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CANADA	
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DENMARK	
M Johansen	Statens Byggeforskningsinstitut, Horsholm
H J Larsen	Instituttet for Bygningsteknick, Aalborg
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U Saarelainen	Technical Research Centre of Finland, Helsinki
FRANCE	
P Crubilé	Centre Technique du Bois, Paris
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K Möhler	Universität Karlsruhe (TH), Karlsruhe
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H Kolb	Otto-Graf Institut, Stuttgart
HOLLAND	
J Kuipers	Technische Hogerschool, Delft
NORWAY	
O Brynildsen	Norsk Treteknisk Institute, Oslo
SWEDEN	
B Norén	Svensak Traforskingsinstitutet, Stockholm
UNITED KINGDOM	
L G Booth	Imperial College, London
H J Burgess	TRADA, High Wycombe
W W L Chan	Consulting Engineer, London
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
T Williams	Hydro-Air International, High Wycombe

(1) Secretary CIB-W18
(2) Co-ordinator CIB-W18 and Chairman for meeting

2 CHAIRMAN'S INTRODUCTION

Mr SUNLEY, as Co-ordinator of CIB-W18 and Chairman of the meeting, welcomed the delegates to the meeting which was the fifth since the group was reformed in March 1973. He outlined the programme for the meeting, pointing out that because of the large number of topics on which the group was working at present it may be necessary to defer discussion on some of them until the next meeting. The delegates accepted this and agreed the programme for the meeting.

Mr SUNLEY introduced a paper "The Work and Objectives of CIB-W18 - Timber Structures" (CIB-W18/5-105-1) which sets out the terms of reference of the group and describes the current programme of work. The paper also describes the relationships between CIB-W18 and other international organisations working in the same field and draws attention in particular to the co-operation of CIB-W18 with the Joint Committee on Structural Safety to draft a unified systems of international codes covering all materials.

Mr SUNLEY also informed the meeting of a symposium which was being organised by the State Committee of the Council of Ministers of the USSR for Building Affairs (Gosstroy, USSR). The title of the symposium was "Design, Production and use of Timber Structures in Building" and it is to be held 7 to 12 June 1976 in Kiev. Mr SUNLEY said that although CIB-W18 had not been consulted about the organisation of the symposium it did appear that it was to be promoted as a CIB-W18 event and he hoped to be able to attend. He said that at present there was very little active participation in CIB-W18 by Eastern Europe and he hoped that the symposium would generate a greater interest. He understood that all members of CIB-W18 would be invited to attend.

3 DESIGN STRESSES FOR PLYWOOD

Dr BOOTH introduced his paper "The Determination of Design Stresses for Plywood in the Revision of CP 112" (CIB-W18/5-4-1) which describes the problems involved in deriving a consistant set of design stresses for use in a limit state design method, for the various types and species of plywood which are available at present. Dr BOOTH said that the problems arose because plywood was available from many sources and the test data provided by these sources was not strictly comparable as there was not an internationally recognised set of standard test methods. This led to an unacceptable variation on the design stress derived from the various test data. He said there was a particular need for information on sampling techniques for test specimens, the effects of specimen size and the effects of defects in relation to specimen size. Dr BOOTH drew attention to Table 2 in his paper which gave the strengths, stiffnesses and stresses for different types and species of plywood relative to Douglas fir plywood. This shows an apparent large variation in these properties which in Dr Booth's opinion is difficult to believe and contrary to his experience.

Dr WILSON commented that tests on small clear plywood specimens do not reflect a true picture of large panels containing the usual defects and therefore tests on large panels were essential.

All delegates agreed that there was a serious problem concerning the types of plywood in use at present and the available test data. Dr BOOTH suggested that manufacturers might be persuaded to carry out new tests using standard methods similar to those used in the most recent Canadian tests.

Dr NORÉN said he had a Finnish report which discussed the effect of specimen size and he would send a copy to Dr Booth.

Mr SUNLEY said there was obviously a clear need for a set of standard test methods for plywood and he reminded delegates of an earlier paper (CIB-W18/3-4-1) presented by Dr Kuipers on the subject. It was agreed that Dr Booth in co-operation with Dr Kuipers representing RILEM, and Dr Wilson would draft a set of standard test methods for plywood for the next meeting.

For information Mr SUNLEY drew attention to a draft proposal being considered by ISO/TC 139, Working Group 6 entitled "Veneer Plywood for Construction - Quality Specification" (CIB-W18/5-4-2).

Dr WILSON said the proposed method of presentation of the bending strength and stiffness results (Clause 13) was new and this was being investigated in Canada at present to see if it was acceptable

Dr BOOTH said that he was a member of WG6 which was concerned with specifications for structural plywood and another group WG5 chaired by Prof Noack was dealing with testing methods for plywood. The tests in the draft proposal were for the purpose of specifying plywood and were not intended for use to derive working stresses. Dr BOOTH thought both groups should consider tests on large size specimens and he suggested that WG6 may be prepared to do some work on this. He also proposed that when CIB-W18 and RILEM had agreed a set of standard test methods for plywood they should be submitted to ISO/TC 139 for approval. This was agreed and Dr Booth undertook to inform WG6 on these matters.

Closing the discussion on plywood Mr SUNLEY said that for the next meeting he hoped to have a paper putting forward a method for obtaining characteristic stress values for plywood which would deal with sampling techniques and the statistical evaluation. This would be in addition to the paper on test methods for plywood.

4 STRESSES FOR SOLID TIMBER

Mr CURRY introduced his paper "Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes". (CIB-W18/5-6-1) and he thanked Dr Kuipers for the information he provided for the paper. He said it was his hope to create a data bank of test results obtained using the test methods described in the paper.

Mr SUNLEY asked if the delegates agreed that when the paper was finally accepted after consultation with RILEM, it should be submitted to ISO/TC 55. This was agreed but delegates were anxious that ISO/TC 55 should appreciate the urgent need for standard test methods for structural size timber. Mr SUNLEY said that at a meeting of ISO/TC 55 in June 1974 it had been agreed work on this was necessary.

Dr NORÉN said that the paper which was finally submitted to ISO/TC 55 should not contain all the details included in the present paper. He also requested clarification of the differences between the ASTM test methods and those proposed by Mr Curry. Mr CURRY replied that the present proposals required that the critical zone in the test piece was located in the most critical position for the test. This was not necessarily the case with the ASTM method.

Mr SUNLEY suggested that the delegates should go through the paper systematically and the following comments were made:

Clause 1 It was agreed that "solid rectangular sections" did not include laminated timber.

Prof MOHLER and Dr KUIPERS said they would like sufficient information to give the full stress/strain curve for each test piece. Prof LARSEN said that the tests were designed for a large testing programme and it would not be practical to give the stress/strain curve for each test piece. Mr CURRY and Mr SUNLEY agreed with Prof LARSEN.

Clause 2.2 Mr CURRY drew attention to the requirement that each test specimen should be selected so that the critical zone was at the centre.

Clause 3.1 Mr SAARELAINEN suggested that a complete moisture content history of each test piece should be included. However it was thought that this might not always be possible and it was agreed therefore to add "Any other relevant information" to the list.

Clause 3.3 Dr NORÉN said that he would prefer moisture content to be expressed as a ratio rather than a percentage. Mr CURRY said that in ISO papers moisture content was expressed as a percentage.

Prof MOHLER asked if the 500 mm limit was necessary or if 300 mm would be adequate. Mr CURRY said that he thought the 500 mm limit was an ISO recommendation although 300 mm would probably be sufficient. It was agreed that Mr Curry would check this with ISO and if this was not correct the 300 mm limit would be adopted.

Clause 3.4 Prof LARSEN preferred density to be expressed as a relative density ie a dimensionless ratio. Mr CURRY replied that this departed from the ISO recommendation but it was agreed to use relative density.

Clause 3.5 It was agreed to delete "nominal" and an alternative be substituted.

Clause 4.5 Prof LARSEN said that if the grading was tied to the ECE Standard, confusion would arise in the data bank if the Standard was changed. Mr CURRY disagreed saying that although the ECE grades might change the KAR method of measurement would remain and if the knot distribution was described using a grid system as shown in Fig 1 then the knot plot could always be reconstructed.

Clause 5.2 Prof LARSEN suggested that a full load/deflection curve would be costly and unnecessary as a single load/deflection measurement would give adequate information. Mr CURRY said that a definition of rate of loading was necessary and if a single load only was used it would be necessary to specify at what time after the application of the load, the delfection should be measured.

It was agreed that "± 25 per cent" should be added as a tolerance on the rate of loading and the formula for modulus of elasticity should be rewritten in general terms without the constant and the dimensions. The suffix T in the same formula is also to be omitted.

Clause 6.1.1 Prof LARSEN pointed out that many testing machines had fixed increments of adjustment and therefore the test spans specified should include a tolerance to allow for this. It was agreed to include a note to this effect. Dr NORÉN said he thought the rate of loading was too slow. It was agreed to rewrite the formulae for modulus of elasticity in general terms without constants and dimensions.

Clause 6.1.2 Prof LARSEN expressed doubts that the proposed test method would work as it involves the small difference between two large numbers. Mr CURRY replied that the method had been tried and found to work if sufficient care was taken and at present there was no alternative. Prof MOHLER said there were other test methods for shear modulus. One method involved tests at different spans related to depth of specimen and a second method involved torsional tests. He agreed to investigate these methods and report back to the next meeting.

Clause 6.2.1 It was agreed that the rate of straining should be specified before the rate of cross-head separation and a distinction should be made between gauge length and specimen length.

Clause 6.3.1 Mr CURRY said that there could be problems with this method due to buckling of the specimen but it may be possible to provide some sort of lateral restraint. He agreed to investigate this and report back to the next meeting.

Dr NOREN suggested that with the possible exception of the tension tests, all rates of strain could be increased to 0.003 although ASTM methods specify 0.001. Mr CURRY agreed to investigate this for the next meeting.

In conclusion Mr CURRY agreed to produce a second draft for the next meeting but he suggested that in the interim period tests should be carried out according to the present proposals.

5 DESCRIPTION OF TIMBER STRENGTH DATA

Mr CURRY introduced a paper "The Description of Timber Strength Data" (CIB-W18/5-6-2) by Mr J R Tory of Building Research Establishment, England and asked for comments.

Commenting on Table 3, Prof LARSEN said the tabulated values were not comparable because the confidence levels associated with each value were not the same. Mr CURRY agreed saying that it was doubtful if confidence levels could be established for the Weibull distributions. However the ASTM values were comparable as they were both related to a 95 per cent confidence level and some of the other values may have similar confidence levels although not the same as the ASTM level.

After a protracted discussion on how results should be expressed Mr SUNLEY suggested that the 5 percentile values should be given and each country would then be free to derive their own design stresses based on this value. Alternatively it may be possible to recommend stresses which are based on an arbitrary judgment and not necessarily compatible with the 5 percentile value.

Referring to the Conclusions (page 7) Prof LARSEN proposed that (2) and (5) should be adopted with 5 percentile values, with options on (1) and (4).

Mr SUNLEY asked if the paper should be given wider circulation with a request for (2) and (5) to be adopted. It was agreed that Mr CURRY would send the paper to the ECE Timber Committee and other selected people in the amended form.

6 STRESSES FOR ECE STRESS GRADES

Mr CURRY introduced a paper "Stresses for EC1 and EC2 Stress Grades" (CIB-W18/5-6-3) by Mr J R Tory which he said was intended to give an indication of the design stresses which would be assigned to the recently adopted ECE stress grades.

Prof SONNEMANS drew attention to the tabulated lower fifth percentile values given in Table 2. He said that it appeared that the EC1 and EC2 grading was ineffective as the values to the V and VI quality grades did not correspond to the values assigned to the ECE grades eg EC2 grade timber selected from Vths had a higher stress than EC1 grade timber selected from VIths. Mr SUNLEY replied that when actual stresses were eventually assigned to EC1 and EC2 it would be necessary to choose values based on engineering judgment with qualifications on the methods of selection for the grades together with a weighting of the test results.

7 TESTING OF TIMBER JOINTS AND FASTENERS

Prof MOHLER introduced a paper "Influence of Loading Procedure on Strength and Slip Behaviour in Testing Timber Joints" (CIB-W18/5-7-1) together with an additional note giving details of the most recent test results on joints made with integral nail plates, wood screws, and plain shank nails. Prof MOHLER said that his work suggested that the most suitable loading procedure was one which included an unloading section as proposed by the RILEM 3TT Committee. This was described in a paper (CIB-W18/4-7-1) presented by Dr KUIPERS to the previous CIB/W18 meeting in Paris 1975.

Mr WILLIAMS questioned the need for the unloading portion of the loading curve as according to Appendices 3, 4, 7, 8 and 10 of Prof Mohler's paper it did not appear to effect the general shape of the curve. Dr KUIPERS said that the unloading portion of the curve could be used to give a useful indication of the modulus of elasticity of the joint. This was followed by further discussion which concluded with agreement that one unloading cycle should be included.

Continuing on from Prof Mohler's paper Dr KUIPERS introduced a paper from the RILEM 3TT Committee "RILEM Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures - 5th Draft" (CIB-W18/5-7-2). The comments on the paper were as follows:-

Clause 1.2 This is more suitable in a document dealing with the analysis of test data. Therefore it is recommended that it be deleted.

Clause 4 Mr SUNLEY drew attention to the difference which existed regarding the timber sizes used when testing joints made with integral nail-plates. Denmark, France and UK made the test joints with the minimum standard timber size consistant with the size of the plate under test. However, in Germany the timber was machined to a size related to the actual size of the plate.

Dr KUIPERS pointed out that Comment 7.1b appeared to suggest a preference for the minimum standard size of timber to be used.

Prof MOHLER asked if there was any real difference between the tests shown in Fig 1(a) and Fig 1(b). Dr KUIPERS said no, the direction of loading can be changed without affecting the test result, but some people found one test easier to perform than the other and therefore both types of test were needed. Prof LARSEN suggested that Fig 1 could be omitted and included in another document which gave full details of test specimens. However, Dr KUIPERS and Dipl Ing KOLB wanted it retained. Finally it was agreed that Clause 4 should only contain a sentence which required test joints to accurately reflect real joints and then reference should be made to an Appendix for the details of actual test joints.

Clause 5.1.2 It was agreed that a constant rate of loading should be used up to "7f" after which a constant rate of strain could be adopted if preferred. This would result in a test duration time of between 5 and 10 minutes. However for tests on joints with integral nail plates the loading should be in accordance with the German proposals which will be submitted to the next meeting of CIB-W18 by Prof MOHLER.

Clause 5.1.7 It was agreed that this should be eliminated as it was not necessary.

Clause 5.1.8 Prof LARSEN and Dr NORÉN said the definition of ultimate load should not be in a document on test methods. Therefore the clause should be rewritten to say that it is permissible to stop the test and record the final load after a slip of 15 mm has occurred.

Clause 5.2.2 Dr BOOTH and Dr CHAN suggested that the duration of the long term test should be defined and it was agreed that 90 days was a suitable period of time after which the joints could be considered satisfactory.

Dr CHAN also suggested that shorter load duration tests could be used with correspondingly higher loads but this was not accepted as it was thought that because of the variability of the test joints some would fail much earlier than others.

Clause 5.3 Prof MOHLER agreed to send Dr Kuipers details for dynamic tests if possible, otherwise reference to be made in Clause 1 "Scope" that dynamic tests are not included.

Clause 7.1 Recommended that it be deleted.

It was agreed to leave discussion of the climatic conditions included in the Commentary (page 6) as these were the subject of a paper to be discussed later.

In conclusion Dr KUIPERS undertook to report back to the RILEM 3TT Committee and they would provide an amended draft for agreement at the next CIB-W18 meeting, after which it would be published. Mr SUNLEY agreed with this and said that when the paper was finally published he would recommend that RILEM should submit it to the relevant ISO committee for approval and status.

Prof MOHLER agreed to draft an Appendix to the paper which would deal with the testing of integral nail plates.

Finally on this subject Dr KUIPERS introduced a paper "CIB - Recommendations for the Evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures" (CIB-W18/5-7-3). He said that this paper, which dealt with the analysis of test results, was complementary to the previous paper which dealt with test methods and he asked delegates for their comments. However due to lack of time discussion was not possible and Dr NORÉN, Dr KUIPERS and Mr JOHANSEN agreed to produce a further paper for the next meeting which would define the objectives and the methods to be used to achieve them.

8 LONG TERM LOADING

Dr NORÉN introduced a paper "Strength of a Wood Column in Combined Compression and Bending with Respect to Creep" (CIB-W18/5-9-1) together with an accompanying note on the paper. However due to shortage of time no discussion was possible but delegates were asked to write to Dr Norén with their comments. In addition it was agreed to circulate the paper and note to members of the IUFRO Timber Structures Group with a request for them to write to Dr Noren with their comments and this would be followed up with a discussion of the paper at the IUFRO meeting in June 1976.

Three other papers on long term loading were also submitted but due to a shortage of time it was agreed to hold these over to the next meeting.

9 TIMBER BEAMS

Prof LARSEN introduced his paper "The Design of Timber Beams" (CIB-W18/5-10-1) and suggested that the delegates should go through the paper commenting as necessary.

Clause 3 — Mr SUNLEY said he thought it needed to be stated that the permissible stresses are based on the elastic theory of bending.

Fig 3.01 — Mr SUNLEY asked if everyone agreed that the true stress distribution was a curve as shown in (b) and not linear as normal beam theory assumes. Mr CURRY pointed out that defects in the timber such as knots would cause a deviation from the curve. Dr NORÉN said there was no statement of when this stress distribution occurred ie at the ultimate load or earlier.

Clause 3.2 — It was agreed that there was a depth or size effect but that the Newlin and Trayer approach was no longer satisfactory. Mr CURRY said that experimental work was reouired to determine whether it was a depth effect or a size effect. At present work had been carried out by Finland, Sweden and UK and also between Canada and UK. This suggested that it was a depth effect although the size effect was not properly investigated as the range of widths covered was not sufficiently large. However there was general agreement that the true effect was due to size but it was probably adequately dealt with by assuming a depth effect.

Dr NOREN suggested that whichever effect it was it could be taken into account by modifying the stress grading rules. However it was agreed that this would be difficult in machine grading and very difficult in visual grading. Furthermore the new ECE stress grades do not take this effect into account. It was therefore agreed that it should not be included in stress grading rules. Prof LARSEN recommended the approach put forward by Bohannan (formula 3.03) which was related to a base depth of 200 mm. He said he preferred this to the French treatment which he thought overcompensated - see Fig 3.02. Dr BOOTH suggested that it was necessary to specify a minimum depth.

Clause 3.3 Prof LARSEN said it was necessary to decide if different stresses were required for circular sections and other odd shapes. It was pointed out that this was partially covered by special grading rules and it was generally agreed that form factors for odd shapes were not required.

Clause 3.4 Prof LARSEN recommended the adoption of formula 3.07 with ratio $^E/_G$ put equal to 16 which results in formula 3.10. However other delegates thought $^E/_G$ should be put equal to 20 therefore it was agreed to put the ratio $^E/_G$ in the formula and allow engineers to assign their own values. Everyone agreed to use the Hooley and Madsen approach until further work gives a better method. It was also agreed that for European timbers $^E/_G$ would be set at 20.

Clause 3.5 It was proposed that the ISO/TC 98/SC4 recommendations on deflection should be adopted. Dr BOOTH asked what values should be used for modulus of elasticity. Prof LARSEN recommended the use of the mean E for deflection calculations for the serviceability limit state but a lower value (probably the 5 percentile value) for calculations of strength and collapse conditions. There was disagreement on this as the serviceability limit state was considered to be as important as the collapse limit state and therefore the mean E should not be used for serviceability deflections but a lower value (maybe the 5 percentile value). Mr SUNLEY pointed out that Volume I in the Unified System of Structural Codes produced by the Joint Committee on Structural Safety (JCSS) seemed to imply that the 5 percentile value should be used for all deflection calculations.

The next factor to be considered was the way in which shear deflection should be dealt with. The choice was between using true E values and calculating shear deflection separately or using a modified E values and a separate calculation made for shear deflection. However for a common range of sizes and spans it would be sufficient to assume the shear deflection was 10 per cent of the bending deflection. This was agreed.

Clause 4.2 Prof LARSEN said that the approach adopted by North America and Norway should be discarded but there definitely was a size effect in shear which may or may not need to be taken into account. He suggested this could be achieved by using factors related to the sheared areas. Dr BOOTH asked what was the base area to which the factors should be related if this approach was used. Prof LARSEN replied that the factors in Table 4.04 were based on engineering judgment and were probably suitable for shear stresses developed using the ASTM shear test method.

Clause 4.4 Prof LARSEN said that the formulae for cases 4.04 (a) and 4.04 (b) (ie 4.12) were satisfactory but the formula for 4.04 (c) was not well founded. Therefore he would recommend the adoption of formulae 4.12 and 4.14. He also thought that the use of bolts (case 4.04 (d)) was very dubious and should be avoided. Replying to a question from Mr Sunley, Prof LARSEN said that formula 4.12 was satisfactory for beams with notches cut to half their depth. However he thought the use of formula 4.15 was probably not worthwhile as it was complicated and the benefit it gave was small. This was generally agreed. Dr BOOTH raised the question of notches which occurred within the length of the beam. Prof LARSEN replied that they should be designed on the same basis as notches at the ends of the beam although it would be necessary to calculate the bending stesss at the notch on the reduced section. Dr BOOTH questioned this approach and Prof LARSEN agreed to look into the need for modification factors for these notches and decide whether or not to introduce a limit.

Clause 4.6 Prof LARSEN said that as all the curves were close, Fig 4.09, it would be reasonable to adopt a smooth curve. Dr BOOTH asked if it was possible to calculate deformation at the bearing with force perpendicular to the grain. Prof LARSEN replied that it was and he would include it.

Clause 5.1 Prof LARSEN said none of the present rules for glulam design were satisfactory because finger joints in laminations are random and have a bigger effect than defects in the laminations. Furthermore when glulam beams are tested they generally are found to have inadequate factors of safety. Therefore Prof LARSEN suggests that any of the design methods described in the paper could be used as an interim measure until a better design method was evolved.

Dipl Ing KOLB said that it had to be accepted that the laminations would contain random placed finger joints therefore design stresses should be reduced to take account of this. Prof SONNEMANS said that tests carried out over a period of six years suggested that the finger joint problem was really one of quality control. It was agreed that further discussion on this would be left over to the next meeting when it was hoped to have information from the FEMIB committee on glulam.

Clause 5.2(a) Referring to Fig 5.03 Prof LARSEN said that curvature should be taken into account and this was agreed. He said that whichever curve was adopted they should both come to 1.00 at same value and he suggested this should be when the ratio r/t was equal to 200 → 300. Finally it was agreed to adopt the Wilson curve as this was based on work on large beams.

Clause 5.2(b) Prof LARSEN said that this effect was not dealt with in most codes as it is really a problem in structural mechanics. Dr NORÉN said that this should not be dealt with in a timber code. Prof LARSEN suggested that the delegates should agree on a formula but not necessarily include it in a timber code and he would like to use formula 5.02. This was agreed.

Clause 5.2(d) It was agreed to use the Barrett, Foschi and Fox approach and Prof LARSEN undertook to confer with Prof Mohler on a suitable interaction formula. Dr BOOTH agreed to provide information on a range of parameters greater than those investigated by Barrett, Foschi and Fox.

Clause 5.2(e) Prof LARSEN said that there was no sound basis for the interaction formula 5.04 and he recommended that it should not be used. Prof MOHLER agreed with this and it was decided to ask the IUFRO Timber Structures Group to carry out some research work to derive suitable interaction formulae.

Clause 6.1 Prof LARSEN said there was no need to put this formula in the code as a reference could be made to a standard text book or an appendix could be added to the code. Prof LARSEN said the formulae were based on panels which were pin jointed round the edges but this was not really correct in the practical situation. Dr KUIPERS said that some research which had been carried out suggested that the pin jointed approach was sufficiently accurate. It was agreed that buckling should be dealt with in the timber code.

Clause 6.2 It was agreed to carry out designs based on compression in the top flange and tension in the bottom flange with either one being the limiting criterion.

Clause 6.3 Dr KUIPERS agreed to provide information on the deformation which occurs across the joint between the web and the flange. Dr BOOTH requested information on the effective area of joint between flange and web which transferred the stresses from one component to the other.

In conclusion Prof LARSEN undertook to draft a suitable section on the design of beams for timber code for the next meeting.

10 CLIMATIC GROUPS

Dr NORÉN introduced his paper "Climatic Grading for the Code of Practice" (CIB-W18/5-11-1)

Mr SUNLEY, referring to the grouping defined on page 5, asked if the division of group 1 into two subgroups was really necessary. Prof LARSEN said he thought there were advantages to be gained by adopting the two subgroups. Dr CHAN suggested that the definition of the groups should be in terms of temperature and humidity if they were to be meaningful for timber structures. This was agreed. Dr Wilson pointed out that group 3 extends from $>$18 per cent upwards, with no upper limit, which would have the effect of all buildings in this group being designed for the fibre saturation condition. He thought this would be unnecessarily severe on some structures such as swimming pools and ice rinks, which could be designed for less severe conditions than the fibre saturation condition.

Referring to the test conditions, Mr SUNLEY asked if the conditions 23°C, 50 rh which had recently been proposed by ISO were acceptable as a reference climate. Dr NOREN said he agreed with the ISO proposals and there was general support in Scandinavia for them, as the conditions which they would replace, 20°C, 65 rh, where more expansive to maintain because they required refrigeration plant. However it was generally agreed that 23°C, 50 rh would result in a low moisture content for timber test pieces and a protracted discussion followed on a more suitable rh condition. Finally it was resolved to recommend to ISO that for timber structures the reference climate should be 23°C, 60 rh and Mr SUNLEY agreed to inform ISO of the opinion of CIB-W18.

Prof SONNEMANS suggested that the tolerance referred to on page 7 was really a tolerance on the measuring instruments and not on the actual test conditions.

In conclusion Dr NOREN agreed to provide a further paper on this topic for the next meeting.

11 TIMBER COLUMNS

Prof LARSEN introduced a first draft of a section on the "Design of Solid Timber Columns" (CIB-W18/5-100-1) for the timber code. He pointed out that this represented the concensus of opinions expressed at previous meetings where papers on timber column design had been discussed He said that the proposed method of design was similar to the Dutch method but it allowed the factor of safety to remain constant or to vary.

Dr CHAN referred to the constants K_1 and K_2 in formula 3 saying that the formula could be simplified by putting $K_1 = 0$ and adopting a constant value for the ratio $^{Fc}/_{Fb}$ for all species of timber without seriously affecting designs. Prof LARSEN agreed to investigate the effect of putting $K_1 = 0$ and adopting a suitable value for K_2 assuming the ratio $^{Fc}/_{Fb}$ is a constant.

Dr CHAN also questioned the suitability of increasing the bending stresses by a constant 10 per cent to take account of eccentricity. He said this would not cover the most severe cases because the ratios of bending stress to compressive stress could vary of $0 \longrightarrow 1$. Therefore he proposed that the stresses should be calculated separately and combined using a suitable interaction formula taking into account the secondary bending moment.

Prof LARSEN agreed to produce a second draft for the next meeting.

12 CIB - TIMBER CODE OF PRACTICE

Dr BOOTH introduced a paper "A Draft Outline of a Code for Timber Structures" (CIB-W18/5-100-2) which he said had been written so that it would be compatible with the Unified System of Structural Codes proposed by the Joint Committee of Structural Safety (JCSS). To this end the format and clause numbering were similar to the concrete code which was to be included in the unified system. Prof LARSEN said that there were some serious inconsistencies in the concrete code and therefore it would be wrong to make the timber code similar. Furthermore it was not possible at this stage to decide on the lay-out of the code as this would depend on the contents. He thought that the draft should be used only as a check list of items to be considered for a timber code. This was agreed and the following comments made regarding the contents of the list:

Clause 1 General Requirements - This should include the sub-clause "Partial safety factors (γ_m)" which should be deleted from all the following sections where it occurs and within this sub-clause the "Moisture content" should be changed to "Climatic Conditions". A discussion followed on what factors should actually be included in the sub-clause on partial safety factors but as it was not possible to reach agreement it was decided to leave the matter until Volume I of the Unified System of Structural Codes was available.

Clause 2.2 This should include a reference to specification standards for laminated timber.

Clause 2.5.2 Mr WILLIAMS suggested that nail plate fasteners should be included as a separate item after the sub-clause on bolts.

After discussion it was agreed that the whole of clause 2.5 should come in Clause 3 - Design of Structural components and joints.

Clause 3.3.4.3 Permissible deflections - It was agreed that this should be in volume I and applicable to all materials.

Clause 3.4.1 Beams - The sub-clause 3.4.1.4 "Plywood box and I beams" and sub-clause 3.4.1.5 "Diagonally boarded" should be renumbered 4.1 and 4.2 respectively and placed under a new sub-clause 3.4.1.4 "Built-up". In addition the word plywood should be deleted together with sub-clause 3.4.1.3.2 "Mechanical".

Clause 3.4.2 Columns - Sub-clause 3.4.2 3.2 "Mechanical" should be deleted and sub-clauses 3.4.2.4/5 and 6 should be arranged under a new sub-clause 3.4.2.4 "Built-up" as sub-clauses numbered 4.1, 4.2 and 4.3 respectively. In addition the word plywood should be deleted.

Clause 3.4.6.4 Layered boards - This should be deleted.

Clause 3.4.6.5 Folded - This should be retitled "Folded constructions" and arranged as a new sub-clause 3.4.7 - The existing sub-clauses 3.4.7 "Shells" and 3.4.8 "Space frames" should be deleted.

Clause 4.6 Painting - This should be retitled "Protective or finishing treatments".

Dr CHAN suggested that a section should be added for fibreboards similar to the existing plywood section. He undertook to provide a paper for the next meeting making a case for the inclusion of fibreboard and detailing a check list which will be similar to that for plywood.

In conclusion Dr BOOTH agreed to provide a second draft of an "Outline of a Code for Timber Structures" for the next meeting.

13 INTERNATIONAL STANDARDS ORGANISATION

Prof LARSEN submitted a note by the Dansk Ingeniorforening commenting on the ISO report of the consultation with member bodies concerning ISO/TS/P129 - Timber Structures (CIB-W18/5-103-1). He said that the members of ISO had voted for the setting up of a new Technical Committee to deal with timber structures with the secretariat in Denmark. The matter had now gone to the planning committee. Mr SUNLEY said that it had been agreed that ISO/TC98 would only deal with principles applicable to all materials and other TCs would deal with specific materials. In this respect ISO/TC98 would deal with Volume I of the Unified System of Structural Codes, ISO/TC71 would deal with Volume II - Concrete Structures, and the proposed ISO committee for timber structures would deal with Volume VI - Timber Structures which it was agreed would be drafted by CIB-W18. The establishment of a new ISO committee to deal with timber structures was therefore essential if the timber code which CIB-W18 was drafting was to be accepted internationally.

14 FUTURE PROGRAMME OF WORK

Mr SUNLEY said following discussion with the IUFRO - Wood Engineering Group, it was proposed to hold a joint meeting with CIB-W18, 14-18 June 1976 in Aalborg, Denmark immediately prior to the IUFRO Congress in Oslo. Prof Larsen had kindly offerred to act as host to the joint meeting.which would be arranged so that the first half of the week was devoted to CIB-W18 topics and the second half to IUFRO matters. However members of both groups were invited to participate in all the activities of the week.

Mr SUNLEY said he had tentatively arranged with Dr Norén for the following meeting of CIB-W18 to be held in Stockholm in March 1977.

Mr SUNLEY reminded delegates that one of the papers presented at the present meeting (CIB-W18/5-9-1) was to be circulated to the IUFRO group for discussion in June 1976 and three further papers on long term loading had been deferred to the CIB-W18 meeting in June 1976. Mr SUNLEY also drew the delegates attention to two papers by Mr F H Potter, Imperial College, London - "The Prediction of Load-Deformation Behaviour in Axially Loaded Nailed Joints" and "Codes of Practice and the Load Deformation of Timber Joints". These had been circulated to members of CIB-W18 with a request for members to send their comments and any further information they had to Mr Potter who would consider amendments. The papers would then probably be included for discussion during the IUFRO meeting in June 1976.

Mr SUNLEY said that because of the large number of topics already under discussion in CIB-W18 he thought it advisable not to introduce any further topics for the next meeting, which could be devoted to bringing to a conclusion some of the present work. In this respect he said that at the next meeting it was hoped to agree draft sections for the timber code dealing with:

1 Timber Columns - Prof Larsen to provide a second draft of a code section on the design of solid timber columns together with a first draft of a code section on the design of spaced timber columns.

2 Timber Beams - Prof Larsen to provide a frist draft of a code section on the design of timber beams.

3 Testing of Joints and Fasteners - RILEM 3TT Committee through Dr Kuipers to provide a further draft of paper on methods of test for timber joints and connectors and Prof Mohler to provide an appendix to this paper on methods of test for integral nail plates.

4 Environmental Conditions - Dr Norén to provide a draft of code section defining different climatic groups applicable to timber structures.

In addition, further topics for discussion at the next meeting would be:

1 Long-Term Loading - Three papers held over from present meeting.

2 Plywood - Dr Booth, assisted by Dr Kuipers and Dr Wilson to provide a draft of standard test methods for plywood. Dr Booth to provide a further paper on the derivation of design stresses for plywood.

3 Stresses for Solid Timber - Mr Curry to provide second draft of paper on test methods for structural size timber.

4 CIB Timber Code - Dr Booth to provide further paper on "Outline of a Code for Timber Structures".

5 Joint Committee of Structural Safety - Discussion of Volume I of Unified System of Structural Codes.

Finally Mr SUNLEY thanked delegates for their participation in the meeting and Prof Mohler in particular for his generous hospitality and the interesting programme he had arranged.

15 PAPERS PRESENTED AT THE MEETING

CIB-W18/5-4-1	The Determination of Design Stresses for Plywood in the Revision of CP 112 - L G Booth.
CIB-W18/5-4-2	Veneer Plywood for Construction - Quality Specification - ISO/TC 139 - Plywood, Working Group 6.
CIB-W18/5-6-1	Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes - W T Curry.
CIB-W18/5-6-2	The Description of Timber Strength Data - J R Tory.
CIB-W18/5-6-3	Stresses for EC1 and EC2 Stress Grades - J R Tory.
CIB-W18/5-7-1	Influence of Loading Procedure on Strength and Slip Bebaviour in Testing Timber Joints. - K Mohler
CIB-W18/5-7-2	Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures- RILEM 3TT Committee.
CIB-W18/5-7-3	CIB-Recommendations for the evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures - J Kuipers.
CIB-W18/5-9-1	Strength of a Wood Column in Combined Compression and Bending with respect to Creep - B Källsner and B Norén.
CIB-W18/5-10-1	The Design of Timber Beams - H J Larsen.
CIB-W18/5-11-1	Climate Grading for the Code of Practice - B Norén
CIB-W18/5-100-1	Design of Solid Timber Columns - H J Larsen.
CIB-W18/5-100-2	A Draft Outline of a Code of Practice for Timber Structures - L G Booth.
CIB-W18/5-103-1	Comments on the Report of the Consultation with Member Bodies concerning ISO/TS/P129 - Timber Structures - Dansk Ingeniorforening.
CIB-W18/5-105-1	The Work and Objectives of CIB-W18-Timber Structures - J G Sunley.

CIB-W18/5-4-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

THE DETERMINATION OF DESIGN STRESSES FOR PLYWOOD IN THE REVISION OF CP 112

by

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KARLSRUHE - OCTOBER 1975

1 INTRODUCTION

This paper is a summary of an investigation undertaken on behalf of the Building Research Establishment with the purpose of providing data on plywood for the revision of CP 112 - The structural use of timber.

Strength data was submitted by plywood manufacturers in Canada, Finland and Sweden, and the main purpose of the report was to use this test data to derive design stresses in a Limit State format using the procedures recently adopted for solid timber, and to compare these stresses with those in current use in CP 112 (1971).

The acceptability of the resulting design stresses was to be measured against the following criteria.

- i the adoption of the stresses in actual practice should not result in structural components which, for no good reason, contain more plywood than existing components, and
- ii the characteristic stresses derived from the test data for the various species should, when compared, be realistic.

If these criteria were not satisfied through, say, large safety factors, inadequate sampling or inappropriate test specimens, it was a purpose of the report to consider alternative procedures of deriving design stresses and/or the undertaking of further tests.

At the time of writing no firm decisions have been made by the Sub-committee on the recommendations of the report and the main reason for submitting this paper to CIB W18 is to hear whether similar problems have been encountered in other countries and how these have been resolved.

Decisions are also still required on

- i the species and grades that will be included in CP 112
- ii the basis of the design stress presentation (full section, parallel plies, etc)
- iii the method of presentation (stresses and moduli, or strengths and stiffnesses)

None of these topics will be discussed in this paper.

2 CP 112 (1967) AND CP 112 (1971)

The first edition of CP 112 - The structural use of timber in buildings - was published in 1952 and made reference only to Douglas fir plywood manufactured in the USA. During the following fifteen years until the publication of CP 112 (1967), designers made little use of this data and relied mainly on Canadian literature for design methods and on British Columbian Douglas fir plywood for the majority of their structures. CP 112 (1967) recognised this fact and replaced American plywood by Canadian Douglas fir. At the same time Finnish European birch plywood and British structural plywood manufactured from certain tropical hardwoods were added to the Code.

The design stresses for North American plywoods have always been based on the parallel plies only theory and this method was adopted in CP 112 (1952). In the case of Finnish birch plywood, the design stresses submitted to CP 112 were based on the full cross-sectional area. Rather than specify Douglas fir on the parallel plies only and birch on the full cross-section the drafting committee of CP 112 chose to specify both plywoods and British hardwood plywood on the full cross-sectional area.

The design stresses given in CP 112 (1967) used the nomenclature grade stress, where the grade stress was defined as "the stress which can safely be permanently sustained by plywood of a particular grade". Hence the grade stress was applicable to long-term loading. Dry grade stresses were defined at 18 per cent moisture content. The permissible stress (to be used in design calculations) was found by multiplying the grade stress by a series of modification factors which took into account the effects of duration of load, moisture content, etc.

3 LIMIT STATE PRINCIPLES

For the purpose of this paper the next edition of CP 112 will be referred to as CP 112 (Limit State).

3.1 <u>Concepts of Limit State design</u>: The concepts of Limit State design are now well known and although international agreement is steadily being reached on most aspects there are still variations of philosophy and nomenclature between countries. The adoption of Limit State procedures has now been formally agreed by the drafting committee of CP 112, but there are still many details to be resolved.

3.2 <u>Partial safety factors</u>: In the UK, the lead has been taken by concrete, and timber has found itself in the invidious position of being morally forced to accept the partial safety factors for loads adopted by concrete. Insufficient information was, and still is, available during the preparation of the Concrete Code to determine the partial safety factors on a strict probabilistic basis and they were, in general, determined by calibrating against the previous permissible stress code so that similar designs were produced by the two methods. In the case of timber it may be questioned if the same values of γ_f are appropriate.

Since the partial safety factors for strength for solid timber have already been determined by the sub-committee dealing with solid timber, plywood finds itself in much the same position relative to solid timber as did solid timber relative to concrete.

The partial safety factors for load are given in Appendix 1, which is an extract from the latest draft of CP 112. The design load is determined by multiplying the characteristic load (F_k) by the appropriate partial safety factor for loads (γ_f). Hence design load = $\gamma_f\ F_k$. Of the four design loads (long, medium, short and very short term) specified in Appendix 1, the long-term and the medium-term loads are the most frequently occuring loads on plywood (floors and shuttering respectively). In both cases the design load is of the form

$$1.4G_k + 1.6\ Q_k$$

where G_k and Q_k are the characteristic dead and imposed loads, respectively.

3.3 Characteristic stress: The characteristic stress for the grade should be determined by testing in-grade material. Test methods and the sampling of specimens are being considered by CIB - W18 and it is hoped to adopt their recommendations. The sample should be of sufficient size for a meaningful statistical analysis to be undertaken for the characteristic stress to be determined as the 5 percentile value. Further information is required before we can recommend the best distribution to be adopted and, in the absence of any evidence to the contrary, a normal distribution was assumed throughout the report.

The duration of a laboratory test will be about 5 to 10 minutes and may be considered as short-term loading. Little information is available on the effects of duration of load on plywood and it is assumed that the effects are the same as for solid timber. The long-term ultimate stress is assumed to be found by dividing the short-term ultimate stress by 1.6. It is assumed that duration of load has no effect on the moduli of elasticity and rigidity.

Test specimens taken from mill production will usually have a moisture content in the range 7 to 10 per cent. "Dry" service conditions will usually be about 15 per cent moisture content and characteristic stresses at this level may be found either by conditioning the specimens to 15 per cent or by testing at 7 to 10 per cent and applying a modification factor found from a special test programme; the latter approach is usually adopted. In CP 112 (Limit State) dry service conditions for plywood have been defined as 15 per cent moisture content. (For solid timber the same environmental conditions will give 18 per cent moisture content).

3.4 Design stress: The design stress is determined by dividing the characteristic stress by the partial safety factor γ_m. For **plywood**, the same value of γ_m adopted for small clear specimens of timber has been used ie $\gamma_m = 1.15$. Although there is strong evidence to indicate that the size of the specimen influences both the mean and the standard deviation, and hence the characteristic strength, the evidence is insufficient to recommend values of γ_m which depend on size. The value of 1.15 was adopted for all sizes of specimen and for all species, but there is no doubt that this is incorrect.

Hence we may summarise for plywood as follows:

$$\sigma_{gd} = \frac{\sigma_{gk}}{1.6\gamma_m} \text{ and } E_{gd} = \frac{E_{gk}}{\gamma_m}$$

where σ_{gd} (E_{gd}) = grade design stress (modulus of elasticity) for long-term loading

σ_{gk} (E_{gk}) = grade characteristic stress (modulus of elasticity) for short-term loading as calculated from laboratory tests on either large or small test specimens.

γ_m = partial safety factor. A value of 1.15 is used for stress and 1.0 for moduli.

Hence $\sigma_{gd} = \dfrac{\sigma_{gk}}{1.84}$

$E_{gd} = E_{gk}$

4 STRENGTH TEST DATA

The species of plywood considered were as follows:

Canadian:	Douglas fir (faced) plywood
:	Softwood plywood (CSP)
Finnish :	Birch plywood
:	Combi (birch faced) plywood
Swedish :	Structural plywood (P40 and P30)

It should not be assumed that all the above apecies will be presented in the Code in the same manner. Nor should it be assumed that the Code will be limited to the above species. These decisions have yet to be made by the drafting sub-committee.

The form and scope of the test data submitted by the countries of origin of the plywoods varied greatly and this led to many intractable problems.

The most comprehensive information was supplied by the Council of Forest Industries of British Columbia on Douglas fir (faced) plywood (Smith, 1974): rather less, but adequate, data was submitted on their Canadian softwood plywood (CSP). The data had been obtained over several years by sampling in-grade plywood from various mills. The length and scale of test programme, although in many respects an asset in determining variability, was a disadvantage in that the specification of the veneers and their lay-up changed during the period of test: the analysis of the test results however took these changes into account. A significant feature of the test programme was the experiments on different sizes of test specimens to determine the most appropriate size for plywood with defects: these experiments led to the adoption of large test specimens (eg the bending specimen was 48 in x 48 in).

The COFI test programme represents the most extensive set of tests ever undertaken on a particular species of plywood and as such it has been used as a norm against which to consider the data submitted on other plywoods. In general the data on other species is based on small test specimens and considerably less data.

Data on Finnish Combi plywood is less extensive than that from Canada (Suominen, 1973: Kilpelainen and Suominen, 1973). The size of the sample is probably adequate. The tests were however made on small specimens (eg for 15 mm thick plywood the bending specimen is 480 mm x 50 mm) and it is very doubtful if the design stresses so obtained are a realistic measure of the strength of the material, as used in large pieces. Having made this reservation it must in fairness be emphasised that the Finnish industry was asked to supply test data using the small size specimens defined in BS 4512 - Methods of test for clear plywood. At the time of the request it was appreciated that small specimens were not ideal, but insufficient information was then available on the effect of specimen size. Even in the case of Douglas fir it is doubtful if complete information is now available and it is certainly not available for birch or birch-faced plywood. In the absence of knowledge on size effects in birch plywood it has been assumed that the test results give as good a measure of strength as the Canadian large specimens: this assumption is very doubtful.

No further data was supplied on birch plywood and the data originally supplied for CP 112 (1967) (Niskanen, 1963) has been re-analysed for a limit state format.

The extent of the data on Swedish structural plywood falls between the Canadian and Finnish data for some strength properties and for other properties it is very restricted. Routine quality control sampling is undertaken to check the bending and tension strengths of the three grades manufactured. The sample data submitted for one of these grades (P30) is probably adequate, but it would only be possible to allocate stresses to P40 (and P20) by exercising considerable engineering judgement. For the shear properties no systematic testing has been undertaken but although the data is restricted it is probably adequate to allocate design stresses. The quality control specimens may be described as medium size (eg the bending specimen is 1000 mm x 500 mm) and although no information is available on the effect of size on the strength of Swedish plywood it is reasonable to assume that the size gives an adequate measure of the strength in actual structural components (Noren, 1974).

From the above it will be seen that the method of sampling, the number of specimens tested, and the size of the specimens tested is different for each country of manufacture. In no case has the same test specimen been used and when it is remembered that the size of specimen has a significant effect on the mean and the variability of the ultimate stress (and hence on the characteristic stress), that this effect varies according to the species, and that there is insufficient data on these effects for all species, it will be appreciated that it is impossible to derive satisfactory design stresses from the test data. The analysis of the test results was however undertaken in the same way for each species and although the resulting design stresses may appear to be reasonable for each individual species, when a comparison is made between species the design stresses are open to doubt. These doubts will only be removed by a programme of comparative testing.

5 COMPARISONS BETWEEN CP 112 (1967) AND CP 112 (LIMIT STATE)

To examine the effect of the introduction of Limit State design methods a comparison of the strengths was made under various stress conditions. The full derivation of the comparative equations will be given for bending only: similar equations may be derived for other stress conditions.

Let the plywood be subjected to a bending moment M. To design according to CP 112 (1967) we find the required section modulus Z_{67} from

$$Z_{67} = \frac{M_{67}}{\sigma_g} = \frac{kW}{\sigma_g}$$

where M_{67} = bending moment caused by a total load W

σ_g = grade stress

k = constant depending on support conditions, type of load, span, etc.

To design according to CP 112 (Limit State) we find the required section modulus Z_{LS} from

$$Z_{LS} = \frac{M_{LS}}{\sigma_d} = \frac{k\ \gamma_f\ F_k}{\sigma_d}$$

where M_{LS} = bending moment caused by the design load

σ_d = design stress

γ_f = partial safety factor for loads

F_k = characteristic load

The dead and imposed load we have from clause 4.3.1 of Appendix 1.

$$Z_{LS} = \frac{k(1.4\ G_k + 1.6\ Q_k)}{\sigma_d} = \frac{k\ \bar{\gamma}_f\ (G_k + Q_k)}{\sigma_d}$$

where $$\bar{\gamma}_f = \frac{1.4\ G_k + 1.6\ Q_k}{G_k + Q_k} = \frac{1.4 + 1.6(Q_k/G_k)}{1.0 + (Q_k/G_k)}$$

G_k = characteristic dead load

Q_k = characteristic imposed load

Using the same nomenclature for loads we have

$$Z_{67} = \frac{k(G_k + Q_k)}{\sigma_g}$$

Therefore

$$\frac{Z_{LS}}{Z_{67}} = \frac{\sigma_g}{\sigma_d/\bar{\gamma}_f}$$

Hence to compare CP 112 (Limit State) and CP 112 (1967) we compare $\sigma_d/\bar{\gamma}_f$ and σ_g. There will be no change in the amount of plywood required if $\sigma_d/\bar{\gamma}_f = \sigma_g$ and more material will be required if $\sigma_d/\bar{\gamma}_f < \sigma_g$. The value of $\bar{\gamma}_f$ depends on the ratio Q_k/G_k, and hence $\bar{\gamma}_f$ lies in the range 1.4 to 1.6 (eg when $Q_k/G_k = 6$, $\bar{\gamma}_f = 1.57$). For many uses of plywood (eg shuttering and flooring) $Q_k >> G_k$ and a value of 1.6 for $\bar{\gamma}_f$ is a good approximation. The ratio Z_{LS}/S_{67} is maximum when $\bar{\gamma}_f$ is maximum and hence the use of $\bar{\gamma}_f = 1.6$ represents the worse case.

Hence to find the worst effect of the change to CP 112 (Limit State) from CP 112 (1967) we compare $\sigma_d/1.6$ and σ_g.

A comparison of birch in CP 112 (1967) and CP 112 (Limit State) can be made directly through σ_g and $\sigma_d/\bar{\gamma}_f$ since both are specified on the full cross-section and the lay-ups have not changed. Similarly we may compare Combi with birch in both Codes.

A comparison of Douglas fir in CP 112 (1967) and CP 112 (Limit State) cannot be made through σ_g and $\sigma_d/\bar{\gamma}_f$ since, firstly, σ_g is based on the full cross-sectional area and σ_d on parallel plies only, and secondly, the lay-ups have changed. The comparison must now be made of the moments of resistance through

$$m_{67} = \sigma_g Z_{67} \text{ and } m_{LS} = \frac{\sigma_d Z_{LS}}{\bar{\gamma}_f}$$

A comparison of Douglas fir with birch and/or Combi may only be made through the moment of resistance, since the Canadian stresses are based on parallel plies only and the Finnish on full cross-sectional area.

The stresses for Swedish structural plywood are based on parallel plies only.

Similarly expressions for stresses and stress resultants may be derived for tension and compression. The panel shear stress is always based on the full cross-sectional area and a comparison of τ_g and $\tau_d/\bar{\gamma}_f$ can be made in all cases: rolling shear and the modulus of rigidity may be similarly compared directly.

The deformations in bending, tension and compression can be compared directly through E_g and E_d if they have the same basis and if the lay-ups are the same for a given species. Otherwise comparisons must be made through the bending stiffness (EI) and the direct force stiffnesses (EA).

A comparison of all grades and stresses is not possible within the confines of this paper. The reason for this limitation is clear when it is remembered that CP 112 (1967) gave grade stresses for five grades of Douglas fir and one of birch: the plywoods could be sanded and unsanded, and dry or wet: the stress conditions were bending, tension and compression (parallel and perpendicular to the face grain), panel and rolling shear: deformation for each should be compared. In CP 112 (Limit State) there will be only one grade of Douglas fir plywood, but three new plywoods (Combi, Swedish, CSP), some with more than one grade, will be available.

The major limitation that has been adopted is to make a comparison of dry values and the main emphasis has been placed on bending parallel to the face grain and panel shear. Unsanded sheathing grade Douglas fir and sanded birch and Combi have been used. Strengths and stiffnesses have only been calculated for representative thicknesses.

6 CP 112 (LIMIT STATE)

6.1 Grade design stresses: The grade design stresses (σ_{gd} and τ_{gd}) and moduli (E_{gd} and G_{gd}) for each species have been determined from existing test data provided by the countries of manufacture of the plywoods. The method of sampling and the size of the test specimens were different in each case and as it is known that both factors influence the test results they should have been taken into account in the analysis. Correction factors are however not available and no attempt has been made to introduce arbitrarily determined factors.

At this stage only the dry values have been calculated: there is however adequate data available to predict wet stresses.

The most important properties of plywood are bending with the face grain parallel to the span (as used in flooring and shuttering), and panel and rolling shear (as used in webs and walls). The bending strength and stiffnesses have been plotted for the five species in Figures 1 and 2. The strengths and stiffnesses in bending, tension and compression have been calculated for one thickness of plywood (12.5 mm nominal) in Table 1 and have been compared with Douglas fir in Table 2. Table 1 also contains the shear values, which are applicable to all thicknesses.

It must be emphasised that the tabulated design stresses are those which are derived in a consistent manner from the available test data, but it is doubtful that, due to sampling and size effects, they are relatively correct. This problem, and an examination of their absolute values in a Limit State format, must now be discussed.

6.2 Comments on the calculated grade design stresses: Of the various test data available, that for Douglas fir is the most extensive and its use of large specimens probably gives the best measure of the strength in actual components. The other species will therefore be compared using Douglas fir as a norm.

The strength and stiffness of a plywood test specimen depends on the following:

i the strength and stiffness of the veneers

ii the lay up, which in turn depends on the ordering of the mixed species and the veneer thicknesses

and iii the effect of the defects, which in turn depends on the veneer specification, the specimen size and the sampling.

At the moment there is insufficient information to make reliable theoretical predictions of strength and an accurate comparison of the various plywoods could only be achieved through the same programme of testing. Since this is not available any comparison must be, to a large extent, subjective.

There is no reason to doubt that birch is the strongest plywood. It does however seem doubtful that the bending strength with the face grain parallel to the span is 2.25 times the value for Douglas fir (see Table 2), especially when the bending stiffness is only 0.81 times Douglas fir. These differences must, in part, be attributed to the fact that Douglas fir used large (48 in wide) specimens and birch small (2 in wide) specimens. Admittedly birch contains more veneers and less defects, but the differences still appear to be excessive. Similar comments could be made about the other properties. On the whole birch appears to be overrated with respect to Douglas fir.

Combi and birch used the same small specimens for most of the strength properties and should therefore be comparable. The test data showed that in most cases birch was, as to be expected, stronger than Combi: there were however inconsistent results with 5 ply in bending (parallel to span) and the panel shear results were suspect. Although the data could be accepted with some small changes as giving a reasonable relationship between the species it is doubtful if the Combi values are correct relative to Douglas fir. Combi appears to be overrated. If large specimens were used it is probable that the relationship between birch and Combi would also change.

Swedish plywood was tested using medium-size specimens and there may not be much difference if they were re-tested using the Canadian large specimens. The main query must be placed against the limited test data, which was in the main obtained from quality control tests. The knot specification is very different and comparison of the effect of defects is difficult. The Douglas fir faces are probably stronger than the Swedish redwood or whitewood faces, but the cores of the Douglas fir faced plywood may contain weaker species. Despite these imponderables, cursory examination of panels would lead one to expect that the Douglas fir would be stronger or perhaps equal to, but certainly not weaker than, the Swedish P30 grade. This is however a subjective view.

The test data on CSP is limited but the Canadian proposal of giving it the perpendicular to the grain values of Douglas fir is reasonable.

There is however a further difference between North America and Europe. It will be recalled that the parallel plies approach adopted in North America only gave the same basic stresses for plywood and solid timber if a factor K, which depended on the direction of the stress with respect to the span and the number of veneers, was applied to the bending stress. The ultimate moments found during the recent COFI test programme were modified by the factor K and were then reduced to parallel plies only stresses. In contrast the Swedish ultimate test moments are immediately reduced to parallel plies only stresses and in effect use a value of one for K in all cases. Similarly the Finnish ultimate moments are reduced to stresses (on the full cross-section) using a value of one for K.

Summing up, the overwhelming feeling is one of disquiet. The criterion, proposed in the Introduction, that the resulting grade stresses "for the various species should, when compared, be realistic" has not been satisfied. What to do about this failure is more difficult, but it will now be discussed against the general background of the derivation of characteristic and design stresses.

6.3 Comments on Limit State procedures:

6.3.1 Partial safety factors for loads (γ_f): The partial safety factors for the loads given in Appendix 1 have no effect on the relative position of the species and only on the absolute values of the sizes required in design. Comparison of past and future designs is dependent partly on the product $\gamma_f\gamma_m$ and since it may be argued that γ_f is independent of the material, the chosen values will not be discussed further.

6.3.2 <u>Determination of characteristic stresses</u>: The choice of the 5 percentile value for the characteristic stresses is now generally accepted as an arbitrary, but reasonable, procedure.

An examination of the distributions for bending stresses and modulus of elasticity was made for Combi and Douglas fir plywood.

The analysis of the Combi results was undertaken by PRL and "There is greater than 10 per cent probability that these sets of data originate from a normal population, and this is too large a probability to reject the hypothesis of normality". Similar comments may be made about the Douglas fir.

There would appear to be no reason for the hypothesis of normality to be rejected. The effect of the sample size should however be taken into account.

6.3.3 <u>Partial safety factor for strength (γ_m)</u>: The purpose of γ_m is "to adjust from the test conditions under which the strength was measured to conditions associated with the structure" (Sunley, 1974).

Tests are usually of short duration and if we wish to define the design stress as being appropriate for long-term loading we must reduce the test values. There is little data available on the effect of duration of load on plywood and it can only be assumed that the behaviour is the same as that of solid timber. The long-term value is obtained by dividing the short-term value by 1.6.

For small clear specimens of timber a value of 1.15 was taken for γ_m, the numerical value having been determined by calibrating CP 112 (1967) and CP 112 (Limit State). For plywood the same value of 1.15 has been used for stresses and 1.0 for moduli. An alternative approach would have been to determine a new value of γ_m for plywood by calibrating CP 112 (1967) and CP 112 (Limit State) for Finnish birch plywood, which is the only plywood unchanged in both codes. If this procedure had been adopted, either an average value of the coefficient of variation would have to be used or the value of γ_m would vary from strength property to strength property: also calibration can only have been achieved for one value of $\overline{\gamma}_f$. If we take $\overline{\gamma}_f = 1.6$, the γ_m becomes 1.22 and 1.31 for coefficients of variation of 10 and 20 per cent respectively. If Douglas fir and birch used the same values of γ_m, we would still have the same large difference.

There is however evidence to indicate that the size of the test specimen influences both the mean and the variability of ultimate stresses. It may therefore be argued that γ_m also depends on the size of the specimen and should be different for the Canadian large specimens and the Finnish small specimens. It could be argued that γ_m should be taken as 1.0 for Canadian test specimens and that a value of 1.15, or larger should be applied to birch and Combi. It is not suggested that this should be done at this stage but it is proposed that the principle of γ_m varying with test specimen size should be investigated.

7 CONCLUSIONS

7.1 Manufacture: All the plywoods submitted for consideration (Douglas fir, CSP, birch, Combi and Swedish structural) are covered by satisfactory manufacturing specifications containing satisfactory quality control procedures.

7.2 Test data submitted: The Canadian data on Douglas fir (faced) plywood is the most extensive and as such has been considered as the norm against which to measure the other plywoods. It is also the best measure of structural behaviour since it is based on large-size specimens. The data on CSP is less extensive, but is adequate in relation to the stresses proposed.

The Finnish data on birch and Combi is less extensive than the Canadian, but may be considered adequate. There are however doubts about the use of small-size specimens as a measure of structural strength.

The Swedish data has been obtained from a quality control programme and, although not extensive, probably gives a satisfactory measure of the ultimate bending and tension stresses. The data on modulus of elasticity is less satisfactory. The remaining properties have not been measured during quality control and have been determined from tests at various laboratories and at various dates: it is difficult to assess their consistency and the determination of design stresses for these properties requires considerable engineering judgement.

Satisfactory data on the effect of moisture content is available for all the species.

No information is available on the effect of duration of loading. It has been assumed to be as given by Madison for solid timber: recent work at UBC Vancouver by Madsen suggests this is not correct, but is conservative.

7.3 Limit State procedures:

7.3.1 Determination of characteristic values: On limited evidence, the distribution of strength values may be assumed to be normal. The choice of a five percentile value for strength is acceptable, but it may be questioned if the same value is appropriate for stiffness. The use of the Composite Dispersion Factor adopted by the Canadians is not recommended.

7.3.2 Partial safety factors for loads (γ_f): It is assumed that these are independent of material and are already predetermined.

7.3.3 Partial safety factor for strength (γ_m): A value of γ_m = 1.15 has been used for stresses and γ_m = 1.0 for moduli. It is recommended that the values of γ_m for both stresses and moduli should depend on specimen shape and size: at the moment this information is not available. Sampling procedures should also be included in γ_m or elsewhere.

7.4 Effect of adopting Limit State procedures in CP 112: It has been assumed in the following comments that γ_m is the same for all the plywoods and consequently the effects of specimen size and sampling have been ignored.

A true comparison of CP 112 (Limit State) and CP 112 (1967) can only be made for Finnish birch plywood: for Douglas fir the manufacturing specification has changed since 1967, and the other plywoods are new. For birch, at comparable levels, Limit State stresses will be higher than 1967 values by amounts which vary with the strength property (eg bending stresses are between 22 and 27 per cent greater, panel shear 12 per cent greater). The moduli are all decreased by about 10 to 15 per cent. (Figures 3 and 4).

Although for Douglas fir the comparison of Limit State and 1967 is not justified due to the change of specification it is of commercial interest. Bending strength and stiffness are both 7 to 27 per cent smaller: rolling and panel shear are 8 per cent smaller: shear modulus is up to 63 per cent smaller. (Figures 3 and 4).

The effect of introducing CP 112 (Limit State) will require less birch when stress is the active design constraint and more when deflection is active. For Douglas fir more plywood will be required in all conditions.

The order of strengths (based on the data submitted) depends on the property considered. In general birch is the strongest, followed by Combi, Swedish P30, Douglas fir and CSP. An engineering judgement of the relative strengths, using Douglas fir as the norm, is that birch and Combi are much too high, Swedish P30 is probably too high, and CSP is correct. The reason for these discrepancies can be attributed mainly to test specimen size and to a less extent to sampling.

Although consistently derived design stresses have been given for all the species, it is considered that if they are adopted they will contain inherently different factors of safety.

7.5 Future work: Before design stresses are included in CP 112 it is recommended that comparative testing should be undertaken on all the species using the same size specimens. Ideally all properties should be investigated, but tests on only bending and panel shear would provide useful guide lines.

In the long-term, work is required on:

i statistical distributions, especially for those properties with large coefficients of variation.

ii the effect of size of test specimens, and

iii duration of load effects at characteristic stress levels.

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Table 1

CP 112 (Limit State)

Comparative strengths, stiffnesses and stresses 12.5 mm (nominal thickness) plywood. Long term loads. 15% mc

PROPERTY		Douglas fir	Birch	Combi	Swedish P30	CSP
Net thickness (mm)		12.2	12.0	12.0	12.7	12.2
Bending strength	$\parallel$	186	418	372	188	158
(N. mm x 10^3/m)	$\perp$	81.3	317	276	127	81.3
Bending stiffness	$\parallel$	1362	1100	887	948	962
(N.mm^2 x 10^6/m)	$\perp$	296	706	518	327	296
Tension strength	$\parallel$	59.4	181	120	54.8	50.4
(N x 10^3/m)	$\perp$	33.6	139	93.7	43.2	33.6
Comp strength	$\parallel$	70.9	95.0	91.2	49.1	71.4
(N x 10^3/m)	$\perp$	47.6	70.2	56.8	34.6	47.6
Bearing						
Panel shear	$\parallel$	1.18	3.47	0.792	1.50	1.14
(N/mm^2)	$\perp$	1.14	3.47	0.792	1.50	1.14
Rolling shear	$\parallel$	0.340	0.965	0.462	0.38	0.332
(N/mm^2)	$\perp$	0.332	0.965	0.462	0.38	0.332
Modulus of rigidity	$\parallel$	431	716	175	405	283
(N/mm^2)	$\perp$	283	716	175	405	283

In the above table

Bending strength = $\dfrac{\sigma_{gd}}{\bar{\gamma}_f}$ Z with $\bar{\gamma}_f = 1.6$

Bending stiffness = E_{gk} I

Tension and compression strength = $\dfrac{\sigma_{gd}}{\bar{\gamma}_f}$ A with $\bar{\gamma}_f = 1.6$

Bearing = $\dfrac{\sigma_{gd}}{\bar{\gamma}_f}$ with $\bar{\gamma}_f = 1.6$

Panel and rolling shear = $\dfrac{\sigma_{gd}}{\bar{\gamma}_f}$ with $\bar{\gamma}_f = 1.6$

Modulus of rigidity = G_{gk}

Table 2

CP 112 (Limit State)

Strengths, stiffnesses and stresses relative to Douglas fir 12.5 mm (nominal) plywood.

Long-term loading. 15% mc

PROPERTY		Douglas fir	Birch	Combi	Swedish P30	CSP
Net thickness (mm)		1.00	0.98	0.98	1.04	1.00
Bending strength	$\parallel$	1.00	2.25	2.00	1.01	0.85
	$\perp$	1.00	3.90	3.39	1.56	1.00
Bending stiffness	$\parallel$	1.00	0.81	0.65	0.70	0.71
	$\perp$	1.00	2.39	1.75	1.10	1.00
Tension strength	$\parallel$	1.00	3.05	2.02	0.92	0.85
	$\perp$	1.00	4.14	2.79	1.29	1.00
Comp strength	$\parallel$	1.00	1.34	1.29	0.69	1.01
	$\perp$	1.00	1.47	1.19	0.73	1.00
Bearing						
Panel shear	$\parallel$	1.00	2.94	0.67	1.27	0.97
	$\perp$	1.00	3.04	0.69	1.32	1.00
Rolling shear	$\parallel$	1.00	2.84	1.36	1.12	0.98
	$\perp$	1.00	2.91	1.39	1.14	1.00
Modulus of rigidity	$\parallel$	1.00	1.66	0.41	0.94	0.66
	$\perp$	1.00	2.53	0.62	1.43	1.00

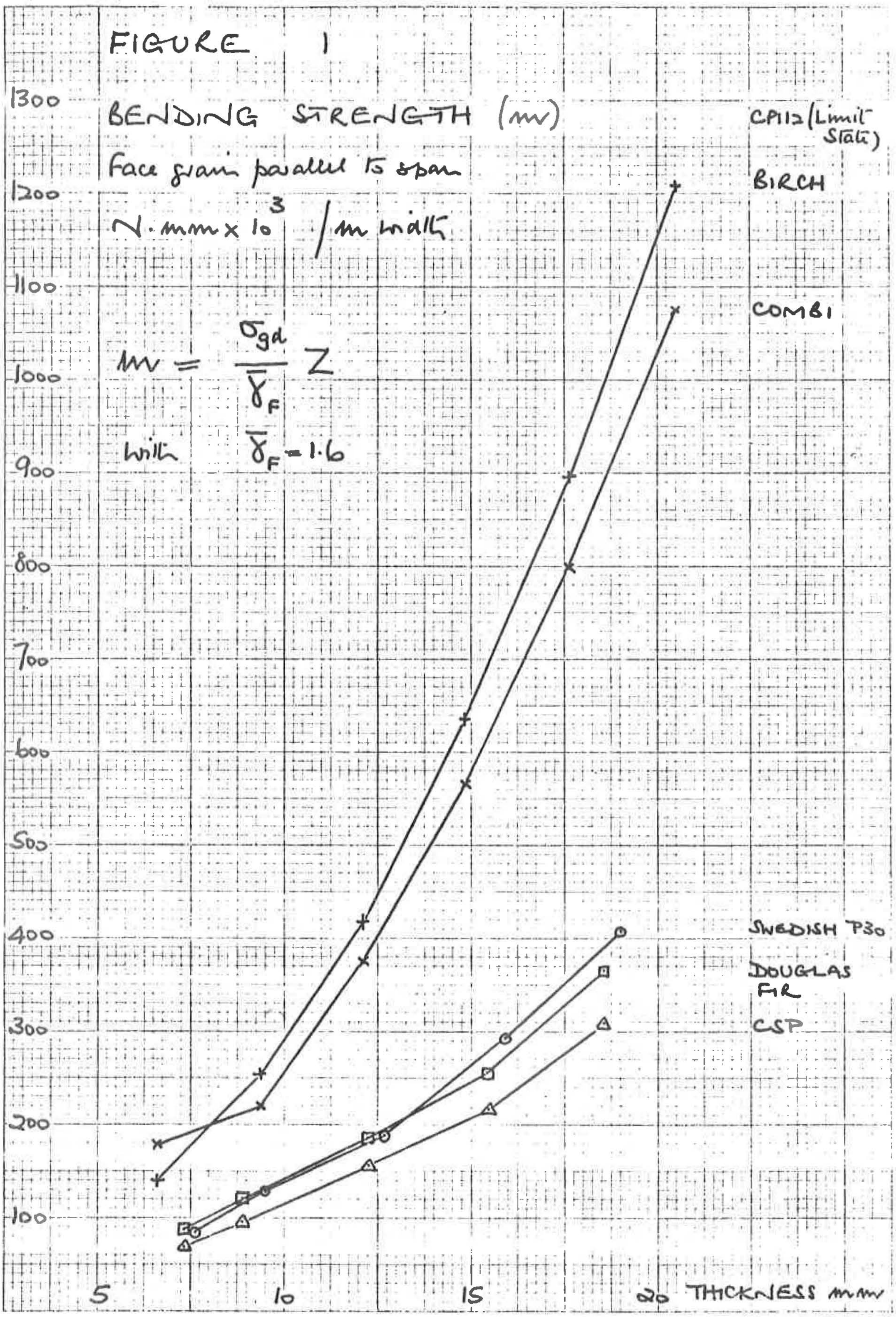


FIGURE 1
BENDING STRENGTH (mu)
Face grain parallel to span
N·mm x 10^3 / m width
mu = σgd / γF Z
with γF = 1.6
CP112 (Limit State)
BIRCH
COMBI
SWEDISH P30
DOUGLAS FIR
CSP
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
5
10
15
20
THICKNESS mm

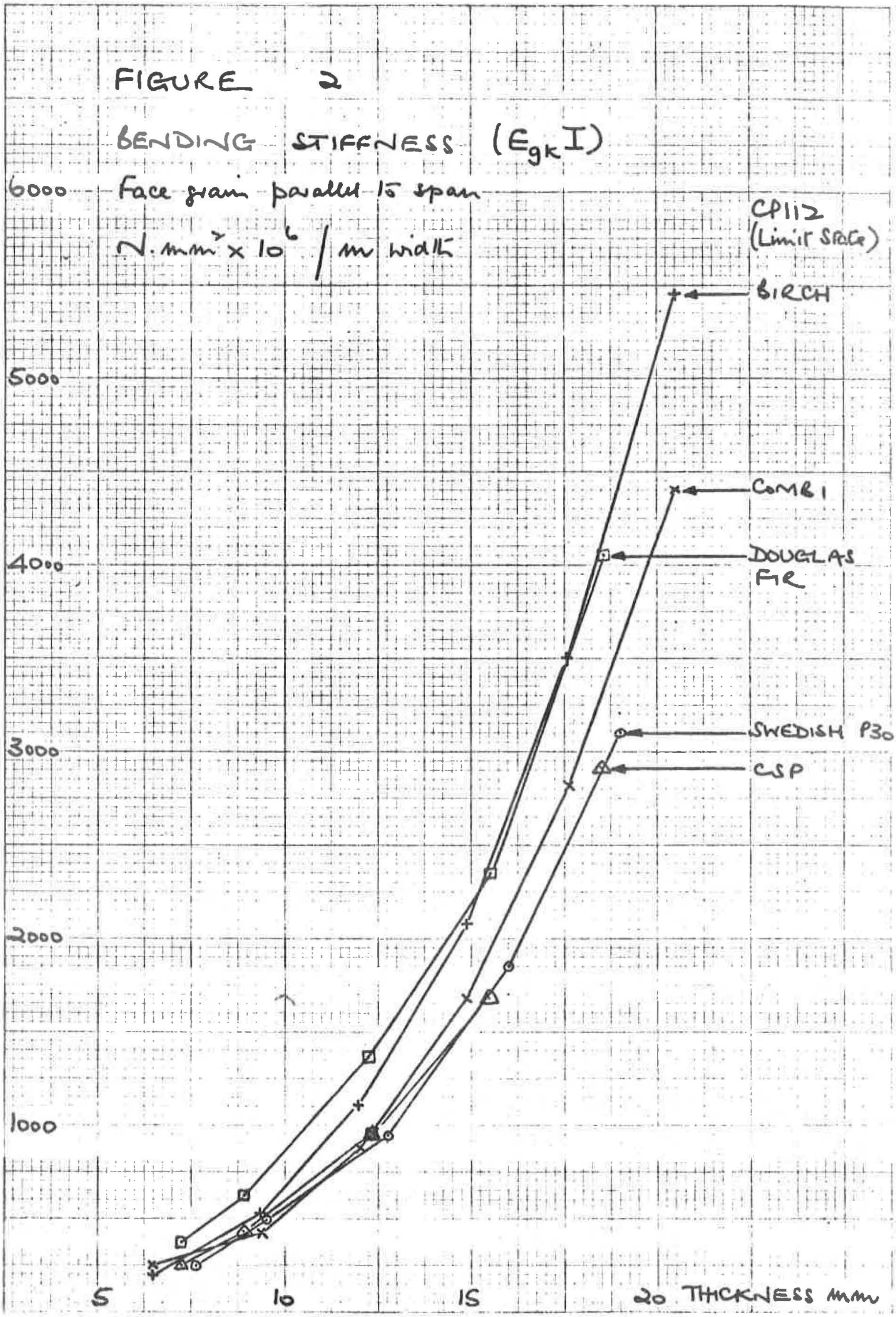


FIGURE 2
BENDING STIFFNESS ($E_{gk}I$)
Face grain parallel to span
$N.mm^2 \times 10^6$ / m width
6000
5000
4000
3000
2000
1000
5
10
15
20 THICKNESS mm
CP112 (Limit State)
BIRCH
COMB I
DOUGLAS FIR
SWEDISH P30
CSP

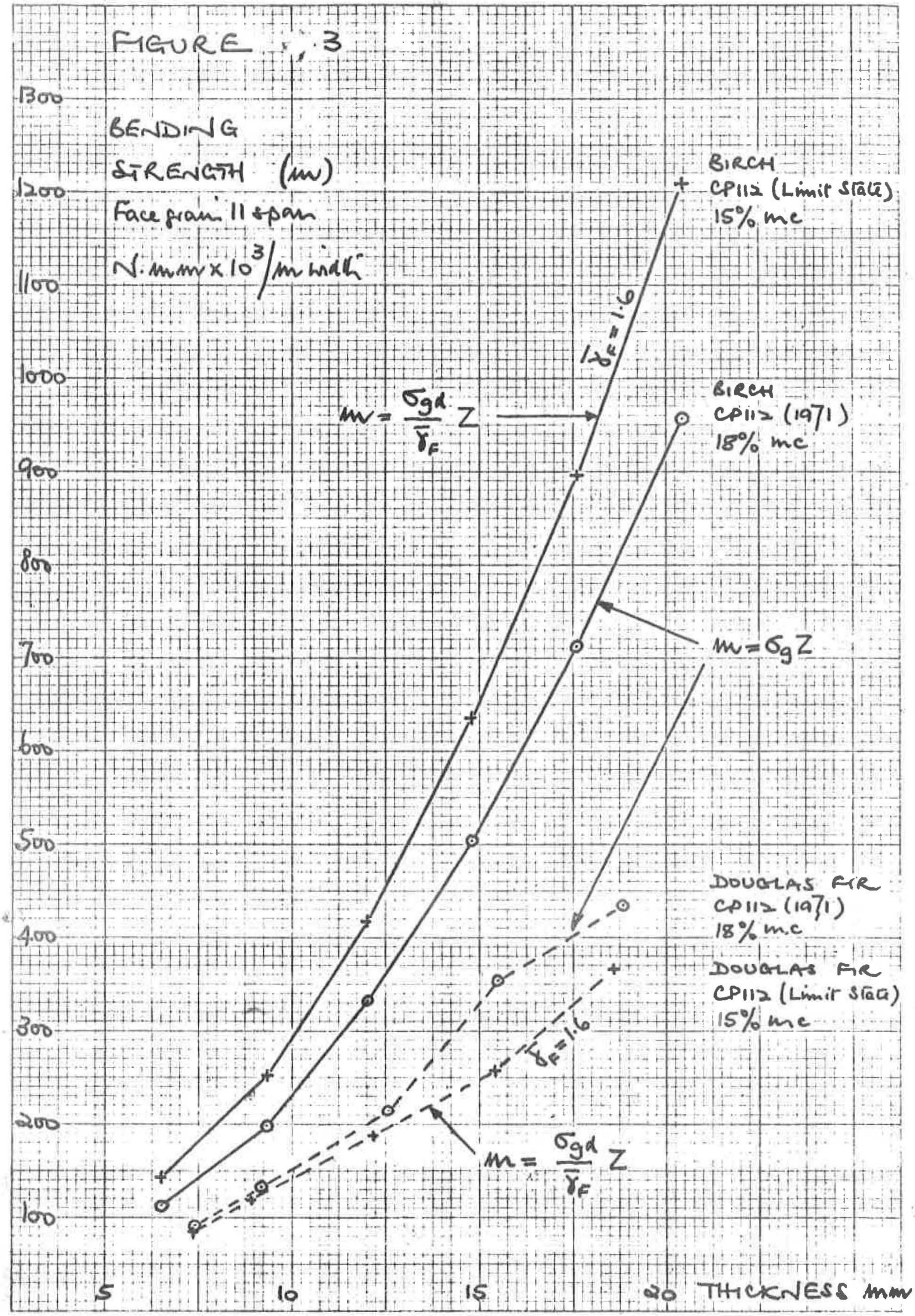


FIGURE .3
BENDING
STRENGTH (m)
Face grain ll span
N.mm x 10^3/m width
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
5
10
15
20
THICKNESS mm
BIRCH
CP112 (Limit State)
15% mc
1/γF = 1.6
m = σgd/γF Z
BIRCH
CP112 (1971)
18% mc
m = σg Z
DOUGLAS FIR
CP112 (1971)
18% m.c
DOUGLAS FIR
CP112 (Limit State)
15% mc
γF = 1.6
m = σgd/γF Z

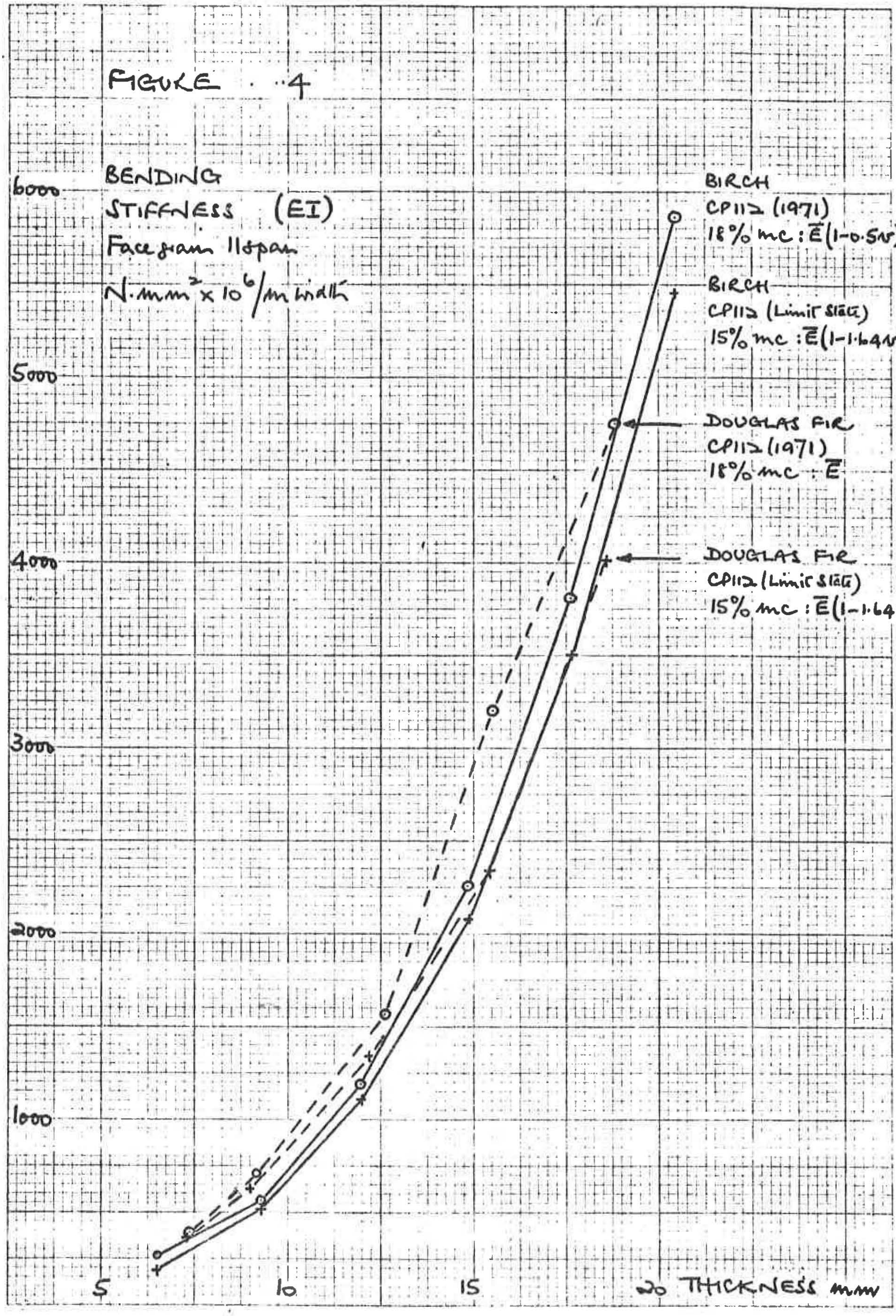


FIGURE 4
BENDING
STIFFNESS (EI)
Face grain ll span
N·mm² x 10⁶/m width
6000
5000
4000
3000
2000
1000
5
10
15
20 THICKNESS mm
BIRCH
CP112 (1971)
18% mc : Ē(1-0.5v)
BIRCH
CP112 (Limit state)
15% mc : Ē(1-1.64v)
DOUGLAS FIR
CP112 (1971)
18% mc : Ē
DOUGLAS FIR
CP112 (Limit state)
15% mc : Ē(1-1.64v)

EXTRACT FROM DRAFT CP 112

4 DESIGN OBJECTIVES

4.1 Limit State Design:

The object of design is the achievement of an acceptable probability that the structure being designed will not become unfit for the use for which it is required during its intended service life, ie that it will not reach a limit state. The stresses and loads used in design should therefore take account of variations in the properties of the materials and in the loads to be supported. Where the necessary data are available the characteristic stresses and loads are based on statistical evidence, and where they are not, on an appraisal of experience. In addition two partial safety factors are used, one for material strength γ_m, and one for load and load effects γ_f. These partial safety factors should have the values indicated in this Code.

4.2 Limit State Requirements:

All relevant limit states should be considered to ensure adequate safety and serviceability. The usual approach will be to design on the basis of the most likely critical limit state and to check that other limit states will not be reached. In most cases it will be sufficient to design for the ultimate strength and deflection limit states.

The methods of analysis used in assessing compliance with the requirements of the various limit states should be based on as accurate a representation of the behaviour of the structure as is practicable, but the methods and assumptions given in this Code will generally be adequate. When elastic analysis is used to determine the force distribution and/or displacements within a structure the stiffness of the members should be based throughout on the modified design stress values for modulus of elasticity.

4.2.1 Ultimate Strength:

The strength of a member or structural unit should be such that under the action of the design loads, the stresses induced in the materials, and the forces in the joints, do not exceed the modified design values, due account being taken of the effects of fabrication and erection.

In design calculations the modified design stresses, the modified design strength for fasteners and the design loads should be those specified in the appropriate sections of this Code or derived in accordance with the recommendations of this Code

The most unfavourable combination of the presence or absence of loads likely to occur should be considered, and any special hazards due to the nature of the occupancy or use of a structure or building should be taken into account. An assessment should also be made, where appropriate, to ensure that ultimate strength limit state is reached as a result of instability, and that progressive collapse will not occur as a result of accident or mis-use to an extent disproportionate to the original cause.

4.2.2 Deflection:

The deflection of a member or structural unit under the forces and loads that will be encountered in service, should not adversely affect serviceability, due regard being paid to the possibility of damage to surfacing materials, ceilings, partitions, the functioning of doors and windows, and to aesthetic and psychological effects.

In all cases the engineer should satisfy himself that deflections will not be excessive having regard to the loading conditions and requirements of the structure. When determining deflections account should be taken of joint slip and rotation and of any tolerances in fit permitted at the joints. (See Section).

For the purposes of calculating the deflections of principal members the modified design values for modulus of elasticity and modulus of rigidity, and the design loads for this serviceability limit state, should be as specified in this Code or derived in accordance with the recommendations of this Code.

As a guide and in the absence of criteria indicating a higher or lower value, the following may be regarded as reasonable limits for deflection:-

a) The deflection of a flexural member under the design load should not exceed 0.003 of its effective span.

b) Subject to the possible effects of the greater total deflection, members may be precambered to off-set the calculated deflection under the dead load and in this case the deflection under imposed load should not exceed 0.003 of the effective span.

c) The deflection of a vertical member under the action of wind forces should not exceed 0.003 of its height.

d) The deflection, normal to the length of rafters in roofs, under design load should not exceed 0.004 of the effective span.

e) The deflection of purlins in roofs, under design load, should not exceed 0.003 of the effective span.

f) The deflection of beams over windows or other openings, under design load, should not exceed 0.002 of the effective span.

g) The deflection of domestic flooring, under design load, should not exceed 0.003 of the effective span, or 2.0 mm, whichever is the smaller.

Under continuous loading, timber and board materials are subject to increasing deflection with time, the amount depending on the species or material, the magnitude of the induced stress, the moisture content at the time of loading and any subsequent changes in moisture content that take place while under load. The slip and rotation of mechanical joints also exhibit the same effect. Account may have to be taken of this in design and in the absence of specific information the following general recommendations should apply:

a) For the wet exposure condition, and irrespective of the initial moisture content, the deflection of solid timber or plywood members should be calculated using the modified design values of modulus of elasticity and/or modulus of rigidity for the wet exposure condition. The deflection under long-term load should be taken as twice the calculated value.

b) For the dry exposure condition, the deflection of solid timber members, of more than 100 mm least dimension, should be calculated using the modified design values of modulus of elasticity and/or modulus of rigidity for the wet exposure condition. The deflection under long term load should be taken as 1.5 times the calculated value.

c) For the dry exposure condition, the deflection of solid timber members of not more than 100 mm least dimension, and of laminated timber and plywood members should be calculated using the modified design values of modulus of elasticity and/or modulus of rigidity for the dry exposure condition. However where such a solid timber or plywood member is installed at a high moisture content (in excess of 20 per cent) and dries out under continuous loading, the deflection under this load should be taken as 1.5 times the calculated value.

d) For the wet exposure condition, the deflection of glued laminated timber structural members should be calculated using the modified design values of modulus of elasticity and/or modulus of rigidity for the wet exposure condition, see Section

4.3 DESIGN LOADS

The characteristic load on a structure should ideally be determined from a consideration of the actual values, and the variability of the loads which occur in practice. Adequate data are not yet available to enable this approach to be generally adopted and in the absence of such data the following characteristic lo 's should be used in design:

1) Characteristic dead load: The characteristic dead load G_k is the mass of the structure complete with finishes, fixtures and partitions and should be taken as equal to the dead load as defined in and calculated in accordance with CP3: Chap V:Part 1.

2) Characteristic imposed load: The characteristic imposed load Q_k should be taken as the imposed load as defined in, and calculated in accordance with CP3:Chap V:Part 1.

3) Characteristic wind load: The characteristic wind load W_k should be taken as the wind load as defined in, and calculated in accordance with CP3:Chap V:Part 2.

The loading conditions during erection and construction should be considered in design and SHOULD BE such that the subsequent compliance of the structure with the limit state requirements is not impaired.

The design load for a given type of load and limit state is obtained by multiplying the characteristic load (F_k) by the appropriate partial safety factor for loads (γ_f) ie

$$\text{Design load} = \gamma_f F_k$$

γ_f is introduced to take account of:

i) Possible unusual increases in load beyond those considered in deriving the characteristic value.

ii) Inaccuracies in assessment of the effects of loading, and unforeseen stress redistribution within the structure.

iii) Variations in dimensional accuracy achieved in construction.

The value of γ_f depends upon the importance of the limit state being considered and on the number of characteristic loads that act simultaneously on the structure or member.

4.3.1 Ultimate Strength:

For the ultimate strength limit state the duration of each design load, whether of long, medium, short or very-short term should be identified so that the appropriate modification factor, for the duration of load effect on material strength, may be included in the determination of the modified design stress (See Section).

The characteristic dead (G_k), imposed (Q_k) and wind (W_k) loads should be classified according to their estimated duration as:

Long term loads (G_{k1}, Q_{k1}) which may be either dead or imposed loads and including all loads which act, or may be considered to act, permanently on a structure or member, as for example dead loads, uniformly distributed imposed loads for floors, and loads in roof spaces due to storage.

Medium term loads (G_{k2}, Q_{k2}) which may be either dead or imposed loads and including al loads which act, or may be considered to act, for prolonged periods on a structure, or member, as for example uniformly distributed imposed loads for roofs.

Short term loads (Q_{k3}, W_{k3}) which may be either imposed or wind loads and including all loads which act, or may be considered to act from time to time for short periods on a structure or member, as for example wind loads of Class C (CP 3: Chapt V:Part 2, 15 sec averaging time) and concentrated imposed loads for roofs and ceilings.

Very short term loads (Q_{k4}, W_{k4}) which may be either imposed impact loads or wind loads and including all loads which act, or may be considered to act, from time to time for very short periods on a structure or member, as for example wind loads of Class A or Class B (CP 3: Chapt V:Part 2, 3 and 5 sec averaging time).

The design loads for the ultimate limit state should be taken as:

Long term design load

$$1.4\ G_{k1} + 1.6\ Q_{k1}$$

Medium term design load

$$1.4\ (G_{k1} + G_{k2}) + 1.6\ (Q_{k1} + Q_{k2})$$

Short term design load

$$1.2\ (G_{k1} + Q_{k1} + Q_{k2} + Q_{k3} + W_{k3})$$

Very short term load

$$1.2\ (G_{k1} + Q_{k1} + Q_{k2} + Q_{k3} + Q_{k4} + W_{k4})$$

It should be noted that while each of the design loads is a summation of all imposed loads (Q_k) of that duration category, and all longer duration categories, it is unlikely that all of the imposed loads will occur simultaneously.

When considering the design of part of a structural unit or member under a combination of loads, if a more unfavourable condition results from the presence or absence of a load, or by taking γ_f equal to 1.0 or 1.4 for dead load (G_{k1} G_{k2}), in any other part of the structural unit or member, then this condition or these factors should be used.

When considering overturning or stability the γ_f factor for dead load (G_{k1} G_{k2}) should be taken as 0.9 or 1.4, whichever produces the worst condition.

4.3.2 Deflection:

For the deflection limit state it is not necessary to distinguish between the different duration of load categories for the imposed and wind loads. The design loads should be taken as:

1 The sum of the characteristic loads when one or two types of characteristic load act simultaneously, ie

$1.0\ G_k$ $1.0\ Q_k$

$1.0\ (G_k + Q_k)$

$1.0\ (G_k + W_k)$

2 The sum of the characteristic loads, multiplied by $\gamma_f = 0.8$, when three or more types of characteristic load act simultaneously, ie

$0.8\ (G_k + Q_k + W_k)$

The most unfavourable combination of characteristic loads should be considered in design, and if a more unfavourable condition is created by selecting only parts of a structure to be loaded with the imposed loads then the arrangement of these loads should be such as to cause the greatest deflection.

4.4 STRENGTH OF MATERIALS

For timber and board materials, and for joints, the strength properties are defined for the dry exposure condition, for long term loading and, for timber in the case of bending strength, for a beam depth of 200 mm.

Three stages are involved in the determination of modified design stresses, (or modified design strength values for joints) from which the strength of a section or joint or the deflection of a member or structure, should be assessed.

1 Characteristic stresses and fastener strengths for the different properties at the dry exposure condition are determined from the results of standard laboratory tests on representative samples and are the values below which not more than 5 per cent of the results fall. The characteristic values are assumed to apply to the particular species or grade of timber and board material, and to the particular type of fastener, so that special care must be paid to the selection of samples for testing.

2 The characteristic stresses and fastener strengths are reduced by dividing by the partial safety factors for strength (γ_m) and adjusting to the standard condition of long term loading, and in the case of bending strength for timber to a section depth of 200 mm. Depending on the grade or quality of the material tested the resulting stresses are the basic design stresses, the grade design stresses or fastener design strengths.

3 Finally these design stresses or strengths are multiplied by modification factors given in this Code for loading and service conditons, and for section size, when these differ from the standard conditions. The resulting stresses, the modified design stresses (or strengths for fasteners), are the values to be used in all design calculations.

CIB-W18/5-4-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

VENEER PLYWOOD FOR CONSTRUCTION
QUALITY SPECIFICATIONS

(1st DRAFT 1.6.75)

by

ISO/TC 139 - Plywood, Working Group 6

KARLSRUHE - October 1975

June 9th 1975

TO THE MEMBERS OF WORKING GROUP 6 ISO TC 139 - PLYWOOD

Dear Sirs,

Re First Draft Proposal
dated June 1st 1975
"Veneer Plywood
for Construction - Quality Specification"

It has taken some time to develop the enclosed draft. The draft is based on the guidelines sent to you for review on November 9th 1973. There were no negative comments on the guidelines.

Clause 1 - Scope - states the objectives of the draft. Since ISO Standards must be useful on a world-wide basis an attempt has been made to cover all types of plywood that are suitable for the purpose covered by the Standard.

Cross references are made to establish standards for some test procedures - Clauses 7.1; 7.2; 11.2; 12.2; 13.3. It is assumed that Committee members will have access to these Standards. If they are not available, I will, wherever possible be, pleased to supply xeroxed copies of the pertinent Clauses on request.

The sections of the draft dealing with classification are tentative. Sampling and statistical techniques for the classification levels are being reviewed by scientists at the Western Forest Products Laboratory of the Canadian Government. The results of this review will be used to develop the final draft.

Members of Working Group 6 are asked to comment promptly on the draft proposal. These comments will be used to prepare a second draft for consideration at a meeting of the Working Group to be held in the week prior to the next meeting of TC 139 (not yet scheduled).

Yours very truly,

Fred Walsh

FNW:kk

F.N. Walsh

Chairman
Working Group 6

ISO/TC 139 Working Group 6

I S O

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION

TECHNICAL COMMITTEE ISO/TC 139 - PLYWOOD

P L Y W O O D

VENEER PLYWOOD FOR CONSTRUCTION

QUALITY SPECIFICATIONS

FIRST DRAFT PROPOSAL - June 1, 1975

For Working Group 6 Consideration

FIRST DRAFT June 1, 1975

ISO CONSTRUCTION PLYWOOD

CONTENTS

* To be added later.

1. SCOPE

This International Standard establishes methods for measuring, testing and reporting characteristics of construction plywood. It is recognised that some characteristics may vary between different types of construction plywood and still permit the plywood to meet the requirements of the end use. Limits are set for those characteristics which, if unlimited, would result in failure of the plywood to meet the requirements of the end use.

The standard is not concerned with the appearance of construction plywood and only the structural requirements most important to the end use are considered.

Test procedures for construction plywood panels are limited to bond, stiffness and bending strength. Data on bond type, stiffness and bending strength may be derived by procedures recognised in the country of manufacture. However, if verification is required, the standard against which construction plywood performance is to be measured is set forth in this Standard.

2. FIELD OF APPLICATION

This Standard applies to flat rectangular plywood panels for use in construction (primarily as roof, wall or floor sheathing). Sapwood or heartwood of any species of wood, for which a botanical description exists, may be used. Panels shall be long grained or cross grained veneer plywood. Plies may be of sliced, sawn or rotary cut veneer.

Clauses 3 References; 4 Definitions; 5 Terminology are to be added later.

6. MANUFACTURER'S SPECIFICATION

6.1 Format

The Manufacturer's specification shall be in the format shown on page 4.

6.2 Data

Where no limitation of size is imposed it shall be so stated (no limit). Where any characteristic or defect is not permitted it shall be so stated (none). The Clause relevant to each item is given in parentheses. The following data shall be provided:

6.2.1 Identification

Name and address of manufacturer, manufacturer's reference number and date.

6.2.2 Panel Dimensions

(a) Length (Clause 7.1)

(b) Width (Clause 7.1)

(c) Thickness (Clause 7.2)

6.2.3 Panel Manufacture

(a) Number of Plies

(b) Orientation of each ply given as "parallel" (||) or "perpendicular" (⊥) to the long edge of the panel. The grain direction of the face ply shall be taken as the long direction in the case of square panels.

(c) Species of wood in each ply identified by its common name and botanical name or by species group. (Where identification is by species group, the common and botanical name of each species in the group shall be appended to the Specification in the form shown on page 5.)

(d) Maximum size of knots (sound and tight), loose knots or knot holes, splits and the maximum area of decay (inactive). (Clause 8.1)

(e) Maximum size of patches. (Clause 8.3)

(f) Surface: in "face" or "back" columns or both columns as appropriate, write "yes" if the panel is sanded or if open defects are filled. (Clause 8.7)

6.2.4 Bond Type

Bond type Exterior - 'A' or
Interior - 'B' (Clause 11&12)

6.2.5 Stiffness and Bending Strength

Appropriate numerical levels for stiffness and bending strength both parallel and perpendicular to face grain from Table 1. (Clause 14)

6.2.6 Classification

The classification consists of a series of numbers and letters representing bond type, stiffness and bending strength. (Clause 14)

6.2.7 Panel Marking

All the marks appearing on the panel must be listed indicating whether the marks appear on the face, back or edge of the panel. (Clause 15)

MANUFACTURER'S SPECIFICATION FOR ISO CONSTRUCTION PLYWOOD

Name of Manufacturer :	Reference Number :
Address of Manufacturer :	Date :

PANEL DIMENSIONS (mm)	Length :	Width :	Thickness :

PANEL MANUFACTURE		Number of Plies :
Ply Number	Orientation	Species (Common Name, Botanical Name).
1. (face)		

	face	inner	back		face	inner	back
Maximum Patch Size (mm)				Max. Knot Size (mm)			
				Max. Loose Knot or Knot Hole Size (mm)			
Surface : sanded		–		Max. Split Size (mm)			
: open defects filled		–		Max. Decay Area (%)			

PANEL BOND TYPE :

PANEL STIFFNESS (Nm^2/m)	Parallel :	Perpendicular :
PANEL BENDING STRENGTH (Nm / m)	Parallel :	Perpendicular :

PANEL CLASSIFICATION :

PANEL MARKING (including ISO logo indicating construction plywood)	face	back	edge
Classification :			
Manufacturer :			
Country :			
Other Marks :			

SPECIES GROUP DESIGNATION:	
Species in Group	
Common Name	Botanical Name

Manufacturer:

Attachment to Specification for ISO Construction Plywood,

Plywood Classification............ sheet.....of..... sheets.

7. PANEL DIMENSIONS

7.1 Size

Panel length and width shall be measured in accordance with ISO Recommendation R 1097.. The maximum permissible tolerance shall be +0 -3 mm.

7.2 Thickness

Panel thickness shall be measured in accordance with ISO Recommendation R 1097. The minimum and maximum panel thicknesses shall be 7 mm and 20 mm respectively. The maximum permissible tolerance shall be + ___ - ___ for unsanded panels and + ___ - ___ for sanded panels.

7.3 Edge Straightness

Panel edges shall not deviate from straight more than 1.5 mm.

7.4 Squareness

The deviation from squareness of two adjacent edges at the four angles of a panel shall not exceed 1 mm per metre of panel length.

8. PANEL MANUFACTURE

8.1 Veneer Characteristics

The size of sound tight knots, loose knots or knot holes and splits shall be measured across the grain. Roughness, grain imperfections, insect damage, streaks and discolouration, bark pockets or other defects that do not impair the bond and serviceability of the panels, shall be permitted. Inactive decay shall be permitted and shall be measured and expressed as a percentage of the panel area. Active forms of decay shall not be permitted.

8.2 End Joints in Veneer

End butt joints shall not be permitted. Veneer that has been scarfed or otherwise end jointed shall be acceptable provided the bond is as strong and durable as the bond between plies.

8.3 Patches

The size of patches made of wood or synthetic materials shall be the width measured across the grain of the ply.

8.4 Gaps and Laps

Face, back and inner plies may be made of more than one piece of veneer. Every effort shall be made to closely butt adjoining pieces of veneer to minimise the width of gaps in face, back and inner plies, and to avoid laps in inner plies. No laps shall be permitted in face or back plies.

8.5 Open Defects - Any ply

In panels with a type "A" bond (see Clause 11.1), the maximum width of open defects shall be 40 mm and the maximum length 160 mm. In panels with a type "B" bond (see Clause 11.2), the maximum width of open defect shall be 90 mm and the maximum length 160 mm. There is no restriction on the length of open defects which do not exceed 25 mm in width for panels with type "A" or type "B" bond. The width of open defect shall be measured perpendicular to grain direction.

8.6 Short and Narrow Plies

Face and back plies may be narrow on one edge or short on one end only by no more than 6 mm for half the panel length or width respectively. Inner plies may be short or narrow by not more than 6 mm in depth and 200 mm in length.

8.7 Surface

The face and/or back of panels may be sanded or unsanded. Open defects may be filled or unfilled.

9. SAMPLING FOR BOND, STIFFNESS AND BENDING STRENGTH TESTS

9.1 Method

Sample panels shall be taken at random from manufacturing plant, inventory, storage or job site, as appropriate to the circumstances.

9.2 Number of Panels

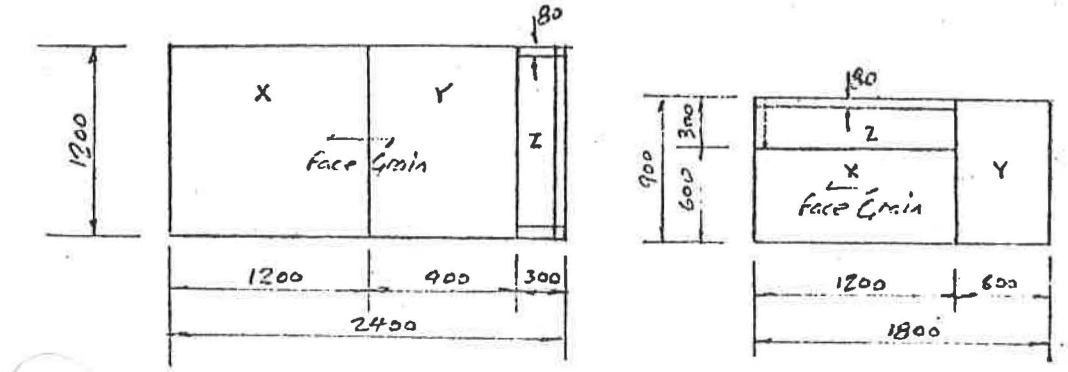
The number of panels in the sample shall be a minimum of 10 up to a maximum of 50 in increments of 10 panels. Property values will vary depending upon the sample size and this factor should be assessed when selecting the number of panels in the sample (see Clause 13.4).

10. CUTTING SPECIMENS

Panels shall be cut to give all the test pieces listed in Clauses 11, 12 and 13. The length and width of all specimens for a given test shall be identical. Where the required test specimens cannot be obtained from a single panel, the specimen for bending parallel to the face ply shall be cut from one panel and the specimen for bending perpendicular to face ply and the test piece for glue bond shall be cut from a second panel. Test pieces for glue bond tests shall be cut to exclude plywood within 80 mm of the edge of the panel. Typical cutting plans are shown in Figure 1.

Figure 1

X - Specimen for parallel to face grain test.
Y - Specimen for perpendicular to face grain test.
Z - Test piece for bond test.



1200 x 2400 panel

900 x 1800 panel

11. BOND TYPE "A"

11.1 General

Bond type "A" denotes an exterior bond type.

11.2 Test

The cutting of test specimens, conditioning, testing and evaluation of test results shall be based upon Clause 5.1 of CSA O121.

12. BOND TYPE "B"

12.1 General

Bond type "B" denotes an interior bond type.

12.2 Test

The cutting of test specimens, conditioning, testing and evaluation of test results shall be based upon Clause 5.1 of CSA O153.

13. STIFFNESS AND BENDING STRENGTH

13.1 General

The stiffness is the product of the moment of inertia and the modulus of elasticity in bending. The bending strength is the product of the modulus of rupture and the section modulus.

13.2 Specimen Size

For parallel to face grain testing the size shall be:

length parallel to face grain - 1200 mm,

width - 600 mm minimum, 1200 mm maximum.

For perpendicular to face grain testing the size shall be:

length perpendicular to face grain - 900 mm minimum,

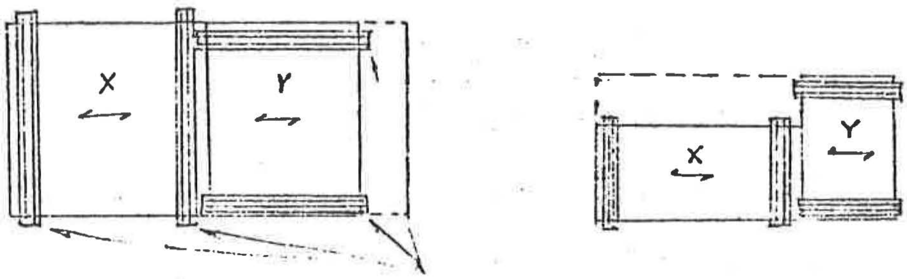
width - 600 mm minimum, 900 maximum.

13.3 Test

Conditioning of specimens and bending and stiffness tests shall be based upon ASTM D3043-73 "Testing Plywood in Flexure - Method C." Where the face and back plies of panels differ significantly, the panels shall be tested in their less stiff direction. Equal numbers of parallel and perpendicular to face grain specimens shall be tested. The orientation of specimens in the test machine shall be as shown in Figure 2.

Figure 2

X - Specimen for parallel to face grain test.
Y - Specimen for perpendicular to face grain test.
↔ - Face grain direction.



loading frames of test machine

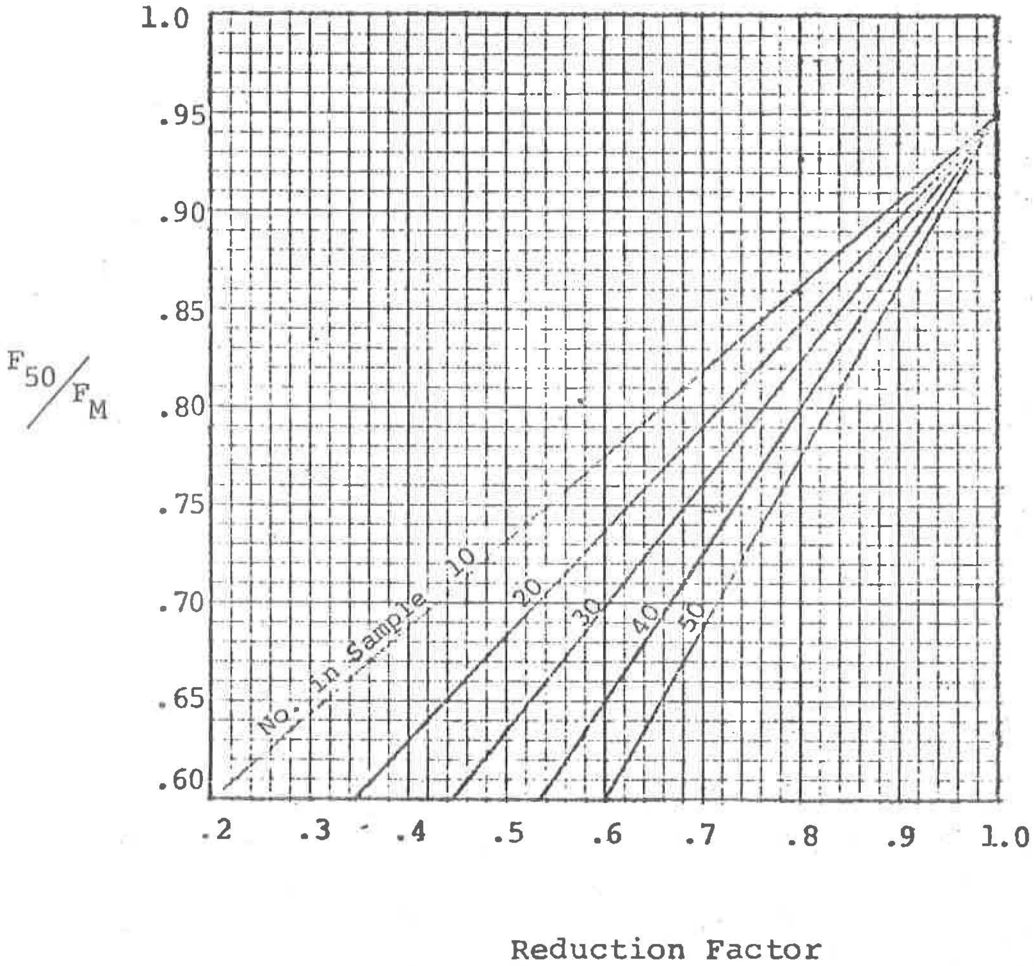
Diagrams show orientation of loading frames in relation to typical specimens cut as shown in Figure 1.

13.4 Evaluation of Test Results

The test results for each property shall be ranked and the property value shall be taken as the mean value of the test results multiplied by the reduction factor obtained from Figure 3. To use Figure 3 calculate the mean value of the lower half of the ranked test results (F_{50}) and calculate the ratio of F_{50} to the mean value F_M.

Enter Figure 3 from the vertical axis with the ratio of $^{F}50/^{F}M$ and proceed horizontally until the line representing the number of test specimens is reached. From this point proceed down vertically to the horizontal axis which gives the appropriate reduction factor.

Figure 3



F_{50} is the mean value of the lower 50% of test results.

F_M is the mean value of all the test results.

14. PANEL CLASSIFICATION

Panels shall be classified according to their bond type, stiffness parallel to face grain, bending strength parallel to face grain, stiffness perpendicular to face grain and bending strength perpendicular to face grain, in that order.

Bond type shall be denoted by "A" or "B" as appropriate (see Clause 11 and 12).

Property values for stiffness and bending strength evaluated from test results shall be rounded up or down to the nearest numerical level given in Table 1. Should the property value fall exactly midway between two classification levels, the property value shall be rounded up. Stiffness shall be given in the Classification by the numerical level and bending strength by the equivalent letter given in Table 1. For example, panels with an exterior type glue bond and having property values of 1618 Nm^2/m for stiffness parallel, 915 Nm/m for bending strength parallel, 266 Nm^2/m for stiffness perpendicular and 350 Nm/m for bending strength perpendicular, would be classified as A 1500 U 300 R.

CLASSIFICATION LEVELS									
UNITS		Stiffness: Nm^2/metre width Bending Strength: Nm/metre width							
Number	Letter	Number	Letter	Number	Letter	Number	Letter	Number	Letter
4	D	40	J	200	P	900	U	3000	-
7	E	60	K	300	Q	1200	V	3500	-
10	F	80	L	400	R	1500	W	4000	-
20	G	100	M	500	S	2000	-	4500	-
30	H	150	N	700	T	2500	-	5000	-

Table 1

15. PANEL MARKING

15.1 ISO Mark

Panel shall be marked on the face or back with the ISO logo or mark indicating "construction plywood" and the appropriate classification.

15.2 Identification Marks

Panels shall be marked to identify the manufacturer and country. The name of an agent or association representing the manufacturer may be used in place of the name of the manufacturer.

15.3 Optional Marking

Panel grade, panel thickness, species and other descriptive information may be marked on the panel. Such marks shall be at least 100 mm from the ISO mark.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STANDARD METHODS OF TEST FOR DETERMINING SOME PHYSICAL AND MECHANICAL PROPERTIES OF TIMBER IN STRUCTURAL SIZES

by

W T CURRY

BUILDING RESEARCH ESTABLISHMENT
PRINCES RISBOROUGH LABORATORY
UNITED KINGDOM

KARLSRUHE - OCTOBER 1975

STANDARD METHODS OF TEST FOR DETERMINING SOME PHYSICAL AND MECHANICAL PROPERTIES OF TIMBER IN STRUCTURAL SIZES

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INTRODUCTION

At the request of the W18 Commission of CIB a first draft standard was prepared and circulated to Messrs Kuipers and Saarelainen for comment. It was not however found possible, by correspondence and in the time available, to arrive at a final agreed document. This paper can therefore only provide a basis for discussion so that decisions may be made as to the scope and contents of the standard. Various comments have been included which are not proposed as part of the standard but are intended rather to explain why certain recommendations have been made. These are identified by the type-set. Consideration should also be given as to how far the standard should be self-contained or to what extent it should refer to other ISO and ECE documents.

DRAFT STANDARD

1 Scope:

This standard gives preferred methods of test for the determination of the following physical and mechanical properties of solid rectangular sections of timber in structural sizes.

The physical properties are:
- dimensions
- moisture content
- density
- nominal density

The mechanical properties are:
- modulus of elasticity in bending
- ultimate bending strength
- shear modulus
- modulus of elasticity in tension
- ultimate tension strength
- modulus of elasticity in compression
- ultimate compression strength

In addition recommendations are made for recording the grade determining properties for visual and machine grading. Reference is also made to sampling and specimen selection.

Because of the cost of testing timber in structural sizes consideration should be given, not only to the immediate objectives of a particular project, but also to establishing a data bank on the strength properties and growth characteristics of structural timber. This could involve recording additional information but data storage and access are no longer problems with computers. What additional information should be recorded might be identified within the following broad areas of interest, where more experimental data are needed:

a visual stress grading
b machine stress grading
c harmonisation of stress grading
d derivation of characteristic strength values
e effect of such factors as moisture content, load duration and size, on strength.
f monitoring the 'quality' of structural timber.

2 Sampling and Specimen Selection:

2.1 Sampling

No specific recommendations can be given as to what constitutes an acceptable sample since this will depend on the objectives of the tests and the application of the results. A sample should contain a sufficient number of specimens, selected at random or according to a defined method, to permit the use of mathematical statistics and the achievement of an acceptable confidence in the results.

2.2 Specimen selection

No specific recommendations can be given as to how individual specimens should be selected since this will depend on the objectives of the particular investigation. Each specimen shall however be selected so that its critical zone, ie the weakest zone as judged by visual inspection or by machine selection, shall be at the centre of its length.

The location of the critical zone can influence both modulus of elasticity and shear modulus determinations in bending and should be controlled. There is a need to distinguish between end-use testing and testing for the determination of grade stresses generally. In end-use testing, where the specimens are supported and loaded to simulate service conditions, the defects can occur anywhere and consequently the strength values may relate to sections with better characteristics than those permitted for a grade. Although more in line with a probability approach to the design of the particular members, end-use testing provides data of limited application. If timber is to be graded without its end-use being known then it would be preferable to determine grade stresses on the strength of critical zones containing defects to the limits specified for a grade. Extending this one stage further, for the development of grading rules and stresses generally, a data bank on the strength and 'quality' characteristics of unit lengths of timber would be required.

3 Physical Properties:

3.1 Identification

Each specimen shall as necessary be identified for:

a species
b nominal size
c country of origin, region or mill
d grade or any relevant pre-selection
e project reference
f date

It is not of course essential in a particular project for all of these identifications to be carried out. However if maximum advantage is to be gained from the correlation and use of data from different sources a longer term objective could be the establishment of a standard form of data logging.

3.2 Dimensions

The width, thickness and length of each specimen shall be determined to three significant figures. In the case of width and thickness the dimensions shall be the average of three measurements taken throughout the length of the specimen. When strength tests are made the dimensions shall be determined at the moisture content condition for these tests.

3.3 Moisture content

Unless otherwise required each specimen shall be conditioned prior to test in air at the desired exposure conditions of temperature and relative humidity until equilibrium moisture content is attained. Kiln-drying may be used to accelerate the drying prior to a final stabilisation at the desired exposure conditions. When tests are required at the wet condition, ie at a moisture content higher than the fibre saturation point, this may be obtained by submersion in water for about three months.

The moisture content of each specimen shall be determined from a cross-section disc, free from knots, and taken close to the fracture in an ultimate strength test, or otherwise not nearer than 500 mm to an end of the specimen. The disc shall have the full cross-section dimensions of the specimen and shall have a length along the grain of 25 ± 5 mm. Moisture content shall be calculated as:

$$W = 100\ (m_1 - m_0)/m_0 \text{ per cent}$$

where m_1 is the mass (g) of the disc before drying
m_0 is the mass (g) of the disc after drying

The disc shall be dried to constant mass (m_0) at a temperature of 103 ± 2°C, constant mass being reached when the loss in mass between two successive weighings carried out at an interval of 6 hours is not greater than 0.5 per cent.

Moisture content shall be expressed to the nearest 1 per cent.

There is evidence that the effect of the initial drying of timber after conversion from the log, on the geometrical and strength properties is significantly different than after re-wetting and drying. It may be desirable therefore to require wet strength to be determined only after a drying/re-wetting cycle, since this will reflect practical conditions. Also, with kiln-drying the severity of the drying rate can affect the properties particularly with timber containing large defects, and may therefore have to be controlled to reflect practical methods of drying.

3.4 Density

The density of each specimen shall be calculated as:

$$\rho_w = m_w/v_w \text{ kg/m}^3$$

where m_w is the mass (kg) of the specimen
v_w is the volume (m^3) of the specimen at the moisture content w per cent

Density shall be expressed to three significant figures.

3.5 Nominal density

The nominal density of each specimen shall be determined from a cross-section disc free from knots and taken close to the fracture in an ultimate strength test, or otherwise not nearer than 500 mm to an end of the specimen. Nominal density shall be calculated as:

$$\rho_n = m_o/v_w \text{ g/cm}^3$$

where m_o is the mass (g) of the disc after drying in accordance with 3.3.
v_m is the volume (cm^3) of the disc at the moisture content w per cent.

Nominal density shall be expressed to three significant figures.

There is a need to distinguish between the density of a complete specimen containing knots, moisture etc and the density of the dry wood fibre material of the test specimen. The first defines self weight and the second is important in determining the influence of defects on strength and seasoning sample characteristics against those of other samples or species. Whether nominal density should be determined using oven-dry volume or the volume of the test moisture content is a matter of choice, the latter has been chosen since it is easier to measure.

4 Grade Determining Properties (Visual):

4.1 General properties

The slope of grain, rate of growth, fissures and wane shall be determined for each specimen in accordance with ECE Standard No - 'Stress Grading of Coniferous Sawn Timber'. The value of each at the critical zone shall be recorded. If the pith is present in a specimen this shall also be recorded.

4.2 Knots

Knots shall be determined and assessed by the knot area ratio (KAR) method in accordance with ECE Standard No - 'Stress Grading of Coniferous Sawn Timbers'. The KAR for both the margin and full cross section conditions at the critical zone shall be recorded to the nearest 0.01.

Consideration should be given to recording knots at the critical zone, or at the fracture section in an ultimate strength test if this is different, by numerical code so that account may be taken of their actual shape, size and location. One such method is illustrated in Fig 1.

The effects on yields and stress values of changes in the permissible limits for defects and the establishment of visual grades boundaries are likely to be of continuing interest and it is desirable that as much basic information as possible should be accumulated to enable these to be studied. Although there is presently considerable interest in 'in-grade' testing this would seem to be an inefficient exercise unless supplementary information is recorded to examine the effects of changes in the grading rules. If visual grading is to be improved them more knowledge of the relations between strength and defects is needed.

5 Grade Determining Properties (Mechanical):

5.1 General

The development of machine stress grading depends on the knowledge available on the relations between the ultimate strength properties of timber and indicating parameters such as deflection, wane velocity, vibration frequency, acoustic impedance, etc, which can be determined by non-destructive tests. These indicating parameters have the same significance in machine stress grading as do the visible growth characteristics in visual stress grading. Consideration should be given to including their measurement in individual projects which involve the determination of ultimate strength values.

5.2 Modulus of elasticity, E_T

An important indicating parameter is modulus of elasticity measured in pure bending, ie free from shear, over a relatively short span. The following standard procedure should be used for the determination of this property.

Each specimen, or the piece from which it will be cut, shall be loaded in bending as a joist as shown in Fig 2. The distance between the inner load points shall be 1.0 m with the critical zone of the piece located centrally, (see 2.2). The deflection at the centre shall be measured over a gauge length of 900 mm, with the deflectometer located at the centre of depth of the section. Load shall be applied at a continuous rate to induce a rate of straining in the extreme fibres of 0.001 per min. The rate of cross-head movement shall be:

$$R = (2a + 3000)\ Z a/3d$$

where a is the distance between the inner load point and the nearest support (mm). This distance shall be such as to keep the shear stresses to an acceptably low value and generally should not be less than 3d.
d is the nominal depth of the section (mm)
Z is the rate of straining, 0.001 per min.

From a record of the load/deflection characteristics below the proportional limit the modulus of elasticity, E_T, shall be calculated as:

$$E_T = 607.5\ P'a/bd^3\ \delta\ kn/mm^2$$

where δ is the deflection (mm) under a total load increment of P'(N)
b is the actual width of the section (mm)
d is the actual depth of the section (mm)
a is the distance between the inner load point and the nearest support (mm)

The value of modulus of elasticity, E_T shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy.

The indicating parameters for machine stress grading are all related to general, or local, values of modulus of elasticity and to density. The measurement of modulus of elasticity under standard conditions can consequently be taken as a basic parameter. This can be determined using standard laboratory equipment, thus enabling research centres to accumulate data which could ultimately be used to assist with the introduction and development of machine stress-grading. It should be noted that this approach depends for its effectiveness on establishing a calibration between the actual indicating parameter of a machine and E_T. This should however involve less testing than examining each indicating parameter separately.

It is not the intention of this test to provide a measure of modulus of elasticity for use in design but rather to provide data for study of the relations between strength and E_T and of how these are affected by species, size, moisture content and dimensional tolerances.

It is recognised that Section 5 may not be suitable for a standard but is included here in line with the general approach which has been to broaden the scope of standard testing to the creation of data banks which could be of considerable value in the future.

6 Strength Tests:

6.1 Bending

These tests provide for the measurement of modulus of elasticity and ultimate strength in bending, and for the measurement of the shear modulus of structural timber.

6.1.1 Modulus of elasticity: Each test specimen shall have a length equal to 18 times the nominal depth of the section, plus 150 mm, with the critical zone at its centre. It shall be loaded in third point bending over a span of 18 times the nominal depth, as illustrated in Fig 3. It shall be supported on rollers and a fixed knife edge reaction, or by devices which achieve an acceptable free support condition. Small metal plates of a width not greater than 50 mm shall be inserted between the specimen and the loading heads and supports to minimise indentation. If the depth to width ratio exceedes 4, lateral restrain shall be provided both outside and inside the loading heads, as necessar to prevent buckling. The restraints shall permit vertical movement without significant frictional resistance.;

The deflection at the centre shall be measured with the deflectometer attached at the centre of the depth of the section.

a relative to the supports (δ_1) and/or
b relative to a gauge length of 5 times the nominal depth located centrally in the middle third of the span (δ_2)

Load shall be applied at a continuous rate to induce a rate of straining in the extreme fibres of 0.001 per min. The rate of cross-head movement shall be:

$$R = 0.06\ d\ \text{mm/min} \pm 25\%$$

where d is the nominal depth of the section (mm)

From a record of the load/deflection characteristics below the proportional limit the modulus of elasticity shall be calculated as:

a $$E_{(a)} = P'\ell^3/4.7\ bd^3\delta_1 \times 10^3\ \text{kn/mm}^2$$

where δ_1 is the deflection (mm) under a total load increment of P'(N)
b is the actual width of the section (mm)
d is the actual depth of the section (mm)
ℓ is the total span (mm)
$E_{(a)}$ is the apparent modulus of elasticity, ie unadjusted for shear

b $$E = 1.6075\ P'\ell^3/bd^3\delta_2 \times 10^6\ \text{kn/mm}^2$$

where δ_2 is the deflection (mm) under a total load increment of P'(N)
b is the actual width of the section (mm)
d is the actual depth of the section (mm)
ℓ is the total span (mm)

The value of modulus of elasticity shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy.

It is known that in pieces of structural timber the moduli of elasticity (and shear modulus) of critical zones containing knots or inclined grain, are lower than for the remainder of the clear material. There will consequently be differences in the gross values of modulus of elasticity for the same gauge length under bending and compression and tension tests. There will also be differences in the values obtained from bending tests, depending on the gauge length and on the location of the critical zones within this length. There are obviously arguments for and against adopting a fixed gauge length or one that changes with section depth, and while a gauge length of 5 times the depth may be satisfactory for deep sections, it will hardly give a value of modulus of elasticity, which is realistic for design purposes, for sections as shallow as 100 mm.

Provision is made for the measurement of two values of modulus of elasticity, one containing shear deflection and the other shear free. In some test procedures an arbitrary ratio is assumed between E and G so that the centre full-span deflection may be reduced to provide some allowance for shear. If G is to be determined by a bending test then the shear free value of modulus of elasticity must be determined.

The decision as to which modulus of elasticity should be determined, and what test procedure should be used, must be very largely an arbitrary one. Attention should be paid to the magnitude of the deformations that have to be measured, and although it is felt that a fixed gauge length is better, practical considerations may favour one which is a multiple of section depth. However, if this is adopted it should apply not only to bending but to direct stress determinations of modulus of elasticity.

6.1.2 Shear modulus: For the determination of shear modulus each specimen, after being tested in accordance with 6.1.1 shall have the gauge length retested in the same direction under a centre load bending test, ie with a span of 5 times the nominal depth of the section and with the critical zone at its centre. The deflection at the centre shall be measured relative to the supports, with the deflectometer attached at the centre of depth of the section, as shown in Fig 3. Load shall be applied at a continuous rate to induce a rate of straining in the extreme fibres of 0.001 per min. The rate of cross-head movement shall be:

$$R = 0.004\ d\ \text{mm/min} \pm 25\%$$

From a record of the load/defeection characteristics below the proportional limit the shear modulus shall be calculated as:

$$G = 0.3/\{(bd\delta_3/P'\ell) - (0.25\ \ell^2/d^2\ E \times 10^3)\}\ \text{N/mm}^2$$

Where δ_3 is the deflection (mm) under a load increment of P'(N)

ℓ is the span (mm)
b is the actual width of the section (mm)
d is the actual depth of the section (mm)
E is the modulus of elasticity (kn/mm^2) determined as in 6.1.1

The value of shear modulus shall be expressed to two significant figures and the equipment used shall be capable of achieving at least this accuracy.

6.1.3 Ultimate strength: The test arrangement and rate of straining shall be the same as in 6.1.1. Each specimen shall be loaded continuously to fracture and the ultimate bending stress calculated as:

$$\sigma_b = P\ell/bd^2\ \text{N/mm}^2$$

Where P is the maximum total load (N)
ℓ is the total span (mm)
b is the actual width of the section (mm)
d is the actual depth of the section (mm)

The value of ultimate stress shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy. The mode of fracture shall be recorded together with the grade determining properties at the fracture zone if this is different from the previously identified and recorded critical zone.

6.2 Tension:
These tests provide for the measurement of modulus of elasticity and ultimate strength of structural timber parallel to the grain.

6.2.1 Modulus of Elasticity: Each specimen shall be of the full cross-section and shall be loaded continuously in tension using gripping devices which ideally permit the application of uniform tension without introducing bending moments. It is recognised that in practice it may not be possible to achieve the necessary alignment and rotational freedom to satisfy the ideal test condition. The actual gripping devices and loading employed should therefore be recorded. The length of each specimen outside the grips shall be at least 9 times its nominal width, ie the greatest dimension of the section, and shall have the critical zone within 1½ times the nominal width from its centre.

Deformation shall be measured over a gauge length of 5 times the nominal width of the section located not closer than twice this width to the ends of the grips. The gauge length shall include the critical zone. Two extensometers shall be attached at diagonally opposite points on the faces to minimise the effects of distortion and permit the determination of the average deformation of the full gauge length. The rate of cross-head separation shall be:

$$R = 0.001\,\ell \text{ mm/min} \pm 25\%$$

where ℓ is the specimen length (mm) between the grips.

If there is significant movement associated with the functioning of the grips, eg as with wedge grips with unrestricted closure, preliminary tests should be made to establish a rate of cross-head separation which induces an average rate of straining of 0.001 per min.

From a record of the load/deformation characteristics the modulus of elasticity shall be calculated as:

$$E = P\ell/bd\delta \times 10^3 \text{ kN/mm}^2$$

where δ is the average deformation (mm) under a load increment of P (N)
ℓ is the gauge length (mm)
b is the actual thickness of the section (mm)
d is the actual width of the section (mm)

The value of modulus of elasticity shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy.

6.2.2 Ultimate strength: The test arrangement and rate of straining shall be the same as in 6.2.1. Each specimen shall be loaded continuously to fracture and the ultimate tension stress shall be calculated as:

$$\sigma_t = P/bd \text{ N/mm}^2$$

where P is the maximum load (N)
b is the actual thickness of the section (mm)
d is the actual width of the section (mm)

The value of ultimate stress shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy. The mode of fracture shall be recorded together with the grade determining properties at the fracture zone if this is different from the previously identified and recorded vertical zone.

6.3 Compression
These tests provide for the measurement of modulus of elasticity and ultimate strength of structural timber parallel to the grain.

6.3.1 Modulus of Elasticity: Each specimen shall be loaded continuously in compression using spherical seated loading heads or other devices which ideally permit the application of uniform compression without introducing additional bending moments. The end surfaces of each specimen shall be accurately prepared to ensure they are plain and parallel to each other. The length of each specimen shall be 7 times the nominal width, ie greatest dimension of the section and shall have the critical zone within 1½ times the nominal width from its centre. To prevent buckling the faces and edges shall be restrained at a sufficient number of points so that there is no free length greater than 5 times the nominal dimension of the section in both directions.

Deformation shall be measured over a central gauge length of 5 times the nominal width of the section. Two compressometers shall be attached at diagonally opposite points on the faces to minimise the effects of distortion and to permit the determination of the average deformation of the full gauge length.

The rate of closure of the loading heads shall be.

$$R = 0.001\ \ell \text{ mm/min} \pm 25\%$$

Where ℓ is the specimen length (mm)

From a record of the load/deformation characteristics the modulus of elasticity shall be calculated as:

$$E = P\ell/bd\delta \times 10^3 \text{ kn/mm}^2$$

Where δ is the average deformation (mm) under a load increment of P (N)
ℓ is the gauge length (mm)
b is the actual thickness of the section (mm)
d is the actual width of the section (mm)

The value of modulus of elasticity shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy.

6.3.2 Ultimate strength: The loading conditions and the preparation of each specimen shall be the same as for 6.3.1 except that the length of the specimen shall be reduced to 6 times its nominal thickness, ie the least dimension of the section. The specimen shall contain the critical zone at its centre.

The rate of closure of the loading heads shall be:

$$R = 0.001\ \ell \text{ mm/min} \pm 25\%$$

Where ℓ is the length of the specimen (mm)

Each specimen shall be loaded continuously in compression and the ultimate compression stress shall be calculated as:

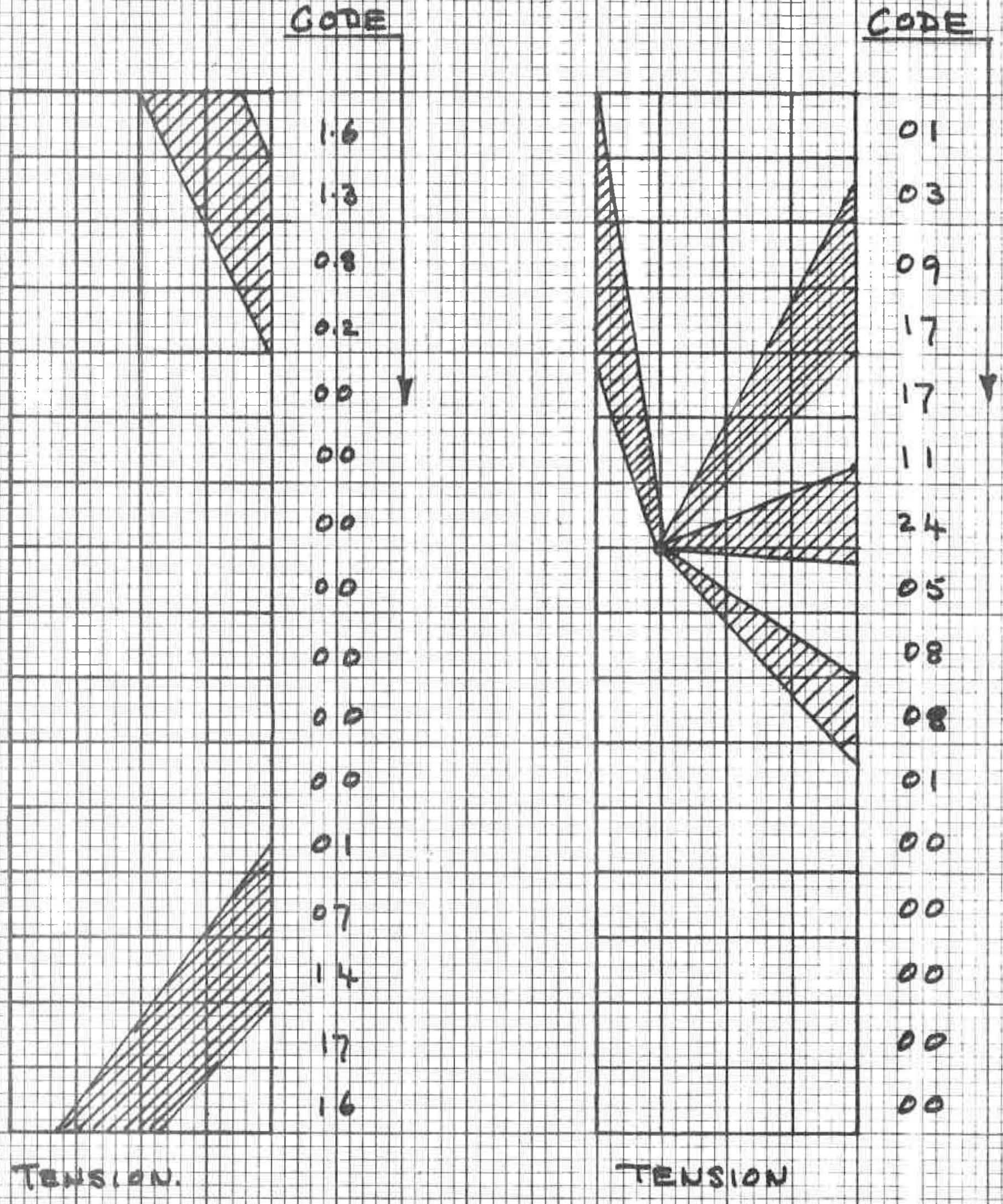
$$\sigma_c = P/bd \text{ N/mm}^2$$

Where P is the maximum load (N)
b is the actual thickness of the section (mm)
d is the actual depth of the section (mm)

The value of ultimate stress shall be expressed to three significant figures and the equipment used shall be capable of achieving at least this accuracy.

The mode of fracture shall be recorded together with the grade determining properties of the fracture zone if this is different from the previously identified and recorded critical zone.

Although there may be some difficulty in achieving sufficient accuracy in the measurement of deflection, the bending test is preferred to a torsion test for the determination of shear modulus since it relates directly to beam shear deflection. A constant national rate of straining of 0.001 per min has been maintained for all three tests with the result that the duration of a compression test will be very much greater than for bending and tension. The alternative is to increase the rate of straining in compression and accept that there is likely to be a different effect of rate of straining for this property. The determination of modulus of elasticity and ultimate strength have been defined separately since this simplifies the presentation and recognises that both properties may not always be required, nor need they necessarily be determined under the same conditions. Finally it should be noted that the properties, tension perpendicular to grain, compression perpendicular to grain and shear parallel to grain have not been covered in this draft.



STANDARD 4×16 GRID.

FIG. 1.

KNOT PLOTS AND NUMERICAL CODE

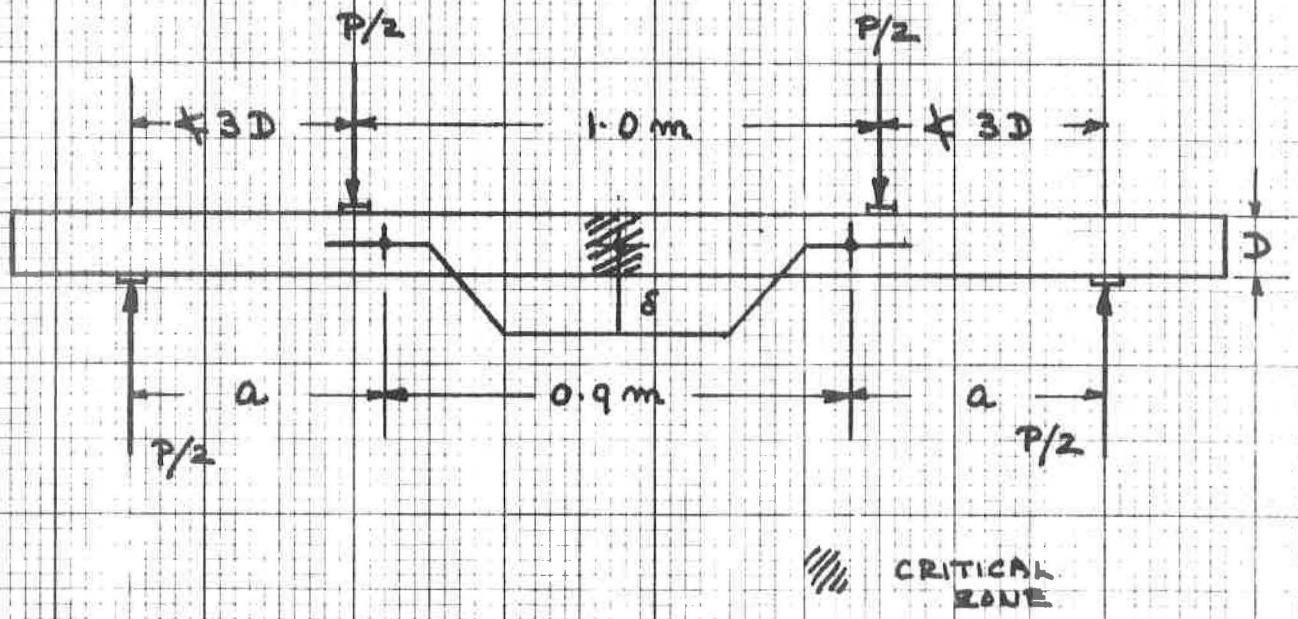


FIG 2.

DEFLECTION MEASUREMENT IN BENDING.

GRADE DETERMINING PROPERTIES MECHANICAL — E_T.

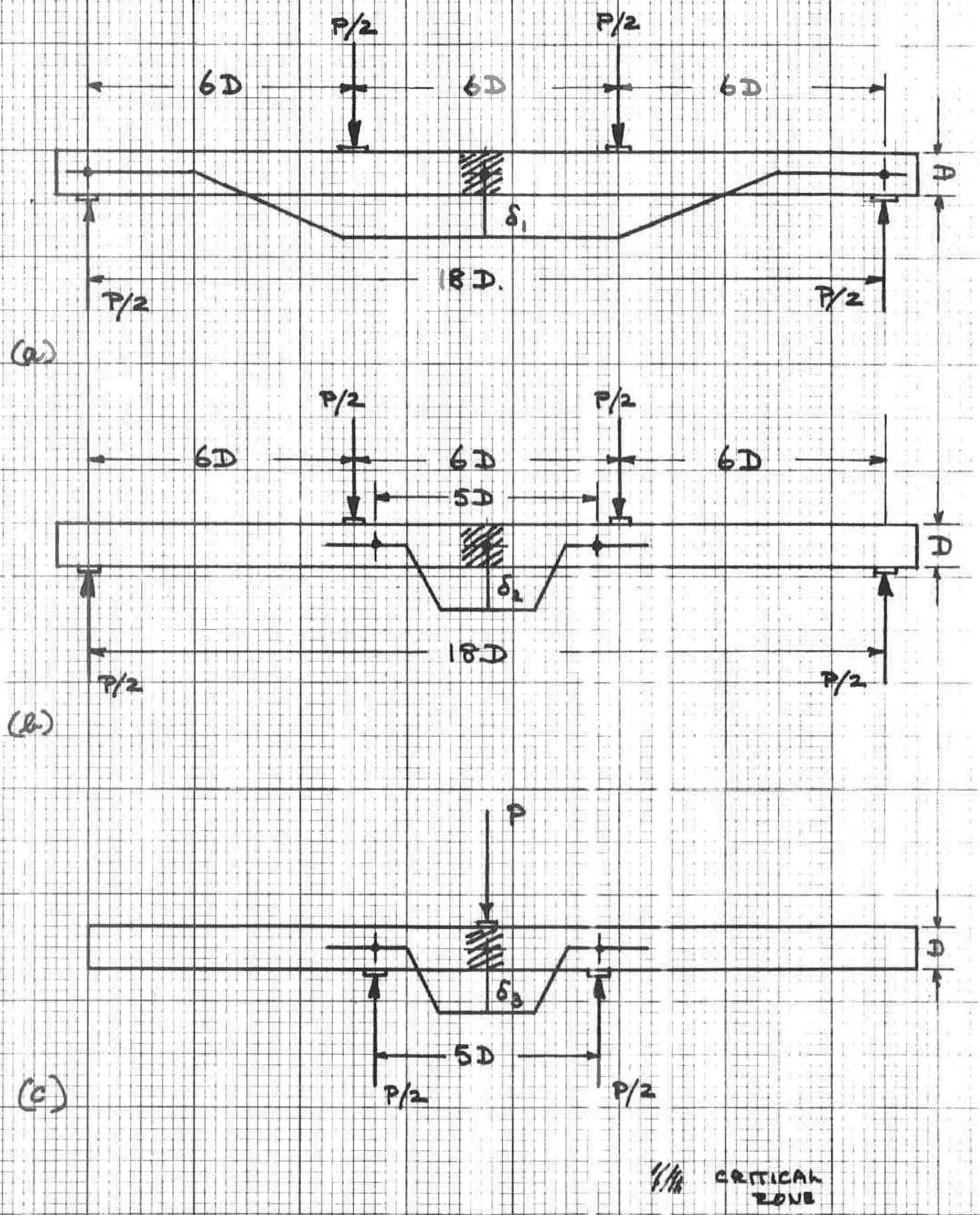


FIG. 3.

DEFLECTION MEASUREMENTS IN BENDING.

MODULUS OF ELASTICITY AND SHEAR MODULUS

CIB-W18/5-6-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE DESCRIPTION OF TIMBER STRENGTH DATA

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Karlsruhe - October 1975

THE DESCRIPTION OF TIMBER STRENGTH DATA

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INTRODUCTION

In the development of the ECE stress-grading rules for sawn softwoods the first consideration was to define limits for the grades which would ensure acceptable yields from commercial parcels of timber. The question of appropriate stress values was deferred because of foreseeable difficulties in reaching an agreed method for their determination. Since acceptance of the grades will finally depend not only on yields but, perhaps equally, on the stress values associated with them, the question of the determination of stresses was subsequently referred to the W18 Commission of CIB. Although the basic approach to the determination of stresses has the standard form:

$$\sigma_g = \sigma_m G/K$$

where σ_m is an estimate of a minimum value associated with a specified probability level

G is a grade strength ratio having a value of 1.0 if in-grade specimens are tested and

K is a composite reduction factor, or the product of a number of factors to allow for rate of loading, size effect, moisture content, safety factor etc.

The actual levels and magnitudes assigned to σ_m and K vary considerably between countries and each must retain the freedom to incorporate their own concepts of design and safety. The range of species and strength properties, the need for adequate samples and the cost of destructive testing of in-grade pieces of timber makes it highly desirable to develop a standard approach which will permit an interchange of test results. This involves adopting standard methods of test and a standard method of presenting results.

Since the derivation of grade stresses must start from the analysis of test results, the most explicit presentation would be to list the individual values. This would permit the combination of data from different sources and provide for the better estimate of the grade stress for a species as a whole. This may be impracticable, and even unnecessary, depending on the interpretation of what is required for a characteristic stress. The alternative is to consider condensing the data to correspond to the second stage of the analysis. This second stage may be taken as the definition of a minimum value σ_m, the determination of which would have to be standardised, or a sufficient range of statistics produced to enable it to be computed for any required distributional assumption and probability level.

For the determination of minimum values the choice of method lies initially between the assumption and fitting of a suitable probability function or the use of a non-parametric technique. In this paper a few commonly used methods are outlined and a comparison is made of the minimum values obtained by applying them to some actual test results.

PARAMETRIC METHODS

Normal Distribution

In the past the minimum stresses for structural timber were calculated on the assumption that strength values were normally distributed. This applied whether the small clear or full size specimen approach was used, the minimum value being determined as:

$$\sigma_m = (\bar{\sigma} - ts)$$

where $\bar{\sigma}$ is the mean of the test results

s is the standard deviation and

t is the appropriate value of students t[(1)] for the required probability level.

Only quite recently has this method been questioned, particularly as a result of the analysis of strength data obtained from tests on graded timber in structural sizes, where it was found that, unlike the results from small clear tests, the distributions of strength showed noticeable skew. Thus the theoretically infinite extension of the tails of a normal distribution, coupled with its use on positively skewed data, led to the calculation of negative minimum stress values. In such extreme cases the error is obvious but other errors, leading to overconservative estimates of minimum values have likely arisen and been undetected. The assumption of normal distribution is no longer generally favoured when the full size specimen approach is used for the determination of stresses.

Log-Normal Distribution

With positively skewed data an obvious choice of distribution was the log-normal distribution. This is also easy to use, since estimates of minimum values for the transformed variable can be obtained using normal distribution statistics. In its simplest form the distribution has a zero lower boundary value. A non-zero value could be introduced and although it would lead to lengthier calculations it could be expected to improve the fit of the distribution to some data. The probability density function for the three parameter log-normal distribution is

$$f(x;\ \mu,\ \sigma,\ \varepsilon) = \frac{1}{\sigma(x - \varepsilon)\sqrt{2\pi}} \exp\left[-\frac{1}{2\sigma^2}(\ln(x - \varepsilon) - \mu)^2\right]$$

$x \geq \varepsilon,\ -\infty < \mu < \infty,\ \sigma > 0,\ -\infty < \varepsilon < \infty$

where $\mu = \frac{1}{n}\Sigma\ln(x - \varepsilon)$, the scale parameter

$$\sigma = \left[\frac{\Sigma\left[\ln(x - \varepsilon)\right]^2}{n-1} - \frac{\left[\Sigma\ln(x - \varepsilon)\right]^2}{n(n-1)}\right]^{\frac{1}{2}}$$, the shape parameter

ε = the location parameter or lower boundary value of x for the distribution

To some extent the log-normal distribution lacks flexibility since it can only accommodate positive skew of an amount which is probably often extreme in relation to the strength data.

Weibull Distribution

The Weibull distribution, a fairly recent introduction to timber statistics, is currently popular in North America and Europe for describing strength data and determining minimum values. The probability density function for the three parameter distribution is

$$f(x;\ \eta,\ \sigma,\ \mu) = \left[\frac{\eta}{\sigma}\left(\frac{x - \mu}{\sigma}\right)^{\eta - 1} \exp\left(-\left(\frac{x - \mu}{\sigma}\right)^{\eta}\right)\right]$$

$x \geq \mu,\ -\infty < \mu < \infty,\ \sigma > 0,\ \eta > 0$

where σ is the scale parameter

η is the shape parameter

μ is the location parameter or lower boundary value of x for the distribution

If the location parameter μ is set to zero a two parameter Weibull distribution results. This would be easier to use but less suitable since as Pierce[2] has shown it cannot accommodate positive skew unless the coefficient of variation is greater than 30 per cent.

The three parameter distribution has advantages over the normal and log-normal distributions. It may have a positive lower boundary value of strength which is intuitively more realistic than a zero or negative value. It has some theoretical justification for representing the breaking strength of materials and it is sufficiently flexible to cover from slightly negative (-1.14) to substantially positive (2.0) skew. In addition Warren[3] suggests that the errors in estimates of minimum values resulting from fitting a Weibull to a truly normal distribution appear to be of less consequence than the errors resulting from fitting a normal curve to Weibull data.

The estimation of the three distribution parameters from test data generally requires the use of a computer for the solution of non-linear equations. A graphical method[4] is possible but is not very satisfactory. Once the parameters are estimated the minimum value of the variable for the required probability p is calculated from

$$\exp\left\{\left[\ln \ln \left[1/(1 - p)\right]\right] / \eta + \ln\sigma\right\} + \mu$$

Although the Weibull three parameter distribution has advantages its use entails considerably more computing than the normal or log-normal distributions.

NON-PARAMETRIC METHODS

As an alternative to assuming a particular type of distribution estimates of minimum values can be obtained from a ranking of test results using so-called non-parametric methods.

The method consists of arranging the results in ascending order x_1, x_2, x_3 ..... x_n, calculating for each value x_i its rank $i/(n + 1)$ and obtaining for example the 5 per cent minimum from the value of x whose rank is 0.05, or by interpolating between two values of x whose ranks span 0.05.

In ASTM D 2915-74[5] a method is introduced for establishing tolerance limits at the 95 and 99 per cent confidence levels to the estimates of 5 per cent minimum values, $x_{0.05}$ determined as above. For example the tolerance limit x_t with 95 per cent confidence is taken as the lowest test result from 58 tests or the second lowest from 93 tests. Then if $x_{0.05}$ is less than or equal to $1.05x_t$ the 5 per cent minimum is taken as $1.05x_t$ or the sample size is increased until $x_{0.05}$ is greater than $1.05x_t$. Warren[6] suggests that the conservative tolerance limit $1.05x_t$ would frequently be used and that this would be closer to a 2 per cent than a 5 per cent minimum value. It would be possible to extend the tables of Bendsten and Rattner[7] on which the ASTM tolerance limits are based, to other percentiles and confidence levels. Thus to obtain an estimate of the tolerance limit for a one per cent minimum with 95 per cent confidence would require the lowest test result from a sample of about 300 to be determined, a prohibitive test programme.

Another non-parametric method has been suggested by Madsen[8] which requires the fitting of an odd order polynomial to the normalised ranked data and using an "interpretation point" to estimate the 5 per cent minimum. The method was aimed at providing reasonable estimates of minima from small samples (probably less than 50) or from destructive tests which are confined, by a process of proof loading, to the weaker members in a sample. The method appears to have little if any advantage over the other methods and becomes increasingly suspect as the reciprocal of the number of tests approaches and exceeds the required probability level.

OTHER STATISTICS

Although it is the definition of a minimum value of ultimate strength which is the most important single statistic for the derivation of grade stresses, and for the comparison of the strength properties of different grades, consideration should also be given to the inclusion of other statistics in the presentation of the data. This may in fact be unavoidable if there is no agreement as to what standard minimum value should be used. In this case it would be necessary to specify the parameters for the continuous distributions and the actual values and ranks for the non-parametric methods determined for the sample test results. Consideration might also be given to including other statistics, such as the mean, median, standard deviation, skewness and kurtosis as indicators of the central tendency, dispersion and symmetry of the data. Tolerance limits for some of the statistics where they can be determined, could also be included.

APPLICATION

To indicate what would be involved in the presentation of the data and to provide a comparison of the minimum values obtained by the different methods, three sets of timber data were analysed. The first comprised modulus of rupture values for 438 pieces of 38 x 150 mm Swedish redwood/whitewood of mixed U/S, V and VI commercial grades. The other two sets of data comprised the strength values for individual pieces selected from these by a stress grading machine as conforming to two arbitrary stress grades.

Table 1 lists the general statistics for the three sets of data from which it can be seen that they exhibited both slightly negative and positive skewness and, as would be expected, that the samples selected by the machine showed less dispersion in their strength values. The distribution parameters for the five continuous distributions that have been described are given in Table 2. In the case of the three-parameter log-normal distribution the lower boundary of strength was chosen as 0.9 of the minimum value in each set of data. Histograms, with superimposed probability density functions, for the normal, log-normal and two- and three-parameter Weibull distributions are shown in Figures 1 to 3 and it can be clearly seen that the three-parameter Weibull provides the most satisfactory fit. As a further illustration of the closeness of fit the cumulative frequency distributions of the test data and the cumulative functions for the three-parameter Weibull distributions are given in Figures 4 to 6.

As regards minimum values of strength most interest is centred on the 5 per cent values and to a lesser extent on the one per cent values. These have been determined, where possible, for both the parametric and non-parametric methods and are given in Table 3. Taking the three-parameter Weibull values for reference it can be seen that, depending upon the method used, the minimum values range from 0.80 to 1.10 for the 5 per cent, and from 0.51 to 1.39 for the one per cent, times these values. Obviously the method employed can have a considerable influence on estimates of minimum values and a preference for a particular method can only be based on judgment.

CONCLUSIONS

Unless it becomes possible to agree on a standard method for deriving characteristic stresses, or permissible grade stresses, using the same values for the various partial coefficients or reduction factors that are involved, the following alternatives for the presentation of strength data for stress-graded timber should be considered.

1. Listing individual test results so that each country can employ its own method of analysis.

2. Listing the general statistics indicating central tendency, dispersion and symmetry of the test results.

3. Listing the distribution parameters for the continuous distributions commonly used, or for a standard type of distribution if this could be agreed.

4. Listing the individual results and their ranking for the non-parametric or distribution-free methods commonly used.

5. Listing estimates of minimum values for various probability levels, or for one level if this could be agreed.

Finally it should be recalled that the objects of considering how strength data should be presented were:

a. To allow comparisons to be made between the strength characteristics of the ECE proposed grades and present national grades. These can only be made if comparable data are available for all the grades involved, and this is not likely to be the case.

b. To permit assessments to be made of the effect on the strength characteristics of possible changes in the ECE grade limits. This could be achieved by comparisons of minimum values for comparable samples, using an agreed method for their determination.

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Table 1

SAMPLE DESCRIPTIONS FOR BENDING STRENGTH (N/mm^2)

Sample	Sample Size	Median	Mean	Standard Deviation	Skewness	Kurtosis	Minimum Value	Maximum Value
1	438	43.55	43.56	13.43	-0.13	2.30	11.70	75.40
2	106	55.95	55.23	9.56	-0.72	3.59	22.30	75.40
3	86	28.75	29.21	9.94	0.52	3.22	11.70	61.80

Table 2

DISTRIBUTION PARAMETERS

Distribution	Parameter	Sample		
		1	2	3
Normal	location (mean)	43.56	55.23	29.21
	scale (std deviation)	13.43	9.56	9.94
Log-normal	scale	3.718	3.994	3.315
	shape	0.3540	0.1953	0.3548
Log-normal 3-parameter	location; $0.9x_1$	10.53	20.07	10.53
	scale	3.377	3.503	2.738
	shape	0.5661	0.4015	0.7056
Weibull 2-parameter	scale	48.35	59.05	32.63
	shape	3.69	7.12	3.16
Weibull 3-parameter	location	0.00	9.03	8.31
	scale	48.40	50.53	23.71
	shape	3.54	4.85	2.14

Table 3

ESTIMATES OF THE 5 PER CENT AND 1 PER CENT MINIMUM VALUES OF BENDING STRENGTH (N/mm^2)

METHOD		5 PER CENT MINIMUM						1 PER CENT MINIMUM					
Distribution	Ref	1		2		3		1		2		3	
		Value	Ratio*	Value	Ratio*	Value	Ratio*	Value	Ratio*	Value	Ratio*	Value	Ratio*
Normal	A	21.5	1.03	39.4	1.08	12.7	0.89	12.3	0.93	32.7	1.14	5.7	0.51
Log-normal	B	23.0	1.10	39.3	1.08	15.3	1.07	18.1	1.37	34.2	1.20	11.9	1.07
Log-normal	C	22.1	1.06	37.1	1.02	15.3	1.08	18.4	1.39	33.0	1.15	13.4	1.21
Weibull	D	21.6	1.03	38.9	1.07	12.7	0.89	13.9	1.05	30.9	1.08	7.6	0.69
Weibull	E	20.9	1.00	36.4	1.00	14.2	1.00	13.2	1.00	28.6	1.00	11.1	1.00
Ranking	F	20.8	1.00	37.5	1.03	14.0	0.98	13.9	1.06	22.8	0.80	10.2	0.92
ASTM tolerance limit	G	17.2	0.82	29.1	0.80	11.7	0.82	-	-	-	-	-	-
ASTM allowable	H	18.1	0.86	30.6	0.84	12.3	0.86	-	-	-	-	-	-
Nearest order statistic	J	20.8	1.00	36.9	1.01	13.8	0.97	13.8	1.04	22.3	0.78	-	-

*The ratio is the factor by which the minimum value, estimated from the assumption of a three-parameter Weibull distribution, must be multiplied to obtain the minima estimated by the other methods.

Ref METHOD

A $(\bar{\sigma} - 2.33S)$ for one per cent and $(\bar{\sigma} - 1.64S)$ for 5 per cent

B Lower boundary value zero

C Lower boundary value 0.9x, where x_1 is the lowest test result.

D Two-parameter Weibull

E Three-parameter Weibull

F Minimum value given by $\left[p(n + 1) - (j - 1)\right]\left[x_j - x_{j-1}\right] + x_{j-1}$
where p is required probability level, n sample size, j the first order value for x where $i/(n + 1) \geq p$.

G Tolerance limit at 95 per cent confidence level

H 1.05 times the tolerance limit if $(x_{0.05} - x_t) \leq 0.05x_t$

J The nearest order value of x given by $\leq (n + 1)p$

Fig 1 : Sample 1

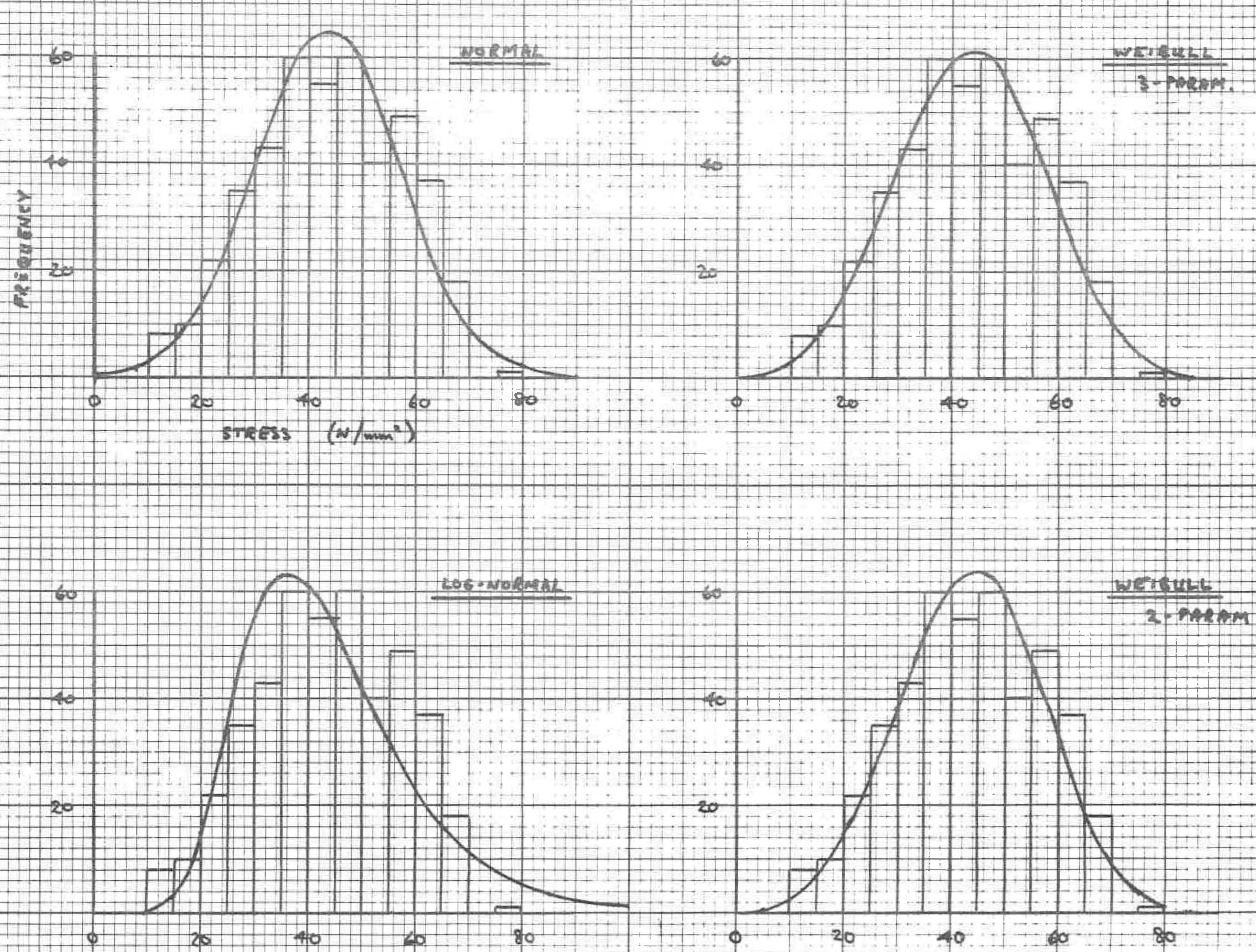


Fig 2 : Sample 2

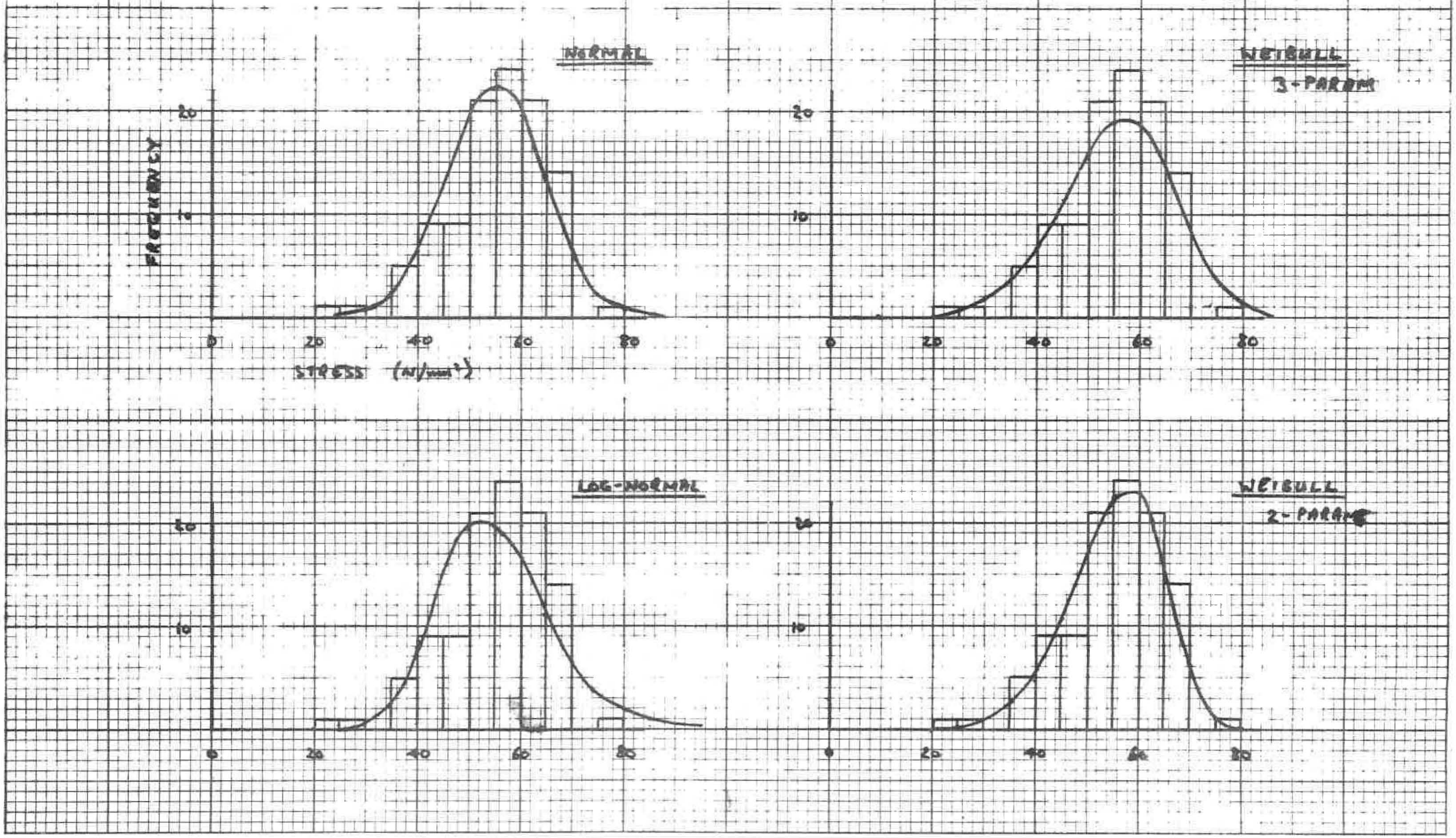
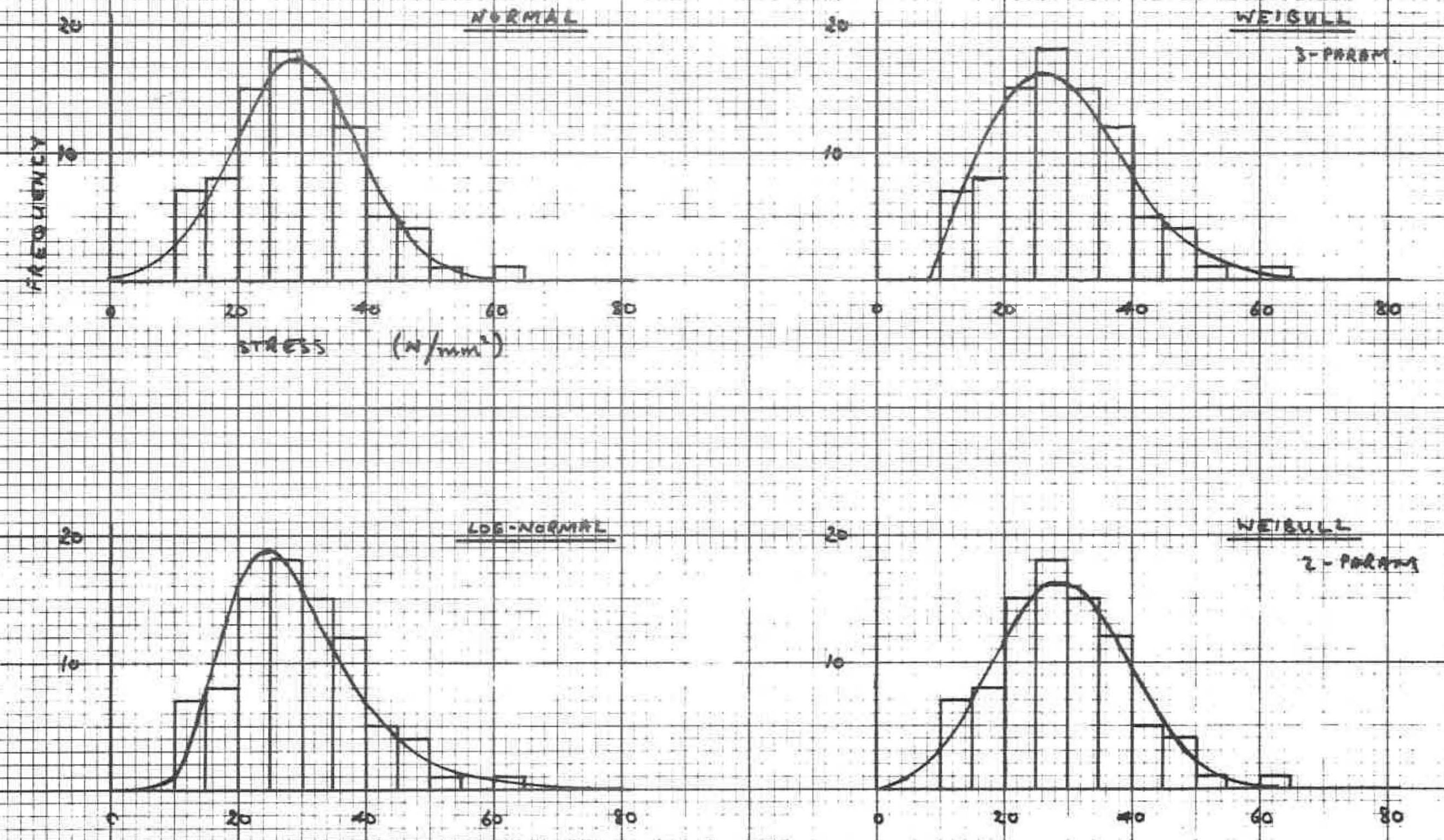


Fig 3 : Sample 3



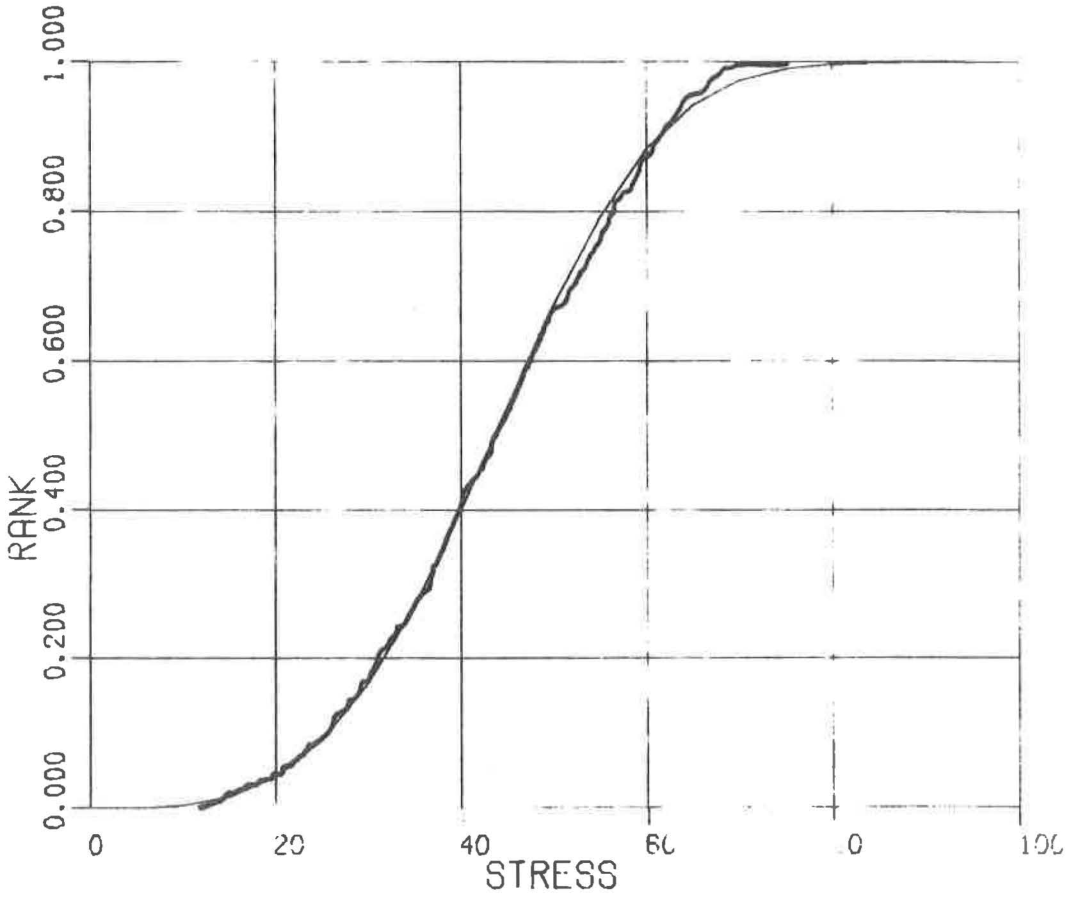


Fig 4 SAMPLE 1. Cumulative distribution and 3-parameter Weibull cumulative function

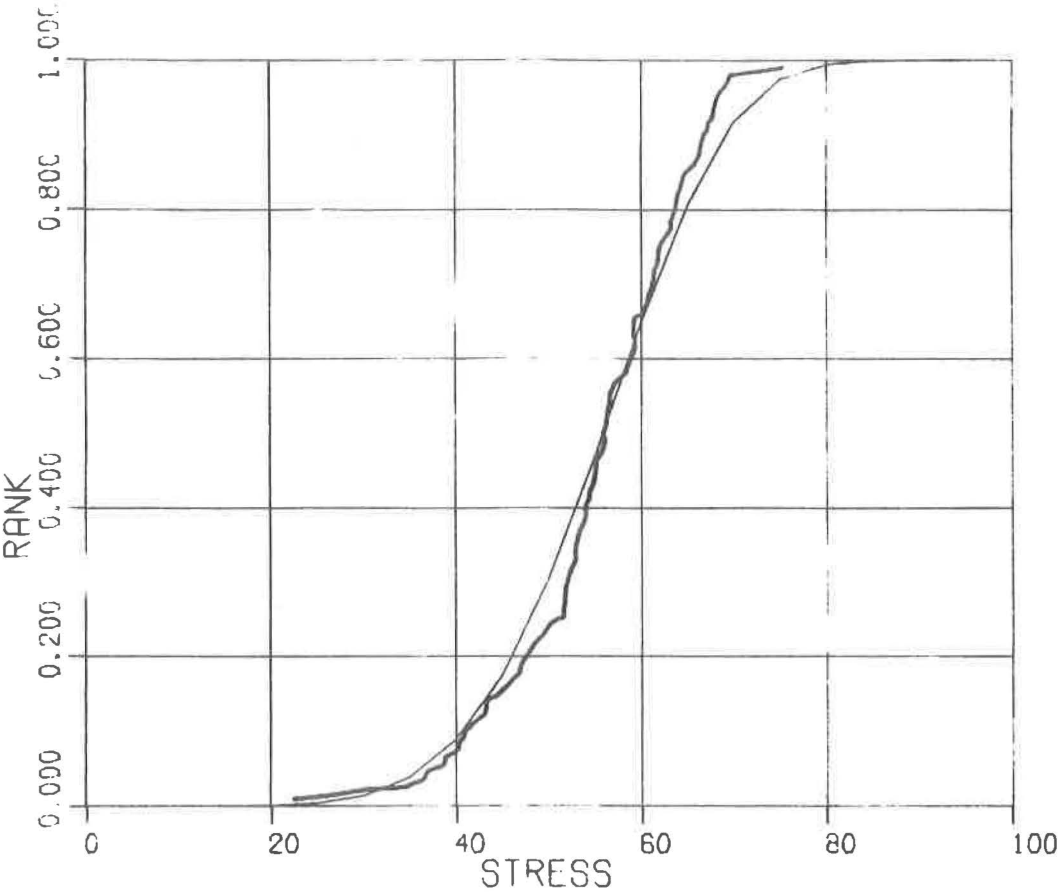


Fig 5 SAMPLE 2. Cumulative distribution and the 3-parameter Weibull cumulative function

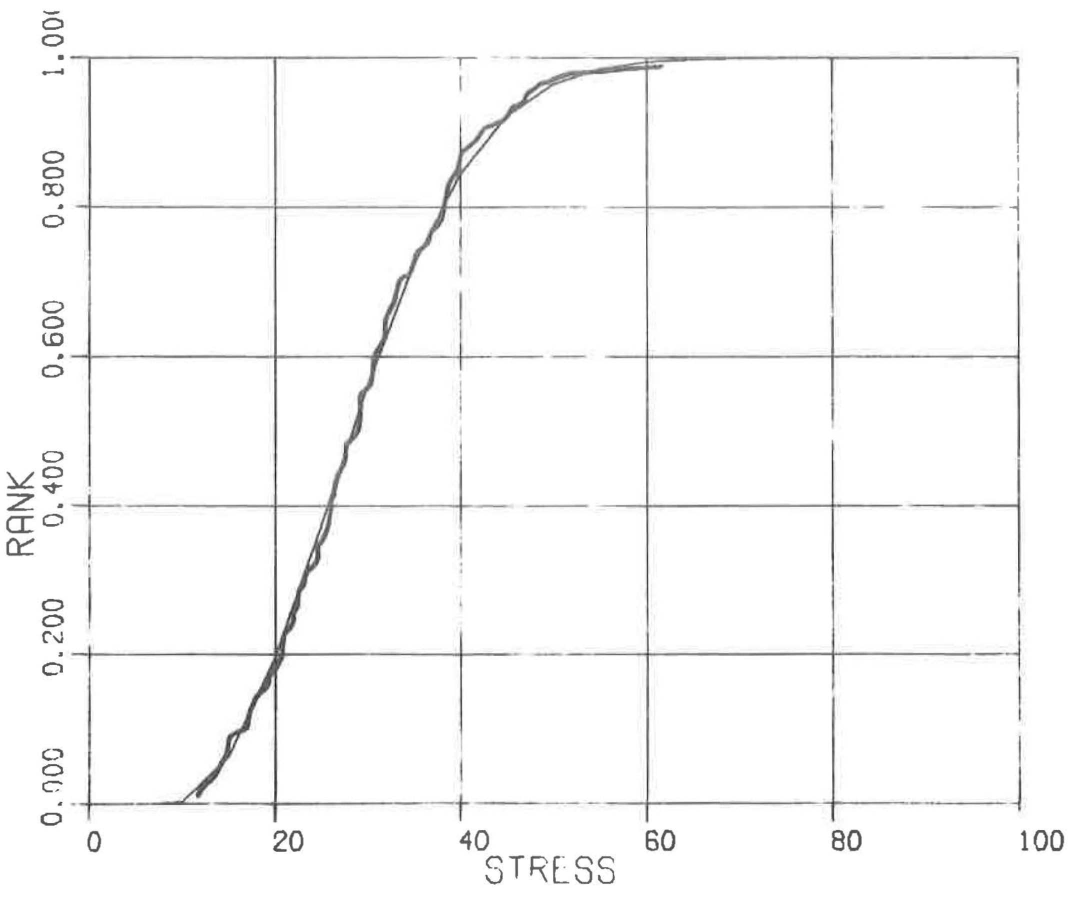


Fig 6 SAMPLE 3. Cumulative distribution and the 3-parameter Weibull cumulative function

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STRESSES FOR EC1 AND EC2 STRESS GRADES

by

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KARLSRUHE - OCTOBER 1975

STRESSES FOR EC1 AND EC2 GRADES

by J R Tory, Princes Risborough Laboratory, United Kingdom

This paper provides an indication of the likely stresses for a variety of samples of EC1 and EC2 material of mixed Swedish/Finnish redwood/whitewood. The samples were selected from the results of the Anglo/Scandinavian project on bending strength and comprised four sizes from Vth and VIth quality graded material. EC1 and EC2 grades were selected from the size samples on the basis of knot area ratios alone. Relevant statistics to describe the samples are given in Table 1. Analysis of the results to provide estimates of the first and fifth percentiles was by two non-parametric methods and by four different distributional assumptions. The calculated fifth and first percentiles are listed in Table 2. Many of the samples were too small to permit the non-parametric estimation of the first percentiles and so these have been excluded from the Table. The individual samples were combined after they had been adjusted to 200 mm depth.

Examination of the fifth percentiles in Table 2 shows that EC1 grade selected from VIths is assigned lower stresses than EC2 grade selected from Vths. It can also be seen that in a few instances in the VIth material EC2 is assigned higher stresses than EC1. These somewhat disturbing results could perhaps have been brought about by the relatively few pieces whose failure could not have been attributed to knots or whose KAR grade should have been determined by other defects.

The quite significant differences between the stresses for the grades when selected from Vth quality material and those for the same grades selected from VIths suggests that before finally assigning stresses to the grades some care should be taken to ensure that the sample from which the stresses are to be calculated is reasonably representative of the relevant population of carcassing timber. For example; should one expect stress grading to follow the present commercial grading operations or will stress grading in the future be performed on saw-falling timber from which the joinery material has been creamed off?

Table 1

SAMPLE STATISTICS. MODULUS OF RUPTURE. SWEDISH/FINNISH REDWOOD/WHITEWOOD

Sample	Quality Grade	KAR Grade	Sample Size	Mean	Standard Deviation	Relative Skewness	Coeff of Variation %
38 x 100 mm	V	EC1	16	45.5	11.9	0.06	26.1
		EC2	71	35.4	8.0	0.54	22.7
	VI	EC1	11	36.9	14.4	0.38	39.1
		EC2	69	34.7	9.7	0.53	28.0
38 x 150 mm	V	EC1	80	45.2	10.6	0.20	23.5
		EC2	142	36.6	9.5	0.23	25.8
	VI	EC1	68	37.3	10.9	-0.26	29.2
		EC2	115	30.8	10.8	0.62	35.0
50 x 150 mm	V	EC1	75	42.8	10.3	0.42	24.1
		EC2	84	36.4	7.8	0.07	21.3
	VI	EC1	65	40.6	11.6	-0.03	28.6
		EC2	51	34.8	9.9	0.67	28.4
50 x 200 mm	V	EC1	54	43.2	10.3	0.10	23.9
		EC2	106	33.0	7.4	0.29	22.6
	VI	EC1	56	40.1	14.0	-0.43	34.9
		EC2	79	29.4	9.0	0.21	30.7
Combined	V	EC1	225	41.9	10.1	0.24	24.0
		EC2	403	33.6	8.0	0.31	23.8
	VI	EC1	200	37.5	11.9	-0.13	31.8
		EC2	314	30.2	9.5	0.48	31.4

Table 2

FIRST AND FIFTH PERCENTILES OF MODULUS OF RUPTURE. SWEDISH/FINNISH REDWOOD/WHITEWOOD

Sample	Quality Grade	KAR Grade	Lower 5%						Lower 1%			
			Cumulative Ex Level	ASTM D2915 Allowable	Normal	Log-Norm	Weibull 2-param	Weibull 3-param	Normal	Log-Norm	Weibull 2-param	Weibull 3-param
38 x 100 mm	V	EC1	-	-	24.8	27.2	25.6	25.1	14.8	21.6	17.7	20.6
		EC2	23.5	19.5	22.0	23.6	20.3	22.6	16.2	20.1	14.3	18.8
	VI	EC1	-	-	11.0	16.1	15.0	12.7	-ve	11.0	8.6	7.3
		EC2	19.1	12.1	18.5	20.4	17.4	18.2	11.6	16.5	11.3	12.1
38 x 150 mm	V	EC1	27.7	21.3	27.5	29.2	26.1	27.2	19.9	24.5	18.4	20.9
		EC2	22.3	18.7	20.9	21.9	19.7	19.9	14.4	18.0	13.3	13.5
	VI	EC1	18.5	9.9	19.2	20.0	19.6	17.6	11.4	15.7	13.0	11.0
		EC2	14.9	14.5	12.9	16.0	13.1	15.2	5.4	12.5	7.7	12.1
50 x 150 mm	V	EC1	27.1	24.3	25.6	27.6	23.9	26.2	18.3	23.2	16.5	21.5
		EC2	22.9	18.9	23.5	24.5	22.2	23.0	17.9	20.9	16.2	18.3
	VI	EC1	19.3	14.6	21.2	22.5	21.0	19.9	12.8	17.8	13.8	12.7
		EC2	21.6	-	18.3	21.1	17.4	21.3	11.0	17.2	11.3	19.4
50 x 200 mm	V	EC1	24.5	-	25.9	27.4	24.7	25.4	18.4	22.8	17.4	19.8
		EC2	22.3	17.7	20.6	21.8	19.2	20.6	15.4	18.5	13.6	16.4
	VI	EC1	10.7	-	16.7	16.7	18.0	13.7	6.5	11.9	11.0	7.0
		EC2	16.6	14.1	14.4	16.4	14.4	15.7	7.9	13.0	9.2	13.1
Combined	V	EC1	25.8	24.4	25.4	27.0	23.7	25.7	18.5	22.8	16.5	20.9
		EC2	21.5	21.5	20.4	21.6	18.9	20.0	15.0	18.2	13.1	14.4
	VI	EC1	19.1	13.8	17.9	18.8	18.0	16.7	9.8	14.4	11.3	10.1
		EC2	15.8	15.1	14.6	16.8	14.1	15.8	8.1	13.5	8.7	12.1

CIB-W18/5-7-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

INFLUENCE OF LOADING PROCEDURE ON STRENGTH AND SLIP-BEHAVIOUR IN TESTING TIMBER JOINTS

by

Prof Dr Ing K Mohler

Universitat (TH) Karlsruhe

KARLSRUHE - OCTOBER 1975

INFLUENCE OF LOADING PROCEDURE ON STRENGTH AND SLIP-BEHAVIOUR AT TESTING TIMBER JOINTS

The loading procedure for testing timber joints is very different in various countries. This was stated by P Vermeyden, Stevin Laboratory Delft, in his publication in "Heron" Nr. 2, 1963. In Germany the procedure used is according to DIN E 4110, sheet 8, but the time taken for one test is long (about 30 to 60 minutes). The tests proposed by VERMEYDEN have a duration of 6 to 8 minutes.

Using modern mechanical or hydraulic testing machines, especially those which are regulated electronically, the rate of displacement of the movable head of the testing machine can be fixed step-by-step. On some machines this is also possible for the rate of loading.

For testing timber joints it seems to be useful to work within the range below 70 per cent of the ultimate load with a low rate of deformation or with an equivalent rate of loading. For the remaining part of the test the rate of deformation should be increased, because large displacements occur in mechanical timber joints when the load exceeds 70 per cent of the ultimate load. If no failure of the timber joint occurs before the deformation has reached a certain limit-value, the load at this limit should be defined as the "ultimate load". Values of 7.5 mm and 15 mm have both been proposed as a suitable "limit-deformation". In research work started a few months ago preliminary tests were made with double shear nail joints (8 nails 34/90) using 8 different loading procedures. These are described in appendix 1. Appendix 2 gives the test results and shows the ultimate loads, the total slips at a loading of 1.2 times the allowable load (which corresponds to about 40 per cent of the ultimate load), and the total time required for each test. The load - slip diagrams of all the test procedures are shown in appendix 3 to 10.

CONCLUSIONS

With procedure A 1 according to DIN E 4110, sheet 8, the lowest ultimate loads and the largest slips (.44 mm) were reached at 1.2 times the allowable load. The time needed for one test was 27.5 minutes.

Using the procedures B with constant rates of deformation the ultimate loads were always higher and the slips were smaller. Similar results were found with loading procedures C. Procedure C 2 corresponds to the procedure described by Vermeyden as that used at the Stevin Laboratory during the last few years.

Comparing procedure C 2 with procedure A 1 showed an increase of the ultimate load of about 9 per cent and a smaller slip, at 1.2 times the allowable load, of about 5 per cent. The time needed for one test was about 8 minutes. This is less than one third of the time needed for the German loading procedure.

A further series of tests were began in September 1975 using the loading procedures A 1, B 4 and C 2 with nails, nail-plates and wood-screws in order to establish the differences between these three procedures more exactly. This will enable definite proposals to be made for the loading procedures to be used for testing mechanical timber joints.

The safety coefficient for ultimate loads found by tests and the limit-values of slips at the allowable loads accepted must be agreed together with the number of test specimens and the method for the statistical evaluation of test results.

Appendix 1 - 10

Loading Procedures Used for the Preliminary Tests

Series A According to DIN E 4110, sheet 8

2 minutes waiting time at each loading step
$p = 0.1 \cdot \max P$ ($\max P$ = ultimate load),
first loading until $4 \cdot p$ and removal of the load until zero,
eleven times renewed continuous increase and decrease of the load between zero and $4 \cdot p$,
finally load increase step-by-step until ultimate load.

A 1: Loading each step with a rate of deformation of 2 mm/min.

A 2: Loading each step within 30 seconds (rate of loading = p/30 sec = $2 \cdot p$ kp/min).

Series B Loading with a constant rate of deformation (r.o.d.)

B 1: Loading with a r.o.d. of 2 mm/min continuously until ultimate load.

B 2: B 1, but with a r.o.d. of 1 mm/min.

B 3: Loading with a r.o.d. of 2 mm/min continuously with one removal of load from $4 \cdot p$ to p.

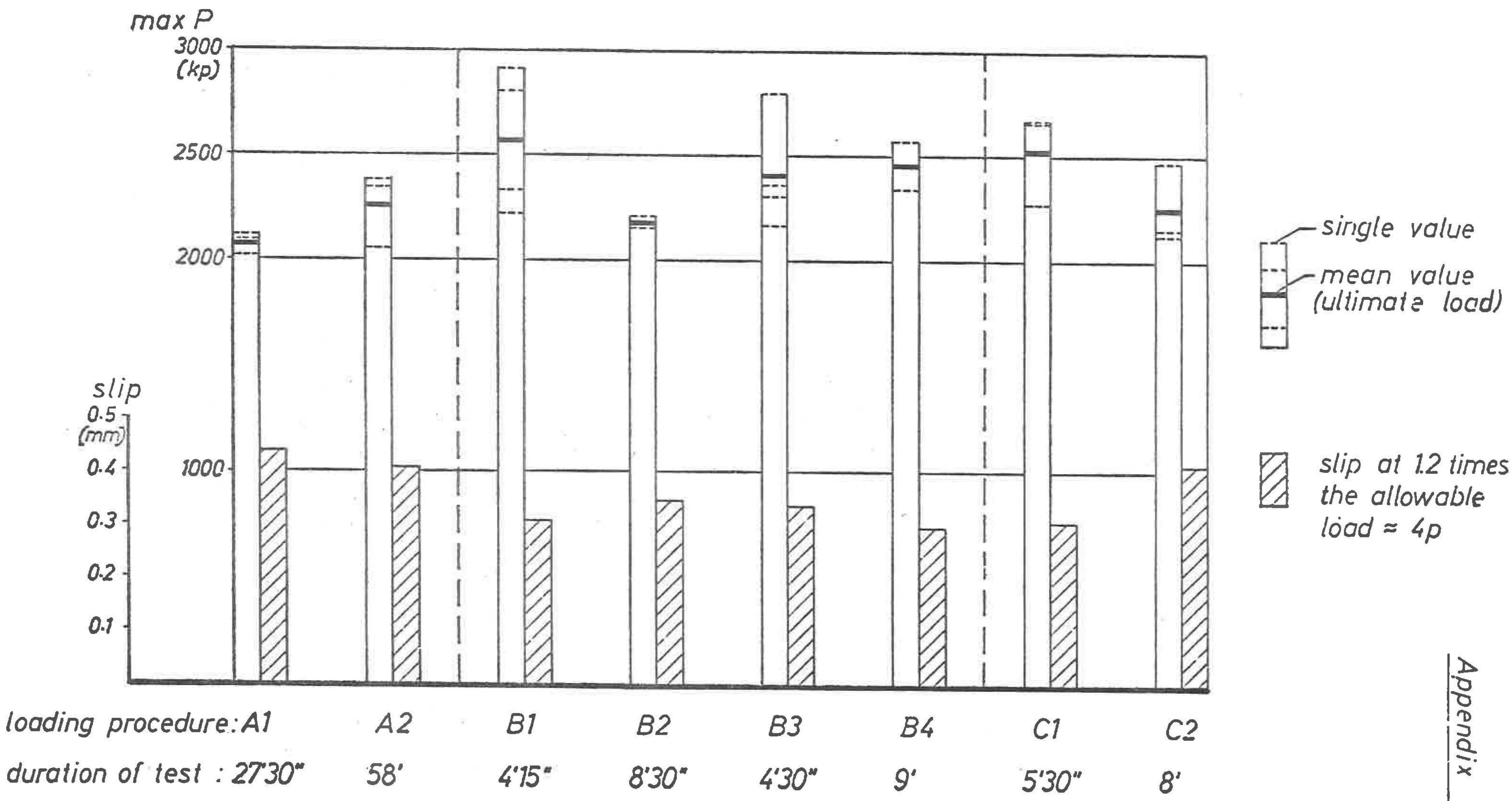
B 4: B 3, but with a r.o.d. of 1 mm/min.

Series C Loading with a constant rate of loading

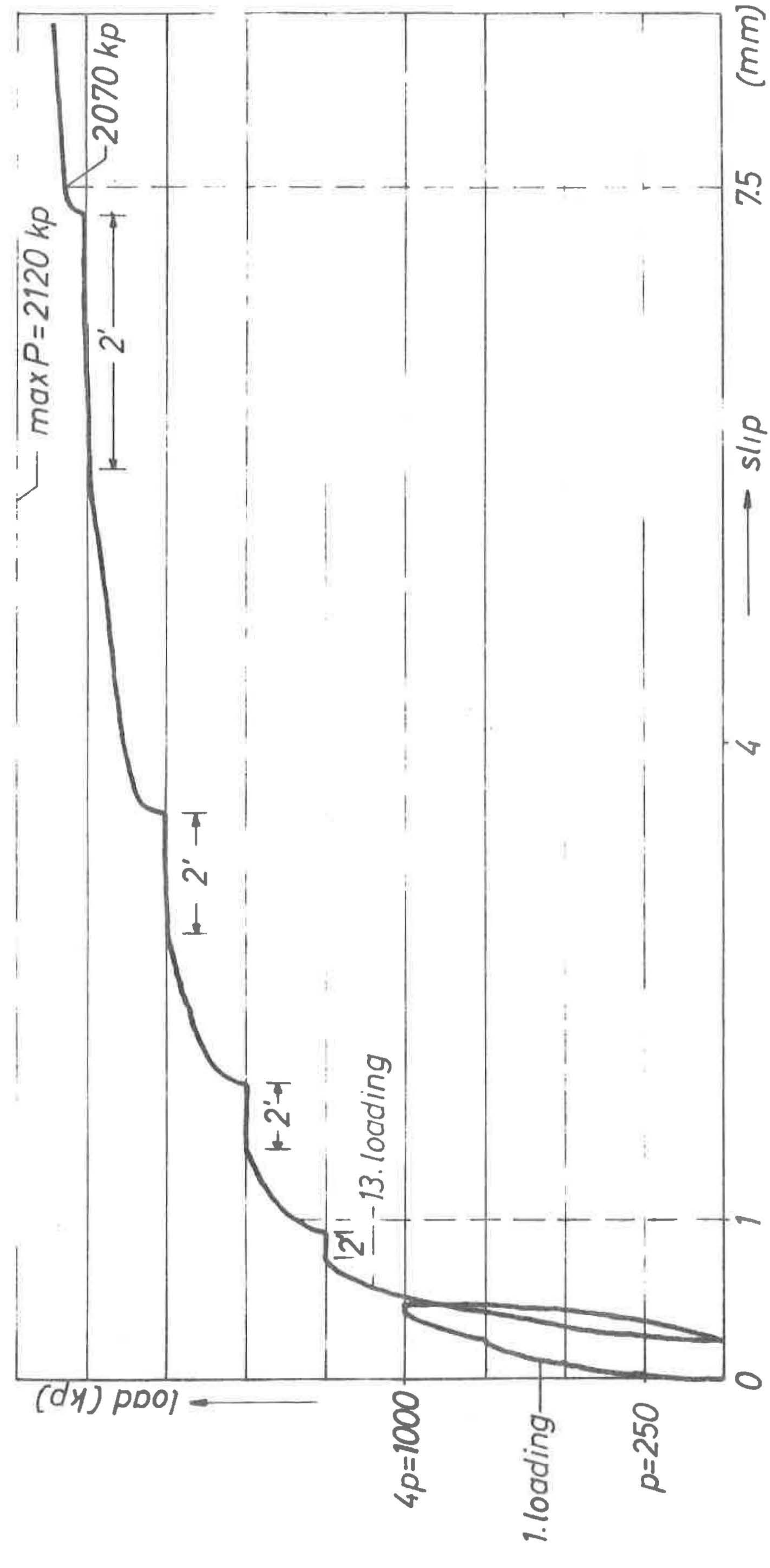
C 1: Rate of loading continuously p/30 sec until ultimate load.

C 2: Rate of loading continuously p/30 sec until $7 \cdot p$ with one removal of load from $4 \cdot p$ to p. After the load has reached $7 \cdot p$, the rate of deformation just arrived is to be kept constant until ultimate load.

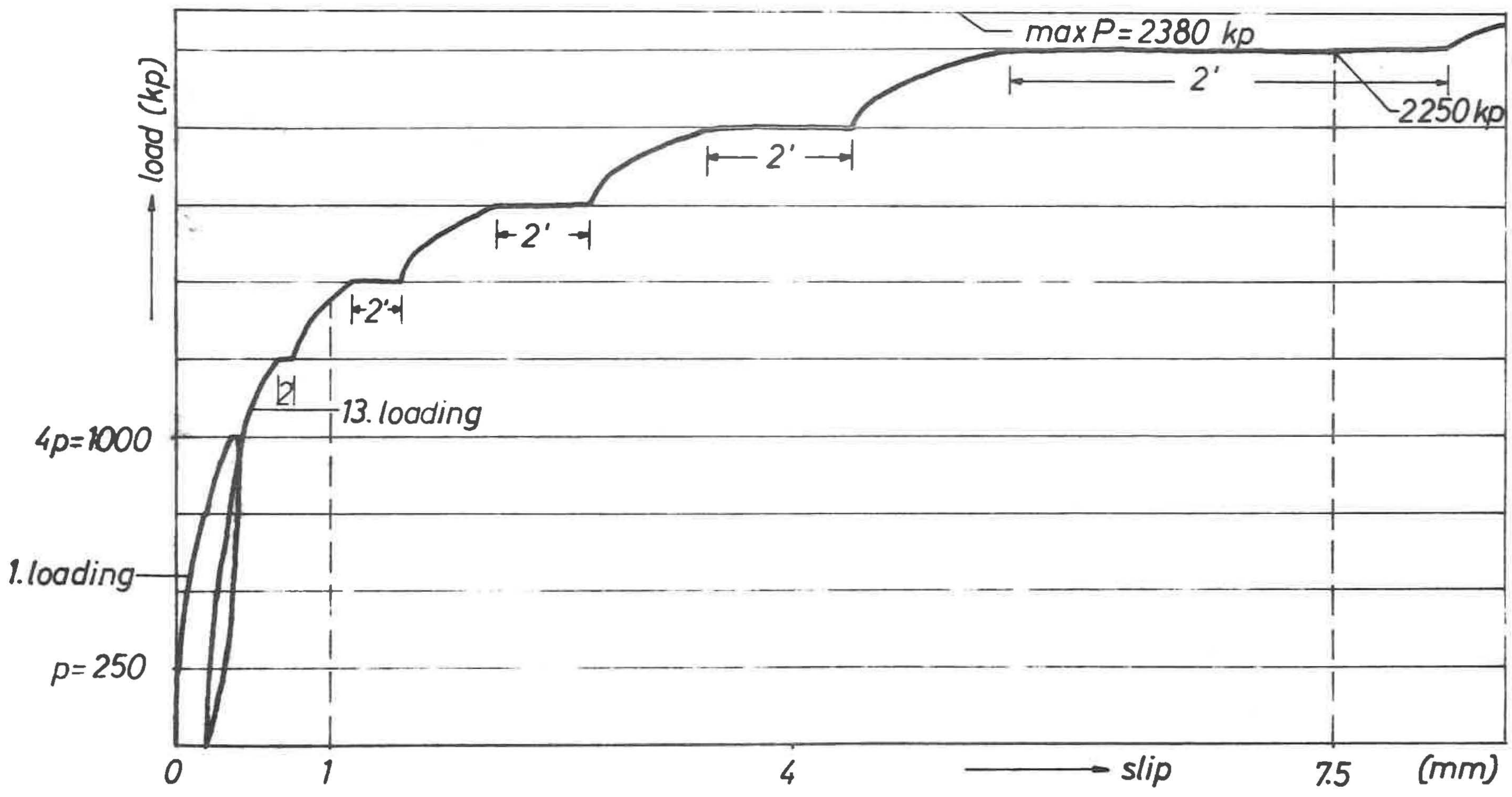
Preliminary test with nail joints and different loading procedures



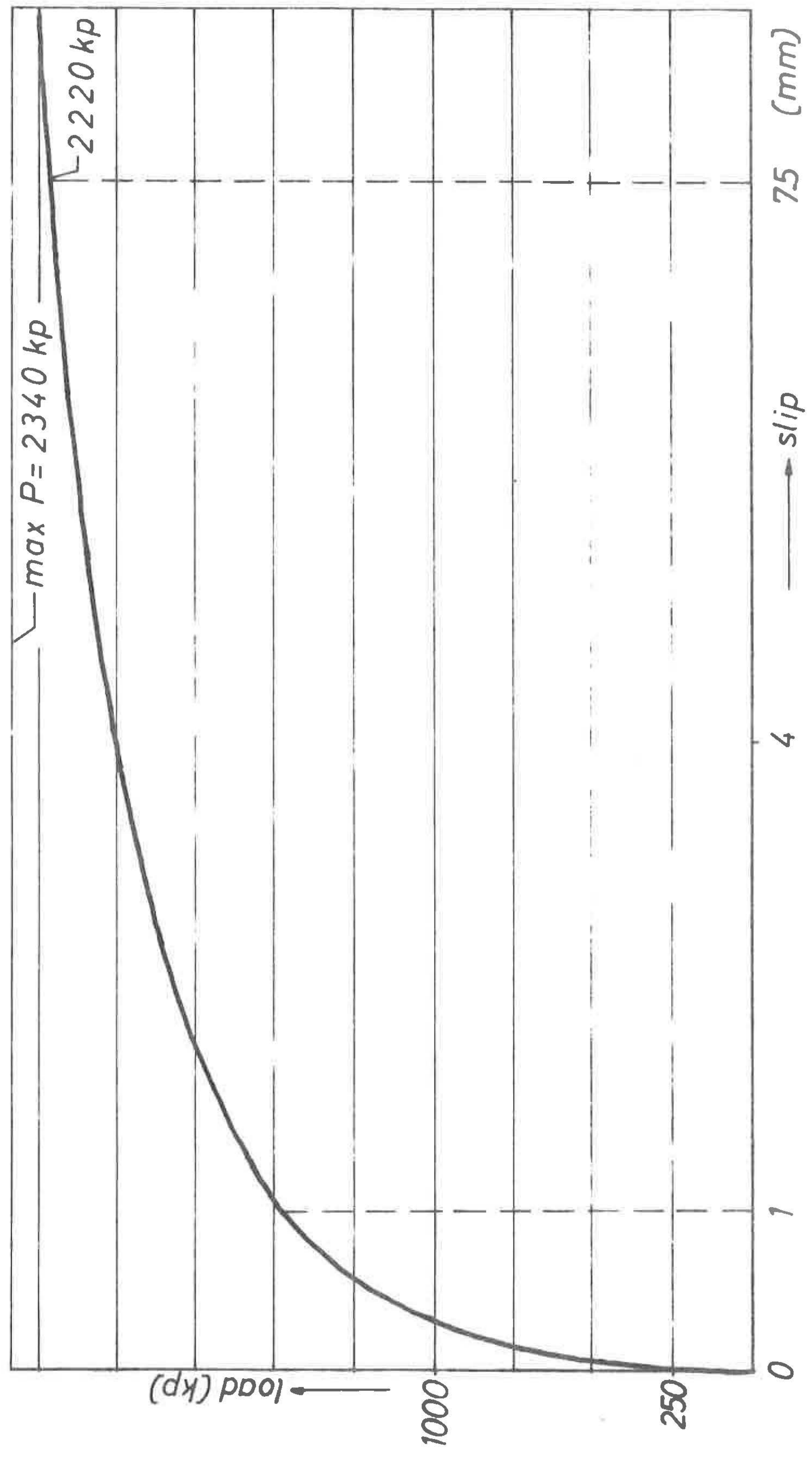
Appendix 2



Load - slip diagram for A1

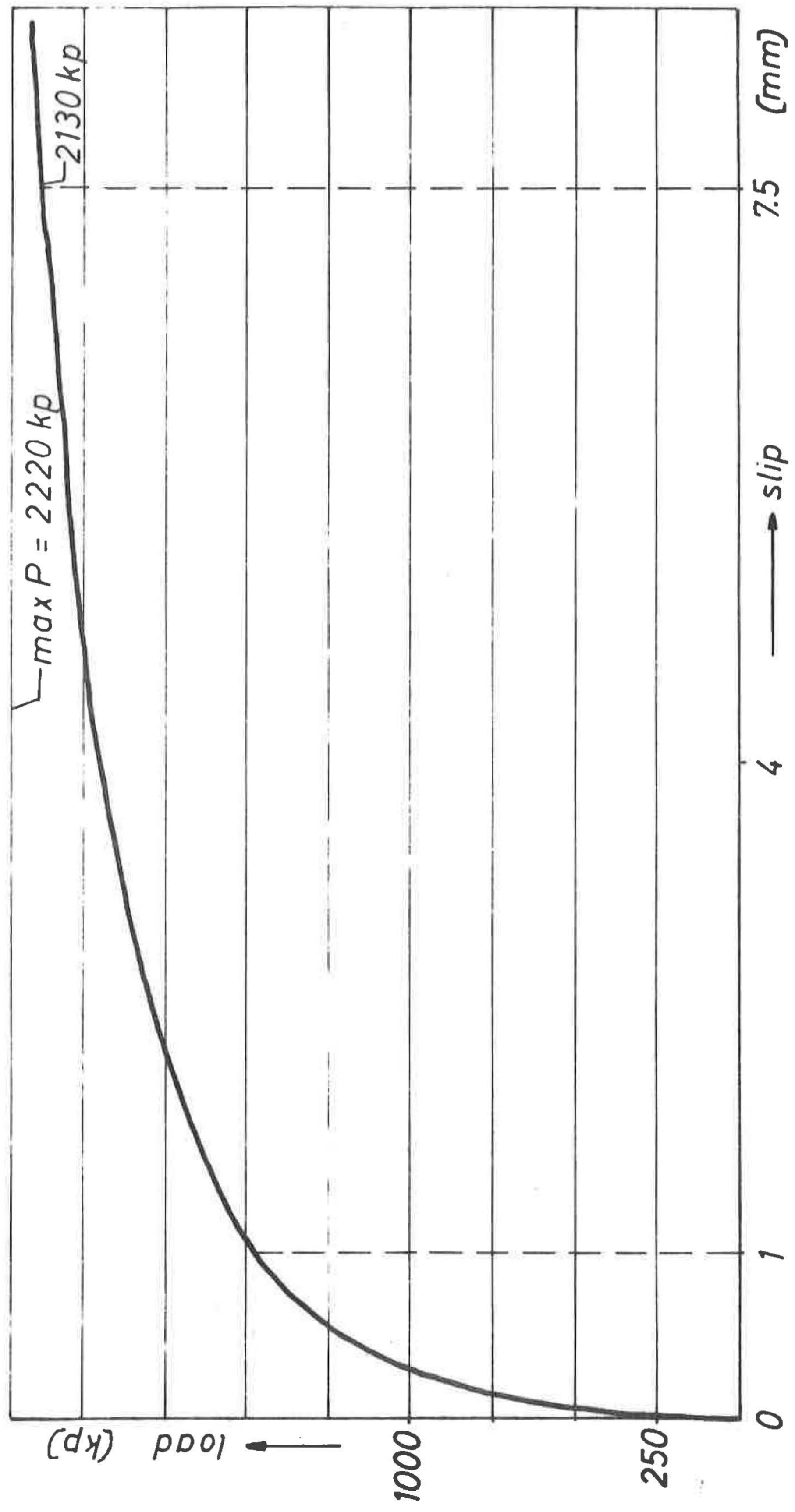


Load-slip diagram for A2

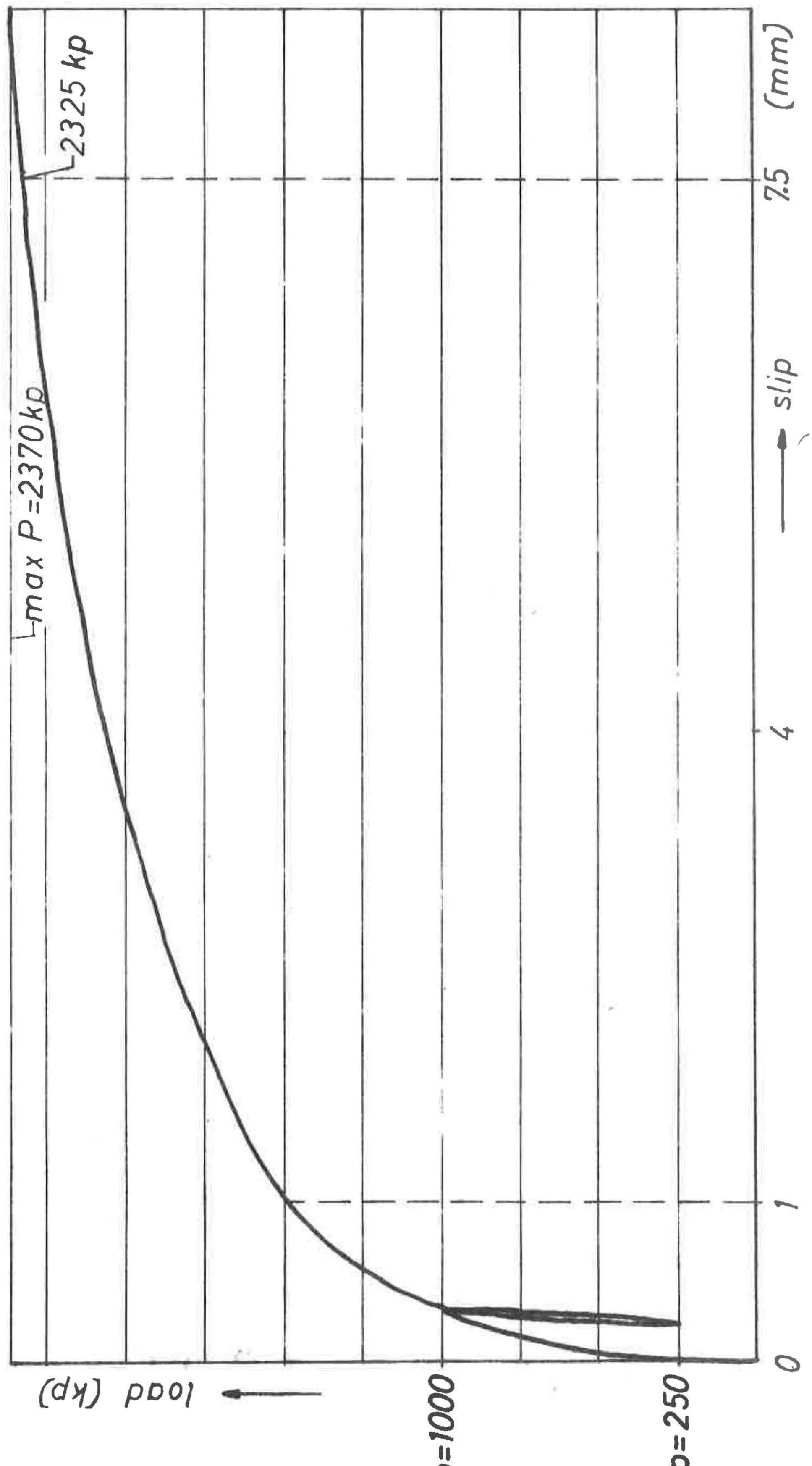


Load-slip diagram for B1

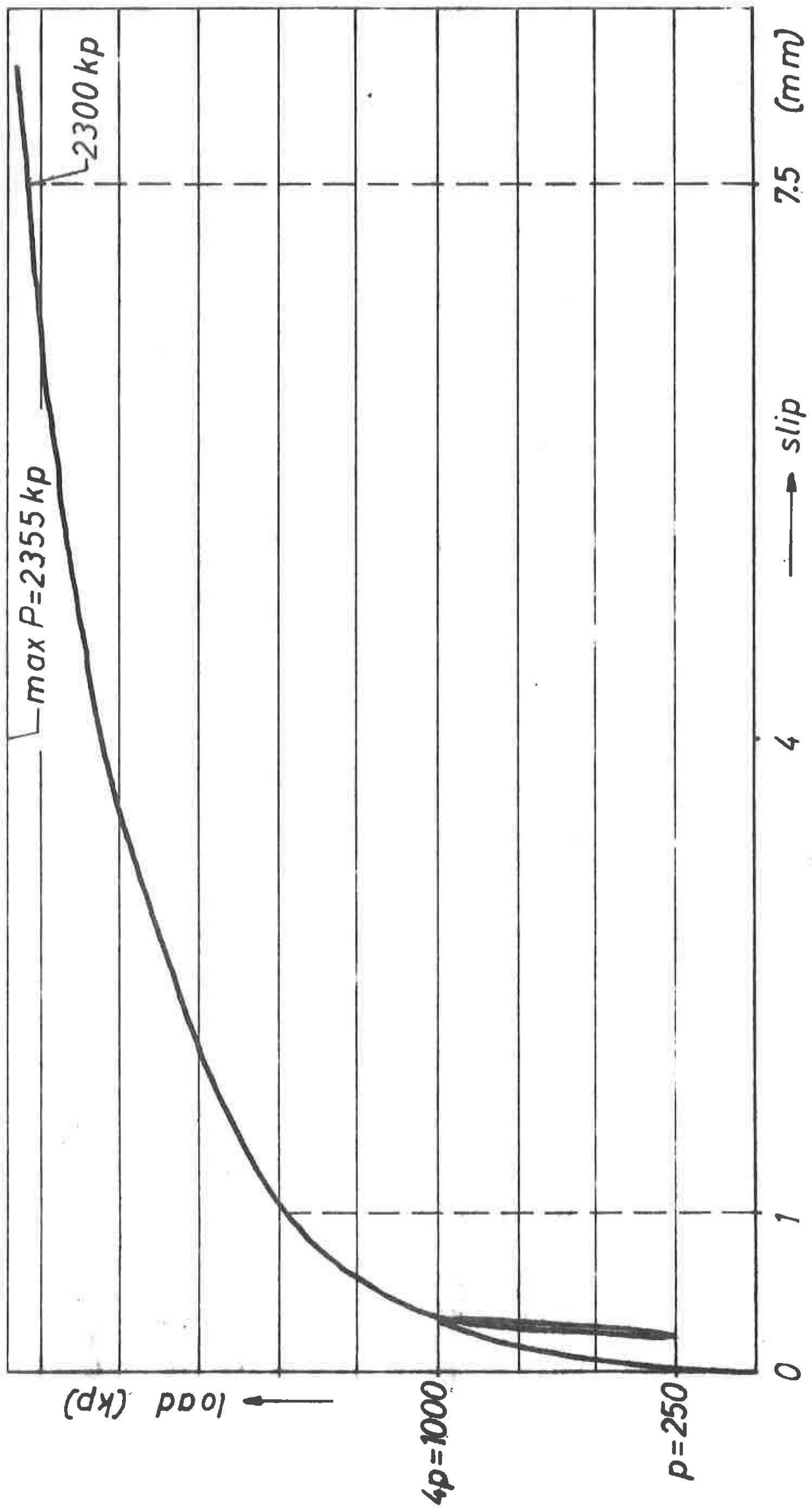
Appendix 6



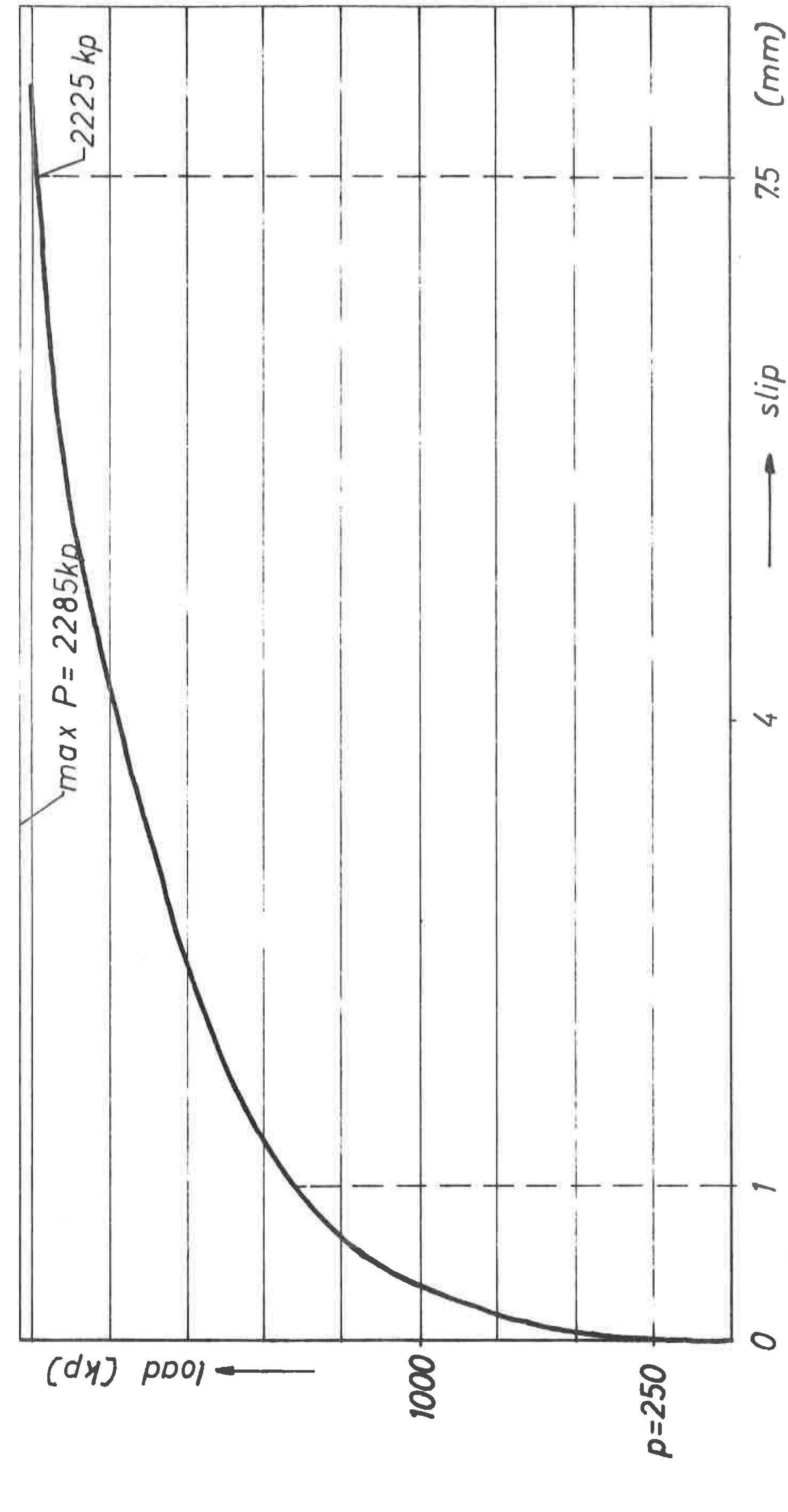
Load - slip diagram for B2



Load - slip diagram for B3

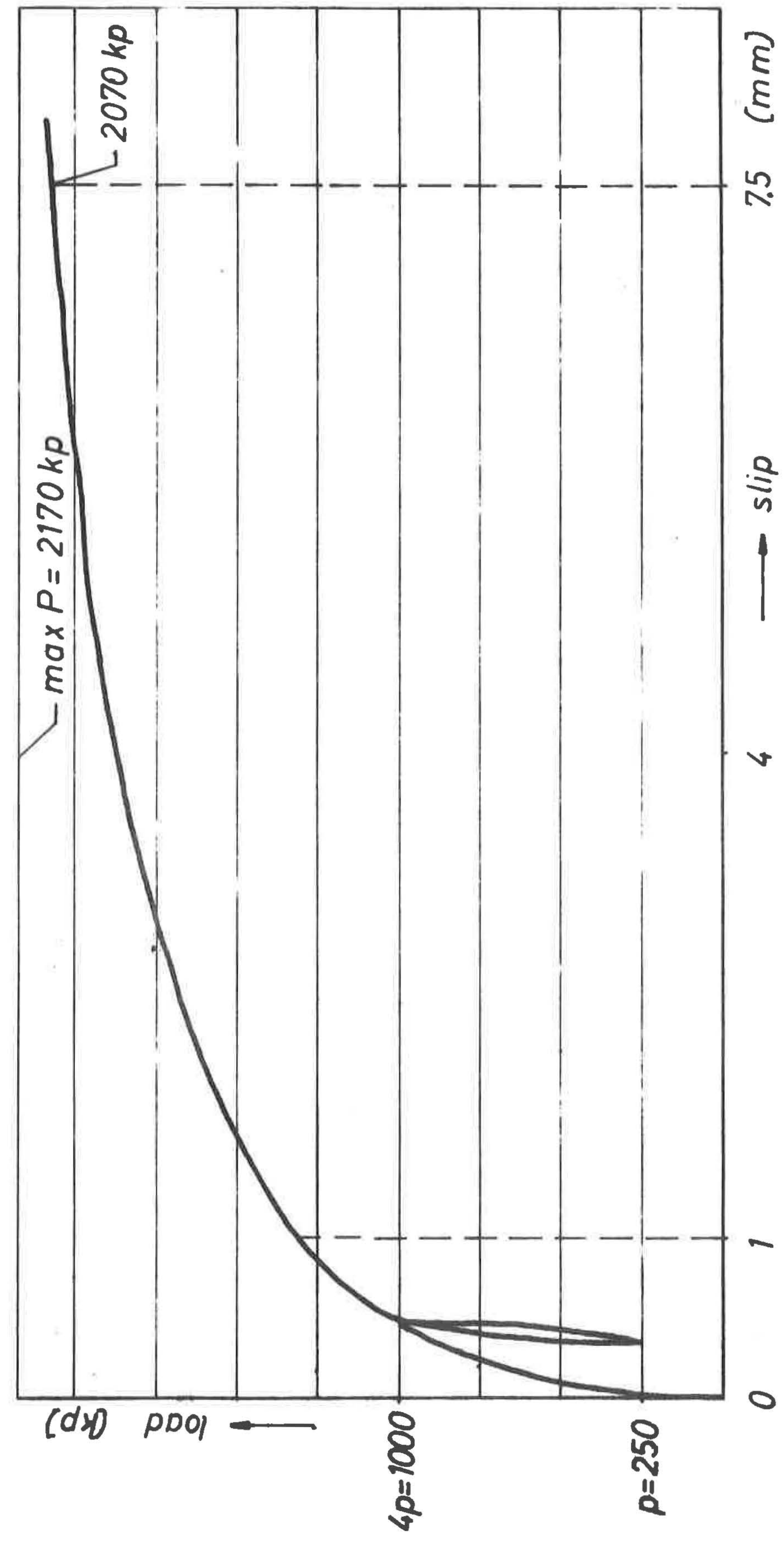


Load - slip diagram for B4



Load-slip diagram for C1

Appendix 10



Load - slip diagram for C 2

Loading Procedures Used for the Tests

A : According to DIN E 4110, sheet 8

Loading each step with a rate of deformation of

1 mm/min,

2 minutes waiting time at each loading step (load increment)

$p \approx 0{,}1 \max P$ ($\max P$ = ultimate load),

first loading until 4p and removal of the load until p, eleven times renewed continuous increase and decrease of the load between p and 4p with a rate of deformation of 4mm/min,

finally load increase step by step until ultimate load with a rate of deformation of 1 mm/min.

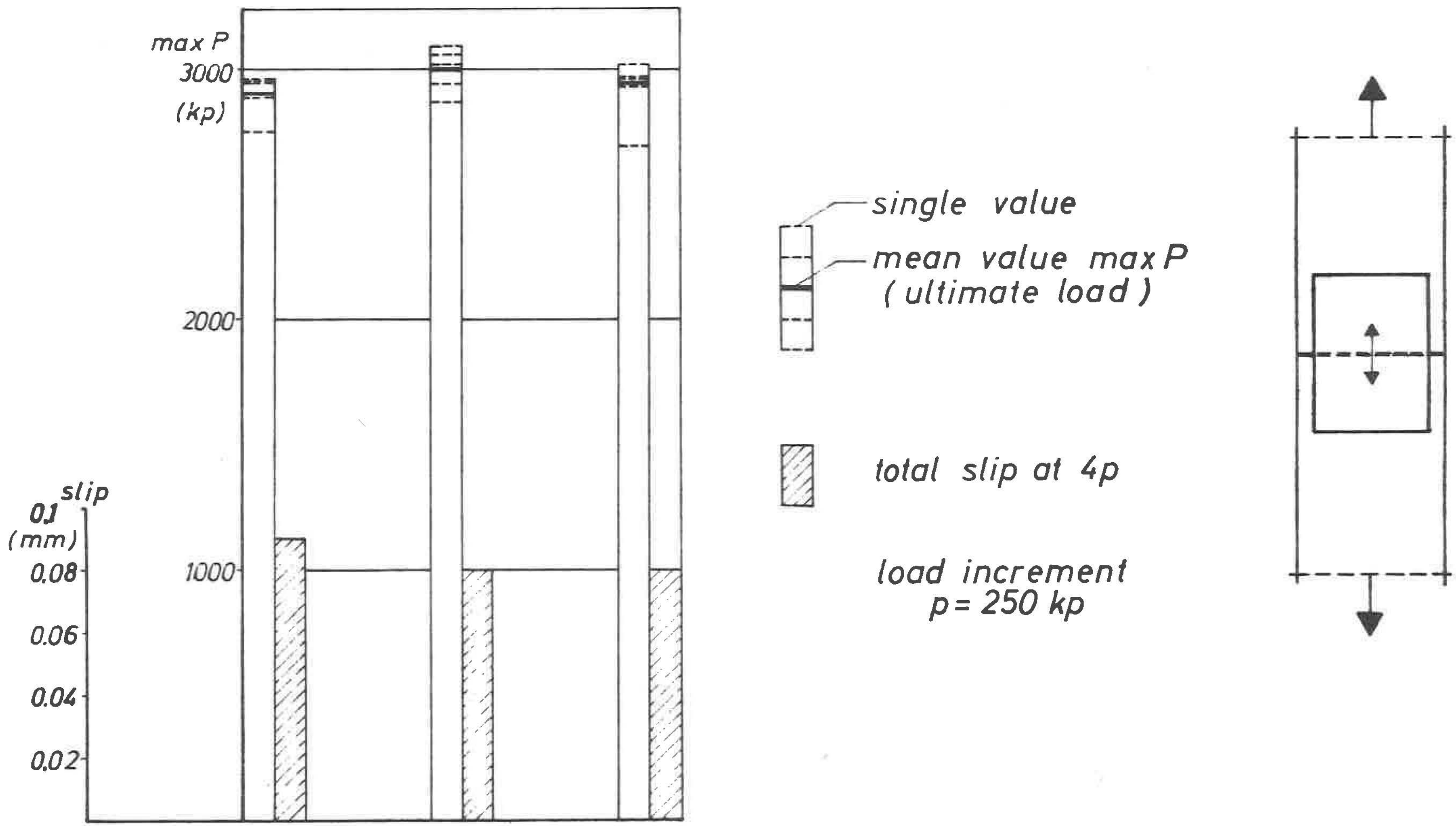
B : Loading with a rate of deformation of 1 mm/min continously until 7p with one removal of load from 4p to p.

After the load has reached 7p, the rate of deformation is 4 mm/min until ultimate load.

C : Loading with a rate of loading continously p/30 sec until 7p with one removal of load from 4p to p.

After the load has reached 7p, the rate of deformation just arrived is to be keept constand until ultimate load.

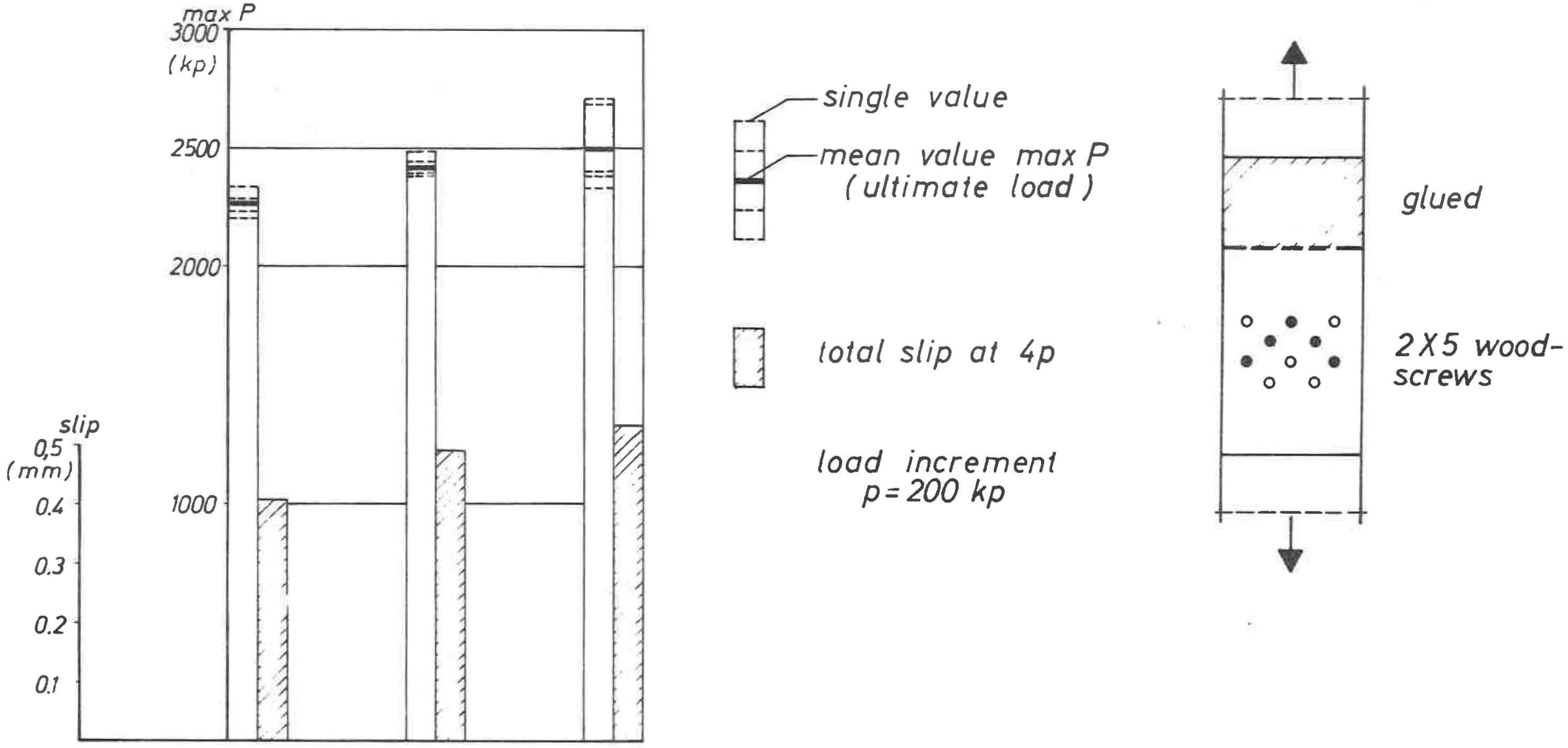
Test with nail-plates "System Truswal" 76.2 X 101.6 mm



loading procedure : A B C

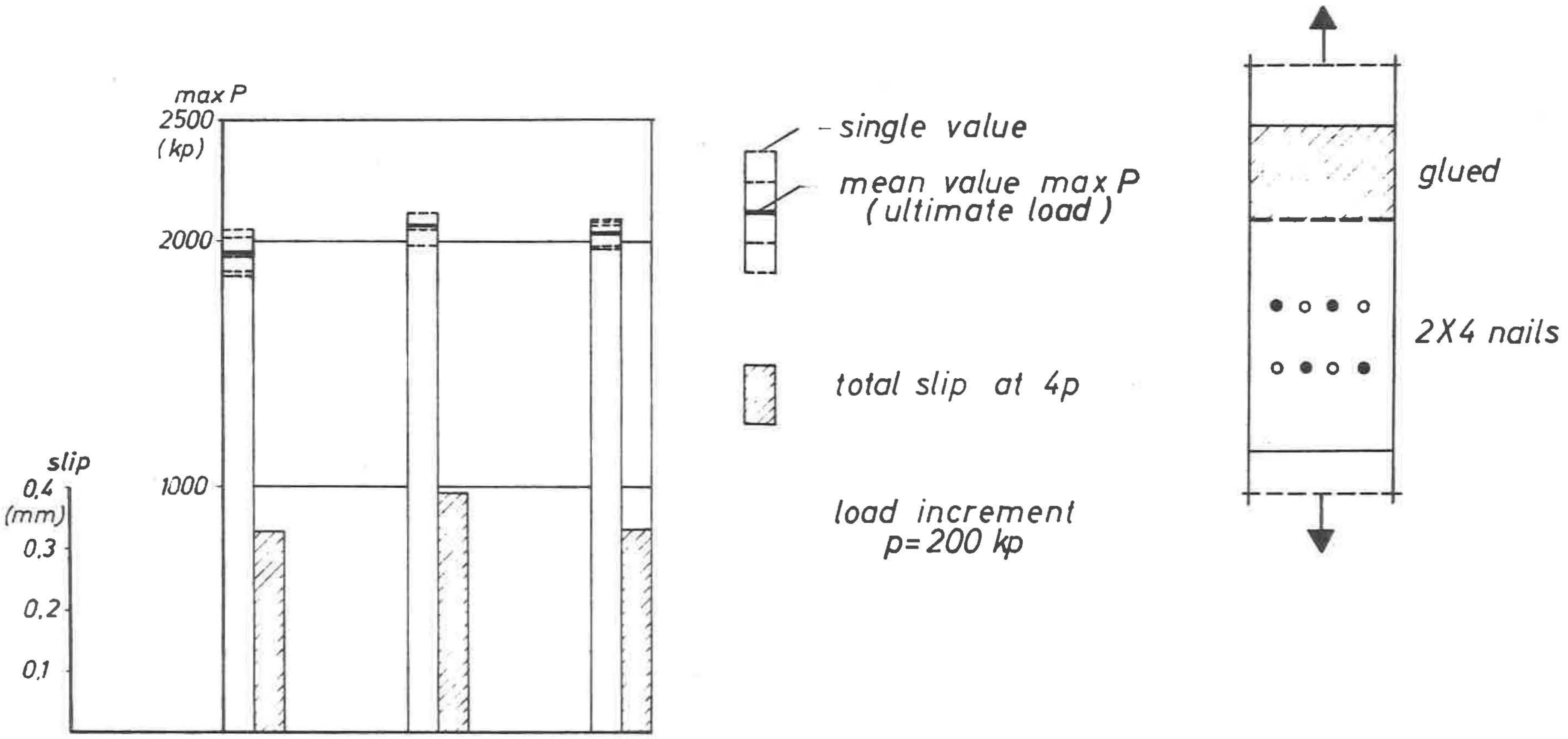
total duration of test : 32' 5'30" 10'20"

Test with wood-screws 5 X 60 DIN 96



loading procedure :	A	B	C
total duration of test :	42'10"	9'35"	9'40"

Test with plain shank nails 34 X 90 DIN 1151



loading procedure : A B C

total duration of test : 35'50" 7'30" 10'30"

CIB-W18/5-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

RECOMMENDATIONS FOR TESTING METHODS FOR JOINTS WITH MECHANICAL FASTENERS AND CONNECTORS IN LOAD-BEARING TIMBER STRUCTURES - 5TH DRAFT

by

RILEM 3TT COMMITTEE

KARLSRUHE - OCTOBER 1975

Kuipers, Delft

RILEM - 3TT
5th draft; August 1975

RILEM recommendations for testing methods for joints with mechanical fasteners and connectors in load-bearing timber structures.

Introduction

Developments in the field of load bearing timber structures give way to the problem that very often joint connectors should be tested to gather new or more information about their deformations and load-bearing capacity.

The present recommendations give some general principles which should be followed in order to reach a better comparability of results from investigations carried out in different laboratories.

1. Scope

A method is developed:

1. to investigate the mechanical properties of timber joints with mechanical fasteners and connectors.
* 2.[1] to permit to calculate from the test results values of the characteristic strength and/or of the allowable loads;
3. to determine values of the deformation in the joint which enable designers to introduce these in their calculation.

* 2. Fields of application

This code of testing practice is applicable to joints destined for application in statically loaded timber structures with mechanical fasteners and connectors.

* 3 Conditioning of test specimens

Attention should be paid both to theconditioning of the timber before the manufacturing of the joint as well as to the conditioning of the joints as a whole before testing. The preconditioning should be conducted in such a way that the moisture content ψ of the wood, the effects

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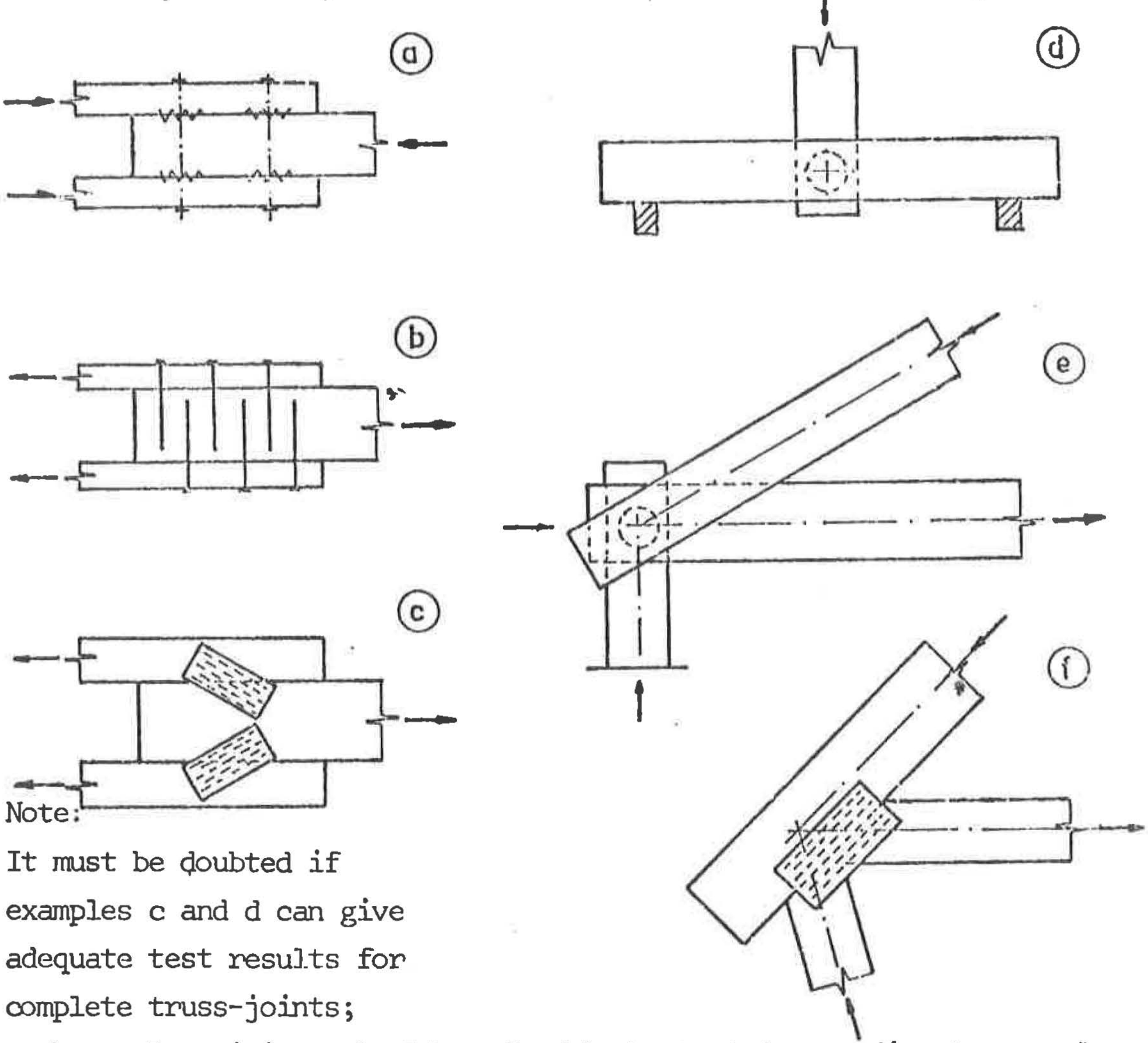
[1] A comment is given about paragraphs marked with a * on page 6 etc.

of shrinkage, etc. can influence in a realistic manner the strength properties of the wood, the occurrence of slits, etc., so as to guarantee a good comparability between the performance of the test joints and the joints in a structure.

4. Form and dimensions of test specimens

1. The joints to be tested in the investigation must be of such realistic form and dimensions that the necessary information about strength and deformation in actual service can be achieved.
2. If necessary the angle of load to grain and/or the position of the connector with respect to directions of load and grain must be varied.
3. In most cases not only simple joints in tension and in compression must be tested, but also joints where some of the members are loaded with an angle to the grain. Attention should be paid to the possibility to transform test values into data for practice



Note:
It must be doubted if examples c and d can give adequate test results for complete truss-joints;
such complete joints should preferably be tested according to examples e or f.

Fig. 1. Examples of test specimens.

4.4 The number of connectors in a joint should be chosen in accordance with 4.1. and correspond to the character of the joint.

5. Loading procedures

5.1. Short duration test or standard test

.1. An expected value of the ultimateload $\hat{F}$ of the joint to be tested has to be determined on the basis of former experience, calculations, preparatory test or elsewise.

.2. The loading procedure has been given in fig. 2.

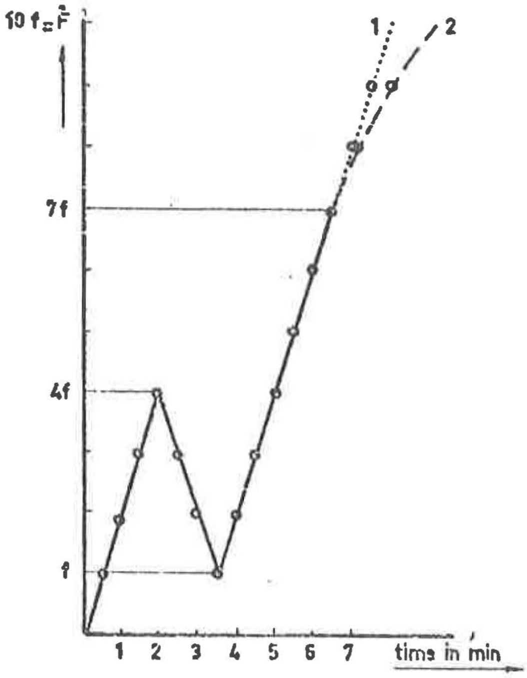


fig. 2 Loading procedure

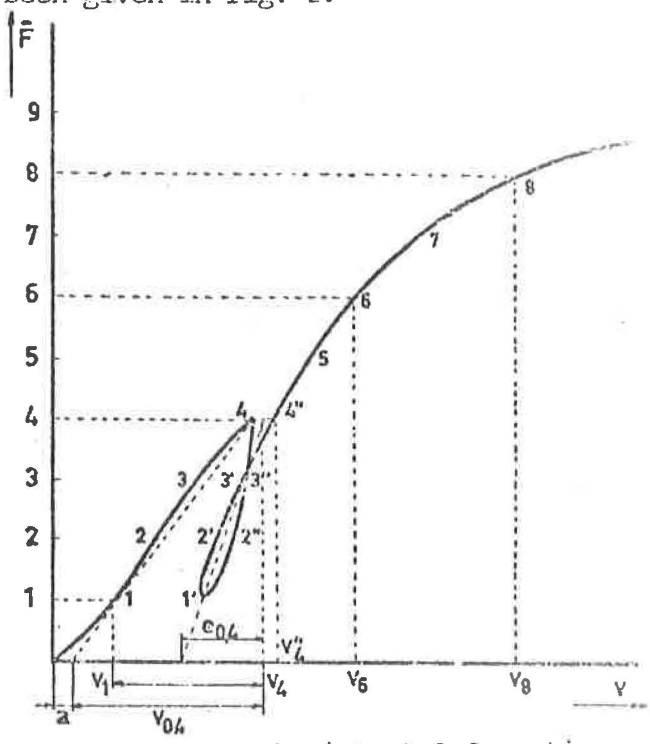


fig. 3 Idealised load-deformation curve and measurements.

The test load is raised with a constant velocity up to 4f, then diminished to f and raised again to 7f. Dependent on the possibilities of the testing machine and of the goals one has in mind the test can then be continued with a constant rate of loading or a constant rate of deformation.

In the first case one must be aware of the fact that high rates of deformations may be reached, as well as higher values of ultimate load $\hat{F}$ than in case two.

.3. At each load-increment or - decrease the deformations should be measured in such a way that the continuity of the loading procedure is not be disturbed to an essential degree.

5.1.4. The loading procedure up to 7f should be carried out with a rate of loading of f (= 0,1 $\hat{F}$) pro 30 sec., so each load increment f will be passed in 30 seconds.

5.1.5. From the measurements the following data can be calculated:

virgin displacement $v_{0,4} = 4/3\ (v_4 - v_1)$

joint slip $a = v_4 - v_{0,4}$

elastic displacement $e_{0,4} = 4/3\ (\frac{v_4 + v_4''}{2} - v_1')$.

joint stifness $k_{0,4} = \frac{v_{0,4}}{4f}$

displacement at overload $v_{0,6} = v_{0,4} + v_6 - v_4$

$v_{0,8} = v_{0,4} + v_8 - v_4$

.6. If during the execution of the investigation the average ultimate load $\hat{F}$ of two or more executed tests turn out to differ more than 20% of the expected $\hat{F}$-value adjustments of the loading procedure for the following tests should be made.

The already obtained values of $\hat{F}$ may be maintained in the final results; the values of v_4 etc. must be recalculated or estimated from the load slip diagram.

.7. As an alternative for more simple cases the load may also be raised continuously from 0 to $\hat{F}$ as in fig. 4.

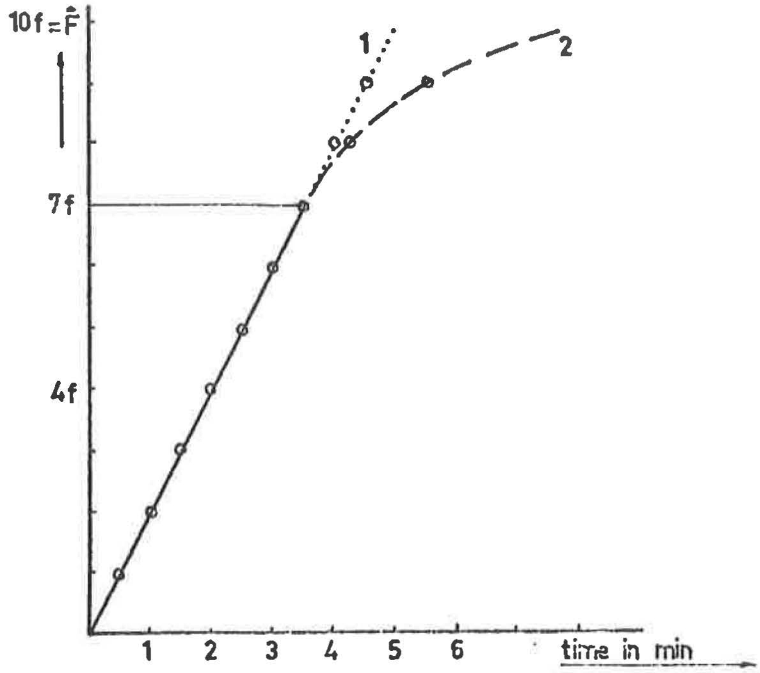


fig. 4

5.1.8 The ultimate load is taken to be the maximum load reached within a displacement in the joint of 15 mm.

5.2 Long duration tests

.1. If information is wanted about the trust-worthiness of joints on the long run long-duration-tests may be carried out.

* .2. Suggestion is made to use two load levels:
  - . at a continuous load of 0,80 $\hat{F}$; joints are expected to fail within a period of 3 month.
  - . at a continuous load of 0,40 $\hat{F}$; a creep factor $\frac{v_{creep}}{v_4}$ of 0, may be expected in 3 month; a distinct flattening of the creep curve will normally be observed.

5.3. Dynamic tests

For non-statically loaded structures results of dynamic tests may be required. Definite guide lines are not yet available.

6. Test reports

Reports on tests must give all reliable data about the tests carried out and the results. They shall therefore contain data about:

- . species and quality of the wood (relevant strength properties, strength grade following standard........... and in any case relative density)
- . material, quality and strength properties of the connectors; eventual measures against corrosion.
- . exact data about the dimensions of the joints, number of connectors therein loading method in the test machine etc.
- . conditioning of timber and test specimens before and after manufacturing; moisture content at the time of testing, slits between members, etc.
- . loading procedure followed
- . all individual test results; mean values and standard deviations
- . mode of failure.

* 7. Evaluation of test results

1. The extent of the investigation must be such that a statistical treatment of the available data can take place. The number of tests depend upon the goals of the investigation.
2. Recommendations about the deduction of admissable loads and other data which may be used in calculations will be given in a CIB - W18 - brochure.

COMMENTARY

1.2. Recommendations about the method of derivation of design values for strength and stiffness will be published by CIB-Commission W18 "timber structures".

2. In case of time-dependent live loads where variance of intensity occur with frequencies higher than ½ to 1/3 of the lowest frequency of the structure itself, dynamic effects must be expected. In many cases, like floors of ballrooms, gymnastic halls, etc. these effects have already been taken into account by the introduction of equivalent live loads in the loading standards and/or by stiffness requirements.

3. The climatic conditions in which a joint is supposed to function influence its strength and its deformations. Four basic conditions can be distinguished:
   - normally heated, and sufficiently ventilated buildings;
   - not heated, closed buildings;
   - not heated, open buildings but with covered structure;
   - unprotected open air exposure.

   Although the basic conditions vary considerably between geografic positions it may be possible to circumscribe the average climatic data for certain regions and to derive therefrom a range from which the moisture content of the timber will not differ for longer than 2 weeks in a period of 5 years, disregarding surface conditions.

For great parts of Western Europe such figures are given in table 1.

Table 1. Average moisture contents to be expected in European softwoods, used in Western Europe

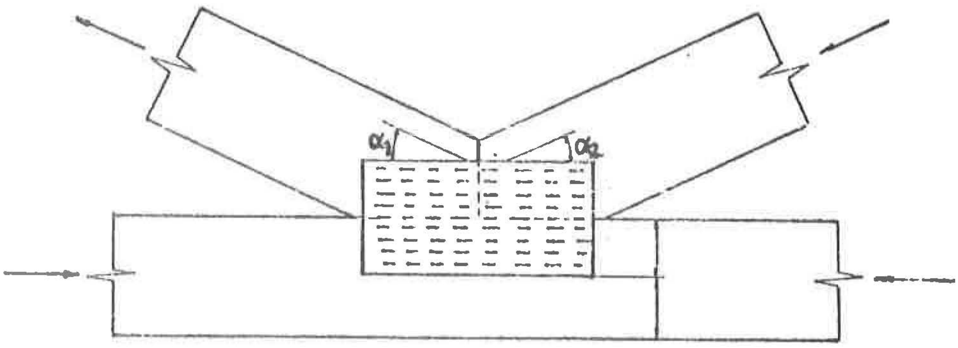
Climatic conditions	moisture content %
heated and ventilated buidings	10 ± 3
not heated, closed buildings	13 ± 4
not heated, covered buildings with open walls	17 ± 4
open air	22 ± 8

5.2.2 Results of long-duration-tests on normally used types of connectors as well as on clear wood show that tests on a level of 0,8 $\hat{F}$ will last not longer than the period of 3 month. In the CIB-W18-recommendations it is said that: "Collapse of the total number (of test specimens with this load) shall not occur within a period of 100 hours".

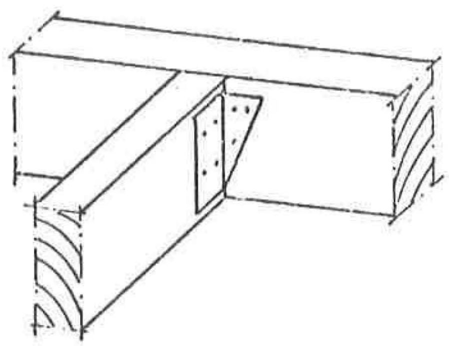
7.1 Dependent on the wanted information different types of investigation can be distinguished:

a. in a systematic investigation information is wanted in a very general way, including dimensions of the connector, the timber, angle of load to grain etc.

b. in a limited investigation information is wanted about the behaviour of a certain type of connector in different positions, e.g. with respect to angle of load to grain but with pre-fixed minimum-values of timber dimensions, edge- and end-distances etc.
For instance a joint with punched metal plates



c. In a special investigation information is wanted about the behaviour of a certain joint with fixed dimensions and in known circumstances.
For instance an "grip" - connector for the connection of the secondary beam to a primary beam of certain dimensions.



CIB-W18/5-7-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

CIB-RECOMMENDATIONS FOR THE EVALUATION OF RESULTS OF TESTS ON JOINTS WITH MECHANICAL FASTENERS AND CONNECTORS USED IN LOAD-BEARING TIMBER STRUCTURES

by

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KARLSRUHE - OCTOBER 1975

CIB-W18 Kuipers; Delft.

1st draft; May 1975

CIB-recommendations for the evaluation of results of tests on joints with mechanical fasteners and connectors load-bearing timber structures.

1. Introduction

Developments in the field of load-bearing timber structures give way to the problem that very often joint connectors should be tested to gather new or more information about their deformations and load-bearing capacity.

The present recommendations give some general principles which could be followed if results of such tests have to be evaluated.

1. Scope

A system is developed to evaluate results of "standard" (short duration) tests as well as results of long duration tests om timber joints.
Ultimate loads of standard-tests lead to the determination of characteristic joint strengths and/or to allowable working loads.
Requirements have been laid down with respect to the behaviour of joints under long duration loads (up to 3 months loading).

2. Fields of application

.1. The recommendations are applicable to results of tests, carried out according to "RILEM-recommendation for testing methods for joints with mechanical fasteners and connectors in load-bearing timber structures".

×.2. The RILEM-recommendations above are applicable to joints in statically loaded timber structures.

.3. Evaluation of joints must take place a) with respect to short-duration tests (cf chapter 3)

b) with respect to long-duration tests (cf chapter 4).

3. Interpretation of results of standard tests

.1. All test results must be adjusted to the mean strength properties of the timber-species under consideration. This may be done by multiplying the ultimate load from the tests $\hat{F}_{test}$ with a factor $\left\{ \frac{\hat{\sigma}_c}{\hat{\sigma}_{ctest}} \right\}^c$, where

$\hat{\sigma}_c$ = mean compressive strength of timber species

$\hat{\sigma}_{ctest}$ = mean compressive strength of timber used in the test-programm.

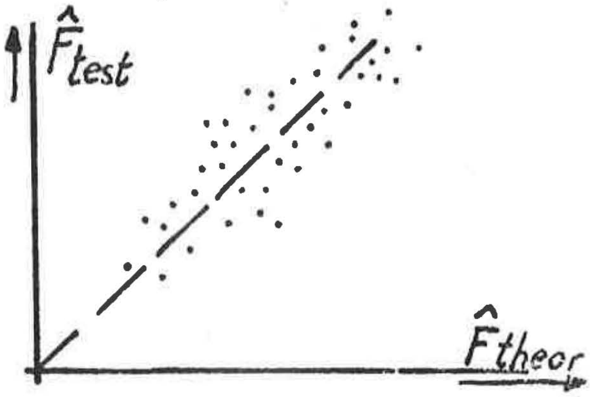
c = coëfficiënt, dependent on woodspecies and on type of connector, as a guide following values may be used:

c = o for $\hat{\sigma}_{ctest} < \hat{\sigma}_c$

c = 1 for $\hat{\sigma}_{ctest} \geqq \hat{\sigma}_c$

*.2. In the case of systematic and limited investigations, the influence of different variables must be studied and a theory or method to forecast the ultimate load $\hat{F}$ must set up.

.3. From the distribution of the ratio $\phi = \frac{\hat{F}\ test}{\hat{F}\ theory}$ a mean value ϕ_m as well as a coëfficiënt of variation v_ϕ can be calculated. In both values both the variation in strength as a result of materials properties as well as simplications of the theory are incorporated.



*.4. With the aid of the mean value and the variation mentioned in .3. a 5%- fractile value ϕ char can be calculated, and this may be used to calculate characteristic strength values for the joints following $F_{char} = \phi_{char} . \hat{F}_{theor}$.

For the determination of a coëfficient of safety w use may be made of one of the following formules.

a) $W_a = \dfrac{1-\sqrt{1-0,9375\ \left[1-6,25(v/100)^2\right]}}{0,9375}$, or

b) $W_b = \dfrac{1-2,33v/100}{1,25}$, or

c) $W_c = \dfrac{1-1,96v/100}{1,33}$.

Allowable basic loads on joints may be found from

$\bar{F}_{basis} = wt\hat{F}_{theor}$, where

w = a safety factor and

t = a factor for duration of load; unless more than normal information is available a value of t = 9/16 (= 0,56) may by maintained; if shearing or splitting of the wood appear to be the governing strength properties for the joint a somewhat lower value of t - e.g. t = 0,5 - may be chosen.

4. Long-duration tests

.1. It is strongly recommended that the evaluation of the trustworthyness of timber joints is not only based on standard strength tests, but also on the results of long-duration tests.

.2. Results of the long-duration-tests as suggested in the RILEM-recommendations may be judged in the scope of the following requirements:

.1. From the total number of 0,80 $\hat{F}$-long-duration-tests not more than 50% shall be collapsed within a period of 100 hours. Collapse of the total number shall not accur within a period of 100 hours.

.2. The creep-deformation of 0,40 $\hat{F}$-longduration-tests after a period of 1000 hours shall be not more than 70% of the initial deformation. Within this period collapse of such test specimens shall not occur.

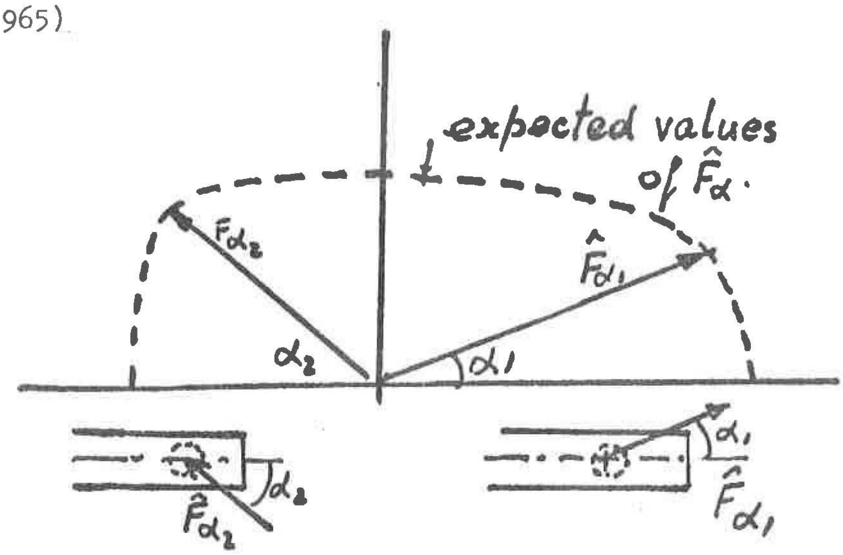
.3. If the long-duration tests primarily do not fulfill the reuirements of 4.2.1. and 4.2.2. the allowable load must be reduced to such limits that the requirements can be fullfill ed at load-levels of 80% resp. 40% of 3 x the reduced allowable load.

Commentary

.3.1. Values of $\hat{\sigma}_c$ and of $\hat{\sigma}_{ctest}$ must be based on the compressive strength prisma 20x20x60mm, loaded paralell to the grain.
A sufficient number of tests on the wooden members of the joints under consideration must be taken to base a mean value of $\hat{\sigma}_{ctest}$ upon.
For the following species-independent of the grade - a value of $\hat{\sigma}_c = 35$ N/mm^2 - may be uaed:

. abies alba (syn. A pecinata), abied sp. div. Tanne, whitewood
. Pinus silvestris, pinus sp. div. European Redwood Kiefer.
. Larix spe. div; larch, Lärche
. Picea abies (syn. P excelsa); european spruche, Fichte
. Picea sitchensis; sitka spruce
. Tanga heterophylla; Western henlock

.3.2. Such theories may have the character of the study of A. Meyer about the load-bearing capacity of nails (A. Meyer, Die Tragfähigkeit von genägelten Verbindungen; diss. Karlsruhe) or of the study about the behaviour of joints (c.f. Kuipers and Vermeyden: "research on timber joints in the Netherlands" and " The ratio between the strength and the alloable load on timber joints"; Papers for the TRADA/CIB-International symposium on joints in timber structures, 1965)



.3.4. If use is made of a normal distribution of ϕ- the validaty of which may be controlled - a multiplication factor $\phi_k = \phi_m - 1,64\ v_\phi$ can be used to find the characteristic values of the theoretical strength

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WORKING COMMISSION W18 - TIMBER STRUCTURES

STRENGTH OF A WOOD COLUMN IN COMBINED COMPRESSION AND BENDING WITH RESPECT TO CREEP

by

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STOCKHOLM, SWEDEN

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STRENGTH OF A WOOD COLUMN IN COMBINED COMPRESSION AND BENDING WITH RESPECT TO CREEP

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Introduction

The purpose of this paper is principally to show the result of calculations of strength and deformation of initially curved homogeneous wood columns when creep is considered. Though the subject is not completely dealt with, the presented results will give an idea of the influence of creep on long-term strength. The calculations are based on a method presented by Åke Samuelsson /1/.

Theory

The buckling theory is elementary. For the numerical solution of the equations finite differences are used. The problem is treated as if the column was divided in elements, lengthwise and crosswise. The following basic assumptions have been made:

1. The column section is symmetrical with respect to the plane in which it buckles.
2. The cross sectional area and the moment of inertia are constant along the column.
3. The column is divided into a finite number of imaginative laminae, the bending stiffness of the laminae themselves being neglected. Shear deformation within and between the laminae have been neglected.

It is possible to introduce layers between the laminae in the computer program. These intermediate layers are supposed not to take normal forces. By introducing such layers one can simulate the real bending stiffness of the total cross-section and still use a limited number of laminae. The laminae are of equal thickness.

4. The cross-sections are assumed to remain plane at the deformation of the column.

5. The deflections are considered as small in relation to the length of the column.
6. The deformation of the wood is supposed to have one part appearing instantaneously with a change in stress and one part which is depending on the time and the level of stress. The material is assumed to behave linearly.

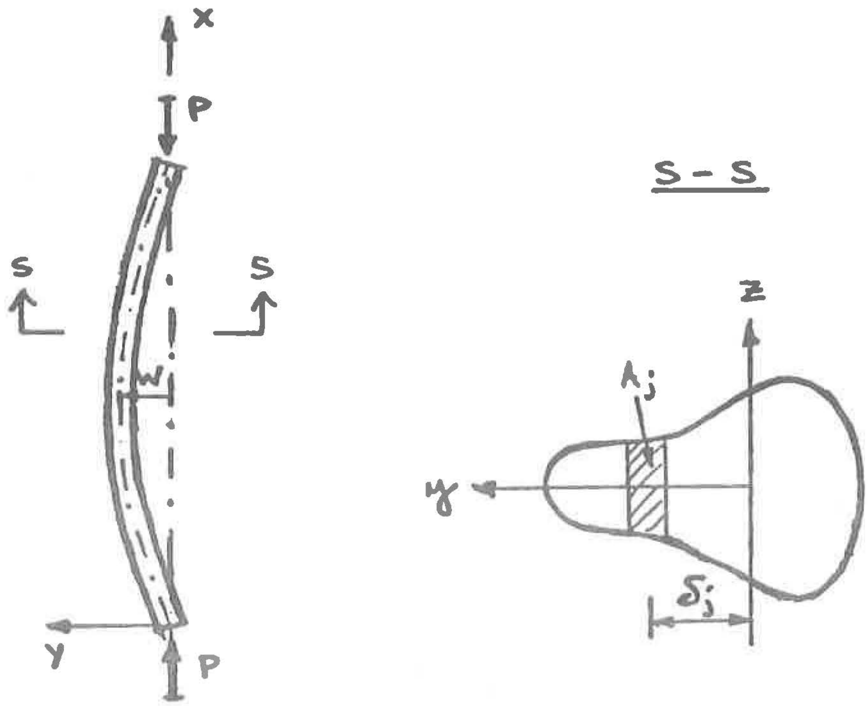


Figure 1 Geometry and loading of the column

The geometry and loading of the column is seen from Figure 1. The fundamental equations for determining unknown quantities are given briefly in the following. For the more detailed derivation is refered to /1/. For each cross-section there is an equation of equilibrium with reference to external and internal axial forces

$$P = -\Sigma A_j \sigma_j \quad (1)$$

and one equation with reference to bending moment

$$P\, w = \Sigma A_j\, \sigma_j\, \delta_j \tag{2}$$

Further, the condition that the cross-section remains plane gives n_x -1 equations

$$\varepsilon_j - \varepsilon_i = (\delta_j - \delta_i)\, w'' \tag{3}$$

At last one can formulate the relation between strain and stress for each lamina in the cross-section

$$\dot{\varepsilon} = \frac{\dot{\sigma}}{E} + \dot{\varepsilon}_{cr} \tag{4}$$

For each cross-section of the column we thus get $2n_x$+1 equations and the total number of equations for the column will be $n_y(2n_x+1)$.

The number of unknown quantities are $n_x n_y$ strains, $n_x n_y$ stresses and n_x+2 deflections, i.e. the total being $n_y(2n_x+1)+2$. Thus we need two additional equations. They are given by the boundary conditions for the column:

$w(=\dot{w}) = 0$ for $x = 0$ and $x = L$

The relation between stress, strain and time can be expressed by the equation

$$\dot{\varepsilon} = \dot{\sigma}/E + k_2 \sigma^{n_2}\, t^m + k_3 \sigma^{n_3} \tag{5}$$

In (5) the first term gives the rate of strain directly referable to the rate of stressing. The second term refers to primary creep and the third to secondary creep (Samuelsson /1/ has not included the second term). The parameters in (5) can be determined by creep testing at different stress levels. The computer program is based on a more general creep function (at constant stress)

$$\varepsilon = \frac{\sigma}{E}\,(1 + \phi) \tag{6}$$

The creep function ϕ is assumed to consist of two independent factors: a stress function and a time function

$$\phi(\sigma,t) = \psi(\sigma)f(t) \tag{7}$$

By dividing the time in relatively short intervals the stress can be considered constant within each interval and the equation (6) can be applied. The increase of the stress is illustrated by Figure 2. The law of superposition is assumed to be applicable, i.e. the stress at a time t_n can be divided into components:

$$\sigma_n = \sigma_o + \Delta\sigma_o + \Delta\sigma_1 + \ldots\ldots\ldots + \Delta\sigma_{n-1} \tag{8}$$

The duration of the respective stress components is

$$t_n - t_o \text{ , } t_n - t_1 \text{ , } t_n - t_2 \text{ , } \ldots, t_n - t_{n-1}$$

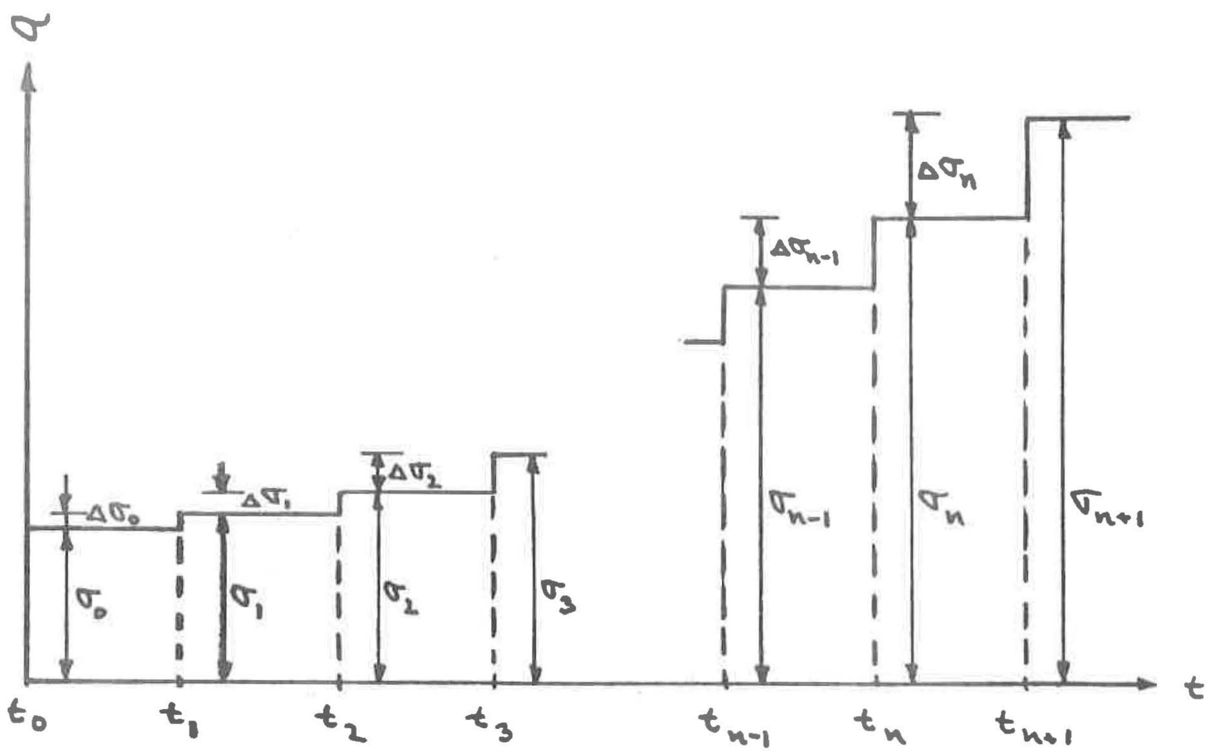


Figure 2 Assumed variation of stress with respect to time

The rate of creep in successive time intervals is given in equation (9):

For $t_o < t \leq t_1$

$$\dot{\varepsilon}(t) = \frac{\dot{\sigma}}{E} + \frac{\sigma_o}{E} \psi(\sigma_o) \dot{f}(t-t_o) \tag{9a}$$

For $t_1 < t \leq t_2$

$$\dot{\varepsilon}(t) = \frac{\dot{\sigma}}{E} + \frac{\sigma_o}{E} \psi(\sigma_1) \dot{f}(t-t_o) + \frac{\Delta\sigma_o}{E} \psi(\sigma_1) \dot{f}(t-t_1) \tag{9b}$$

For $t_n < t \leq t_{n+1}$

$$\dot{\varepsilon}(t) = \frac{\dot{\sigma}}{E} + \frac{\sigma_o}{E} \psi(\sigma_n) \dot{f}(t-t_o) + \frac{\psi(\sigma_n)}{E} \sum_{0}^{n-1} \Delta\sigma_i \dot{f}(t-t_{i+1}) \tag{9c}$$

Numerical examples

In the examples refered to in the following the function $\psi(\sigma)$ in (7) has been given the value 1. The function f(t) chosen is

$$f(t) = 0,0224 \, t^{0,356} \tag{10}$$

Thus the last term in (1) is omitted. The coefficients should be regarded as an example, see appendix 1.

It is refered to five examples (no. 1 - 5). In all cases the cross-section is 45 x 95 mm. Variables are the length of the column (L) and the initial bow (w_o). Numerical values are given in Table 1. At the calculation the cross-section was divided into 15 equally thick laminae and the length of the column into 20 parts (21 cross-sections). As shown by Samuelsson /1/ one could have used a less number, especially fewer laminae.

TABLE 1 Length, initial bow and slenderness used in the calculations

Example	L mm	w_o mm	λ	w_o/L
1	2700	10	98.5	1: 270
2	2700	25	98.5	1: 108
3	2700	1	98.5	1: 2700
4	4050	15	147.8	1: 270
5	1350	5	49.3	1: 270

Stresses

The stress in the lamina furthest out in the middle of the column (maximum stress) is shown as a function of the duration of load and the levels of load in the Figures 3 - 7. The lowest level of load is refered to as 1.0. Actually, this is the presently permissed long-term load, with respect to buckling, for the Swedish structural quality T 200 (Swedish code of practice SBN 67).

In example 1 (Figure 3) the bow (10 mm on a length of 2.7 meter) fairly well corresponds with the maximum bow allowed in the structural quality T 200. At the lowest load the shape of the stress curve is characterized by a very small influence of time in the first part and a steep increase of stress in the last stage, just before failure. In this case the stress at the time t = 1h is very near the stress immediately after loading. The stress after one hour´s duration is slightly higher. It is further seen from the figure that at a load about 3.4 times admissible load at long-term loading the stress almost instantaneously reaches the short-term strength of the timber, 20 N/mm^2.

In example 2 the bow is increased to 25 mm. As seen from Figure 4 the short-term stress thereby will be higher and the time influence on the stress increased, so the curves are more smoothly curved. The ratio of short-term load, giving the stress 20 N/mm^2, to admissible long-term load is decreased to about 2.5. Should the initial bow have been still larger the stress curves at low load levels had approached the creep function f(t).

The stresses in a column with small initial bow are shown by Figure 5. The load 1.0 gives an instantaneous stress close to the long-term stress admitted which increases very slowly during a long time. In this case the short-term load giving a stress of 20 N/mm^2, is comparatively high: about 4.6 times the admitted long-term load.
Figure 6 gives the stresses for the example 4 in which the length and thus the slenderness is increased by 50%. Also the initial bow is increased by 50%. Thus the case is comparable to that in example 1, except for the slenderness. The curves at equal relative load have similar shape.

In the last example, Figure 7, length and bow are half the length and bow as given in the first example, Figure 3. In this short column the ratio axial force to bending moment is comparatively large. Consequently, the bending stresses are relatively small. The curves are almost horizontal for a long period even at higher load levels.

Ultimate load

The ultimate load at short-term loading (1h) can be defined as the load which gives a stress equal to the characteristic strength, here 20 N/mm^2. (This is not absolutely correct as the value 20 refers to a bending strength but the stress is a result of compression and bending). In order to design a curve showing the time influence on ultimate load, that is a "Madison-curve" for columns, one has to define a long-term ultimate stress. One possibility is to assume that the ultimate stress decreases as found in testing at constant stress. Such a decrease of stress is examplified by the line AB in Figure 3. The time to failure at a given load level is then found from the intersection of the line and the respective load curve. If we, for example, assume that the relative level of load is 2.2, then the time to failure will be 1.6×10^4h. The stress at this time is 13.5 N/mm^2 while the corresponding short-term stress was 9.3 N/mm^2. The difference is not too big so it is reasonable in this case to use the reduction of the ultimate stress found in tests at constant stress. If instead we have a load level as represented by the curve 1.0, the ultimate stress at the intersection of the line AB is 9.8 while the instantaneous stress is 3.5. In this case the stress has been less constant. Still it has been at least 6 N/mm^2 during 2/3 of the time so one will not be too conservative if assuming a material strength reduction in accordance with line AB.

The assumption that the intersection of the line AB and the curves directly gives the time to failure is slightly on the safe side. It is possible to modify the time to failure with respect to the increasing stress. However, from the shape of the curves one can estimate the error by assuming the line AB to be reasonably small.

Strength reduction curves

If in Figure 3 the intersection of the curve representing the relative load 2.2 and the line AB is once again regarded, it represents a failure load of 2.2 after 1.6×10^4h duration. An interpolated curve which intersects the line AB at the time 1h represents a relative load 3.37. In other words: The ratio of the ultimate load at 1.6×10^4h duration to the load at 1h duration is 2.2/3.37 = 0.65. This ratio is marked in Figure 8. If the procedure is repeated for the rest of the curves in Figure 3 one finally arrives at the full curve in Figure 8. It shows the time influence on the ultimate load at constant load (P). It should replace the reduction curve for the strength of the wood (dotted line in Figure 8). The deviation from the line is due to deformations of second order (deflection due to axial force). Corresponding curves for the other examples are found in Figures 9 - 12. It is observed that the calculated load reduction curves give less reduction at short-term duration and greater reduction at long-term duration than the line for the strength of the material.

The dash-dotted curve in Figure 8 shows the load reduction which one would get if there was no creep, that is if the curves in Figure 3 remained horizontal. The strength of the material has of some other reason decreased as shown by the line AB. This is a case of pure aging without creep.

The more influence from second order deformations, the less is the influence on ultimate load of a reduction of the material strength. If the strength of the wood decreases from 20 to 10 N/mm^2, the ultimate load decreases from 3.37 to 2.3 (Figure 3), that is by less than 50%. Consequently the dash-dotted curve will always be above the straight line (dashed) representing reduction in material strength. The curve approaches the line when the ultimate long-term load approaches zero.

For timber columns the case of pure aging is merely academic. Reduction of strength of the wood is almost always related to creep. The real reduction curve for the ultimate load on the column will therefore initially lay between the curve for pure aging and the straight line assumed for the strength reduction of the material. If, however, either the excentricity (bow) is great or the column short, the influence of second order deformation will be small. In this case the ultimate load will be near proportional to the material strength, i.e. the three curves will lay close to each other. This can be seen from example 5, Figure 12 and - to some extent - from example 2, Figure 9.

At the assumed conditions, especially the chosen creep function for the material, one is on the safe side in reducing the ultimate load by the same factor as the material strength, provided the duration of load exceeds 10^4h, that is about one year. If the column is loaded for longer time one must increase the reduction as the load reduction curve falls below the reduction line for the material strength. This is the most important conclusion of the calculations.

The influence of the slenderness on the reduction can be estimated from the examples shown in Figure 8 (λ = 98.5) and Figure 11 (λ = 147.8). The excentricity is normal, that is, the bow is close to what is allowed in the timber grade in question. The reduction curves for the loads are almost identical. On the other hand when the slenderness is lower, the reduction curves deviate, see Figure 12.

Strain and deflection

The extreme strain and the deflection in the middle of the column are given as function of time in the Figures 13 and 14. The time to failure at different load levels is marked on the curves and the marks have been connected to a dashed curve indicating the strain at failure. The curves' first drop indicating that the ultimate strain decreases with load duration. This is hardly to be expected. However, the curves are nothing but a result of the assumed creep function and the assumed linear reduction of the strength of the wood. These assumptions can be modified to the effect that the strain at failure will be constant or increase with duration of load. Of course the result has nothing to do with the second order deformations. This is clear if one regards a case of pure bending (large excentricity). Strain at failure in this case is

$$\varepsilon_B = \varepsilon_0 (1 + 1 \cdot f(t)) = \varepsilon_0 (1 + 0.0224\, t^{0.356})$$

The ultimate short-term strain, ε_0, decreases proportional to the logarithm of the time in agreement with the assumption of the reduction of the ultimate stress. Contrary to this the factor which includes the creep function increases with the time. In the beginning the decrease of ε_0 is dominating, see Table 2.

TABLE 2 Ultimate strain at pure bending (pure compression)

t	$1 + f(t)$	ε_0	$\varepsilon_0 (1 + f(t))$
0	1	1	1
10	1.051	0.92	0.967
10^2	1.115	0.835	0.931
10^3	1.262	0.76	0.959
10^4	1.595	0.68	1.085
10^5	2.350	0.60	1.41
10^6	4.064	0.525	2.13

The deflection curves are similar in shape as the curves for the maximum strain. Figure 15 shows the rate of deflection at the middle of the column. Also in this figure the values at failure are marked and connected to give a curve. This curve does not entirely coincide with the curve through the minimum points of the deflection rate curves, i.e. the points where a deflection retardation changes to acceleration.

Notation

A_j	cross sectional area of lamina j
E	modulus of elasticity
f	time dependent function
L	length of the column
n_x	number of sections in the x-direction
n_y	number of laminae
P	axial force
t	time
w	lateral deflection
x, y, z	coordinates (Figure 1)
δ_j	distance between the center of gravity and lamina j
ε	strain
λ	slenderness - ratio
σ	stress
ϕ	creep function
ψ	stress dependent function

Reference

1. SAMUELSSON, Å: Creep deformation and buckling of a column with an arbitrary cross section. *The Aeronautical Research Institute of Sweden, Report No.107 (September 1966)*

Figures 3 to 7 Maximum stress as function of time at different load levels

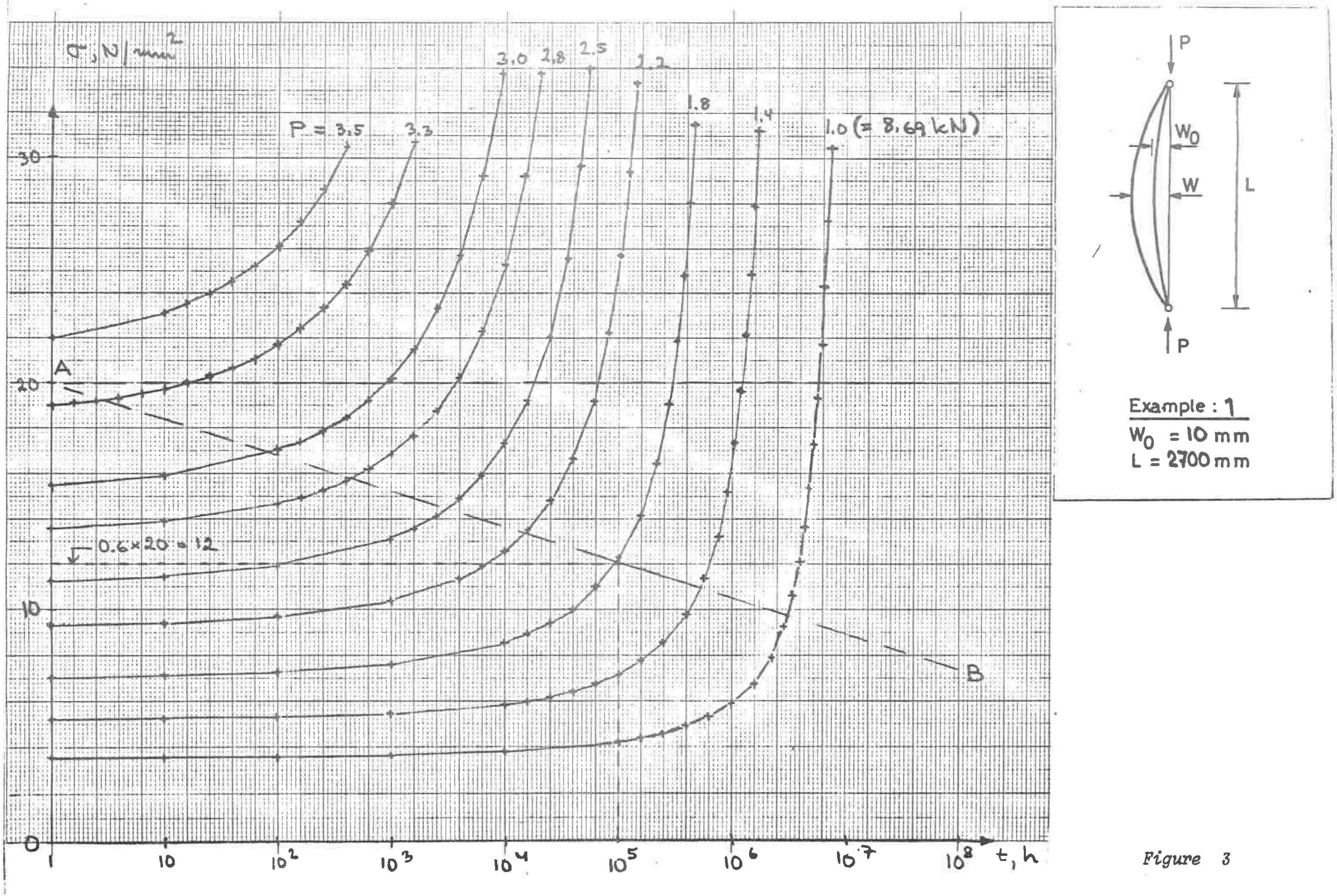
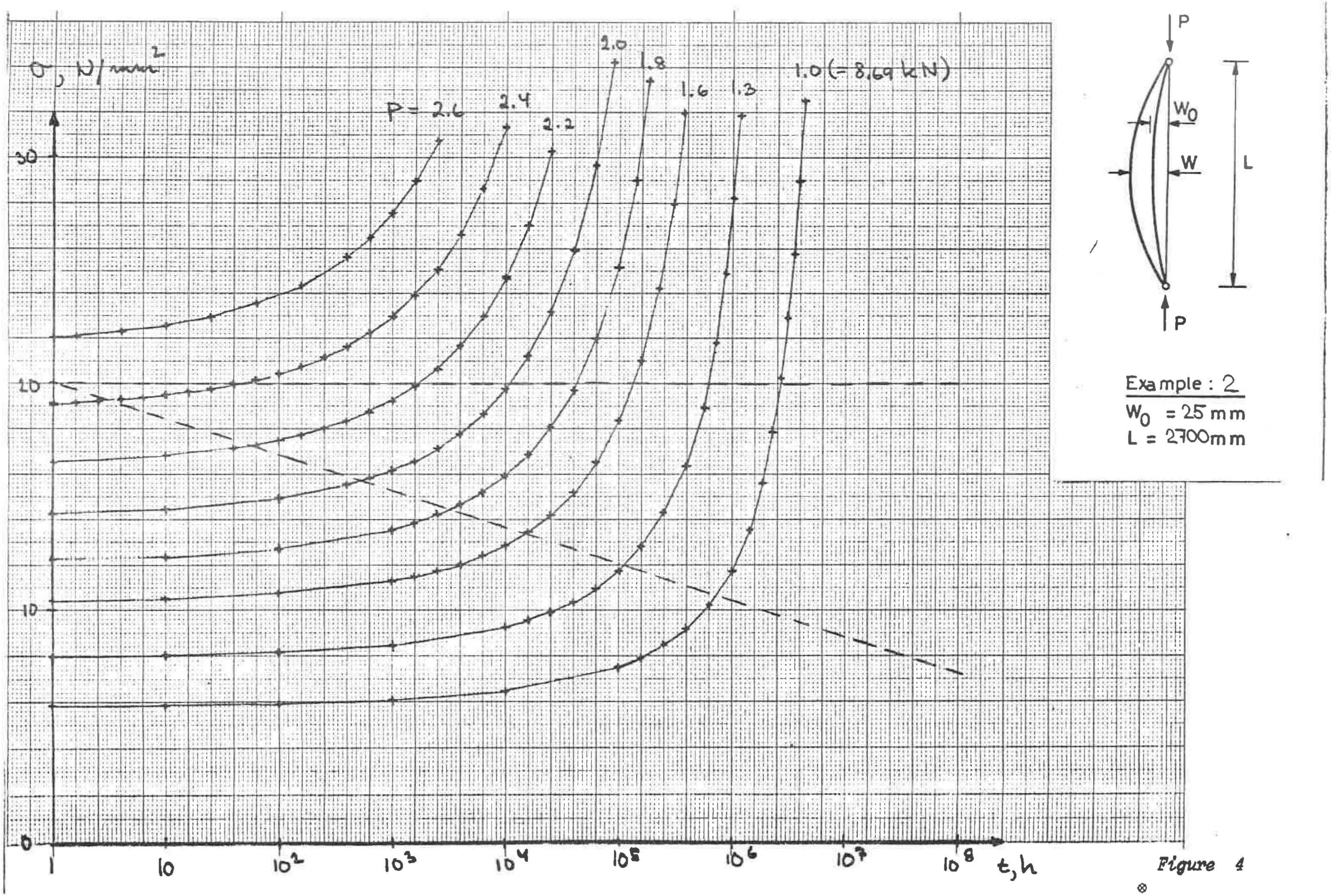

σ, N/mm²
P = 3.5
3.3
3.0
2.8
2.5
2.2
1.8
1.4
1.0 (= 8.69 kN)
A
B
0.6×20 = 12
30
20
10
0
1
10
10²
10³
10⁴
10⁵
10⁶
10⁷
10⁸
t, h
P
W0
W
L
Example : 1
W0 = 10 mm
L = 2700 mm

Figure 3



σ, N/mm²
30
20
10
0
P = 2.6
2.4
2.2
2.0
1.8
1.6
1.3
1.0 (= 8.69 kN)
1
10
10²
10³
10⁴
10⁵
10⁶
10⁷
10⁸
t, h
P
P
W0
W
L
Example : 2
W0 = 25 mm
L = 2700 mm

Figure 4

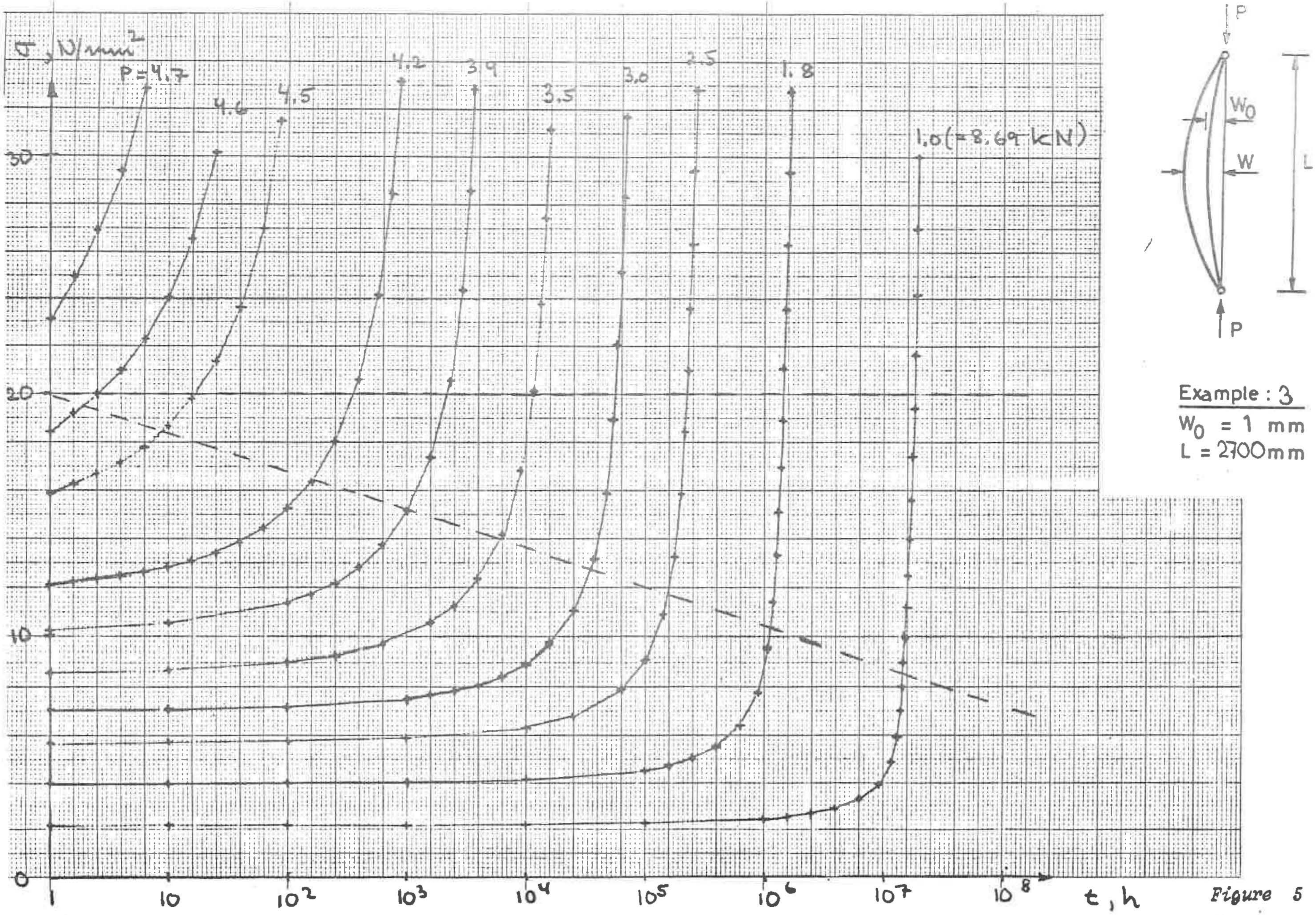

σ, N/mm²
P = 4.7
4.6
4.5
4.2
3.9
3.5
3.0
2.5
1.8
1.0(=8.69 kN)
30
20
10
0
1
10
10^2
10^3
10^4
10^5
10^6
10^7
10^8
t, h
P
W_0
W
L
Example : 3
W_0 = 1 mm
L = 2700mm

Figure 5

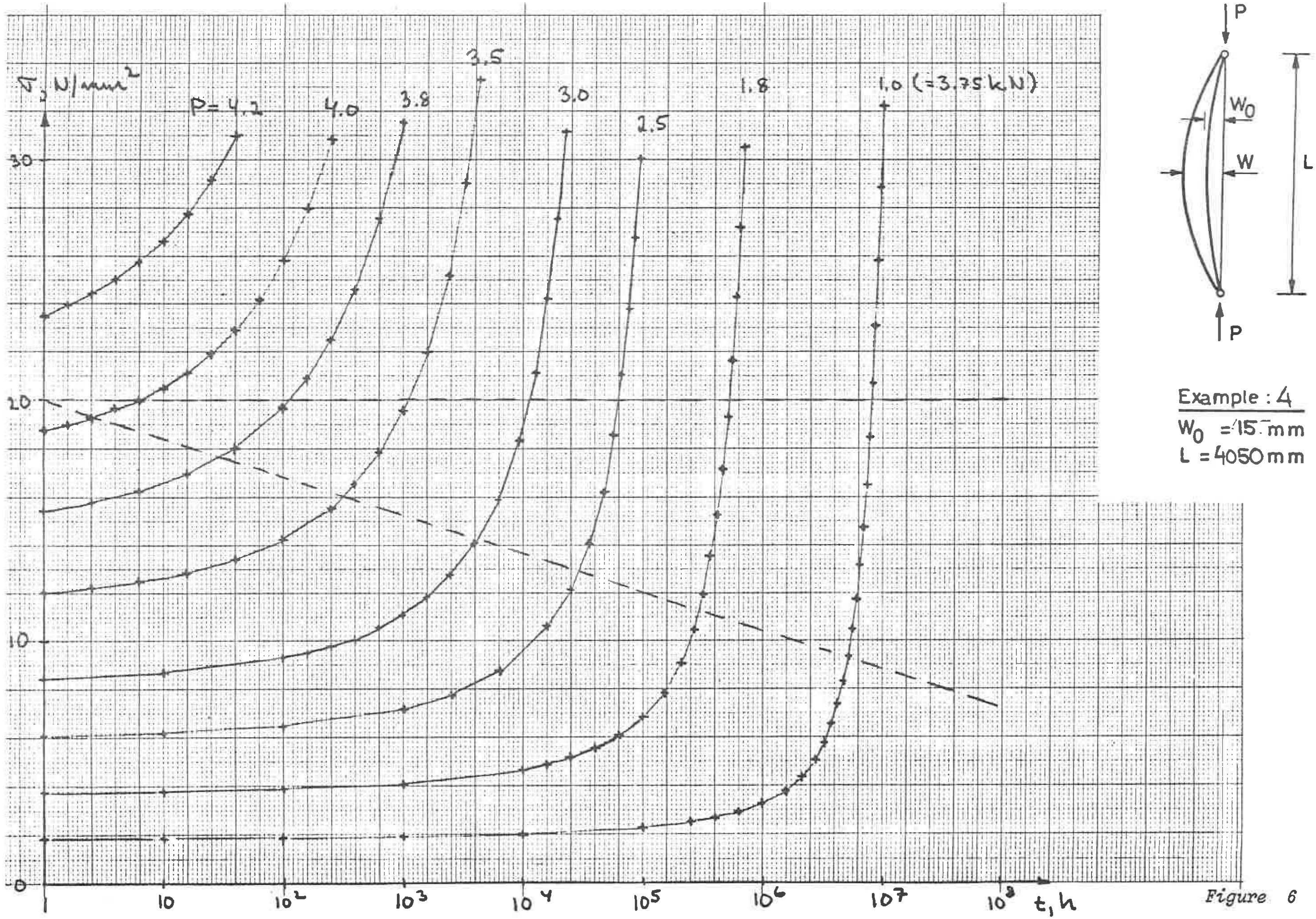
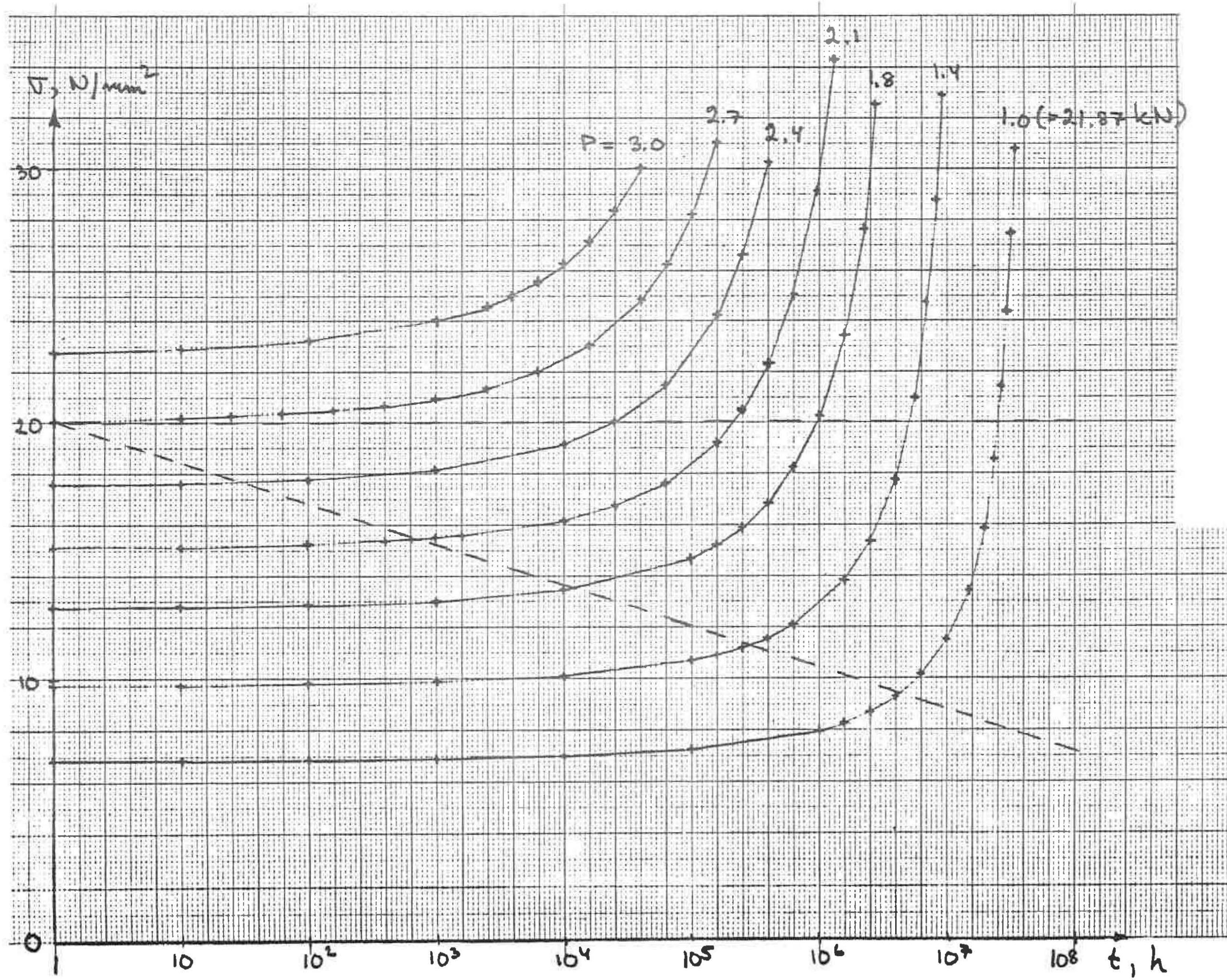
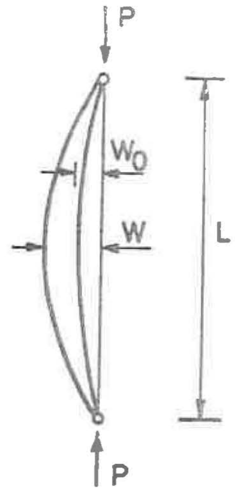

σ, N/mm²
P = 4.2
4.0
3.8
3.5
3.0
2.5
1.8
1.0 (=3.75 kN)
30
20
10
0
1
10
10²
10³
10⁴
10⁵
10⁶
10⁷
10⁸
t, h
P
W0
W
L
Example : 4
W0 = 15 mm
L = 4050 mm

Figure 6





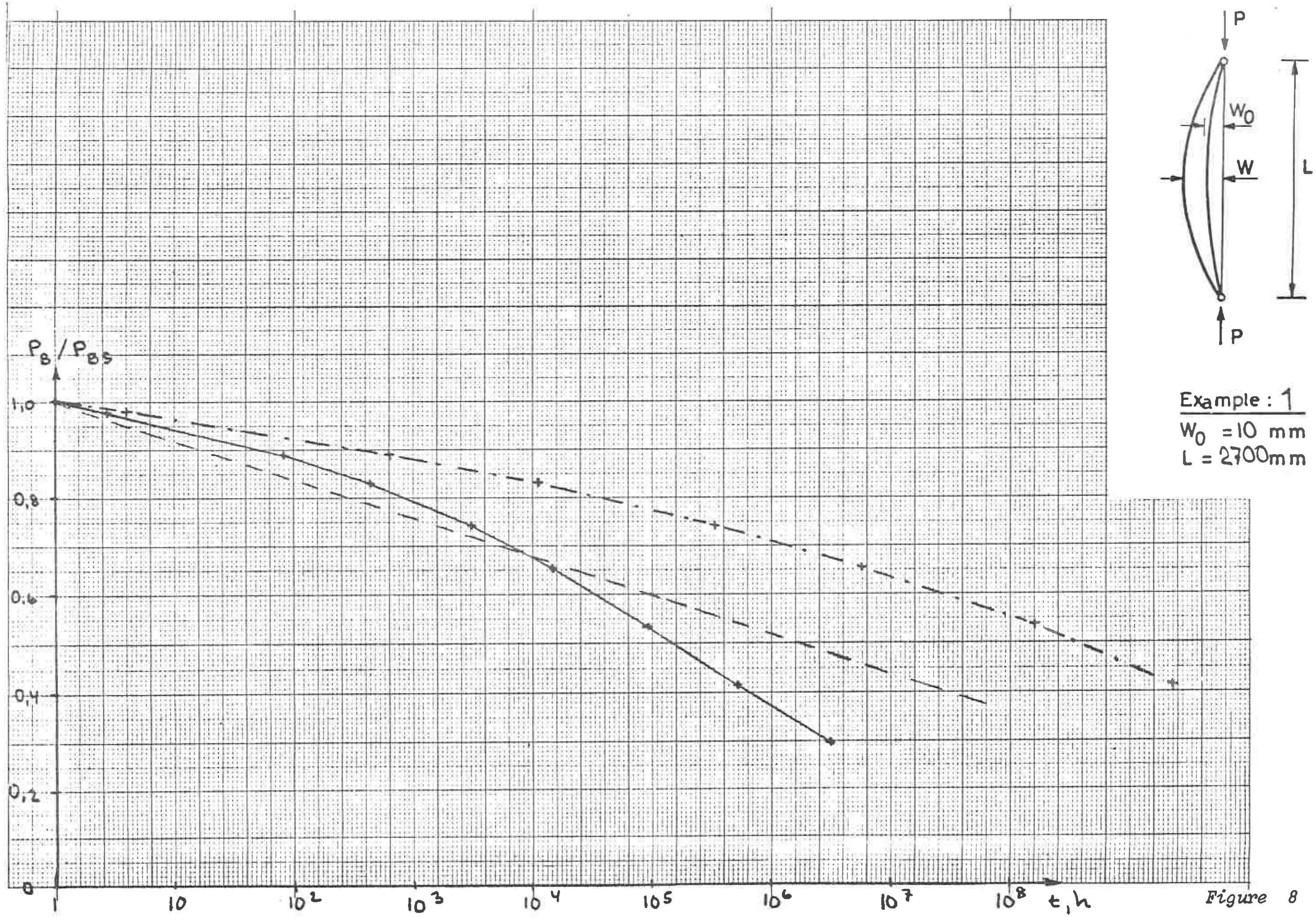
Example : 5

W_0 = 5 mm

L = 1350 mm

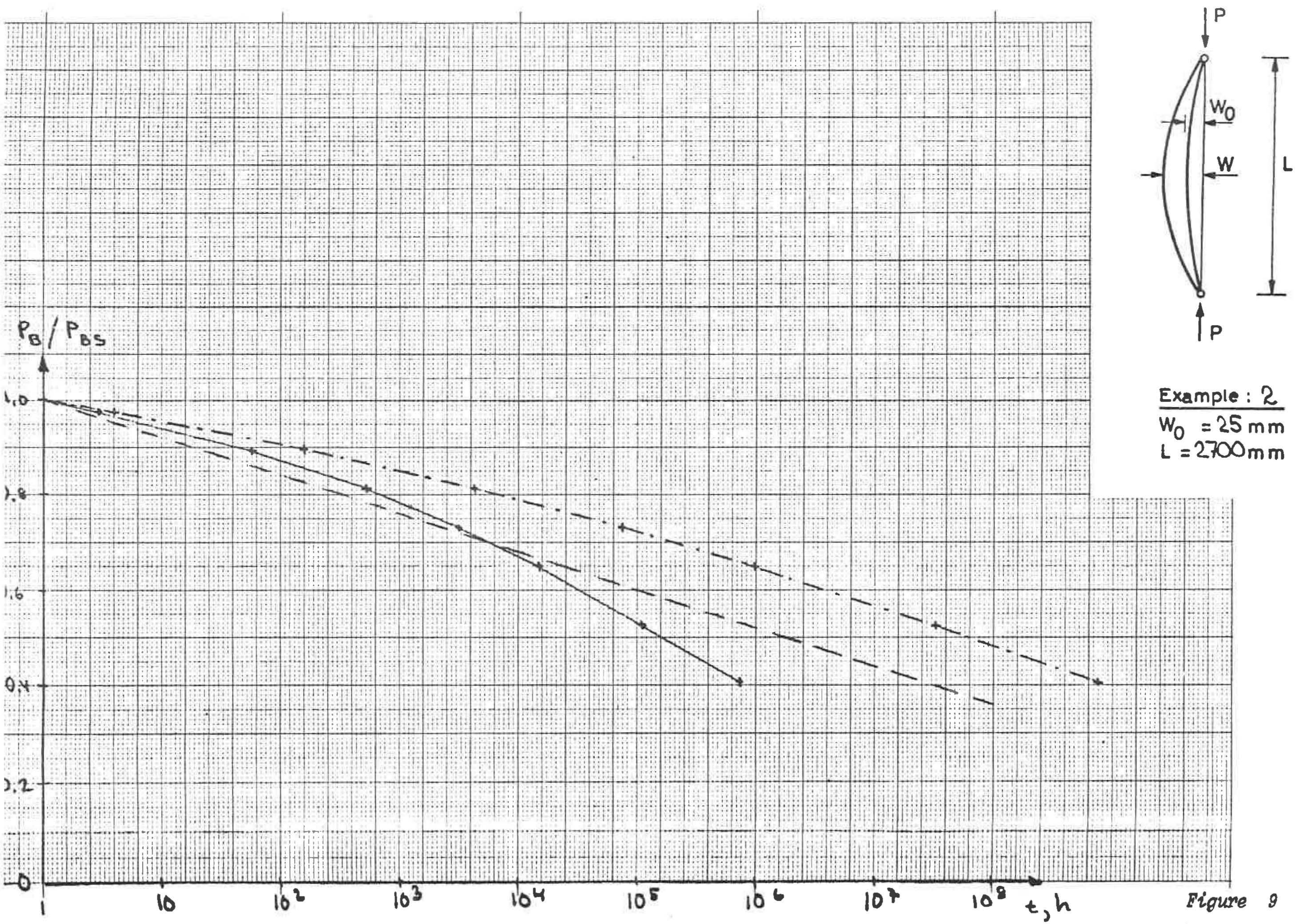
Figure 7

Figures 8 to 12 Time influence on ultimate load



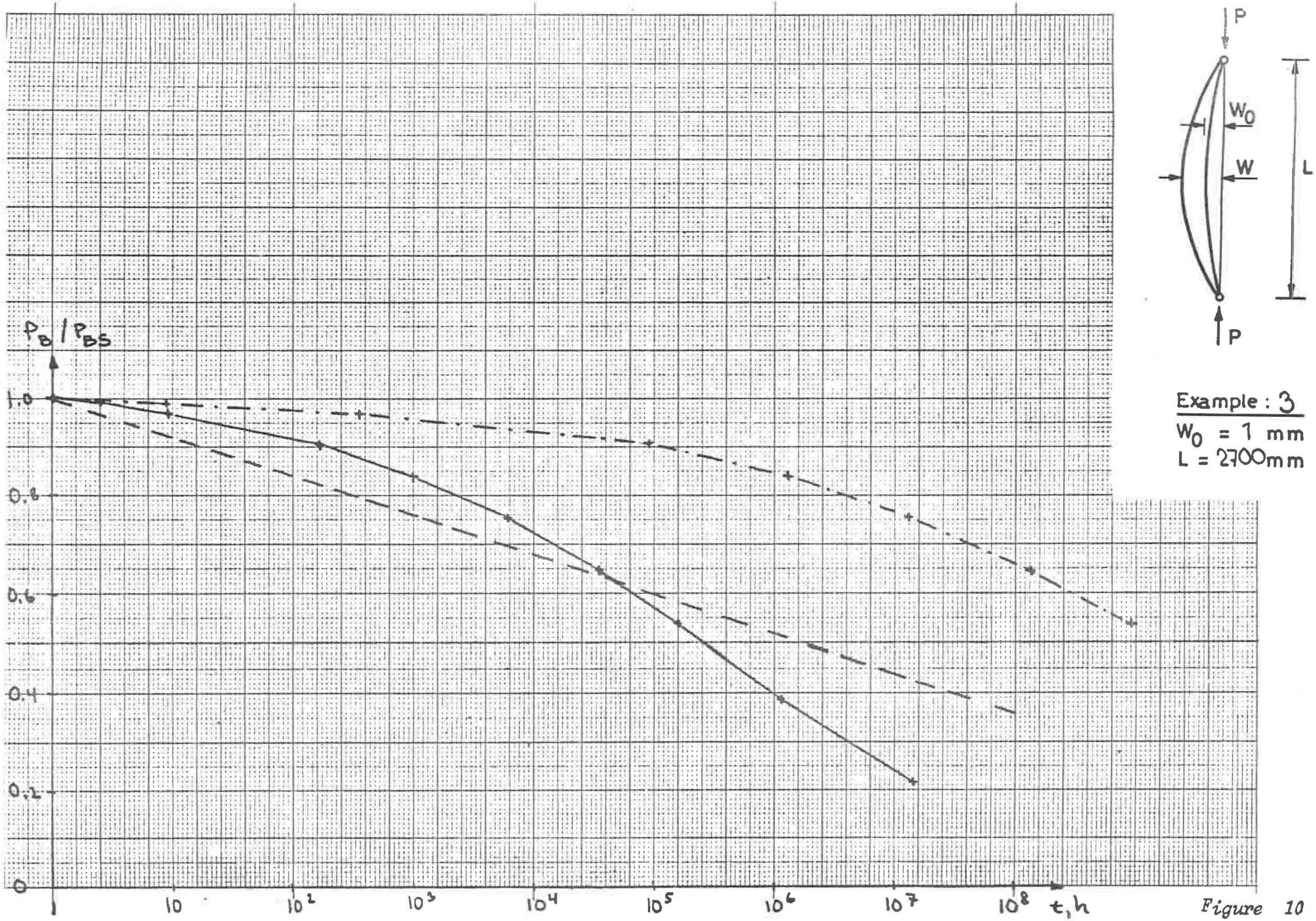
P
W0
W
L
P
Example : 1
W0 = 10 mm
L = 2700mm
PB / PBS
1,0
0,8
0,6
0,4
0,2
0
1
10
10²
10³
10⁴
10⁵
10⁶
10⁷
10⁸
t, h

Figure 8



P_B / P_{BS}
1,0
0,8
0,6
0,4
0,2
0
1
10
10^2
10^3
10^4
10^5
10^6
10^7
10^8
t, h
P
W_0
W
L
Example : 2
W_0 = 25 mm
L = 2700 mm

Figure 9



P
W0
W
L
P
Example : 3
W0 = 1 mm
L = 2700mm
PB / PBS
1.0
0.8
0.6
0.4
0.2
0
1
10
10 2
10 3
10 4
10 5
10 6
10 7
10 8
t, h

Figure 10

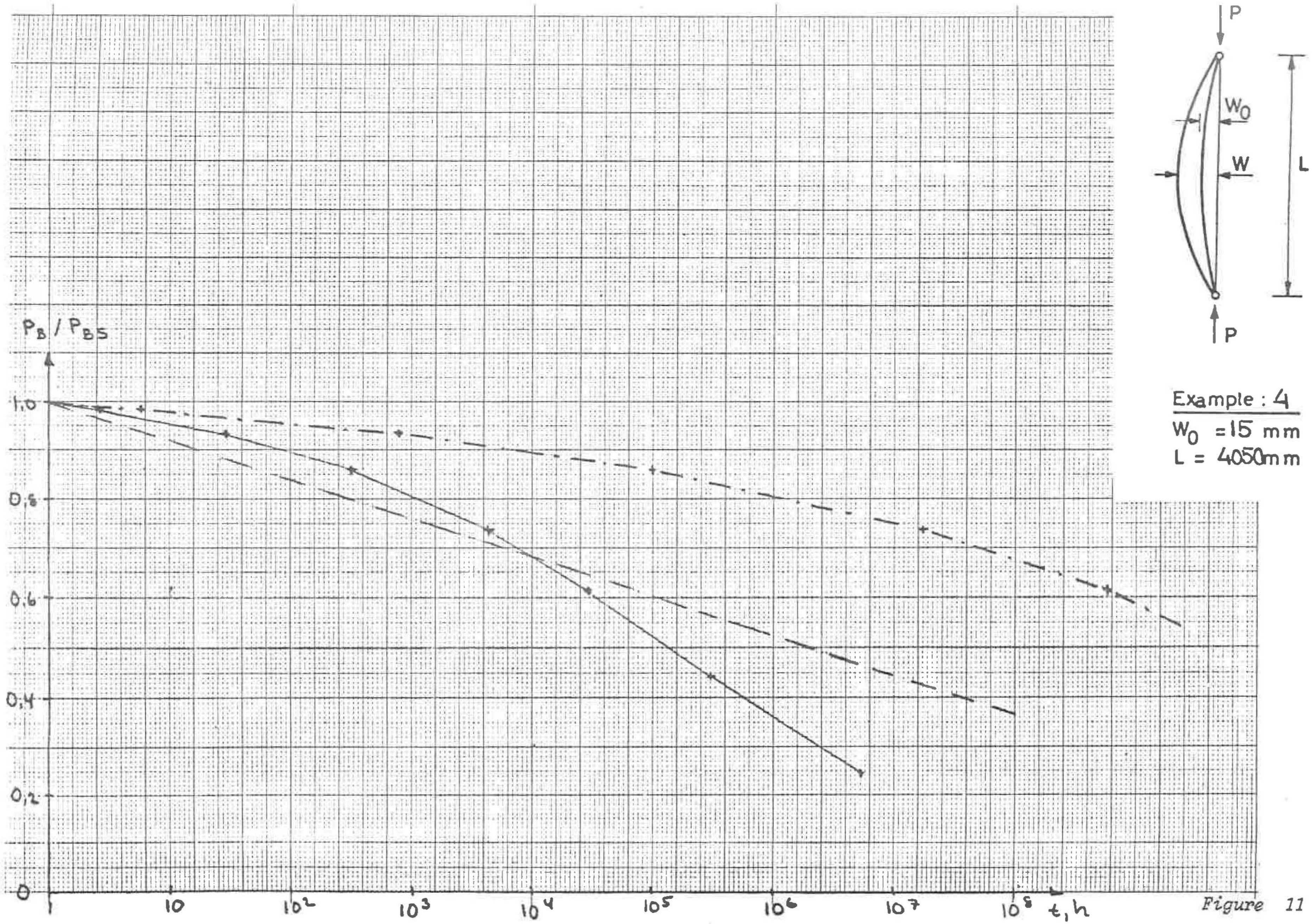
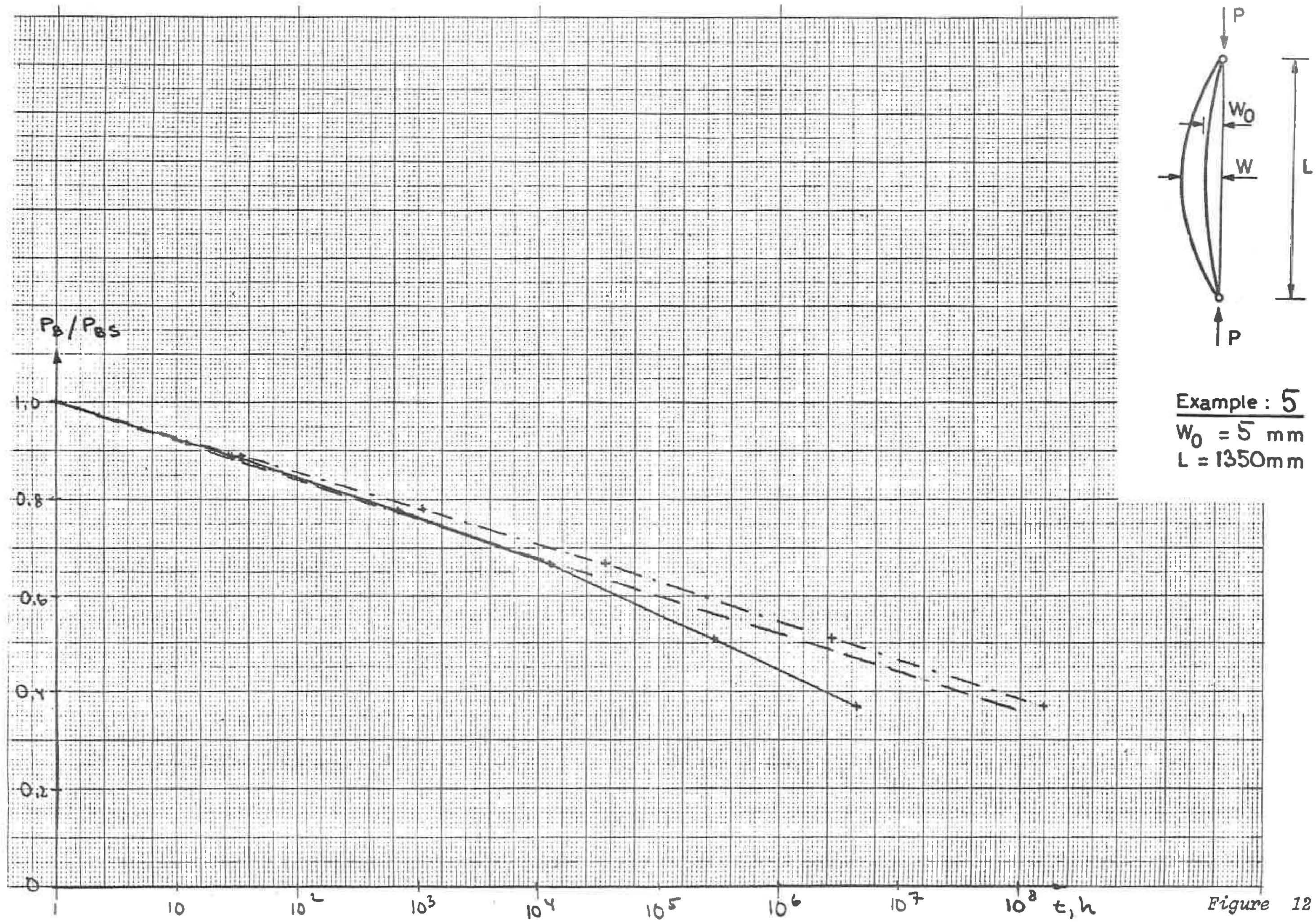

P
W0
W
L
P
Example : 4
W0 =15 mm
L = 4050mm
PB / PBS
1,0
0,8
0,6
0,4
0,2
0
1
10
10²
10³
10⁴
10⁵
10⁶
10⁷
10⁸
t, h

Figure 11

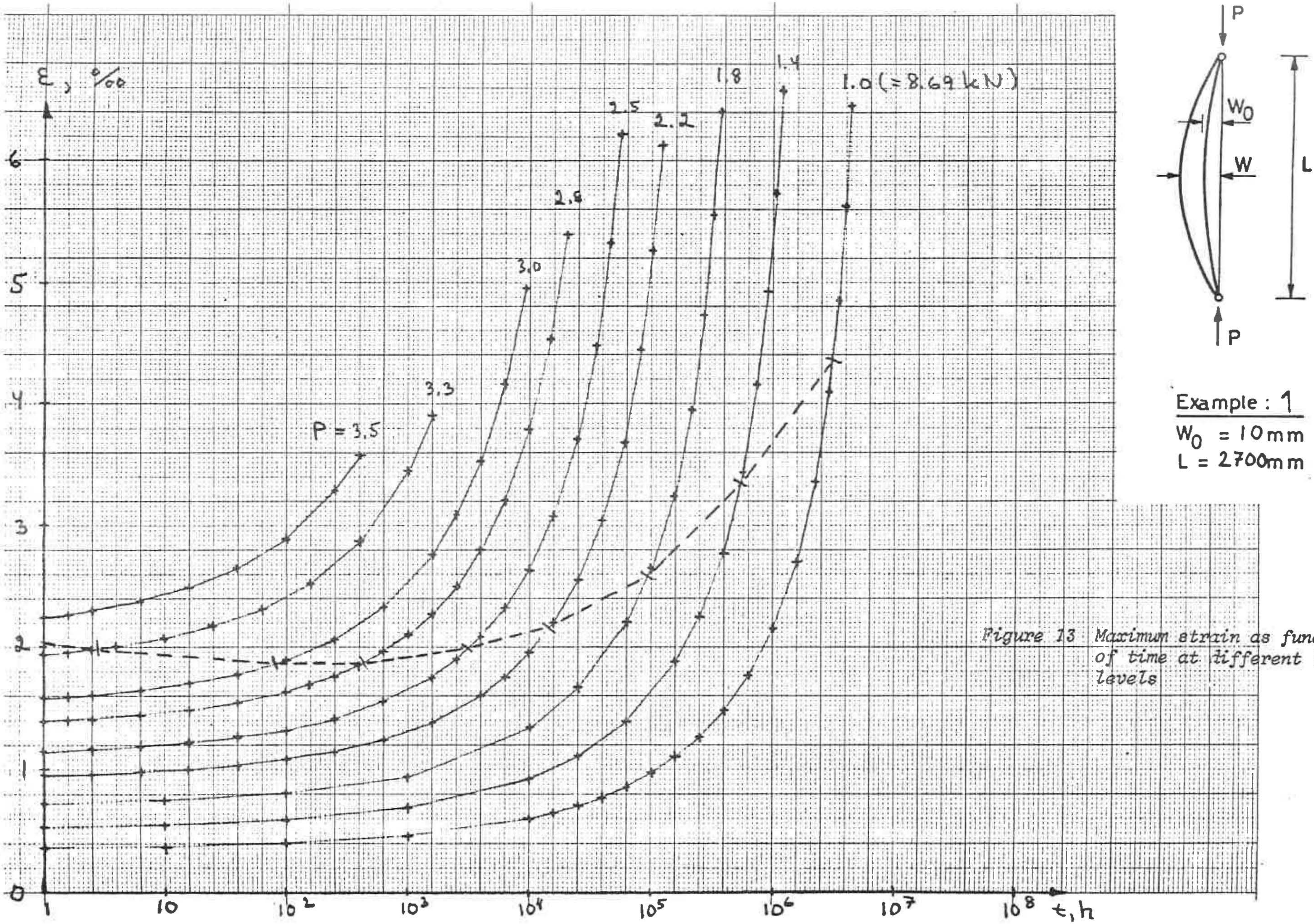


Example : 5

$W_0 = 5$ mm

L = 1350mm

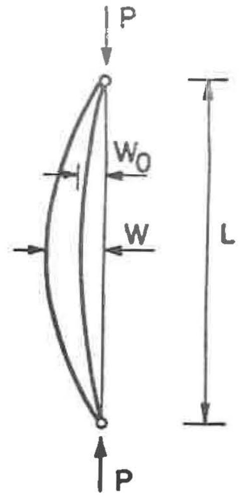
Figure 12



Example : 1

W_0 = 10 mm
L = 2700 mm

Figure 13 *Maximum strain as function of time at different load levels*



Example : 1

W_0 = 10 mm
L = 2700 mm

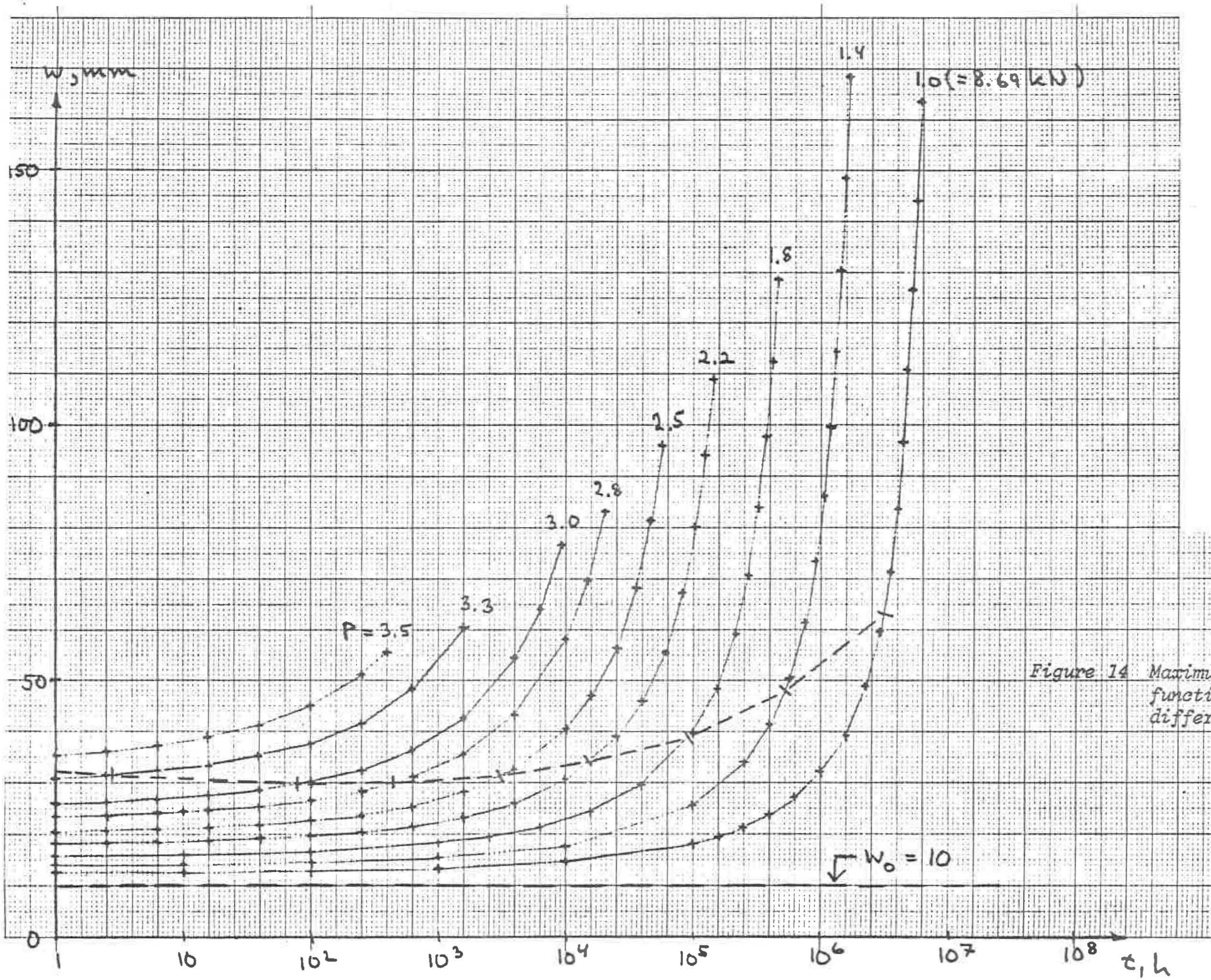


Figure 14 Maximum deflection as function of time at different load levels

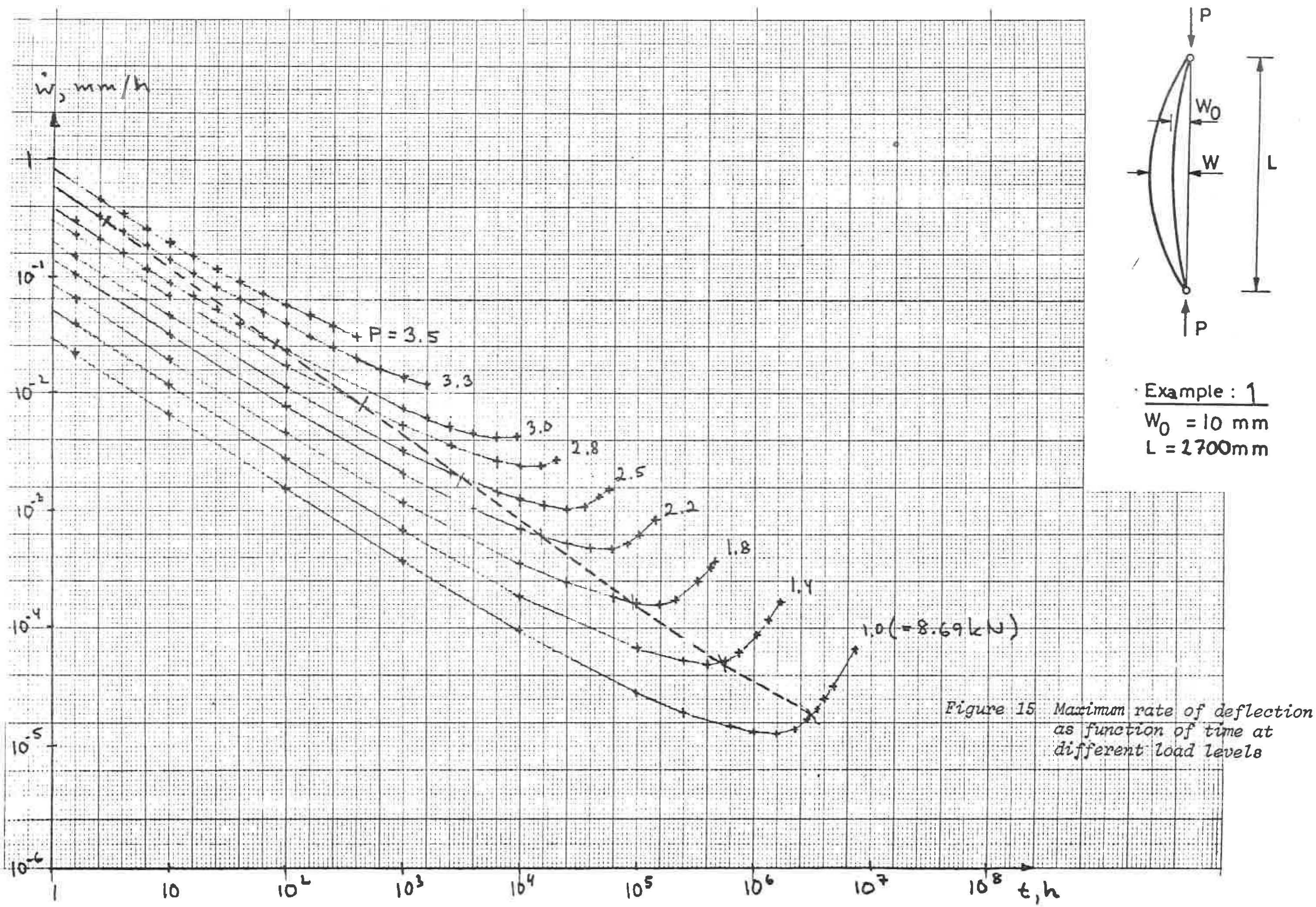


Figure 15 Maximum rate of deflection as function of time at different load levels

The Creep Function

The creep function used was fitted to results of tests with clear pine wood (Pinus sylvestris). In these tests the load was kept constant at comparatively low levels. As the time for the test was short one can neglect the secondary creep. The creep velocity is then from eq. (5):

$$\dot{\varepsilon}_c = k_2 \sigma^{n_2} t^m \qquad (1:1)$$

and the creep function:

$$f(t) = k_2 E \sigma^{n_2 - 1} t^{m+1}/(m+1) = a t^{m+1} \qquad (1:2)$$

The values a = 0.0224 and m+1 = 0.356 were chosen giving the circled curve in Figure A 1. The corresponding creep velocity is

$$\dot{\varepsilon}_c = 0.008\ t^{-0.64} \varepsilon_o \qquad (1:3)$$

where $\varepsilon_o = \sigma/E$ is the instantaneous strain at loading (t = o).

The adopted coefficients correspond to a relative creep of 0.12 after 10^2h and 1.35 after 10^5h. The corresponding fictive MOE are

$$E_2 = \frac{E}{1+0.12} \text{ and } E_5 = \frac{E}{1+1.35}$$

$$E_5/E_2 = 1.12/2.35 = 0.48$$

If the E-value given in the Code of practice is applicable for an effective duration of 10^2h, this E-value should be multiplied by 0.48. This factor is slightly smaller than the relation of the MOE at long-term loading (dead load) to the MOE at short-term loading (exceptional load) given by the Swedish Code (SBN 75):

$$E_{LT}/E_{ST} = 0.7/1.3 = 0.54$$

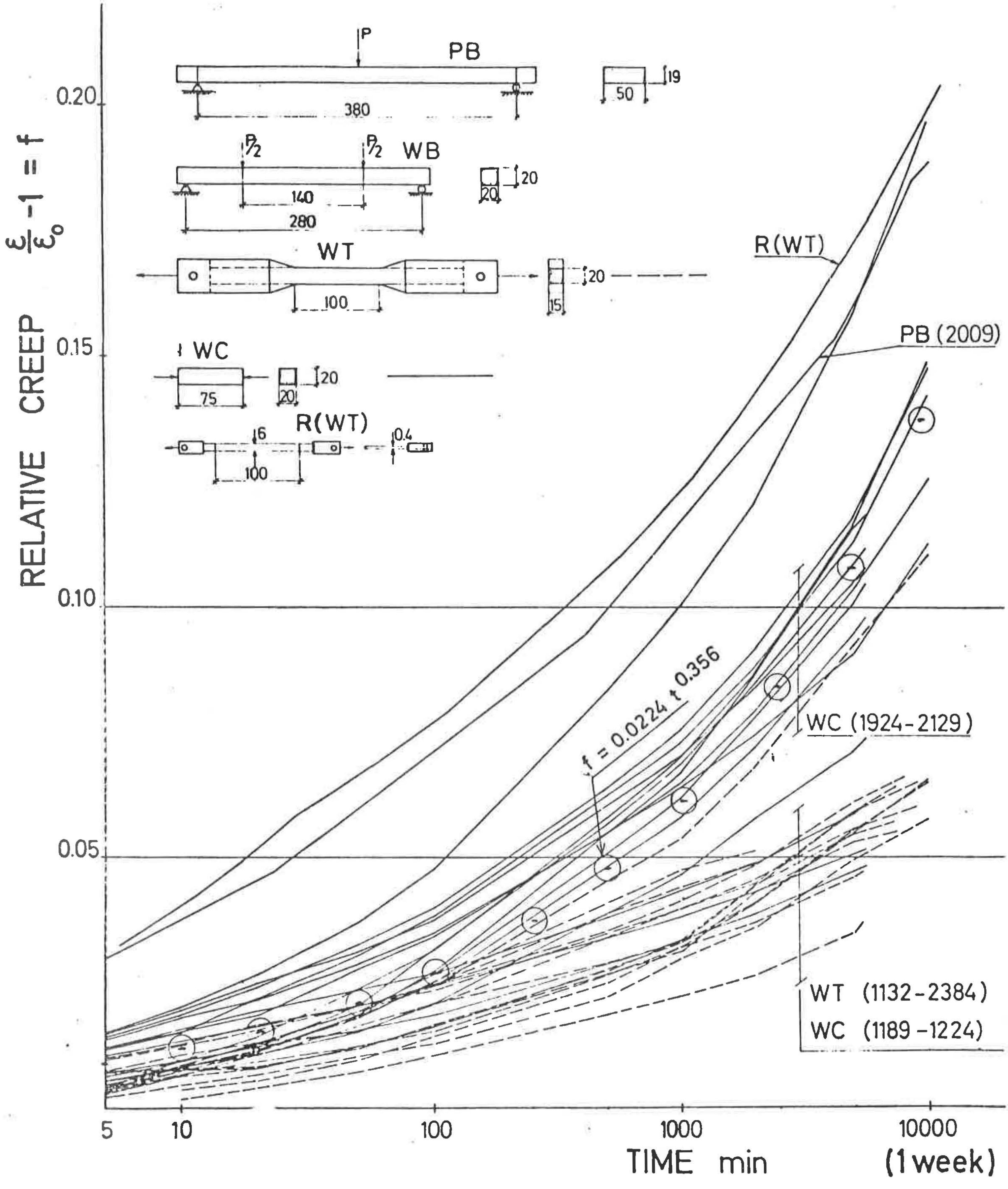


Figure A1 Result of creep tests and fitted curve (circles)

Note to the Creep-in-column paper by Källsner and Norén

The time influence on the strength of the column could alternatively have been calculated simply by substituting the modulus of elasticity E in the elementary formulas by a fictive modulus

$$E_f = \frac{E}{1 + f(t)}$$

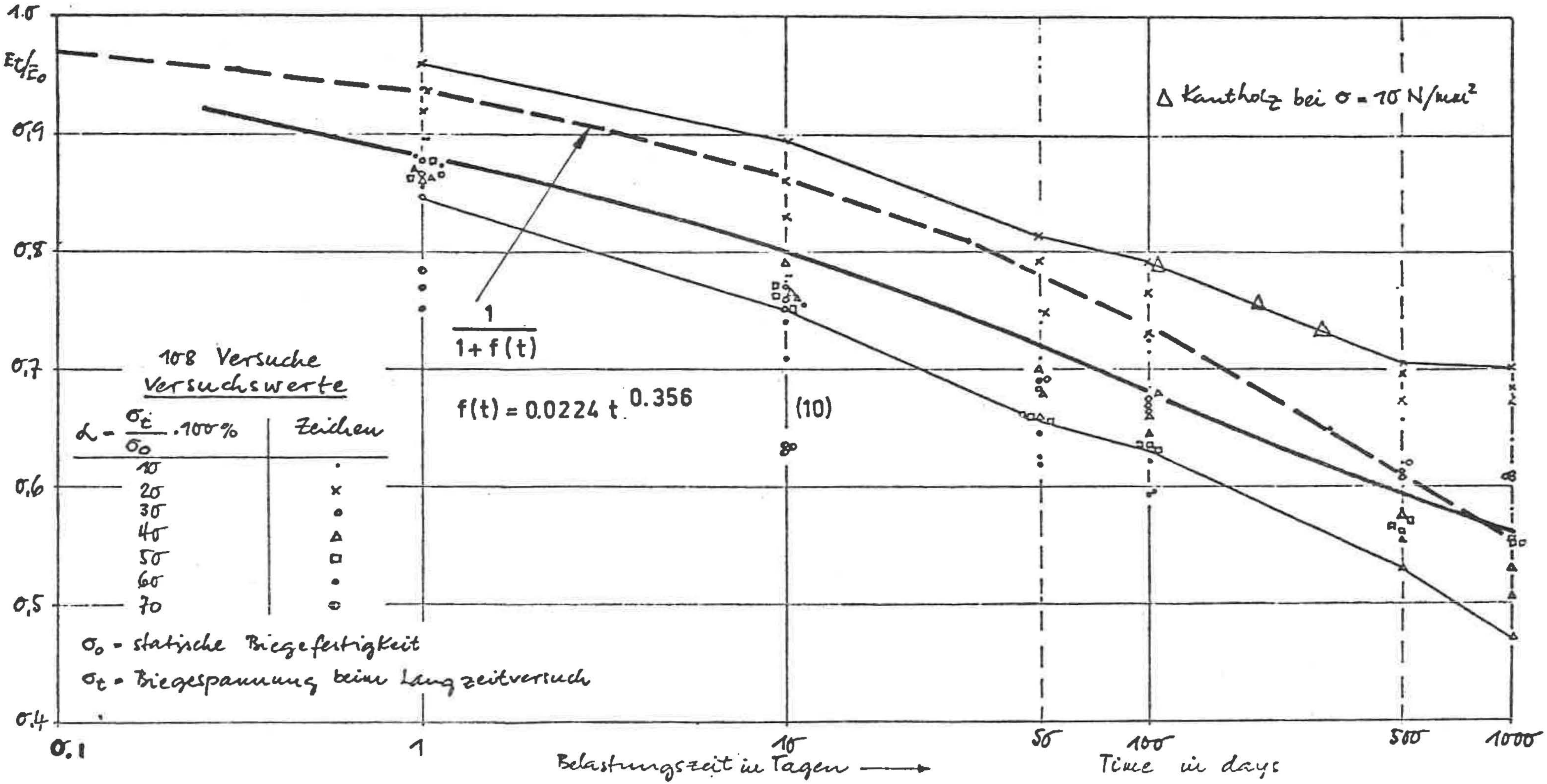
This would have given stress curves curving slightly earlier than the curves in Figures 3 to 7. The corresponding strength reduction curves will after some time fall below the full curves in Figures 8 to 12. The drop at $t = 10^6$h is about

Example	$\Delta(P_B/P_{BS})$
1	23%
2	10%
3	35%
4	20%
5	10%

Thus the approximation gives result on the safe side.

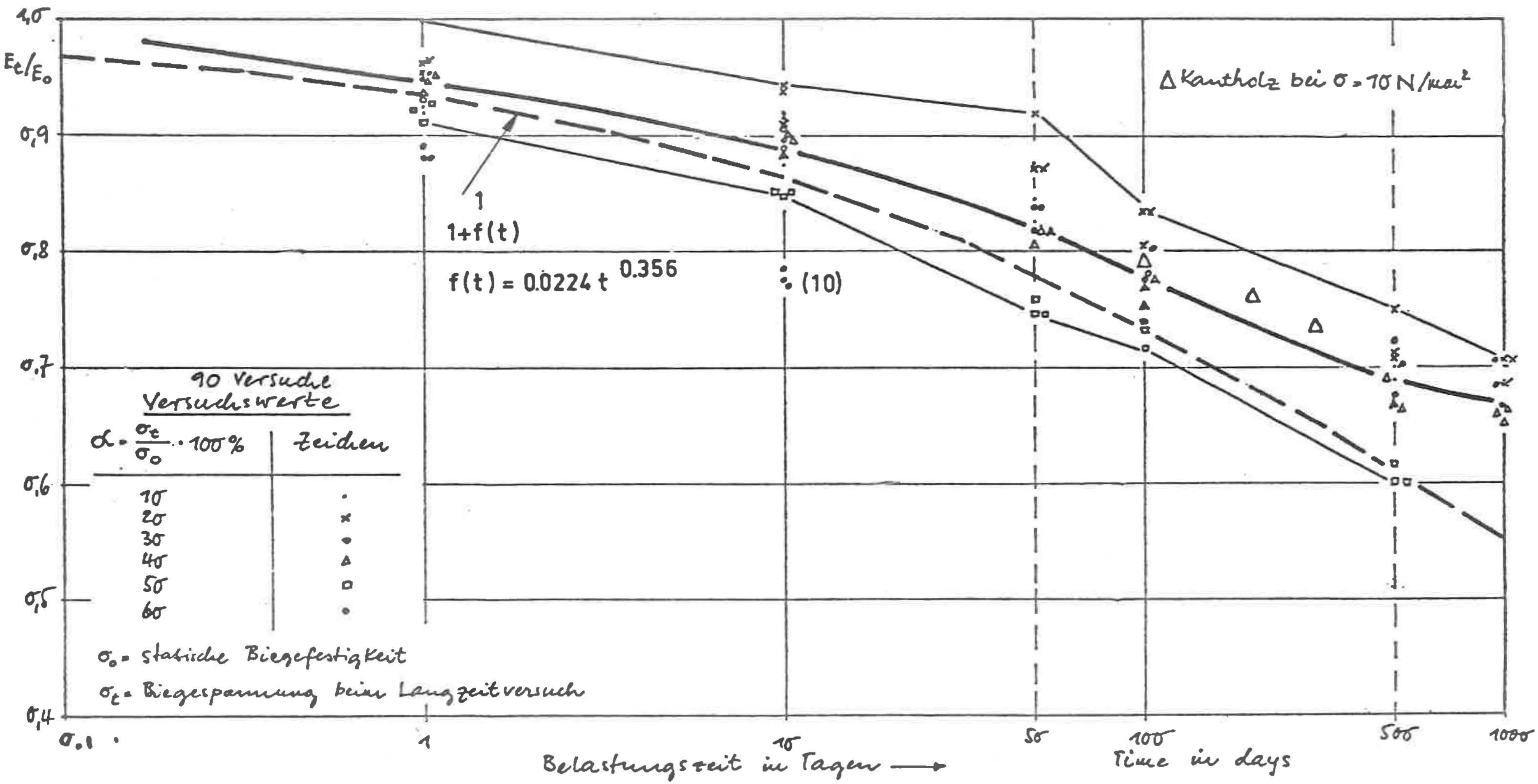
Acknowledgement

Bernt Johansson (Statens Planverk) initiated the calculations and Åke Samuelsson (FFA) made his programme available. The authors express their gratitude.



Biegeversuche

E_t / E_0 - Zeit - Abhängigkeit bei liegenden Jahrringen



Biegeversuche

E_t/E_0 - Zeit - Abhängigkeit bei stehenden Jahrringen

CIB-W18/5-10-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION - TIMBER STRUCTURES

THE DESIGN OF TIMBER BEAMS

by

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DENMARK

KARLSRUHE - OCTOBER 1975

CONTENTS

1. INTRODUCTION

The present paper has been prepared as a background for the work of the CIB/W18-Timber Structures in setting up the basis for an international standard for timber structures.

The report deals with the design of beams of both solid timber and glulam, and glued thin-webbed or thin-flanged I- and box beams (flat stressed-skin panels).

A number of West European and North American timber codes have been studied and an attempt has been made to clarify and evaluate the background for the regulations of the codes.

2. SYMBOLS

In this review of code regulations and their basis no opinion has been given on safety systems, etc. Loads and strength parameters (which might also be moduli of elasticity) are assumed to be supplied with safety factors or partial coefficients in conformity with the safety rules chosen.

The applied symbols and designations are in conformity with Draft International Standard ISO/DIS 3898, 1975 and the proposal from the CIB-Timber Structures group.

The following main symbols have been used:

E	modulus of elasticity	f	strength parameter
(EI)	bending stiffness		f_b bending strength
G	shear modulus		f_c compressive strength
(GI_v)	torsional stiffness		f_t tensile strength
I	moment of inertia (geometric)		f_v shear strength
M	bending moment	h	depth
V	shear force	ℓ	span
W	section modulus	r	radius
X	gravity axis of beam	t	thickness
Y	main axes of cross-section	α	dimensionless factor
Z	main axes of cross-section	λ_v	slenderness ratio in lateral buckling
a	length	τ	shear stresses
b	width	σ	axial stresses
			σ_b axial stress in bending
			σ_{crit} critical stress

Other symbols will be defined when used.

3. SOLID TIMBER, BENDING

3.1. Rectangular cross-section, bending in one plane

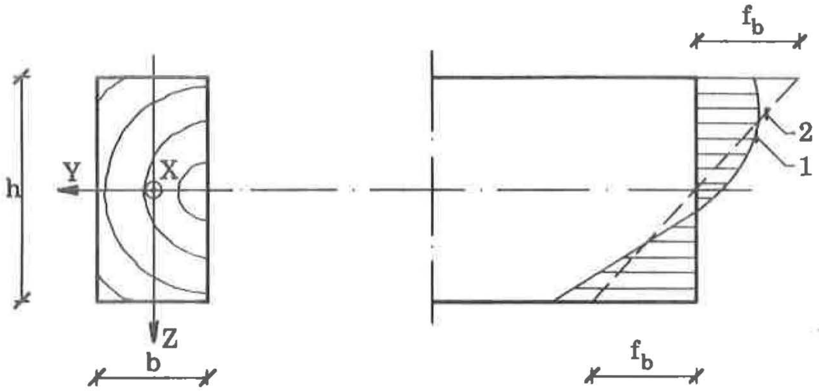
All the codes dealt with in this paper assume the following condition to be satisfied in bending about one of the principal axes (here the Y-axis, cf. fig. 3.1):

$$\sigma_b = M_y/W_y \leqslant f_b \tag{3.01}$$

where σ_b is the bending stress from the moment M_y, f_b is the bending strength, and W_y is the section modulus:

$$W_y = bh^2/6 \tag{3.02}$$

In a few cases (3.01) is given directly, but in most cases it is just implied in the code text.



a) b)

Fig. 3.01. Curve 1 of fig. b) shows the real stress distribution in principle, while curve 2 is the approximation of the theory of elasticity.

Although eq. (3.01) corresponds to the theory of elasticity, this has not been assumed. It is, however, assumed that the stress distribution and strength parameters are independent of the dimensions of the rectangular cross-section, and - of course - that (3.01) - (3.02) are used to determine f_b from rupture tests.

3.2. Rectangular cross-section, size effect

Tests performed by, for example, Newlin & Trayer*, Dawley & Youngquist**, Camben [21], Bechtel & Norris [16] and Bohannan [17] have shown that the bending strength decreases as the size of the beam increases.

Originally, the phenomenon was considered solely a depth effect and explained (Newlin & Trayer) by the so-called »support theory», which is described in [20] as follows:

> The fibres in the compression zone of a beam act individually as small columns and the more highly stressed fibres near the edge of the beam are restrained by those relatively unstressed fibres near the neutral axis. In a shallow beam these restraining fibres are nearer (this distance is absolute and note relative since the size of the fibres is the same for all depths of beam) to the compression edge than in the case of a deeper beam and hence a greater modulus of rupture is achieved.

The explanation now generally recognized is based on Weibull's statistical rupture theory for brittle materials [40]. This theory assumes that exceeding of the ultimate strength in a single point (part volume) will result in total collapse of the whole member. According to this theory it will not only be a question of depht effect, but the strength will depend on the volume, which is exposed to large stresses. Thus, the theory also explains another phenomenon observed, i.e. that the strength depends on the type of load: e.g. strength with four-point load is less than with three-point load (= mid-span load).

* The result is mentioned in [4].

** Referred to in [27].

On the basis of tests, Bohannan has evaluated the theory [17] and found that the correspondence between theory and tests is not particularly impressive. There will be good correspondence, however, if the front area of the beam is used as a parameter instead of the volume, i.e. if the influence of the width is not taken into account. Bohannan tries to explain this, but his explanation is not convincing, and for the time being it must be accepted merely as a fact.

On the basis of Bohannan's work the following rules, cf. [5], can be given for determining the bending strength, $f_{b,h}$, of a beam with the depth h (in mm) in relation to the bending strength $f_{b,200}$ of a beam with the depth of 200 mm, uniformly loaded with $\ell/h = 21$:

$$\frac{f_{b,h}}{f_{b,200}} = \alpha\left(\frac{200}{h}\right)^{\frac{1}{9}} \tag{3.03}$$

where α is given in table 3.03 dependent on the load type and the ratio ℓ/h between the span ℓ and the depth h. For other load types the factor can be determined by comparison and interpolation. The factor $\left(\frac{200}{h}\right)^{\frac{1}{9}}$ is shown in fig. 3.02.

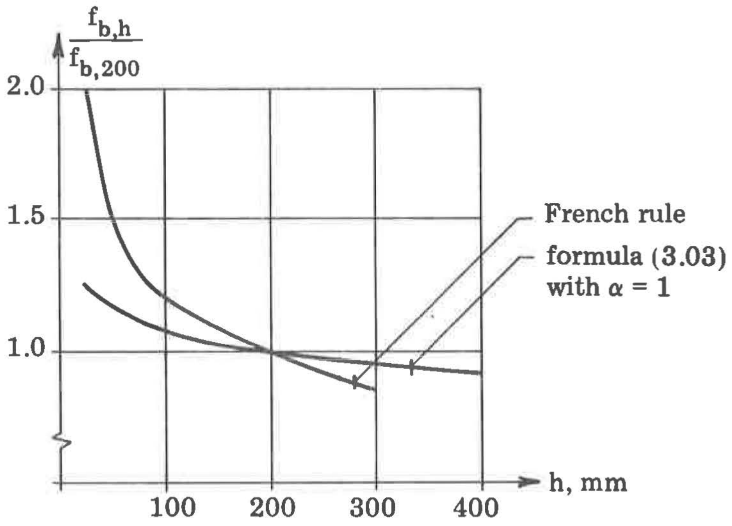


Fig. 3.02

$\frac{\ell}{h}$	mid-point ($\ell/2$, $\ell/2$)	uniform (ℓ)	third-point ($\ell/3$, $\ell/3$, $\ell/3$)
7	1.15	1.06	1.03
14	1.10	1.02	0.99
21	1.08	1.00	0.97
28	1.06	0.98	0.95
35	1.05	0.97	0.94

Table 3.03. Factor α in eq. (3.03).

The conclusion is that there is a marked depth effect (size effect), and that this has been tested so thoroughly, at any rate for North American structural timber, that it is possible to give relatively simple rules. The only question is therefore whether the better utilization of the materials justifies the calculations being made more difficult.

So far, most code-writers have apparently judged that it was not worth-while, as only the French Timber Code [10] gives a depth factor for ordinary structural timber.

Indeed there are depth rules in the British Timber Code [6] and in the Timber Construction Manual [5], but they only give a strength reduction for $h \geqslant 300$ mm and with European timber dimensions they apply to glulam only. Therefore, the peculiar situation arises that a set of rules which, with a few exceptions*, are based on ordinary structural timber of depths below 300 mm (most tests even with h equal to 25-50 mm), are solely applied to glulam with depths considerably exceeding 300 mm.

The French rules [10] which do not take the load types, etc. into account, are drawn in fig. 3.02, taking as point of origin h = 200 m (in [10] h = 150 mm is used as a standard depth). It is seen that a considerably larger depth effect than that according to Bohannan's results has been assumed.

If it is decided to introduce a depth factor it must be a prerequisite that reasonable test series are carried out with European timber.

3.3. Other cross-sections and bending in two planes

The bending strength is not a material parameter, but only a design factor related to the rectangular cross-section. In bending tests with other cross-sections other formal bending strengths are found.

To take this into consideration a form factor α_f can be used so that the requirement by bending about one of the principal axes can be written

$$\sigma_b = M/W \leqslant \alpha_f f_b \qquad (3.04)$$

The stress distribution is suggested in fig. 3.01 b and implies that the fibres near the gravity axis become more involved than by the theory of elasticity. Therefore, the form factors are greater than 1 for circular cross-sections, square cross-sections stressed in bending about a diagonal, and others where the area is concentrated near the axis, but less than 1 for box beams and I-beams.

An impression of the size of the form factor can be obtained by taking into account the stress distribution shown in fig. 3.04, where ideal-elastic plastic conditions are allowed for in the compressive side and ideal-elastic conditions in the tensile side, with a tensile strength of f_t, which is α times the compression strength $f_c (\alpha \geqslant 1)$.

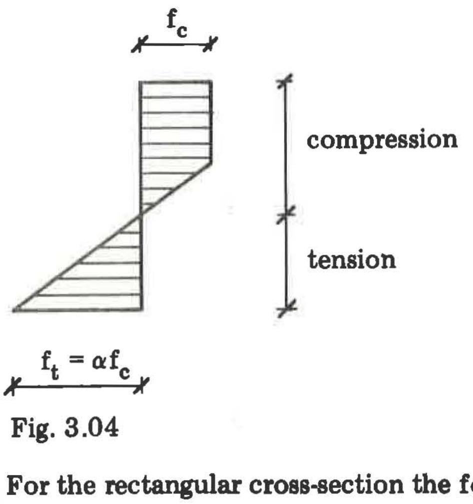


Fig. 3.04

For the rectangular cross-section the following expression is found in this case:

$$f_b/f_c = 3 - 4/(1 + \alpha) \qquad (3.05)$$

* A few of the tests in [27] were performed with h = 400 mm (16"), and Bohannan's tests comprise 3 glulam beams with h ~ 800 mm and ℓ = 14.6 m, but these results cannot be immediately applied to European conditions.

As an extreme value, α = 2 is used in the following, giving f_b/f_c = 1.67.

For a square section on edge f_b/f_c = 1.82 is found, i.e. the theoretical form factor is 1.82/1.67 = 1.10.

For the cross-section shown in fig. 3.05 a, f_b/f_c = 1.45 is found, i.e. α_f = 1.45/1.67 = 0.87.

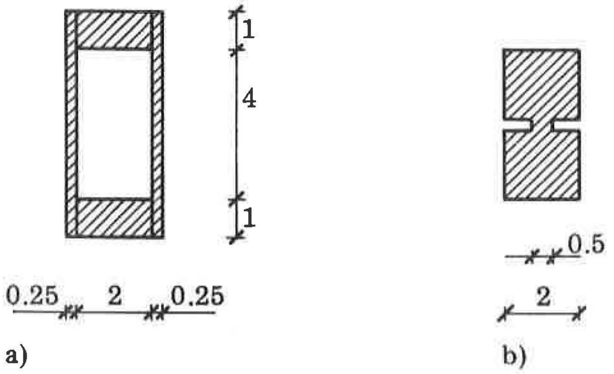


Fig. 3.05

Further, the square section on edge is just a special case of bending of a rectangular cross-section about both principal axes, where the form factor depends on the depth-width ratio and moment direction. Generally, the form factor is found greater than or equal to 1 (for bending about one of the principal axes), rarely, however, above 1.05.

Newlin & Trayer [3], who determined the form factors by experiments, found $\alpha_f \sim 1.2$ for the circular cross-section and about 1.4 for a square section on edge. For a number of I- and box sections, α_f was found in the interval between 0.65 and 0.90; thus, for a cross-section corresponding to fig. 3.05 a $\alpha_f \sim 0.7$ was found. According to these tests the shape therefore has a relatively strong influence.

Newlin & Trayer explain the form factors by the fact that the outermost fibres, being essential to the bending strength, are fixed better in the compressive side of the cross-section with material near the neutral axis than in I- and box beams, and an empirical design model is set up. As seen from above, however, the form factors might also be explained just by the form of the stress-strain curve. However, an unexpected result from the tests should be mentioned, namely that a form factor of 0.9 was found for the cross-section shown in fig. 3.05 b.

Of the codes dealt with in this paper only the British and certain USA rules give form factors. Here, 1.18 is stated for circular beams and 1.41 (= $\sqrt{2}$) for a square section on edge.

The justification for these rules is doubtful. The basis is Newlin & Trayer's tests, which were performed only for very small cross-sections (50 mm side length or diameter) using absolutely perfect Sitka (»No material was used having knots or pitch pockets, no matter how small»).

The suitability is also doubtful. In practice, circular cross-sections will only occur in the form of logs for which the grading rules - if they exist at all (they do not exist in the UK-Code) - are quite different from those of sawn timber, the strength of which is used as a reference. Therefore, it appears more reasonable, as e.g. in the German Timber Code, to give individual strength parameters, also for other cases than bending. As regards the square section on edge, it seems unreasonable to give a form factor for this case, which hardly ever occurs in practice, while for the ordinary case - bending about both axes - there is none, neither for square nor for rectangular cross-sections. Here the factor 1.4 is unaccountably high.

For I-beams, cf. fig. 3.06, the French Timber Code [10] gives the following form factor:

$$\alpha_f = 0.58 + 0.42\left[\frac{t}{h}\left(1-\frac{a}{b}\right)+\frac{a}{b}\right] \tag{3.06}$$

as taken from [34].

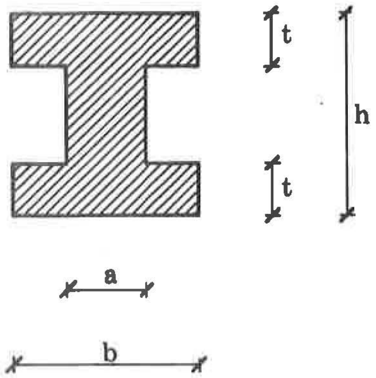


Fig. 3.06

The other countries give no form factors for I- and box beams, although there is a distinct effect. In the Danish and German timber codes, among others, there is, however, a special rule, having the same effect in practice, namely the requirement that the mean stresses in the flanges must be kept below the tensile and compressive strength. Having f_t/f_b about 0.6 - 0.7 there will generally be good approximation between the strength reductions according to this rule and those experimentally determined in [4]. Thus, for the cross-section in (3.05) $0.6\frac{6}{5} = 0.72$ is found for $f_t/f_b = 0.6$ and 0.84 for $f_t/f_b = 0.7$.

As regards the form factors it also appears to be necessary, if it is considered reasonable to introduce them, to have them determined also for European timber.

3.4. Lateral buckling

For beams with a small stiffness transverse to the load direction or for long beams, failure may occur for smaller loads than one would expect from the stresses occurring. For a certain load the initial undeformed stage will not be stable any more, but at the slightest load increase buckling will occur in the form of a combined flexural and torsional mode. The load for which the structure is no longer stable is in the following denoted the critical load and indicated by subscript $_{crit}$.

In the following, a number of results are given concerning the critical load based on, among others [39] and [29]. It is true that they are derived for ideal-elastic, isotropic materials, but tests by Hooley & Madsen [29] have shown that they are also applicable to timber.

A cross-section subjected to bending about the strong principal axis is considered, i.e. loaded in the Z-direction, cf. fig. 3.01. The bending stiffness of the cross-section around the main axes is EI_y and EI_z. The torsional stiffness is GI_v, and it is assumed that the cross-section remains plane, which can be reckoned to be the case for solid sections, box sections, etc. However, the following does not apply to open thin-walled sections, e.g. I-beams.

When lesser secondary terms are not taken into account (for example $\sqrt{1 - I_y/I_z} \sim 1$ is assumed), the ratio between the critical stress, σ_{crit}, and the bending strength may be written as

$$\frac{\sigma_{crit}}{f_b} = \frac{\frac{E}{f_b}\sqrt{\frac{G}{E}}}{\lambda_{v1}^2} \tag{3.07}$$

where the slenderness ratio λ_{v1} in lateral buckling is defined by

$$\lambda_{v1}^2 = \frac{W\ell}{k\sqrt{I_z I_v}} \tag{3.08}$$

k is seen from table 3.07 dependent upon the types of beam and load and the position of loading, i.e. whether the load is acting on the top side of the beam (T), in the gravity axis (M) or in the bottom (B).

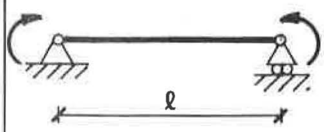
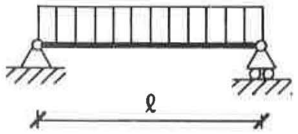
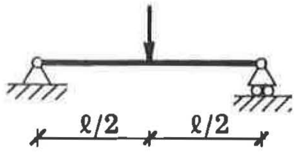
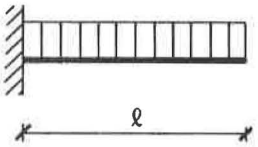
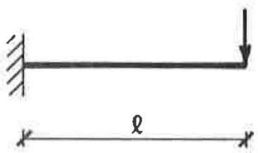
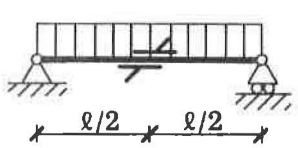
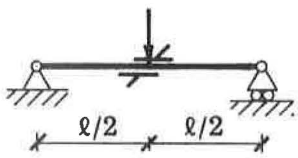
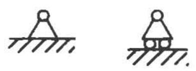
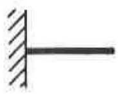
	position of loading	k	$\frac{\ell_{e1}}{\ell}$	$\frac{\ell_{e2}}{\ell}$	$\frac{\ell_{e3}}{\ell}$	$\frac{\ell_{e4}}{\ell}$
	T M B (all)	π	0.34	1.60	1.8	1.84
	T M B	~ 3.3 3.5 3.7	0.33 0.31 0.29	1.42	1.6	1.92
	T M B	3.9 4.2 4.5	0.28 0.26 0.24	1.19	1.4	1.61
	T M	6.4	0.17	0.78	0.9	1.06
	T M	4.0	0.27	1.25	1.4	1.69
	T M B	7.7 8.2 8.9	0.14 0.13 0.12			
	M	12	0.09			
	simply supported in both Y- and Z-direction, fixed against torsion					
	built-in both in Y- and Z-direction, fixed against torsion					
	beam fixed against displacement in Y-direction, fixed against torsion					
	T = Top, M = Middle (gravity axis), B = Bottom					

Table 3.07. Factor α in eq. (3.07) and effective lengths.

For rectangular beams $W = bh^2/6$, $I_z = b^3h/12$ and $I_v \sim \frac{1}{3} b^3 h(1 - 0.63\frac{b}{h})$. For $b/h \sim 0.20$, $I_v \sim 0.29\, b^3 h$, and

$$\lambda_{v1}^2 = \frac{1.07}{k}\frac{\ell h}{b^2} = \frac{\ell_{e1} h}{b^2} \tag{3.09}$$

The effective length ℓ_{e1} defined by (3.09) is given in table 3.07.

Using $E/G \sim 16$, (3.07) will be

$$\frac{\sigma_{crit}}{f_b} = \frac{0.25\frac{E}{f_b}}{\lambda_{v1}^2} \tag{3.10}$$

which is used in the Danish Timber Code [8].

As suggested by Hooley & Madsen [29] the following expression has been chosen in some codes:

$$\frac{\sigma_{crit}}{f_b} = \frac{1.20\frac{E}{f_b}}{\lambda_{v2}^2} \tag{3.11}$$

where the slenderness ratio λ_{v2} is defined by

$$\lambda_{v2}^2 = \frac{1.20}{0.25}\lambda_{v1}^2 = 4.8\,\lambda_{v1}^2 = \ell_{e2}h/b^2 \tag{3.12}$$

and the effective length ℓ_{e2}, cf. table 3.07, by

$$\ell_{e2} = 4.8\,\ell_{e1} \sim 5.0\,\ell/k \tag{3.13}$$

To take into consideration the difficulties in establishing the total support restraint against torsion, which is assumed, Hooley & Madsen [29] have suggested increasing the effective length by 15%, i.e. in (3.11) to use

$$\lambda_{v3}^2 = \ell_{e3}h/b^2 = 1.15\,\ell_{e2}h/b^2 = 1.15\,\lambda_{v2}^2 \tag{3.14}$$

The Norwegian and Canadian timber codes use (3.11) + (3.14). The effective length is given only for loads acting in the gravity axis, as the effective lengths - according to suggestions by Hooley & Madsen - for loads on the upper side are found by increasing ℓ_{e3} by 3h and for loads in the bottom by reducing by h.

In cases where ℓ_e depends on the position of loading, the Timber Construction Manual [5] gives values 35% larger than the theoretical values for load in the middle; these values, however, can be used in general, also for loads on the top edge, which is most common. The corresponding effective lengths are denoted ℓ_{e4} in table 3.07.

The theoretical expressions (3.07) or (3.11) apply only as long as σ_{crit} is less than the proportional limit, i.e. for σ_{crit}/f_b less than $0.4 - 0.67$.

For beams with λ-values below a certain limit, $\sigma_{crit}/f_b = 1$.

As f_b is normally determined with beams having $\ell > 16h$ and $h \sim 2b$ this limit is, when ℓ_e is assumed to be $1.5\,\ell$

$$\lambda_{v3}^2 = 1.5 \cdot 16h \cdot h/(0.5h)^2 \sim 100 \tag{3.15}$$

$$\lambda_{v3} \sim 10 \tag{3.16}$$

A suitable transition curve is used between $\lambda_{v3} \sim 10$ and the proportional limit. Hooley & Madsen have suggested a parabola, which is also applied in the Timber Construction Manual, while just a straight line is used in the Danish and Norwegian codes.

For $E/f_b = 300$ which is used in the Norwegian and Danish codes and also is a typical value for North American timber, σ_{crit}/f_b has been drawn in fig. 3.08, in conformity with the codes mentioned and the Timber Construction Manual. $E/f_b = 300$ might immediately appear to be a high value; however, it is due to only a small part of the cross-section being exposed to the large stresses which normally reduce the stiffness.

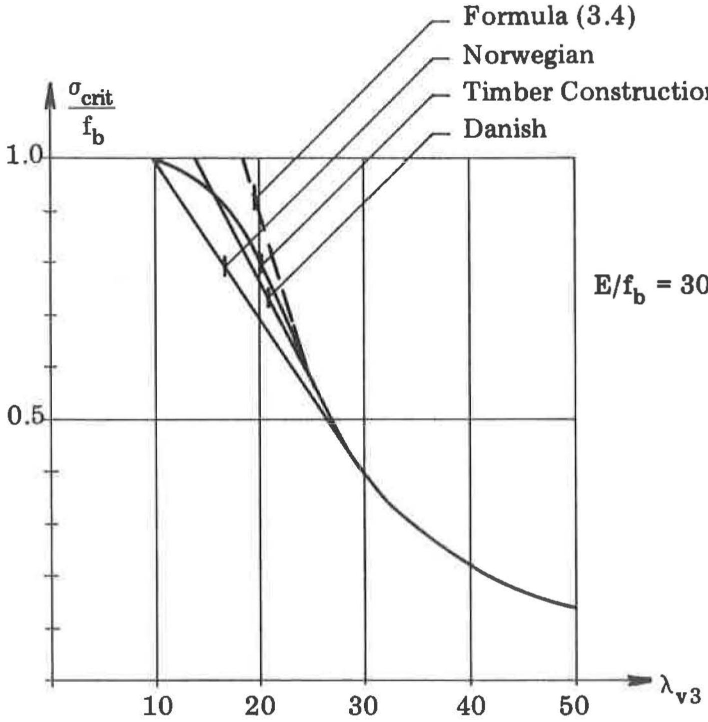


Fig. 3.08

A number of codes only allow for lateral buckling by limiting the depth-width ratio. The most common rules of this type are the Canadian-British rules, limiting depth-width as stated in table 3.09.

degree of lateral support	maximum depth to width ratio
no lateral support	2
ends held in position	3
ends held in position and member held in line, as by purlins or tie rods	4
ends held in position and compression edge held in line, as by direct connection of sheathing, deck or joists	5
ends held in position and compression edge held in line, as by direct connection of sheathing, deck or joists, together with adequate bridging or blocking spaced at intervals not exceeding 6 times the depth	6
ends held in position and both edges firmly held in line	7

Table 3.09. Maximum depth to width ratios (solid and laminated members)

As seen from the above, the determining parameter is not h/b, but $\ell h/b^2$, and rules of the kind mentioned are hard to defend, as they are not always on the safe side, not even in cases normally occurring.

3.5. Deflections

The deflection calculations present no problem; but the problems arise in connection with determination of the acceptable deformations. The estimate of these, however, is outside the scope of the present report. The matter is dealt with by ISO/TC98/SC4, which has issued proposals for an ISO-recommendation.

There is a difference, however, between the codes of the various countries. Some of them state the pure modulus of elasticity in bending, while others state a reduced valueto allow for the fact that apart from bending deformations, which for beams are normally the only ones directly calculated, there are also shear deformations. Typically, the reduction is about 10%.

As the moduli of elasticity are also to be used for other purposes, where it is important to know the values as correctly as possible, such a procedure seems unfortunate. The codes should rather state that the requirements apply to the sum of bending and shear deformations, and that the latter - if more exact calculations are not desired or necessitated by special conditions - may be calculated to 10% (or what is thought reasonable) of the bending deformations, cf. section 4.5, where the value is estimated.

4. SOLID TIMBER, SHEAR AND BEARING

4.1. Rectangular cross-section

Generally, the ordinary theory of elasticity is used, i.e. in bending about one of the principal axes (here the Y-axis, see fig. 3.1) with the shear force $V = V_z$ it is assumed that the shear stresses τ vary parabolically with the maximum value

$$\tau_{max} = \frac{3}{2}\frac{V}{bh} \tag{4.01}$$

and

$$\tau_{max} < f_v \tag{4.02}$$

is required, where f_v is the shear strength.

In fact it should be taken into account when determining τ that the bending stress distribution, cf. fig. 3.01, deviates from the linear distribution, but normally this is not done.

4.2. Rectangular cross-section, size effect

For a long time North Americal timber codes, see e.g. [5] and [7], have used a modified shear force V′, which is less than the true shear force V, as seen from the statical calculations. A corresponding rule has now also been included in the Norwegian Timber Code [12].

The basis for this modification is an article by Newlin, Heck & March [37] from 1934, where the so-called »two-beam action» was suggested for the first time.

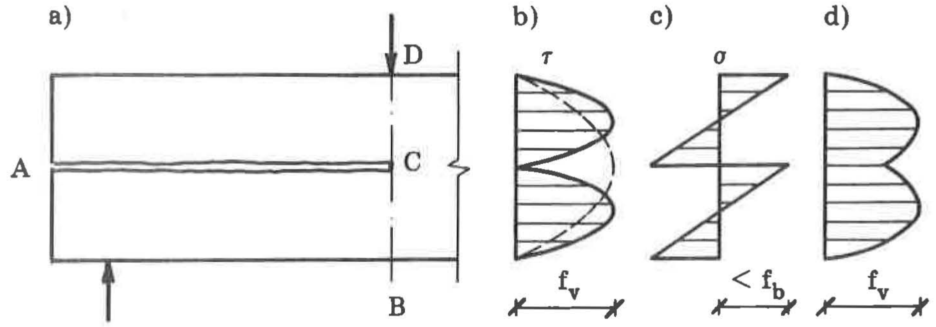


Fig. 4.01. In b) - - - denotes stress distribution in an unchecked beam, and —— in a checked beam.

Shear failure along the middle shear plane in a beam need not cause total collapse. The shear capacity of the two beams, into which the original beam has been split, is not necessarily less than in the unchecked beam, and provided the bending strength is not exceeded in the plane denoted BCD in fig. 4.01, the rupture may theoretically be stopped (in practice, however, it will be difficult owing to the dynamic effects of the rupture).

Newlin et al now assumed that the beam could check partly and that the original beam, when the part-beams were fully utilized, still could absorb part of the shear force, corresponding to the stress distribution as shown in fig. 4.01 d. The assumption was theoretically and experimentally supported. On the basis of this, the following principal rules are given in North America, cf. fig. 4.02:

- Loads applied closer to the support than the beam depth h are not taken into account.
- The following influence line is used for the shear force

$$\frac{V'}{P} = \frac{10}{9}\left(1 - \frac{x}{\ell}\right)\frac{\left(\frac{x}{h}\right)^2}{2 + \left(\frac{x}{h}\right)^2} \qquad (4.03)$$

  instead of

$$\frac{V}{P} = 1 - \frac{x}{\ell} \qquad (4.04)$$

  which results in a substantial reduction of the effect of forces acting on the outermost quarter of the beam, cf. fig. 4.02.

- For uniformly distributed load q

$$V = q\left(\frac{\ell}{2} - h\right) \qquad (4.05)$$

  is assumed in accordance with the first rule.

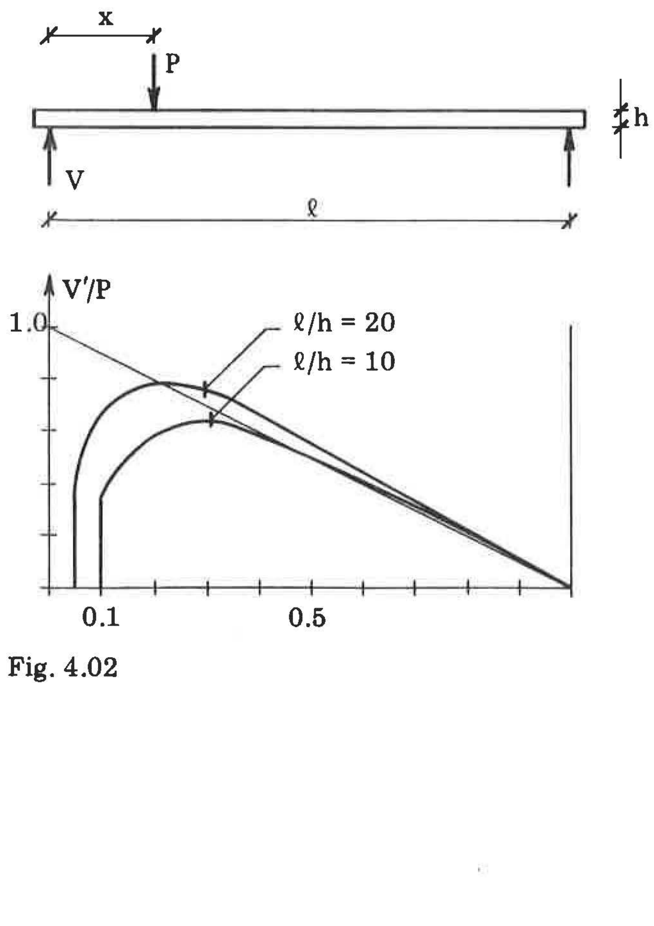


Fig. 4.02

It seems incredible that this theory has been able to remain unchallenged for 40 years and that it has been admitted in otherwise respectable codes. It makes no sense that checks - either caused by drying or a beginning, but halted rupture - should improve the shear capacity. And if common sense is not adequate, the theoretical derivations might have been looked into, which would have shown that they are based on senseless geometric assumptions along the check from A to C. It is of course quite unacceptable that the results have been applied also to glulam, when it is certain that the producer will be sued if the beams are delivered with the checks which form the background of the theory.

Keenan [31] has evaluated a large number of shear tests. As a start it was investigated whether the increased shear strength, which is indisputably found when the load is close to the support, might be caused by friction from the transverse compression stresses. This, however, had to be rejected. Then it was investigated by the finite element method whether the shear stress distribution assumed by Newlin et al might occur. This was found not to be the case, which is quite obvious, as continuity was assumed in the calculations, while it is a discontinuity phenomenon which should be investigated.

Finally, the connection between the shear strength f_v and sheared area A_v, defined as

$$A_v = bx \tag{4.06}$$

where b is the width of the beam and x the distance to the nearest force, cf. fig. 4.02, was investigated.

Fig. 4.03 (taken from Keenan) shows the results of a large number of tests with shear in bending, shear in torsion (hollow tubes) and block shear. Not taking the latter into account, the following regression line is found:

$$f_v = 19.20 - 3.03 \log_{10} A_v \quad \text{N/mm}^2, \; A_v \text{ in mm}^2 \tag{4.07}$$

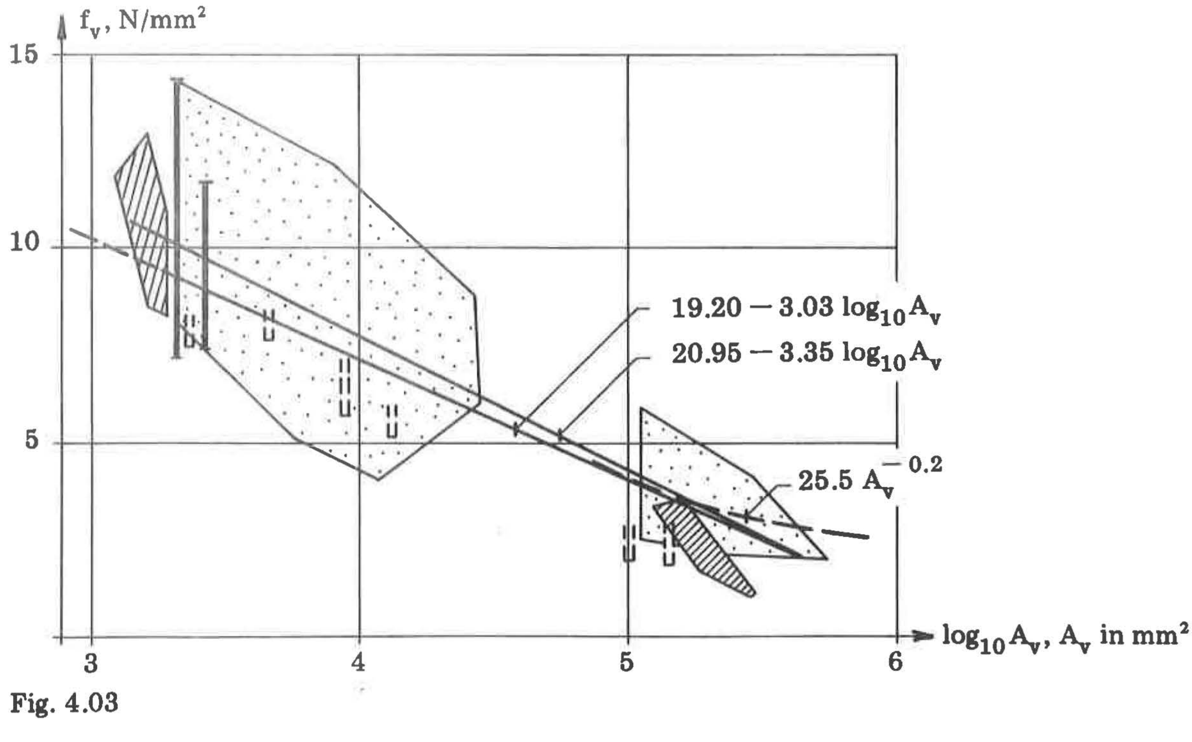


Fig. 4.03

A_v is the sheared area in mm^2. The framed values show the variation for large test series with varying A_v-values, while vertical lines give the deviation for small, identical test series. Dotted vertical lines correspond to tests where only the low values are recorded.

If the shear block tests are also taken into account, the following expression is found:

$$f_v = 20.95 - 3.35 \log_{10} A_v \quad N/mm^2, \quad A_v \text{ in } mm^2 \tag{4.08}$$

which only deviates a little from (4.07). This is in fact peculiar, as the stress distribution in the block tests is quite different from the others.

If Weibull's rupture theory for brittle materials is assumed to apply, it is found that the ratio between the strength parameters σ_1 and σ_0 related to the volumes V_1 and V_0 is given by

$$\frac{\sigma_1}{\sigma_0} = \left(\frac{V_0}{V_1}\right)^{\frac{1}{k}} \tag{4.09}$$

where k is the shape factor. For tension perpendicular to the grain Barrett, Foschi & Fox [15] have found k ~ 5, and assuming the same value to apply to shear and further, that the volume and the sheared area are proportional, we find

$$\frac{f_v}{f_{v0}} \sim \left(\frac{A_0}{A_v}\right)^{0.20} \tag{4.10}$$

Choosing arbitrarily $A_0 = 10^4$ mm^2 as an initial point, where the strength according to (4.07) is about 4.0 N/mm^2, we find

$$f_v = 4\left(\frac{10^4}{A_v}\right)^{0.2} = 25.5\, A_v^{-0.2} \tag{4.11}$$

which is shown in fig. 4.03. The difference between (4.07) and (4.11) is negligible.

If it is desired to take into account the influence of the sheared area in the codes it might be done as follows:

The shear strengths given in the code apply to sheared areas A_v greater than or equal to $2 \cdot 10^5$ mm^2 ~ 0.2 m^2. If A_v is smaller, the values can be multiplied by the factors given in table 4.04.

A_v, m^2	⩾ 0.20	0.18	0.16	0.14	0.12
factor	1.00	1.02	1.04	1.07	1.11
A_v, m^2	0.10	0.08	0.06	0.04	⩽0.02
factor	1.15	1.20	1.27	1.40	1.58

Table 4.04. Increase factors for shear strength.

As the shear strength is seldom decisive, it is doubtful whether it is worth-while introducing such a complication.

4.3. Other cross-sections and bending in two planes

Usually the formulas of the ordinary theory of elasticity are applied, to which is referred.

4.4. Notches

Notches cause a weakening further to that caused by the area reduction. This is due to the stress concentrations which particularly occur when tensile stresses perpendicular to the grain are produced.

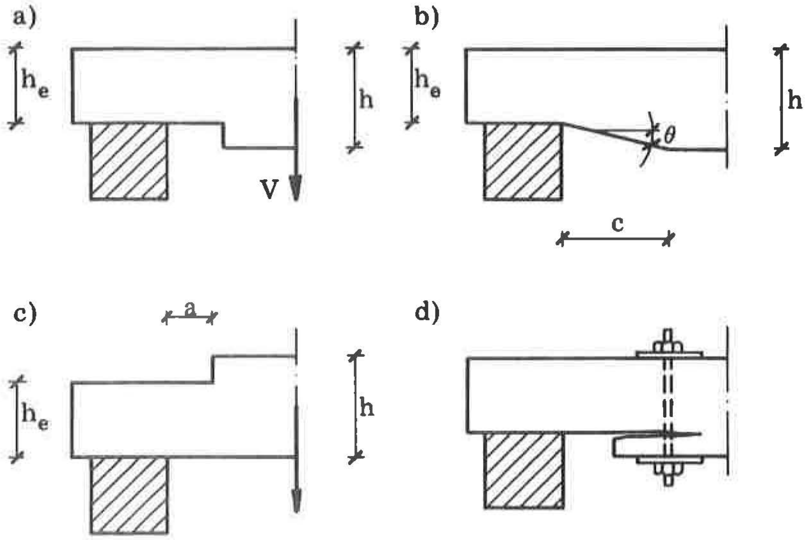


Fig. 4.05

For the case in fig. 4.04 a, i.e. with notches in the bottom side, most codes state that notch effect and tension perpendicular to the grain can be taken into account by reducing the load-carrying capacity calculated on the effective depth h_e by the notch-factor $\alpha_n = h_e/h$, i.e.

$$\frac{3}{2}\frac{V}{bh_e}\frac{h}{h_e} < f_v \tag{4.12}$$

is required (for rectangular beams).

The influence from the notch can be reduced by cutting obliquely as shown in fig. 4.05 b. By cutting flatter than corresponding to $\theta = 30°$, the Norwegian Timber Code states that the reduction need not be taken into account (apart from calculating the shear stresses on the reduced depth h_e). Moreover, for $90° \geqslant \theta \geqslant 30$, the Norwegian Timber Code gives the following interpolation formula:

$$\alpha_n = \frac{h_e}{h} + \frac{\sqrt{3}}{3}\frac{c}{h} \tag{4.13}$$

The Swedish Timber Code gives

$$\alpha_n = \frac{h_e}{h}\left(1 + \frac{c}{3h_e}\right) \tag{4.14}$$

for $c \leqslant 3(h - h_e)$, i.e. correspondong to $\theta \geqslant$ about 20°.

In a few codes it is stated that the total load-carrying capacity, i.e. without the factor h_e/h can be obtained by fastening the parts for example with bolts, cf. fig. 4.05 d. The value of this must be considered questionable, even if it should be possible to find a method to keep the bolts tight.

The origins of eq. (4.12) are lost in the dark, but are presumably attributable to the Forest Products Laboratory, whose tests, according to [20] form the basis for the expressions given in the North American, British and Norwegian codes for notches in the upper side, cf. fig. 4.05 c.

In conformity with these, the load-carrying capacity calculated from the reduced depth ($V = \frac{2}{3} f_v b h_e$) is corrected by the factor

$$\alpha_n = \frac{h}{h_e} - \frac{a}{h_e}\left(\frac{h}{h_e} - 1\right) \qquad (4.15)$$

The Timber Construction Manual and the British Timber Code assume that $h_e/h > 0.6$. It is not clear whether this is a constructive assumption or an assumption for the equation; in other USA-rules, however, cf. for example [38], it is stated that the equation only applies to $h_e/h > 0.6$ and $a/h < 1$. Otherwise $\alpha_n = 1$. In the Norwegian version the limits are set as $h_e/h > 0.6$ and $a/h < 0.6$.

The expression (4.15) has been drawn in fig. 4.06. It seems difficult to find much sense in it with the discontinuities for $h_e/h = 0.6$ and for $a/h = 1$, where there is a jump to $\alpha = 1.0$ in certain of the rules.

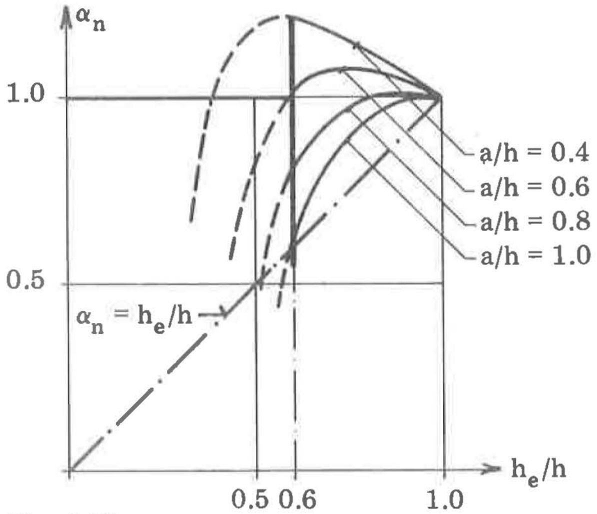


Fig. 4.06

Why a is introduced into the equations can also be hard to understand immediately, as an effect in the corner of the notch and not a reaction effect is dealt with.

The reason for the special Norwegian limitation - $a/h < 0.6$ - is that otherwise we would obtain $\alpha_n < 1$, which might strike some as exactly what should be possible. Here, however, there is conformity with the Canadian rules, giving a modification factor greater than or equal to 1, i.e.

$$\alpha_n = \frac{h}{h_e} - \frac{a}{h_e}\left(1 - \frac{h_e}{h}\right) \qquad (4.16)$$

for $a/h < 1$, otherwise $\alpha_n = 1$ is assumed. The expression has been drawn in fig. 4.07.

As the background material is very inadequate and the effect probably modest, the most reasonable thing to do is to put $\alpha_n = 1$, i.e. - on the safe side (?) - just allow for the reduced depth.

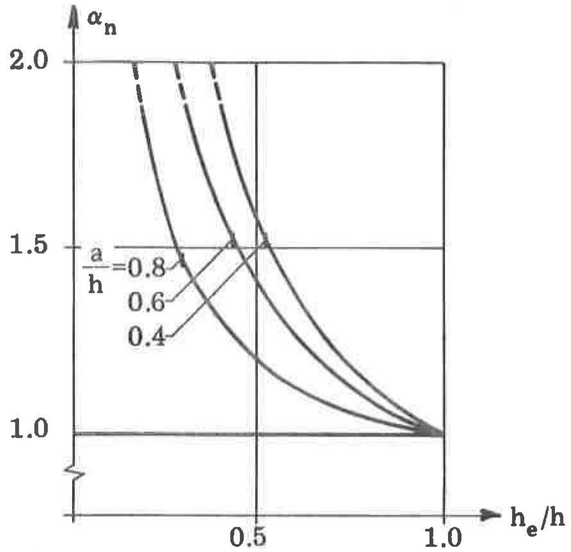


Fig. 4.07

4.5. Deflections

The shear deflections w_{shear} are calculated by the ordinary theory of elasticity, and for a simply supported beam, cf. fig. 4.08, they are:

$$w_{shear} = \alpha_w \frac{M}{GA} \tag{4.17}$$

and for a fixed-end beam

$$w_{shear} = \alpha_w \frac{M - M_0}{GA} \tag{4.18}$$

where M is the bending moment, M_0 the fixed-end moment, G the shear modulus, A the cross-sectional area of the beam and α_w a factor dependent upon the shape of the cross-section.

For a rectangle, α_w = 1.20, for a circle, α_w = 32/27 ~ 1.20.

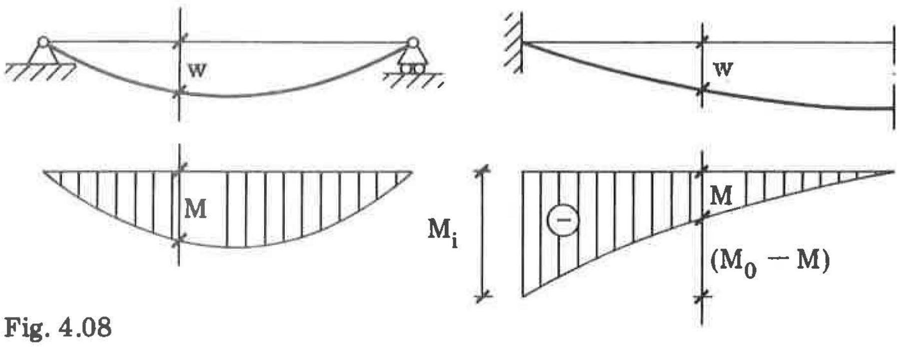


Fig. 4.08

As E/G ~ 20, the ratio between the contributions from shear and bending for a simply supported, rectangular beam with a concentrated force in the centre will be:

$$w_{shear}/w_{bending} \sim 24 \left(\frac{h}{\ell}\right)^2 \tag{4.19}$$

Only when $\ell/h <$ about 16, will w_{shear} constitute more than 10% of the total deflection.

4.6. Bearing

The bearing strength perpendicular to the grain, $f_\perp$, depends upon the loaded length. The reason is that for a short loaded length there is a relatively considerable contribution from the fact that the fibres are not only to be compressed but also cut through. Furthermore, there might be an arch effect.

Backsell [14], among others, has investigated the variation of the strength with the length and found the following connection:

$$\frac{f_\perp}{f_{0\perp}} \sim \left(\frac{a_0}{a}\right)^{0.4} \tag{4.20}$$

where $f_\perp$ is the strength, when the loading length is a, while $f_{0\perp}$ is the strength for a reference length a_0.

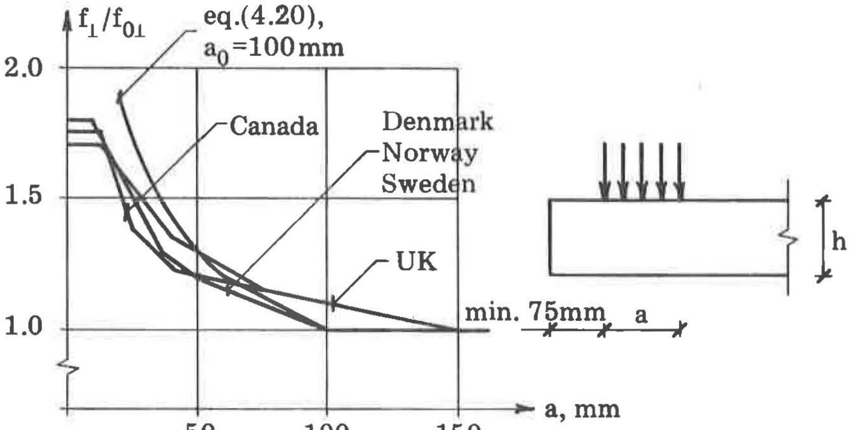
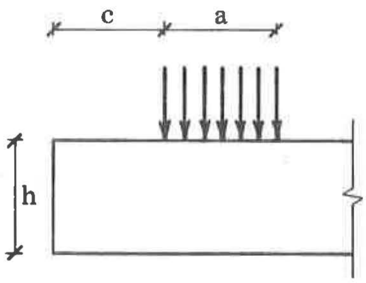


Fig. 4.09

The expression is drawn in fig. 4.09 with $a_0 = 100$ mm chosen as reference. In this figure the rules stated in the codes of a number of countries have also been drawn. The condition in all cases is that the unloaded length to the end is at least 75 mm (and in certain cases also at least 1.5 h). It is seen that there is a good correspondence both mutually and also with the theory.



$f_\perp/f_{0\perp}$

a/h \ c/h	⩾ 1.5	1	0.5	0
1	2	1.5	1.25	1
2	1.5	1.25	1.12	1
⩾ 3	1	1	1	1

Fig. 4.10

In France a slightly different rule is applied, as stated in fig. 4.10. Backsell's tests do not support the assumed strong dependence on h; it is rather the absolute distance which is decisive.

5. GLULAM

5.1. Bending and shear

In this section only horizontally laminated beams are dealt with, i.e. the moment vector in bending is parallel to the glued joints.

For straight beams of constant depth the conditions mentioned in sections 3 and 4 apply; however, size effect and lateral instability, for example, might be of greater importance than for solid timber.

In many cases the strength parameters have been determined from quite modest tests with representative beams, and the theoretical considerations limited to what was necessary in order to be able to extend the test results to, for example, other combinations of lamella qualities than those directly investigated. Most frequently the results are given as modification factors to the normal strength and stiffness parameters for boards of the same or similar quality as used for the lamellas. As an example some values from different timber codes are given in table 5.01. The deviation in the factors for f_b according to the French rules is due to the fact that for glulam the depth factor (between 0.8 and 1.0) which is otherwise applied to beams with h ⩾ 150 mm, may be disregarded.

	Nordic countries			Germany	Netherlands		France
timber quality	T40	T30	T20		C*	S*	
bending, f_b	1.2	1.3	1.4	1.1	1.2	1.4	1.1 - 1.4
compression ∥, f_c	1.2	1.3	1.4	1.0	1.2	1.2	1.1
tension ∥, f_t	1.2	1.3	1.4	1.0	1.2	1.8	1.1
shear, f_v	1.2	1.2	1.2	1.3	1.0	1.0	1.0
MOE, E	1.2	1.2	1.2	1.1	1.1	1.1	1.0

* C = Constructiehout, S = Standardbouwhout

Table 5.01. Modification factor for glulam

In North America and England the strength and stiffness parameters have been determined on the basis of the so-called I_k/I_g-theory. According to this theory it is assumed that the reduction in relation to perfect timber is solely dependent upon the I_k/I_g-ratio, where I_g is the total geometric moment of inertia of the investigated cross-section, while I_k is the contribution to I_g from the area of the knots assumed to belong to the cross-section (this is normally assumed to be the knots within 150 mm (6″) of either side of the section). The theory has been further dealt with by Freas & Selbo [27] and especially by Curry [22], who verified the theory by experiments and determined the statistical distribution function for I_k/I_g and thus the strength variation for different cross-section set-ups and number and quality of lamellas.

The theory has been somewhat attacked, because, among other things, it does not take into account the facts that knots in the tensile side are much more significant than those in the compressive side, that knots in the outermost lamella have greater influence than assumed in the theory, and that the effect of knots in the outermost tensile lamella is not only dependent upon the size but also upon the location. Moody & Bohannan [34] and [18] have thus shown that, generally, the quality of the outer lamellas of the test beams for verification of the I_k/I_g theory was above average - at any rate in the American tests - and that an evaluation of the strength of glulam should be based on a combination of the I_k/I_g ratio and the quality of the outer lamellas.

These objections, however, do not question the soundness of the rules given in the British Timber Code on the basis of [22], but they do create doubt whether it is possible to apply the theory directly to cases not covered by the tests.

According to the British Timber Code the strength and stiffness parameters vary with the number of lamellas, the depth of the beam and the quality of the joints. If, for example, the bending strength of a beam with ten 22 mm-lamellas is put arbitrarily at 100, the strength parameters will vary as shown in table 5.02. Plain scarf joints 1 : 10 or corresponding finger joints are assumed.

lamella thickness		22 mm			33 mm		
lamella quality		LA	LB	LC	LA	LB	LC
number of	4	100	88	75	100	88	75
lamellas	5	100	94	83	100	94	83
	10	100	100	100	100	98	98
	15	100	102	107	100	94	99
	20	100	98	104	100	92	98
	30	100	95	104	100	91	101
	50	99	92	104	98	91	102
	100	98	93	108	96	93	107

Table 5.02. Relative bending strengths for glulam

When the extreme case with just 4 lamellas is disregarded, the variations do not seem large or logical enough to justify the very complicated rules. If a more realistic joint quality is used, the variation will be even smaller.

Most of the material forming the background of code rules concerning the strength of glulam must now be considered obsolete; it does not correspond to present-day production technique and raw material supply. Most important in this connection is the fact that in almost all cases the code values are based on tests with beams having full-length lamellas without joints or with plain scarf joints, while to-day's lamellas without exception are finger-jointed. In far too many cases these have proved to have a definitely impairing influence, even in cases where there has been no objection against the joint quality in general.

However, a condition for a substantial, renewed effort to make more up-to-date and uniform rules is that production standards, desired lamella qualities, etc. are harmonized.

5.2. Special conditions

Owing to the production methods or the kinds of construction made possible by glulam a number of strength or design problems arise, which do not occur in connection with ordinary timber structures.

In the following, such problems as appear reasonable to mention in a timber code are dealt with, i.e. problems which are not to be found in the usual text-books on Structural Mechanics. It is obvious that to some extent it will be a question of estimation.

a. Strength reduction in curved members

In the manufacture of curved members considerable bending stresses arise in the individual lamellas, often of the same order of magnitude as the bending strength. To minimize the risk of rupture during production and to ensure satisfactory assembling a lower bound for the ratio r/t between the radius of curvature, r, and the lamella thickness, t, are normally given in the production regulations. This bound varies from country to country owing to tradition and differences in timber properties; a normal requirement, however, is $r/t \geqslant 125 - 150$. The bending of the lamellas results in a reduction of the bending strength of the member, although it is a surprisingly modest reduction. It has been investigated by Wilson [41] and Hudson [30], and their results are given in fig. 5.03.

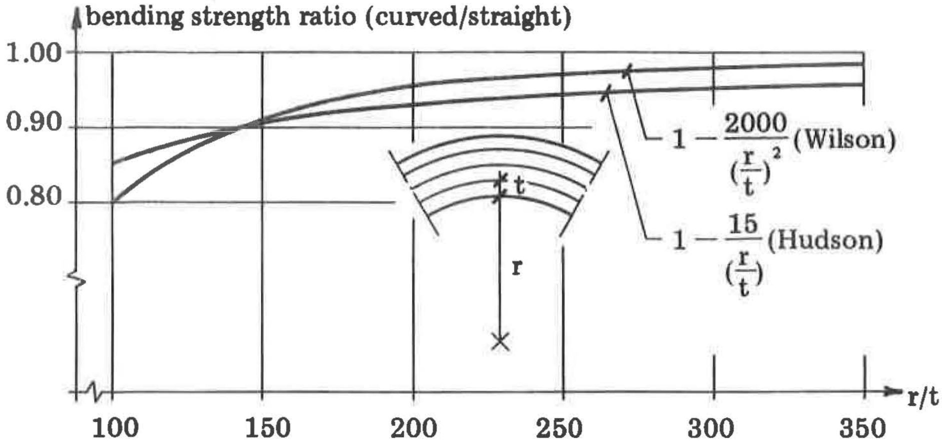


Fig. 5.03

In the area of interest the difference is insignificant on the background of the scanty test material which is available. Certain codes give Wilson's expressions, others the lowest value. To avoid discussion on whether a cambered beam must be considered a curved beam, the curves in fig. 5.03 should be »lifted» so that the value for e.g. $r/t = 300$ is put to 1.0. The German Timber Code contains no reduction rule, as Möhler [36] states that the reduction in the actual area is insignificant according to the American tests and could not be documented in his own tests.

b. Stress distribution in curved members

In bending the stress distribution deviates from the one occurring in a straight beam, the stresses in the inner side being larger and in the outer side smaller than in a straight beam. Generally speaking, the reason is that there is a relatively smaller length in the inner side to absorb a given deformation, and a relatively greater length in the outer side. For a rectangular beam the stresses in the inner side may be written as

$$\sigma_i = \alpha_i \frac{6M}{bh^2} \qquad (5.01)$$

where α_i is a function of the ratio r_m/h, where r_m is the average radius and h the depth of the cross-section.

α_i is shown in fig. 5.04 both for an isotropic and for an anisotropic material (with $E_{\parallel}/E_{\perp} = 6$). Furthermore, the figure shows the approximation expression

$$\alpha_i = 1 + \frac{h}{2r_m} \qquad (5.02)$$

as suggested by Möhler.

To avoid discussions on when to allow for the curvature, the expression should perhaps be modified or limited to $r_m/h < 10 - 15$.

c. Lateral stresses in curved members

In curved beams subjected to bending radial stresses occur. These can, with approximation for both isotropic and anisotropic material, see for example [36], be written as

$$\sigma_{\perp} = 1.5\, M/(r_m bh) \qquad (5.03)$$

where the notation is seen from fig. 5.04.

When the moment tends to increase the radius of curvature (direction of bending as shown in fig. 5.04), tensile stresses occur and only this case is of interest.

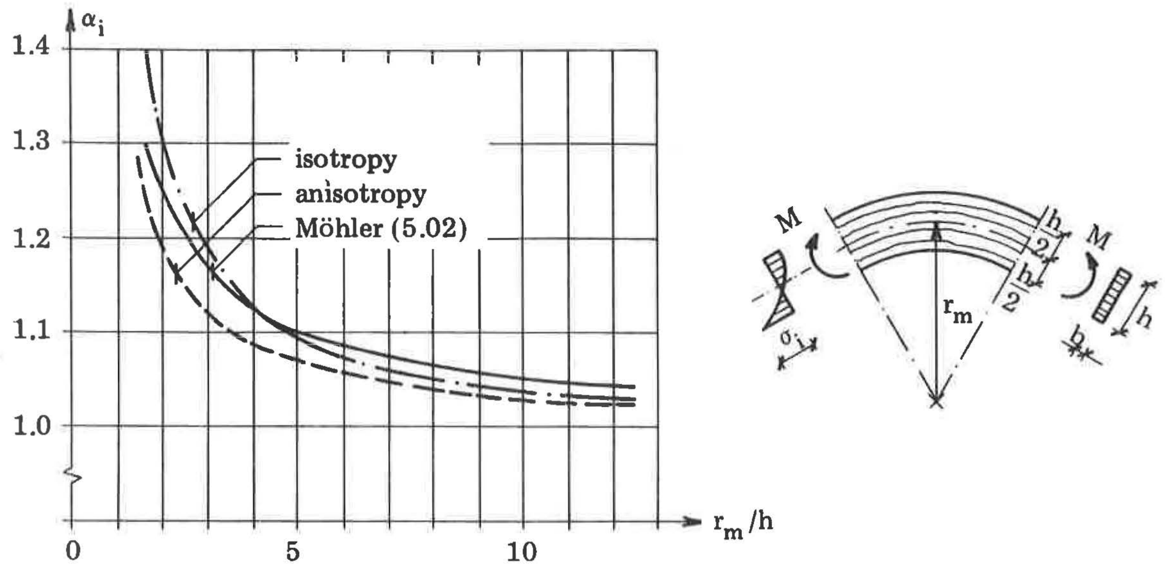


Fig. 5.04

d. Stresses in pitched cambered beams

The use of (5.03) unconditionally requires a uniform cross-section in the entire curved part. If the depth of the cross-section varies, considerably larger stresses may occur.

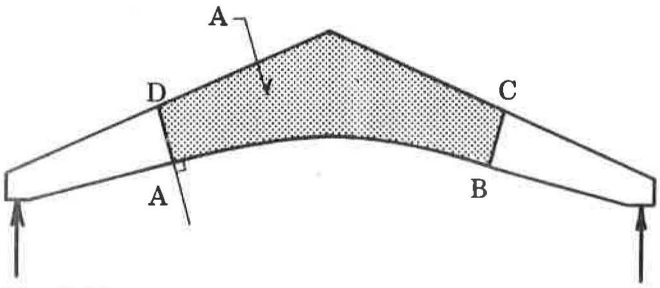


Fig. 5.05

Of practical importance are pitched cambered beams, see fig. 5.05, where there are both large lateral tensile stresses near the apex (ridge), and a bending stress distribution which is quite different from ordinary beams.

The case has been looked into by, among others, Möhler and Foschi & Fox [25], [26], and the conditions are so clarified that they can be incorporated into clear code rules, see for example the Canadian Timber Code [7].

It is not enough, however, to know the stresses occurring; the fact that the strength parameter for tension perpendicular to the grain is highly dependent upon the volume must also be taken into account.

According to investigations by Barret, Foschi & Fox [15] the perpendicular tensile strength $f_t(V)$ of a structure with the volume (V) may be assumed to be

$$f_t(V) = f_t(1) \cdot V^{-0.20} \tag{5.04}$$

where $f_t(1)$ is the strength of a unit volume, cf. eq. (4.10). The stress distributions are assumed to be the same.

In the structure shown in fig. 5.05 the lateral tensile stresses occur particularly in the marked zone ABCD the area of which is denoted A. If it is assumed that the stress distribution in different beams is rather similar, the volume Ab (b = width of beam) may be used as a parameter, i.e. the strength can be assumed proportional to $(Ab)^{-0.20}$.

As Ab for normal beam shapes may vary with a factor of about 100 the strength may thus vary with a factor of about 2.5. As the lateral tensile strength is often decisive for such structures, this effect should be taken into account.

e. Tapered straight beams

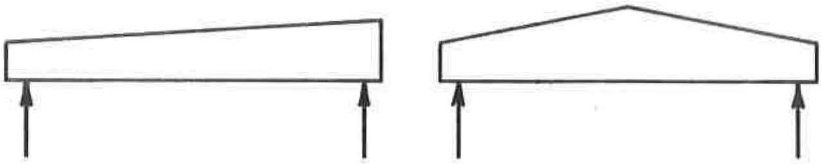


Fig. 5.06

For tapered straight beams, see fig. 5.06, there are a large number of special phenomena, as for example:

- the maximum bending stresses do not occur in the mid-section,
- the maximum bending stresses in the upper side occur on sections perpendicular to the upper surface,
- stresses perpendicular to grain occur at the upper side of the beam,
- shear stresses occur on vertical sections in the upper side, and they may often be larger than in the gravity axis,
- the contribution to the deflections caused by shear stresses may be large.

These are, however, effects of a general nature which are outside those for which rules should be given in a timber code. One problem should perhaps be resolved, namely which inter-action formula to use in the points on the upper side of the beam, where axial stresses both perpendicular and parallel to the grain ($\sigma_{\parallel}$ and $\sigma_{\perp}$) and also shear stresses (τ) occur.

The Timber Construction Manual [5] uses the requirement

$$(\sigma_{\parallel}/f_{b\parallel})^2 + (\sigma_{\perp}/f_{b\perp})^2 + (\tau/f_v)^2 \leq 1 \qquad (5.04)$$

where f is the strength parameters belonging to the stresses.

The basis for application of this equation to timber, however, seems very slender.

6. GLUED BEAMS AND PANELS

This section will deal with I- and box beams and flat stressed-skin panels as shown in fig. 6.01. For these the buckling stability of the panels is often decisive, and therefore a general introductory paragraph on this subject is given, followed by a more detailed discussion of the two types of structure.

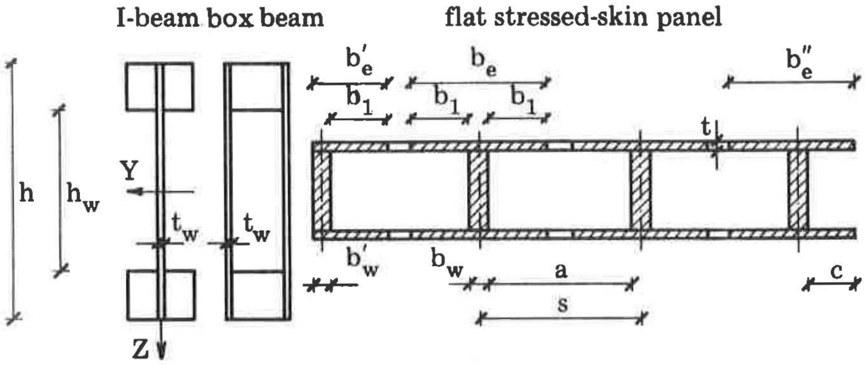


Fig. 6.01

6.1. Buckling stability

The following description of the buckling phenomena aims at the relatively coarse methods that might be given in a code, and therefore, a large number of simplifications have been made, mostly on the safe side.

A panel as shown in fig. 6.02 is considered. The panel is assumed simply supported along all 4 sides and loaded either with stresses σ independent of x in sections parallel to the Z-axis or with constant shear stresses τ.

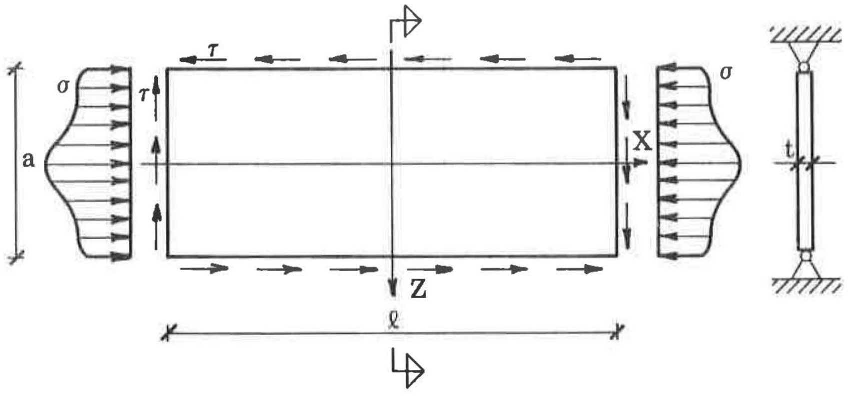


Fig. 6.02

The following notations are used:

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

$(EI)_z$ $(EI)_x$, but in bending about the Z-axis.

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

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

β_2 $= 0.5\,(GI)_v/\sqrt{(EI)_x(EI)_z}$. For an isotropic panel, $\beta_2 = 2G/E$. As $0 \leqslant G/E \leqslant 0.5$, $0 \leqslant \beta \leqslant 1$ thus applies in this case.

According to the theory of elasticity, which Halasz & Cziesielski [28] and the very comprehensive reports from Forest Products Laboratory, Madison [1] and [3], have proved can also reasonably be used for wood-based materials, the critical value of σ is found as

$$\sigma_{crit} = \alpha \frac{\pi^2 \sqrt{(EI)_x (EI)_z}}{ta^2} \tag{6.01}$$

and the critical value of τ as

$$\tau_{crit} = \alpha \frac{\pi^2 \sqrt[4]{(EI)_z (EI)_x^3}}{ta^2} \tag{6.02}$$

where α is a factor which is dependent upon the stress distribution and the parameters β_1 and β_2, cf. fig. 6.03, where α is given for the most common cases. Other cases are dealt with in [1], [2], and [28]. The expressions only apply as long as the proportional limit is not exceeded, but is often used until failure. In the cases a) and b) this can be justified by the fact that only part of the cross-section is highly stressed, while the bending stiffness in the case d) is only insignificantly reduced owing to τ. However, it would be correct - as in the case of the other stability phenomena - to introduce a suitable transition curve.

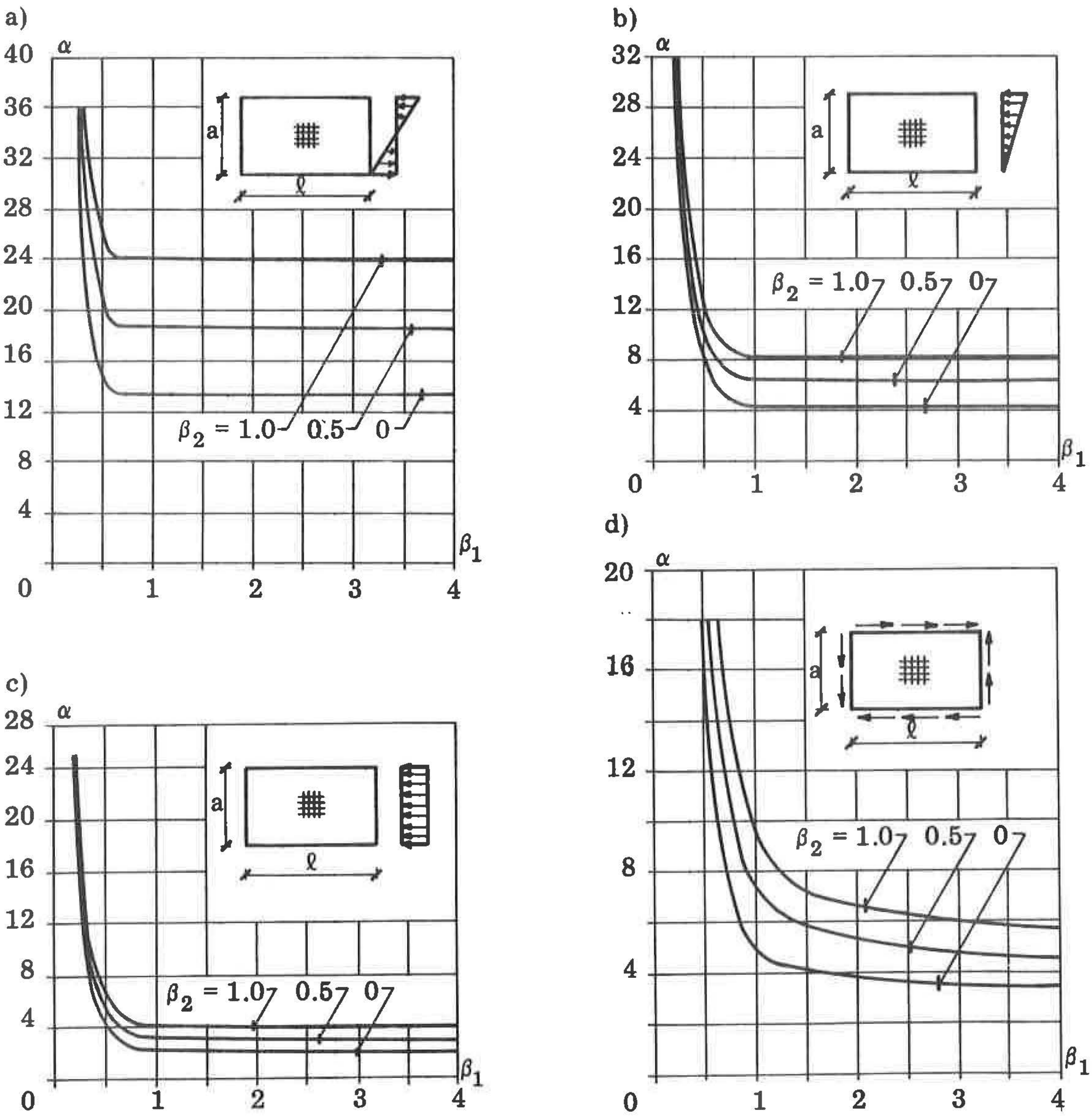


Fig. 6.03

Moreover, a slightly reduced safety is often used for this buckling phenomena, as a favourable redistribution of the stresses occurs in buckling, so that further load can be applied before collapse.

6.2. I- and box beams

The calculation of the stresses and deflections for these is carried out according to general principles, the fact that web and flange might have different strength and stiffness properties being taken into consideration. For very thin webs the redistribution of stresses owing to the shear deformations may be allowed for. Of special interest, therefore, in connection with a timber code are form factors, rules for lateral buckling of the beam and rules for buckling of the web plate. Concerning form factors, refer to section 3.3.

Regarding lateral buckling it is possible to extend the rules given in section 3.4 to apply to I-beams as well (box beams are directly covered by the rules) with the slenderness ratio also a function of the warping resistance of the beam.

In practice, however, it is fully sufficient - and slightly on the safe side - to consider the compressive flange as a column with a free length corresponding to the outer restraints.

Rules of the type given in the British and Canadian timber codes, where the requirements for lateral support are given solely as a function of the ratio I_y/I_z, seem unacceptable, as they do not allow for the essential parameters. (As an example, a beam is required to be braced at 8 ft intervals - irrespective of the dimensions of the beam - provided I_y/I_z ranges between 30 and 40).

For the web plate, calculations according to section 6.1 show that the load-carrying capacity will not be limited by buckling, as long as $h_w < 40\ t_w$. If $h_w > 40\ t_w$, a further investigation must be carried out. The limit $40\ t_w$ is calculated from rather unfavourable assumptions concerning the ratio $(EI)_x/(EI)_z$, and considerably larger plate heights can often be allowed.

6.3. Flat stressed-skin panels

For notations, refer to fig. 6.01.

The behaviour of this structure is very complex, and even a tolerably exact analysis requires the use of computer.

In practice, however, it will be fully satisfactory and on the safe side as shown by Booth in [19] to consider the structure as a number of I-beams, each of which is to carry the load imposed on the width, s, see fig. 6.04.

The width of the panels, b_1, which can be taken into account to either side is less than a/2, where a is the free spacing between the webs. The reason is that the stresses, σ, in the flanges will vary, in principle as shown in fig. 6.04. Therefore, the calculations are carried out with a fictitious cross-section with an effective flange width, b_e, where $b_e \leq s$.

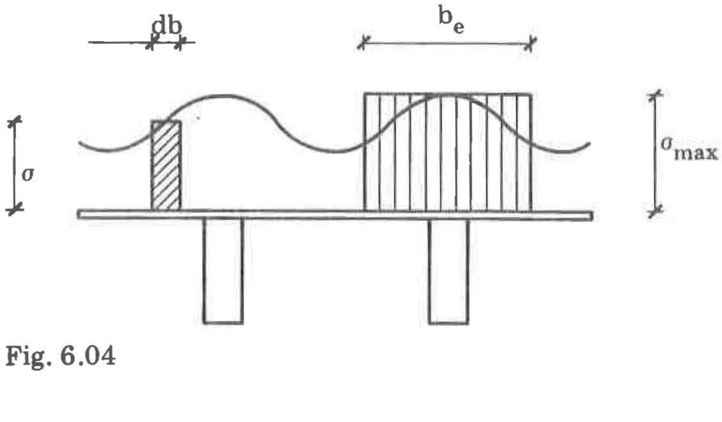


Fig. 6.04

b_e is defined by

$$\sigma_{max}\, b_e = \int_s \sigma db \qquad (6.03)$$

b_e depends on a number of factors, among others the span, ℓ, the panel thickness, the bending stiffness of the panel in both directions and its torsional stiffness. Most important, however, are ℓ/a and E/G.

With a sinusoidal moment curve with n half-waves ($M = M_0 \sin \frac{n\pi x}{\ell}$) Möhler, Abdel-Sayed & Ehlbeck [35] have calculated the effective panel width for a simply supported panel as shown in fig. 6.05.

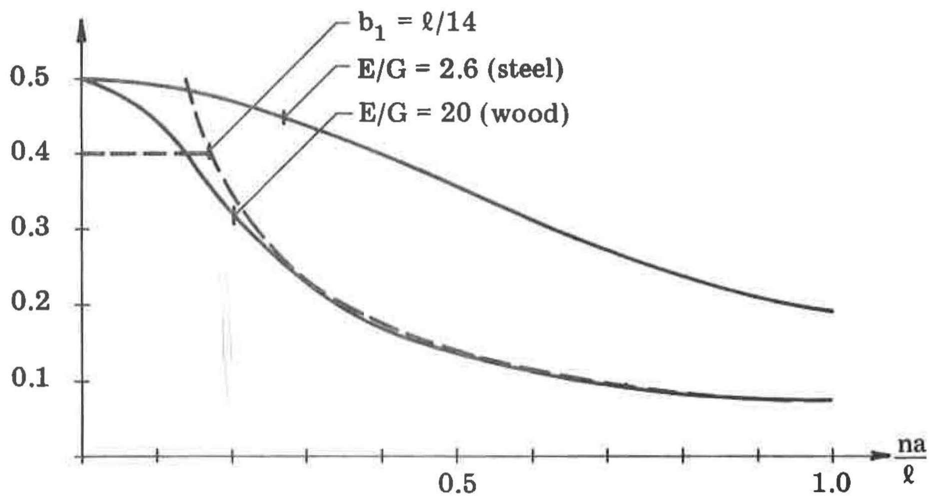


Fig. 6.05

For a uniformly distributed load n = 1 can be used, and it is seen that a reasonable value is $b_1 = \ell/14$, however, $b_1 < 0.4$ a, i.e.

$$b_e = \min \begin{cases} \ell/7 + b_w \\ 0.8\,b + b_w \end{cases} \qquad (6.05)$$

and

$$b'_e = \min \begin{cases} \ell/14 + b_w \\ 0.4\,b + b_w \end{cases} \qquad (6.06)$$

On the basis of the same investigations the following can be assumed for a panel with an overhang, c:

$$b''_e = \min \begin{cases} \ell/14 + b_w + c \\ 0.6\,b \end{cases} \qquad (6.07)$$

In the case of concentrated load in the middle it is necessary to transform the moment curve by a Fourier series and the effective width for each term can be calculated with the aid of fig. 6.05. As b_1 is reduced for $n > 1$ the average width for the total load is smaller than for a uniformly distributed load. For a concentrated load $b_1 = \ell/20$ can be used with sufficient accuracy, the expressions (6.05) - (6.07) being modified accordingly.

For the flange panels calculations according to section 6.1 show that as long as $a < 30$ t, the load-carrying capacity will not be reduced by buckling. For larger values further investigations must be carried out; considerably larger widths may often be permissible.

In the USA and Canada the effective width for plywood is determined as follows:

- the effective width, b_e, for plywood with n plies is given by

$$b_e = \begin{cases} 31t\sqrt{t/t_{\parallel}} & \text{for } n = 3 \\ 36t\sqrt{t/t_{\parallel}} & \text{for } n \geq 5 \end{cases} \tag{6.08}$$

  where t is the thickness of the plywood and $t_{\parallel}$ the thickness of the plies parallel to the span. Of course b_e cannot be greater than s,
- the free spacing between the ribs should not exceed $2b_e$,
- the permissible compressive stress, f_c, in relation to the permissible compression strength of the panel, f_{co}, is determined as shown in fig. 6.06.

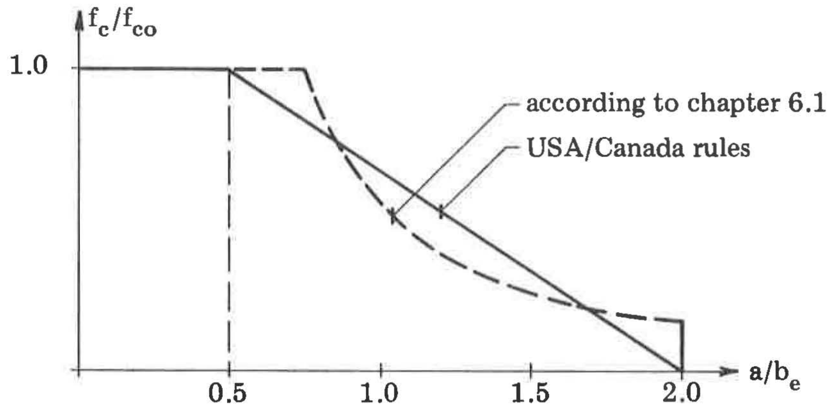


Fig. 6.06

These rules are given in numerous publications in addition to the official codes, unfortunately, however, in all cases without references.

Immediately, eq. (6.08) appears hard to understand, as the effective width increases the more unbalanced the plywood is, which is contrary to the theoretical results, but on the whole the results are reasonable.

Thus the rule in (6.08) gives $b_e \sim 40t - 50t$, i.e. no reduction is required for $b_e < 20t - 25t$. In fig. 6.06 the strength reduction has been drawn according to the rules in section 6.1 combined with the buckling problems being estimated to begin at a = 30 t - 40 t. Apparently the rules are thus based solely on buckling considerations, and it is not taken into account that the effective width is less than the spacing. It seems difficult to extend the results to other board materials.

For more exact - but still manageable - calculation methods for this type of structure, see for example Foschi [23] and [24]. Furthermore, Kuenzi & Zahn [32] is referred to.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CLIMATE GRADING FOR THE CODE OF PRACTICE

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KARLSRUHE - OCTOBER 1975

CLIMATE GRADING FOR THE CODE OF PRACTICE

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General

By climate is here meant temperature and humidity and their variations. It is, thus, only a part of the conception environment. The climate may influence the loads acting on the structure as well as the load carrying capacity of the structure. When dealing with the influence on the loads, one can often do without a climate grading in the Code of Practice simply by giving different loads for different geographic zones. This belongs to the loading code.

Climate influence giving deformations (swelling, shrinkage, thermal strain) and, thereby, often stress, is generally treated as load, but may also be referred to as an influence on behaviour and resistance of the structure. The influence of moisture content on elasticity and strength is obviously an example of influence on resistance to be considered in the Code for Timber Structures. Other examples are such time-dependent factors as corrosion and ageing which decrease strength or hardening which increases it. Creep and corresponding change of strength are also influenced by the climate.

Climate grading is principally of interest for its effect on the resistance of material and structures and is generally believed to belong to the Code of Practice for specified materials, such as the Code for Design and Construction of Timber Structures. This is acceptable, provided that the grading is harmonized between the codes for different materials.

Basis of grading

There are principally three methods of defining climate in the present codes of practice:

1. Directly by temperature (T) and relative humidity (RH).
2. Indirectly by the moisture content (MC) of the material.
3. Indirectly by grouping structures and structural parts.

Only the first alternative is an unambigous definition of the climate itself. It can simultaneously be made a basis for a testing standard. The disadvantage is that the designer will have difficulties in applying a grading based on temperature and relative humidity. If referred to as a regulation, it must anyway be supplied with recommendations, e.g. in accordance with alternative 3.

In the second alternative the climate is defined by an effect - the (equilibrium) moisture content. This has been the most common method, no doubt because the strength of wood is depending on the MC and the aim has been to give working stresses for different climates. In some codes, there has merely been defined a value of MC, e.g. $\psi = 0.18$, to separate "dry" from "wet".

By giving MC-values, the climate is, of course, not defined exactly in terms of temperature and relative humidity. The correlation between RH and MC is depending on temperature and not quite the same after desorption as after adsorption. It differs between wood species and products such as plywood, particle board and wood fibre board.

If - as an acknowledgement of tradition - one choses to keep the definitions of climate by moisture content, one ought at least to refer the MC-values to a specified (standard) wood species. Anyway, the problem of application remains and the code must give advice as to what structures that are supposed to get a moisture content in the material which is equivalent to the grade-value.

The conclusion is that the code must include some kind of grouping of buildings and building parts with respect to climate, but that a fundamental grading of the climate, preferably in terms of temperature and relative humidity, serves an important purpose for the designers' own judgement (e.g. for objects or conditions which are not reckoned in the official grouping) and as referendum at testing.

Elementary climate classes

The variation of temperature at normal service of wood in buildings and structures has a limited influence on the moisture content. In an elementary grading, one could, therefore, possibly refer to the relative humidity at constant temperature. Here the temperature 23^{o} C is suggested with reference to a proposed standard climate at testing. The relative humidity can be denoted RH(23) or $\phi(23)$. It is understood that these RH values can be transformed to give equivalent MC in the reference wood at other temperatures.

The restriction to one temperature limits the possibility to consider various climates but special classes of climate can be added to the elementary classes, should this be necessary. The elementary grading is suggested principally to be based on the influence of moisture content on deformation and strength of wood and wood products as well as on a sensible grouping of the most important structures. It is then natural to the author to suggest something along the guide-lines issued by the Nordic Building Regulations Committee (Guide-line XII, NKB report No.18 Dec, 1973). The climate classes are shown in table 1 as they appear in the Norwegian standard (NS 3470, Oct. 1973).

The most important feature is Class 1, which better corresponds to an interior climate than does the previous "MC 0.18 or lower". Such a dry climate was suggested for codes many years ago but was rejected as unnecessary with reference to the not very big influence of moisture content on the properties of wood. The approving of wood fibre board and particle board as structural materials, however, changed the situation.

Due to the substantial creep in these materials at increased moisture content, one could no longer defend a "dry class" which included such a high value of MC in wood as 0.18.

TABLE 1 Elementary climate grading (NS 3470)

Climate class	1	2	3
Relative humidity (ϕ) [1]	<=0.65	0.65<RH<=0.85	>0.85
Approx. moisture content (ψ) (pine, spruce)	<=0.12	0.12<ψ<=0.20	>0.20

1) During a short time (a few successive days) the upper limits of ϕ may be exceeded by 0.10.

The value ψ = 0.20 can be regarded as rounded off from 0.18. The tolerance given by the foot note to the table implies that, at least in thin panels, the MC may reach 0.20 or even slightly higher values (at 23^{o} C). The tolerance applied on dry class 1 in the same way may increase the MC from 0.12 to 0.14. The classes 1 and 2 put together give a climate grade comparative to the previous climate grade, defined by $\psi \leq$ 0.18, although the Nordic grouping of objects for class 2 might be looked upon as too liberal to fit into the old "dry" grade.

Supplementary climate grading

Sometimes there is a need for special climate classes. An example is the climate class denoted T1 in the NKB recommendations mentioned (class 0 in the Swedish code). It is a special case of class 1 in table 1 by a stipulation that the mean value of RH during one year must not increase 0.40. The introduction of this special class of climate made it possible to increase the fictive elasticity and rigidity moduli (in which creep is considered) by 30 % for particle and fibre board at a majority of interior applications.

For other reasons, e.g. durability of glued joints, it may be desirable to separate a special grade at the wet end of the scale. One might also, with respect to creep and glued joints, base special climate grades on variations in RH. Thus, it is generally desirable that the number of special climates, defined in the Code of Practice, is limited.

This can be achieved if the tests on which strength and elasticity values are based are carried out in such climate, possibly cycled, that unfavourable cases within the elementary climate grades are reasonably well simulated.

Grouping of objects

It appears from present codes that the grouping of constructions as an indirect definition of climate can be subject to international harmonizing, let be not in all details. The German standard DIN 1052 gives the following groups including the corresponding moisture content of wood (the groups are numbered here, not in the DIN):

1a.	Buildings, closed on all sides and heated	(9 ± 3) %
1b.	Buildings, closed on all sides without heating	(12 ± 3) %
2.	Open but covered constructions	(15 ± 3) %
3.	Constructions, exposed to weather	(≥ 18) %
4a.	Scaffolding	
4b.	Sub-water constructions	

The code, which deals with wood and plywood, not other panels, does not make any difference between stresses allowed in classes 1a, 1b and 2. The aim of the division into heated and non-heated buildings is obviously to give a background for estimating shrinkage and for chosing adhesives for gluing. The German climate grading could as well have been used to differentiate working stresses. Anyway, it coincides rather well with the Nordic grouping which has this purpose:

T1 Structures in permanently heated buildings without air conditioning.

1 Roof structures in ventilated, cold spaces on top of a (permanently) heated building. External walls, inside a well ventilated wall covering.

2 Constructions in ventilated but not permanently heated buildings, e.g. recreation houses, garages and store houses, crawl spaces, roof panels, scaffoldings and concrete formwork.

3 Other constructions (sub-water constructions, etc.)

A close examination shows that this grouping is not in complete agreement with the guide-values of relative humidity in table 1. Thus, in beams over crawl spaces and in scaffoldings and concrete forms, the moisture content may well - and not only during a couple of days - be higher than corresponding to a RH of 0.85. The reason for the grouping in class 2 was that they are temporary constructions. This could instead (and preferably) have been considered by a general increase of the allowed stresses.

The difference in moisture content in roof trusses in non-heated and indirectly heated ("in spaces on top of a heated building") spaces is that the last mentioned will have a lower moisture content than the non-heated during the winter when the design loads (snow loads) can be expected. In the summer-time when there is no heating, there will of course be no difference.

Testing climate

The trend that performance is regulated rather than material and design of the product will also have influence on the testing standards. One consequence ought to be an intensified coordination of the work of ISO-committees which is now too much tied to specific materials. The standard climates at testing should, as close as possible, be referred to the climates to which the building regulations and Code of Practice refer their performance stipulations.

ISO R554 has established three testing climates, see table 2. The climate 20^{o} C, RH 65 % (20/65), given as a reference climate is often referred to as the "normal climate". In revising the standard (by ISO/TC125), this reference climate is supposed to be replaced by 23^{o} C, RH 50 % (23/50). This somewhat dryer climate, which has advantages as a testing climate, can be accepted as a standard climate at verification of the strength properties to be applied in climate class 1 (table 1). A reservation with respect to the suggested tolerances must be made, though. If the standard condition at testing as suggested is stipulated as $(23\pm2)^{o}/(50\pm5)$ % RH, testing at 21/45 is allowed, corresponding to a moisture content of 0.083. This is, maybe, somewhat dry compared with the "normal" upper limit in class 1 ψ = 0.12. Anyway, authorities responsible for the Code will hardly approve of the results from testing in the reference climate 23/50 as verification of properties for a climate 20/85 (climate class 2), neither for a climate "$\psi \leq$ 0.18". Naturally, one can use transforming factors, but these are bound to be on the conservative side.

The Code of Practice for Timber Structures, or for approvals of materials and structures in connection with it, therefore require at least one testing climate at higher RH. If 23/50 is accepted for verification to climate class 1 it seems reasonable to choose 23/80, corresponding to ψ = 0.16, for class 2. An alternative would be 23/65 for class 1 and 23/85 (ψ = 0.18) for class 2. The 23/80 for testing has the advantage that it gives an MC not much deviating from the 0.15 at which structural timber and joints as a rule are tested when the intention is to derive stresses for the now common climate class "$\psi \leq$ 0.18".

Some views on testing in cycled climate can be added. Climate variations give alternating swelling and shrinkage, which can generate forces and decrease the resistance of structural elements, such as glued joints. The alternating adsorption and desorption in itself increases the creep in stressed wood and wood products. Standard routines for creep testing of wood products have so far not been much discussed. As for climate cycling, a starting point could be the levels between which the variation should take place. Here is suggested that the levels 23/30, 23/50 and 23/80 are chosen for a standard, possibly with a supplementing 23/90. The 23/30 should be compared to the levels 20/33 and 25/40 suggested by ISO-committees for wood fibre board and particle board respectively to be used in testing dimensional stability.

The amplitude should generally be from the level, related to the climate class as a "normal upper limit", to the next lower level. Of course it can be increased to cover the three or four levels suggested if desirable for a certain reason.

Also the frequency of the cycling should be standardized. A period of two days to one week is here mentioned merely as a starting point for further discussion.

TABLE 2 Climate at testing

	Temperature, ^{o}C	Relative humidity (RH), %
ISO R 554	20	65
	27	65
	23	50
Here suggested	23	80 For climate class 2
	23	50 For climate class 1
	23	30 Limit at cycling

CIB-W18/5-100-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION - TIMBER STRUCTURES

DESIGN OF SOLID TIMBER COLUMNS

1ST DRAFT FOR CIB TIMBER CODE

KARLSRUHE - OCTOBER 1975

1. SCOPE

A simply supported column loaded by a central axial force and a lateral load resulting in deflection along one of the main axes of the cross-section (in the figure the z-axis) is dealt with.

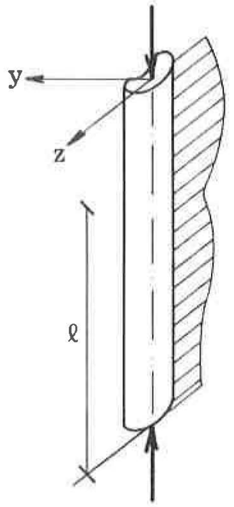


Figure 1

2. NOTATIONS

See figs. 1 and 2.

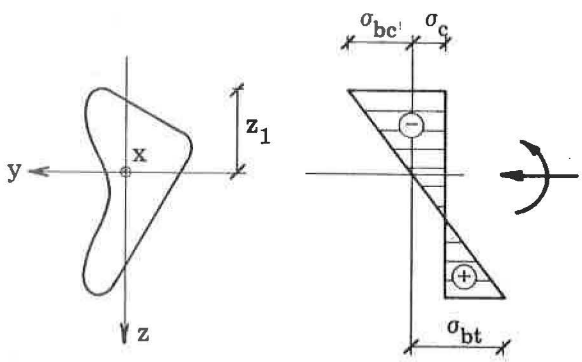


Figure 2. y and z are the main axes of the cross-section.

A	Cross-sectional area
E	Modulus of elasticity
$I = I_y$	Moment of inertia about the y-axis
$W_c = I_y/z_c$	Section modulus for the outermost fibre in compression
e	Eccentricity
f_b	Strength in bending
f_c	Strength in compression
$i = \sqrt{I/A}$	Radius of gyration
k_1, k_2	Constants, see section 3
ℓ	Free length
$k_E = \frac{\pi^2 E}{\lambda^2 f_c}$	Constant
z_c	Distance from centroid to outermost fibre in compression
$\lambda = \ell/i$	Slenderness ratio
σ_b	Bending stress
σ_{bc}, σ_{bt}	Bending stresses (numerical values) in the outermost fibres in compression or tension, respectively, due to the lateral load and calculated according to the theory of elasticity
σ_c	Compressive stress (numerical value)

3. ASSUMPTIONS

The column is assumed to have initial curvature corresponding to the axial force in the centre of the column having an eccentricity e, expressed as

$$e = (k_1 + k_2\lambda)\frac{I}{Az_c} \qquad (1)$$

where $I/(Az_c)$ is the core radius corresponding to the compressive side.

If exact values are not known, the following can be used for the European softwoods allowed for structural use

$$k_1 = 0{,}1 \qquad k_2 = 0{,}005 \qquad (2)$$

The bending moment is assumed to vary from zero at the ends to a maximum value at the middle and is assumed not to exceed the value corresponding to a sinusoidal or parabolic variation. By moment distributions not satisfying these conditions, the criterion in section 4 may be applied, if the bending stresses are increased,for constant moment for example (corresponding to eccentric axial force) by 10%.

If I_y is the greatest moment of inertia of the cross-section, the column is assumed to be secured against deflection in the y-direction so that the failure will occur by deflection along the z-axis and not by lateral and torsional deflection.

4. DESIGN CRITERIONS

The permissible combinations of stresses from the axial force and the moment are given by the following expressions:

Compressive side:

$$2\left(1 - k_1 \frac{f_c}{f_b}\right) \frac{\sigma_c}{f_c} \leqslant 1 + \left(1 + k_2 \lambda \frac{f_c}{f_b}\right) k_E$$

$$- \sqrt{\left(1 + \left(1 + k_2 \lambda \frac{f_c}{f_b}\right) k_E\right)^2 - 4\left(1 - \frac{\sigma_{bc}}{f_b}\right)\left(1 - k_1 \frac{f_c}{f_b}\right) k_E} \qquad (3)$$

Tensile side:

$$\frac{\sigma_{bt}}{f_b} \leqslant \left(1 + \frac{\sigma_c}{f_b}\right)\left(1 - \frac{\sigma_c}{k_E f_c}\right) - (k_1 + k_2 \lambda) \frac{\sigma_c}{f_b} \qquad (4)$$

5. LIMITATION OF λ

For primary and secondary structural members λ should not exceed 170 and 200, respectively.

6. EXTENSION OF THE AREA OF APPLICATION

The stated method of dimensioning may also be applied (with approximation) to other kinds of support, if their influence on the critical length is allowed for in the usual manner.

7. REFERENCES

The background for the method are given in the following reports prepared for CIB-W18:

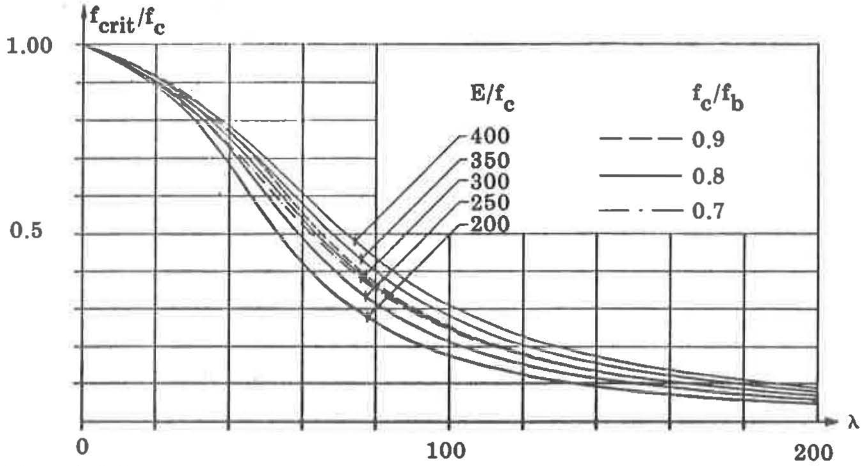
H. J. Larsen: The Design of Solid Timber Columns. Aalborg University Center, Pure and Applied Mechanics, Report R7406, 1974.

H. J. Larsen and S. S. Pedersen: Tests with Centrally Loaded Timber Columns. Report R7405, 1974 - as above.

APPENDIX 1

Load-bearing curves for columns loaded with a central axial force.

Load-bearing curves for different values of E/s_c and f_c/f_b are given in fig. A 1.1. σ_{crit} is the value of σ_c corresponding to the equality sign in eq. (3) with $\sigma_{bc} = 0$.

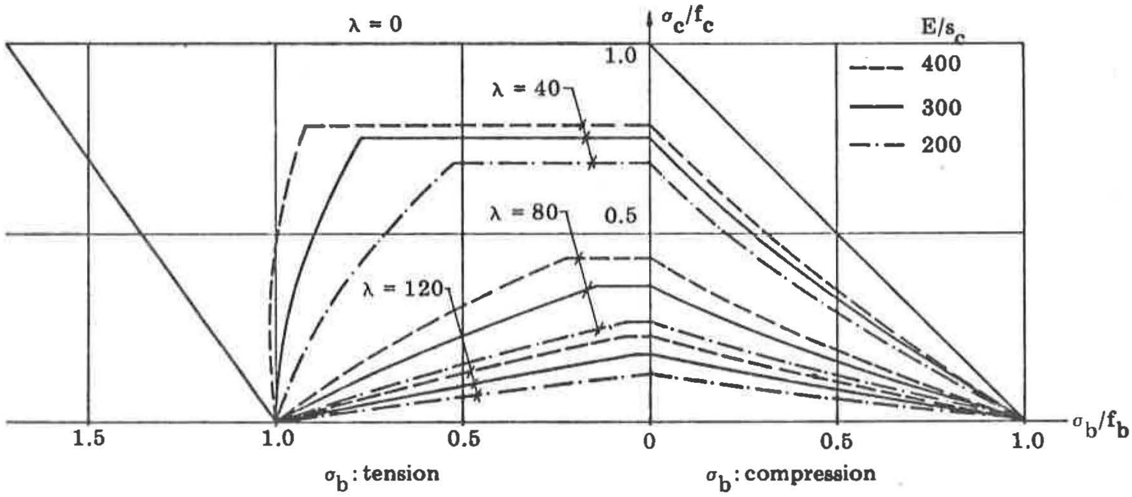


Figur A 1.1

APPENDIX 2

Load-bearing curves for columns loaded with a central axial force and a lateral load. The diagram gives related values of σ_b/f_b and σ_c/f_c corresponding to the equality sign in eq. (3) or (4).

In the diagram $f_c/f_b = 0.8$ has been assumed.



Figur A 2.1

CIB-W18/5-100-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

A DRAFT OUTLINE OF A CODE FOR TIMBER STRUCTURES

by

L G BOOTH

Department of Civil Engineering

Imperial College, London

KARLSRUHE - OCTOBER 1975

A CODE FOR TIMBER STRUCTURES

At the last meeting of W18 the Chairman reported that discussions had taken place with CEB/FIP with the aim of producing a Code for Timber Structures which will link with similar codes for concrete and steel.

It has been suggested that Volume I of the unified code would be common to all the materials and would contain the principles of limit state design, partial safety factors for actions and details of the actions to be resisted by structural components. A draft of Volume I was considered by CEB in Lisbon in May 1975. A copy of the contents list of this draft is attached as Appendix A.

A first draft of Volume II (the Concrete Code) was considered by CEB in Lisbon in May 1975 and a second draft has been prepared for their next meeting in Helsinki in September 1975. Both drafts are written partly in French and partly in English. An English contents list of the first draft is attached as Appendix A of this paper (the contents of the second draft are similar). Many of the clauses have not been finalised and at this stage the intention of some of the outline clauses is not clear.

The attached draft outline for the Timber Code (Volume X) has been prepared with the aim of following the pattern and clause numbering of the Concrete Code as much as possible. In some parts the pattern was inappropriate and the Timber Code has a different form. For some of the clauses the same headings have been used as for the Concrete Code where it appeared that the outline in the Concrete Code was also applicable to timber, but at this stage it is not completely clear what is the intention of the concrete code. From these brief remarks it will be appreciated that the outline is very tentative and as individuals begin to prepare draft clauses it is likely that the overall pattern will need to be changed.

L G BOOTH
August 1975

VOLUME X

CODE FOR TIMBER STRUCTURES

PRELIMINARY OUTLINE

1 General requirements

1.0 Introduction

1 Scope

2 General design requirements: safety, serviceability and durability

3 Specific notations for timber structures

4 Units

5 Documentation and records

1.5.1 Calculations

2 Drawings

3 Diary

2 General design data

2.1 Data for timber

2.1.1 General

2 Species of timber

3 Grading

2.1.3.1 Visual

2 Mechanical

4 Method of test

5 Method of determining characteristic stresses

6 Characteristic stresses

2.1.6.1 Clear timber

2 Graded timber

7 Partial safety factors (γ_m)

2.1.7.1 General

2 Duration of load

3 Moisture content

4 Size

8 Geometrical properties of sections

2.2 Data for laminated timber

2.2.1 General

2 Species of timber

3 Grading

4 Partial safety factors (γ_m)

2.2.4.1 General

2 Duration of load

3 Moisture content

4 Size

5 Number of laminations

2.3 Data for plywood

2.3.1 General

2 Species of plywood

3 Grading

4 Method of test

5 Method of determining characteristic strengths

6 Characteristic strengths

7 Partial safety factors (γ_m)

2.3.7.1 General

2 Duration of load

3 Moisture content

4 Size

2.4 Data for other panel products

(Clauses similar to 2.3)

2.5 Data for fasteners

2.5.1 General

2 Types of fastener

2.5.2.1 Nails

2 Screws

3 Bolts

4 Other mechanical fasteners

5 Glued

5.1 Scarf

5.2 Finger

- 3.4.1.1 General
  - 2 Solid
  - 3 Laminated
    - 3.1 Glued
    - 3.2 Mechanical
  - 4 Plywood box and I beams
  - 5 Diagonally boarded
- 2 Columns
  - 3.4.2.1 General
    - 2 Solid
    - 3 Laminated
      - 3.1 Glued
      - 3.2 Mechanical
    - 4 Plywood box and I beams
    - 5 Diagonally boarded
    - 6 Spaced
- 3 Arches
- 4 Portals
- 5 Trusses
- 6 Plates
  - 3.4.6.1 General
    - 2 Plywood
    - 3 Stiffened plywood plates
      - 3.1 Single skin
      - 3.2 Double skin
    - 4 Layered boards
    - 5 Folded
- 7 Shells
- 8 Space frames

3.5 Specific problems related to particular joints

- 3.5.1 General
  - 2 Nailed
  - 3 Screwed
  - 4 Bolted
  - 5 Other mechanically fastened
  - 6 Glued

4 Construction

4.1 General

2 Machining

3 Joints

4 Assembly

5 Storage

6 Painting

7 Transportation

8 Erection

5 Maintenance

5.1 General

2 Maintenance

3 Inspection

4 Repair criteria

5 Repairs

L G BOOTH
August 1975

APPENDIX A - EXTRACT FROM CEB BULLETIN 109 APRIL 1975

Contents	Page	Sommaire

Volume II

Model Code for concrete structures

Volume II

Code de référence pour les structures en béton

Contents	Page	Sommaire

Contents	Page	Sommaire

CIB-W18/5-103-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMENTS ON THE REPORT OF THE CONSULTATION WITH MEMBER BODIES CONCERNING ISO/TS/P129 - TIMBER STRUCTURES

by

DANSK INGENIORFORENING

KARLSRUHE - OCTOBER 1975

COMMENTS ON THE REPORT OF THE CONSULTATION WITH MEMBER BODIES CONCERNING ISO/TS/P129 - TIMBER STRUCTURES

DANSK INGENIORFORENING

We agree that cooperation should be established with the organizations suggested by Austria and the Netherlands, but - in order to avoid confusion in the lines of cooperation - we suggest that such cooperation should be ensured by adding:

"The committee shall, to the extent such liaison is not already secured through the cooperation with CIB/W-18, establish liaison with relevant organizations working in this field, for example UN-ECE, CEB, RILEM and FEMIB/Sous-Commission "GLULAM"."

Regarding the scope proper - and with special reference to §3.1.2.1 of Directives for the technical work of ISO - we quite understand the Norwegian comment. Bearing in mind, however, the great number of already existing ISO/TC's working inside the field of Norway's proposed scope, we feel that it will be important to limit the scope of the new TC to the task of preparing the actual structural "code" supplemented only with the task of ensuring, that also the ancillary standards necessary for the application of that code are prepared or collected. "Code of practice" is indirectly defined in the scope. It seems to us indispensible for characterizing a standard of this type.

We have nothing against it being stated, that the work of the committee shall be based on the work of the ISO/TC's mentioned by France, but we find that special importance must be given to the cooperation with ISO/TC 98. We therefore find, that the present wording should be retained, but with the following addition after the mention of the specialized ISO/TC's: ", with which the necessary liaison must be established, to avoid duplication of work and to ensure the best distribution of the tasks involved. Relevant here are the TC's 55, 59, 89, 92, 99, 139 and 151."

We have understood the remarks from the United Kingdom and from USA regarding cooperation with CIB W-18 simply as un underlining of the intention of the proposal.

Regarding the exact limitation of the scope of work, this must be decided by the committee in accordance with § 3.1.2.1 of the directives, but it is our opinion, that specific structures of the type; telephone poles, together with special standards regarding topics such as fire protection and rot, lie outside an appropriate limitation - at least for the present - of the scope for the proposed new TC.

Based on the results of the consultation with the member bodies, we would propose the adjusted SCOPE FOR THE TECHNICAL COMMITTEE as follows:

Standardization in the field of Timber Structures with particular regard to the preparation of an International Standard for the structural design and construction of load-bearing timber structures.

The Standard (or standard code of practice), in the following denoted "the code", shall comprise the relevant technical requirements for the design and the work of construction, with appurtenant requirements regarding materials, components and connections.

The code shall be formulated in such a manner, that it gives the greatest freedom for design and construction compatible with satisfactory technical performance, durability and safety.

The code shall be supplemented with the necessary supporting standards, in particular regarding test methods necessary for the verification of stipulated requirements.

The work shall, where applicable, build on the work and results of existing ISO/TC's with which the necessary liaison must be established to avoid duplication of work and to ensure the best distribution of the work involved. Relevent in this respect are the present TC's 55, 59, 89, 92, 99, 139 and 151."

As far as basic and general principles are concerned, the work shall be based on the work and results of ISO/TC 98, Bases for the Design of Structures, with which committee a close liaison shall be maintained.

The main drafting work shall be entrusted to CIB W-18 which has agreed to act as a working group for the committee. Close liaison shall be ensured.

To the extent that this is not already secured through the cooperation with CIB W-18, liaison shall further be established with other relevant organizations working within the scope of the committee, for example UN-ECE, CEB, RILEM and FEMIB/Sous-Commission "GLULAM".

26 September 1975.

CIB-W18/5-105-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

THE WORK AND OBJECTIVES OF CIB W18 - TIMBER STRUCTURES

by

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KARLSRUHE - OCTOBER 1975

THE WORK AND OBJECTIVES OF CIB W18 - TIMBER STRUCTURES

by J G Sunley, Princes Risborough Laboratory, United Kingdom

The terms of reference of CIB W18 are:

> "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."

In carrying out this work I suggest we have two main roles:

1 As an independent group of timber engineering experts which should publish its own recommendations through CIB so the rest of the world is aware of our views and of recommended ways in dealing with timber in structural codes.

2 We should ensure that our recommendations are dealt with in a proper way by the appropriate organisations which wish to use them. We can I feel only ensure that our information is correctly used if we take part in the activities of these organisations.

In carrying out the second part of our work I have agreed that CIB W18 will liaise with other appropriate organisations concerned both with other materials and the publication of codes and standards.

There are a number of organisations involved in the harmonisation of codes and standards, etc, concerned with structural engineering. However, all of them appear to consider CIB W18 as the recognised authority on timber.

Described below are some of the activities of various organisations concerned with harmonisation.

The first initiative came from a CEB/CECM/FIP/CIB/IABSE Joint Committee on Structural Safety (JCSS) which with a strong lead from CEB is trying to draft a series of codes covering all materials. Volume I will contain information general to all materials. Other Volumes will deal with specific materials eg Volume II Concrete and Volume VI Timber. The first draft of Volume I is nearly complete. I am a member of an editing committee on Volume I and have also said that CIB W18 will supply Volume VI Timber to link with Volume I and the other material Volumes.

The Nordic Group NKB are taking part in the general activities of the above Group but have some differences of opinion on the principles involved and a number of meetings have been held between the Groups in an attempt to resolve these. However, members of the NKB Group have ensured that their views are discussed in other groups as well.

The JCSS at least as far as concrete is concerned have tried to form the necessary links with ISO. I understand that they have agreed that ISO TC 98 "Bases for Design" will only consider matters of principle and not specific details on particular materials. They will be responsible for processing the draft Volume I prepared by JCSS into an ISO document. Volume II on Concrete will be similarly processed by ISO TC 71 the Concrete Material Group. Volume VI Timber when produced would logically be processed by the new ISO TC on Timber (I think number TC 129, the Secretariat of which is held by Denmark).

Although there appears to be an agreement between ISO TC 98, TC 71 and JCSS recent papers for TC 98 meetings indicate that the Eastern European Secretariat of TC 98 interpret the general bases of design in much greater detail than was expected. For example, they have produced a document entitled "General Principles for the Verification of Safety of Wood Structures". This document is very specific in its detail (and not very good either). Similar documents have also been produced for Concrete and Steel.

Another two organisations are also involved in the harmonisation of structural codes namely EEC and ECE in Geneva (ECE covers the whole of Europe and North America). The involvement of EEC and ECE appears to have arisen through their activities in harmonisation of Building Regulations. Representation at EEC and ECE meetings tends to be at governmental level and frequently is non-technical.

As part of their activities the ECE Housing, Building and Planning Committee have commissioned reports on problems of harmonisation of structural codes on different materials.

In this context Jessome from the Canadian Forest Products Laboratory in Ottawa prepared a document giving views on problems in timber in this area. Generally I do not object to the conclusions of his paper which says in effect 'leave it to CIB W18'. However the contents of the paper were somewhat naive and even annoying in so far as it was recommending how CIB W18 should do its job and asking us to do things which we are already carrying out.

My own recommendations to the Group are that we should not get involved in the "politics" of the situation and that we should ensure that we remain the best technical forum in the subject of timber structural codes. I also recommend that we comment on the Volume I from JCSS and prepare Volume VI to link with this. We should support the sending of Volume I to ISO TC 98 for processing and ensure that our own Volume VI is sent to the new ISO Timber Group for processing. We should maintain our links with IUFRO Wood Engineering Group which I consider a research forum and RILEM to ensure adequate test methods exist for timber and timber products.

I would recommend retaining our existing format of 3 day meetings in which we discuss the background to codes in 2 days and restrict code drafting to the final day.