used locally in the text are not included.

2. GENERAL THEORY FOR BUILT-UP BEAMS.

2.1. General expressions.

A beam with a single symmetric cross-section consisting of N rectangular lamellas, as shown in fig. 1 is considered.

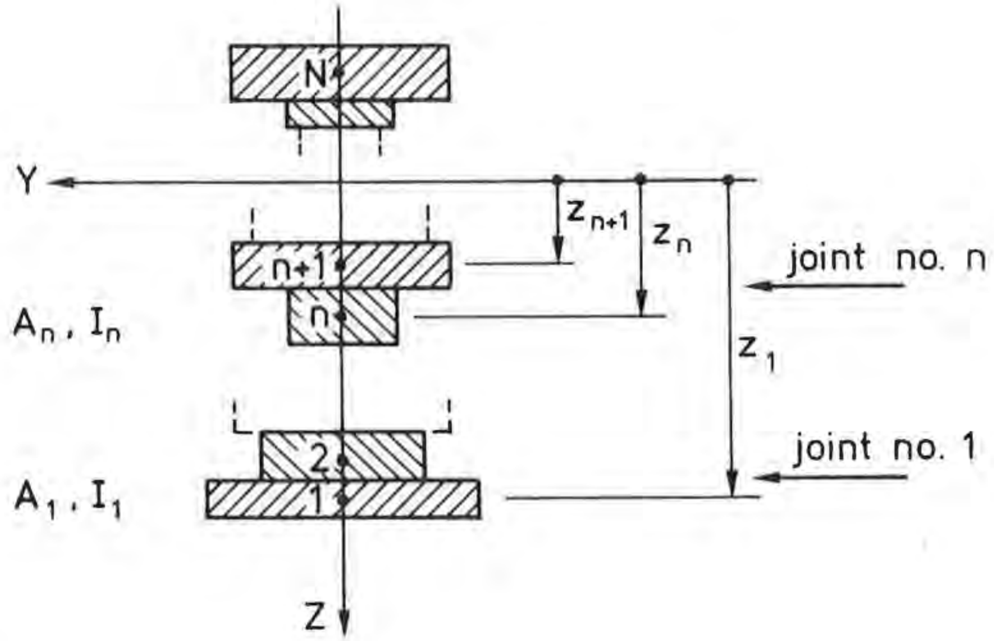


Fig. 1. Cross-section.

The areas of the individual lamellas are A_1 ..... A_N, and their moments of inertia about their own centre of gravity axes are I_1 ..... I_N. The centres of gravity of the lamellas are given by z_1 ..... z_N. The position of the Y-axis is temporarily arbitrary. The notation

$$I_o = \sum_{i=1}^{N} I_n = \beta^2 I \tag{1}$$

is introduced. I is the total geometric moment of inertia about the Y-axis:

$$I = I_o + \sum_{n=1}^{N} A_n z_n^2 \tag{2}$$

The lamellas are supposed connected by semi-rigid connections, for which the following is valid:

$$P = kS \tag{3}$$

where P is the load on a connection, and S is the slip. k is named the stiffness number of the connection.

If H is the shearing force per unit length in the joint, and a is the distance between the connections, then

$$P = aH \tag{4}$$

H, a and k can vary in the longitudinal direction of the beam as well as from joint to joint, which will be indicated by the index n on these parameters. It is assumed that a is so small that the structure acts as though the connections were continuous.

The usual assumptions in the theory of linear elasticity are made. In the following, all lamellas are considered as having the same modulus of elasticity E. The case in which E varies from lamella to lamella can be treated in the usual manner by weighing the individual lamellas in proportion to their stiffness. The longitudinal axis of the beam is indicated by x. Differentiation with respect to x is indicated by' .

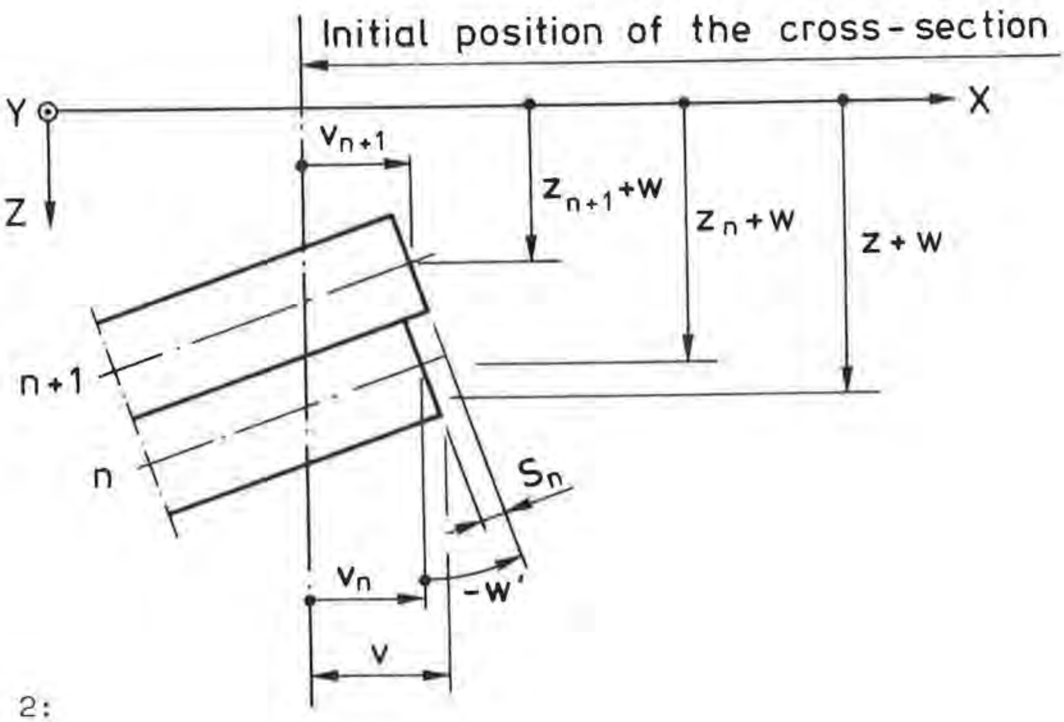


Fig. 2:

The displacement of the beam in the Z direction is called w, and its curvature at any arbitrary point is -w".

The centre of gravity of the n^{th} lamella is assumed to have a displacement v_n in the longitudinal direction, i.e. an arbitrary fiber in the lamella has the displacement

$$v = v_n - w'(z - z_n) \tag{5}$$

As v' is the strain in the longitudinal direction, the axial stress σ_n in the n^{th} lamella varies according to the expression

$$\sigma_n = Ev' = E(v_n' - w''(z - z_n)) \tag{6}$$

The resultant force from the stresses in the n^{th} lamella is an axial force

$$N_n = EA_n v_n' \tag{7}$$

and a moment

$$M_n = -EI_n w'' \tag{8}$$

The total axial force is called N (positive as tension) and the total moment M (positive when there is tension in lamella no. 1). The equations of equilibrium give

$$M = -EI_o w'' + \sum_{n=1}^{N} EA_n z_n v_n' \tag{9}$$

og

$$N = \sum_{n=1}^{N} EA_n v_n' \tag{10}$$

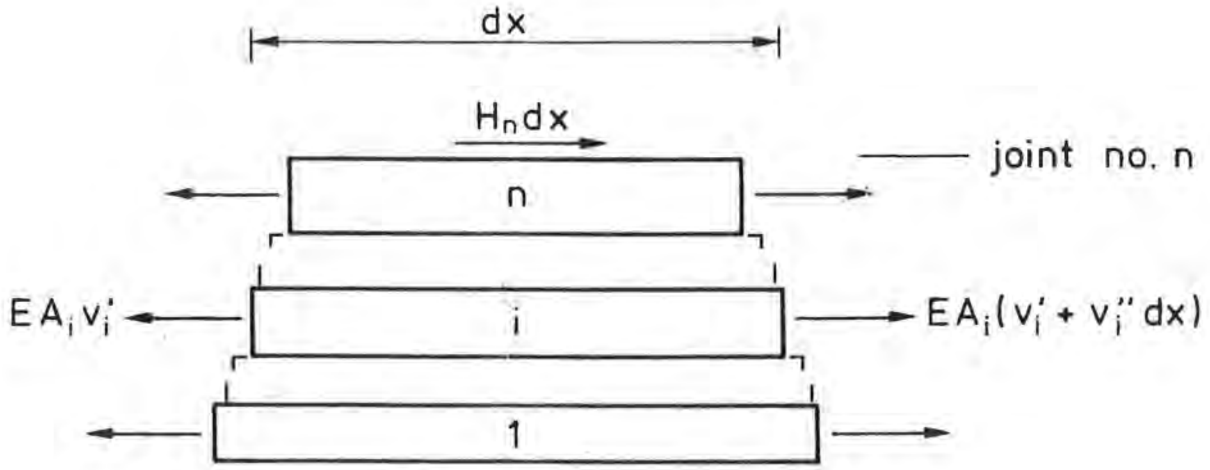


Fig.3

The shearing force per unit length H_n in the n^{th} joint is,(see fig. 3.)

$$H_n \mp - \sum_{i=1}^{n} EA_i v_i'' \tag{11}$$

In this joint the slip, referring to fig. 2 and formula (5) is

$$S_n = v_{n+1} - v_n - w'(z_n - z_{n+1}) \tag{12}$$

Substitution of (11), (12) and (3) into equation (4) gives

$$v_n - v_{n+1} + w'(z_n - z_{n+1}) = \frac{a_n}{k_n} \sum_{i=1}^{n} EA_i v_i'' \tag{13}$$

(9), (10) and (13) are $1 + 1 + (N-1) = N + 1$ equations for the determination of w and the N unknown displacements $v_1 \ldots\ldots v_N$.

A first order theory of elasticity is used, therefore a unique solution is found for a given load. Furthermore, the law of superposition is valid. As the solution for N alone is seen to be

$$\sigma = \text{constant} = \frac{N}{A}$$

it is only necessary in the following to investigate M.

The solution of the given equations is very cumbersome and can seldom be given in a form suitable for practical use. In the following section approximate expressions for the deflections and stresses will therefore be given.

2.2. Approximate expressions.

It is assumed that $\frac{k}{a}$ is constant, i.e. independent of x and the same for all joints.

It is further assumed that the displacement of the centre of a lamella can be written as

$$v_n = cz_n \tag{14}$$

with the X-axis placed at the centre of gravity of the total cross-section, i.e. so that

$$\sum_{i=1}^{N} A_i z_i = 0 \tag{15}$$

For single symmetric cross-sections with two lamellas, and for double symmetric cross-sections with three lamellas (14) is satisfied, i.e. for these the only new approximation is that $\frac{k}{a}$ is assumed constant. As explained above N = O.

It is seen that (1O) is satisfied as

$$\sum_{n=1}^{N} EA_n v_n' = Ec' \sum_{n=1}^{N} A_n z_n = 0$$

Equation (14) is substituted into equation (9)

$$M = -EI_o w'' + Ec' \sum_{n=1}^{N} A_n z_n^2 = -EI_o w'' + Ec'(I-I_o) \tag{16}$$

and into the (N-1) equations (13)

$$c + w'' = \frac{aE}{k} c'' \sum_{i=1}^{N} \frac{A_i z_i}{z_n - z_{n+1}} \tag{17}$$

The addition of these (N-1) equations gives

$$(N-1)(c+w'') = \frac{aEA^*}{k} c'' \tag{18}$$

as the following notation is introduced:

$$A^* = \sum_{n=1}^{N-1} \sum_{i=1}^{n} \frac{A_i z_i}{z_n - z_{n+1}} = \frac{A_1 z_1}{z_1 - z_2} + \frac{A_1 z_1 + A_2 z_2}{z_2 - z_3} \ldots\ldots$$

$$\ldots\ldots + \frac{A_1 z_1 + A_2 z_2 + \ldots\ldots + A_{N-1} z_{N-1}}{z_{N-1} - z_N} \tag{19}$$

The constants A and N of the cross-section are given in fig. 4 for a number of cross-sections. For the cross-sections 1-7 (14) is satisfied, while this is not the case for cross-section 8. Cross-sections 6-8 are composed of 2-4 actual lamellas with spaces. As it is assumed in the above that the lamellas lie close to one another (see fig. 1), 1-3 lamellas with area zero must be added, corresponding to the spaces.

Differentiating (18) and substituting c' from (16) gives

$$EI_o w'''' + M'' - \frac{(N-1)k}{aEA^*}(EIw'' + M) = 0 \tag{20}$$

From this equation w can be found and then from (16)

$$c' = \frac{M + EI_o w''}{E(I - I_o)} = \frac{-w_o'' + \beta^2 w''}{1 - \beta^2} \tag{21}$$

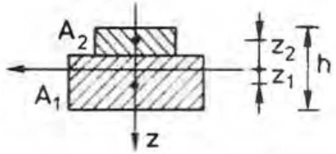
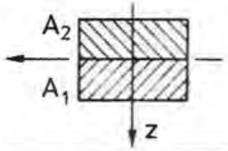
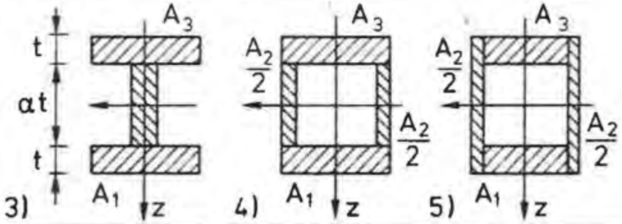
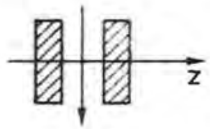
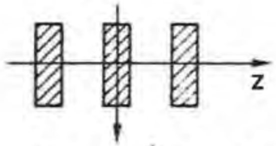
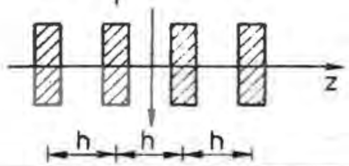
where β^2 is defined in (1) and w_o is the deflection that will be found by complete interaction:

$$w_o'' = -\frac{M}{EI} \quad . \tag{22}$$

Substitution of (21) into (6) gives the axial stresses in the n^{th} lamella

$$\sigma_n = \frac{w''}{w_o''} \frac{M}{I} \left(z - z_n \frac{1 - \frac{w_o''}{w''}}{1 - \beta^2} \right) \tag{23}$$

Fig. 4.

No.	Cross section	N	A^*	$\frac{A^*}{N-1}$
1		2	$\frac{2A_1z_1}{h} = \frac{2A_2(-z_2)}{h}$	$\frac{2A_1z_1}{h}$
2	 $A_1 = A_2$	2	$\frac{A_1}{2}$	$\frac{A_1}{2}$
3-5	 $A_1 = A_3$	3	$2A_1$	A_1
6	 All laminations (shafts) have the area A_1	3	$2A_1$	A_1
7		5	$8A_1$	$2A_1$
8		7	$20A_1$	$\frac{10}{3}A_1$

The following is found from (11), (14) and (21):

$$H_n = -Ec'' \sum_{i=1}^{n} A_i z_i = \frac{Ew_o'''S_n}{1-\beta^2}\left(1-\beta^2 \frac{w'''}{w_o'''}\right)$$

$$= \frac{QS_n}{I} \frac{1-\beta^2\frac{w'''}{w_o'''}}{1-\beta^2} = H_{on} \frac{1-\beta^2 \frac{w'''}{w_o'''}}{1-\beta^2} \tag{24}$$

S_n is the static moment about the y-axis of the lamellas 1 to n, and H_{no} is the shearing force found by complete interaction.

$$Q = EIw_o''' \tag{25}$$

is used where Q is the shear force.

2.3 Sinusoidal Load Intensity.

As an example the conditions in a simple supported beam with length l and loaded with a sinusoidal load which gives the moment

$$M = M_o \sin \frac{\pi x}{l}$$

are investigated (see fig. 5)

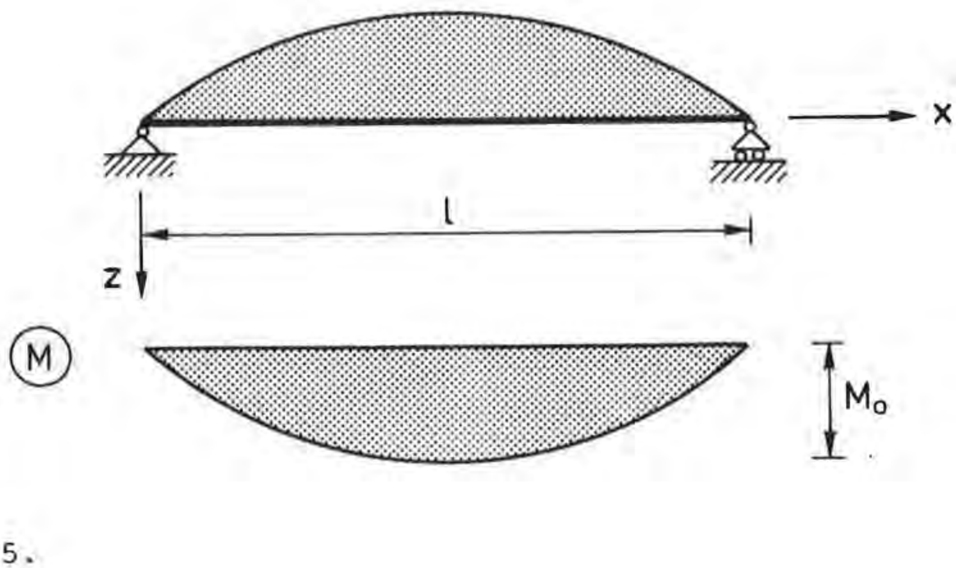


Fig. 5.

It is found from (20) that the deflection curve is also sinusoidal, and that the following is valid at any point:

$$\frac{w}{w_o} = \frac{w'}{w'_o} = \frac{w''}{w''_o} = \frac{w'''}{w'''_o} = \frac{1 + \mu}{1 + \mu\beta^2}$$

where

$$\mu = \frac{\pi^2 aEA^*}{(N-1)kl^2} \tag{26}$$

i.e. the deflections can be determined in the usual way if an effective moment of inertia I_e is used instead of the moment of inertia I

$$I_e = \frac{1 + \mu\beta^2}{1 + \mu} I = I_o + \frac{I - I_o}{1 + \mu}$$

$$= I_o + \gamma \sum_{n=1}^{N} A_n z_n{}^2 \tag{27}$$

where

$$\gamma = \frac{1}{1 + \mu} \tag{28}$$

It is emphasized that this is valid only for the sinusoidal load in question; in all other cases w/w_o and thereby the relationships between the derived parameters vary from point to point.

From (23)

$$\sigma_n = \frac{M}{I_e}\left(z - z_n \frac{1 - I_e/I}{1 - \beta^2}\right) \tag{29}$$

is found, and from (24)

$$H_n = H_{no} \frac{1 - \beta^2 I/I_e}{1 - \beta^2} \tag{30}$$

3. LOAD-CARRYING CAPACITY OF COLUMNS, THEORY.

3.1 Perfect columns.

A perfect column will have a stability failure and the load-carrying capacity (determined by the Euler formula) is found in the usual manner by investigating the possibility of equilibrium in an undeflected position. In this case one must therefore investigate whether (20) has solutions when M = Pw (see fig. 6).

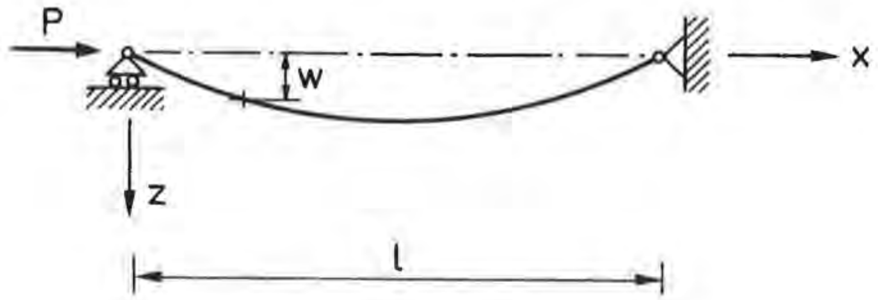


Fig. 6.

The result of the calculations, which will not be described here, is that

$$P_E = \frac{\pi^2 EI_e}{l^2} \qquad (31)$$

where I_e is the effective moment of inertia defined in equation (27). The result is immediately apparent from Section 2.3, bearing in mind that the Euler investigation results in sinusoidal deflection curves.

3.2 Practical columns.

As in the case of solid columns the Euler load represents an upper limit which is approached asymptotically for slender columns.

For shorter columns it is normally assumed - by analogy with (31) - that the load-carrying capacity of built-up columns can be determined from the expressions valid for solid columns as long as the effective slenderness ratio λ_e is used:

$$\lambda_e = \frac{\ell}{i_e} \qquad (32)$$

where i_e is the effective radius of gyration

$$i_e = \sqrt{\frac{I_e}{A}} \qquad (33)$$

and A is the total area

$$A = \sum_{i=1}^{N} A_i \qquad (34)$$

This assumption may seem immediately obvious, but it is not. The initial eccentricities and curvatures can be different from those of solid columns, and the stress conditions are markedly different.

To evaluate these conditions the load-carrying capacity will be determined for a column with a sinusoidal initial curvature with the amplitude e at the centre (refer to Section 2.2. in [10]). The column force is

$$P = s_{cr} A$$

where s_{cr} is the ultimate stress in the column.

As

$$k_E = \frac{\pi^2 E}{\lambda_e^2 s_c} \qquad (36)$$

where s_c is the compression strength, the original amplitude will be increased to

$$u = e \frac{k_E}{k_E - \frac{s_{cr}}{s_c}} \qquad (37)$$

as will be seen from (4) in [10].

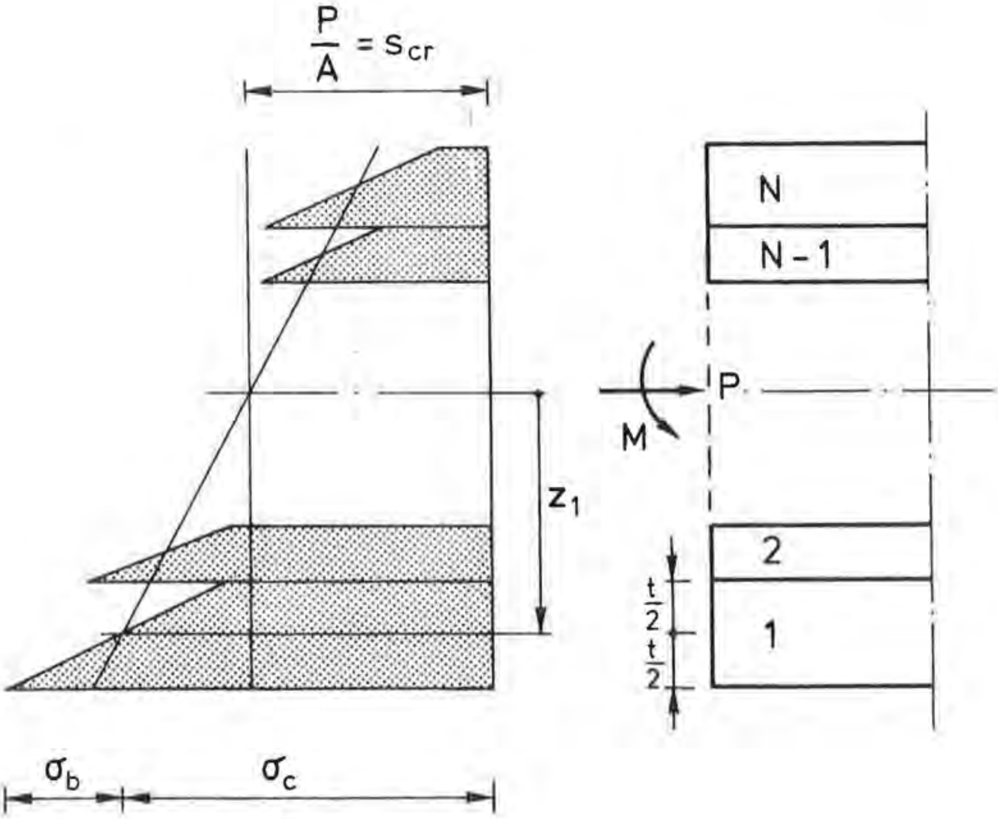


Fig. 7.

The axial stress at the centre of the outer lamella in the compression side (no. 1 in fig. 7) from P alone is s_{cr}, and the axial stress at the same point from the moment Pu is found by introducing $z = z_1$ into (29), i.e. the total axial stress is

$$\sigma_c = s_{cr} + \frac{Pu}{I_e} z_1 \left(1 - \frac{1 - I_e/I}{1 - \beta^2}\right)$$

The bending stress σ_b in the outer fibre according to (29) will be

$$\sigma_b = \frac{Pu}{I_e} \cdot \frac{t}{2}$$

where t is the thickness of the lamella.

From the ultimate condition (7) in [10]

$$\frac{s_{cr}}{s_c} \left[1 + e \frac{s_c}{s_b} \frac{A}{I_e} \left(z_1 \left(1 - \frac{1 - I_e/I}{1 - \beta^2}\right) \frac{s_b}{s_c} + \frac{t}{2}\right) \frac{k_E}{k_E - \frac{s_{cr}}{s_c}}\right] = 1 \qquad (38)$$

is found.

For a solid column $\frac{s_{cr}}{s_c}$ is determined by (11) in [10]. In this formula $\beta = \frac{s_c}{s_b}$, and assuming that $\Psi = 1$ and $\varepsilon = \frac{e}{k} = \frac{eA}{W} = \frac{eA(z_1 + \frac{t}{2})}{I}$, the following is found:

$$\frac{s_{cr}}{s_c}\left[1 + e\,\frac{s_c}{s_b}\,\frac{A}{I}(z_1 + \frac{t}{2})\,\frac{k_E}{k_E - \frac{s_{cr}}{s_c}}\right] = 1 \;. \qquad (39)$$

It can thus be seen that is is not enough to replace I with I_e for the built-up column, but that in addition e must be replaced with

$$e\,\frac{\frac{t}{2} + z_1\left(1 - \frac{1 - \frac{I_e}{I}}{1 - \beta^2}\right)\frac{s_b}{s_c}}{\frac{t}{2} + z_1} \qquad (40)$$

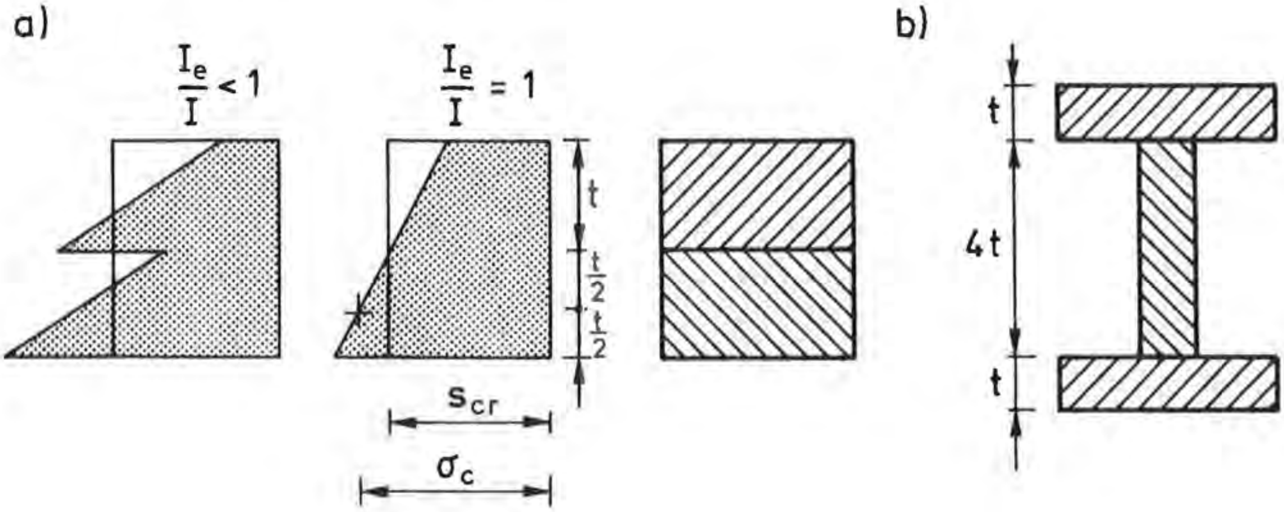


Fig. 8.

Assuming that $\frac{s_c}{s_b} = 0.75$, the factor for e for the cross-section in fig. 8a, where $z_1 = \frac{t}{2}$ and $\beta^2 = 0.25$, will then lie between 0.30 (for $I_e = 0$) and 1.17 (for $I_e = I$). For the cross-section in fig. 8b where $z_1 = 2.5t$ and $\beta^2 \sim 0.10$, the factor lies correspondingly between 0.10 and 1.28. For $I_e = I$, i.e. a rigid connected cross-section, one would immediately expect the value 1. That this is not the case is due to the fact that σ_c has here been compared with the low strength s_c (see fig. 8), while in the case of the solid cross-section only s_{cr} has been compared with s_c.

The factor will therefore always lie below 1, and for the cases found in practice around 0.5 approximately.

With respect to e one will presumably get less curvature seen from the outside, but on the other hand the applied load will probably be more eccentric. It is difficult to transfer the loads evenly over the individual parts, in addition to which the individual lamellas will normally have varying E, i.e. the static and geometric centres of gravity do not coincide.

As the column expressions are not sensitive to minor variations in e (see fig. 2 in [10]), it seems reasonable to use the usual column expression just with λ_e.

3.3 Loads on the connections.

When the column's deflection curve is known the shear force at any point is determined. The following is found from fig. 9:

$$Q \sim Pw' = As_{cr}w' \qquad (41)$$

The deflection curve being sinusoidal, and as the initial deflection at the centre is e, the following is found from (37):

$$w' = e \frac{k_E}{k_E - \frac{s_{cr}}{s_c}} \cos \frac{\pi x}{l} \qquad (42)$$

Q reaches its largest value at the ends, and when Q is set in

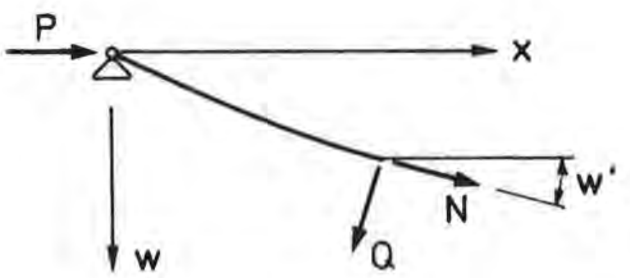


Fig.9

relation to the compression strength without column effect, i.e. $s_c A$,

$$\frac{Q}{s_c A} = e \frac{k_E}{k_E - \frac{s_{cr}}{s_c}} \cdot \frac{\pi}{l} \frac{s_{cr}}{s_c} \tag{43}$$

is found.

As reasonable values $e = 0.005\ \lambda$ (formula (3) in [10]) and $i \sim 30\%$ of the height of the cross-section, have been taken. $\frac{s_{cr}}{s_c}$ is determined from (12) in [10] with $\frac{E}{s_c} \sim 300$. Fig. 10 shows then $\frac{Q}{s_c A}$ dependent on λ_e.

It is seen that the largest shear force occurs for $\lambda_e \sim 75$ and is approximately 1.5% of $s_c A$.

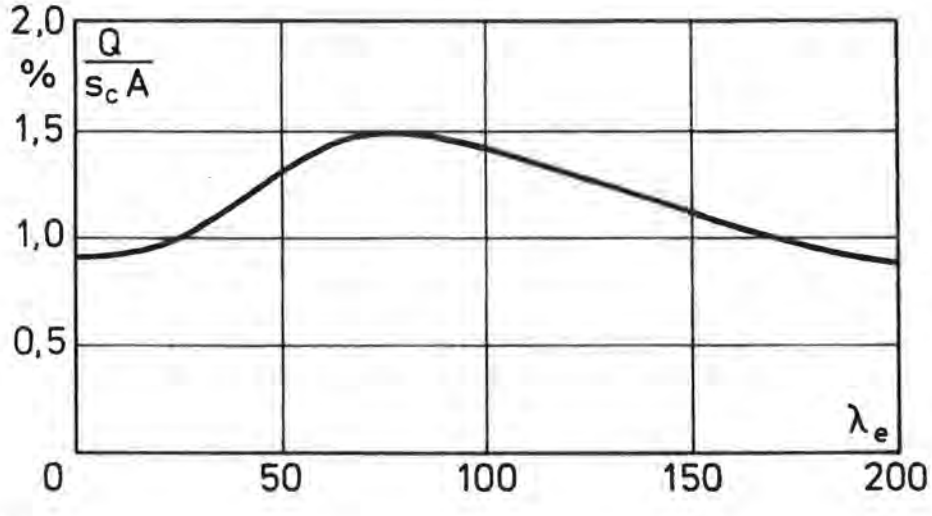


Fig.10

4. DESIGN OF VARIOUS TYPES OF COLUMNS

4.0 General

Referring to previous sections built-up columns are designed as solid columns where, instead of the total geometric moment of inertia I, the effective moment of inertia I_e is taken.

$$\left.\begin{aligned} I_e &= I_o + \gamma(I - I_o) = \frac{1 + \beta^2\mu}{1 + \mu} I && (27) \\ \gamma &= \frac{1}{1 + \mu} && (28) \\ \mu &= \frac{\pi^2 A^* E}{(N-1)l^2} \cdot \frac{a}{k} && (26) \end{aligned}\right\} = (44)$$

The connections are characterized by their distance a and their stiffness k.

1/k is the displacement which occurs when the connection must carry the force 1 through the joint. a/k, which is denoted the compliance, is the displacement when the force 1 is transferred per unit length.

In the following a/k will be determined for a number of column types. It should be noted that it is necessary to distinguish between the total compliance in relation to the transfer of force between the individual parts, (this is denoted by a/k), and the compliance in a single joint. Moreover, it is necessary to distinguish between the compliance of the entire connection in a joint and the individual connections.

4.1 Continuously jointed cross-section.

Typical cross-sections are given in fig. 1 and fig. 4, types 1-5.

Provided the shear strain in the individual parts is neglected, these columns can be designed directly from (44) when the stiffness numbers of the connections are known.

For I and box-formed cross-sections with thin webs it may be necessary to take the shear deformation of the web into account.

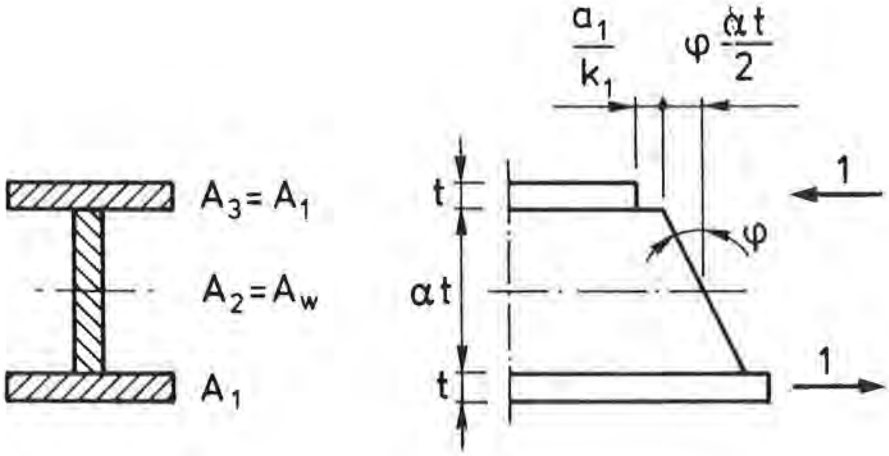


Fig. 11.

The compliance of the joint is called a_1/k_1, while the shear strain in the web gives

$$\varphi \frac{\alpha t}{2} = \frac{\alpha t}{2Gt_w}$$

where t_w is the area of the web and G is the shear modulus, see fig. 11.

Thus is found

$$\frac{a}{k} = \frac{a_1}{k_1} + \frac{\alpha t}{2Gt_w} \qquad (45)$$

4.2 Spaced columns with packs.

Columns with cross-sections as shown in fig. 4, types 6-8, are considered. First a column with two identical lamellas is investigated, (refer to fig. 12), where various notations are given.

As mentioned in connection with fig. 4, the cross-section must be considered as consisting of three lamellas where the area of the middle lamella is zero. It should be noted that one of the assumptions made - that a is small - is not particularly well satisfied in this case.

a/k is determined as the displacement between an outer lamella and the fictitious middle lamella, when each pack has to transfer the force $1 \cdot l_1$.

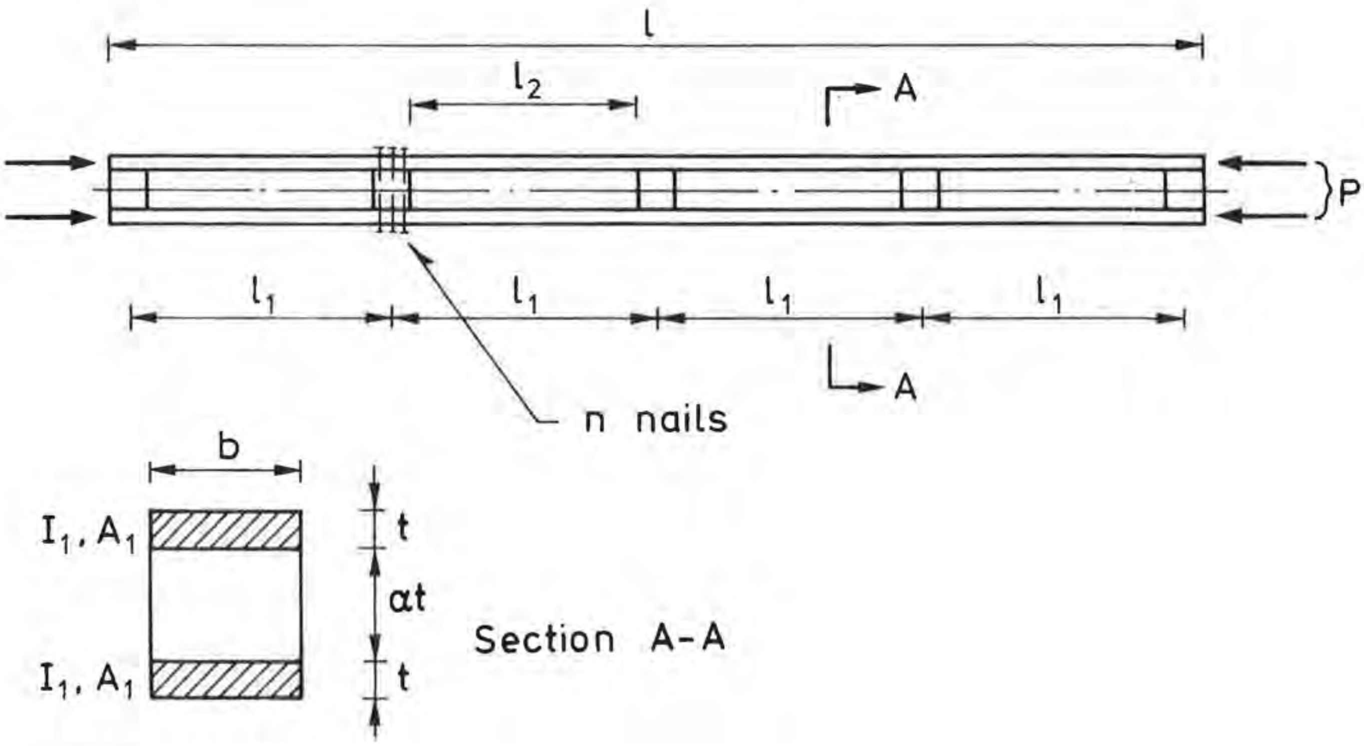


Fig. 12.

The stiffness number for the connection between the pack and the flange is denoted by K. The contribution from this to the displacement becomes l_1/K. Should the connection for example consist of n nails, each with the stiffness number k_1, then $K = nk_1$.

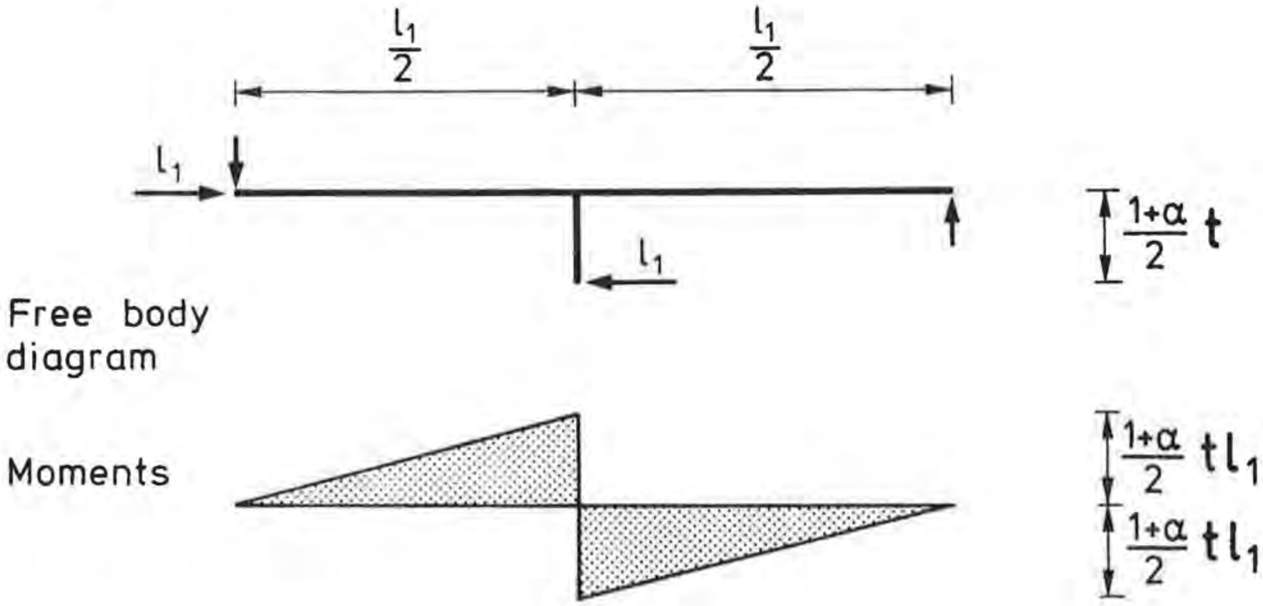


Fig. 13.

The displacement due to the deformation of the lamellas is found from the diagram of moments in fig. 13, where as usual the value of the moment in the middle of the spans is taken as zero. By applying the principle of virtual work

$$\frac{l_1}{3EI_1} \cdot \frac{1+\alpha}{4} t l_1 \cdot \frac{1+\alpha}{4} t$$

is found.

$\frac{a}{k} \hat{=} \frac{e'}{c}$

As a whole is found

$$\frac{a}{k} = \frac{l_1}{K} + \frac{(1+\alpha)^2 t^2 l_1^2}{48EI_1} \tag{46}$$

Provided the packs are glued, $l_1/K = 0$, and if in this case only the deformations on the distance l_2 between the packs are taken into account, then

$$\frac{a}{k} = \frac{(1+\alpha)^2 t^2 l_2^3}{48EI_1 l_1} \tag{47}$$

is found.

For cross-sections with more than two lamellas the above-mentioned values are often used. A slightly more accurate expression, (see [15]), similar to (46) is

$$\frac{a}{k} = \frac{l_1}{K} + \frac{(N-1)(1-\beta^2)l_1}{12EA^*\beta^2} \tag{48}$$

4.3 Spaced columns with battens.

For spaced columns with battens (fig. 14) instead of packs, the stiffness will be further reduced, partly due to the deformation of the battens and partly to the possible rotation of the connection of the battens.

The bending stiffness (corresponding to the rotation in the direction of k_M, see fig. 14) of the batten is characterized by EI_W, and the stiffness of the connection by the moment k_M that will give an angular rotation of 1 in the joint. If there are more battens in each place the total stiffness is taken into account.

Analogous to (46), taking into account the shear deformation

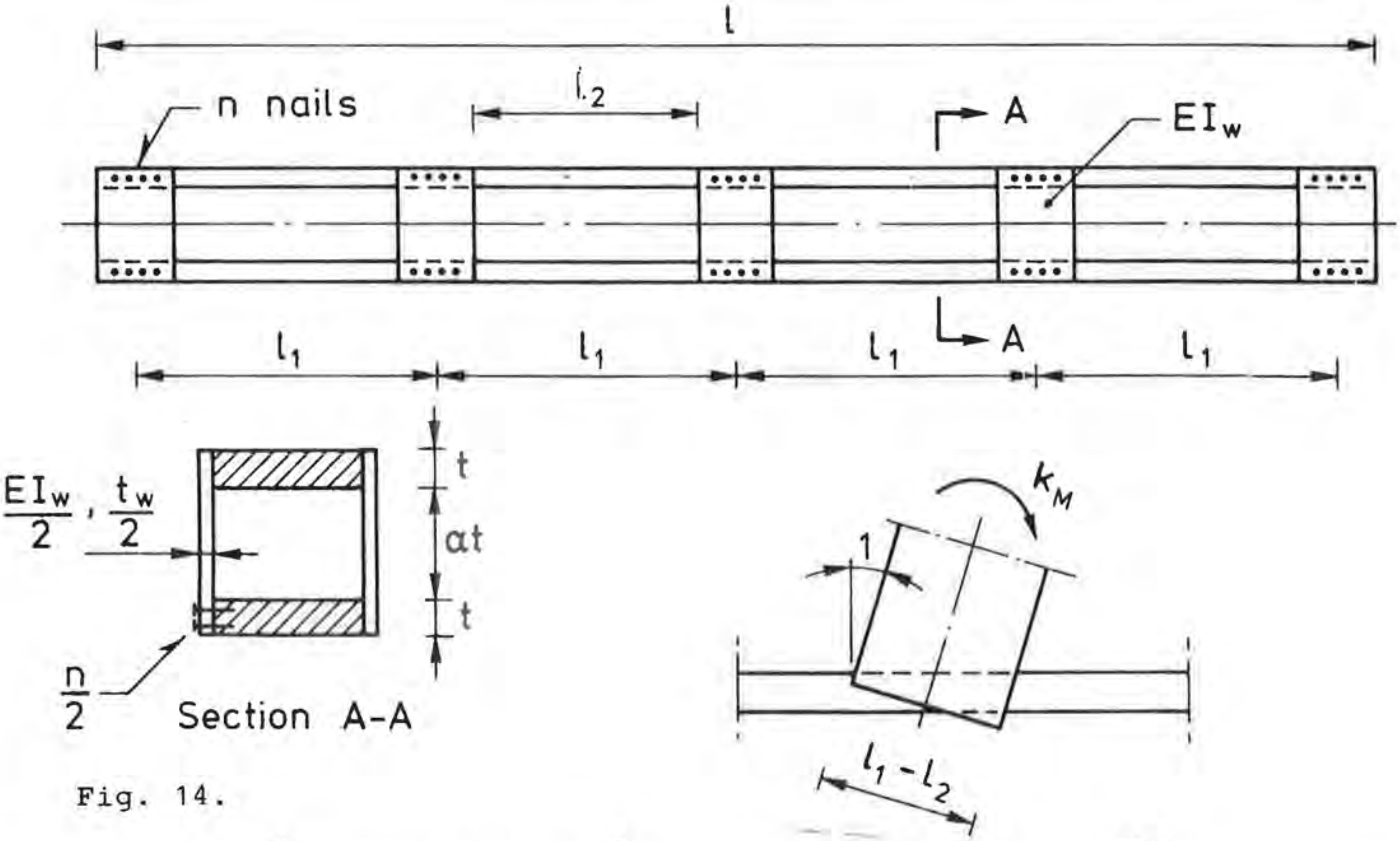


Fig. 14.

$$\frac{a}{k} = \frac{l_1}{K} + (1+\alpha)^2 t^2 l_1 \left[\frac{l_1}{48EI_1} + \frac{(1+\alpha)t}{24EI_w} + \frac{1}{4k_M}\right] + \frac{atl_1}{2Gt_w(l_1-l_2)} \quad (49)$$

is found (refer to Section 4.1).

4.4 Lattice-columns.

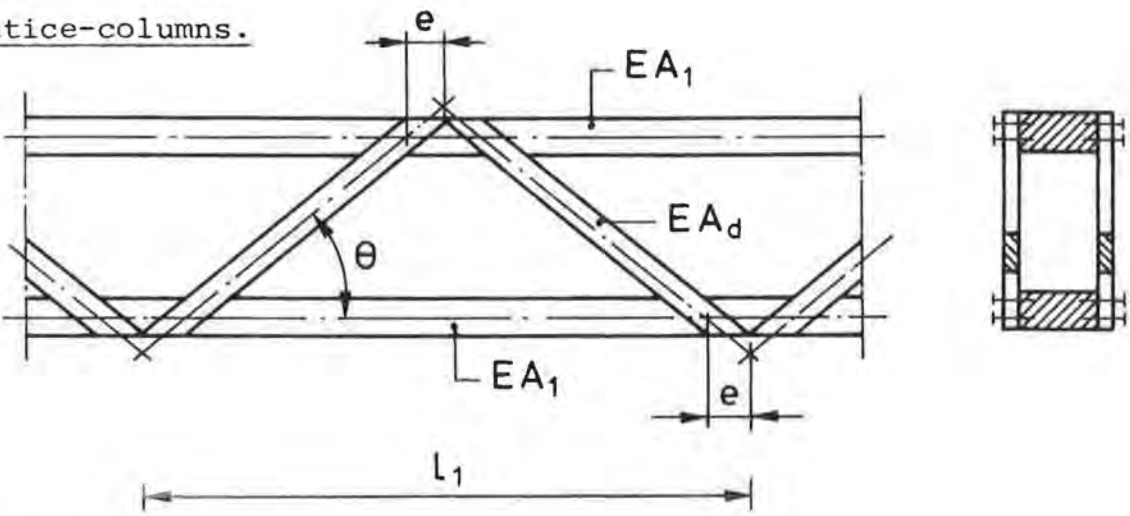


Fig. 15.

A column with V-lattice is considered, and the following new notations are used (refer to fig. 15):

- A_d: The area of a diagonal. A diagonal may consist of more parts; A_d is then the total area.
- k_d: The stiffness of a diagonal connection. Where the joint is made with n nails, $k_d = nk_1$.

e : Possible eccentricity at the intersection of diagonals.

θ : The slope of the lattice.

It is found that

$$\frac{a}{k} = \frac{l_1}{2 \cdot \cos^2 \theta} \left[\frac{l_1}{4EA_d \cos\theta} + \frac{1}{k_d} + \frac{l_1 e^2}{6EI_1} \left(1 - \frac{2e}{l_1}\right)^2 \sin^2 \theta \right] \qquad (50)$$

Note that this is still a case of a cross-section with N = 3.

Similar expressions can be established for other types of lattices.

5. TESTS

5.0 General

Quite a large number of reports exist concerning tests with built-up columns. However, in most cases the authors have been so pleased at finding a new empiric formula which covered their modest experimental results that they have not found it necessary to give the requisite data for an actual evaluation of the tests. Exceptions are the tests performed by Egner and Möhler [6],[7], [12] and [13], Granholm [8], Niskanen [15] and Nijenhuis [14], which will be used in the following.

Glued columns will be treated first, following by nailed columns, and finally columns having other types of connections.

For the individual types the effective moment of inertia I_e, according to formula (44), the effective slenderness ratio λ_e and the Euler Load-carrying capacity P_E^c (c = calculated) are calculated. This last value is compared with the load-carrying capacity determined in the tests P_{cr}^m (m = measured). The relationship P_{cr}^m/P_E^c versus λ_e is plotted in diagrams where also the theoretical (expected) relationship corresponding to various initial deflections is shown.

The theoretical relationship is determined from (12) in [10], with $\beta = 0$ and $\varepsilon = 0.001\ \lambda_e$, $\varepsilon = 0.002\ \lambda_e$ and $\varepsilon = 0.005\ \lambda_e$, i.e. in accordance with current practice in the United Kingdom.

In a number of cases an introductory determination of the bending stiffness is made. In these cases a comparison of the measured effective bending moment of inertia I_{eb}^m and the calculated I_{eb}^c *) is made.

In all cases the same test enumeration is used as that in the original experiments.

----

*) For a load in the middle

$$\frac{I}{I_{eb}} = 1 + \frac{12(1-\beta^2)}{\pi^2}\left(1 - \frac{\tanh(\pi/(2\beta\sqrt{\mu}))}{\pi/(2\beta\sqrt{\mu})}\right)$$

is found (see,for example, [8]).

5.1 Glued columns.

5.1.1 Glued I and box-columns.

Glued columns with cross-sections as shown in fig. 16 were investigated in [7]. The length was 7.00 metres (column length 7.30 metres); in the case of the cross-section with plywood webs the length 3.00 metres (3.30 metres) was also investigated.

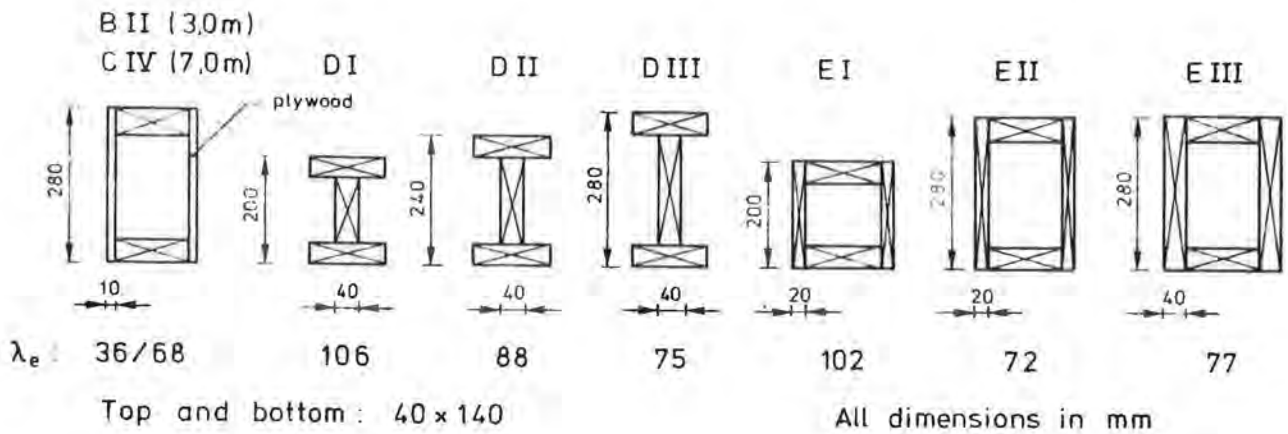


Fig. 16.

The ratio I^m_{eb}/I^c_{eb} between the measured and calculated effective moments of inertia corresponding to bending varied from 1.00 to 1.05 for all the 7 metre columns, while it was 0.83 for the short column (B II). It should be noted that information on the plywood is lacking, and the calculations are therefore carried out as though the plywood were of ordinary timber with E/G $\sim$ 20.

The ratio I_{eb}/I was about 0.60 for B II (plywood, 3.0 metres), 0,90 for C IV (plywood, 7.0 metres), and further about 1.00. It is thus seen that it is only necessary to take into account the influence of the shear deformation in short columns with very thin webs.

The ratio P^m_{cr}/P^c_E between measured column -load and calculated Euler load is given in fig. 17. The agreement with the expected curves is satisfactory in all cases.

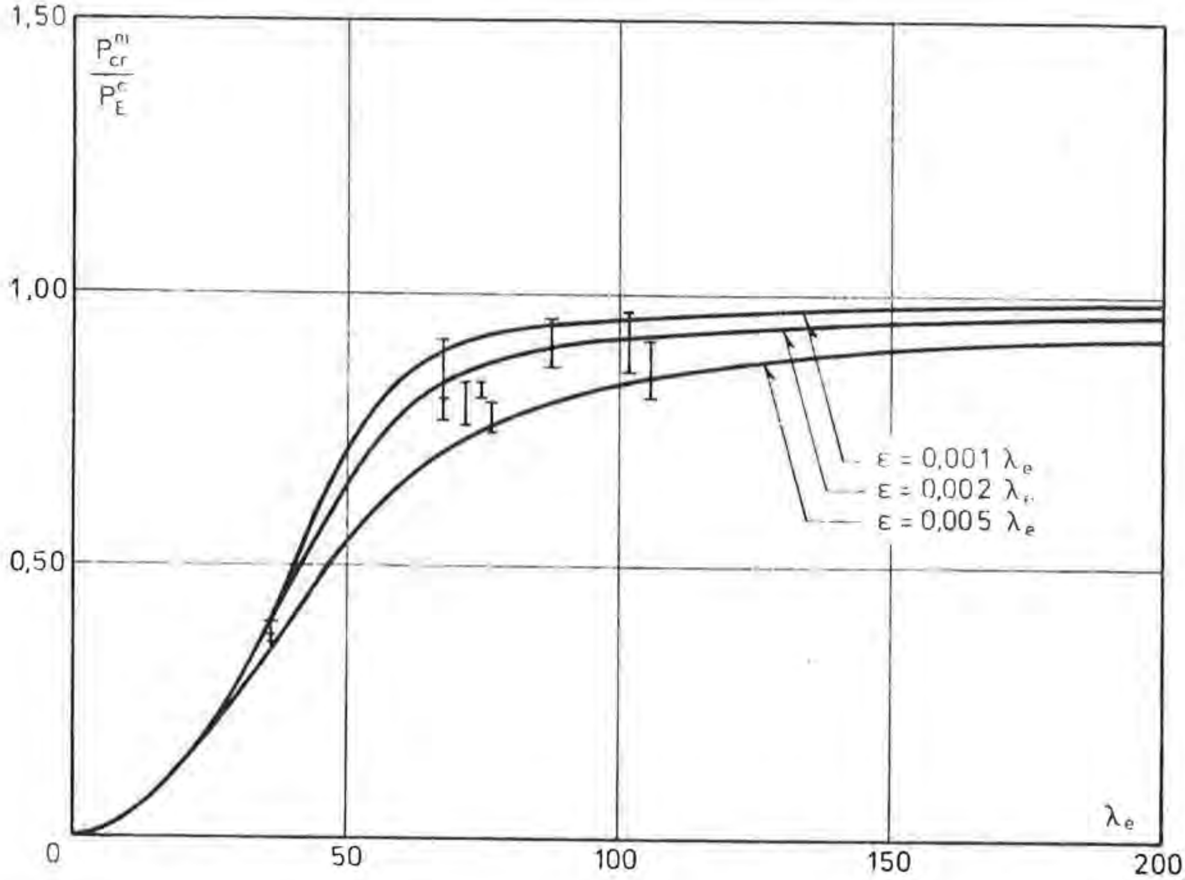


Fig. 17.

5.1.2 Spaced columns with glued packs.

65 bending and column tests are described in [6] as well as a further 10 bending tests with columns of this type. The parameters are given in Table 1. Fig. 12 may be referred to regarding notation. The column length was in all cases 0.30 metres greater than the length of the structure. The column width was 3.5t. In all cases the calculations were performed using formula (46) as this proves to give substantially better results than (47), with the exception of the case with very long packs.

For the 3 metre beams I_e^m/I_e^c is 1.02 on average excluding no. XII which inexplicably falls outside the complete picture. Including XII the average is found to be 1.08. For the 7 metre beams I_e^m/I_e^c is 1.18 on average.

TABLE 1

Identification	No. of	l m	t mm	α	l_1 mm	l_1-l_2 mm	Bending I_e^m/I %	Bending I_e^m/I_e^c %	Axial Compression I_e^m/I %	Axial Compression λ_e	Axial Compression P^m_{cr}/P^c_E %
I				0,5	950	50	67	92	69	247	104
II				1	940	60	53	91	55	215	100
III			20	2	920	80	32	96	35	183	97
IV				3	900	100	22	96	24	168	95
V				4	880	120	16	109	17	150	89
VI				5	860	140	12	122	13	152	101
VII				1	470	60	80	98	82	175	100
VIII				3	450	100	54	88	57	108	99
IX	3	5,00		5	430	140	33	119	36	91	107
X				1	880	120	56	99	58	104	95
XI				3	800	200	26	130	28	77	96
XII				5	720	280	16	203	17	56	90
XIII				0,5	450	100	89	94	90	108	91
XIV				1	440	120	82	89	84	87	80
XV			40	2	420	160	68	101	71	64	73
XVI				3	400	200	57	116	60	53	60
XVIIa	2			3	2130	200	22	104	25	188	83
XIX	5			5	2050	280	12	127	13		
XXII	3			2	1090	160	64	94	66	147	95
XXIII	4			3	1070	200	50	100	53	124	101
XXIV	5			4	1050	240	40	111	43	111	104
XXV				5	1030	280	32	117	35	103	102
XXVII				2	2010	320	36	111	38	95	82
XXVIIA	3	7,00	80	3	1930	400	25	132	27		
XVIII				4	1850	480	18	157	20		
XXX				2	1010	320	67	103	70	72	70
XXXA				3	970	400	55	120	68		
XXXI				4	930	480	45	145	49		

I_e/I decreases with increasing α but I_e however increases. For α > 4 the increase is very modest.

The effective moment of inertia for columns is a little greater than in the case of bending, which is due to the fact that the shear force conditions are substantially better.

The ratio P^m_{cr}/P^c_E is plotted in fig. 18 for each tests.

There is seen to be good agreement between the found and expected values for P^m_{cr}/P^c_E. For the slender columns (λ_e > app. 100) P^m_{cr}/P^c_E is on average 0.98.

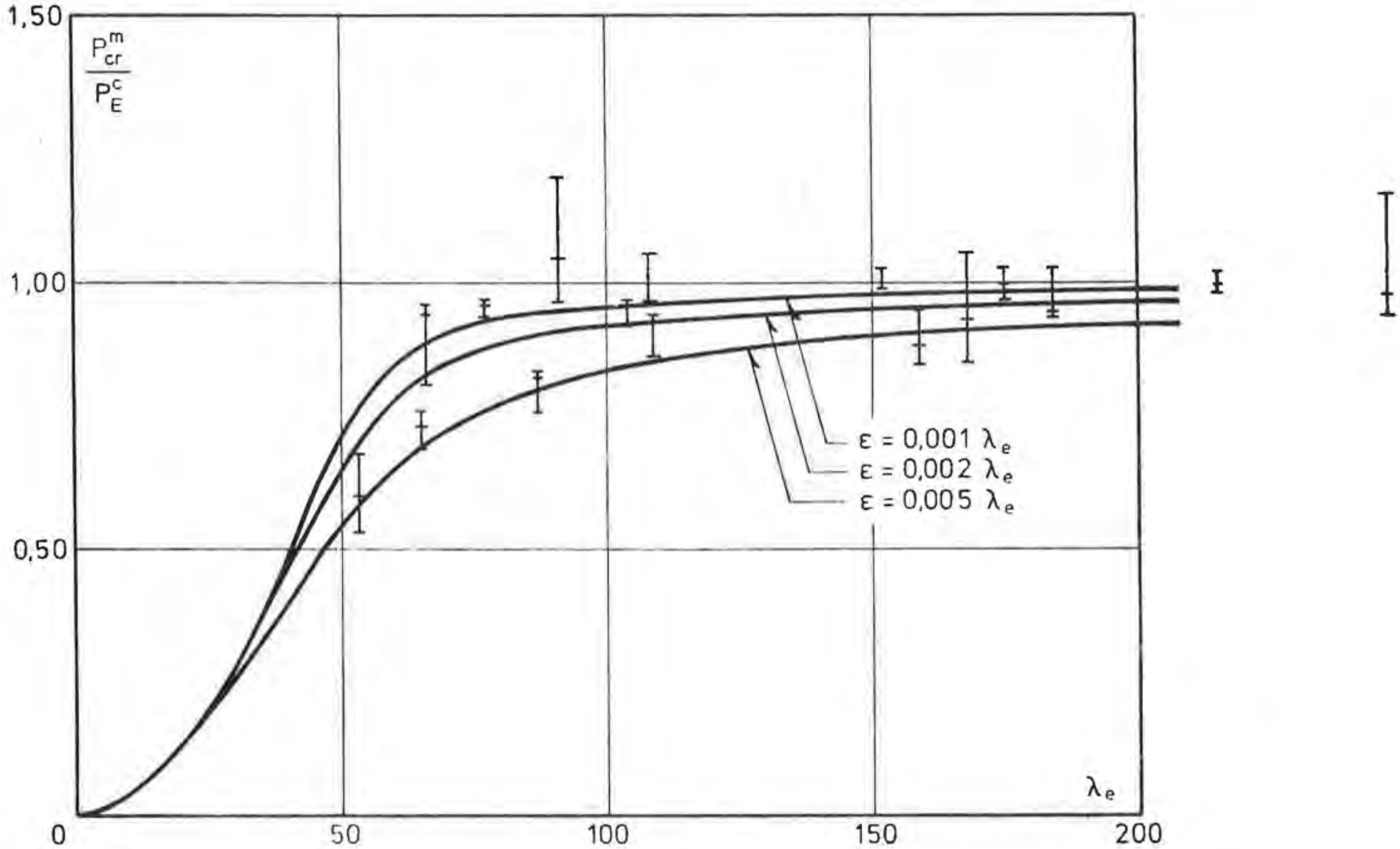


Fig. 18.

5.1.3 Spaced columns with glued battens.

This type is investigated by Egner in [7] and by Möhler in [12]. Egner's experiments are shown in Table 2a. Fig. 14 should be referred to regarding the notation. Each series consisted of three separate tests.

TABLE 2a

Identification	l m	t mm	n	l_1 mm	l_2-l_1 mm	t_w mm	Contribution to $\frac{a}{k}$ in pct. bending shafts	bending battens	shear battens	Bending I_e/I %	Bending I^m_e/I^c_e %	Axial Compression I^c_e/I	λ_e	P^m_{cr}/P^c_E
A I			1	470	60		65	3	32	73	83	75	183	103
A II	3,00	20	3	450	100	2x10	80	1	19	46	87	49	117	106
A III			5	390	140		86	1	13	36	82	40	87	97
B I			5	360	280		60	3	37	30	107	33	48	54
C I			3	1070	200		70	2	28	42	91	44	135	100
C II	7,00	40	5	1030	280		81	1	18	28	99	30	110	103
C III			5	1030	280	2x20	89	1	10	30	96	33	106	104

The battens were made of 10 mm plywood, with the exception of C III where 20 mm plywood was used. No information about the plywood is given. In the following it is assumed that E/G = 20 and that there is an effective thickness in bending of $t_e = 0.6t_w$, while the full thickness is assumed in shear.

a/k is calculated from (49) with $1/K = 0$ and $1/k_M = 0$. Table 2a shows the contribution to a/K from the bending of the flanges (the term containing I_1), the bending of the battens (the term I_w) and the shear of the battens (the term t_w). It can be seen that the bending of the flanges is dominant but that the contribution from the shear in the battens is not to be ignored, whereas in all cases the contribution from the bending of the battens is negligible.

The measured effective moments of inertia in bending are a little lower than theoretically expected (the average for I_e^m/I_e^c is 0.92), but in the column tests there is very satisfactory agreement between the measured and theoretical values, (see Table 2 and fig. 19). For the slender columns (λ_e > app. 100) P_{cr}^m/P_E^c is on average 1.02.

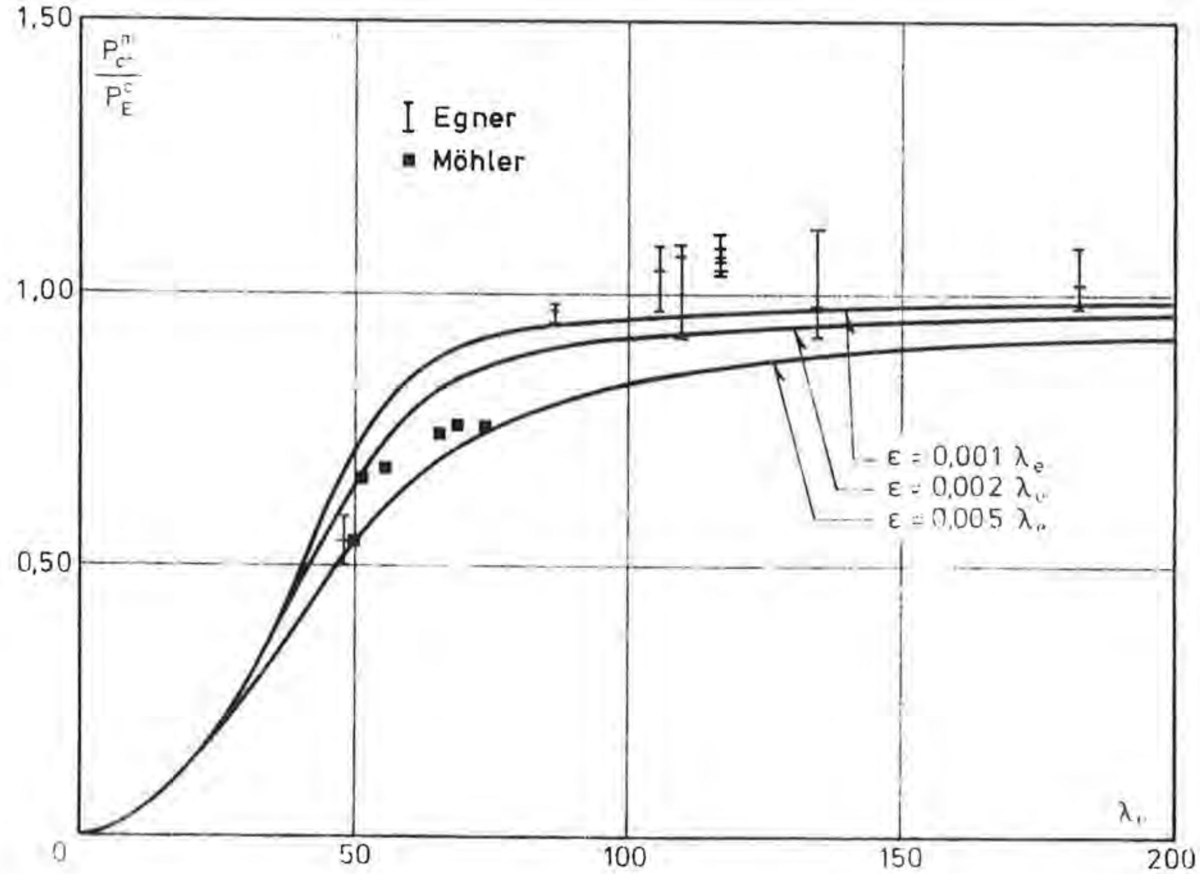


Fig. 19.

Möhler's experiment is shown in Table 2b which contains the same information as Table 2a. However, it should be noted that no bending experiments were carried out. Each series consists of only one test.

TABLE 2b

Ident.	l m	t mm	α	l_1 mm	l_2-l_1 mm	t_w	Contrib. to $\frac{a}{k}$, pct. bend. flange	bend. batt.	shear batt.	Axial comp. I_e^c/I %	λ_e	P_{cr}^m/P_E^c %
3 1	[illegible]	20	3	400	150	2x20	79	1	20	35	56	68
3'1	[illegible]	↓	3	550	↓	↓	84	1	15	34	74	75
4 1	[illegible]	↓	4	400	↓	↓	81	1	18	26	52	66
4'1	[illegible]	↓	4	550	↓	↓	86	1	13	25	69	76
5 1	[illegible]	↓	5	400	↓	↓	83	1	16	20	50	54
5'1	[illegible]	↓	5	550	↓	↓	87	1	12	19	66	74

The battens were ordinary boards for which E/G = 20 was assumed. It can be seen that the flange bending is dominant and that the shear must be considered.

The calculated ratio P_{cr}^m/P_E^c is also plotted in fig. 19.

There is seen to be completely satisfactory agreement with the expected values.

In his calculations of I_e and λ_e Möhler has assumed that the Euler formula is valid, which must be considered incorrect.

5.1.4 Lattice columns.

[7] describes four series with glued lattice columns with a length of 7 metres. The flanges were 40 x 140 mm and the lattices 2 at 20 x 80 mm. For F I, F II and F IV the lattices were identical in the two sides, while for F III they were displaced so that the flanges were held on one side per 750 mm. The data are further shown in Table 3 (refer to fig. 15 for notations).

Table 3 gives two sets of values, one corresponding to a case where the lattice is assumed to be joined to the flanges without eccentricity, and the other corresponding to the given eccentricity e. e is not given directly in the test report, but is estimated by sketching the various joints. In bending there is reasonable agreement between the calculated and measured stiffnesses provided

TABLE 3

Identification	α	l_1 m	θ^{o}	e mm	Bending I_e^c/I %	Bending I_e^m/I_e^c %	Axial compression μ	Axial compression I_e^c/I %	Axial compression λ_e	Axial compression P_{cr}^m/P_E^c %
FI	5	1.00	32	0	92	93	0,072	93	63	60
				4,0	86	100	0,139	88	65	63
FII	7	1.00	41	0	89	83	0,102	91	48	40
				3,5	80	91	0,203	83	50	44
FIII	7	1,50	28	0	85	88	0,144	88	49	33
				4,5	75	101	0,287	78	52	37
FIV	9	1,00	45	0	89	70	0,102	91	38	25
				3,5	78	80	0,235	81	40	28

the influence of the eccentricities is taken into account.

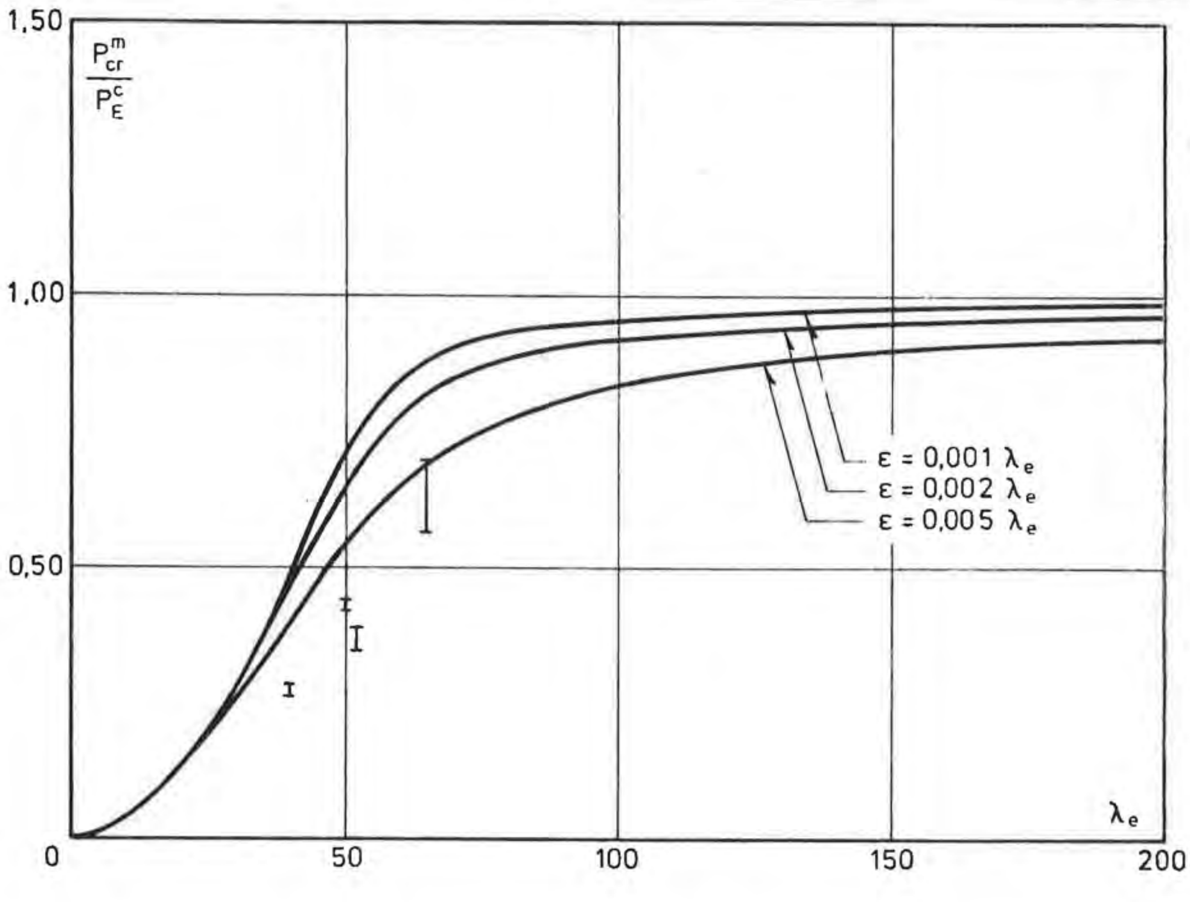


Fig. 20.

The agreement of the column test results is less satisfactory than in the above examples as the measured values in all cases are too low (refer to fig. 20 which only shows the cases where $e \neq 0$; if e is neglected the values are even lower). This is probably due to the fact that the slenderness ratio for the flanges is of the order 60-120 (corresponding to full rigidity and no rigidity at the joints respectively), i.e. higher than for the total column, so that local fracture in the flanges may have occurred.

5.1.5 Conclusion.

Fig. 21 gives a summary of all the described column tests, the average figure for P^m_{cr}/P^c_E being given for each series.

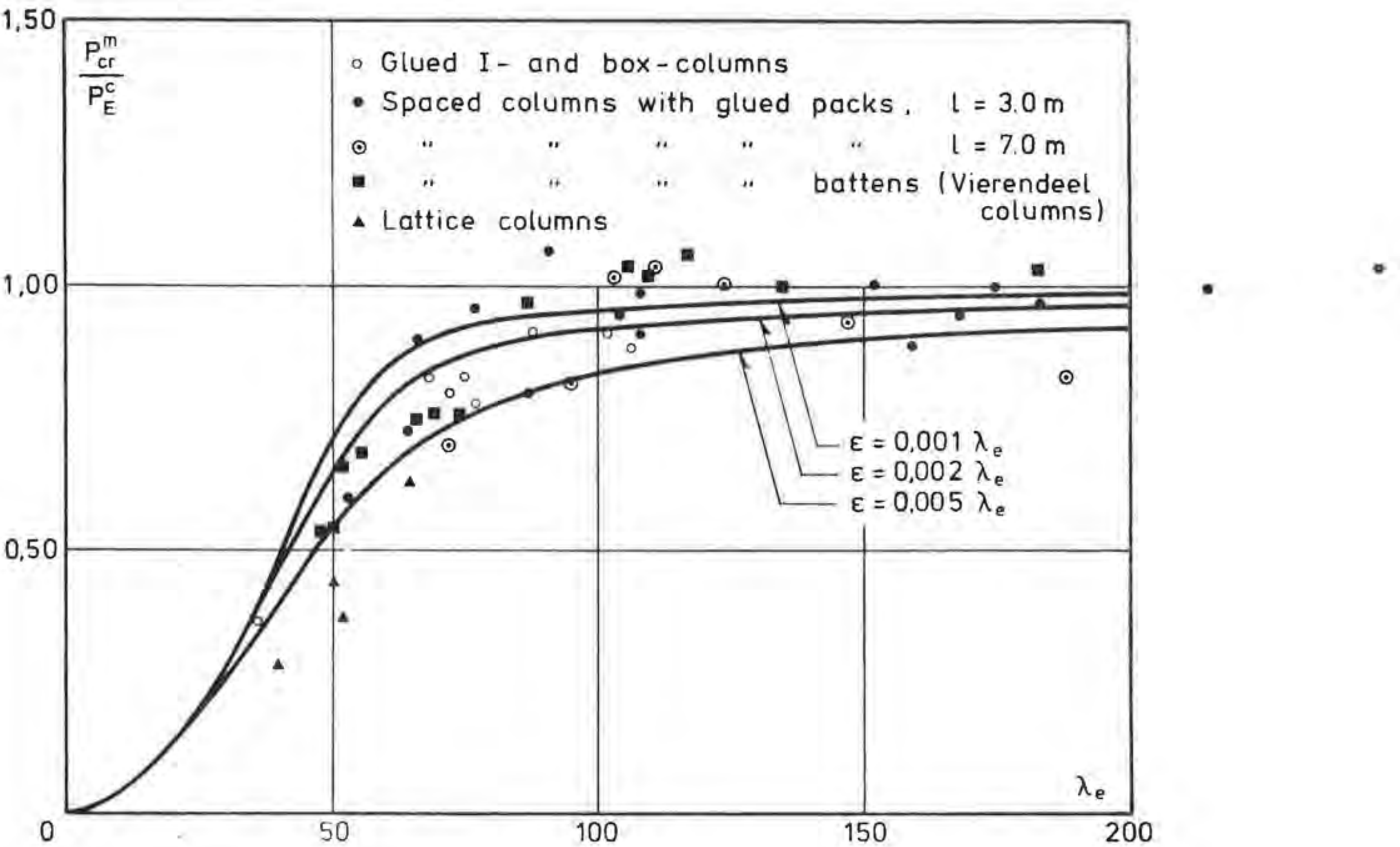


Fig. 21.

There is seen to be fully satisfactory agreement between the theory and the tests, with the exception that the lattice columns are rather low as previously explained.

5.2 Nailed columns.

5.2.0 General.

Under service load the slip in a nailed connection can reasonably be calculated by considering the nail as an elastic beam on an elastic foundation where the pressure p per unit length for the deformation u can be written as

$$p = K \cdot u$$

The slip in the joint will then, according to [16], be

$$S = k_S \frac{P}{\sqrt[4]{4(EI)_n K^3}} \tag{51}$$

where $(EI)_n$ is the bending stiffness of the nail, and k_s ($\gtrapprox 4$) is a parameter which depends on the thickness of the wood members in relation to the diameter of the nail. For nails the thickness of the timber will as a rule be so large that $k_s \sim 4$.

Norén, on the basis of experiments with connections, has shown that for Nordic whitewood under short-term loads it may be assumed that $K \sim 500$ N/mm^2.

For a circular nail with diameter d

$$k/d \sim 380 \text{ N/mm}^2 \tag{52}$$

is then found.

For a quadratic nail with side-length d, k will be about 15% greater.

The values correspond to tight and well-constructed connections. If there is a substantial gap between the parts, the slip can be considerably greater.

For long-term loads K is reduced; Norén gives a reduction down to about 45% of the value for short-term loads.

The values, which are valid for normal connections, can not immediately be used for built-up columns. As the stiffness of the connections is the only important-parameter which is not directly determined, all differences between theory and practice will show up in the value of k. This therefore becomes, to a certain extent, a formal parameter. The possibility of a reasonable determination of this parameter is therefore decisive in whether or not the presented theory can be considered reasonable.

5.2.1 Continuously jointed columns.

Möhler [13] and Niskanen [15] have carried out tests with continuously jointed columns with I cross-sections. Part of the test results is given in fig. 22 where k/d is used as an ordinate.

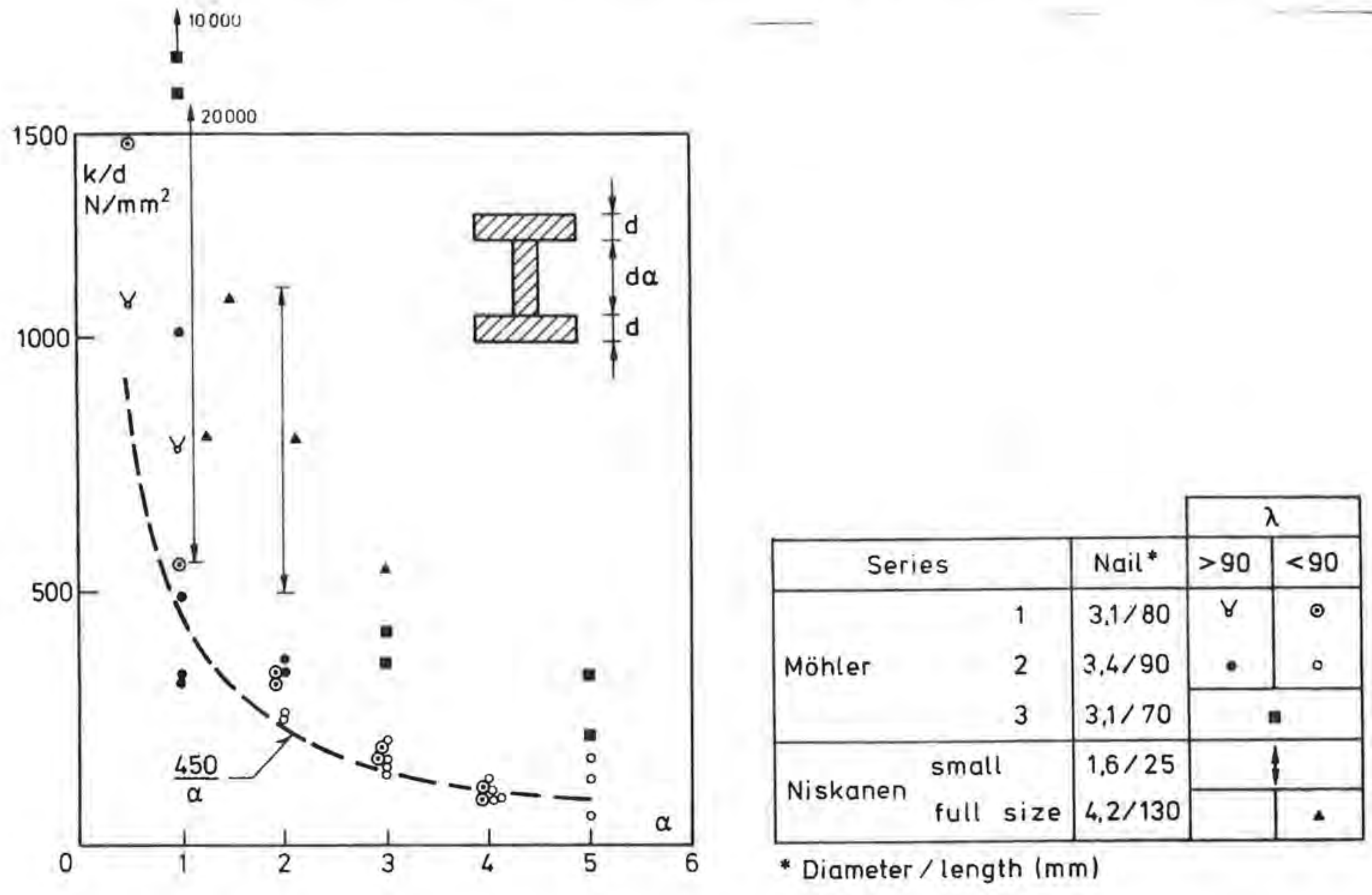


Fig. 22.

The purpose of Möhler's series 1 and 2 was to determine k's dependence on α (the spacing) and λ. A clear dependence on α corresponding approximately to $k/d = 450/\alpha$ was found from these experiments. No dependence on λ was found.

Möhler's series 3 was carried out to investigate the influence of the intensity of the nails. The highest intensity of the nails corresponded - as in series 1 and 2 - to a transverse force of app. 1.5% of the compression strength of the cross-section. Furthermore, the doubled and quadrupled distance was used for comparison.

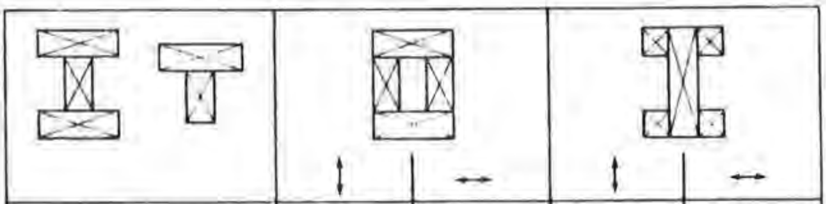
Only the values for the highest intensity of the nails i.e. values which are comparable with the values from series 1 and 2, are shown in fig. 22, but is is seen that they are considerably higher. With greater nail distances k/d was even greater. The use of more nails results in all cases in an increase in the load-carrying capacity, but the effect of the individual nail decreases.

The main part of Niskanen's experiments was carried out with small scale columns. For these experiments the graph only shows the interval for the found values. The value of model tests in timber is often limited because too high values are usually found. Full-scale experiments were carried out with $\alpha = 1, \frac{3}{2}$, 2 and 3. If one takes into account that a relatively large nail distance is used, Niskanen's results are approximately the same as Möhler's from series 3.

For $\alpha = 0.5$ and 1.0 quite large variations in the values of k are to be expected (I_e/I is of the order 0.8 and therefore not very sensitive to variations in k), but they are nevertheless unusually high even for timber experiments. The reason is probably that shrinkage and the working of the timber can open gaps which greatly reduce the stiffness of the connections. A substantially improved connection could probably be obtained by using, for example, ring nails. (Ring nails were used in a series of tests with box cross-sections carried out at the Virginia Polytechnic Institute [9]. Much higher values for k were found from these experiments).

Möhler has also investigated box columns similar to types 4 and 5 in fig. 4. The values for k/d did not differ fundamentally from those found for I-beams, and no unequivocal difference between types 4 and 5 was found.

On the basis of Möhler's tests the German and Dutch codes for timber structures show the values which are given in fig. 23.



600 (700)	600	900	900	600 (700)

Fig. 23. k-values in N/mm. The arrows show the direction of deflection. The values in brackets are valid where the nails are in double shear.

It should be noted that the results do not take into account the influence from the duration of load or the found dependence on d and a.

5.2.2 Spaced columns with nailed packs.

This section is based on tests carried out by Nijenhuis [14], Möhler [12] and Niskanen [15]. Tests with this type of column are also described in [11] and others, but in these cases so much important information on materials etc. is lacking that an evaluation of the results has not been possible. The tests are summarized in Table 4. λ is calculated from the geometry of the cross-section and the column length, while λ_e is based on I_e^m. I_e^m is calculated with the help of the Euler formula from the measured failure load and E values. An exception is the short columns in one Möhler series. Here it is assumed that $P_{cr}/P_E \sim 0.8$ (see note 2 in Table 4).

The table shows how large a part of μ (and thereby a/k) is caused by bending of the flanges (μ_b) and how large a part is due to the slip in the nailed connections (μ_n) - see (46). μ is defined in (44). In addition to the k values, k/d is also given.

In the Dutch tests each line represents three individual tests, one with high and one with low E modulus. The Möhler tests, marked 3n, 3n' .... are individual experiments while the others are average figures from two tests. The Niskanen results are average figures from three tests.

TABLE 4

	Identi-fication	σ	no. of packs	λ	λ_e	I_e^m/I %	μ_b %	μ_n %	k N/mm	k/d N/mm^2		Notes
Nijenhuis	z.1.4	1	4	142	191	50	66	34	3010	793	512	①
	z.1.5	"	5	"	184	53	48	52	1980	522		
	z.1.8	"	8	"	162	57	17	83	1190	314		
	z.2.4	2	4	97	171	28	67	33	1980	521	426	*
	z.2.5	"	5	"	150	35	51	49	1410	371		
	z.2.8	"	8	"	135	45	28	72	1470	386		
	z.3.4	3	4	73	167	17	65	35	1120	294	271	*
	z.3.5	"	5	"	156	20	48	52	900	238		
	z.3.8	"	8	"	121	30	28	72	1060	280		
Möhler	3n	3	5	38	67	32	60	40	10400	3000	2020	②
	3'n	"	"	52	100	27	71	19	3550	1040		
	4n	4	"	29	70	18	59	41	1450	426	479	* ②
	4'n	"	"	43	98	„20	72	28	2170	638		
	4"n	"	"	40	99	16	56	44	1270	374		
	5n	5	"	26	70	13	62	38	1050	309	432	* ②
	5'n	"	"	38	95	16	81	19	1890	556		
Möhler	1	1	5	75	121	38	40	60	2590		433	*
	2	2	"	51	104	24	46	54	2150		390	*
	3	3	"	39	98	15	51	49	1560		286	*
	4	4	"	31	114	07	30	70	660		120	
Niskanen		1	4	78	144	29	35	65	6990		1660	
		1,5	"	63	117	29	51	49	8640		2060	
		2	"	53	117	20	45	55	6080		1450	
		3	"	40	114	12	43	57	4520		1080	

① 2 Tests only. z1..4.1 excluded, see text.
② I_e calculated on the assumption that $P_{cr}/P_E \sim 0,8$

In the German and Dutch experiments nailing corresponding to a transverse force of approximately 1.5% of the cross-section's compression strength was used, while in Niskanen's tests nails with only 1/3 of the above value were used. This may be part of the explanation why Niskanen's results are considerably higher than the others. It should be noted that the values in Table 4 are higher than those given by Niskanen, which is due to the fact that he used an expression similar to (47), i.e. assuming the influence of the flange bending to be considerably less than here where (46) is used. Similarly, this partly applies to the Dutch tests.

There is generally good agreement between the Dutch and German experiments. There is a suggestion that k/d decreases with α.

Möhler's series 4 is a little outside the general picture, but as there were only two tests it is impossible to determine whether this is accidental. In this case the test results are a little surprising since the absolute value of I_e is reduced by increasing α from 3 to 4.

Based on the tests indicated with * in Table 4 the average figure

$$k/d = 400 \text{ N/mm}^2$$

is found. This value corresponds to the theoretical value found in (52).

As well as the tests referred to in Table 4 Niskanen has carried out a large number of experiments with small-scale columns (scale app. 1/2.5). The results of these tests are summarized in Fig. 24.

Owing to the large scattering in the results it is difficult to draw essential conclusions. There is however no suggestion of an α-dependency, but perhaps a slight tendency for k to increase with larger λ values.

The results are high, partly because a very light nailing was used and partly because of the use of models.

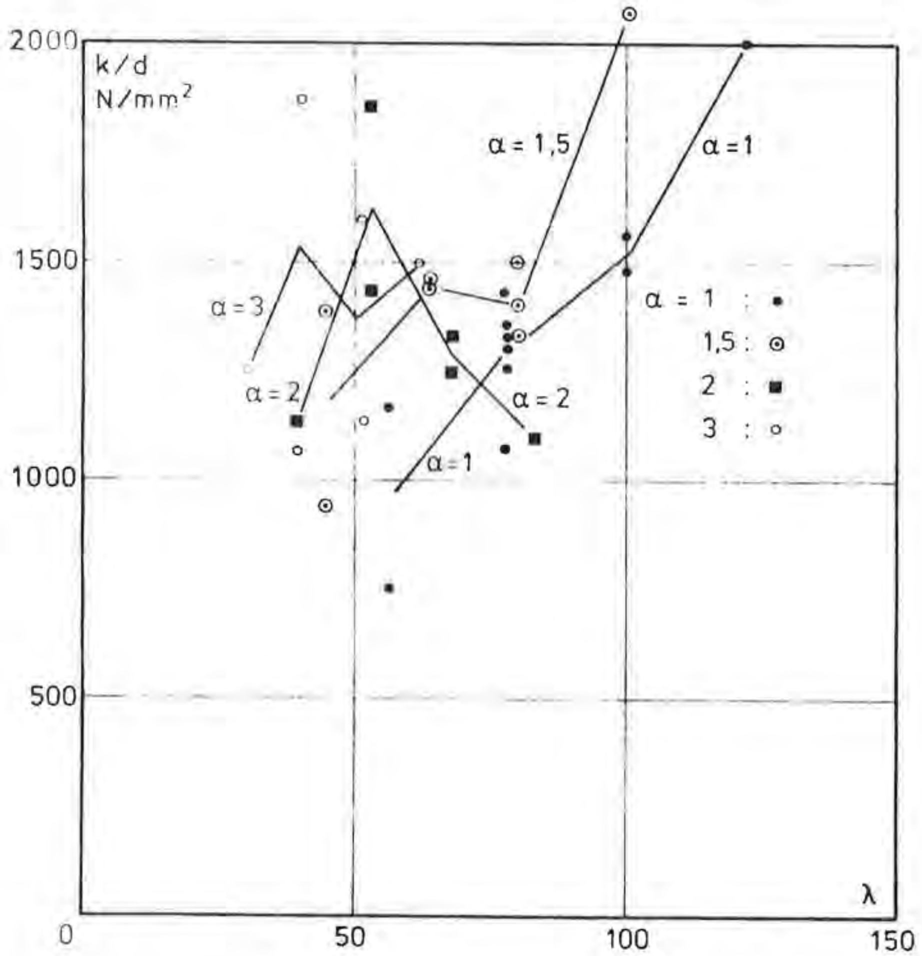


Fig. 24.

5.2.3. Spaced columns with nailed battens.

There were only found two series of tests with these columns, namely those conducted by Möhler [12] and Granholm [8]. A resumé of Möhler's results is given in Table 5. Due to the lack of some information, it is assumed that the nails are placed as shown in Fig. 25. To determine k_M it is assumed that the forces applied to the individual nails vary linearly according to the distance from the centre, and that they are perpendicular to the radius vector. The battens were made of 20 mm plywood with a length of 160 mm. The nailing, carried out with 12 nails of a diameter 2.5 mm$^{\phi}$, corresponded to $Q/(s_cA)$ being approximately 1.5% (refer to Section 3.3)

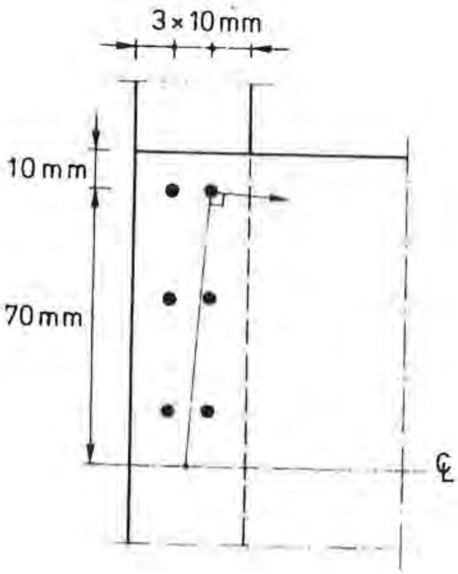


Fig. 25.

An approximate calculation shows that only the contributions from the bending of the flanges and slip and rotation of the connections of the battens are of significance. The contributions to μ and thereby to a/k are indicated in Table 5 by μ_b (bending), μ_s(slip of nails) and μ_M (rotation). The flanges were 30 x 90 mm and the column length approximately 2.00 and 2.60 metres.

TABLE 5. Columns with 5 battens.

Identi-fica-tion	α	λ	λ_e	I_e^m/I	μ_b %	μ_n %	μ_M %	k N/mm	k/d N/mm^2
3n	3	33	99	0,11	19	32	49	860	400
3'n	"	43	124	0,12	23	30	47	1120	
4n	4	26	95	0,075	20	23	57	780	340
4'n	"	34	115	0,088	26	21	53	930	
5n	5	22	81	0,074	30	15	55	920	400
5'n	"	29	106	0,075	33	15	52	1070	

It can be seen that there is some dependency on α. k/d is, on average, 380 N/mm^2, i.e. practically the same value as for columns with packs.

The results of Granholm's experiments are given in Table 6. The flanges were approximately 4.5 x 9.5 mm^2, α was approximately 4.5 and the column length app. 4.10 m corresponding to $\lambda \sim 33$. The battens were made of 19 x 95 mm boards. The nailing was quite light, corresponding to $Q/(s_c A)$ varying between 1% and 2%. The nailing was carried out with a view to eliminating the friction.

TABLE 6

No.	No. of battens	λ_e	I_e^m/I	Nails number	$d^\square$ mm	k N/mm	k/d N/mm^2
2	8	150	0,047	4	2,50	1510	600
3	10	135	0,059	4		1420	570
4	11	150	0,048	4		1130	450
5	11	105	0,093	4		1920	770
6	14	120	0,076	4		1100	440
7	8	195	0,028	2		1320	530
8	8	165	0,038	2		875	350
9	8	125	0,068	4		935	370
10	8	145	0,044	2	2,80	3550	1270

The average figure for k/d in tests 2-9 is 510 N/mm^2, i.e. a little higher than found above. It must be noted that Granholm's experiments were performed with square nails, and the 510 should therefore be reduced by about 15% before making comparisons.

5.2.4 Nailed lattice columns.

Granholm [8] has carried out four series of tests with geometry as shown in Tables 7 and 8.

TABLE 7.

No.	l m	Flanges mmxmm	Diagonals mmxmm	α	θ	e mm	λ_1
1	14,2	80x230	2x38x120	3,1	45°	113	50
2.1 - 2.3	5,6	38x120	2x11x34	3,0	"	43	45
3.1 - 3.8	6,1	35x95	"	3,5	"	44	50
6.1 - 6.7	6,1	35x98	"	4,6	"	43	55

Series 1, 2 and 3 were nailed normally and tested shortly after fabrication i.e. friction occurred in varying degrees which means that there was quite a large variation in the stiffness of the connections, and that a number of values are apparently higher than obtainable in practice.

In series 4 the nailing was performed with a 0.3 mm spacer block between the diagonals and the flanges. This was afterwards removed. The ratios therefore corresponded to what might be expected in practice.

The results of the tests are given in Table 8. The number of nails stated is the total at each end. The nailing corresponds to $Q/(s_c A)$ being between 1.5% and 3%, apart from the series with 2 nails where it is about 0.75%.

It is shown how the contributions to μ (and thereby a/k) are due to the diagonal extension (μ_d), the bending of the flanges caused by the eccentric intersection of the diagonals (μ_b) and the slip of the nails (μ_n). It will be seen that the last contribution is completely dominant.

The stiffness number for the individual nail is denoted with k.

It can be seen that k/d is more or less as found in Section 5.2.3 for series 1 and 3. Two average figures are given for series 3; in one case the two lightly nailed tests, where as previously mentioned one would expect relatively high values, have been ignored.

TABLE 8.

No.	λ	λ_e	Nails no. $d^{\square}$ mm	I_e/I	μ_d %	μ_b %	μ_n %	k N/mm	k/d N/mm^2	
1	85	105	10x5,16	0,655	7	15	78	2680	520	
2.1	80	95	8x2,30	0,701	23	14	63	2755	1200	
2.2	"	115	4x -	0,483	12	5	83	2075	900	1070
2.3	"	125	2x -	0,382	8	4	88	2525	1100	
3.1	80	95	8x -	0,694	15	14	71	1445	630	
3.2	"	100	8x -	0,663	13	13	74	1240	540	
3.3	"	100	8x -	0,631	14	11	75	1245	540	
3.4	"	100	8x -	0,614	13	10	77	1130	490	565
3.5	80	120	4x -	0,450	9	5	86	1505	650	
3.6	"	125	4x -	0,401	8	4	88	1235	540	625
3.7	"	120	2x -	0,456	6	5	89	1995	870	
3.8	"	125	2x -	0,415	6	4	90	1710	740	
6.1	65	115	4x -	0,314	6	8	86	1070	470	
6.2	"	115	4x -	0,318	6	8	86	1095	480	
6.3	"	140	2x -	0,212	3	5	92	1160	500	
6.4	"	100	6x -	0,396	6	12	82	875	380	455
6.5	65	110	4x -	0,339	5	9	86	970	420	
6.6	"	110	4x -	0,350	5	10	85	1030	450	
6.7	"	130	2x -	0,243	3	6	91	1130	490	

The most interesting results are in series 6. The average figure is 455 N/mm^2, which when converted to round nails is app. 390 N/mm^2, i.e. the same value as that in Möhler's tests. In this case the coefficient of variation corresponds to app. 10%, which is very modest for timber experiments.

5.2.5 Conclusion.

It appears from the comprehensive test material described that for spaced columns and lattice columns with short-term loads one can in practice assume

$$k/d = 400\ N/mm^2 \tag{53}$$

for round nails. For quadratic nails 15% more can be assumed.

For continuously nailed columns where shrinkage apparently has greater influence, especially for high α values, one would not recommend taking into account more than

$$k/d \sim 450/\alpha \ N/mm^2 \quad (54)$$

for round nails, again with 15% more for quadratic nails. The possibility of obtaining higher and more reliable values by using ring nails should be investigated.

The results are valid for short-term loads. There are no experimental results for long-term loads. On the basis of experience from tests with ordinary connections one cannot recommend assuming more than half the given values for permanent loads.

5.3. Bolted columns.

Bolts alone are unsuitable because of the high initial slip which must be taken into account in normal construction. This applies also to bolts with laid-in connectors (Split-ring, Shear-plate), whereas bolts with toothed-plate connectors can be used.

Möhler has, in [12] also investigated a number of columns assembled with quite small toothed-plate connectors ($P_{all.} \sim 10$ kN), partly with $\alpha = 0$ and partly with packs (with $\alpha = 1, 2, 3$ and 4).

For $\alpha = 0$ k values over 60,000 N/mm were found. k decreased substantially with greater α values. Thus for $\alpha = 4$ $k \sim 6000$ N/mm was found.

The last value is probably the one which can realistically be assumed, as the high values for $\alpha = 0$ are apparently due to friction. In all cases a good tightening was sought, but the possibilities for keeping it decrease greatly with increased thickness of the packs, and the influence of the friction is not so reliable when $\alpha > 1$. In DIN 1052 it is permitted to assume for $\alpha = 0$ that $k = 15000$ for small toothed-plates ($P_{all.} < 16$ kN) and that $k = 30000$ for large ($P_{all} > 30$ kN).

6. APPROXIMATE FORMULAS

6.0 General.

The use of the expressions derived above can be a little complicated, and in various codes etc. approximate expressions are given.

These can be wholly or half empirical, or based on simplifications of the "exact" expressions.

As an example of the latter, the effective slenderness number determined with the help of (44)

$$\lambda_e^2 = \frac{l^2 A}{I_e} = \frac{1+\mu}{1+\beta^2\mu} \frac{l^2 A}{I}$$

approximately - when $\beta^2\mu$ is small - can be written as

$$\lambda_e^2 \sim (1+\mu) \frac{l^2 A}{I} = \lambda^2 + \frac{\pi^2 A A^* E a}{(N-1) I k} \qquad (55)$$

In certain cases this form will be very convenient (refer to Section 6.1).

The English CP 112[2] mentions only spaced columns with two lamellas and with packs, either glued or nailed.

It is stated that the largest of the slenderness ratios λ_e and λ_1 must be used, where λ_e/λ is shown in Table 9 and λ_1 is the slenderness ratio corresponding to local stability failure of the lamellas with column length l_1, i.e.

$$\lambda_1^2 = \frac{l_1^2 A_1}{I_1} \qquad (56)$$

TABLE 9. λ_e/λ according to CP 112:1967.

Method of connection	α			
	0	1	2	3
Nailed	1.8	2.6	3.1	3.5
Screwed or bolted	1.7	2.4	2.8	3.1
Connectored	1.4	1.8	2.2	2.4
Glued	1.1	1.1	1.3	1.4

In the German DIN 1052[1] and the Dutch NEN 3852[3] the following approximate expression is given:

$$\lambda_e^2 = \lambda^2 + f\,\frac{m}{2}\,\lambda_1^2 \tag{57}$$

where m is the number of actual lamellas and f_1 is a factor which depends on the column type as given in table 10.

TABLE 10. The factor f_1 in (57)

	Spaced columns with	
	packs	battens
Glue	1,0	3,0
Nails	3,0	4,5
Bolt with toothed-plate connectors	2,5	

In the Swedish building code SBN 67[4], according to a suggestion made by Niskanen, it is stated that for built-up columns with nailed packs λ_e can be determined by

$$\lambda_e^2 = f_2(\lambda^2 + C(\frac{l}{3l_1})^2 \tag{58}$$

where C is dependent on λ and α as shown in Table 11. f_2 is 1 for $l/l_1 \sim 3$; 0.94 for $l/l_1 \sim 4$ and 0.92 for $l/l_1 \geq 5$.

TABLE 11. The factor C in (58).

No of shafts	α	λ ≤ 20	30	40	50	≥ 60
2	≤ 1	2,5	2,2	2,0	1,8	1,6
	2	2,1	2,0	1,9	1,7	1,6
	3-5	1,7	1,7	1,7	1,6	1,5
3	≤ 1	2,1	2,0	1,8	1,7	1,6
	2	1,9	1,7	1,5	1,5	1,5
	3-5	1,7	1,5	1,4	1,4	1,4

For $l/l_1 \sim 3$ and 2 lamellas, (58) becomes

$$\lambda_e^2 = \lambda^2 + C\lambda_1^2 \tag{59}$$

i.e. the same as (57), but with another and smaller factor on λ_1^2.

For continuously nailed columns the Swedish code gives $\lambda_e/\lambda = f_3$ where f_3 is 1.6 for the cross-section type 2 in fig. 4 and is shown in fig. 26 for types 3-5 in the same figure.

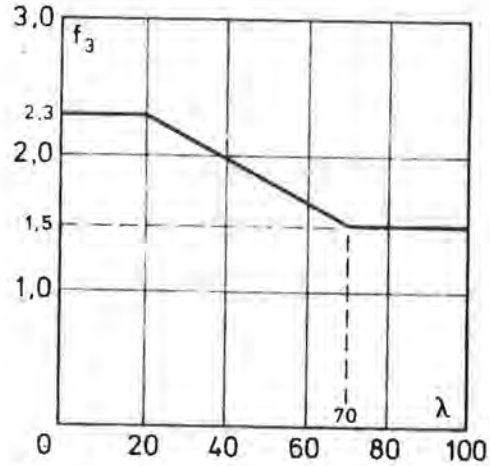


Fig. 26.

6.1. Spaced columns with glued packs.

In Table 12 the column with two flanges is investigated where the variation of the two determining parameters l/l_1 and α amply cover the cases which can occur in practice.

The theoretical expression for λ_e/λ is calculated from (44). Provided λ_1/λ is greater than λ_e^L/λ, the load-carrying capacity will be determined from local stability failure of the flanges. In Table 12 the determining slenderness number is framed. For this column type $A^*/(N-1) = A/2$, and if the approximate assumption $I \sim (1+\alpha)^2 t^2 A/4$ is made,

$$\lambda_e^2 \sim \lambda^2 + \frac{\pi^2}{12}\lambda_1^2 \tag{60}$$

is found by substituting (46) into (55).

λ_e/λ calculated from (60) is also shown in Table 12. It can be seen that there is a very satisfactory approximation, which also appears from the fact that $\beta^2\mu$ is small. The table also shows λ_e/λ in relation to the English and the German/Dutch approximate expressions discussed in section 6.0. It is seen that in this case the difference between (55) and (60) is only the factor for $\lambda_1^2(\frac{\pi^2}{12}$ or 1).

TABLE 12.

l/l_1		3					5					8				
α		0	1	2	3	5	0	1	2	3	5	0	1	2	3	5
μ		0,274	1,097	2,467	4,386	9,870	0,099	0,395	0,888	1,579	3,553	0,039	0,154	0,347	0,617	1,388
$\beta^2\mu$		0,069	0,084	0,088	0,090	0,091	0,025	0,030	0,032	0,032	0,033	0,010	0,012	0,012	0,012	0,013
λ_1/λ		0,67	1,20	1,76	2,35	3,48	0,40	0,72	1,06	1,40	2,09	0,25	0,45	0,66	0,88	1,31
λ_e/λ	(60)	1,09	1,[illegible]	1,79	2,22	3,16	1,04	1,16	1,35	1,58	2,10	1,01	1,07	1,15	1,26	1,54
according to	(55)	1,13	1,4[illegible]	1,86	2,32	3,30	1,05	1,18	1,37	1,61	2,13	1,02	1,07	1,16	1,27	1,55
	CP 112	1,1	1,3	1.76	2,35	3,48	1,1	1,1	1,3	1,4	2,09	1,1	1,1	1,3	1,4	-
	DIN 1052	1,20	1,56	2,03	2,54	3,62	1,08	1,23	1,46	1,72	2,32	1,03	1,10	1,20	1,33	1,64

6.2. Spaced columns with nailed packs.

For this type of column with two flanges there are four parameters, namely l/l_1 (the number of free sections), l_1/t (the length of the individual sections), α and the stiffness of the nail connections (K in formula (46)). The geometric parameter values selected are shown in Tables 13 and 14. It will be seen that all the cases which might occur in practice are covered.

The stiffness number K is established as follows:

- The design is carried out for a transverse force of $Q = 0.015s_cA$, see section 3.3. The force per connection will then be $P = 0.015s_cA\frac{l_1}{(1+\alpha)t}$ (see fig. 13).
- It is assumed that the nail thickness d is app. t/8, and that the load-carrying capacity in N is $130d^2 \sim 2t^2$ (d and t in mm). For the individual nail it is assumed that $k/d = 400$ N/mm^2 $\sim k/t = 500$ N/mm^2 (refer to section 5.2.2).

With $E/s_c \sim 300$

$$\mu = \mu_{bending} + \mu_{nail} \sim \frac{\pi^2(1+\alpha)^2}{4(\frac{l}{l_1})^2} + \frac{4000(1+\alpha)}{(\frac{l}{t})^2} \qquad (61)$$

is then found from (44) and (46).

The result of the calculations is given in Table 13.

The first line in each section shows λ_1/λ. Only in one single case is λ_1 rather than λ_e decisive.

The next two lines give λ_e and λ_e/λ calculated according to the theory with μ as shown.

The last two lines give λ_e/λ calculated according to the approximate expressions in DIN 1052 (DIN) and British Standard CP112 (BS) - refer to section 6.0.

It will be seen that the approximations, both in DIN and BS, are justifiable as far as pure safety is concerned, but that in many cases they are very much on the safe side. This is especially true of the BS approximation which must be considered unreasonably rough.

The approximation corresponding to (55) is not shown in the table, as it will in this case often be unreasonably on the safe side because $\beta^2\mu$ is relatively large.

The Swedish approximation (59) gives values between 1.14 and 1.34 for $\alpha = 0$; between 3.20 and 2.74 for $\alpha = 3$ and between 4.64 and 3.85 for $\alpha = 5$. As there is a very rough approximation, which is indeed very much on the unsafe side in certain cases, this is not discussed further.*

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*After completion of the manuscript a new Norwegian Code of Practice (Norsk Standard NS 3470-1973) was received, which gives the approximate formula

$$\lambda_e = (c\lambda)^2 + \lambda_1^2$$

where c is a factor dependent on λ ($\lambda = 10 \Rightarrow c = 3.3$, $\lambda \geqq 100 \Rightarrow c = 1.5$).

The approximation is in many cases very much on the safe side, but is otherwise acceptable.

TABLE 13.

α	l/l_1		3			5			8		
	l_1/t		10	20	40	10	20	40	10	20	40
0	λ_1/λ			0,67			0,40			0,25	
	λ_e	Theory	84	138	243	120	199	(370)	165	296	(571)
	λ_e/λ	Theory	1,62	1,33	1,17	1,38	1,15	1,07	1,19	1,07	(1,03)
		DIN		1,53			1,22			1,09	
		BS					1,8				
1	λ_1/λ			1,20			0,72			0,45	
	λ_e	Theory	72	109	176	91	136	236	113	183	(338)
	λ_e/λ	Theory	2,49	1,85	1,53	1,90	1,42	1,23	1,47	1,19	(1,10)
		DIN		2,31			1,60			1,27	
		BS					2,6				
2	λ_1/λ			1,76			1,06			0,66	
	λ_e	Theory	65	92	155	78	110	190	90	140	251
	λ_e/λ	Theory	3,28	2,37	1,96	1,36	1,69	1,45	1,73	1,33	1,20
		DIN		3,21			2,09			1,52	
		BS					3,1				
3	λ_1/λ			2,33			1,40			0,88	
	λ_e	Theory	59	86	144	69	98	167	78	117	209
	λ_e/λ	Theory	3,99	2,89	2,42	2,78	1,98	1,69	1,97	1,48	1,32
		DIN		4,16			2,52			1,82	
		BS					3,5				
5	λ_1/λ			3,48			2,09			1,31	
	λ_e	Theory	53	78	139*	59	85	148	64	96	171
	λ_e/λ	Theory	5,30	3,90	3,37	3,55	2,57	2,23	2,42	1,80	1,61
		DIN		6,10			3,76			2,48	
		BS					-				

*$\lambda_1 (>\lambda_e)$.

Table 14 - with respect to a later point regarding safety problems and consideration of the effects of time - shows λ_e/λ provided $k/d = 200\ N/mm^2$ is assumed instead of $k/d = 400\ N/mm^2$, i.e. with the contribution to μ from the nails (μ_{nail}) twice as large as that shown in (61).

TABLE 14. λ_e/λ for k/d = 400 and $k/d = 200\ N/mm^2$, and the ratio between the critical stresses for the two cases.

l/l_1	3				5				8			
l_1/t	10		40		10		40		10		40	
k/d	200	400	200	400	200	400	200	400	200	400	200	400
$\sigma = 0$	1,77	1,62	1,23	1,17	1,53	1,38	1,10	1,07	1,32	1,19	1,04	1,03
	0,85		0,90		0,82		0,95		0,81		0,98	
$\alpha = 3$	4,75	3,99	2,59	2,42	3,45	2,78	1,79	1,69	2,44	1,97	1,38	1,32
	0,79		0,87		0,69		0,89		0,69		0,91	
$\alpha = 5$	6,37	5,30	3,56	3,37	4,43	3,55	2,35	2,23	3,02	2,42	1,68	1,61
	0,80		0,90		0,73		0,90		0,72		0,92	

The table also gives the ratio between the load-carrying capacities corresponding to the relative slenderness numbers. It will be seen that there can be a reduction of 30%, although it will normally lie below 20%.

6.3. Spaced columns with battens.

The number of parameters is so extensive that it was not found to be feasible to undertake general evaluations in this case.

For the previously described tests the following is found for the factor f_1 in (57):

Glued battens

Egner (Table 2a)	0.96 - 1.25
Möhler (Table 2b)	0.89 - 0.96

Nailed battens

Möhler (Table 5)	2.53 - 4.12
Granholm (Table 6)	7.3 - 22

It can be seen that the approximate expression for the nailed battens can be extremely unsafe, and that it can in any case only be used provided one ensures that the construction is similar to that used in Möhler's tests, where the nailing was substantially stronger than Granholm's.

For the glued columns Möhler himself gives f_1 values between 1.45 and 2.86, and has therefore proposed f_1 = 3.0 in DIN. As mentioned in connection with Table 2b, the difference is due to Möhler's incorrect assumption that the Euler formula was valid for the calculation of λ_e.

6.4. Spaced columns with bolts and toothed connectors.

Sufficiently reliable stiffness information to justify a comprehensive calculation is not available. However, the impression given by the tests discussed in Section 5.3 is that toothed connectors have a slightly greater stiffness than a nailed connection of the same strength. The DIN approximation mentioned in Section 6.0 would therefore seem at first sight to be reasonable. On the other hand, it seems as though the approximated values in BS CP112 (Table 9 in Section 6.0) are based on an overestimation of the stiffness.

7. CONCLUSIONS.

7.1. Theory for determination of load-carrying capacity.

The conclusion from the previous sections is that the load-carrying capacity can be calculated with satisfactory accuracy from the presented expressions, i.e. that the calculations are carried out as for ordinary solid columns except that λ is replaced by λ_e, where

$$\lambda_e = \lambda \sqrt{\frac{1+\mu}{1+\beta^2\mu}}$$

μ is defined in (26), and the parameter a/k which forms part of μ is determined as follows:

Continuously jointed columns: a/k is determined from formula (45) where the last term, corresponding to the shear deformation, is immaterial, unless I or box columns with thin webs, for example of plywood, are concerned. For round nails k_1/d can be taken as $450/\alpha$ N/mm^2 in short-term loads, and $270/\alpha$ for long-term loads. This is increased by 15% for square nails.

Spaced columns: a/k is determined from (46) for columns with packs, and from (49) for battens. In the latter case the term containing I_w can generally be ignored.

The stiffness number for the nails is established as given in (53), i.e. for round nails and long-term loads k/d $\sim$ 400 N/mm^2 and 60% of this, i.e. 240 N/mm^2, for short-term loads. 15% is added for square nails.

Lattice columns: a/k is determined from (50) with the stiffness number for the nails as shown above. The last term in (50), which refers to a possible eccentric intersection of the diagonals, is often of substantial significance.

It applies in all cases that it is necessary to show that local failures do not occur.

7.2. Approximate expressions.

Some of the expressions are a little complicated, and therefore simpler rules for the most ordinary types, i.e. spaced columns, should also be given in codes. For spaced columns with packs the German/Dutch approximate expressions (57) discussed in section 6 appear to be the most recommendable.

It reflects to some extent the influence of the determining parameters without - as in the English case (corresponding to Table 9) - being unreasonably on the safe side for certain cases, and the expression could hardly be simpler as, in all cases, it is necessary to determine both λ and λ_1.

The values for f_1 given in Table 10 appear very reasonable for the glued columns and for short-term loads for the nailed ones (corresponding to k/d = 400 N/mm^2, see Tables 13 and 14). For long-term loaded nailed columns higher values for f_1 should be assumed. A calculation for nailed spaced columns with packs shows that f_1 can reasonably be increased from 3 to 4. The other values for columns with packs are similarly increased without documentation (refer to Table 15 which replaces Table 10).

TABLE 15. The factor f_1 nr. (57) for spaced columns with packs or battens.

Method of connection	Short term loading		Long term loading	
	packs	battens	packs	battens
Glue	1,0	(2,0)	1,0	(2,0)
Nails	3,0	((4,5))	4,0	((6,0))
Bolts with toothed-plate connectors	2,5		3,5	

For columns with battens the material for the establishment of f_1 is very limited. For the glued columns, in accordance with 6.3, f_1 = 2.0 is suggested.

For nailed columns the German/Dutch values of 4.5 for short-term loads and a 30% higher value for long-term loads are reproduced - with greater doubtfulness. It should be noted that these values in each case assume that the connections are designed for loads corresponding to $Q = 0.015 s_c A$ (refer to Section 7.3).

7.3. Design basis.

It must of course be ensured that the connections have the necessary strength, which can be expected to be the case provided

- the length of packs or battens is about 1.5 times the free distance between the lamellas (1.5α)
- the connections as well as the battens and diagonals in lattice columns are designed according to a transverse force $Q = 0.015 s_c A_{necc}$ where s_c is the compression strength and A_{necc} is the necessary area.

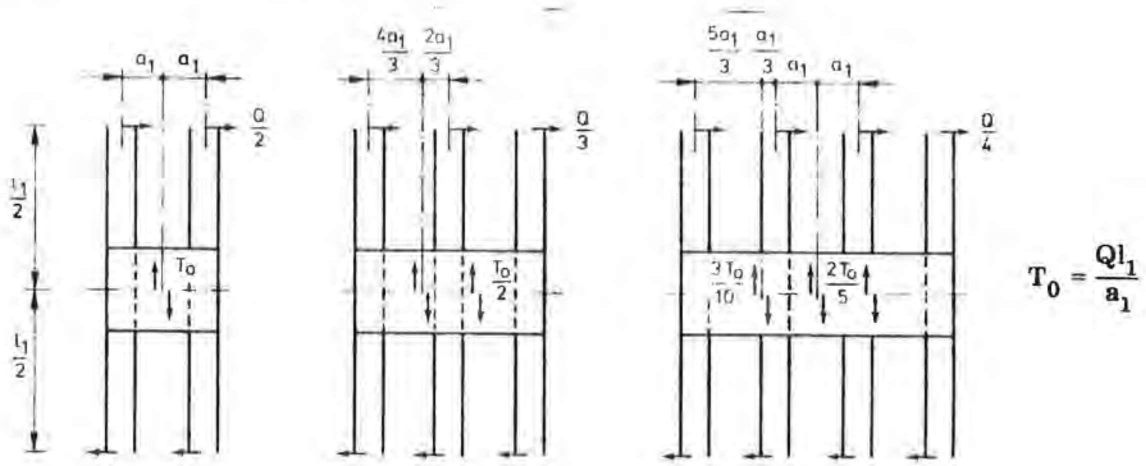

Fig. 27.

Following a suggestion from Möhler, DIN 1052 states that Q can be assumed to give forces in battens as shown in Fig. 27.

8. LITERATURE

[1] Deutsche Normen, DIN 1052, Blatt 1, Holzbauwerke. 1969.

[2] British Standard Code of Practice, CP 112:1967, The Structural Use of Timber.

[3] Nederlandse Norm, NEN 3852, 1973. Technische grondslagen voor de berekening van bouwconstructies.

[4] Svensk Byggnorm, SBN 1967.

[5] Booth, L.G. & Reece, P.O.: The Structural Use of Timber. London 1967.

[6] Egner, K.: Versuche mit zweiteiligen, geleimten Holzstützen. Fortschritte und Forschungen im Bauwesen, Reihe D, Heft 9, p. 12-26. 1953.

[7] Egner, K.: Versuche mit geleimten Holzstützen, II Teil. As [6], Heft 20, p. 7-14. 1955.

[8] Granholm, H.: On composite beams and columns with particular regard to nailed timber structures (in Swedish). Chalmers Tekniska Högskolas Handlingar. No. 88. 1949.

[9] Kinzey, B.Y.: Wood Box Columns and their Design. Bull. Virginia Polytechnic Institute. Eng. Exp. Station Series No. 79, 1951.

[10] Larsen, H.J.: The design of solid timber columns. CIB Working Group W18, October 1973.

[11] Morris, E.N. & Farrell, R.M.: Investigation into the strength of spaced timber long columns. TRADA Research Report E/RR/18.

[12] Möhler, K.: Biege- und Knickversuche mit zusammengesetzten Holzdruckstäben. Fortschritte und Forschungen im Bauwesen, Reihe D, Heft 9, p. 27-54, 1953.

[13] Möhler, K.: Biege- und Knickversuche an Stützen mit zusammengesetzten, kontinuerlich vernagelten Holzquerschnitten. As [12], Heft 20, p. 15-26, 1955.

[14] Nijenhuis, W.: Beproevning en berekening van samengestelde houten drukstaven. Rapport 4-62-6-kb-6. Stevin Laboratorium, Delft. 1962.

[15] Niskanen, E.: Investigations on the buckling of compressed
Hk 56 columns assembled by nailing. The State Institute for Technical Research, Finland. Publication 63. 1961.

[16] Noren, B.: Nailed Joints. Proc. of the First International Conf. on Timber Engineering. Southampton 1961.

9. SYMBOLS

This list contains generally used symbols. Symbols which are only used locally are defined in the individual sections. The index n of a parameter indicates the value of the parameter in lamella no. n or in joint no. n.

' indicates differentiation with respect to x.

A Area

- A_d Area of a diagonal in a lattice column
- A^* Cross-section parameter defined in (19)

E Modulus of elasticity

- E Modulus of elasticity for wood

H Shearing force in a joint (force per unit length)

- H_o H if the lamellas were rigidly connected (e.g. with glue)

I Moment of inertia

- I The geometrical I for the cross-section as a whole, see (2)
- I_o ΣI_n for all laminations, see (1)
- I_e Effective moment of inertia for columns
- I_{eb} I_e for bending
- I^c I calculated
- I^m I measured

K Stiffness of joint between shaft and packs or battens in a spaced column

N Normal force

N Number of laminations, see fig. 1

M Moment (bending about the Y-axis)

P Load on a connection

P Load-carrying capacity of a column

- P_E P determined from the Euler formula
- P_{cr} P critical
- P^c P calculated

P^m P measured

Q Shear force

S Slip in a joint

S_n Static moment about the Y-axis for laminations No. 1 to n

a Distance between the connections

d Diameter of nail (side-length for square)

e Initial deflection, see Section 3.2

e Eccentricity of diagonals in lattice column, see Fig. 15

i Radius of gyration

$i = \sqrt{I/A}$

$i_e = \sqrt{I_e/A}$

$i_1 = \sqrt{I_1/A_1}$

k Stiffness of joint, see (3)

k_d Stiffness of diagonal joint in lattice column, see Fig. 15

k_1 Stiffness of a single nail (also denoted k when there can be no misunderstanding)

k_M Stiffness in bending/torsion of a joint in a Vierendeel column, see Fig. 14

$k_E = s_E/s_c$, see (36)

l Free length of column

l_1 Distance between middle of packs or battens for spaced columns and distance between joints in lattice columns.

n Number of nails in a joint

s Strength value

s_b Bending strength

s_c Compression strength

s_{cr} Critical stress (ultimate stress in the column)

s_E Euler strength

t Thickness of a lamination (shaft)

v Translation in the x-direction

w Deflection in the z-direction

w_o w provided the lamellas were rigidly connected to one another, see (22)

x Length co-ordinate

y Cross-section co-ordinate, see Fig. 1

z " " " " " "

α Relative free distance for spaced columns, see for example Fig. 12.

β^2 I_e/I, see (1)

γ See (27) - (28)

θ Diagonal inclination, see Fig. 15

λ Slenderness ratio

$\lambda = \sqrt{l^2 A/I}$

$\lambda_e = \sqrt{l^2 A_e/I}$ = effective slenderness ratio

$\lambda_1 = \sqrt{l^2 A_1/I_1}$ = slenderness ratio for a shaft in a spaced column

μ See (26)

σ Normal stress

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DEFINITIONS OF LONG TERM LOADING FOR THE CODE OF PRACTICE

by

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DEFINITIONS OF LONG-TERM LOADING FOR THE CODE OF PRACTICE

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Introduction

In spite of the fairly extensive research in the field of wood rheology, the influence of the loading time in the design of wood constructions is considered as a rule merely with a single factor of reduction for the strength and elasticity coefficients. The reduction coefficients for strength at long-term loading, for wood generally the 9/16 from the so called Madison-curve, has been argued. In any case it is important to differentiate strength reduction and creep values, with regard to moisture content and type of construction and to kind of loading. How far in detail this should be carried out is a matter of how complicated a code one can accept. For sure one has to stop at a considerable distance from the detailed results available from research. In this paper it will not be dealt with the reduction values for long-term loading of different wood materials but with the possibilities for a more differentiated treatment of loads of different durability. These possibilities are greatly limited by our knowledge of the real loads and of difficulties to formulate this knowledge in the loading code in a reasonably simple way.

In most countries the code of practice for timber structures is, or will be, subordinated a general code for structures, including a code for loads. This general code must have such wording that it can be applied also on materials and structures which have more or less pronounced rheologic properties. This is necassary for wood, concrete and plastics among other building materials. In som cases short-term loading may be defined corresponding to ordinary standard testing and long-term loading equalized to constant load during the expected service life of the structure. It is possible on these concepts to write a simple code which like many of the present codes is satisfactory from the point of safety. Still this is hardly satisfactory from the point of economic design. In an explicit code one must consider that loads generally will fluctuate in intensity during the service time of the structure. This fact has influenced the work within NKB on a common Nordic code for loads and principles of verification of safety.

Specification of safety

The deciding factor for the design of a structure is that during its expected service life, with a certain minimum probability, it should not fail. In failure is here included the relatively small damage which defines the limit of the serviceability state as well as the extensive rupture which defines the limit of ultimate failure. Thus the probability of failure of (new) structures can be expressed by the ratio of the number expected to fail within a scheduled lifetime ~~the~~ to the total number of the structures. Of course the risk of failure will be smaller if the considered period is less than the total lifetime. It is however essential to observe that the risk of failure based on the total lifetime is not always a sufficient basis for judging the resistance of structures, especially when this resistance is changing during the lifetime, e.g. due to rheologic properties. In these cases one may also need a stipulation of a maximum risk of failure during a specified shorter period. This period can be the last 1/10 of the service life or similar.

For an example: Volvo-cars are supposed to have a service life of 13 years. That means that the risk of failure based on this expected term is 0.5. This information may be sufficient from a national economic point of view or for the presumptive buyer of a new car who hopes to use it for 13 years. However, for a buyer of a 12 years old car the risk of failure during the last 1/10 of a 13-years lifetime is of principal interest.

The conclusion is that the safety claim in the general code for structures should be expressed with regard also to structures which lose strength with time.

Long-term resistance

Failure occurs (general: the limit of state is exceeded) when the load exceeds the resistance. The probability for this should not be greater than a certain allowed value P_a:

$$P(L > R) \leq P_a \tag{1}$$

The problem of applying this simple formula in the code of practice, as well known, is due to difficulties of estimating the real risk of failure as well as stipulating a permissible value of this risk. Naturally these obstacles are not less if the resistance (R) is depending on the load (L) by the loading time (t). As a rule the limit state of serviceability is determined by a deformation which makes the structure unfit for its purpose. At this state "failure" often refers to a deformation and more seldom it is a matter of rupture. Thus in this case the code has to treat deformation and creep.

In spite of the fact that many times also the ultimate failure can be related to an extreme limit of deformation (strain at rupture), long-term and short-term strength is often given without reference to deformation. Well known is the "Madison-curve"

$$\sigma_B = a - b \log t \tag{2}$$

With a strength $\sigma_B = 100$ at 12h dead load ($t = 1/60$ month) and the coefficients $a = 82$ and $b = 10$, this strength will be reduced to $\sigma_B = 56$ if the load remains for 30 years, i.e. to 9/16th of the short-term strength. The material may have such properties that it does not make any difference whether t in (2) at periodic loading represents the time of continous loading or the total time. In this case the equation can be used for calculating the short-term strength necessary to make the structure resist the periodic loading during a specified time (without failure), see Table 1.

TABLE 1 Required strength according to the "Madison" line (2) with a = 82, b = 10

Period of loading	Number of periods per year	Total time of loading (months) in 30 years	Relative strength	Required short-term strength
24 min	1	1/60	100	100
12h	2	1	82	122
2 months	1	60	64	156
1 year	1	360	56	179

The strength of material with predominant secondary (Newtonian) creep can be calculated accordingly if failure occurs at constant deformation, that is, at the same deformation independent of the time to failure. For materials, which recover when load is removed, the method may be insufficient. If the retardation time of the creep is small compared with the duration of unloaded state the recovery will be complete. In this case the continuous, not the total, time of load decides deformation and strength during the service life. For instance, in the case of two months snow load per year and a considerably shorter retardation time, the effective continuous time of loading will be two months. The relative strength will be higher than for the material which does not recover: 79 in stead of the value 64 given in Table 1. To conclude: for these extreme materials, which either do not recover or recover completely, we can define a constant load applied during the total life of service, respectively, a load continuously applied during a defined shorter time. These loads replace the real time-spectrum of loads to which the structure is exposed. A further conclusion with respect to cases of complete or partial recovery is, that it would considerably simplify the calculation of the influence of load-duration in strength if a theory of failure could be applied, referring strength to deformation. Actually, for wood products one does not seldom assume that the deformation at failure is independent of the rate of loading. For example, there are results from testing clear wood which indicate that the inflection point of the strain - time-curve, foresaying accelerating creep, appears at constant strain. Likewise the strain at failure in paper and wood fibre board has been found.

relatively independent of the rate of loading. Of course this is generally an approximation as the mechanism of failure is different at different loading speed. But it may well be acceptable within limits where the mechanism of failure changes.

Response to fluctuating load

Though several of the building materials - not in the least wood- have a complicated rheology, it is essential that a Code of Practice is made simple. A first approximation, which can be accepted at least in the serviceability state, is that the material is considered linear in a rheologic sense. This means, that the laws of superposition are valid for loads and deformations also after creep. The response of such a material to load can always be described by a mechanical model consisting of Hookeian springs and Newtonian flow elements. For exampel a model of Kelvin bodies in series (Kelvin = Hooke and Newton parallelly) is particularly useful for describing creep.

A further approximation is, as assumed in the following for the serviceability state as well as for ultimate failure, that the strength in a simple way can be related to a certain limit of deformation. It may then not be correct to say that the strength is decreasing with duration of load, but for a small load it will take longer time than for larger ones to bring the structure to the limit of deformation which defines failure.

In Fig 1 the full curves illustrate the response to intermittent loading of three different Kelvin bodies A, B and C. The period of one cycle is 2T, with constant load during half that time and zero load during the other half ($\eta = 0.5$ in Fig 2). The difference is in the response, expressed by the retardation time τ (or rather by the ratio of retardation time to the period of load duration τ/T). The retardation time τ is the time for the material to creep the fraction 1 - 1/e of the total creep (a), whereby e is the basis of the natural logarithm. The curves also illustrate the response of one and the same material at intermittent loading of different period length.

Case A, curve 1, shows the response of a slow material, the retardation time corresponding to 12.5 cycles. The curve segments are close to linear, i.e. the retardation is almost nil. The full curve 2 shows the creep at constant load. Also this creep curve is located closely below that straight line which represents Newtonian flow, but deviates by and by, approaching the asymtot $\frac{1}{a}(\frac{\varepsilon}{\varepsilon_0} - 1) = 1$. The creep curve for constant load (2) is in case A not drawn further than to $t = 0.28\ \tau$. However, the curve is seen prolonged as curve 2 in cases B and C.

The curves 1 and 2 in cases B and C show the corresponding response to intermittent respcitively constant load at shorter retardation time: τ/T is 1/10 respectively 1/100 of the value in case A. From a comparison of the curves 1 and 2 an important conclusion can be drawn: Curve 2 can be regarded as superimposed a mean curve which after some periods approaches half the level of curve 1. It is easy to prove that this observation can be generalized to the case where the period is split in one interval η 2T where the load is Q and one interval $(1 - \eta)$ 2T where the load is zero. In that case the effective creep is equivalent to the creep under a constant load η Q. If we are looking only for long-term effect of load, and by long-term (LT) it is here understood that the considered time is long compared with the length of the loading period, we can simply replace the fluctuating load by a constant load equal to the <u>mean</u> load

$$\bar{Q} = \frac{1}{LT}\int^{LT} Qdt \qquad (3)$$

Thus, in this case one is not interested in the fluctuation of the load within the time LT (Fig 3) but merely of the total time of application of loads of different size (Fig 4). The surface giving the mean load is to be found below the duration curve

$$\bar{Q} = \frac{1}{LT}\int_0^{Q_{max}} \Sigma\ STdQ \qquad (4)$$

Additional to the long-term creep there is actually short-term deformation, partly instantentious (ε_0) not illustrated in Fig 1, partly short-term creep (ε_{ST}), represented in the figure by the fluctuations of curve 1. The short-term deformation is small compared with the long-term creep in case A. Contrary to this the fluctuations form an important part of the creep in case C, when the mean curve (not shown in Fig 1) already during the first period reaches the ultimate value 0.5, while the short-term creep almost reaches the value 1. (Short-term here must be seen as relative to the retardation time τ. If the period is one year - e.g. snow load during the winter - this is not what is generally meant by short-term.)

For the case A it is sufficient to consider in the code the long-term creep caused by the mean load ($\bar{Q}$ = 0.5 Q) and, reversed, in case C it is sufficient to regard the short-term creep caused by the maximum load Q. However, one can coordinate the cases A and C by adding long-term creep caused by the mean load to the short-term creep caused by the exceeding load ($Q - \bar{Q} = 0.5\ Q$):

$$\varepsilon(Q,t) = \varepsilon(\bar{Q},LT) + \varepsilon(Q - \bar{Q},ST) \qquad (5)$$

In eq. (5) LT can denote the service lifetime and ST is the time of continuous load. The equation can be applied with Q split up according to kind and size. The LT creep, due to the mean load $\bar{Q}$, is not effected by this differentiation, but the joint effect of various short-term loads of different size and durability and appearing at different occasions, is more realistically described that way.

Eq. (5) can be tested by means of the case B in Fig 1. It is seen from Table 2 that the equation results in bigger creep than the creep of the Kelvin model as illustrated by curve 1. The difference, remaining after prolonged time, is due to the fact that (5) does not reproduce the dependence of the short-term creep on the long-term creep, i.e. the dependence of the strain of the Kelvin-body at the time of load application.

TABLE 2 Creep $\Sigma\varepsilon_{(2)}$ from eq. (5) compared with creep $\varepsilon_{(1)}$ as shown by curve 1 in Fig 1, B

$t = LT$	$\bar{Q}$	$\varepsilon(\bar{Q}, LT)$	$\varepsilon(Q - \bar{Q}, ST)$	$\Sigma\varepsilon$	$\varepsilon_{(1)}$
0,4	P	0,33		0,33	0,33
1,2	2/3 P	(2/3)·0,69	(1/3)·0,33	0,57	0,47
2,0	3/5 P	(3/5)·0,85	(2/5)·0,33	0,64	0,54
2,8	4/7 P	(4/7)·0,93	(3/7)·0,33	0,67	0,57
....					
∞	0,5 P	0,5 · 1,0	0,5 · 0,33	0,67	0,59

The importance of recovery

If the period (2T) in absolute time is equally long in the cases A, B and C (for example expressed in h), the full curves in Fig 1 will represent materials of the same rheologic type (Kelvin) but responsing differently fast: Material A shows hardly any recovery of the creep when the load is removed while C is recovering almost immediately within the interval of inloading. A common assumption when the time effect is discussed is that the material able to recover is superior to the material without such an ability. Obviously this is not the case for our examplified materials C and A, providing that the final creep (a) is the same. The reason is that the ability of recovery is linked to the time needed to reach final creep. Eq. (5) is applicable in both cases but with different dominance of short-term respectively long-term creep. It also applies to material, the creep of which can be described as the sum of a number of components with properties varying from case A to case C, or more generally

$$\varepsilon/\varepsilon_0 - 1 = \int_0^\infty a(\tau)\ \phi(\tau)d\tau \qquad (6)$$

Fig 5 shows a frequency curve for $a(\tau)$ and an approximation into three creep parts corresponding to the final creep values of the models C, B and A, multiplied by 1.5/2.0/1.0 respectively.

Alternatively, the evaluation of the recovery creep can refer to material which have equal creep properties under constant load but a smaller or larger part of the creep locked against recovery.

The elastic material, as described in the foregoing periodically loaded, will ultimately creep to a fraction (η) of the final value (a) of creep at constant load. If in the other hand the recovery is completely locked, the final creep will be equal to the creep from infinite constant load. Of course, it vill require longer time to reach the final or any specified creep value. With regard to the long-term creep the periodic loading in this case can be replaced by an equal constant load with the reduced duration η LT compared with the total time LT. Here η LT is the sum of the periods during which the load is applied (generally the total time during which the load level is exceeding a certain value).

A further possible complication would be that a fraction (α) of the creep is recoverable while the residual part $(1 - \alpha)$ is locked. The creep in these respective parts at intermittent load is seen in Fig 1, case B, curves 3a and 3b. The α has the value 0.7, which means that a relatively small fraction (0.3) of the creep is locked. The mean ordinate of curve 3a approaches asymtotically the value $\eta \cdot \alpha \cdot 1.0 = 0.7 \cdot 0.5 \cdot 1.0 = 0.35$. The curve 3b reaches the final value $(1-\alpha) \cdot 1.0 = 0.30$, i.e. the sum of 3a and 3b reaches the (average) final value $0.35 + 0.30 = 0.65$. This is 0.15 above the final creep value for the ideal elastic material (curve 1 with the final value 0.50).

In Fig 1, case A, the creep recovery is very small due to the time scale and it is of minor importance whether it is locked or not. Contrary to this, in case C the final creep is nearly reached during the first loading interval. The total-curve (3) deviates from the curve of the completely elastic material (curve 1) by the recovery not being complete. This has no influence on the design of the structure.
Contrary to η, which is a factor for the loading code, α is a factor for the material belonging to the code of practice for the material in question.

In an even more detailed but also more complicated description of the properties of the material one can introduce a spectrum of $\alpha(\tau)$ in a similar way as the spectrum for the final creep $a(\tau)$. However, this will hardly be feasable for a code of practice in the near future.

Fig 1 Creep response of the Kelvin body to intermittent load (1) and to constant load (2). Dashed curves (3) refer to a case where part of the creep ($1 - \alpha = 0.3$) is locked against recovery. A, B and C show the response at different ratios of time of continuous loading (T) to time of retardation (τ).

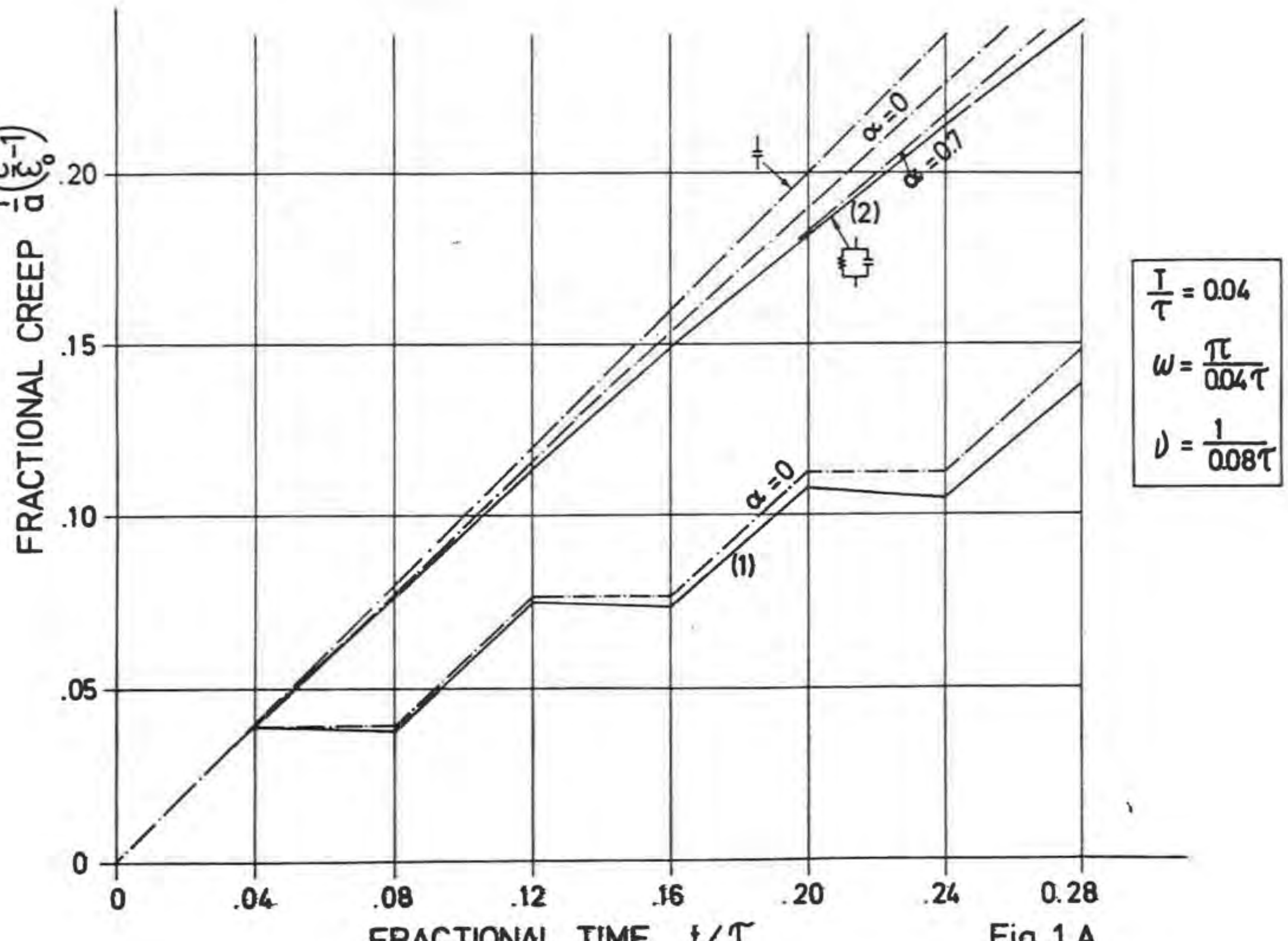
The intermittent loading corresponds to that shown in Fig 2 with $\eta = 0.5$.

Fig 2 Intermittent loading. ST = Time of continuous load (Short-term). LT = Total time (Long-term). $\Sigma ST = \eta LT$ = Total loading time.

Fig 3 Fluctuating load. It can be treated as a sum of periodic loads (dQ) variating as in Fig 2.

Fig 4 Total time for values of the load according to Fig 3, from Q_{min} to Q_{max}. The surface under the curves in Fig 3 and 4 and under the mean value $\bar{Q}$ are the same.

Fig 5 Distribution of creep "amplitudes" (values of final creep) with respect to retardation time. The curve can approximately be substituted by these parts of creep with retardation times as shown in Fig 1, A, B and C.



FRACTIONAL CREEP $\frac{1}{a}\left(\frac{\varepsilon}{\varepsilon_o}-1\right)$
0
.05
.10
.15
.20
0
.04
.08
.12
.16
.20
.24
0.28
FRACTIONAL TIME t/τ
$\alpha=0$
$\alpha=0.7$
(1)
(2)
$\frac{T}{\tau}=0.04$
$\omega=\frac{\pi}{0.04\tau}$
$\nu=\frac{1}{0.08\tau}$

Fig 1 A

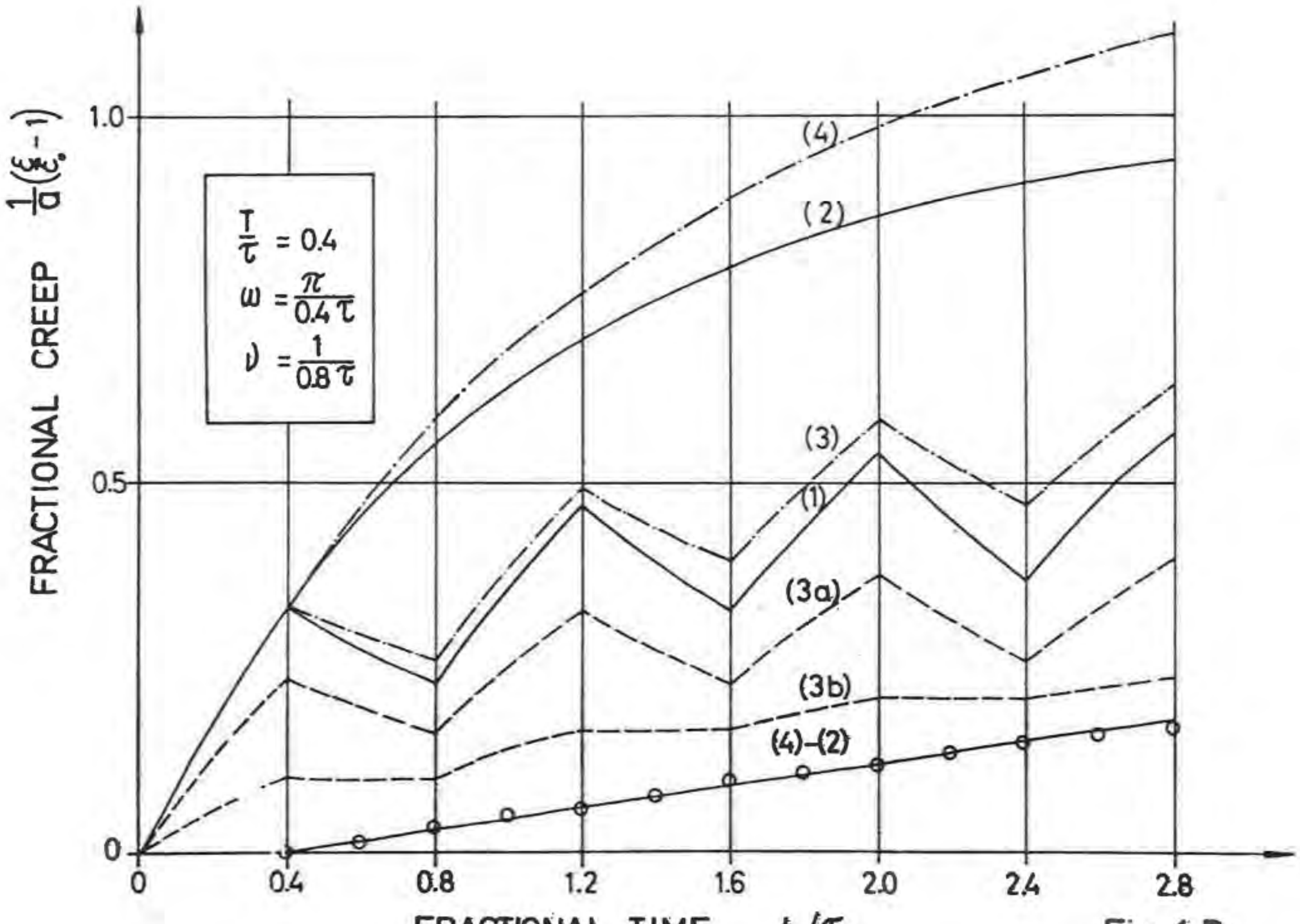

FRACTIONAL CREEP $\frac{1}{a}\left(\frac{\xi}{\xi_0}-1\right)$
$\frac{T}{\tau} = 0.4$
$\omega = \frac{\pi}{0.4\tau}$
$\nu = \frac{1}{0.8\tau}$
1.0
0.5
0
(4)
(2)
(3)
(1)
(3a)
(3b)
(4)-(2)
0.4
0.8
1.2
1.6
2.0
2.4
2.8
FRACTIONAL TIME t/τ

Fig 1 B

APPENDIX

Selected List of References on Load-sharing in timber structures and allied subjects

1. Tentative recommended Practice for determining design stresses for load-sharing lumber members - ASTM, D 2018 - 62T; 1962.

2. Methods of establishing structural grades for lumber - ASTM, D 245 - 68T. (Revision 1969 included provision for 15 per cent increase in bending stress for repetitiously used members in framing assemblies).

3. Random Products method to set stresses for lumber - J. W. Johnson, Oregon Forest Research Center, Report T-20; 1960.

4. Load distribution in wooden floors subjected to concentrated loads - N.H. Kloot and K. B. Schuster, Division of Forest Products Technical Paper No. 29, CSIRO; 1963.

5. Test report on stressed-skin plywood floor panels - T. M. Wardle, TRADA; 1969 (unpublished report on sponsored project).

6. Tentative method for evaluating allowable properties for grades of structural lumber. ASTM, D 2915 - 70T; 1969.

7. Proceeding of ASTM on committee activities: Reports of D-7 committee and sub-committees on wood.

8. Design methods for domestic timber roofs - H. J. Burgess, TRADA, E/IB/8; 1965.

9. Lateral distribution of a concentrated load on timber floor panels - G. D. Grainger, F.P.R.L., reprint from Civil Engineering; 1969.

10. Distribution of concentrated loads on timber floors - T. M. Wardle, New Zealand Forestry Service, Reprint No. 111, Design Notes No. 2.

11. Australian Standard CA 65 - 1972 - SAA Timber Engineering Code (also metric draft of this standard).

12. Deflection characteristics of residential wood-joist floor systems - paper 30, Housing and Home Finance Agency, Washington D.C.; 1954.

13. CLS Hemlock Joists - An investigation of stiffness and permissible spans. FPRC ref. 45/2/81; 1967 (unpublished report on sponsored tests).

14. Strength of multiple-member structures - John J. Zahn, U.S. F.P.L., Report No. 139; July 1970.

15. Allowable working stresses for vertically laminated beams - L. J. Nemeth, Forest Products Journal, Vol. 17, No. 4; 1967.

16. The interaction of walls, floors, roofs and partitions in housing - J. G. Sunley, Timber Research Committee paper, TRC 13/73; 1974 (unpublished).

17. Load-sharing properties of some Australian structural timbers - R. H. Leicester and G. F. Reardon, F.P.L., CSIRO - Paper contributed to 16th Forest Products Research Conference; 1973.

18. Structural reliability of timber - R. H. Leicester, paper presented at seminar 'New concepts in timber engineering', Canterbury University, New Zealand; 1973.

19. The wood frame house as a structural unit - Part I - Floor deflections as influenced by various stages of construction H. T. Hurst, Technical Bulletin 179, Department of Agricultural Engineering, Virginia Polytechnic Institute; 1965.

20. do, Part II - Residential floor vibrations as influenced by various stages of construction - H. T. Hurst, Bulletin 573, Dep. of Agr. Eng., Virginia Poly. Inst.; 1966.

21. Correction factors for deflection of wood flooring systems - H. T. Hurst and D. H. Pletta - paper accepted for publication by International Association for Housing Science, Rolla, Missouri; 1973.

22. Service and overload behaviour of wood-joist floor systems - D. Vanderbilt, J. Goodman, M. E. Criswell, Journal of Structural Division, Colorado State University; 1974.

23. Implications of rational analysis of wood-joist housing floor systems - paper by J. R. Goodman, M. E. Criswell, D. Vanderbilt, J. Bodig - Colorado State University; 1974.

24. Composite and two-way action in wood-joist floor systems - J. R. Goodman et alt., C.S.U.; 1974.

25. Investigation on the load-sharing effects of boarding on a point loaded timber floor - Report @-68-12, P. Vermeyden, T. H. Delft; 1968 (Dutch).

26. Reduction factors for concentrated floor and roof loads - Report 4-70-4, P. Vermeyden, T. H. Delft; 1970 (Dutch).

27. The calculation of timber floors and floor joists of dwelling houses - Report 4-70-21, P. Vermeyden, T. H. Delft; 1970 (Dutch).

28. The structural use of timber - BS CP 112: 1971 - British Standards Institution; 1971.

29. The effect of size on bearing strength of wood members, Research Paper 56, U.S. F.P.L.; 1966.

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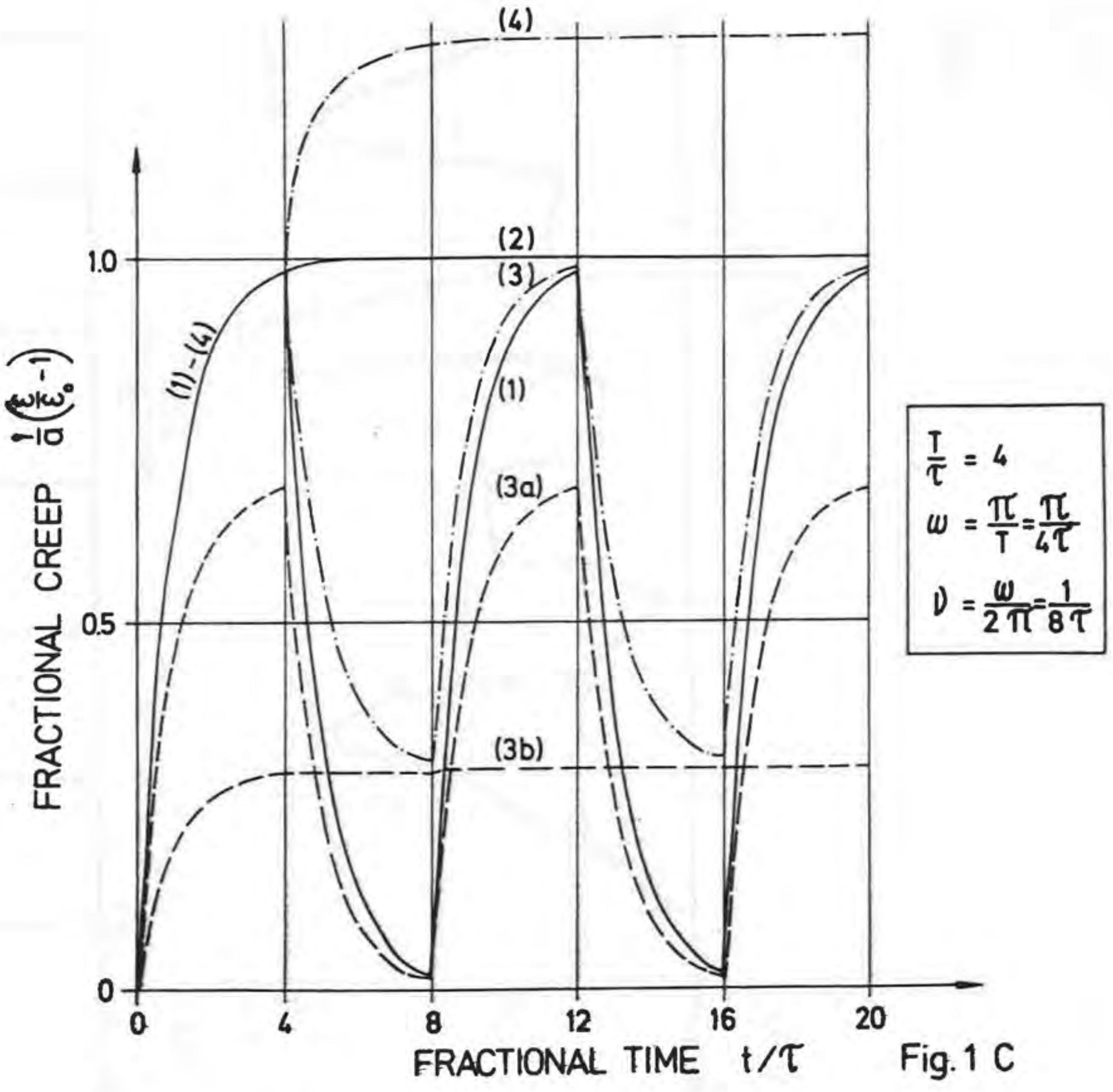

FRACTIONAL CREEP $\frac{1}{a}\left(\frac{\varepsilon}{\varepsilon_0}-1\right)$
1.0
0.5
0
(4)
(2)
(3)
(1)-(4)
(1)
(3a)
(3b)
0
4
8
12
16
20
FRACTIONAL TIME t/τ
$\frac{T}{\tau} = 4$
$\omega = \frac{\pi}{T} = \frac{\pi}{4\tau}$
$\nu = \frac{\omega}{2\pi} = \frac{1}{8\tau}$

Fig. 1 C

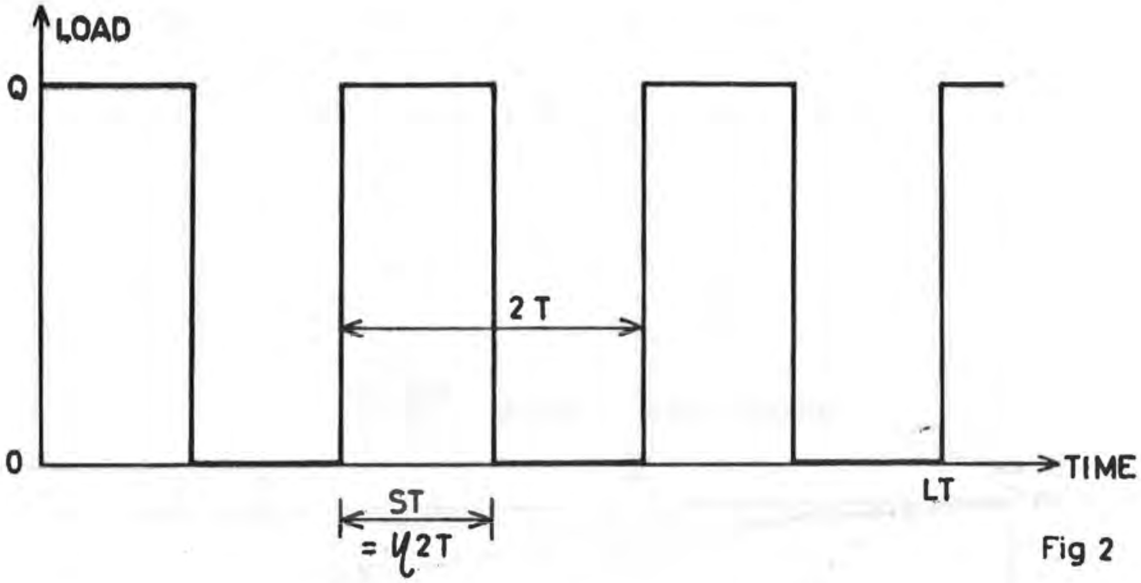

Fig 2

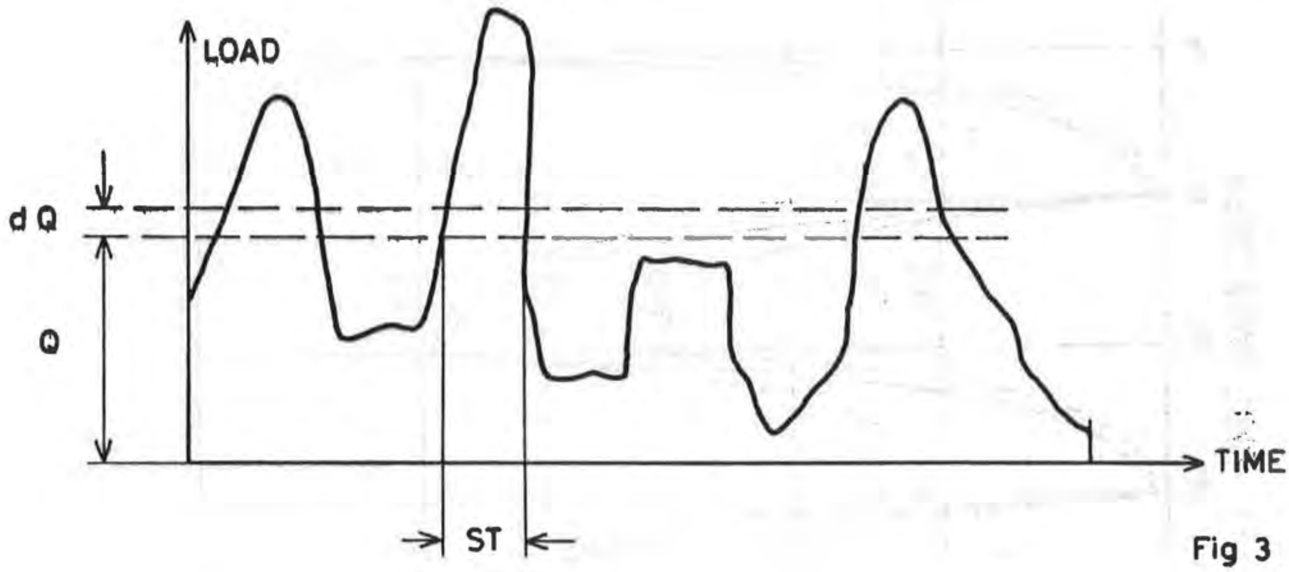

Fig 3

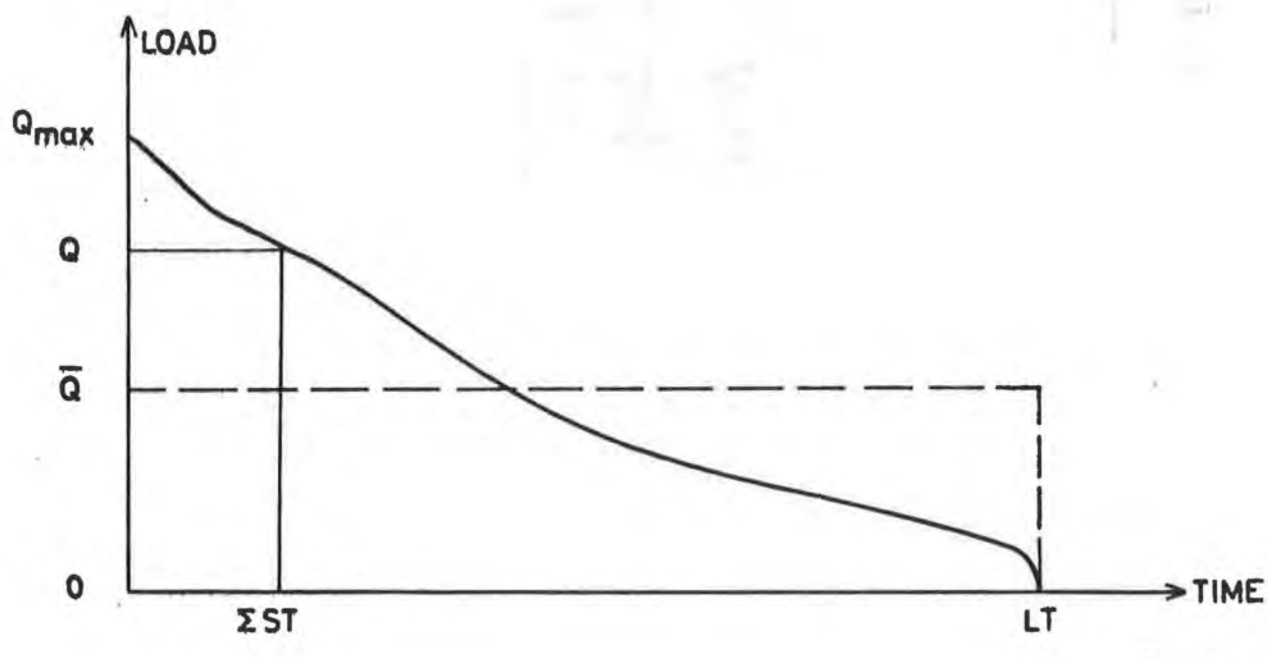

Fig 4

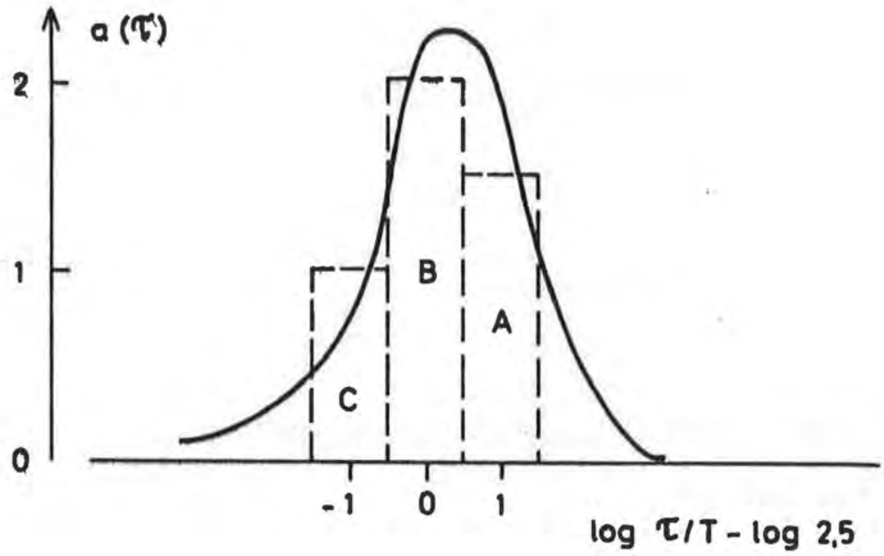

Fig 5

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

APPENDIX 1

to

DEFINITIONS OF LONG-TERM LOADING FOR THE CODE OF PRACTICE

by

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DELFT - JUNE 1974

EXTRACT FROM THE NKB LOADING CODE

Table 2.1.2 Durability classification (Time of coninuous loading[1])

Class of durability	Limits of time of continuous loading lower	upper	Example
A	250 d	-	Dead load
B	15 h	250 d	Snow "
C	2 s	15 h	Human "
D	-	2 s	Impact "

[1] In many cases the time of continuous loading is not sufficient for describing the effect of load during long time. The accumulated effect from several periods of loading can be decisive. In this case conceptions like "relative durability" (η) or mean load ($\bar{Q}$) may be introduced in the respective Codes of Practice (for different structural materials).

....

2.3 Probability of intensity of load

2.3.1 When the Method of Partial coefficients is applied

Loads are grouped as follows:

Constant load (k) For contant load applies:

1) The load is on at any arbitrary chosen occasion

2) The characteristic value is defined by the (geometric) mean value of the intensity

Common load (v) ("normal" load) is a load of an intensity which with a probability $P \geq 0.2$ is exceeded at least once a year. The characteristic intensity is defined by $P = 0.2$

For short-term common load (v1) the relative durability for the characteristic value is $\eta \leq 0.001$.

Exceptional load (ov) is a load of an intensity which with a probability of $0.2 > P > 0.02$ is exceeded at least once a year. The characteristical value is defined by $P = 0.02$

2.3.2 When a Statistic method is applied

When a statistic method is applied for the design the intensity of load is described by the mean value and the coefficient of variation of the highest value appearing during a year.

....

4.4.1 Snow load

4.4.1.1 General

Snow load is with regard to durability to be referred to Class B, table 2.1.2.

For the relative durability η the following values are accepted

	Common	Exceptional
Denmark, South Sweden	0,001	0,0002
S Finland, S Norway Middle Sweden	0.015	0.0005
Northern Finland, N. Norway and N Sweden	0.02	0.001

....

4.4.2 Wind load

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Exceptional windload of maximum intensity is with regard to durability referred to Class D in Table 2.1.2. The relative (accumulated) durability for common and exceptional windload is assumed to be 10^{-2} resp. 10^{-5}.

PAPER 8

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

LOAD-SHARING - AN INVESTIGATION ON THE STATE OF RESEARCH AND DEVELOPMENT OF DESIGN CRITERIA

by

E LEVIN

TIMBER RESEARCH AND DEVELOPMENT ASSOCIATION

HIGH WYCOMBE, ENGLAND

DELFT - JUNE 1974

LOAD - SHARING

AN INVESTIGATION ON THE STATE OF RESEARCH AND DEVELOPMENT OF DESIGN CRITERIA

A PRELIMINARY REPORT

E. Levin, Timber Research and Development Association, High Wycombe, England.

1. At the meeting of CIB Commission W18 - Timber Structures - held in Copenhagen on 25th and 26th October 1973, the author proposed that the study of load-sharing in composite elements and structures and design provisions therefor in codes of practice would be a suitable subject for the Commission to embark upon. Although it was widely recognised as a potential source of important economies in the design of structures, it appeared to be dealt with scantily in some national codes and ignored in others. Members of the Commission readily agreed and promptly entrusted the proposer with the task of undertaking an investigation and preparing a report for the Commission.

2. As a first step, an appeal was addressed to all members of the Commission as well as to other research workers and a number of Forest Products Laboratories, building research establishments and universities which were suggested to the author as likely to provide fruitful information. The intention was (a) to obtain information on the present provision in national codes or other authoritative guides, (b) to obtain research reports on work of a theoretical or practical nature bearing on any aspect of the subject and (c) to find out if possible what work was in progress or planned.

3. Replies and documents were received from many colleagues within the Commission and outside it and grateful acknowledgements are extended to them all.

4. It was not considered desirable at the outset to define and limit the area of the investigation. Rather it was hoped that clear definitions and delimitations of the scope of this enquiry will emerge from the study of the literature and correspondence received. For this reason, quite a number of the documents and research papers received (listed in the Appendix) may appear to have only a tenuous connection with a narrow concept of load-sharing. However, none that the author has so far had the opportunity to study are in fact unconnected with one or another concept or approach to the study of this complex field.

5. Many of the documents listed came to hand only recently and time was not available to study, let alone digest their contents. Moreover, some contain important references which have yet to be obtained; others, in Dutch are yet to be translated. Furthermore, whilst information has been solicited from the areas of Western and Central Europe, North America, Australia and New Zealand, no information from the U.S.S.R. or any of the East and South European countries is yet available.

6. For these reasons, no attempt will be made in this introductory report to analyse or discuss the substantial amount of information already gleaned. This is reserved for future reports. However, a few general comments can be hazarded already, although with the proviso that they may be subject to modification in the light of further study and information to be received.

7.1 Few of the European national codes appear to make any provisions for load-sharing in multiple member systems. The exceptions are the British, Norwegian and Dutch codes, which contain very simplified and strictly limited stress increase factors for closely spaced parallel member systems (the Dutch only for concentrated loads on joisted floors, but with a somewhat more sophisticated formula which brings in consideration of joist spacing and board stiffness).

7.2 Many European codes are now undergoing revision and it may be opportune to attempt a co-ordination of efforts and a search for an identical approach.

7.3 The Australian timber engineering code contains more elaborate provisions for 'parallel support systems effectively connected' and allows higher stress modification factors ranging from 1.14 for 2 'effective members' to 1.33 for 10 or more members. There are separate provisions for systems of laminated members and for grid systems, (i.e. widely spaced beams supporting joists or decking).

7.4 In America, the ASTM tentative recommended practice for determining design stresses for load sharing lumber members (D 2018 - 62 T), produced in 1962, developed a statistical approach based largely on the 'random products' method, and suitable for any number of members and any defined population of timbers. As a simplified measure it recommended a 15 per cent bending stress increase for a system of 3 or more members spaced at most 24 inches apart. This was withdrawn in 1970 for lack of experimental evidence.

7.5 In the light of the potential economics due to the application of methods of design which recognise the effects of load sharing, it is surprising that until quite recently relatively little research effort has been devoted to this field of enquiry, particularly in Europe. This may be due in part to the complexities entailed by the rational analysis of composite systems involving extremely variable materials. Recent work at some centres, and particular mention may perhaps be made of the Colorado State University, has demonstrated that elegant theoretical solutions can be developed which can closely predict controlled test results.