INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

# MEETING OF WORKING COMMISSION W18 - TIMBER STRUCTURES

CENTRE TECHNIQUE du BOIS

PARIS, FRANCE

19-21 FEBRUARY 1975

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| **BELGIUM** | |
| P Sonnemans | National Institute for Housing, Brussels |
| A Visser | Automated Building Components, Brussels |
| **DENMARK** | |
| M Johansen | Statens Byggeforskningsinstitut, Horsholm |
| H J Larsen | Instituttet for Bygningstekn, Aalborg |
| **FINLAND** | |
| U Saarelainen | Technical Research Centre of Finland, Helsinki |
| **FRANCE** | |
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| L O Leparsky | Central Research Institute for Building Structures, Moscow |
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| E Levin | TRADA, High Wycombe |
| W T Curry | BRE, Princes Risborough |
| (1) A P Mayo | BRE, Princes Risborough |
| (2) J G Sunley | BRE, Princes Risborough |
| R Marsh | Arup Associates, London |
| P O Reece | 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 fourth meeting of the reconstituted Timber Structures group. He outlined the programme for the three day meeting and thanked M Hochart of Centre Technique du Bois for his invitation to the group to see the research being carried out at the Centre.

MR SUNLEY said he believed that CIB-W18 had now established itself as an effective working group in the field of timber structures. This was being recognised by other international organisations who were requesting the group to undertake work, and make recommendations regarding the structural use of timber, to be included in both European and international standards and codes of practice. In addition MR SUNLEY suggested that there were some aspects of the work of the group which would be suitable for publication as official CIB recommendations and he hoped that when these publications were available the members of the group would try to get them adopted in their respective countries. Finally, as a further method of making the work of the group, better known he said that the members should consider presenting papers and articles at the various CIB conferences and seminars which were arranged from time to time.

## 3 REPORT ON CIB CONGRESS - OCTOBER 1974

The Sixth CIB Congress which was attended by MR SUNLEY was held in Budapest in October 1974 during which approximately 250 papers were presented in three days. MR SUNLEY said that because of the large number of papers the sessions were necessarily very formal and provided little opportunity for discussion, however it did provide a useful opportunity to meet others involved in building research. A list of the papers presented at the Congress was circulated to CIB-W18 members and MR SUNLEY offered to provide copies of any papers which were of particular interest to members.

The date of the next CIB Congress had been fixed for September 1977 and would be held in Edinburgh. MR SUNLEY said he thought that some of the Working Commissions might be encouraged to hold their own meetings during the Congress and W18 could be included.

## 4 CEB-CECM-CIB-FIP-IABSE JOINT COMMITTEE ON STRUCTURAL SAFETY

MR SUNLEY informed the delegates that since the last meeting of CIB-W18, when proposals by the "Comité Européen du Béton" (CEB) for a unified system of structural codes was discussed, a Joint Committee on Structural Safety (JCSS) had now been set up consisting of CEB,CIB,CECM (European Convention for Structural Steel work) FIP (Fédération Internationale de la Précontrainte) and IABSE (International Association for Bridge and Structural Engineering). The JCSS has adopted the objective originally proposed by CEB for a unified system of structural codes which would include all structural materials. MR SUNLEY said that he is a member of this Committee and would attend a meeting at the end of February when it was hoped to agree Volume I of the unified code. This first volume, which would be applicable to all materials, would lay down general rules covering the requirements of safety and serviceability, requirements for materials and components, and notations and units. Volumes II, III and IV would deal with concrete structures, steel structures and composite steel and concrete structures, respectively. Further volumes would deal with other materials, of which timber would be one, and CIB-W18 had been asked to draft this volume. When Volume I was agreed it would be circulated to all the groups drafting the remaining volumes for each material and it was proposed that when the draft of each material volume was agreed it would be submitted to the relevant ISO group for approval and status, eg "Volume II - Concrete Structures" would be submitted to ISO/TC 71.

## 5 RECENT ACTIVITIES OF ISO

MR SUNLEY said that for some time there had been a considerable body of opinion which thought that in order to produce an international standard for the design of timber structures it would be necessary to establish a new ISO committee. This had been supported at earlier meetings of CIB-W18 and the matter had been discussed at the British Standards Institution. Following a suggestion from BSI, the Danish Standards Institution made a proposal to ISO which was discussed at a meeting of Technical Division 3 - Building TD3 who agreed that, in addition to the work already being undertaken by ISO on concrete structures, similar work should also be started on timber and metal structures. Member bodies of ISO were therefore circulated with the Danish proposals for a new ISO/TC on timber structures (See Paper 1), and asked to complete a questionnaire indicating their interest in the proposals. MR SUNLEY said that BSI had indicated that they would participate in the work of the new ISO/TC if it was formed.

Prof LARSEN said Denmark was hopeful that a new ISO/TC for timber structures would be formed and they were prepared to undertake the secretariat. The work of the proposed ISO/TC would be based on the recommendations of the existing specialised ISO Technical Committees, such as ISO/TC 98 - "Bases for Design of Structures", but the proposals also recognised the value of the work of CIB-W18 and it would be necessary to establish a close liaison between the two groups. Prof LARSEN suggested that CIB-W18 members should contact their own national standards organisations to keep themselves informed of the activities of ISO and to win support for the proposed new ISO/TC.

MR SUNLEY said that CIB-W18 should submit their recommendations on the design of timber structures through Prof LARSEN to the Danish Standards Institution who would then pass them on to the new ISO/TC on timber structures. In addition he suggested that if, after the proposed ISO/TC was set up, further sub-committees were also set up to advise the main committee, different countries should undertake the secretariates of these various sub-committees in order to make the work as broadly based as possible.

Prof MOHLER said that if CIB-W18 is going to make recommendations to ISO it is important that each country is represented in CIB-W18 otherwise it is likely that any recommendations would be rejected by ISO because they would not be acceptable to those countries who were not represented on CIB-W18. MR CURRY disagreed with this as he thought there was a danger of CIB-W18 becoming too large to work effectively. He felt that CIB-W18 should work as a relatively small body of experts who would submit their recommendations to ISO where each country would be represented.

DR KUIPERS asked if the RILEM 3TT Committee should submit their work on test methods to the proposed new ISO/TC. MR SUNLEY replied that he did not think the proposed new ISO/TC would deal with test methods and there was already an established ISO/TC dealing with this work. However he went on to say that when test methods were agreed in CIB-W18 they could be published either as CIB or RILEM recommendations, whichever was the most effective at the time, and then submitted to the relevant ISO group for international status.

## 6 SYMBOLS FOR STRUCTURAL TIMBER DESIGN

Amendments to the CIB-W18 list of "Symbols for Structural Timber Design" (Paper 2) originally presented at the previous CIB-W18 meeting in Delft, June 1974, were discussed and agreed. It was noted that discussions were also taking place at the present time within ISO/TC 98 "Bases for Design of Structures", SC1 "Notations" and it was agreed that CIB-W18 should follow the recommendations of ISO where possible. However as the ISO recommendations did not cover all the symbols required for use in timber design it was decided to publish as a CIB recommendation, a supplementary list related to timber structures. It was further agreed that this CIB list would be sent to ISO with the request that it be included in the recommendations made by ISO/TC98/SC1.

In addition to the amendments previously agreed, DR KUIPERS acting on some recent information from Houtinstituut TNO, Delft and ISO/TC98/SC1, suggested that the following changes be made to the CIB-W18 list of symbols.

1 "RH" for relative humidity should be replaced by "$\varphi$" as this was already current international practice.

2 A symbol for equilbrium moisture content should be added to the list. This should be "$\omega_\varphi$".

3 "D" for density should be replace by "$\rho$" which would agree with the ISO recommendations.

4 In order to denote the various methods of expressing the density of timber, sufficies should be used to describe the conditions under which the mass and volume are measured.

ie $\rho_\omega$ refers to the mass and volume at moisture content $\omega$

$\rho_{0;\omega}$ refers to the mass at zero M/C and volume at M/C $\omega$

$\rho_0$ refers to the mass and volume at zero M/C

$\rho_{0;>35}$ refers to the mass at zero M/C and volume under green conditions

5 Coefficient of shrinkage should be denoted by "$\alpha$" and coefficient of swelling by " "

Items 1 to 4 were agreed for inclusion in the CIB list but it was decided that $-\beta$ should be used for coefficient of shrinkage and $+\beta$ for coefficient of swelling.

Finally it was agreed that MR MAYO (secretary CIB-W18) would compare the amended CIB list of symbols with the ISO/TC98/SC1 recommendations when they became available and, following agreement with DR NOREN and DR KUIPERS on any discrepancies between the lists, would arrange for publication of a CIB list of symbols for structure timber design. This would be included in the Code of Practice for timber structures and would also be sent to ISO/TC98/SC1 and ISO/TC129 (the proposed new ISO/TC for timber structures).

## 7 TIMBER COLUMNS

Prof LARSEN introduced his paper "Tests with Centrally Loaded Timber Columns" (Paper 3) which led a discussion on how eccentricity should be dealt with in the design of columns. It was agreed at an earlier meeting of the group that the Dutch method of column design had much to commend it and should form the basis of a CIB method. In this method the eccentricity (e) is expressed as

$e = (a + b\lambda)K$ where a and b are constants, $\lambda$ is the slenderness ratio and K is the core radius. In the Dutch method two pairs of constants are given related to two grades of timber. For the lower grade (standard bouwhout) a = 0.16 and b = 0.008 whereas for the higher grade (constructiehout) a = 0.10 and b = 0.005. Prof MOHLER said that the same expression was used in Germany but the constants "a" and "b" were 0.10 and 0.008 respectively and were applicable to all grades of North European softwoods. Prof LARSEN drew attention to the test results discussed in his paper and said that these showed that values for "a" and "b" of 0.10 and 0.005 respectively would result in satisfactory values for eccentricity and lead to acceptable designs. However he agreed that in some cases designs based on these values may be slightly conservative. It was finally agreed that unless more accurate values were known for "a" and "b", values of 0.10 and 0.005, respectively, should be used for all grades of European softwoods.

MR SUNLEY asked for comments on how the work on the design of solid timber columns should be published so that it would be suitable for inclusion in a Code of Practice on timber structures. Prof LARSEN suggested that a Code of Practice should contain a summary of the theory and the design method, together with references to the background papers. This was agreed and Prof LARSEN undertook to draft the section on the design of solid timber columns for the Code of Practice.

In the absence of the author, who was unable to attend the meeting, DR NOREN introduced a paper "Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns" by B Johansson (Paper 4). MR SUNLEY asked the delegates whether or not the design method which had already been agreed for centrally loaded columns should be extended to include columns subjected to end moments and lateral loads. MR REECE said he thought this was necessary because frequently the practical design problems facing engineers concerned with the design of columns, involved rather more than the simple case of axially loaded columns. MR SUNLEY agreed with this and Prof LARSEN agreed to include this extension in his draft section on the design of solid timber columns for the Code of Practice.

With regard to the design of spaced timber columns MR SUNLEY asked Prof LARSEN to summarise the present position. Prof LARSEN said that sufficient work had been completed to enable a method of design to be drafted but there was a problem in assigning acceptable values for the stiffness of the joints between the individual columns and the spacer blocks or battens. After discussion it was agreed that Prof LARSEN, Prof MOHLER and DR KUIPERS would provide a joint paper on direct compressive loads on spaced timber columns for the next meeting.

## 8 TEST METHODS FOR PLYWOOD

M HUC introduced a paper "Standard Methods of Testing for the Determination of Mechanical Properties of Plywood" (Paper 5) submitted by DR C WILSON of the Research and Development Department of COFI, Vancouver, who was unable to attend the meeting. He explained that this paper discussed the differences between test methods used for the derivation of design stresses for plywood and those used for quality control. The paper concludes with some specific comments on the test methods proposed by DR KUIPERS in a paper which was presented at the previous meeting of CIB-W18 in Delft, June 1974.

MR SUNLEY posed the question as to whether or not it was reasonable and desirable to include tests on full size boards in any standard set of test methods for the derivation of design stresses. He pointed out that some countries, eg Finland, only require small scale tests at present because of the small variation in the plywoods produced in those countries. DR KUIPERS said he thought there was a need for some large scale tests although these might be more appropriate for quality control testing than for the derivation of design stresses. He said he was aware of only one research project where a comparison was made between small and large scale tests and this suggested that there was a greater variation between the small scale samples than between the full size boards. It was his opinion that small scale tests would yield much more information about the variability of the strength of a particular plywood than tests on full size boards and a good knowledge of the total variability of the material was essential for a satisfactory derivation of suitable design stresses.

A long discussion followed on the most suitable sizes for small test specimens. DR NOREN said that in his experience tests on specimens less than 100 mm wide did not provide suitable data for the derivation of design stresses. He suggested that bending test specimens should be 300 mm long but that tension test specimens should be smaller than those generally used at present. On the whole DR NOREN thought that the ASTM methods of test were the most suitable methods in use at present except for the ASTM tension test and he recommended that CIB-W18 should adopt these methods with the exception of the tension test. DR NOREN also pointed out that the method of sampling small scale test specimens was important when considering the minimum size of test specimen. Large scale tests would always contain the weakest section but this was not the case with small specimens and this caused difficulties when analysing data obtained from small size test specimens. Therefore he thought there was a need to specify a standard method of sampling for small size specimens. MR CURRY said that in UK the variation in test methods and the resulting data for plywoods from different countries caused problems when any attempt was made to derive design stresses which were suitable for inclusion in the British Code of Practice. He suggested this could be overcome by having factors to take account of the size effect of the test specimens. These factors would be related to a preferred size of test speciment and as an example he suggested 250 mm as a suitable size for bending test specimens.

MR SUNLEY suggested that CIB-W18 should ask RILEM to draft a standard set of test methods for the derivation of design stresses for plywood. This could then be submitted to the appropriate ISO/TC with the backing of CIB-W18. However DR KUIPERS thought that this would not be possible in the near future due to the commitments which RILEM had already undertaken. It was therefore decided to leave the matter with the Chairman for the time being.

## 9 TIMBER BEAMS

MR BURGESS introduced his paper "The Design of Simple Beams" (Paper 6) which outlined the methods of design at present in use in a number of European countries. The paper also described the proposed design method which will be included in the revision of the present UK Code of Practice, based on "limit state" principles.

MR SUNLEY asked why some countries used a factor related to the depth of the beam while others did not and he questioned whether or not such a factor was necessary. Prof LARSEN said that in Denmark the design stresses were related to a maximum depth and for small sections the depth effect was assummed to be counter-balanced by greater variations in the strength of the timber. To avoid confusion he explained that there was a depth factor in use in Denmark but this was concerned with the instability of deep beams and nothing to do with timber strength. DR NOREN said that the depth effect could automatically be taken into account by varying the grading rules in relation to member size. MR CURRY disagreed with this saying that stress grading must be consistent and independent of the member size, however he did think there was a need for a modification factor to take account of the depth effect. MR SUNLEY agreed with this but he thought there was a need for further research to assess the effect more precisely. Prof MOHLER said Canada did not use a depth effect factor but one related to volume. He also said that Russia recognised a depth effect in beams and took it into account in design.

MR BURGESS pointed out that most countries seemed to use one or more factors to take account of load duration but there was a large variation in the values of the factors used in different countries, and the time periods to which the factors were related were frequently ill defined. Prof LARSEN said he thought most countries used factors which were based on the "Madison curve" although there was a significant body of opinion that this was no longer satisfactory. He thought that factors for long term and medium term loads were unnecessary and suggested that these should be replaced by a factor of 1.00 for normal duration loading and 1.40 for short term loading. DR NOREN said that the revised NKB Code would contain load duration factors which were related to the total duration of different loading intensities which occurred throughout the life of the structure. In this context a "normal" duration load would be one which was actually supported by the structure for a total period of between 1 and 10 years on a building with a design life of 30 years. In practice this would be equivalent to the type of load which at present is frequently defined as a medium term load.

PROF LARSEN submitted a paper "Calculation of Timber Beams Subjected to Bending and Normal Force" (Paper 7) but owing to a shortage of time it was not possible to allow any discussion. However Prof LARSEN undertook to write a comprehensive paper on the design of timber beams for the next meeting of the group.

## 10 TEST METHODS FOR JOINTS

DR KUIPERS said that there was a growing demand for tests on timber joints made with mechanical fasteners and at present a variety of different methods were in use in different countries. Therefore there was an urgent need for a set of standard test methods to make test results generally acceptable in countries other than that in which the tests were carried out. With this objective the RILEM 3TT Committee had drafted a set of test methods, "Proposal for a Basic Test Method for the Evaluation of Structural Timber Joints with Mechanical Fasteners and Connectors" (Paper 8). DR KUIPERS who is a member of this Committee submitted this to CIB-W18 for comment and the following amendments were recommended:

The wording of the second sentence in Clause 1 should be changed to read as follows: "to permit the calculation from the test results, values ........ "

Unless it is found necessary to specify different test methods for different types of connector, Clause 3 - Classification and Nomenclature of Joints, should be deleted.

Clause 4.1 should be deleted as it has a wider application than the testing of joints and connectors. It should therefore be included in the Code of Practice on timber structures as a separate section to which reference could be made when appropriate.

DR NOREN agreed to write a paper defining the conditions of a range of climatic groups, for the next meeting.

Clause 4.2 should be deleted as it is not strictly part of the test methods.

In Clause 6.9 the temperature condition should be altered to "$T = 20 \pm 3^{o}C$".

The value (4f) of the first peak in Figure 5, Clause 7.1.3 should be slightly in excess of the permissible design load and the rate of loading should be essentially continuous. The unloading portion of the curve should be optional as it may not always be necessary.

In Clause 7.1.6 the 10 per cent limitation may be too small and therefore may need to be larger. The definition of the ultimate load should be the maximum load or the load which causes a deformation of 15 mm in the joint, whichever occurs first.

Clause 7.2 - Long duration tests should be specified including a time limitation. These tests may not be necessary every time and therefore should be optional.

Details of the test method should be omitted from Clause 7.3 as the test should be designed so that it is appropriate to the particular joint or connector under test.

Details of the test method should be omitted from Clause 7.4 in order to allow special tests to be devised to meet the requirements of the particular joint or connector under test.

Clause 8 and Clause 9 should be deleted as interpretation of test results is not strictly part of the test methods, however guidance should be given on the number of replicates required in each test.

In Clause 10 details of the relative density of the timber and the quality of the connectors should be included.

Summing up MR SUNLEY suggested that this subject should be dealt with in two parts. The first part should specify the methods of test, applicable to bolts, nails, screws, integral tooth metal plate fasteners and Bulldog and split-ring type connectors together with details of the method of assembly of the joints, rate of loading and condition of the test specimen at the time of testing. The second part should be a comments section or number of appendices giving supplementary information. The Clauses 4,8 and 9 would be suitable for inclusion in this section. After discussion the delegates agreed to recommend this approach to the RILEM 3TT Committee, through DR KUIPERS, who agreed to try and supply a further draft from RILEM for the next meeting of CIB-W18.

Prof MOHLER submitted a short note "Test Methods for Wood Fasteners" (Paper 9) but as it was in German little discussion was possible and he agreed to provide a more comprehensive paper for the next meeting.

## 11 STRESS GRADING AND DERIVATION OF DESIGN STRESSES

Mr Curry reported on the recent activities of the ECE Timber Committee regarding the establishment of a European system for stress grading of sawn softwood. He submitted a copy of the report "Proposal for an International Standard for Stress Grading of Coniferous Sawn Timber" (Paper 10) which had been accepted and approved by the ECE Timber Committee. Following the approval of these grading rules the ECE Timber Committee had requested CIB-W18 to advise them on suitable design stresses for the new stress grades which had been defined. It was agreed by the members of CIB-W18 to undertake this task and as an introduction to the work Mr CURRY introduced a paper describing the derivation of grade stresses in the UK (Paper 11). Commenting on the paper Prof LARSEN said that it was incorrect to define the "green exposure condition" (page 2) as the condition where the moisture content of the timber exceeds 18 per cent. MR CURRY agreed. Prof LARSEN questioned the wisdom of specifying different stresses for redwood and whitewood (Table 4) because of the difficulty in distinguishing between the two species without microscopic examination and therefore he suggested the same stress values should be assigned to each species. He also doubted the implied accuracy in specifying the stresses to two places of decimals. MR CURRY replied that this was really a question of rounding off and generally stresses would not be quoted to two decimal places. However he pointed out that it had been agreed by BSI to specify stresses to three significant figures although in this case the third figure was not significant. It was generally agreed that there was a need for all countries to specify stresses to the same number of significant figures.

Still referring to Table 4 Prof SONNEMANS requested clarification of the "mean modulus of elasticity". MR CURRY explained that this was an error and the value quoted corresponded to the 5 per cent fractile value. Prof LARSEN said that in Denmark the 30 per cent fractile value was used and DR NORÉN said the 50 per cent fractile value was used in Sweden. MR CURRY replied that the choice of the 5 per cent fractile value was related to the value of the modification factor for load sharing and in the UK the product of these two was equivalent to the mean value.

Following a lengthy discussion regarding the assigning of stresses to the new ECE stress grades it was agreed that the 5 per cent fractile values should be established for each grade but it was considered unlikely that actual stresses could be assigned to each grade which would be acceptable to all countries. Therefore it was agreed that each country should be left to calculate acceptable stresses based on the 5 per cent fractile value. MR CURRY also pointed out the need for a large testing programme on structural size timbers using an agreed standard test method. MR SUNLEY proposed that a small sub-committee be set up to draft a standard test method for structural size timber. MR CURRY, Mr Saarelainen and Dr Kuipers were elected and agreed to serve on this sub-committee and to report back to the next meeting.

## 12 LOAD SHARING

MR LEVIN introduced a paper "A Review of Load-Sharing in Theory and Practice" (Paper 12) and pointed out that it was based largely on work carried out in North America and Australia as there appeared to be little information available from Europe. He went on to say that most of the work which had been carried out on this subject was concerned with floors and largely theoretical with little experimental test evidence. MR LEVIN outlined the distinction between the type of load-sharing which occurs under a concentrated load and the type which occurs under a uniformly distributed load and he drew attention to the variety of effects which were considered by different engineers to contribute to a load-sharing effect. He gave as an example a UDL on a joist and boarded floor where among the effects considered were the T-beam effect, the variation in stiffness between the joists and along the length of each joist and the plate effect of the membrane. It was this variety of effects which led to the variation in the magnitude of the load-sharing factors used by different countries. Prof LARSEN said that the grid system approach in use in Australia is based on the acceptance of a reduced factor of safety for individual members in a system where the failure of one member would not cause a total collapse.

DR NORÉN said that a fresh approach was required to load-sharing which would investigate all the effects which in the past had been considered to contribute to load-sharing. It would be necessary to consider what real contribution each effect made towards load-sharing and this would be followed by the need to quantify those effects which were found to be relevant. DR NORÉN submitted a note on load-sharing to the meeting (Paper 13).

MR SUNLEY suggested that the use of the term "load-sharing" to describe these various effects was misleading, particularly in the context of the partial coefficient method of design where a strict distinction is drawn between factors which modify the loads and those which modify the material characteristics.

After further discussion it was agreed that MR LEVIN would write a paper, for the next meeting, which would discuss the variety of effects which had been considered to contribute to load-sharing.

## 13 LONG-TERM LOADING

MR JOHANSEN introduced a paper "Long-Term Loading of Trussed Rafters with Different Connection Systems" (Paper 14) which is a report on the first six months of a research programme investigating the performance of a number of trussed rafters with different types of joint fasteners, subjected to continuous long-term loading.

MR REECE remarked on the apparent poor performance of the trussed rafters with plywood gussets relative to those with metal plate fasteners and questioned whether or not the designs of the two types of fasteners were comparable. He suggested that a comparison of joint slip may be a more satisfactory way of comparing the performance of plywood gussets and metal plate fasteners.
MR JOHANSEN replied that the design loads for the plywood gussets and the metal plate fasteners were similar and although joint slip was greater for the plywood gussets than for the metal plate fasteners other work suggests that trussed rafters with plywood gussets have higher ultimate loads than comparable trussed rafters with metal plate fasteners. DR NOREN suggested that the larger joint slips for the plywood gussets were because nail slip occurred in both the timber member and the gusset whereas for metal plate fasteners it only occurred in the timber member.

MR JOHANSEN agreed to write a further progress report on the research programme for the next meeting of the group.

MR SUNLEY opened a general discussion on long-term loading and asked members to report briefly on any recent research work on the subject with which they were familar.

DR KUIPERS said that work had been carried out in Holland on trussed rafters in the past over a period of 1½ to 2 years, and some long term tests on joints where still in progress. The test joints included split ring connectors, bulldog connectors and nails, all of which where subjected to a range of continuous loads equivalent to 65 per cent to 95 per cent of the ultimate loads. These tests were started more than 10 years ago and although the split ring and bulldog connectors failed after periods approximately equal to those which were predicted by the "Madison curve" the nailed joints had not yet failed and their performance did not fit the "Madison curve", although nail slips of 50 mm had been measured. DR KUIPERS said these results suggested that the "Madison curve" was not suitable for nailed joints and new tests had been started in controlled temperature and humidity conditions with the object of defining a better maximum load/time dependency curve. Included in this new series of tests were some joints made with "Menig" nail plate fasteners which failed after only short periods of loading. DR KUIPERS said that the type of failures experienced with these fasteners suggested that the nails were too short and not fixed sufficiently rigidly in the plate.

Prof MOHLER said that from a practical viewpoint deformation of joints was a more important factor in long term performance than ultimate load and he submitted a report in German which described some of the research work which had been carried out in Germany on this subject. It was decided however to hold this report over until the next meeting when an English translation would be available. DR MOHLER did refer briefly to some German work which indicated that plywood subjected to long-term loading did tend to follow the "Madison curve". However it was found that the "Madison curve" was not satisfactory for predicting the performance of short beams subjected to high shear and bending stresses. Prof MOHLER said this was due to a high rate of creep deflection caused by the high shear stresses.

MR REECE said that in the design of timber portal frames problems were encountered by the high moments which were assumed to occur at the eaves joints, however if the deflection of the frame was taken into account these assumed moments would be reduced and he asked if this design approach was used for timber. Prof MOHLER said that it was used in Germany and DR NOREN said that some account was taken of this effect in Sweden particularly with regard to the design of trussed rafters although it was disguised in a special design method for trusses.

MR CURRY described some work which had been undertaken at Princes Risborough Laboratory, UK on the long term loading of trussed rafters, finger joints and I beams with fibreboard webs. After a period of approximately six years under continuous loading the trussed rafters showed increases of between 100 per cent and 150 per cent over the initial maximum deflections under full design load. For the I beams the mid-span deflection increased by about 60 per cent over a period of loading of about 300 days. During this time the design dead load was maintained continuously and the imposed load was added for a period and then removed. This was repeated a number of times throughout the loading period. The finger joint tests covered three joint profiles and two adhesives. The test joints where positioned in short beams which were subjected to bending such that the bending stresses at the joint were equal to the maximum permissible long-term stresses appropriate to the highest grade of structural timber. This stress level was maintained continuously but every fourth week it was increased by 25 per cent for one week and then reduced to the long-term level. After six months there were no indications of any joints failing and the additional deflections due to the 25 per cent increases were fully recovered when the stresses were reduced to the long-term level.

MR SUNLEY, in concluding the discussion, said that it appeared that most people used the "Madison curve" for the derivation of load duration factors. However there was a strong opinion that the level of stress was an important factor and recent work, particularly in North America, suggests a need for the "Madison curve" to be modified to take this into account. Prof LARSEN said that frequently the load duration factors based on the "Madison curve" were linked with safety factors and for this reason it would be difficult to establish a set of factors which would be generally acceptable. MR CURRY agreed with this although he pointed out the need to establish the true load duration factors independent of safety factors, so that the duration of different loading intensities could adequately be taken into account in the limit state methods of design currently being introduced.

## 14 EEC TIMBER CODES

As an introduction to the subject MR BURGESS presented a paper "Comparison of Codes and Safety Requirements for Timber Structures in EEC Countries" (Paper 15) which had been prepared by the Timber Research and Development Association, UK.

Prof LARSEN said that this survey drew attention to the differences between the various national timber codes and loading codes, and because these differences, in many cases, were so large he did not consider it would be possible to draft an acceptable European timber code. Therefore he suggested that CIB-W18 should publish "Recommendation Sheets" to which each country could refer in their national codes, when they were appropriate. In order for these sheets to be acceptable to as many countries as possible they should only deal with the principles of design and the basic methods of approach and they should not contain values for the various modification factors, etc. The values of these factors would be set by each country independently having regard to other national codes and standards relevant to the design of timber structures in each country. MR SUNLEY did not agree with this suggestion and he pointed out that it had already been agreed by a Joint Committee on Structural Safety (JCSS) that an attempt would be made to draft a unified system of structural design codes covering all materials. Volume I in this system would specify loadings and basic design criteria applicable to all materials, Volume II would deal specifically with the design of concrete structures and Volume VI with the design of timber structures. MR SUNLEY reported that he was a member of the JCSS and it was hoped to agree the contents of Volume I before the next meeting of CIB-W18. It was also understood by the JCSS that CIB-W18 would draft Volume VI dealing with the design of timber structures. This had been agreed at the previous meeting of CIB-W18, June 1974, Delft. In the general discussion which followed delegates accepted that it would be difficult to agree values for factors in Volume VI although to a large extent this would depend on the contents of Volume I. DR NOREN suggested that CIB-W18 should comment on the contents of Volume I and when this was agreed the group should proceed with the drafting of Volume VI as far as possible. This was agreed by the meeting.

## 15 NORDIC CODES ON LOADS AND SAFETY

MR BRYNILDSEN introduced his paper "Nordic Proposals for Safety Code for Structures and Loading Code for Design of Structures" (Paper 16) which discusses proposals by the Nordic Building Regulations Committee (NKB) for unified codes on safety and loading for Scandinavia. He also tabled copies of the proposals and comments published by NKB, "Proposal for Safety Codes for Load-Carrying Structures" (Paper 17) and "Comments to Proposal for Safety Codes for Load-Carrying Structures" (Paper 18).

MR BRYNILDSEN, in discussing his paper, said that the division of buildings into three safety groups (clause 1.1) had been adopted in Norway already and had been found to work in practice. Prof LARSEN said that the current steel code for Denmark also classified buildings in these groups but he thought that this classification would be radically ammended or deleted and the new timber code "Structural Use of Timber" - DS413, December 1974, did not use this classification. Prof MOHLER said that a similar classification system had been proposed in Germany for some materials but timber was not included. DR KUIPERS said that a classification system similar to that proposed by NKB had been considered in Holland but it had not been adopted and there was little support for such a system.

MR SUNLEY said that the three classifications of limit states (clause 1.2) were different from the three categories proposed by the Joint Committee on Structural Safety which were serviceability, ultimate load, and durability. After general discussion it was agreed that durability was not a real limit state in its own right and it would probably be better to include it within the serviceability or ultimate load categories.

MR BRYNILDSEN drew attention to the need to classify the type of failures which normally occurred in timber structures (clause 1.3). He said that no agreement had yet been reached on the most appropriate "Rupture group" for timber failures and it may be necessary to classify timber joint failures in a different category. However, most delegates thought that timber structures would generally come within Safety group 2 (clause 1.1) and timber failures in Rupture group II (clause 1.3).

Referring to clause 1.4 MR BRYNILDSEN said the tables of load factors were quoted direct from the loading code proposed by NKB and were applicable to all materials. DR NOREN tabled copies of this code and comments published by NKB, "Load Regulations" (Paper 19) and "Comments on the Load Regulations" (Paper 20).

MR BRYNILDSEN said that the partial factor related to the control of materials and construction (clause 1.4, page 5) was intended to take account of the degree of control exercised during the construction of a structure, the variability of the material properties and the degree of competence of the engineer responsible for the structural design. It was understood that the Timber Committee within NKB would set standards by which these criteria could be assessed so that the appropriate partial factor could be used in the design.

Finally MR BRYNILDSEN pointed out that the time intervals specified under Duration of Loads (clause 2.1) were the longest individual periods a structure would have to sustain a specific load continuously. There was a significant body of opinion among the delegates that this was not the best way in which to deal with load duration and a more realistic and satisfactory method would be to specify load duration in terms of the total time that a structure may have to sustain a particular intensity of loading throughout the life of the structure.

For information MR BRYNILDSEN tabled a copy of an extract from the current Norwegian Standard NS 3470 "Timber Sturctures" (Paper 21) as an example of a timber code based on "limit state" principles of design.

6 REVISION OF BRITISH CODE CP 112

MR CURRY introduced a paper "Draft for Revision of CP 112 - "The Structural Use of Timber" (Paper 22) which outlined the format for the new Code and included a draft of the first five chapters.

Prof LARSEN said that in his opinion the Code was far too long and contained too much detailed information. He would prefer a short Code which gave the basic principles and methods of design and made reference to other publications or a series of information sheets which contained the detailed information required by designers. Prof LARSEN also questioned the deflection limitation of 0.003 X the effective span for floor joists (clause 4.2.2). In Denmark the Code requirement for domestic floor joists was a maximum deflection of 0.9 mm under a concentrated load of 1000 N. MR SUNLEY pointed out that in the British Code there was a further recommendation that in order to avoid problems from vibration the maximum deflection of floor joists should not exceed 13 mm under design load (clause 4.2.5).

MR VISSER asked if prototype tests on scaled structures instead of full size structures (clause 1.1) would be accepted as a satisfactory method of proving a design. MR CURRY said model testing would not generally be acceptable as it was not possible to scale down joint details so that their performance was representative of the full size joint.

Prof MÖHLER said that there is a provision in Germany which allows structures to be constructed with "wet" timber ie at a moisture content greater than 22 per cent, although the design and ultimate use of the structure relates to dry conditions. MR CURRY said that this was not allowed in the UK at present and there was no intention to make provision for it in the revised Code.

Prof MOHLER also asked for a definition of the density quoted in Table 1 and secondly if it was intended to include design criteria for chipboard and fibreboard in the revised Code. MR SUNLEY replied that the density quoted was an average figure to allow designers to calculate dead weight. Secondly it was likely that stresses would be included for fibreboard but not for chipboard as at present the chipboard industry in UK did not think there was a sufficient need to justify the large amount of test work which would be required to derive suitable stresses for chipboard.

MR VISSER said that in order to take account of the duration of different loads he would prefer the various modification factors to be applied to the loads rather than to the stresses as they were in the UK Code (clause 5.6.1, Table 11). MR CURRY replied that both methods were satisfactory but by modifying the stresses rather than the loads a closer approximation to the true behaviour of the timber was achieved.

Prof SONNEMANS commented that the title "Fire Resistance" to clause 4.2.4 was incorrect as the subject of this clause was really surface spread of flame. MR CURRY agreed that a more appropriate title could be substituted.

In conclusion MR SUNLEY pointed out that although a large measure of agreement had been reached on the drafts of the first five chapters of the revised Code there was still an opportunity for people to comment and request modifications before it was published.

## 17 FUTURE PROGRAMME OF WORK

MR SUNLEY said that CIB-W18 had now established itself and was accepted as an authorative body in the field of timber structures whose opinions and advice were being increasingly sought by other international organisations. He reminded members that the group had undertaken to advise the ECE Timber Committee regarding suitable stresses for the new timber grades EC1 and EC2. In addition the group had accepted the task of drafting the timber section, Volume VI, of the unified system of structural codes proposed by the Joint Committee on Structural Safety and it was expected that when the new ISO/TC on timber structures, proposed by Denmark, was established CIB-W18 would be a major contributor to the work.

MR SUNLEY told the meeting that Prof Mohler had offerred to act as host to the next meeting in Karlsruhe in October 1975. This was gratefully accepted by the delegates and it was agreed to hold the meeting from 1-3 October, when the major topics for discussion would be as follows:

i Timber beams - Prof Larsen to provide a comprehensive paper on the design of timber beams.

ii Timber columns - Prof Larsen to provide a draft section on the design of solid timber columns for inclusion in the Code of Practice. Prof Larsen, Prof Mohler and Dr Kuipers to provide a joint paper on direct compressive loads on spaced timber columns.

iii Plywood - Dr Booth to provide a paper on the theoretical derivation of design stresses for different plywoods.

iv Joints - Dr Kuipers agreed to report back to the RILEM 3TT Committee regarding the comments from the members of CIB-W18 on the RILEM proposals for test methods for timber joints and fasteners. He hoped he would be able to provide a further draft for the next meeting of CIB-W18. Prof Mohler undertook to provide a more comprehensive paper on joint testing incorporating the short note which he submitted to this meeting.

v Stress grading - It was agreed that Mr Curry, Mr Saarelainen and Dr Kuipers would write a joint paper on test methods for structural size timber to obtain data from which design stresses could be derived. In addition Mr Curry undertook to provide a paper dealing with the analysis of test data for the derivation of design stresses together with an indication of the stress levels which might be considered appropriate to the new ECE stress grades.

vi Load-sharing - Mr Levin to provide a further paper discussing the various effects which contribute to load-sharing.

vii Long-term loading - Mr Johansen agreed to provide a further progress report on a research project to study the performance of trussed rafters under long-term loading. Dr Norén to provide a follow-up paper to his previous paper, which would discuss how long-term loading could be dealt with in the Code of Practice.

viii Climatic groups - Dr Norén to provide a paper defining suitable climatic groups for timber structures.

ix Draft codes of practice - Mr Sunley agreed to provide a draft of the JCSS Unified Code - Volume I and Dr Booth agreed to draft a suitable format for Volume VI, the timber code, based on the recommendations of the JCSS.

Finally MR SUNLEY thanked delegates for their participation in the meeting and M Crubilé and Centre Technique du Bois for providing the necessary facilities for the meeting to take place.

## 18 PAPERS PRESENTED AT THE MEETING

PAPER 1 A Proposal for Undertaking the Preparation of an International Standard on Timber Structures - International Standards Organisation.

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

PAPER 3 Tests with Centrally Loaded Timber Columns - H J Larsen and Svend Sondergaard Pedersen.

PAPER 4 Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns - B Johansson.

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

PAPER 6 The Design of Simple Beams - H J Burgess.

PAPER 7 Calculation of Timber Beams Subjected to Bending and Normal Force - H J Larsen.

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

PAPER 9 Test Methods for Wood Fasteners - K Möhler.

PAPER 10 Draft Proposal for an International Standard for Stress Grading Coniferous Sawn Softwood - ECE Timber Committee.

PAPER 11 Derivation of Grade Stresses for Timber in UK - W T Curry.

PAPER 12 A Review of Load-Sharing in Theory and Practice - E Levin.

PAPER 13 Load-Sharing - B Norén.

PAPER 14 Long-Term Loading of Trussed Rafters with Different Connection Systems - T Feldborg and M Johansen.

PAPER 15 Comparison of Codes and Safety Requirements for Timber Structures in EEC Countries - Timber Research and Development Association.

PAPER 16 Nordic Proposals for Safety Code for Structures and Loading Code for Design of Structures - O A Brynildsen.

PAPER 17 Proposal for Safety Codes for Load-Carrying Structures - Nordic Committee for Building Regulations.

PAPER 18 Comments to Proposal for Safety Codes for Load-Carrying Structures - Nordic Committee for Building Regulations.

PAPER 19 Loading Regulations - Nordic Committee for Building Regulations.

PAPER 20 Comments on the Loading Regulations - Nordic Committee for Building Regulations.

PAPER 21 Extract from Norwegian Standard NS 3470 "Timber Structures".

PAPER 22 Draft for Revision of CP 112 "The Structural Use of Timber" - W T Curry.

PAPER 1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# A PROPOSAL FOR UNDERTAKING THE PREPARATION OF AN INTERNATIONAL STANDARD ON TIMBER STRUCTURES

INTERNATIONAL STANDARDS ORGANISATION

PARIS - FEBRUARY 1975

ISO Central Secretariat

our date 1975-01-16

our reference ISO/TS/P 129

BSI DOCUMENT No. 75/10226 C'TTEE REF. BLCP/17

**INQUIRY**

AMONG MEMBER BODIES ON THE PROPOSAL

| | |
|---|---|
| Title : Timber structures | Reference : ISO/TS/P 129 |
| Originator : Denmark | Date of Proposal : September 1974 |

Member Bodies are requested to comment on the attached proposal for the preparation of International Standards in a new field (ISO/TS/P 129), using for their reply the attached form which should be returned in duplicate to the Central Secretariat.

The Member Bodies' replies should reach the Central Secretariat at the latest by 9 May 1975.

Remarks by the Central Secretariat :

At the last meeting of Technical Division 3 - Building, it was agreed that in addition to its work on concrete structures (ISO/TC 71), ISO should start an activity on wood and metal structures and that two further technical committees should be established for this purpose. It was also agreed that Denmark would originate a proposal for timber structures which is the subject of this present inquiry.

It is to be noted that the UN Economic Commission for Europe has included in its work programme, the harmonization of the technical content of building regulations for designing concrete structures, metal structures and timber structures. The proposed TC would be responsible for preparing the necessary International Standards for timber structures to which reference may be made in building regulations.

A proposal for metal structures will be circulated at a later date.

Attachments : Proposal ISO/TS/P 129
Form for reply (in triplicate)

W. H. Raby
Director
Planning and Programming

cc. President
Immediate Past President
General Secretary IEC



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

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Telephone: (022) 34 12 40
Telex: 23 887 ISO CH
Telegrams: ISORGANIZ

Date :
1975-01-16

**PROPOSAL**

FOR UNDERTAKING THE PREPARATION OF INTERNATIONAL STANDARDS IN A NEW FIELD

| | |
|---|---|
| Title : Timber structures | Reference : ISO/TS/P 129 |
| Originator : Denmark | Date of Proposal : September 1974 |

1. Title

Timber structures

2. Scope

Standardization in the field of structural use of timber and related materials, components and connections including codes of practice for the design and execution of structural timber

3. Explanation

The International Council for Building Research, Studies and Documentation (CIB), in its working group 18 - Timber structures, is well advanced in its work for a unified code for the design of timber structures. The proposed ISO activity would liaise closely with this work and provide the forum for the establishment of the necessary International Standards.

4. Justification

With the development of greater markets, international trade in timber and related products has developed in the direction of more finished products, structural components, complete timber product structures, and connected with it, the exchange of specialized know-how and services is increasing. As with all other buildings and structures, timber structures have to be erected at the place of use; particularly the international harmonization of building regulations and codes of practice for the design and execution of structural timber has become an urgent matter for international standardization. This would also encourage developing countries to enter into the international market with more sophisticated products.

The UN Economic Commission for Europe has included the harmonization of building regulation in the field of timber structures in its priority programme and it is for ISO to provide the necessary technical specifications and/or requirements.

5. **Programme of work**

The initial programme of work of the proposed ISO technical committee will be discussed and established at its first meeting.

It is intended that the establishment of an international standard code of practice for the design and execution of structural timber will be developed on basic functional structural requirements with special regard to the technology of timber materials and related products and applications. The proposed code should be developed for direct application in most countries. It should include levels of requirements for economic application in different regions of the world.

The work will be based on the results of the existing specialized ISO technical committees such as ISO/TC 98 - Bases for design of structures.

6. **National or other standards or other documents to be considered**

Existing National Standards, Codes of Practice, etc., as well as the work of CIB/W18 will be considered.

7. **Participation in the work**

Denmark would actively participate in the new T

# REPLY

OF ....................

ON THE PROPOSAL :

| | |
|---|---|
| Title : Timber structures | Reference : ISO/TS/P 129 |
| Originator : Denmark | Date of Proposal : September 1974 |

a) Agreement with study

We agree
We do not agree
to the question proposed *
being dealt with by ISO

b) Scope

We agree without reserve to the scope suggested by the originator of the proposal *

or We comment on the proposed scope (hereunder or in attached document) *

c) Participation

If a new Technical Committee or Sub-Committee is set up :

We are willing to undertake the Secretariat*

or We wish to participate actively in the work*

or We wish to be kept informed of the work *

or We do not wish to be kept informed of the work *

d) Documentation

List of standards, regulations or other relevant documentation existing in our country (copies attached) and any remarks concerning their application

.......................................

.......................................

.......................................

Comments

Place and date : ........................

Signature : ............................

* Delete as appropriate

COPY

ARM



75/10226

PAPER 2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# SYMBOLS FOR TIMBER STRUCTURE DESIGN

by

J KUIPERS, TECHNISCHE HOGESCHOOL, DELFT

B NOREN, SVENSKA TRÄFORSKNINGSINSTITUTET, STOCKHOLM

PARIS - FEBRUARY 1975

BN/lg 19740320
Revised, Delft 19740620

# SYMBOLS FOR TIMBER STRUCTURE DESIGN

| GEOMETRIC QUANTITIES | SYMBOL | | UNIT | |
|---|---|---|---|---|
| Cross section quantities | Ordinary | Printer | Basic | Multiple |
| Width | $b$ | B | m | mm |
| Depth, height | $h$ | H | | |
| Depth of compression zone | $h_c$ | HJC | | |
| Depth of tension zone | $h_t$ | HJT | | |
| Thickness | $h$ | H | | |
| Thickness | $t$ | T | | |
| Distance | $a$ | A | | |
| Co-ordinate | $y$ | Y | | |
| Co-ordinate | $z$ | Z | | |
| Co-ordinate, polar | $r$ | R | | |
| Radius | $r$ | R | | |
| Radius of gyration | $i$ | I | | |
| Diameter | $d$ | D | | |
| Area | $A$ | AA | $m^2$ | $mm^2$ |
| Area of flange | $A_f$ | AAJF | | |
| Area of web | $A_w$ | AAJW | | |
| Volume | $V$ | VV | $m^3$ | $mm^3$ |
| First moment of area | $S$ | SS | $m^3$ | $mm^3$ |
| Second moment of area | $I$ | II | $m^4$ | $mm^4$ |
| Section modulus | $W$ | WW | $m^3$ | $mm^3$ |

## Composed section quantities

| | | | | |
|---|---|---|---|---|
| Number of members or layers | $n$ | N | | |
| No of member | $i$ | I | | |
| Contribution of member No i to total | | | | |
| height | $h_i$ | HJI | | |
| area | $A_i$ | AAJI | | |
| second moment | $I_i$ | IIJI | | |
| Second moment of area of member No i (based on own centre) | $I_{0i}$ | IIJOI | | |
| Effective second moment of area | $I_e$ | IIJE | | |

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

| Special applications | | | SYMBOL Ordinary | SYMBOL Printer | UNIT Basic | UNIT Multiple |
|---|---|---|---|---|---|---|
| Equal symbols or x,y,z can be used for laminated timber or panels: | | | | | | |
| Laminated wood | Plywood and similar prod | Particle- and fibre board | | | | |
| Fibre direction of laminae | Fibre direction of face veneer | Machine- or longitudinal direction | l<br>x | L<br>X | | |
| Perpendicular to fibre direction of laminae, parallel to plane of joints | Perp. to fibre direction of face veneer,parallel to glue lines | Perp.to machine- or longitudinal direction, parallel to face veneer | t<br>y | T<br>Y | | |
| Perpendicular to plane of joints thickness) | Perp. to face (through | Perp. to machine- or longitudinal direction | r<br>z | R<br>Z | | |
| FORCES AND STRESSES | | | | | | |
| Force in general | | | F | FF | N | kN,MN |
| Normal force, axial force | | | N | NN | | |
| Axial force in member also | | | S | SS | | |
| Shear force | | | V | VV | | |
| Moment | | | M | MM | | |
| Torque | | | T | TT | | |
| Reaction force | | | R | RR | | |
| Horizontal reaction force | | | H | HH | | |
| Vertical reaction force | | | V | VV | | |
| Reaction or force in general, parallel to X-axis | | | X | XX | | |
| Reaction or force in general, parallel to Y-axis | | | Y | YY | | |
| Reaction or force in general, parallel to Z-axis | | | Z | ZZ | | |

| | SYMBOL Ordinary | SYMBOL Printer | UNIT Basic | UNIT Multiple |
|---|---|---|---|---|
| For intensity (force per unit length or area) use lower case letters. See also stresses: | | | | |
| Normal stress | $\sigma$ | SI | | |
| Compressive stress | $\sigma_c$ | SIJC | | |
| Tensile stress | $\sigma_t$ | SIJT | $Pa=N/m^2$ | $MPa=N/mm^2$ |
| Bending stress (M/W) | $\sigma_b$ | SIJB | | |
| Shear stress | $\tau$ | TA | | |
| STRAIN, DEFORMATION AND DISPLACEMENT | | | | |
| Strain (incl. compressive strain) | $\epsilon$ | EP | | |
| Displacement, deflection | a | A | m | mm |
| Displacement, deflection x) | u | U | | |
| Rotation | $\Theta$ | TH | | |
| Shear | $\gamma$ | GA | | |
| <u>Rheology quantities (subscripts)</u> | | | | |
| Creep | c | C | | |
| Recovery | r | R | | |
| Permanent (irrecoverable) | ir | IR | | |
| Creep coefficient (function) | $\phi$ | PH | | |
| Relaxation coefficient (function) | $\psi$ | PS | | |
| <u>Moduli and coefficients</u> | | | | |
| Elasticity modulus | E | EE | | |
| Apparent elasticity modulus $\frac{E}{1+\phi}$ | $E_\phi$ | EEJPH | $Pa=N/m^2$ | $MPa=N/mm^2$ $GPa=kN/mm^2$ |
| Rigidity (shear modulus) | G | GG | | |
| Displacement modulus | K | KK | | |
| Slip modulus | k | K | N/m | N/mm |
| Poisson´s ratio | $\nu$ | NU | | |
| Viscocity for pure flow | $\eta$ | ET | $Ns/m^2$ | |

x) For displacement also v and w may be used.

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

PAPER 3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# TESTS WITH CENTRALLY LOADED TIMBER COLUMNS

by

H J LARSEN, INSTITUTTET FOR BYGINGSTEKN, AALBORG, DENMARK

SVEND SØNDERGAARD PEDERSEN, TECHNICAL UNIVERSITY OF DENMARK

PARIS - FEBRUARY 1975

## 1. SUMMARY

120 tests* have been accomplished with columns of Nordic conifer, partly of normal structural grade (unclassified), partly of high grade (T-300). Cross-sections of 50 × 100 mm and 63 × 125 mm have been applied and for part of the columns tests in both principal axes have been made. Slenderness ratios ranged between approx. 20 and 300.

The tests were carried out with special bearings ensuring that the column was simply supported in the end cross-sections with a very slight friction. The bearings are further described in section 4.

The main conclusion of the tests is that the theory given in section 2 is very satisfactory. The theory is based on the theory of elasticity and the assumption that in the middle cross-section the column force has an initial eccentricity $e$, that - independent of timber grade and direction - can reasonably be put at

$$e = (0.1 + 0.005\,\lambda)k$$

$\lambda$ being the slenderness ratio and $k$ the core radius**. Reference is made to section 5, especially figs. 5.4 and 5.5, showing both test results and theoretical values.

Further conclusions of the tests are that:

- the Euler load-carrying capacity can be determined very satisfactorily by the so-called Southwell-plot (cf. section 2 and fig. 5.2),
- the accordance between the modulus of elasticity determined by edgewise bending tests and by the Euler formula is very satisfactory. On the other hand, the correlation between the modulus of elasticity in compression (determined on 200 mm long prisms) and the modulus of elasticity in bending is weak (cf. sections 3 and 5),
- the compression strength and the modulus of elasticity in bending were found equal for the two grades (cf. section 3),
- the correlation between the initial deflection of the columns in the middle and the initial eccentricity determined from the tests by the Southwell-plot is weak (cf. fig. 5.3).

In all essentials the results are in accordance with those found by similar Dutch tests [4].***

* The tests have been carried out by Svend S. Pedersen in connection with his master thesis at the Structural Research Laboratory of the Technical University of Denmark.

** Symbols are explained in section 6.

*** Figures in [ ] refer to the references in section 7.

## 2. THEORY

The theoretical expressions derived in [5] are assumed to apply. These are based on the following assumptions:

- the material is ideal-elastic till rupture which is assumed to occur when the following condition (inter-action formula) is satisfied:

$$\frac{\sigma_c}{s_c} + \frac{\sigma_b}{s_b} = 1 \tag{1}$$

$\sigma_c$ and $\sigma_b$ are the axial stresses from axial force (compression) and moment (bending), respectively, while $s_c$ and $s_b$ are the corresponding strength values,

- a force placed in the geometric centre of gravity of the end cross-section is assumed to act with an eccentricity $e$ in the middle cross-section due to the initial curvature and differences between the geometric and elastic centres of gravity:

$$e = \epsilon k = [a + f(\lambda)]k \tag{2}$$

$\epsilon$ is the relative eccentricity (relative to core radius $k$). $a$ is a constant (= $\epsilon$ for $\lambda = 0$), while $f(\lambda)$ is a function (without a constant term) of the slenderness ratio $\lambda$. $\lambda = \ell/i$, where $\ell$ is the buckling length and $i$ the radius of gyration,

- the eccentricity of the force is assumed to vary sinusoidally along the column.

The load-carrying capacity of the column expressed by the critical axial stress $s_{cr}$ is then, cf. (12) in [5], determined by

$$\frac{s_{cr}}{s_c} = A - \sqrt{A^2 - B} \tag{3}$$

where

$$A = \frac{1 + [1 + \frac{s_c}{s_b} f(\lambda)]k_E}{2(1 - a\frac{s_c}{s_b})} \tag{4}$$

$$B = \frac{k_E}{1 - a\frac{s_c}{s_b}} \tag{5}$$

$$k_E = s_E/s_c = \frac{\pi^2 E}{\lambda^2 s_c} \tag{6}$$

$s_E$ is the stress determined by the Euler formula.

On the assumptions mentioned the deflections in the middle, $u$, are approximately

$$u = e\frac{P}{P_E - P} \tag{7}$$

$P_E$ being the Euler load-carrying capacity ($P_E = \pi^2 EI/\ell^2$, $I$ is the moment of inertia).

From this is found

$$\frac{u}{P} P_E - u = e \tag{8}$$

**Plotting u as a function of u/P, the points must - if the assumptions are correct - lie on a straight line, from which $P_E$ can be found as the slope of the line and e as the length the line cuts off the u-axis. The method is given by Southwell and the u-u/P-diagram is called a Southwell-plot.**

## 3. THE TEST MATERIAL

Spruce of two grades was used, viz. unclassified and T-300.

The unclassified grade is that most generally found in structural use and corresponds mainly to Swedish fifth grade with the worst pieces taken away. The characteristic (5%-fractile) short-term bending strength is about 20 N/mm². Poorer qualities are not allowed to be used for constructions. Graded according to the Swedish T-grading rules T-300 is the highest grade obtainable for structural use. The characteristic strength is about 30 N/mm².

Two dimensions were used, viz. 50 × 100 mm and 63 × 125 mm. After drying and planing the cross-sections were about 45 × 95 and 58 × 120, cf. table 5.1, column 3.

The timber was oven-dried and then kept in a climate room (18-20° C) at about 65% relative humidity. The moisture content for each test specimen was determined from the weight loss of the prisms mentioned below by drying till the weight was constant. At an average the moisture content was 10.6% and so uniform that variations of the moisture content are estimated to have no influence on the test results.

For each test specimen the prism strength $s_c$ was determined on a 200 mm long prism which had been cut off one end of the test specimen immediately before the test. The loading time till rupture was 3-5 minutes. The result is given in table 5.1, column 4. For T-300 an average of 40.0 N/mm² with a deviation of 5.9 N/mm² was found, while an average of 40.2 N/mm² with a deviation of 6.3 N/mm² was found for unclassified timber. The values of the two grades correspond to a characteristic strength (5%-fractile) of 29 N/mm², i.e. the difference in bending strength of the two grades is not reflected in the compression strength. The reason could be that the prisms for unclassified timber do not contain defects determining the grade to the same extent as the columns used in the tests, as they were often degraded due to a few gross defects. The prism strength expresses the wood fibre quality rather than the timber quality.

In connection with the determination of compression strength the modulus of elasticity in compression, $E_c$, was also measured. There was no difference for the two grades. At an average $E_c$ = 11800 N/mm² with a deviation of 600 N/mm² was found.

No connection was found between $E_c$ and the compression strength or the modulus of elasticity determined by bending- or column tests.

The prisms used for determining the compression strength were also used for determining the density corresponding to weight and volume after drying.

The same value was found for both grades, namely on average 470 kg/m³. No connection was found between density and the other properties.

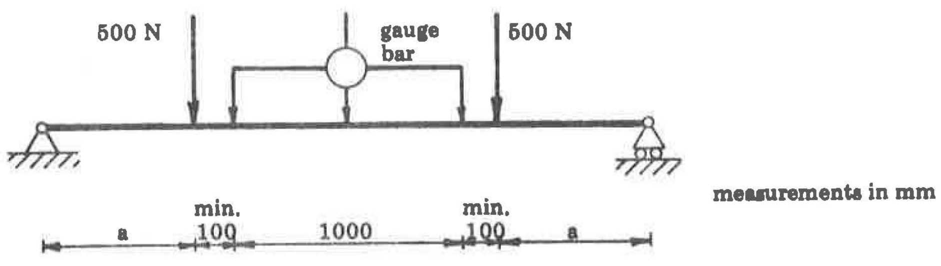


Fig. 3.1

For all columns longer than about 2 m the modulus of elasticity in bending $E_b$ was determined by an arrangement as shown in fig. 3.1. The deflection (curvature) over a length of 1000 mm was measured with a 1/100 mm dial gauge. The values measured are given in table 5.1, column 5.

For T-300 an average of $E_b$ = 13500 N/mm$^2$ with a deviation of 1900 N/mm$^2$ was found corresponding to a characteristic value of 9700 N/mm$^2$. Correspondingly, 11800, 1700, and 8400 N/mm$^2$ were found for unclassified timber. These values are - like compression strength and density - somewhat higher than would normally be expected for the grades in question.

For deflection in the weak direction 7 slenderness ratios $\lambda$ have been tested, about 20, 45, 65, 110, 155, 220, and 310. For each $\lambda$-value, except $\lambda = 310$, both grades and both dimensions of cross-sections have been tested; for $\lambda = 310$ only 50 × 110 mm has been tested.

Furthermore, for $\lambda$ = 110, 155, 220, and 310 deflection in the strong direction has also been tested (the value of $\lambda$ being a little below half of those stated).

A total of 40 series has thus been carried out, each series comprising 3 tests.

## 4. TEST SET-UP AND PROCEDURE

A general arrangement as shown in fig. 4.1 was used. The figure shows deflection in the strong direction of the column.

The load was applied vertically through a frame. Torsion of the column was prevented in its upper and lower sixth sections by supporting bars on which was glued teflon, while aluminium plates were similarly glued on the column. The length between the rails was adjustable so that it corresponded to the actual column dimension. The column was secured against deflection in the weak direction by a steel bar placed at intervals of one-third of the column's length, hinged to a small fitting attached to one side of the column.

The deflections in the middle and at the quarter points were recorded by electric transducers.

For tests with deflection in the weak direction the supports at the one-third intervals were omitted (the column cross-section is rotated 90° in cross-sections A-A and B-B).

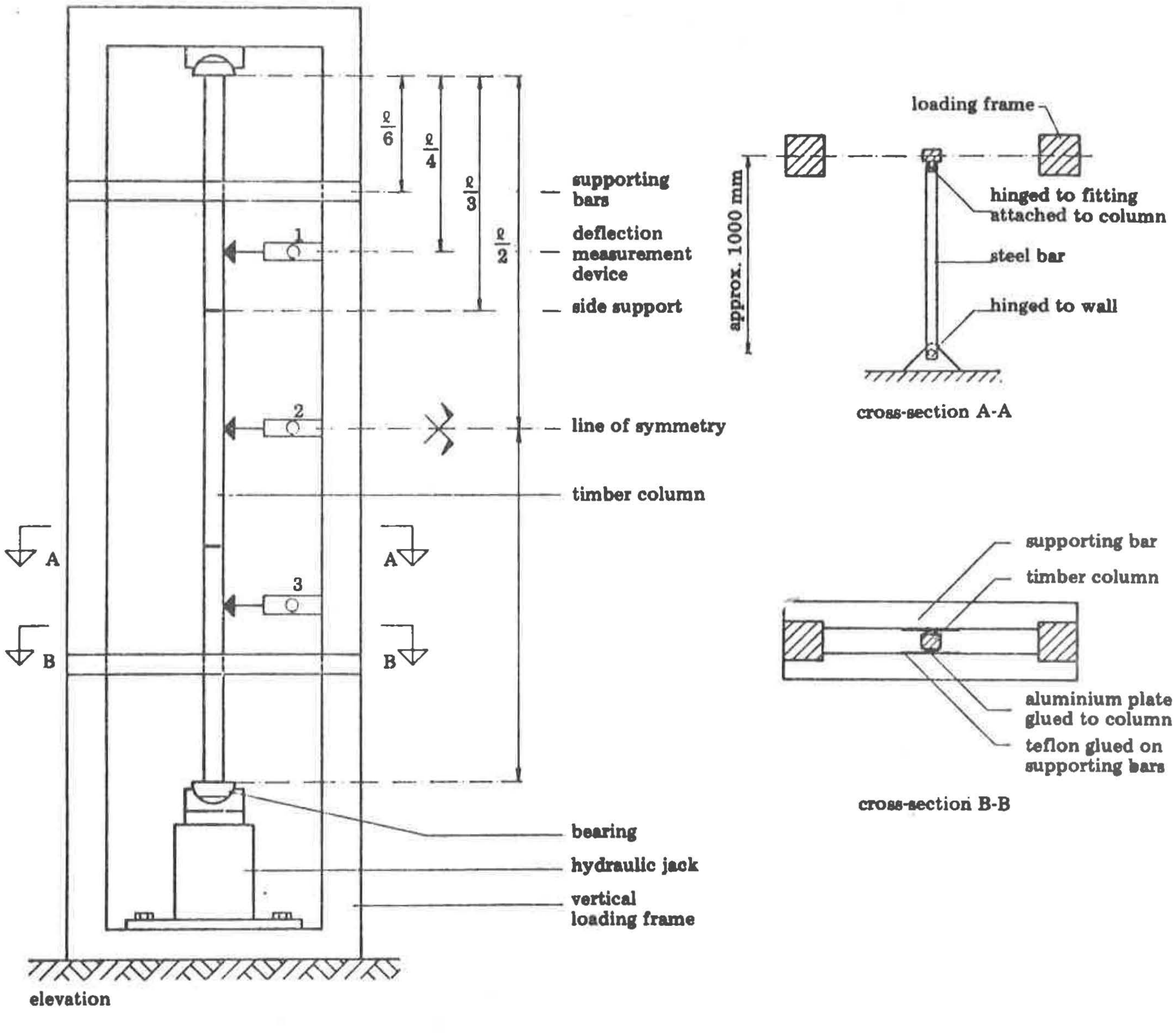


Fig. 4.1

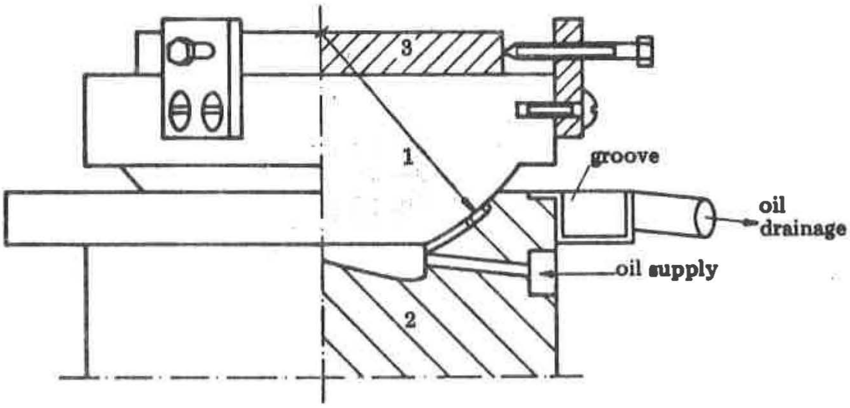


Fig. 4.2

The bearings were in principle as shown in fig. 4.2, which shows a lower bearing. The upper part 1 and the lower part 2 of the bearing were formed in the contact area as part of a spherical surface with milled grooves, which were in connection with an oil chamber milled in the lower part. During the tests oil was pumped into the oil chamber and out through the grooves, so that the upper part came to rest on an oil film.

Surplus oil was gathered in a groove with drainage. The thickness of the steel plate 3 was adapted so that the centre of the bearing surface was in the surface of the plate, and by the adjustment screws shown it was ensured that the centre of the plate coincided with that of the bearing surface. For further description of the bearings refer to [2].

Two sets of bearings were made, one corresponding to maximum loading of approx. 100 kN, the other to approx. 500 kN. By calibration it was found that the initial friction for the small bearings by maximum loading (100 kN) corresponded to a force eccentricity of less than 0.01 mm.

The lower bearing rested on a hydraulic compression jack rigidly fastened to the loading frame.

In the tests - apart from the force - the horizontal displacement in the centre and at the quarter points was measured. The readings were registered by a datarecorder, and load deflection curves and a Southwell-plot, cf. section 5, were automatically drawn.

For short columns only deflection in the weak direction was tested, while tests around both axes were carried out for slender columns ($\lambda_{min} > 100$). In the latter case the failure in the weak direction was so clearly a stability failure that it was unnecessary to carry on till the test specimen was destroyed.

For each column a prism was cut off - as mentioned in section 3 - for determining the prism strength, modulus of elasticity in compression, density, and moisture content, and for long columns the modulus of elasticity in bending $E_b$ was determined.

## 5. TEST RESULTS

Fig. 5.1 a and b show typical load-deflection curves for one of the columns tested for deflection in both directions. $u_1$ and $u_3$ are the displacements measured at the quarter points, while $u_2$ is measured in the centre, cf. fig. 4.1.

The Southwell-plots corresponding to fig. 5.1 (based on the displacement $u_2$), cf. section 2, are given in fig. 5.2 a and b. The figure shows how the Euler force $P_{E,S}$ (S for Southwell) and the initial eccentricity are determined.

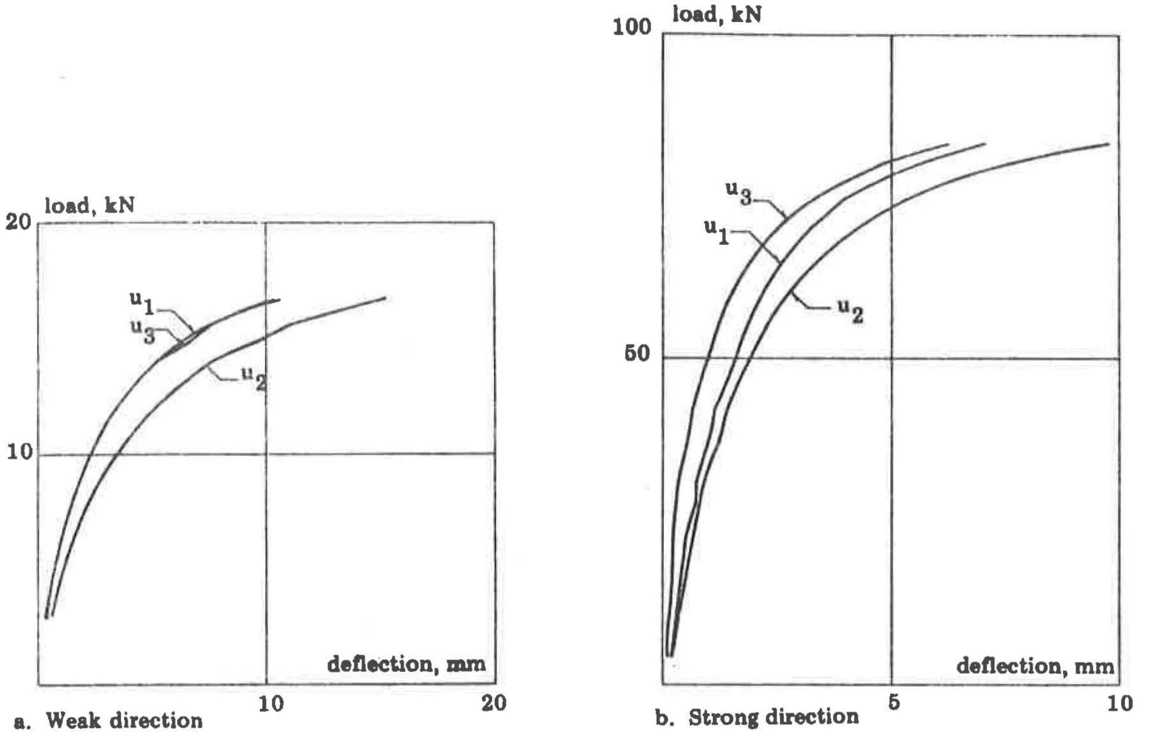


Fig. 5.1. Load-deflection curves

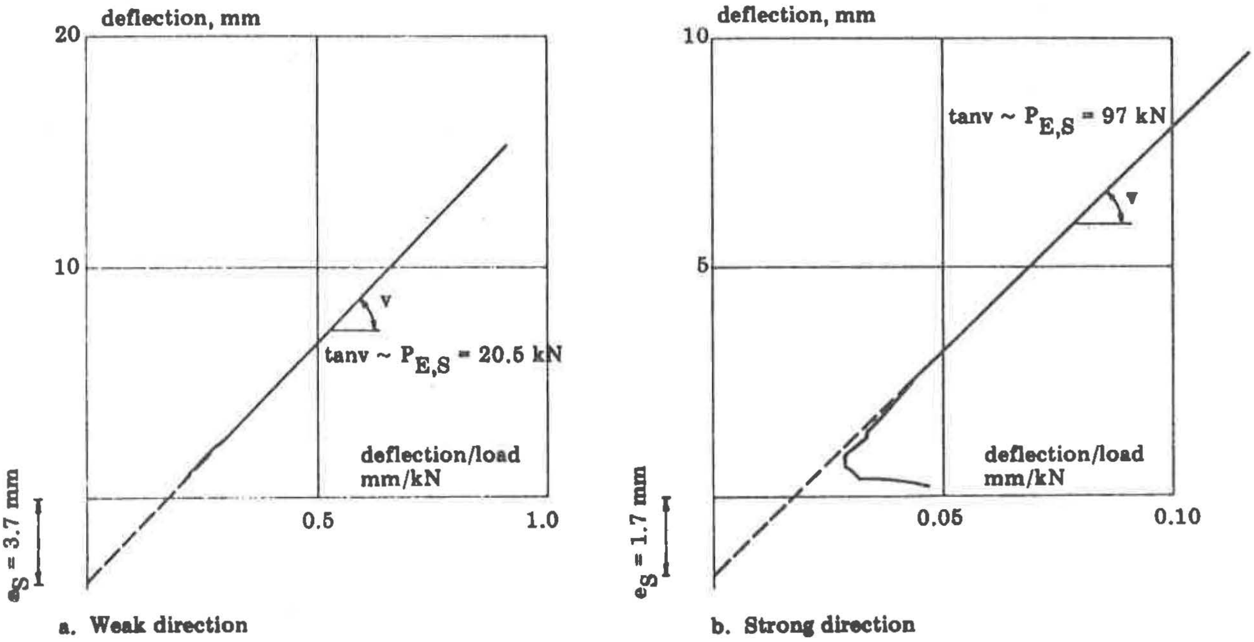


Fig. 5.2. Southwell-plots

The main results are given in table 5.1 (at the end of this paper). The following comments are given to the heads of the columns of the table:

*Columns 1-6* contain general information. As mentioned in section 3, the compression strength $s_c$ has been determined on 200 mm long prisms. Section 3 is also referred to for determination of $E_b$. Edgewise bending was applied, and tests with both narrow sides stressed in tension were carried out. The two values varied slightly; the table states the lower of the two values.

*Columns 7-16* deal with bending in the weak direction, while *columns 17-26* contain the same information for bending in the strong direction.

*Columns 8 and 18* state $e_m$, i.e. the measured, geometric eccentricity in the middle. $e_m$ is measured with a slide gauge (0.1 mm) from a string, held against the ends of the column.

*Columns 9 and 19* state the eccentricity in the middle determined by the Southwell-plot, and *columns 10 and 20* give the corresponding values in relation to core radius (h/6, b/6, respectively).

*Columns 11 and 21* give the ultimate load. For bending in the weak direction for the columns 13-27 the ultimate load was estimated from the load-deflection curves, as the tests were not carried on till rupture (because it was desired also to test the columns in the strong direction).

*Columns 12 and 22* give the Euler load determined by the Southwell-plots, while *columns 13 and 23* give the corresponding moduli of elasticity ($P_{E,S} = \pi^2 E_S I/\ell^2$).

*Columns 13-15 and 23-25* give relations between the values mentioned above. $\sigma_{ult}$ corresponds to $P_{ult}$ divided by the area determined by column 3.

* * *

Fig. 5.3 shows correspondent values of $e_m$ and $e_S$. It is seen that there is only a slight dependence between the values.

The geometric eccentricities allowed in relation to the grading rules are considerably larger than those measured.

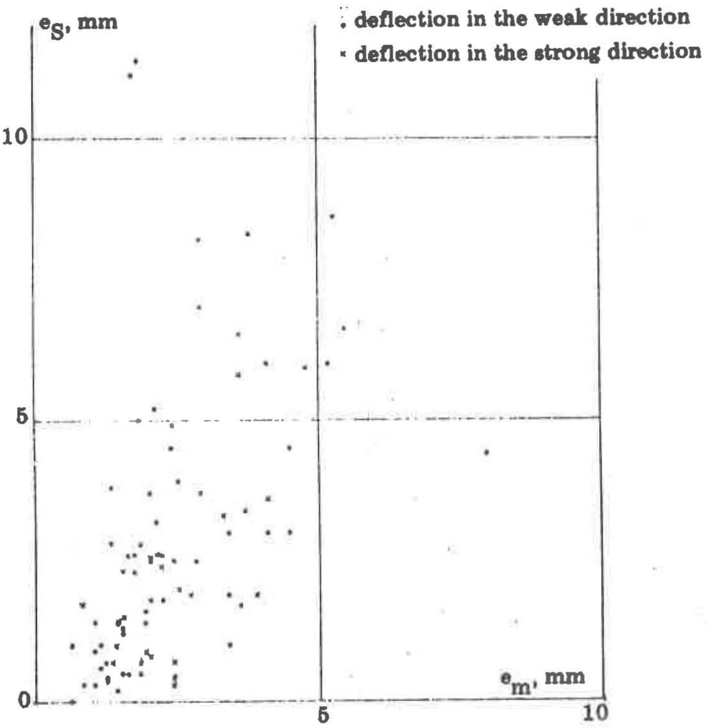


Fig. 5.3

Fig. 5.4 shows the relative eccentricity $\epsilon$ (based on $e_S$, cf. columns 10 and 20 in table 5.1) dependent upon the slenderness ratio $\lambda$. For each series maximum and minimum values and the average figure are stated. It is seen that there is neither basis for distinguishing between the two grades T-300 and unclassified, nor between the two directions of bending.

The figure contains the dependence used in the Netherlands for the best grade of structural timber (Constructiehout),

$$\epsilon = 0.10 + 0.005\,\lambda \tag{9}$$

which is seen to cover the test results very satisfactorily, the essential thing here being the maximum values and not the average values. In Great Britain a considerably lower value is applied, namely $\epsilon$ between 0.001 $\lambda$ and 0.003 $\lambda$.

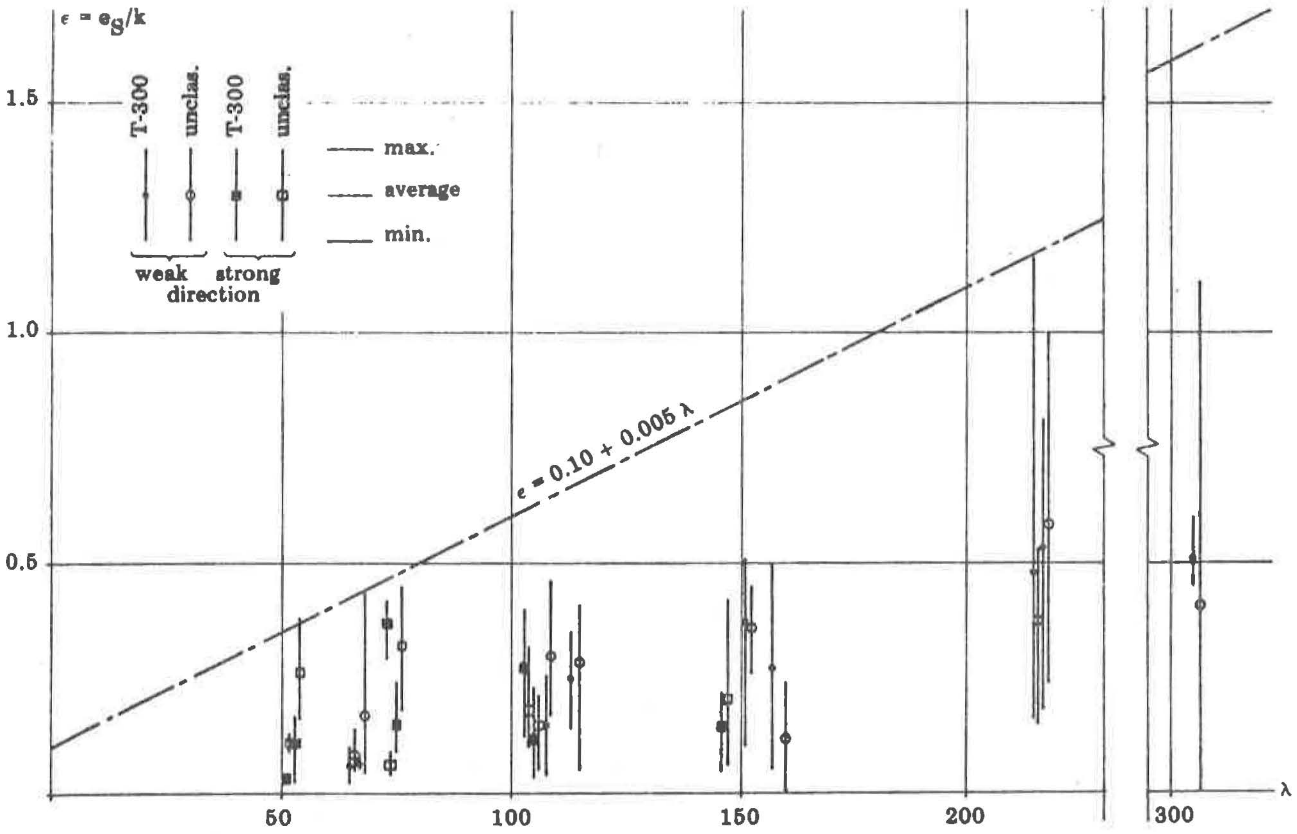


Fig. 5.4

In table 5.1, column 24, the ratio between $E_S$ and $E_b$ for bending in the strong direction is stated. The average figure is 0.965 with a deviation of 0.77 (coefficient of variation = 8.0%). Considering that $E_b$ is determined over a short length in the middle, and $E_S$ is determined from the whole column, the conformity is absolutely satisfactory.

For $E_S$ corresponding to the weak direction the average ratio is 0.933 with a deviation of 0.061 (corresponding to 6.5%). As $E_b$ has been determined by edgewise bending, the conformity must be considered satisfactory in this case too.

* * *

Fig. 5.5 shows the ratio $\sigma_{ult}/s_c$ dependent upon $\lambda$. Although $\lambda$ is slightly different for 50 × 100 and 63 × 125 in corresponding series, the values have been plotted in common. The figure gives mean, maximum, and minimum values.

It is seen that the values for unclassified timber are generally lower than those for T-300 and show a somewhat greater variation. (The reason for the latter may be that $s_c$ does not fully reflect the quality of the timber in the columns, cf. what was stated in section 3).

It is seen that the variation is greatest for small $\lambda$-values, but relatively, the relation between maximum and minimum value is fairly uniform, about 1.4 for T-300 and 1.7 for unclassified timber.

The theoretical curves corresponding to an $\epsilon$-variation as stated in (9) have been drawn in for comparison.

In conformity with the findings in section 3, $E \sim 11800\ N/mm^2$ and $s_c \sim 40\ N/mm^2$, i.e. $E/s_c \sim 300$ have been assumed for both grades.

For similar unclassified timber the bending strength $s_b$ (average figure) has been determined by previous tests [6] as approx. 32 $N/mm^2$, i.e. $s_c/s_b \sim 1.25$. For T-300 it is estimated, among other things, on the basis of [1] and [3], that $s_c/s_b$ is about 0.85.

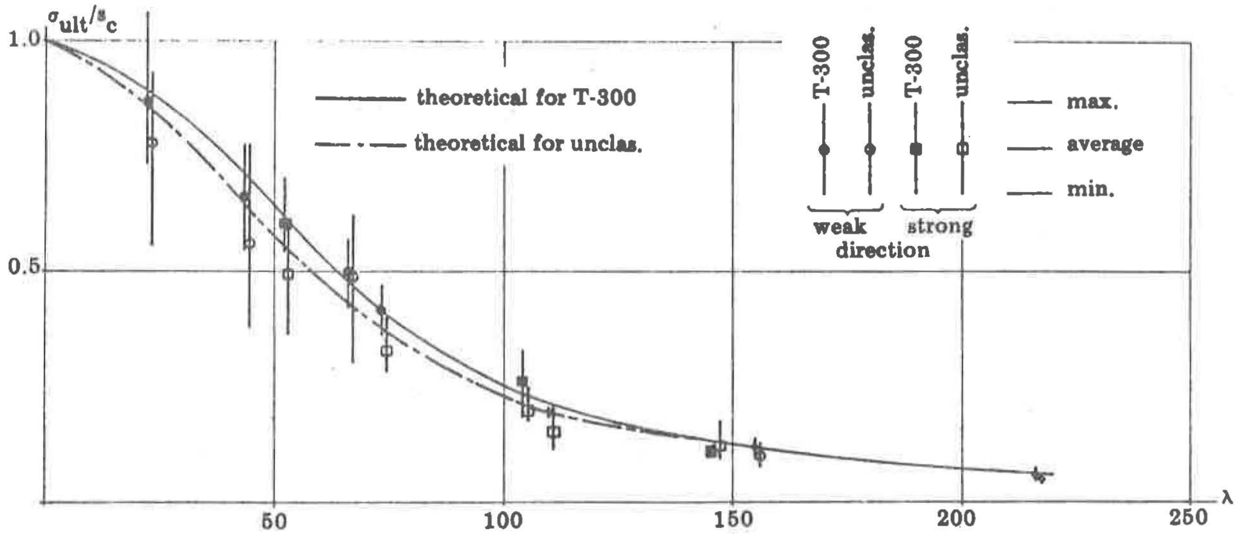


Fig. 5.5

The conformity between theory and tests is extremely satisfactory for T-300 and acceptable for unclassified timber. In the latter case the theoretical curve generally lies a little too high.

Complete conformity between column tests and theory could be obtained by applying a 15-20% lower $s_c$-value, which would also be in better conformity with the values normally found.

* * *

Fig. 5.6 shows the experimentally determined values for the ratio $P_{ult}/P_{E,S}$, and the corresponding theoretical curves determined from the same assumptions as mentioned above.

It is seen that in this case the experimental values are equal for the two timber grades, while T-300 should theoretically be the higher.

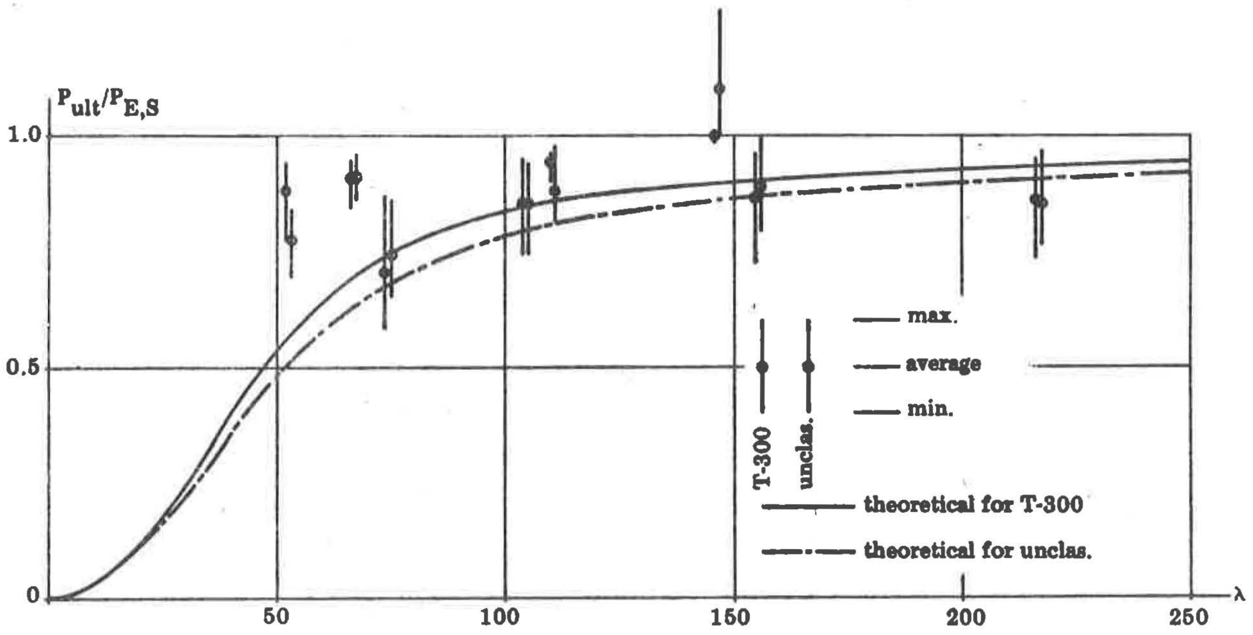


Fig. 5.6

## 6. SYMBOLS

| | | |
|---|---|---|
| $E$ | modulus of elasticity | |
| | $E_c$ | for compression |
| | $E_b$ | in bending |
| | $E_S$ | corresponding to $P_{E,S}$ |
| $I$ | moment of inertia | |
| $P$ | column force | |
| | $P_E$ | Euler load $P_E = \pi^2 EI/\ell^2$ |
| | $P_{E,S}$ | $P_E$ determined by Southwell-plot |
| | $P_{ult}$ | ultimate load force |
| $a$ | constant in the eccentricity expression (2) | |
| $e$ | eccentricity | |
| | $e_S$ | $e$ determined by Southwell-plot |
| $f(\lambda)$ | term in the eccentricity expression (2) | |
| $i$ | radius of gyration | |
| | $i_{min}$ | corresponding to the weak direction |
| | $i_{max}$ | corresponding to the strong direction |
| $k$ | core radius | |
| $k_E$ | $s_E/s_c$ | |
| $\ell$ | column length, buckling length | |
| $s$ | strength parameters corresponding to normal stresses | |
| | $s_c$ | by compression |
| | $s_b$ | in bending |
| | $s_{cr}$ | critical stress |
| | $s_E$ | Euler stress |
| $u$ | deflection | |
| | $u_1$, $u_2$, and $u_3$: see fig. 4.1 | |
| $\lambda$ | slenderness ratio $(\ell/i)$ | |
| | $\lambda_{min} = \ell/i_{min}$ | |
| | $\lambda_{max} = \ell/i_{max}$ | |
| $\epsilon$ | relative eccentricity, $\epsilon = e/k$ | |
| $\sigma$ | normal stresses | |
| | $\sigma_c$ | compression stress from axial force |
| | $\sigma_b$ | bending stress |

## 7. LITERATURE

[1] M. Foslie & K. Moen: Norsk granvirkes styrkeegenskaber 1. Norsk Treteknisk Institutt, Meddelelse nr. 33, Oslo 1968.

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[5] H. J. Larsen: The Design of Solid Timber Columns. CIB-Timber, Structures, September 1973. (Also as Report R 7406 from Aalborg University Center, Pure and Applied Mechanics).

[6] H. J. Larsen og H. Riberholt: Forsøg med uklassificeret konstruktionstræ. Structural Research Laboratory, Technical University of Denmark. Rapport R 31, 1972.

**Table 5.1**
**Test Results**

| General | | | | | | | Deflection in the weak direction | | | | | | | | | | Deflection in the strong direction | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 1 |
| test | grade | dimension | | $f_c$ | $E_b$ | l | λ | $e_m$ | $e_S$ | $\epsilon = \frac{e_S}{k}$ | $P_{ult}$ | $P_{E,S}$ | $E_S$ | $\frac{E_S}{E_b}$ | $\frac{P_{ult}}{P_{E,S}}$ | $\frac{\sigma_{ult}}{f_c}$ | λ | $e_m$ | $e_S$ | $\epsilon = \frac{e_S}{k}$ | $P_{ult}$ | $P_{E,S}$ | $E_S$ | $\frac{E_S}{E_b}$ | $\frac{P_{ult}}{P_{E,S}}$ | $\frac{\sigma_{ult}}{f_c}$ | test |
| | u = unclassified | | | compression strength | bending E-modulus | column length | slenderness ratio | measured e | Southwell e | rel. e | | Southwell $P_E$ | Southwell E | | | | | | | | | | | | | | |
| no. | | mm | mm | N/mm² | N/mm² | mm | | mm | mm | | kN | kN | N/mm² | | | | | mm | mm | | kN | kN | N/mm² | | | | no. |
| 1 - 1 | T-300 | 94.1 | 43.7 | 37.9 | | 288 | 23 | | | | 122 | | | | | 0.78 | | | | | | | | | | | 1 - 1 |
| 2 | | 93.3 | 44.2 | 34.2 | | | | | | | 150 | | | | | 1.06 } 0.86 | | | | | | | | | | | 2 |
| 3 | | 92.2 | 43.6 | 45.5 | | | | | | | 133 | | | | | 0.73 | | | | | | | | | | | 3 |
| 2 - 1 | T-300 | 121.8 | 58.5 | 43.8 | | 364 | 22 | | | | 266 | | | | | 0.85 | | | | | | | | | | | 2 - 1 |
| 2 | | 122.2 | 58.3 | 42.8 | | | | | | | 292 | | | | | 0.96 } 0.86 | | | | | | | | | | | 2 |
| 3 | | 121.7 | 58.4 | 36.3 | | | | | | | 200 | | | | | 0.77 | | | | | | | | | | | 3 |
| 3 - 1 | u | 92.2 | 43.8 | 36.9 | | 288 | 23 | | | | 135 | | | | | 0.91 | | | | | | | | | | | 3 - 1 |
| 2 | | 93.2 | 44.0 | 33.2 | | | | | | | 150 | | | | | 0.80 } 0.75 | | | | | | | | | | | 2 |
| 3 | | 94.2 | 44.5 | 37.0 | | | | | | | 85 | | | | | 0.55 | | | | | | | | | | | 3 |
| 4 - 1 | u | 122.2 | 58.7 | 28.0 | | 364 | 22 | | | | 185 | | | | | 0.93 | | | | | | | | | | | 4 - 1 |
| 2 | | 122.2 | 58.6 | 39.1 | | | | | | | 199 | | | | | 0.71 } 0.80 | | | | | | | | | | | 2 |
| 3 | | 121.3 | 58.2 | 43.9 | | | | | | | 234 | | | | | 0.76 | | | | | | | | | | | 3 |
| 5 - 1 | T-300 | 93.7 | 44.2 | 35.5 | | 576 | 45 | | | | 113 | | | | | 0.75 | | | | | | | | | | | 5 - 1 |
| 2 | | 93.0 | 44.1 | 51.2 | | | | | | | 115 | | | | | 0.55 } 0.67 | | | | | | | | | | | 2 |
| 3 | | 93.3 | 44.2 | 39.8 | | | | | | | 115 | | | | | 0.70 | | | | | | | | | | | 3 |
| 6 - 1 | T-300 | 122.2 | 58.5 | 43.6 | | 728 | 43 | | | | 240 | | | | | 0.77 | | | | | | | | | | | 6 - 1 |
| 2 | | 122.3 | 58.4 | 55.3 | | | | | | | 248 | | | | | 0.63 } 0.65 | | | | | | | | | | | 2 |
| 3 | | 121.8 | 58.6 | 45.5 | | | | | | | 175 | | | | | 0.54 | | | | | | | | | | | 3 |
| 7 - 1 | u | 94.8 | 44.3 | 41.4 | | 576 | 45 | 1.0 | | | 89.0 | | | | | 0.51 | | | | | | | | | | | 7 - 1 |
| 2 | | 93.8 | 44.7 | 41.5 | | | | 1.0 | | | 65.0 | | | | | 0.37 } 0.46 | | | | | | | | | | | 2 |
| 3 | | 94.5 | 44.0 | 40.9 | | | | 1.3 | | | 85.0 | | | | | 0.50 | | | | | | | | | | | 3 |
| 8 - 1 | u | 121.2 | 58.6 | 42.0 | | 728 | 43 | 1.1 | | | 159 | | | | | 0.53 | | | | | | | | | | | 8 - 1 |
| 2 | | 122.2 | 58.5 | 31.8 | | | | 0.8 | | | 174 | | | | | 0.77 } 0.66 | | | | | | | | | | | 2 |
| 3 | | 121.7 | 58.6 | 42.5 | | | | 0.0 | | | 208 | | | | | 0.69 | | | | | | | | | | | 3 |
| 9 - 1 | T-300 | 93.8 | 44.6 | 36.1 | | 864 | 68 | 1.1 | 0.6 | 0.08 | 63.6 | 66.7 | 7250 | | 0.95 | 0.42 | | 0.9 | | | | | | | | | 9 - 1 |
| 2 | | 93.3 | 44.0 | 42.4 | | | | 1.6 | 0.5 | 0.07 | 94.5 | 103 | 12000 | | 0.92 } 0.92 | 0.54 } 0.51 | | 1.0 | | | | | | | | | 2 |
| 3 | | 93.8 | 44.0 | 39.3 | | | | 1.2 | 0.4 | 0.05 | 93.0 | 103 | 11700 | | 0.90 | 0.57 | | 0.7 | | | | | | | | | 3 |
| 10 - 1 | T-300 | 121.6 | 58.2 | 34.9 | | 1092 | 65 | 0.6 | 1.0 | 0.10 | 122 | 145 | 8750 | | 0.84 | 0.49 | | 1.1 | | | | | | | | | 10 - 1 |
| 2 | | 121.5 | 58.1 | 35.7 | | | | 1.2 | 0.7 | 0.07 | 136 | 149 | 9050 | | 0.91 } 0.89 | 0.54 } 0.49 | | 1.6 | | | | | | | | | 2 |
| 3 | | 121.9 | 58.4 | 40.6 | | | | 1.4 | 0.2 | 0.02 | 131 | 143 | 8500 | | 0.92 | 0.45 | | 1.0 | | | | | | | | | 3 |
| 11 - 1 | u | 93.5 | 44.2 | 29.0 | | 864 | 68 | 1.0 | 0.3 | 0.04 | 71.0 | 74.0 | 8300 | | 0.96 | 0.59 | | 0.9 | | | | | | | | | 11 - 1 |
| 2 | | 92.9 | 44.3 | 42.8 | | | | 0.8 | 0.3 | 0.04 | 70.0 | 74.5 | 8350 | | 0.94 } 0.92 | 0.39 } 0.43 | | 1.6 | | | | | | | | | 2 |
| 3 | | 93.4 | 44.1 | 49.5 | | | | 2.1 | 3.2 | 0.44 | 62.5 | 72.0 | 8150 | | 0.87 | 0.30 | | 1.5 | | | | | | | | | 3 |
| 12 - 1 | u | 121.9 | 58.3 | 35.2 | | 1092 | 65 | 1.9 | 1.4 | 0.14 | 107 | 124 | 745 | | 0.86 | 0.42 | | 1.1 | | | | | | | | | 12 - 1 |
| 2 | | 121.7 | 58.4 | 33.3 | | | | 1.5 | 0.5 | 0.05 | 148 | 161 | 965 | | 0.92 } 0.90 | 0.62 } 0.55 | | 0.9 | | | | | | | | | 2 |
| 3 | | 121.9 | 58.2 | 41.2 | | | | 1.8 | 0.5 | 0.05 | 178 | 194 | 11700 | | 0.92 | 0.61 | | 1.2 | | | | | | | | | 3 |
| 13 - 1 | T-300 | 93.6 | 44.0 | 41.3 | | 1440 | 113 | 1.1 | 1.0 | 0.14 | 35.5 | 37.0 | 11650 | | 0.96 | 0.21 | 53 | 2.4 | 0.3 | 0.02 | 94 | 96.0 | 6700 | | 0.98 | 0.55 | 13 - 1 |
| 2 | | 94.1 | 44.4 | 29.9 | | | | 1.7 | 2.6 | 0.35 | 26.0 | 29.0 | 8950 | | 0.90 } 0.93 | 0.21 } 0.20 | | 1.7 | 2.3 | 0.15 | 87 | 113.5 | 7750 | | 0.77 } 0.84 | 0.70 } 0.60 | 2 |
| 3 | | 94.4 | 44.5 | 32.9 | | | | 1.5 | 1.2 | 0.16 | 25.5 | 27.0 | 8200 | | 0.94 | 0.19 | | 2.1 | 2.6 | 0.17 | 75 | 98.0 | 6600 | | 0.77 | 0.54 | 3 |
| 14 - 1 | T-300 | 122.3 | 58.5 | 40.5 | | 1820 | 108 | 1.4 | 1.4 | 0.14 | 53.0 | 56.5 | 9350 | | 0.94 | 0.18 | 52 | 1.3 | 0.7 | 0.03 | 175 | 193 | 7250 | | 0.91 | 0.60 | 14 - 1 |
| 2 | | 122.4 | 58.3 | 52.0 | | | | 1.0 | 0.4 | 0.04 | 73.0 | 76.0 | 12600 | | 0.96 } 0.95 | 0.20 } 0.19 | | 1.0 | - | - | - | - | - | | - } 0.92 | 0.60 } 0.61 | 2 |
| 3 | | 121.8 | 58.5 | 41.0 | | | | 2.8 | 2.5 | 0.26 | 54.5 | 57.0 | 9450 | | 0.96 | 0.19 | | 1.8 | 0.7 | 0.03 | 181 | 193 | 7350 | | 0.94 | 0.62 | 3 |
| 15 - 1 | u | 92.9 | 43.6 | 45.9 | | 1440 | 115 | 4.5 | 3.0 | 0.41 | 21.0 | 23.5 | 7700 | | 0.89 | 0.113 | 53 | 2.4 | 2.5 | 0.16 | 80 | 106 | 7650 | | 0.75 | 0.43 | 15 - 1 |
| 2 | | 93.6 | 44.4 | 38.5 | | | | 1.2 | 0.4 | 0.05 | 31.0 | 31.5 | 9750 | | 0.98 } 0.89 | 0.193 } 0.136 | | 2.5 | 3.9 | 0.25 | 78 | 104 | 7200 | | 0.75 } 0.73 | 0.49 } 0.43 | 2 |
| 3 | | 93.7 | 42.3 | 42.4 | | | | 4.1 | 3.0 | 0.41 | 19.0 | 23.5 | 8400 | | 0.81 | 0.113 | | 4.1 | 6.0 | 0.38 | 61 | 88.0 | 6350 | | 0.69 | 0.36 | 3 |
| 16 - 1 | u | 122.0 | 58.6 | 45.7 | | 1820 | 108 | 4.5 | 4.5 | 0.46 | 47.0 | 54.5 | 8950 | | 0.86 | 0.144 | 52 | 1.6 | 2.6 | 0.13 | 153 | 196 | 7400 | | 0.78 | 0.47 | 16 - 1 |
| 2 | | 122.0 | 58.8 | 46.0 | | | | 1.8 | 2.8 | 0.29 | 56.0 | 64.5 | 10500 | | 0.87 } 0.87 | 0.170 } 0.176 | | 2.2 | 2.4 | 0.12 | 190 | 235 | 8850 | | 0.81 } 0.81 | 0.58 } 0.55 | 2 |
| 3 | | 122.7 | 58.6 | 35.2 | | | | 3.6 | 1.7 | 0.17 | 54.0 | 61.0 | 10000 | | 0.89 | 0.213 | | 2.7 | 1.9 | 0.09 | 150 | 178 | 6600 | | 0.84 | 0.59 | 3 |
| 17 - 1 | T-300 | 93.2 | 44.1 | 33.1 | 13100 | 2020 | 157 | 2.4 | 0.4 | 0.05 | 20.0 | 21.0 | 12900 | 0.98 | 0.95 | 0.144 | 75 | 1.3 | 3.8 | 0.24 | 64 | 89.0 | 12350 | 0.94 | 0.72 | 0.47 | 17 - 1 |
| 2 | | 93.3 | 44.1 | 49.3 | 14200 | | | 2.9 | 3.7 | 0.50 | 18.0 | 20.5 | 12750 | 0.90 } 0.97 | 0.88 } 0.89 | 0.087 } 0.114 | | 0.8 | 1.7 | 0.11 | 84 | 97.0 | 13400 | 0.94 } 0.95 | 0.87 } 0.80 | 0.42 } 0.42 | 2 |
| 3 | | 94.6 | 45.7 | 34.2 | 10100 | | | 3.4 | 1.9 | 0.26 | 16.0 | 19.0 | 10400 | 1.03 | 0.84 | 0.111 | | 1.4 | 1.4 | 0.09 | 62 | 77.0 | 9850 | 0.98 | 0.81 | 0.37 | 3 |
| 18 - 1 | T-300 | 121.9 | 58.2 | 31.4 | 12500 | 2550 | 152 | 1.8 | 5.0 | 0.51 | 26.0 | 33.5 | 10950 | 0.88 | 0.78 | 0.118 | 73 | 5.3 | 8.6 | 0.42 | 94 | 156 | 11700 | 0.94 | 0.60 | 0.42 | 18 - 1 |
| 2 | | 121.2 | 58.1 | 44.3 | 17400 | | | 2.4 | 4.9 | 0.50 | 33.0 | 46.0 | 15250 | 0.88 } 0.89 | 0.72 } 0.82 | 0.106 } 0.120 | | 3.8 | 8.3 | 0.41 | 112 | 192 | 14700 | 0.84 } 0.89 | 0.58 } 0.60 | 0.36 } 0.39 | 2 |
| 3 | | 122.1 | 58.1 | 35.1 | 13300 | | | 3.4 | 1.0 | 0.10 | 34.0 | 35.5 | 11700 | 0.90 | 0.96 | 0.137 | | 4.8 | 5.9 | 0.29 | 98 | 158 | 11800 | 0.89 | 0.62 | 0.39 | 3 |
| 19 - 1 | u | 93.2 | 44.0 | 34.1 | 8300 | 2020 | 160 | 2.2 | 1.8 | 0.24 | 12.5 | 13.5 | 8500 | 1.02 | 0.93 | 0.090 | 75 | 1.3 | 2.8 | 0.18 | 50 | 58.0 | 8100 | 0.98 | 0.86 | 0.36 | 19 - 1 |
| 2 | | 93.8 | 43.3 | 26.8 | 9800 | | | 1.0 | 0.9 | 0.12 | 14.0 | 14.7 | 9600 | 0.98 } 1.01 | 0.95 } 0.96 | 0.131 } 0.108 | | 2.9 | 7.0 | 0.45 | 43 | 66.0 | 9200 | 0.94 } 0.96 | 0.65 } 0.73 | 0.40 } 0.35 | 2 |
| 3 | | 93.0 | 43.9 | 41.9 | 10900 | | | 0.6 | 0.0 | 0.00 | 17.5 | 17.5 | 11100 | 1.02 | 1.00 | 0.103 | | 2.1 | 5.2 | 0.33 | 51 | 75.0 | 10550 | 0.97 | 0.68 | 0.30 | 3 |
| 20 - 1 | u | 121.6 | 58.0 | 43.0 | 9000 | 2550 | 152 | 2.0 | 2.5 | 0.26 | 23.0 | 26.0 | 8700 | 0.97 | 0.88 | 0.077 | 73 | 2.0 | 1.8 | 0.09 | 86 | 109 | 8250 | 0.92 | 0.79 | 0.28 | 20 - 1 |
| 2 | | 121.8 | 58.3 | 41.3 | 11900 | | | 2.0 | 3.7 | 0.38 | 29.0 | 36.5 | 11900 | 1.00 } 0.94 | 0.79 } 0.83 | 0.099 } 0.088 | | 1.9 | 0.9 | 0.04 | 102 | 131 | 9850 | 0.83 } 0.85 | 0.78 } 0.75 | 0.35 } 0.31 | 2 |
| 3 | | 121.6 | 58.2 | 50.6 | 15100 | | | 8.0 | 4.4 | 0.45 | 32.0 | 38.5 | 12750 | 0.84 | 0.83 | 0.088 | | 1.5 | 1.5 | 0.07 | 108 | 162 | 12250 | 0.81 | 0.67 | 0.30 | 3 |
| 21 - 1 | T-300 | 95.0 | 45.7 | 41.5 | 14500 | 2880 | 218 | 2.4 | 4.5 | 0.61 | 9.9 | 11.6 | 12900 | 0.89 | 0.85 | 0.055 | 105 | 2.4 | 0.4 | 0.03 | 53.5 | 56.5 | 14550 | 1.00 | 0.95 | 0.29 | 21 - 1 |
| 2 | | 95.6 | 45.9 | 34.0 | 13200 | | | 1.5 | 1.3 | 0.18 | 11.3 | 11.9 | 13000 | 0.98 } 0.92 | 0.95 } 0.88 | 0.075 } 0.057 | | 1.5 | 1.3 | 0.08 | 49 | 52.5 | 13200 | 1.00 } 0.97 | 0.93 } 0.91 | 0.33 } 0.27 | 2 |
| 3 | | 94.8 | 45.7 | 36.7 | 9800 | | | 5.2 | 6.0 | 0.81 | 6.7 | 7.9 | 8800 | 0.90 | 0.85 | 0.042 | | 4.1 | 3.6 | 0.23 | 29 | 34.5 | 8950 | 0.91 | 0.84 | 0.18 | 3 |
| 22 - 1 | T-300 | 121.8 | 58.4 | 35.1 | 12400 | 3640 | 216 | 1.6 | 1.6 | 0.16 | 18.0 | 19.0 | 12750 | 1.03 | 0.95 | 0.071 | 104 | 2.0 | 2.5 | 0.12 | 78 | 90.0 | 13750 | 1.11 | 0.87 | 0.31 | 22 - 1 |
| 2 | | 121.4 | 58.2 | 47.0 | 14900 | | | 9.2 | 11.1 | 1.14 | 14.0 | 17.5 | 11700 | 0.79 } 0.90 | 0.80 } 0.83 | 0.042 } 0.054 | | 3.6 | 5.8 | 0.29 | 66 | 89.0 | 13800 | 0.93 } 1.02 | 0.74 } 0.79 | 0.20 } 0.26 | 2 |
| 3 | | 121.7 | 58.3 | 35.9 | 13400 | | | 10.8 | 11.3 | 1.16 | 13.0 | 17.7 | 11800 | 0.88 | 0.73 | 0.050 | | 2.9 | 8.2 | 0.40 | 66 | 88.0 | 13500 | 1.01 | 0.75 | 0.26 | 3 |
| 23 - 1 | u | 94.9 | 45.9 | 39.0 | 10200 | 2880 | 218 | 5.0 | 7.4 | 1.00 | 6.8 | 8.9 | 9800 | 0.96 | 0.76 | 0.040 | 105 | 2.4 | 0.7 | 0.05 | 40 | 42.5 | 10950 | 1.07 | 0.94 | 0.24 | 23 - 1 |
| 2 | | 94.9 | 45.6 | 51.8 | 12400 | | | 2.4 | 1.8 | 0.24 | 8.8 | 9.8 | 11000 | 0.89 } 0.93 | 0.90 } 0.84 | 0.040 } 0.045 | | 3.4 | 3.0 | 0.19 | 39 | 47.0 | 12200 | 0.98 } 1.01 | 0.83 } 0.87 | 0.17 } 0.22 | 2 |
| 3 | | 94.6 | 45.6 | 36.4 | 12300 | | | 3.3 | 3.7 | 0.50 | 8.8 | 10.4 | 11700 | 0.95 | 0.85 | 0.054 | | 3.3 | 3.3 | 0.21 | 39 | 46.5 | 12150 | 0.99 | 0.84 | 0.25 | 3 |
| 24 - 1 | u | 121.7 | 58.6 | 43.7 | 12500 | 3640 | 216 | 2.3 | 5.2 | 0.53 | 13.5 | 17.0 | 11050 | 0.88 | 0.80 | 0.042 | 104 | 3.6 | 6.5 | 0.32 | 61.5 | 76.0 | 11600 | 0.93 | 0.81 | 0.17 | 24 - 1 |
| 2 | | 121.6 | 58.4 | 37.7 | 11800 | | | 5.0 | 4.4 | 0.45 | 14.5 | 16.5 | 10900 | 0.92 } 0.91 | 0.88 } 0.88 | 0.054 } 0.047 | | 1.5 | 2.3 | 0.11 | 55 | 74.0 | 11350 | 0.96 } 0.92 | 0.74 } 0.82 | 0.20 } 0.18 | 2 |
| 3 | | 121.8 | 58.3 | 51.8 | 12700 | | | 2.3 | 1.5 | 0.15 | 17.0 | 17.5 | 11800 | 0.93 | 0.97 | 0.046 | | 2.5 | 2.0 | 0.10 | 64.5 | 71.5 | 10950 | 0.86 | 0.90 | 0.17 | 3 |
| 25 - 1 | T-300 | 94.9 | 46.0 | 41.7 | 12600 | 4030 | 305 | 2.2 | 3.4 | 0.47 | 4.8 | 5.2 | 11100 | 0.88 | 0.92 | 0.030 | 147 | 2.2 | 2.6 | 0.17 | 22 | 24.5 | 12350 | 0.98 | 0.90 | 0.120 | 25 - 1 |
| 2 | | 95.1 | 45.8 | 42.2 | 15300 | | | 3.6 | 3.3 | 0.45 | 6.1 | 6.6 | 14250 | 0.93 } 0.91 | 0.92 } 0.91 | 0.033 } 0.030 | | 3.7 | 3.4 | 0.22 | 23 | 28.5 | 14400 | 0.94 } 0.98 | 0.81 } 0.88 | 0.126 } 0.123 | 2 |
| 3 | | 94.9 | 45.7 | 41.0 | 12300 | | | 2.4 | 4.4 | 0.60 | 4.7 | 5.2 | 11350 | 0.92 | 0.90 | 0.026 | | 2.0 | 0.8 | 0.05 | 23 | 24.5 | 12400 | 1.01 | 0.94 | 0.124 | 3 |
| 27 - 1 | u | 95.1 | 46.0 | 42.5 | 11900 | 4030 | 307 | 4.9 | 1.0 | 0.13 | 5.5 | 5.6 | 11850 | 1.00 | 0.98 | 0.029 | 147 | 3.9 | 1.9 | 0.12 | 21 | 24.5 | 12300 | 1.03 | 0.86 | 0.108 | 27 - 1 |
| 2 | | 95.1 | 45.4 | 35.2 | 13100 | | | 9.6 | 8.4 | 1.11 | 4.4 | 4.9 | 10950 | 0.84 } 0.95 | 0.90 } 0.96 | 0.028 } 0.028 | | 1.4 | 1.0 | 0.06 | 28 | 33.0 | 16650 | 1.27 } 1.10 | 0.84 } 0.85 | 0.185 } 0.129 | 2 |
| 3 | | 95.2 | 45.2 | 49.9 | 12500 | | | 5.8 | 0.0 | 0.00 | 5.6 | 5.6 | 12650 | 1.01 | 1.00 | 0.026 | | 5.5 | 6.6 | 0.42 | 21 | 25.0 | 12550 | 1.00 | 0.84 | 0.096 | 3 |

PAPER 4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# LATERAL-TORSIONAL BUCKLING OF ECCENTRICALLY LOADED TIMBER COLUMNS

by

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STOCKHOLM, SWEDEN

PARIS - FEBRUARY 1975

LATERAL-TORSIONAL BUCKLING OF ECCENTRICALLY LOADED TIMBER COLUMNS

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## INTRODUCTION

The problem of lateral-torsional buckling of compression members in timber structures has received little interest. As far as the writer knows there are no test results published but for the related problem of lateral buckling of timber beams in bending. /1/ /2/ /3/. A brief summary of the results of these tests is that they show that the theory of elastic lateral buckling is applicable on timber beams. If correct values of the elastic constants are inserted in the solutions, the theory will accurately predict the buckling load. Hence there are good reasons for expecting the theory of lateral-torsional buckling to render useful results as far as elastic conditions are concerned.

Inelastic conditions have been reached in some of the above-mentioned tests but the results are of little use because the bending strength of the test beams is not known. Accordingly, little is known about inelastic conditions. The influence of initial out-of-straightness is also a white spot as far as test results are concerned.

The present paper is intended to shed some light on the possibilities of lateral-torsional buckling to occur in timber structures and to form a basis for a further discussion.

## BASIC THEORY

The basic theory of lateral-torsional buckling has been presented in a great number of texts of which reference is given to /4/. In order to reduce the complexity of the problem it is assumed that the member has a uniform, doubly-symmetric cross-section e.g. rectangular or I-shaped with equal flanges. The basic case of loading is a central thrust combined with a constant bending moment in the plane of maximum stiffness, cf fig 1.



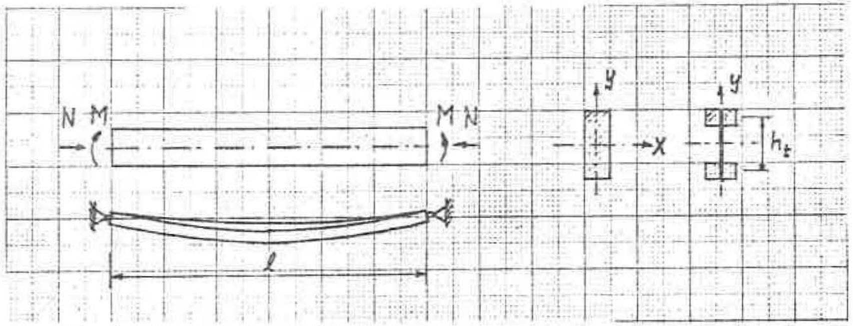

Fig 1. Basic case of loading

The material is assumed to be elastic and the member to be straight immediately before that the lateral-torsional buckling takes place. The latter assumption implies that the member should be slightly pre-curved in the plane of maximim stiffness to account for the pre-buckling deformation of the bending moment. If instead the member is straight before loading, the buckling load may increase or decrease slightly /5/ but it is felt justified to neglect this in order to simplify. The influence of out-of-straightness in lateral direction is discussed later on.

If the member is free to deflect along the span, the theory predicts that the member may become instable for a certain load. The member will start deflecting laterally in combination with twisting at this load. If the member is hinged in its ends in a way that prevents rotation around the longitudinal axis the critical combinations of bending moment M and thrust N are those satisfying the equation

$$\left(1-\frac{N}{N_y}\right)\left(1-\frac{N}{N_t}\right)=\left(\frac{M}{M_{cr}}\right)^2 \qquad (1)$$

in which

$N_y$ = Euler buckling load in lateral direction
$N_t$ = Torsional buckling load
$M_{cr}$ = Lateral buckling moment in absence of thrust

For all practical cases $N_t > N_y$ and hence it is convenient to consider eq (1) as an interaction formula between $N / N_y$ and $M / M_{cr}$ giving different curves for different $N_y / N_t$ as shown in fig 2. For $N_y / N_t = 1$ the interaction curve is straight line and for $N_y/N_t = 0$ it is a parabola with horizontal slope at the left end.

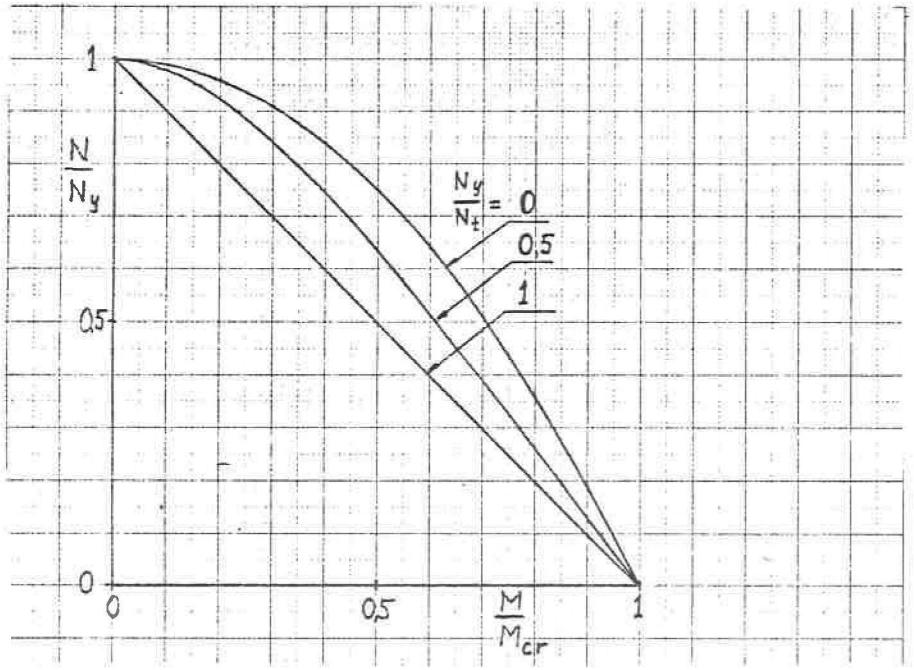


Fig 2. Interaction curves for bending moment and thrust

EFFECT OF VARYING BENDING MOMENTS

For the case of loading with linearily varying bending moment, M in one end and $\mu \cdot M$ in the other the lateral buckling moment in absence of thrust is increased. A good estimate of $M_{cr}$ for this case is /6/

$$M_{cr} = M_{cr,\ \mu = 1} / ( 0.6 + 0.3\mu + 0.1 \mu^2 ) \qquad (2)$$

in which $M_{cr,\ \mu = 1}$ is the lateral buckling moment for constant bending moment. When the bending moment varies along the member the interaction curve will change and also depend on the shape



of the cross-section /7/. However, the deviation from the curves in fig 2 is negligible if the bending moment does not change sign ( $\mu \geq 0$ ) and the error is less than 11 % ( on the conservative side ) for $\mu = -0.5$.

EFFECT OF BOUNDARY CONDITIONS

Other boundary conditions than hinged may approximately be treated by substituting the actual length with a reduced length. For one end clamped in lateral direction the reduced length should be 0.7 l and for both ends clamped 0.5 l. The figures are valid for constant bending moment. If the bending moment varies along the member this approximation may be unconservative if applied when the end with the smaller moment is clamped and the other end hinged. It is believed that a conservative estimate for this case is obtained if the actual length is used when calculating $M_{cr}$ and the reduced length when calculating $N_y$ and $N_t$.

CHOICE OF INTERACTION CURVE

For design purposes a simple interaction formula is to be chosen and the most simple choice is

$$\frac{N}{N_y} + \frac{M}{M_{cr}} = 1 \qquad (3)$$

For steel structure this equation is used, but in some cases with an amplification factor ( $1 / ( 1 - N/N_x )$ ) on the bending moment. This factor is usually only slightly exceeding 1 and it is very likely that neglecting this factor eq (3) will still be conservative. On the contrary it is felt justified to check whether a more favourable curve could be applied for timber structures. To discuss this we need actual values of $N_y / N_t$.

For a rectangular section the following data is used

$I_y = b^3h / 12$

$K_v = 0.3\ b^3h$

$i_p^2 = ( b^2 + h^2 ) / 12 = d^2 / 12$

$h_t = 0$

$E/G = 20$

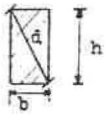


Fig 3.

which render

$$N_y / N_t = 4.6\ d^2 / l^2 \qquad (4)$$

It is clearly seen that eq (4) may hardly exceed $N_y / N_t = 0.2$ for pratical structures. (0.2 corresponds to a member with the length equal to 4.8 times the length of the diagonal of the cross-section ).

For an I-section some more variables enter. With notation according to fig 4 and neglecting the web the following data is obtained

$I_y = b^3h_f / 6$

$K_v = 2c_1\ b\ h_f^3$

$i_p^2 = ( b^2 + h_f^2 ) / 12 + h_t^2 / 4$

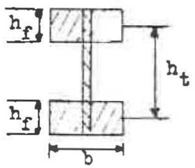


Fig 4.

The coefficient $c_1$ varies between 0.141 and 0.333 as $b / h_f$ varies from 1 to infinity. In the following $b / h_f = 1$ is assumed and again putting $E / G = 20$ we obtain

$$\frac{N_y}{N_t} = \frac{1 + \frac{2}{3}\frac{b^2}{h_t^2}}{1 + 0.034\ \frac{l^2}{h_t^2}} \qquad (5)$$

Some values of $N_y / N_t$ are given in the subsequent table.



| $\frac{l}{h_t} \diagdown \frac{b}{h_t}$ | 0.1 | 0.2 | 0.3 |
|---|---|---|---|
| 5 | 0.54 | 0.55 | 0.57 |
| 7 | 0.38 | 0.38 | 0.40 |
| 10 | 0.23 | 0.23 | 0.24 |

As a variation in the assumed value of $b/h_f$ will give lower figures, $N_y / N_t = 0.5$ may be taken as an upper bound for practical structures.

An interaction curve largely representing the case $N_y / N_t = 0.5$ may be written

$$0.8\ \frac{N}{N_y} + \frac{M}{M_{cr}} = 1 \quad \text{if } 0.2 < \frac{M}{M_{cr}} \leq 1$$

$$\frac{N}{N_y} = 1 \quad \text{if} \quad \frac{M}{M_{cr}} \leq 0.2 \qquad (6)$$

EFFECT OF OUT-OF-STRAIGHTNESS AND INELASTIC BEHAVIOUR

For the design of structures the above solutions may not be used directly. Due regard has to be taken to deviations from the assumptions of elasticity and straightness in lateral direction. Unfortunately there are no results available but for the two special cases in-plane buckling with thrust only and lateral-torsional buckling with bending moment only. As the suggested design procedure will be based on an interaction formula which may be interpreted as an interpolation between those two special cases it is believed that the possible errors will not be too serious.

The in-plane buckling has been discussed in a previous CIB-paper /8/. Only one additional comment to this problem should be given. As far as the writer knows the effect of creep has not been included in a rational manner in any theoretical basis for the design of timber columns. The question of the effect of creep will be left open in this paper too, but it is still felt unsatisfactorily not to know more about it.

An attempt has been presented in /9/ to quantify the influence of inelastic deformations and initial deformations on the lateral buckling of a timber beam loaded in bending. The results indicate that the inelastic deformations due to fibre buckling in the compression zone is of minor importance. The influence of an initial out-of-straightness in lateral direction is less serious in lateral buckling than it is in in-plane buckling of columns. It is shown in a review of different design procedures for lateral buckling of timber beams that this has been accounted for. The critical stresses used as basis for design are closer to those given by the elastic theory than the case is for in-plane buckling of columns.

DESIGN CRITERIA

If eq (3) is chosen as basis for the design and the equation is rewritten in terms of stresses with appropriate safety factors entered, the following design criteria is obtained

$$\frac{\sigma_c}{\sigma_{ca}} + \frac{\sigma_b}{\sigma_{ba}} = 1 \qquad (7)$$

in which

- $\sigma_c$ = computed average compressive stress
- $\sigma_{ca}$ = allowable compressive stress with regard to buckling in lateral direction
- $\sigma_b$ = computed bending stress
- $\sigma_{ba}$ = allowable bending stress with regard to lateral buckling

The interaction formula (7) has the obvious advantage of simplicity and it may be extended to cover in-plane-bending combined with thrust e.g. if the member is braced in lateral direction or if the bending takes place in the plane of minimum stiffness. To cover this case $\sigma_{ca}$ should be taken as the allowable compressive stress with regard to buckling in the plane of bending and $\sigma_{ba}$ as the unreduced allowable bending stress.

If bending moment varies along the member (M in one end and $\mu$M in the other end ) the maximum bending moment in the plane of maximum stiffness will not coincide with the maximum of the unintended bending moment in the plane of minimum stiffness. This is favourable for the capacity of members of small or intermediate slenderness and may be taken into account by the use of an



equivalent constant bending moment e.g. $M_e = (0.6 + 0.4\mu)$ M but not less than 0.4 M. In this case $\sigma_{ba}$ should be taken as for constant bending moment. The use of an equivalent moment is probably a rough estimate but it has the advantage of being simple and the same expression may be used in the design for in-plane bending. A check for the maximum stresses is required in the section with maximum bending moment in addition to the check with eq (7).

A closer approximation of the theory is given by eq (6) which may be rewritten to

$$\begin{aligned} 0.8\frac{\sigma_c}{\sigma_{ca}} + \frac{\sigma_b}{\sigma_{ba}} &\leqq 1 \quad \text{if } 0.2 < \frac{\sigma_b}{\sigma_{ba}} \leqq 1 \\ \frac{\sigma_c}{\sigma_{ca}} &\leqq 1 \quad \text{if } \frac{\sigma_b}{\sigma_{ba}} \leqq 0.2 \end{aligned} \qquad (8)$$

The use of the conditions (8) is thought to give a more accurate design. However, the advantage of covering both in-plane bending and lateral-torsional buckling with the same formula is lost. If this double use of the design formula is dropped one may as well choose a more accurate method for the design when the bending moments varies along the member, e.g. as suggested in /6/.

CONCLUSIONS

The above study indicate that timber members subjected to bending and compression may fail by lateral-torsional buckling. However, in most timber structures the members are braced in lateral direction and hence the problem is mostly avoided. In some large structures in which the main frames are braced by secondary members, the unbraced lengths may be long enough to make lateral-torsional buckling possible. In such cases the bending moment mostly varies linearily between the braced points and the design may be carried as suggested in the preceeding section if the member is straight. For curved members additional considerations are required.

Most design specifications for timber structures do not cover lateral-torsional buckling. There is probably not too much danger to be expected from this, but the possibility of an unsuccessful design exists. If something would go wrong in this respect it is unfortunately most likely to happen on large structures. It is therefore felt justified to include a design criterion for lateral-torsional buckling in design specifications and, as it has been shown above, this may be done in a simple way.

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PAPER 5

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

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

COUNCIL OF FOREST INDUSTRIES

RESEARCH AND DEVELOPMENT DEPT

BRITISH COLUMBIA

PARIS - FEBRUARY 1975

COUNCIL OF FOREST INDUSTRIES - RESEARCH AND DEVELOPMENT DEPT.

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

## 1. GENERAL ANALYSIS

Dr. Kuipers' report, "Standard Methods of Testing for the Determination of Mechanical Properties of Plywood", presented test procedures which are used for:

- Determination of plywood strength properties intended for derivation of stresses suitable for design purposes.
- Purposes of quality control.

It is generally accepted fact that there is a great difference between these two areas of testing and evaluation. Therefore, the following makes some general observations on these two areas and then concludes with some specific comments on individual test methods.

## 2. TESTING OF PLYWOOD FOR STRENGTH PROPERTIES TO BE USED IN DERIVATION OF STRESSES FOR DESIGN

Economical considerations, scarcity of some materials and continuous struggle for better, more realistic design has stimulated the emergence of limit states design which is closely associated and inter-related with probablistic approach to complex design problems.

In connection with this, numerous studies and testing programs were or are being carried out by several laboratories aimed at development of the necessary related data for plywood. The following are some of the more important North American findings:

(a) The ratio between strength properties found for small, clear wood specimens (Eastern Forest Products Laboratory, Ottawa, Canada - Report 1104) and strength properties determined on large in-grade plywood specimens having the same grade of veneer and plywood, and the same plywood contructions (Council of Forest Industries, Laboratory, North Vancouver - "Bending Strength of Canadian Softwood Plywoods"), varies significantly from species to species. Therefore, it concludes that the plywood conversion process influences differently various species and hence transformation of clear wood strength properties into plywood strength properties using veneer grade and plywood grade and plywood grade factors is not an adequate method for derivation of plywood strength properties.

(b) The statistical correlation between strength properties determined on small relatively clear plywood specimens and large in-grade specimens cut as matched pairs from the same panel is poor and namely the correlation of their 5% exclusion value is very low. Other variables as plywood cross section geometry and species mix further complicate the case (Western Forest Product Laboratory, Vancouver, Canada, and COFI, Vancouver, Canada).

It can be concluded that the correlation of small, clear vs large in-grade test results is so low so that prohibitively large numbers of pairs would have to be tested to find statistically meaningful correlations. Factors developed on certain species-geometry-grade combinations could be used for other combinations only with a risk of relatively large errors.

(c) There are systems of plywood and veneer grade factors which have been proven to be practical and workable (APA-USA) but they were developed on the basis of observation, methodic description and testing of large in-grade panels of plywood. They can be used only to convert strength properties determined on large specimen in-grade testing of one plywood grade by means of a conversion factor into strength properties for some other grade of plywood, the difference in grade being specified using predetermined method.

(d) Studies on plywood size effect are now being conducted at WFPL. Preliminary results indicate that the probability of failure of in-grade plywood varies significantly with the size of the specimen. It is believed that in the future size factors will be developed wherever the size effect would be of critical importance. Such size factors could be applied only to strength properties developed by testing large in-grade specimens.

The above information is some of the many factors which have lead to the conclusion that in-grade testing using large specimens is the most suitable approach to development of basic strength properties for plywood.

When the choice of a particular test method for each type of test is made, it is imperative that the method selected is theoretically sound, practically feasible and in both above respects backed up by as much experimental work as possible.

Several years ago the Council of Forest Industries of British Columbia undertook an extensive in-grade plywood strength test program. During Phase I of the project considerable time was spent developing suitable in-grade test methods and evaluating the effect of specimen size and geometry. On the basis of experience found through this program we came to a conclusion that the following methods of in-grade testing of plywood appear to be most suitable for the purpose of determining plywood strength properties.

(a) ASTM D 3043-72 Standard Methods of Testing Plywood in Flexure, Method C.

(b) ASTM D 2719-71 Standard Methods of Testing Plywood in Shear Through-Thickness, Method C.

(c) ASTM D 2718-71 Standard Methods of Testing Plywood in Rolling Shear (Shear in-Plane-of-Plies).

(d) ASTM Proposed Standard - Standard Methods of Testing Plywood in Tension, Method B.

(e) ASTM Proposed Standard - Standard Methods of Testing Plywood in Compression, Method B.

Note: Appendix A, COFI Report 105, "The Derivation of Allowable Unit Stresses for Unsanded Grades of Douglas Fir Plywood from In-grade Strength Test Data", presents abstracts of the reports available on the COFI in-grade plywood strength test program.

## 3. TESTING OF PLYWOOD FOR QUALITY CONTROL

As the objective of quality control testing is not one to determine plywood strength properties, the use of small clear specimens is justified. Tests commonly used in quality control of plywood can be subdivided into three classifications. These are -

A - Tests for quality control certification.

B - Tests for the current quality control evaluation.

C - Tests for quality control monitoring.

In some QC systems the differences between A, B and C are only in the amount of tests required and in the number of types of pretreatment of specimens prescribed. In other QC systems also the type of tests required differ. Apart from measurement of squareness, thickness control, etc., the following tests are the most common:

(1) Bond test by knifing.

(2) Bond test by tensile rolling shear.

(3) Bending test.

(4) Tension or compression test.

Most common types of pretreatment are:

For bond test specimens -

(1) cold water soaking

(2) vacuum-pressure cycle

(3) boil-dry-boil cycle

For bending, tension and compression specimens -

(4) Conditioning to the equilibrium moisture content within specified ambient conditions defined by percentage of relative humidity and dry-bulb temperature.

COFI has an extensive quality control system employing "tensile rolling shear" type bond test (specified in CSA Standard 0121). It has been found that once the bending, tension, compression (etc.) strength properties are established, by in-grade tests, for certain constructions of plywood as specified in the national product standard, the bond test and close checking of ply and panel thickness, species, grades and workmanship gives an excellent level of control on the in-grade strength of the plywood panel. It has therefore been concluded that the "tensile rolling shear" type of bond test on small specimens pretreated by boil-dry-boil cycle and vacuum-pressure cycle be used as the main quality control test for exterior type of plywood. For interior types of plywood the same bond test with less severe pretreatment (as cold water soaking) should be used.

## 4. COMMENTS ON INDIVIDUAL TEST METHODS

The following make some general observations on these in-grade plywood strength tests used in North America.

1. Flexure Test

   ASTM D 3043-72 Standard Methods of Testing Plywood in Flexure, Method C.

- This method is ideally suited for evaluating effects of natural growth characteristics and manufacturing variables.
- Measured deformation and elastic constants are free of shear deformation effects.
- A specially designed testing machine applies pure moments to opposite ends of the test panel through loading frames.
- When non-uniform material containing knots, knot holes, etc. is tested a minimum specimen width of 24 inches (610 mm) is recommended and in no case shall width be less than 12 inches (300 mm).
- Rotation of load frames with respect to each other shall take place at a constant rate throughout the test within $\pm$ 25 percent of the rotation rate, so that the resulting rate of outer fiber strain within plywood will be 0.0015 in./in.min. (mm/mm.mm).
- Panel stiffness (EI) can be calculated from curvature measurements.

2. <u>Shear Through-Thickness</u>

ASTM D 2719-71 Standard Methods of Testing Plywood in Shear Through-Thickness, Method C.

- It employs large specimens and responds well to manufacturing variables and growth characteristics.
- The specimen fabrication and test procedure are somewhat simpler than in other similar methods.
- Specimens 24 inches (600 mm) long and at least 16 inches (400 mm) wide are used with a shear exposed area 24 inches by 8 inches (200 mm).

- Load is transferred to the specimen through two pair of rails glued to the specimen parallel to the longer edge.
- The average deformation is measured across the 8 inch (200 mm) gauge length between pin holes by gauges on each side of the specimen and then averaged.

3. <u>Shear in-Plane-of-Plies</u>

ASTM D 2718-71 Standard Methods of Testing Plywood in Rolling Shear (Shear in-Plane-of-Plies).

- The ASTM rolling shear test uses specimens at least 6 inches (150 mm) wide by 18 inches (450 mm) long, thus it is large enough to contain the main manufacturing variables and natural growth characteristics and therefore is suitable for determination of plywood strength properties.
- Primarily, the test measures the "rolling shear" strength of the weakest ply with grain oriented perpendicular to the direction of shear force.
- The specimen having the form of a rectangular flat plate is bonded between steel plates, beveled at opposite ends of the specimen to provide knife edges for loading.
- The test is conducted by loading the knife edges in compression at a uniform rate while a suitable gauge measures slip between the plates due to specimen deformation.

- Ultimate rolling shear stress and modulus of rigidity in rolling shear can be determined from the test.
- Some concern has been expressed about the safety of the technical staff conducting this test because at the time of failure relatively heavy plates tend to fall aside and can cause injury. COFI Laboratory staff has developed and used a simple holding frame to prevent plates from falling aside. When such frame is used the test is perfectly safe and does not need to be replaced by a similar but tensile type shear test as sometimes recommended in order to improve its safety.

4. <u>Tension</u>

ASTM Proposed Standard - Standard Methods of Testing Plywood in Tension, Method B.

- This method employs large specimens and responds well to manufacturing variables and growth characteristics influencing the tensile properties of plywood.
- Rectangular specimens of a constant cross section are used. Specimens shall be at least 10 inches (254 mm) wide and 48 inches (1219 mm) long.
- Self aligning grips should be used to assure axial specimen alignment as soon as the load is applied.

5. <u>Compression Test</u>

ASTM Proposed Standard - Standard Methods of Testing Plywood in Compression, Method B.

- This method employs large specimens and responds well to manufacturing variables and growth characteristics which influence compression properties of plywood.

- Specimens shall be precisely cut with all adjacent edges at right angles. The dimension of the specimen shall be 7-1/2 inches (190 mm) wide by 15 inches (380 mm) long, measured to an accuracy of not less than $\pm$ 0.3 percent.
- To eliminate buckling, the following length to thickness ratios shall be used:

  A ratio of 20 or less if data are to be recorded up to the proportional limit only.

  A ratio of 10 or less if strength data only are required.

In order to obtain the specified length to thickness ratios, it is necessary in some cases to glue two or more specimens face to back.

- The load shall be applied through a hinged connection to allow for any deviations from parallel of the ends of the specimens and permit adjustment to the end of the specimen in one direction. The specimen shall be loosely held by the side restraining rail.
- The load shall be applied continuously throughout the test at rate of moveable head motion which will produce failure within three to ten minutes after initiation of loading.

PAPER 6

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH
STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# THE DESIGN OF SIMPLE BEAMS

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PARIS - FEBRUARY 1975

# CONTENTS

# DESIGN OF SIMPLE BEAMS

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United Kingdom

The first object of this note is to demonstrate the method of design of simple rectangular beams according to the current UK Code Of Practice, CP 112, and a proposed revision. In trying to achieve a simple presentation it quickly becomes evident that any discussion of UK loading requirements must be left out because a full discussion would be very complicated.

On the other hand, when loading considerations are omitted the subject becomes very simple because the only features requiring explanation are the stress and E values and their modification factors, together with limitation on deflection. The rest of the calculation involves only the mathematical relationships which are common to all countries.

It is nevertheless felt worthwhile to give even such a simple demonstration, partly because the comparison between present and so-called limit state methods will be of interest, and partly to isolate a part of the design procedure for comparison with practice in other countries. The UK loadings will be described in a PRL paper being prepared for another forum, and loadings are best treated separately since independent negotiations towards their international unification are in progress.

## DESIGN TO UK CODE OF PRACTICE

One way of demonstrating the UK calculation would be to design a simply-supported beam for a selected uniformly-distributed load, that is to find the required I and Z values and select a standard cross-section providing both these values while having a height-to-breadth ratio giving reasonable economy without being too flexible laterally.

More than one such example would have to be chosen to demonstrate designs governed by bending stress, deflection and perhaps shear stress, and since both long-term load and a shorter-term load should be catered for, a total of eight calculations would be required. It should perhaps be mentioned that such calculations are rarely required in design practice nowadays, because of the availability of design aids allowing beams to be sized very rapidly without calculation.

A more economical procedure for the present purpose, and again one which helps to keep the treatment as simple as possible, is to calculate the maximum permissible uniform load on a given beam with and without a deflection limitation of 0.003 times the span, first taking all the load as of long duration and then assuming a large part of it is medium-term. For easy recognition the units used are the centimetre and the bar (deca-Newton/cm$^2$ = 0.1 N/mm$^2$ = 1.02 kg/cm$^2$).

(a) LONG DURATION LOAD

- taken as applied to floor joists spaced not more than 0.6 m, for which a loadsharing factor of 1.1 is applied to the stress.

Without deflection limitation

For a 5 x 20 Redwood beam, $Z = 333$ cm$^3$

Using the SS grade for which the permissible long-term bending stress is 73 bar

$$M = fZ = (73 \times 1.1) \times 333 = 26{,}800 \text{ daN-cm} = \frac{Wl^2}{8} = \underline{268 \text{ daN-m}}$$

Taking the span as 3.6 m, the permissible uniform load per metre is

$$W = 268 \times \frac{8}{(3.6)^2} = \underline{165.7 \text{ daN/m}}$$

With deflection limitation

The limitation of 0.003 x span is widely applied in the UK for floor design although not definitely required by CP 112.

The calculation gives

$$0.0031 = \frac{5}{384} \frac{wl^4}{EI}$$

$$0.003 = \frac{5}{384} \times \frac{w \times (3.6)^3 \times 10^6}{100{,}000 \times 3{,}333}$$

giving $w = 1.647$ daN/cm

$= \underline{164.7 \text{ daN/m}}$

The "mean E" value of 100,000 bar used above is only applicable in "loadsharing" situations, i.e. where the joists are spaced not more than 0.6 m and the loading is distributed laterally by boarding. Without loadsharing, the "minimum E" value of 57,000 bar must be used.

The above work completes the most important part of routine design calculations. Shear stress, bearing stress and lateral stability are rarely limitations but will be included for completeness.

Shear stress

For the load of 164.7 deca-Newton/m obtained above, the end reaction would be

$$\frac{164.7 \times 3.6}{2} = 296 \text{ deca-Newton}$$

The maximum shear stress is calculated as

$$\frac{3}{2} \times \frac{296}{5 \times 20} = 4.44 \text{ bar}$$

- compared with a permissible value for the SS grade of 8.6 bar.

Bearing

The minimum end bearing area is calculated to limit the bearing stress to 17.1 bar for SS grade Redwood (15.5 for Whitewood) but if there is no wane these values may be raised to the "basic" values 22.1 and 20.7 bar respectively.

Lateral stability

The present CP 112 includes a table indicating that no special provision need be made for lateral instability in the section size considered. In the revision of CP 112 it is proposed to treat this subject in an entirely different manner similar to that adopted for the Australian code.

(b) MEDIUM TERM LOAD

For the second example it is supposed that three-quarters of the load is "medium term". Such a loading is commonly met in the design of joists for flat roofs and again it will be assumed that a loadsharing situation exists.

Without deflection limitation

$$M = fZ = (73 \times 1.1 \times 1.25) \times 333 = 33{,}400 \text{ daN-cm}$$

$$\frac{wl^2}{8} = \underline{334 \text{ daN-m}}$$

- in which the figure 1.25 is the stress increase factor for medium term load. For wind loading on a vertical member the short-term factor 1.5 would be used.

Taking the span as 4.8 m, the permissible uniform load per metre is

$$w = \frac{334 \times 8}{(4.8)^2} = \underline{116 \text{ daN/m}}$$

With deflection limitation of 0.003 times the span:

$$0.003 = \frac{5}{384} \times \frac{w \times (4.8)^3 \times 10^6}{100{,}000 \times 3{,}333}$$

$$w = \frac{0.003 \times 384 \times 100{,}000 \times 3{,}333}{5 \times (4.8)^3 \times 10^6}$$

$$= 0.695 \text{ daN/cm}$$

$$= \underline{69.5 \text{ daN/m}}$$

Shear stress, bearing stress and lateral stability would be checked as mentioned previously. A further point of interest is that the span used in calculations is strictly the effective span taken from centre to centre of the minimum bearing lengths necessary to limit bearing stress to the permissible value.

PROPOSED FUTURE UK METHOD

The calculations above will be repeated according to the new approach proposed for CP 112, but only those relating to bending stress and deflection.

Ultimate moment calculation

The most important load case will be examined as follows:

(1.4 x dead + 1.6 x imposed) resisted by (modified design stress x Z)

The published values will be grade design stresses which already incorporate $\gamma$ m values to reduce characteristic test results to long duration values for a standard member depth of 20 cm.

The grade design stress published for SS grade Redwood is 110 bar, and no adjustment for depth need be applied because the depth of member in the calculation is equal to the standard depth.

(a) LONG DURATION LOAD

No modification factors are required for depth or load duration in this case, so the only modification is for loadsharing and in the proposed new method the appropriate value will be 1.2 for four members. The ultimate moment for the section will then be

modified design stress x Z = (110 x 1.2) x 333

= 44,000 daN-m

$\frac{wl^2}{8}$ = 440 daN-m

Ultimate load w = 440 x $\frac{8}{(3.6)^2}$ = 272 daN/m

To find the permissible imposed load, a dead load must be assumed and this will be taken as 30 daN/m to bear some relation to floor design in the UK.

1.4 x dead + 1.6 x imposed = 272 daN/m

1.4 x 30 + 1.6 x imposed = 272

imposed = $\frac{230}{1.6}$ = 143.7 daN/m

Total load = 143.7 + 30

= 173.7 da/Nm

- compared with 165.7 daN/m found before. However the result by the new method will vary depending on the assumed value of dead load.

With deflection limitation

The deflection is assessed under the non-factored load i.e. dead + imposed, which appears as w in the equation

$$0.003 = \frac{5}{384} \times \frac{w \times (3.6)^3 \times 10^6}{(69,000 \times 1.31) \times 3,333}$$

The value of E is the bracketed product in the denominator, in which 69,000 is the published design value corresponding to the 1 in 20 lower exclusion limit for the grade and 1.31 is a loadsharing factor. The product of these two is only 90,400 compared with 100,000 in the present form of calculation as given earlier. The permissible load with deflection limited to 0.003 times the span is

$$w = \frac{0.003 \times 384 \times (69,000 \times 1.31) \times 3,333}{5 \times (3.6)^3 \times 10^6}$$

$$w = \underline{148.8 \text{ daN/m}}$$

This is less than the previous result (164.7) in the ratio of the old and new E values, i.e. 100,000 to 90,400.

(b) MEDIUM TERM LOAD

Without deflection limitation

ultimate moment = modified design stress x Z

= (110 x 1.2 x 1.25) x 333

- using the loadsharing factor 1.2 and a load duration factor 1.25

This gives $\frac{wl^2}{8} = 550$ daN-m

Ultimate load $w = \frac{550 \times 8}{(4.8)^2} = \underline{191 \text{ daN/m}}$

Assuming a dead load of 30 daN/m,

1.4 x 30 + 1.6 x imposed = 191 daN/m

imposed = $\frac{149}{1.6}$ = 93.1 daN/m

Total load = 93.1 + 30

= $\underline{123.1 \text{ daN/m}}$

- compared with 116 daN/m found before. Again the result by the new method will vary depending on the assumed value of dead load.

With deflection limitation of 0.003 times the span:

$$0.003 = \frac{5}{384} \times \frac{w \times (4.8)^3 \times 10^6}{(69{,}000 \times 1.31) \times 3{,}333}$$

$$w = \frac{0.003 \times 384 \times (69{,}000 \times 1.31) \times 3{,}333}{5 \times (4.8)^3 \times 10^6}$$

$$= \underline{62.7 \text{ daN/m}}$$

- somewhat less than the former result, 69.5 daN/m

METHODS IN SOME OTHER EUROPEAN COUNTRIES

In the following notes the permissible loads on the beams in the above examples are assessed in relation to a number of other European codes, but as will be seen it is not always possible to do this precisely. The study is only a small scale and superficial one, but does seem to give a good indication of how designs in the different countries compare with one another, provided their codes have been interpreted correctly.

## FRANCE

### (a) LONG DURATION LOAD

For the first example above, the most restrictive loading requirement will be (dead + 1.2 x imposed) resisted by permissible stress.

The load than can be carried by a 5 x 20 redwood floor joist under the present CP 112 was 165.7 daN/m. If a dead load of 30 daN/m is assumed, the imposed load is 135.7 daN/m and the French loading would be

$$30 + 1.2 \times 135.7 = 193 \text{ daN/m}$$

for 3.6 m span, $\frac{Wl^2}{8} = \frac{193 \times (3.6)^2}{8} = 313$ daN-m

A grade stress of 100 bar is quoted for Resineux II timber. Dividing this by the basic value 185 bar gives a grade ratio of 54% corresponding closely to that of SS grade, so Resineux II and SS will be taken as approximately equivalent for the present purposes.

A depth factor of 0.9 is applicable for section depths from 15 to 25 cm, so the moment of resistance of a 5 x 20 piece is

$$\begin{aligned} fZ &= (100 \times 0.9) \times 333 \\ &= 30{,}000 \text{ daN-cm} \\ &= \underline{300 \text{ daN-m}} \end{aligned}$$

This is very close to the assumed applied loading of 313 daN-m. The much larger grade stress compared with the UK figure is compensated by the French depth factor and especially by the factor 1.2 applied to the imposed load.

With deflection limitation

It is not known if domestic floor loading is regarded as of long duration in France, but the calculation will be made as though this were the case. It will be assumed further that the French deflection calculation takes the load as (dead + imposed) and not (dead + 1.2 x imposed). Taking the 5 x 20 beam as just adequate for the given loading, the stress due to the (dead + imposed) load is

$$\frac{30 + 135.7}{193} = 85.8\% \text{ of the permissible}$$

and the creep coefficient for zero moisture change may be interpolated as

$$1.56 + \frac{5.8}{20} \times (1.75 - 1.56) = 1.62$$

The "conventional" E for Resineux timber, using the value which allows for the effect of shear deflection given in the "simplified rules" and not the one in section 4.011, is

$$9000\sqrt{\text{grade stress}} = 9000\sqrt{100}$$
$$= 90{,}000 \text{ bar}$$

This is to be divided by the creep coefficient, giving

$$\frac{90{,}000}{1.62} = 55{,}600 \text{ bar}$$

It is not clear whether the E value 90,000 allowing for shear deflection or the value of 105,000 which excludes it should actually be used. In either case it is evident that the creep coefficient will severely limit the long duration load, compared with the 164.7 daN/m in the UK code calculation. Taking the higher E value and assuming the French deflection limitation of $\frac{1}{300}$ is appropriate, the permissible load is found from

$$\frac{1}{300} = \frac{5}{384} \; \frac{w \cdot x\,(3.6)^3 \times 10^6 \times 1.62}{105{,}000 \times 3{,}333}$$

giving w = <u>118.4 daN/m</u>

(b) <u>MEDIUM TERM LOAD</u>

For the second example, it will be assumed that three-quarters of the load is due to snow, taken as "normal climatic" loading, so the load case is dead + imposed + normal climatic with imposed = 0.

The permissible load will be found from

$$\frac{wl^2}{8} = fZ = (100 \times 0.9) \times 333 = 300 \text{ daN-m}$$

$$w = 300 \times \frac{8}{(4.8)^2} = 104 \text{ daN/m}$$

- compared with 116 in the UK calculation.

Another load case to be considered is 1.1 x dead + 1.5 x imposed + 1.1 x extreme climatic resisted by 1.75 times the bending stress. Without information on code loadings this case cannot be considered but it seems unlikely to be critical.

With deflection limitation

A long duration load in the French code is one applied for three months or longer, or applied on average for 50% of the time or more. Assuming the snow load here is not a long duration load, the creep coefficient for zero moisture change with the dead load forming 25% of the total will be only

$$1.0 + \frac{0.05}{0.2} \times 0.19 = 1.0475$$

so the higher E value will be

$$\frac{105{,}000}{1.0475} = 100{,}000 \text{ bar approximately}$$

- and the permissible load will differ from the UK calculation only to the extent caused by the French deflection limitation of $\frac{1}{300}$ compared with the UK value 0.0031.

CONCLUSION

The indications of this tentative study, undertaken without information on the French loading code, are that the UK and French design methods lead to similar member sizes where stress governs the design or where the proportion of long-term load is low. With a high proportion of long-term load, the French member sizes will be much bigger if deflection governs the calculation.

## GERMANY

In the German code there seem to be no stress modifications for load duration, loadsharing or beam depth, and no provision for varying the E value depending on load duration. The permissible bending stress for grade II pinewood is 100 $kp/cm^2$ and a general E value for European conifers is quoted as 100,000 $kp/cm^2$. These values are similar to those for French Resineux II, but the grade ratio is not known for the German timber.

The deflection calculation under the German code would be similar to the UK method except for a general deflection limitation of 1/300 that seems to be implied, compared with the UK limitation of 0.003l; this would make little difference.

The stress calculations show considerable differences as follows:

(a) LONG DURATION LOAD

$$M = fZ = 100 \times 333 = 33300 \text{ kp-cm} = 333 \text{ kpm}$$

$$\frac{wl^2}{8} = 333$$

$$w = \frac{333 \times 8}{(3.6)^2} = \underline{205 \text{ kp/m}}$$

- compared with 165.7 daN/m by the UK calculation

(b) MEDIUM TERM LOAD

$$M = fZ = 100 \times 333 = 333 \text{ kp-m as above}$$

$$w = \frac{333 \times 8}{(4.8)^2} = \underline{115.5 \text{ kp/m}}$$

- approximately the same as in the UK because the UK stress is increased by modification factors to 73 x 1.1 x 1.25 = 100 bar

## CONCLUSION

With a small proportion of long term load the German calculation gives results similar to the UK or French codes. With a high proportion of long-term load, the German result is much higher than under the UK code if stress-governed, and much higher than the French result if deflection-governed. However the German domestic imposed floor load of 200 kp/cm$^2$ will tend to compensate fr the difference from the UK result since the UK floor load is 146.5 kp/cm$^2$, and for the difference from the French result since the French floor load is 175 kp/cm$^2$.

## BELGIUM

The bending properties of the grade in STS 31 are 100 $kp/cm^2$ for the stress and 100,000 $kp/cm^2$ for E. There are no modification factors for loadsharing, but the stress is multiplied by

1.15 for dead + imposed + normal wind
(and snow if compatible with wind)

1.5 for dead + imposed + exceptional wind (no snow)

and the E value is reduced by $33\frac{1}{3}\%$ to obtain deflections for long term loads. The deflection limit for joists and beams is $\ell/300$.

(a) LONG DURATION LOAD

w = 205 kp/m as in the German calculation

With deflection limitation

$$\frac{1}{300} = \frac{5}{384} \quad \frac{w \times (3.6)^3 \times 10^6}{(0.667 \times 100{,}000) \times 3{,}333}$$

giving w = 122 kg/m

- similar to the French result since the modified E value 66,700 is similar to the modified French value

$$\frac{105{,}000}{1.62} = 64{,}800$$

(b) MEDIUM TERM LOAD

The load-duration increase factors are not applicable in the case (dead + snow), so the result is the same as the German, i.e. 115.5 kp/m.

With deflection limitation

- same as German (not calculated because it differs little from U.K. value).

## HOLLAND

The Construction grade has a bending stress of 100 $kp/cm^2$ and an E of 110,000 but its grade ratio has not been determined. There is a loadsharing factor for concentrated load on a boarded floor, but this will not be taken into account because all the other calculations relate only to uniformly-distributed loading. The stress due to medium term load may be multiplied by 0.85 and that due to short term load by 0.70. The deflection limitation is 0.0025l for domestic floors; the available translation is not clear on the limitation for domestic flat roofs.

w = 205 kp/m as in German and Belgian calculations

With deflection limitation

The Dutch code suggests taking one-third of the imposed load as permanently present (unless a greater part follows from the nature of the construction) when calculating creep deflection, which should be taken as of similar magnitude to the elastic deflection under the permanent load.

It is not known what proportion of a domestic floor load would be taken as permanent, but it will be supposed that the one-third factor is applicable. It appears that the domestic floor loading in $kp/m^2$ is taken as 60 dead + 150 imposed = 210. The load used for calculating creep will be $60 + \frac{150}{3} = 60 + 50 = 110$ $kp/m^2$, so the equivalent load for calculating (initial + creep) deflection will be $210 + 110 = 320$ $kp/m^2$, and a reduced E value having a similar effect would be $110{,}000 \times \frac{210}{320} = 110{,}000 \times 0.657$. The factor 0.657 may be compared with the Belgian 0.667 used in conjunction with an E of 100,000 and the French $\frac{1}{1.62} = 0.617$ used with an E of 105,000.

A calculation based on the above would give

$$0.0025 = \frac{5}{384} \; \frac{w \times (3.6)^3 \times 10^6}{(0.657 \times 110{,}000) \times 3{,}333}$$

giving w = 99.2 kg/m

The difference from the Belgian result arises because of the more severe deflection limitation, the ratio of 0.0025 to 0.00333 being 0.75. This more than outweighs the ratio of 1.083 between the Dutch E and the Belgian E.

The Dutch requirement for roofs is equivalent to (dead + 0.85 x imposed) resisted by (design stress x Z)
As in the French calculation it will be assumed that three-quarters of the load is due to snow, i.e. the snow load is three times the dead load.
The permissible "equivalent" load is found from

$$w = fZ \times \frac{8}{(4.8)^2}$$

$$= 100 \times 333 \times \frac{8}{(4.8)^2} = 115.5 \text{ kp/m}$$

$$\text{dead} + 0.85 \times \text{imposed} = 115.5$$

$$\text{dead} + 0.85 \times 3 \times \text{dead} = 115.5$$

$$\text{dead} = \frac{115.5}{3.55} = 32.5$$

$$\text{imposed} = 3 \times 32.5 = \underline{97.5}$$

$$\text{total} = 130 \text{ kp/m}$$

- compared with 116 in the U.K. calculation

<u>With deflection limitation</u>

The deflection limitation for domestic roofs is not clear in the available translation. Assuming it is of the same order as in the U.K., a bigger permissible load would be obtained because of the 10% greater E value, provided no part of the snow load is taken as permanent. If a third of the snow load is taken as permanent as seems to be required by the Dutch code, the permissible load on the 5 x 20 beam will be found using an E calculated as follows:

Load for calculating creep = dead + $\frac{1}{3}$ x imposed

Load for calculating initial deflection = dead + imposed

Total effective load for calculating deflection = 2 x dead + $\frac{4}{3}$ x imposed

Or since imposed = 3 x dead, total effective = 2 x dead + 4 x dead = 6 x dead

= 6 x ($\frac{1}{4}$ x actual load)

= 1.5 x actual load

The reduced E value having a similar effect would be 110,000 x 0.667, similar to the value just found in connection with floor loading. Taking the U.K. value of $0.003\ell$ as the deflection limitation, the permissible load would be found from

$$0.003 = \frac{5}{384} \times \frac{w \times (4.8)^3 \times 10^6}{(110,000 \times 0.667) \times 3,333}$$

giving w $= \underline{5\,0.9 \text{ kp/m}}$

- compared with 69.5 daN/m in the U.K. calculation, but this result has to be adjusted to the correct deflection limitation and it has to be confirmed that one-third of the snow load must be included in the load causing creep.

## DENMARK

### (a) LONG DURATION LOAD

The design bending stress in the Danish code for the T200 grade is 110 $kp/cm^2$. There is difficulty in interpreting the available translation, but it will be assumed that a load factor of 1.5 is to be applied to imposed loading and 1.0 to dead load. Calculating in a manner similar to that adopted for France above, the Danish loading is

$$30 + 1.5 \times 135.7 = 233.5 \text{ daN/m}$$

$$\text{for } 3.6 \text{ m span, } \frac{wl^2}{8} = \frac{233.5 \times (3.6)^2}{8} = 378 \text{ daN-m}$$

All the loading appears to be GROUP A, for which no duration factor is applicable and the permissible load on the 5 x 20 beam would be found from

$$fZ = 110 \times 333 = \underline{366 \text{ kg-m}}$$

As in the French case this is very close to the applied loading. The much larger grade stress compared with the UK figure is compensated by the factor 1.5 applied to the imposed load.

#### With deflection limitation

The Danish code calls for a reduction of 20 per cent in E when calculating deflections arising from self weight and other similar loads. The deflection limit for a boarded floor is 1/500 or 0.0021 under imposed load. With the E value of 70,000 for the T200 grade (a 30% exclusion value) the calculation would be

$$0.002 = \frac{5}{384} \; \frac{w \times (3.6)^3 \times 10^6}{(0.8 \times 70{,}000) \times 3{,}333}$$

$$\text{giving } w = \underline{61.4 \text{ kg/m}}$$

- which is very restrictive because of the characteristic value used for E as well as the severe deflection limitation.

(b) MEDIUM TERM LOAD

The Danish code seems to place snow load explicitly into the long term Group A loading. The permissible "equivalent" load is found from

$$w = fZ \times \frac{8}{(4.8)^2}$$

$$= 110 \times 333 \times \frac{8}{(4.8)^2} = 127.2 \text{ kp/m}$$

Assuming again that the load factor 1.5 is to be applied to the imposed loading and that the snow load is three times the dead load,

$$\text{dead} + 1.5 \times \text{imposed} = 127.2$$

$$\text{dead} + 1.5 \times 3 \times \text{dead} = 127.2$$

$$\text{dead} = \frac{127.2}{5.5} = 23.1$$

$$\text{imposed} = 3 \times 23.1 = \underline{69.3}$$

$$\text{Total} = 92.4 \text{ kp/m}$$

- compared with 116 in the UK calculation.

With deflection limitation

The imposed load deflection of plastered ceilings must not exceed 1/500. The UK calculation would envisage a dry-lined ceiling probably allowing a more tolerant view, and since the corresponding total deflection would be $0.002 \times \frac{4}{3} = 0.00267$ times the span, it will be supposed that the UK figure of 0.0031 under total loading would be acceptable. The calculation with no reduction for E would then be

$$0.003 = \frac{5}{384} \quad \frac{w \times (4.8)^3 \times 10^6}{70,000 \times 3,333}$$

$$w = \frac{0.003 \times 384 \times 70,000 \times 3,333}{5 \times (4.8)^3 \times 10^6}$$

$$\left.\begin{matrix}\text{giving}\\ w\end{matrix}\right\} = \underline{48.6 \text{ kp/m}}$$

- compared with 69.5 daN/m in the UK calculation

PAPER 7

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# CALCULATION OF TIMBER BEAMS SUBJECTED TO

# BENDING AND NORMAL FORCE

by

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PARIS - FEBRUARY 1975

## 1. SUMMARY.


According to the relevant standards, the ordinary requirement for dimensioning beams subjected to bending and tension is as follows:

$$- 1 \leq \frac{\sigma_N}{s_t} + \frac{\sigma_M}{s_b} \leq 1 \qquad (1)$$

See section 2 for symbols.

It is shown that for cross-sections that are not double-symmetrical this expression leads to unreasonable results in cases in which the tensile side is that subject to the most dangerous loading, and that the left side of (1) should be substituted by

$$- s_b \leq \sigma_M + \sigma_N \qquad (8)$$

An expression corresponding to (1) is given for beams (columns) subjected to bending and compression:

$$- 1 \leq \frac{(-\sigma_N)}{s_c} + \frac{\sigma_M}{s_b} \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} \leq 1 \qquad (13)$$

On the compressive side, (13) can be approximated by

$$\frac{|\sigma_N|}{s_{cr}} + \frac{|\sigma_M|}{s_b} \leq 1 \qquad (15)$$

In cases in which the tensile side is decisive, it is not possible to formulate a corresponding expression; here, (13) must be used, with the right side substituted by

$$\sigma_N + \sigma_M \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} \leq s_b \qquad (16)$$


## 2. SYMBOLS

| | |
|---|---|
| E | Initial modulus of elasticity |
| E' | Modulus of elasticity as a function of $(-\sigma_N)$ |
| M | Bending moment |
| N | Normal force (positive as tension) |
| l | Free length of columns |

| | |
|---|---|
| $r$ | Radius of gyration |
| $z_1, z_2$ | See fig. 2 |
| $s_b$ | Strength parallel to the grain in bending |
| $s_c$ | " " " " " " compression |
| $s_t$ | " " " " " " tension |
| $s_{cr}$ | Critical stress (columns) |
| $s'_{cr}$ | See (12) |
| $s_E$ | Euler stress |
| $\alpha$ | $s_t/s_b$ |
| $\beta$ | $z_2/z_1$ |
| $\sigma_N$ | Normal stresses due to N |
| $\sigma_M$ | " " " " M |
| $\sigma_r$ | See (8) |

## 3. BEAMS SUBJECTED TO TENSION AND BENDING

For beams subjected to simultaneous tension and bending there is no difficulty in determining the normal stresses $\sigma_N$ from the normal force and $\sigma_M$ from the moment.

As the tensile strength $s_t$ for ordinary grades differs from (is considerably lower than) the bending strength $s_b$ in all modern standards, it does not suffice just to consider the resultant stress when it is to be decided whether a given combination of stresses is acceptable. All standards require instead a condition of the following type to be fulfilled:

$$- 1 \leq \frac{\sigma_N}{s_t} + \frac{\sigma_M}{s_b} \leq 1 \qquad (1)$$

The stresses $\sigma_N$ and $\sigma_M$ are assumed to be positive as tension. The acceptable combinations of stresses conforming to (1) are specified in fig. 1

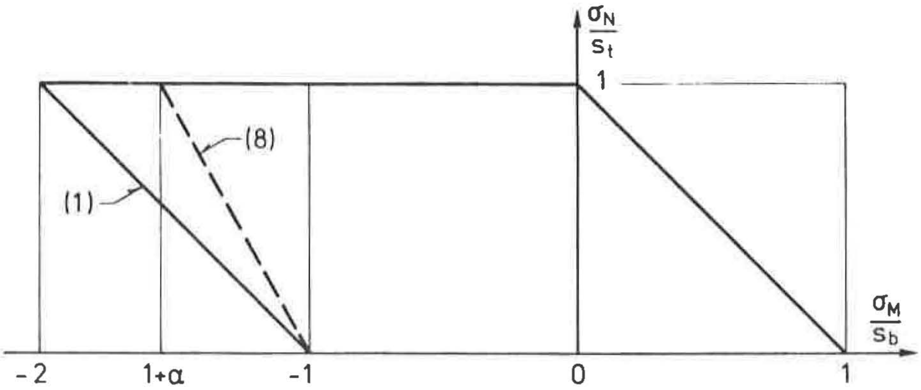



Fig. 1.

At the first glance, the formula seems plausible, but for cross-sections that are not double-symmetrical, it can lead to unacceptable results for parts of the cross-section in which $\sigma_M$ is negative.

Consider, for example, a T-shaped cross-section loaded as shown in fig. 2a. The numerical value of the normal stress corresponding to the bending moment in the outermost compressive fibre is denoted $\sigma_{MC}$.

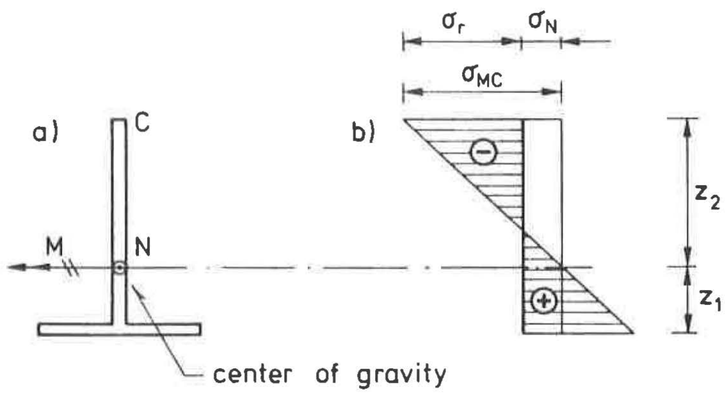



Fig. 2.

The resultant stress, denoted $\sigma_r$, is, see fig. 2b,

$$\sigma_r = \sigma_{MC} - \sigma_N \tag{2}$$

On the assumption of full utilization of the cross-section, the left side of (1) gives

$$\frac{\sigma_{MC}}{s_b} = 1 + \frac{\sigma_N}{s_t}$$

i.e.

$$\frac{\sigma_r}{s_b} = 1 + \frac{\sigma_N}{s_t} - \frac{\sigma_N}{s_b} \qquad (3)$$

With the term, $s_t/s_b = \alpha$, we find

$$\frac{\sigma_r}{s_b} = 1 + (1 - \alpha)\,\frac{\sigma_N}{s_t} \qquad (4)$$

For structural timber, all modern standards specify $\alpha$-values of between 0.4 and 0.7. Thus, provided the compressive zone is decisive, we will always have $\sigma_r/s_b > 1$.

Using the term,

$$\beta = z_2/z_1 \qquad (5)$$

see fig. 2, we find the compressive zone is decisive when

$$\frac{\sigma_N}{s_t} \leq \frac{\beta - 1}{\beta + 1} \qquad (6)$$

in which it is assumed that $\beta > 1$; otherwise, the tensile zone will always be decisive.

Inserting (6) in (4), we get

$$\frac{\sigma_r}{s_b} \leq 1 + (1 - \alpha)\,\frac{\beta - 1}{\beta + 1} \qquad (7)$$

For very high values of $\beta$ and for $\alpha$ between 0.4 and 0.7, $\sigma_r/s_b$ lies between 1.6 and 1.3; for a more normal section, where $\beta \sim 3$, $\sigma_r/s_b$ lies between 1.30 and 1.15.

$\sigma_r/s_b > 1$ appears unacceptable, and the left side of (1) should be substituted by the requirement,

$$\sigma_r = -\sigma_M - \sigma_N \leq s_b \qquad (8)$$

cf. fig. 1.

## 4. CENTRALLY LOADED COLUMNS

For centrally loaded, slender columns with constant cross-section, the bearing capacity can be reasonably determined from Euler's formula, i.e.

$$s_{cr} = s_E = \frac{\pi^2 E}{(\frac{l}{r})^2} \tag{9}$$

where

$s_{cr}$ is the critical stress,

$s_E$ is the Eulerian stress

E is the modulus of elasticity

l is the free length

r is the radius of gyration corresponding to the direction of deflection, and

l/r is the slenderness ratio

For short columns, the bearing capacity of which depends on inhomogenities, eccentricities and existing curvatures, determination of the bearing capacity is more difficult and is usually done by means of empirical or semi-empirical expressions. The tests and assumptions on which these formulae are based vary from one country to another, but the final result, as expressed in the respective standards, is approximately the same, cf. fig. 3, which shows $s_{cr}/s_c$ in accordance with different standards [1]-[7]. $s_{cr}$ is the compression strength.

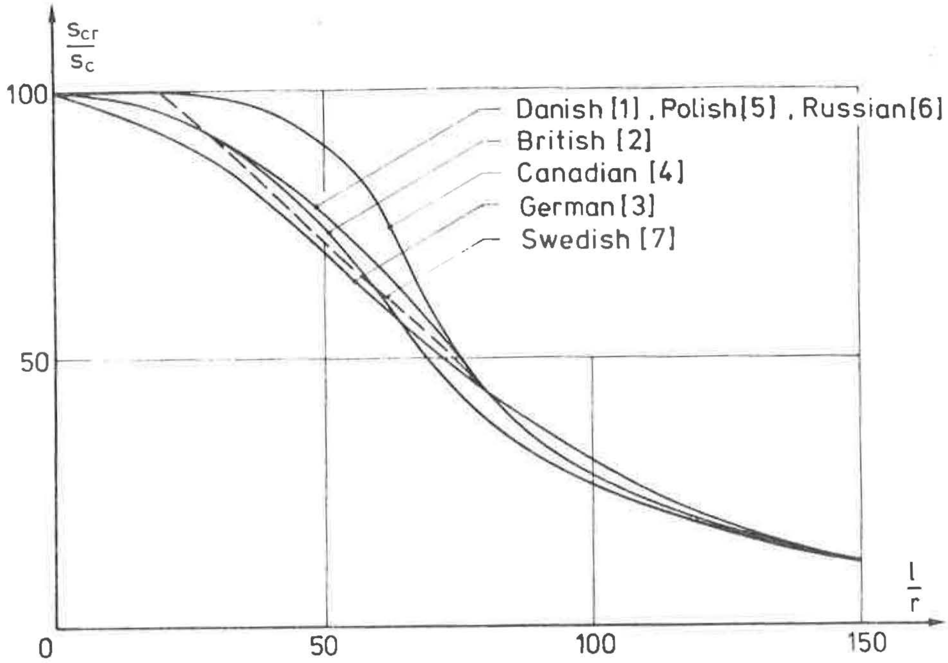



Fig. 3.

The curves are plotted for a common ratio $E/s_c \sim 280$, whereby the influence of different safety systems is removed. If this had not been done, the British curve for long columns would have deviated considerably from the others, because part of the safety factor is "forgotten" for these, cf. [8].

For the remaining calculations it is convenient to express the bearing capacity of the column by means of Engesser's formula

$$s_{cr} = \frac{\pi^2 E'(s_{cr})}{(\frac{l}{r})^2} \tag{9}$$

where $E' = E'(-\sigma_N)$ is a formal modulus of elasticity depending on the compressive stress$(-\sigma_N)$. In (9) it is indicated that $(-\sigma_N) = s_{cr}$ at rupture. By the formulation in (9), the influence of imperfections and existing curvatures is thus converted to a reduced E-modulus, i.e. a reduced stiffness.

Corresponding to the Danish curve [1], we find

$$E' = \begin{cases} E & \text{for } (-\sigma_N) \leq s_c/2 \\ 4(\frac{-\sigma_N}{s_c})[1 - (\frac{-\sigma_N}{s_c})]E & - \quad (-\sigma_N) \geq s_c/2 \end{cases} \tag{10}$$

Minus signs occur in the formulae because normal stresses are still taken as positive for tension.

## 5. LATERALLY LOADED COLUMNS

If it is assumed that the column is subjected to a sinusoidal moment with the maximum value M calculated in the undeformed state, then, due to the normal force, the actual moment at the mid-point will be

$$M' = M \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} \qquad (11)$$

where

$$s'_{cr} = \frac{\pi^2 E'(-\sigma_N)}{(\frac{l}{r})^2} \qquad (12)$$

We often see (12) written with $s_{cr}$ instead of $s'_{cr}$, but this is incorrect because E' must correspond to the relevant normal stress $(-\sigma_N)$, which is, of course, less than $s_{cr}$.

Denoting the stresses from the moment M as $\sigma_M$, taken as positive for tension, the resultant compressive stress will be

$$(-\sigma_N) + \sigma_M \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} \qquad (13)$$

As the compressive strength $s_c$ and the bending strength $s_b$ are also different, (13) cannot be used directly for dimensioning purposes; instead, it is usually required, analogously to (1), that:

$$-1 \le \frac{(-\sigma_N)}{s_c} + \frac{\sigma_M}{s_b} \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} \le 1 \qquad (14)$$

The just acceptable combinations of $\sigma_N/s_c$ and $\sigma_M/s$ for various slenderness ratios are shown in fig. 4.

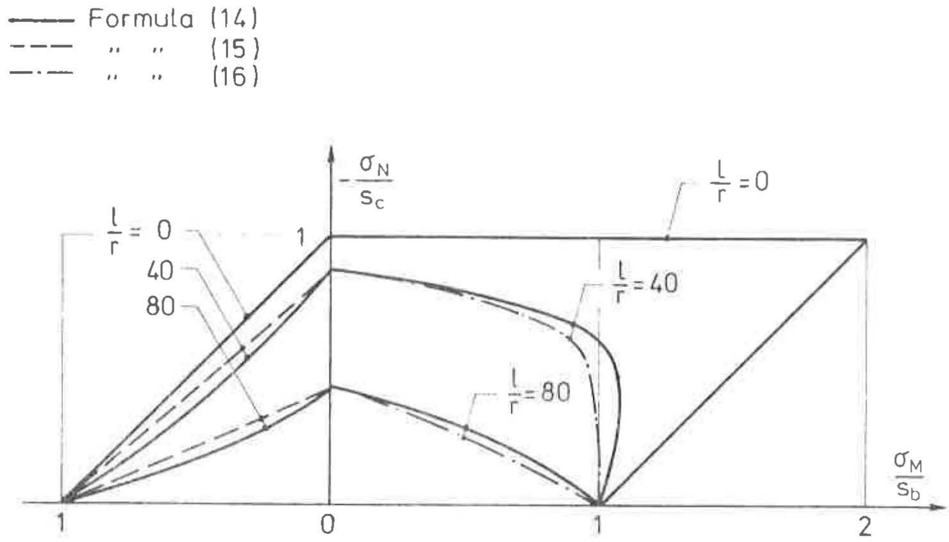



Fig. 4.

In all the standards examined - with the exception of [1] - it is implicitly assumed that the compressive side is decisive, and (14) is substituted by expressions of the form

$$\frac{|\sigma_N|}{s_{cr}} + \frac{|\sigma_M|}{s_b} \leq 1 \tag{15}$$

which is an excellent approximation for the compressive side; the slightly concave curves in fig. 4 are replaced by straight lines.

If, on the other hand, it is the tensile side that is decisive, e.g. in cross-section of the type shown in fig. 2, (15) becomes far too conservative, and it is not possible, for example by removing the numerical signs, to obtain corresponding expressions of any relevance. In this case it is necessary to return to the expression (13). However, as the compressive strength is lower than the bending strength, it is also necessary here to modify the condition on the tensile side, namely to

$$\sigma_M \frac{s'_{cr}}{s'_{cr} - (-\sigma_N)} + \sigma_N \leq s_b \tag{16}$$

The corresponding curves are plotted in fig. 4, where it is assumed that $s_c/s_b = 0.7$.

## REFERENCES

[1]-[7] Code of Practive for the Structural Use of Timber from

[1] Denmark (DS 413,1968)

[2] Great Britain (CP 112 : 1967)

[3] Germany (DIN 1052)

[4] Canada (CSA 086 - 1970)

[5] Poland (PN - 53/b - 031150)

[6] USSR (N og TU 122-55)

[7] Sweden (BABS 1967)

[8] D.A. Newton: The Design of Solid Timber Columns. IUFRO, Structural Utilization. Madison 1971.

PAPER 8

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

PROPOSAL FOR A BASIC TEST METHOD FOR THE EVALUATION OF STRUCTURAL TIMBER JOINTS WITH MECHANICAL FASTENERS AND CONNECTORS.

RILEM 3TT COMMITTEE

PARIS - FEBRUARY 1975

Third draft; February 1975

# Proposal for a basic test method for the evaluation of structural timber joints with mechanical fasteners and connectors.

## 1 Scope

A method is developed:

to investigate the mechanical properties of timber joints; with mechanical fasteners and connectors;

to calculate from the test results values of the characteristic strength and/or of the allowable loads;

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 made in load-bearing timber structures with mechanical fasteners and connectors.

## 3 Classification and nomenclature of joints

3.1 The members to be jointed lie in parallel planes.

3.1.1 Members jointed by connectors through the contact surfaces (eg nails, screws, bolts, etc)

3.1.2 Members jointed by connectors in the contact surfaces (eg all types of dowels and comparable connectors).

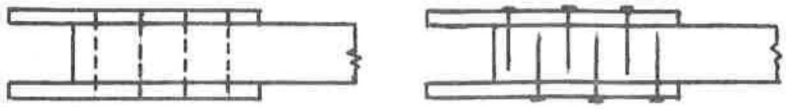
3.2 The members to be jointed have equal thickness and lie in the same plane.

3.2.1 Members jointed by gussets on their outer surfaces (eg punched metal plates, nailing plates, etc).

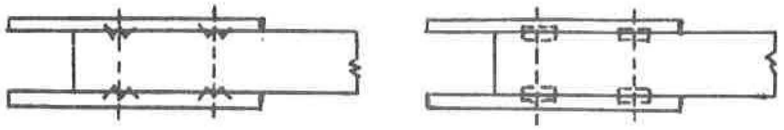
3.2.2 Members jointed by gussets inserted in incised slits (eg nailing plates; Greimbau system).

3.3 Other joints

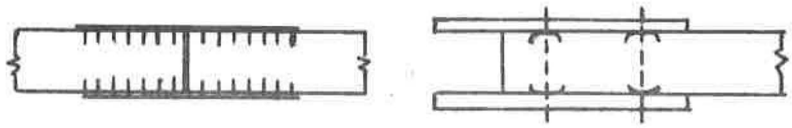
3.1.1



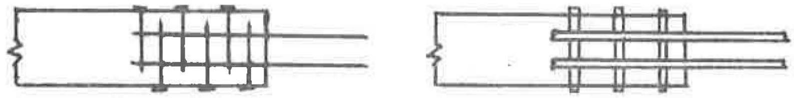
3.1.2

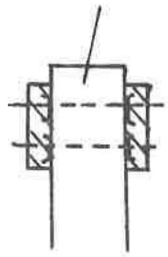


3.2.1



3.2.2






4.1 <u>Climatic conditions</u>

4.1.1 The climatic conditions in which the joint is supposed to function influence its strength and its deformations. Four conditions can be distinguished:

normal heated and sufficiently ventilated buildings

not heated, closed buildings

not heated, open buildings but with covered structure

the open air

4.1.2 Although the basic conditions vary considerably between geographic 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.

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

Table 1

AVERAGE CLIMATIC CONDITIONS IN WESTERN EUROPE
AND MOISTURE CONTENTS TO BE EXPECTED

| Climatic conditions | Temperature $^oC$ | Rel hum % | Moisture content % |
|---|---|---|---|
| Heated and ventilated buildings | $20 \pm 4$ | $55 \pm 15$ | $10 \pm 3$ |
| not heated, closed buildings | $18 \pm 7$ | $65 \pm 15$ | $13 \pm 4$ |
| not heated, covered buildings with open walls | $12 \pm 12$ | $80 \pm 10$ | $17 \pm 4$ |
| open air | $10 \pm 15$ | $85 \pm 15$ | $22 \pm 8$ |

4.2 Loading conditions

Live loads on a structure are changing with time. Dependent on the frequency of the changes three loading conditions are distinguished.

4.2.1 All joints in building structures that are not exposed to the circumstances as described in 4.2.2 and 4.2.3 may be considered as statically loaded. Static loads may have different duration, which can be differentiated as follows:

| | |
|---|---|
| expected total duration of loading during lifetime | 50–100 years |

4.2.2 Structures which are frequently exposed to changing loads with a frequency of about 2 to 5 Hz, like floors of ballrooms, gymnastic halls, etc are semi-dynamic loaded. With respect to the calculation such structures may be considered as statically loaded with an equivalent live load.

4.2.3 Structures exposed to changing loads with higher frequencies than about 5Hz, like crane beams, highway and railroad bridges, are dynamically loaded and must be calculated as such.

5 Types of investigations

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

5.1 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.

5.2 In a limited investigation information is wanted about the behaviour of a certain type of connector in different positions, eg 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

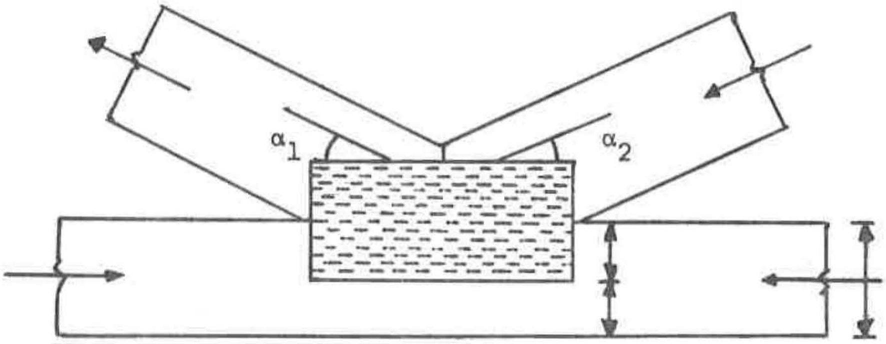



fig 1

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

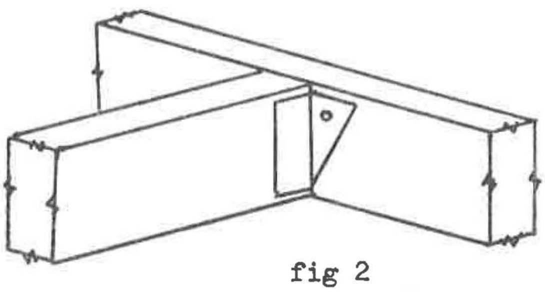


fig 2

5.4 A combination of related series of special and/or restricted investigations as far as the object of investigation has remained unchanged.

## 6 Test specimens

6.1 The number and character of variables introduced in the test program determine the type of investigation, and form the base of type and number of test specimens.

6.2 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.

6.3 In most cases not only simple joints in tension and in compression must be tested (fig 3), but also joints where some of the members are loaded with an angle to the grain. For these latter joints the test specimens must be built up according to 6.2, for instance like fig 4.

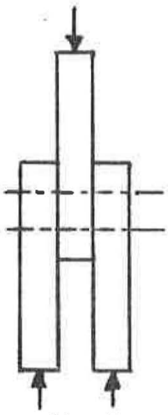


fig 3

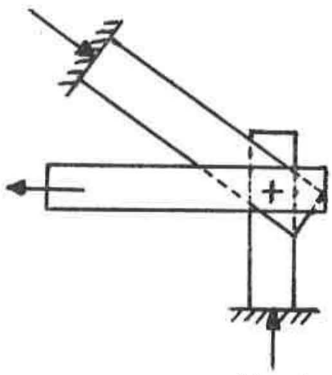


fig 4

6.4 The number of connectors in a joint must be chosen in accordance with 6.2.

6.5 In special and limited investigations (5.1 and 5.2) in any case the minimum predetermined dimensions of the members, the end- and edge-distance, etc must be incorporated in the test series.

6.6 During the test the deformation of the members in the joint must not be hindered by the testing apparatus, measuring devices, etc.

6.7 Species and quality of the timber must be according to that used in reality.

6.8 The principle test must be carried out with wood of a moisture content corresponding with that to be expected in service conditions.

If the structure is expected to be manufactured in practice at another moisture content of the timber the test specimens must be made accordingly and subjected before the test to a conditioning at expected service conditions.

6.9 Normally joints must be made from wood that has been conditioned at $T = 20 \pm 2^{o}C$ and at a relative humidity of $RH = 65 \pm 3\%$ (the timber will then adopt a moisture content of $12 \pm 2\%$). Conditioning may be ended if in 24 hours the weight of a test specimen has not changed by more than 0.2% of the total weight. Before testing is carried out the test specimens must be stored in the same conditions.

## 7 Test procedures

In order to obtain full information about the behaviour of joints in load bearing structures short-duration - (or "standard) tests as well as long-duration tests should be carried out. Procedures, therefore, have been given in 7.1 and 7.2 respectively.

As a compromise which gives a mixed information about short and long duration behaviour also the method of 7.3 may be used.

Some general remarks about dynamic tests have been made in 7.4.

### 7.1 Short-duration test or standard test

Short duration test and long duration test as described in 7.2 belong together.

7.1.2 An expected value of the ultimate load $\hat{F}$ of the joint under test has to be determined on former experience, calculations, preparatory tests or else.

7.1.3 The loading procedure has been given in fig 5 in which the loading or measuring step $f = 0.1\ \hat{F}$. Each loading step f must take not less than 30 seconds.

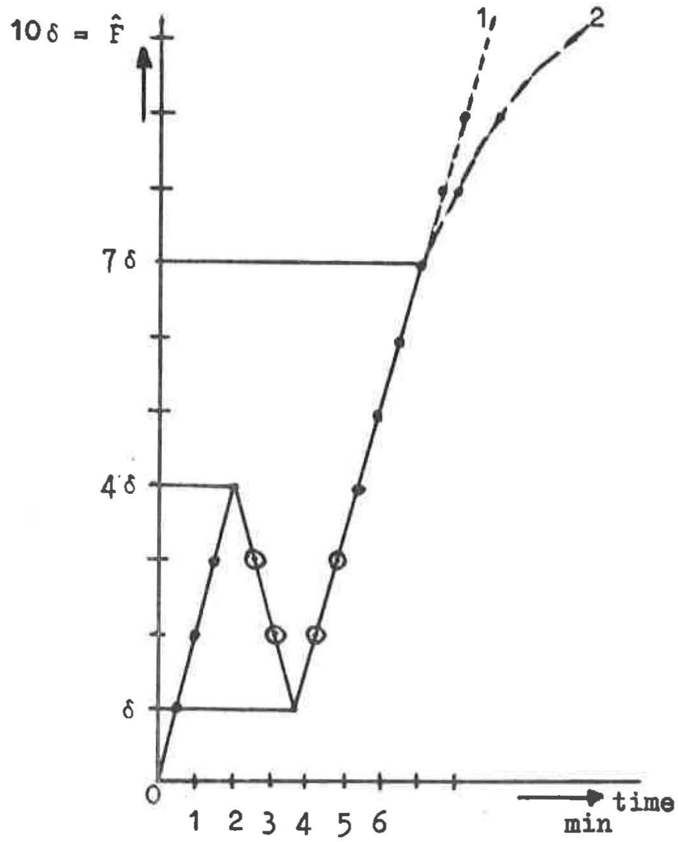



fig 5

7.1.4 According to this fig 5 the test load is increased at a constant rate 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 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 the ultimate load $\hat{F}$ than in case two.

7.1.4 After each loading step deformations must be measured, without interrupting the loading.

From these measurements the following displacements are calculated:

$$V_{0.4} = \frac{4}{3}\left[V_4 - V_1\right]; \quad k_{0.4} = \frac{V_{0.4}}{0.4F}$$

$$V_{0.6} = V_{0.4} + V_6 - V_4$$

$$V_{0,8} = V_{0.4} + V_8 - V_4$$

$$e_{0.4} = \frac{4}{3}\left[\frac{V_4 + V_4''}{2} - V_1'\right]$$

$$a = V_4 - V_{0.4}$$

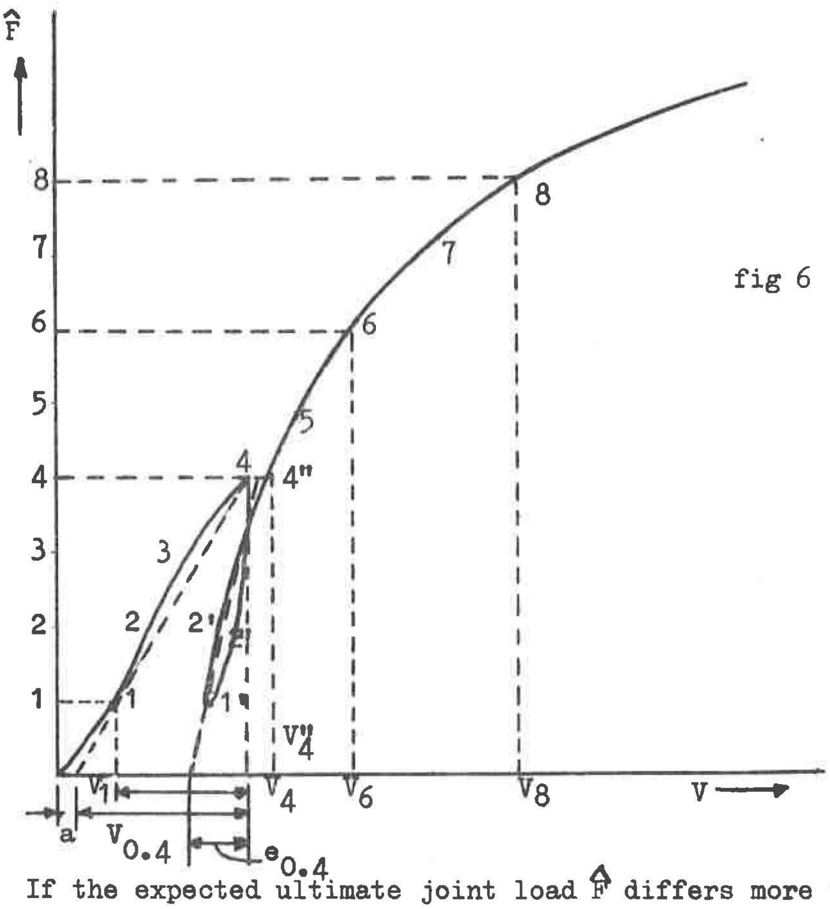



fig 6

7.1.6 If the expected ultimate joint load $\hat{F}$ differs more than 10 per cent of the average result of the executed tests, adjustments of the loading procedure should be made. Also the values of $V_{0.4}$, etc of the already carried out tests must be re-calculated accordingly.

7.2 <u>Long duration tests</u>

7.2.1 Long-duration tests shall be carried out as a control upon the trustworthiness of the joints on the long run. This control intends to check:

a the long duration strength of the joints

b the time-dependent deformations of the joints. Long duration tests must be carried out in conjunction with standard tests of 7.1. These may also serve as an addition to the tests of 7.3.

7.2.2. At least 5 tests specimens must be loaded up to 80 per cent of the mean strength found with short duration tests.

7.2.3 At least 5 tests specimens must be loaded up to 40 per cent of the mean strength found with short duration tests.

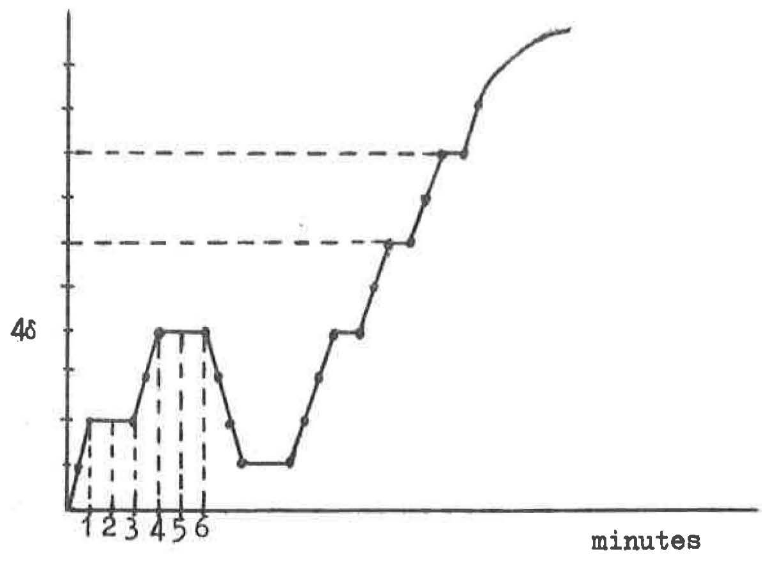
7.2.4 In both cases .2 and .3 the constant load must be reached as good as possible in an uniform rate after maximum 5 minutes.

7.2.5 For the conditioning of the timber and the circumstances in the testing room 6.8 and 6.9 are valid.

7.2.6 During the test deformation must be measured in regular and effective intervals so that a continuing information about the development of the deformation becomes available. The time of the beginning of the measurements and of the end of the test must be given in the test report.

7.3 Combined short-duration or modified short-duration tests

.1 In special and in limited investigations combined, but less detailed than in 7.1 and 7.2, information may be based on tests carried out according to the procedure given in fig.



7.4 Dynamic tests

7.4.1 In principle dynamic tests should be carried out on joints if they are expected to be used in structures where vibrations with frequency above 5 Hz will occur.

7.4.2 The test frequency should not be higher than that in the service conditions foreseen, because that might influence considerably the moisture content and distribution in the test specimen.

7.4.3 With respect to moisture content of test specimens and conditioning of test room 6.8 and 6.9 are valid.

8 General requirements

8.1 Based on static long duration loads.

8.1.1 From the total number of 80 per cent long duration tests not more than 50 per cent may be collapsed within the period of .... hours.

8.1.2 If the long duration test specimens described in 7.2.2 collapse within a period under load of ... hours, or if more than 50 per cent of these tests fail within ... hours, at least 5 tests specimens with the same dimensions must be loaded up to 70 per cent and another 5 to 60 per cent of the mean strength found with short duration tests.

8.1.3 If the long-duration tests described in 6.2.3 show after a period of ... weeks a deformation of more than 1 of the initial deformation the total investigation must be repeated with a two fold number of tests or at least 10 test specimens. From the total number of the 40 per cent - long duration tests not more than 50 per cent may give a deformation of more than .... of the initial deformation.

8.1.4 If the long-duration tests primarily do not fulfil the requirements of 8.1.1 and 8.1.3 the allowable load must be reduced to such limits that the requirements can be fulfilled at load-levels of 80 per cent resp. 40 per cent of 3 x the reduced allowable load.

8.2 Required duration of dynamic tests must be determined in accordance with specific cases of application.

## 9 Interpretation of the results

9.1 In the case of a special investigation (5.1) values of the mean ultimate loads and of the standard deviation may be taken from the test results. The number of tests must be 10 or more.(1)

9.2 In the case of a limited investigation (5.2) the influence of the angle between the direction of the load and of the grain must be studied and a theory or method to forecast the ultimate load $\hat{F}$ must be set up.

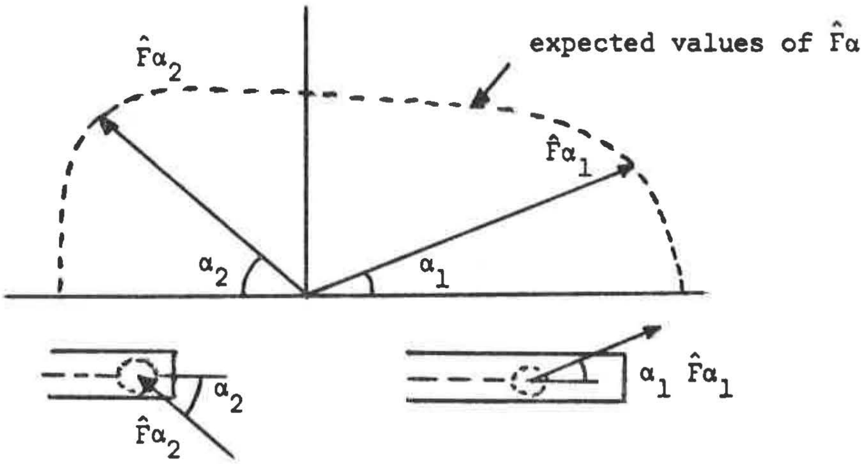



fig 7

(1) In a definite proposal something more must be said about sampling and number of tests.

9.3 From the distribution of the ratio $\frac{\hat{F}\ \text{test}}{F\text{–theory}}$ a coefficient of variation can be calculated, in which not only the variation in strength but also the simplifications of the theory are incorporated.

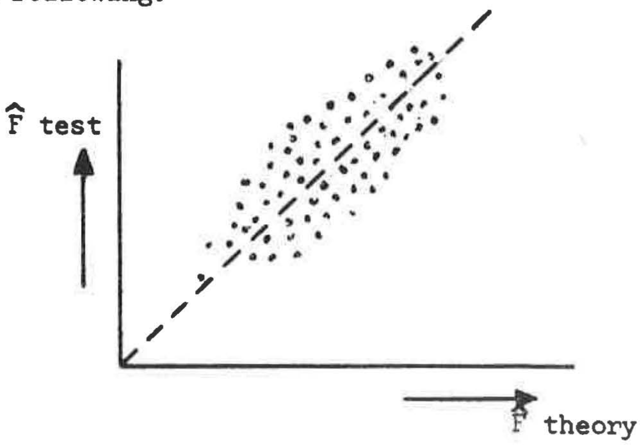
9.4 If the requirements of 8 are fulfilled safe working loads can be calculated following:

$\bar{F} = \frac{\hat{F}}{w} \cdot m_1 \cdot m_2$ , where

$\bar{F}$ = allowable load on the joint

$\hat{F}$ = mean strength of the joint

w = coefficient of safety




$m_1$ , $m_2$ modification factors for service conditions.

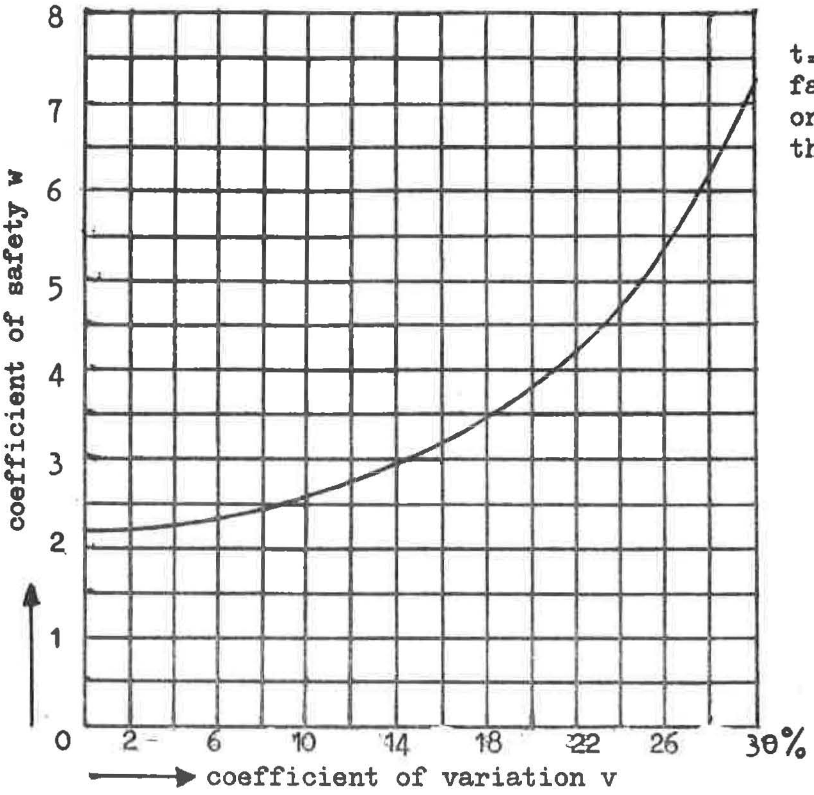
9.5 The coefficient of safety w can be calculated according to

$$w = \frac{1 + \sqrt{1-0{,}94\left[1-6{,}25\left(\frac{v}{100}\right)\right]^2}}{1 - 6{,}25\left(\frac{v}{100}\right)^2} \cdot t$$

v = coefficient of variation = $\frac{\text{standard deviation}}{\text{mean strength}} \cdot 100$

t = time coefficient, for which values of 1.8 to 2 can be taken.

9.6 Values of w can also be taken from fig 8.




t= 2 should be taken if failure depends highly on shear strength of the wood.

Fig 8 Relation between coefficient of variation v and coefficient of safety w; t = 1,8

9.7 From the measurements of the deformations values of direct deformation, elastic deformation, modulus k and expected creep values must be calculated (of fig 5 and 5a.)

## 10 Test reports

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

the species and quality of the wood plus relative density

kind and number of connectors plus quality of connector

exact data about the dimensions of the joint, the loading conditions in the test machine

moisture content of the timber when the joint was made, conditioning before testing and moisture content at the time of the tests

loading procedure followed

measurements

mode of failure

PAPER 9

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# TEST METHODS FOR WOOD FASTENERS

by

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UNIVERSITÄT KARLSRUHE (TH)

WEST GERMANY

PARIS - FEBRUARY 1975

# Test methods for wood fasteners

In Germany there is no officially prescribed test method for wood fasteners.
A method was established as long ago as 1944 for dowel joints. This was incorporated in a draft German Standard (DIN E 4110, Part 8) and is used mostly as a guideline. According to this Standard, compression shear tests may be made for the provisional determination of the ability to withstand loading parallel to the grain, but the principal tests in this respect are those in which test pieces are subjected to shear in tension. The resistance perpendicular to the grain is checked by testing specimens in compression. The wood to be used is spruce with a compressive strength of 350 20 kgf/cm$^2$. The procedure is very time consuming, as the graph presented in the enclosure shows.

The permissible load is established with a factor of safety of 2.75 from the mean value of 3 tests in each case, or the displacement under the permissible load (zul P) 1.5 mm shall not be exceeded.

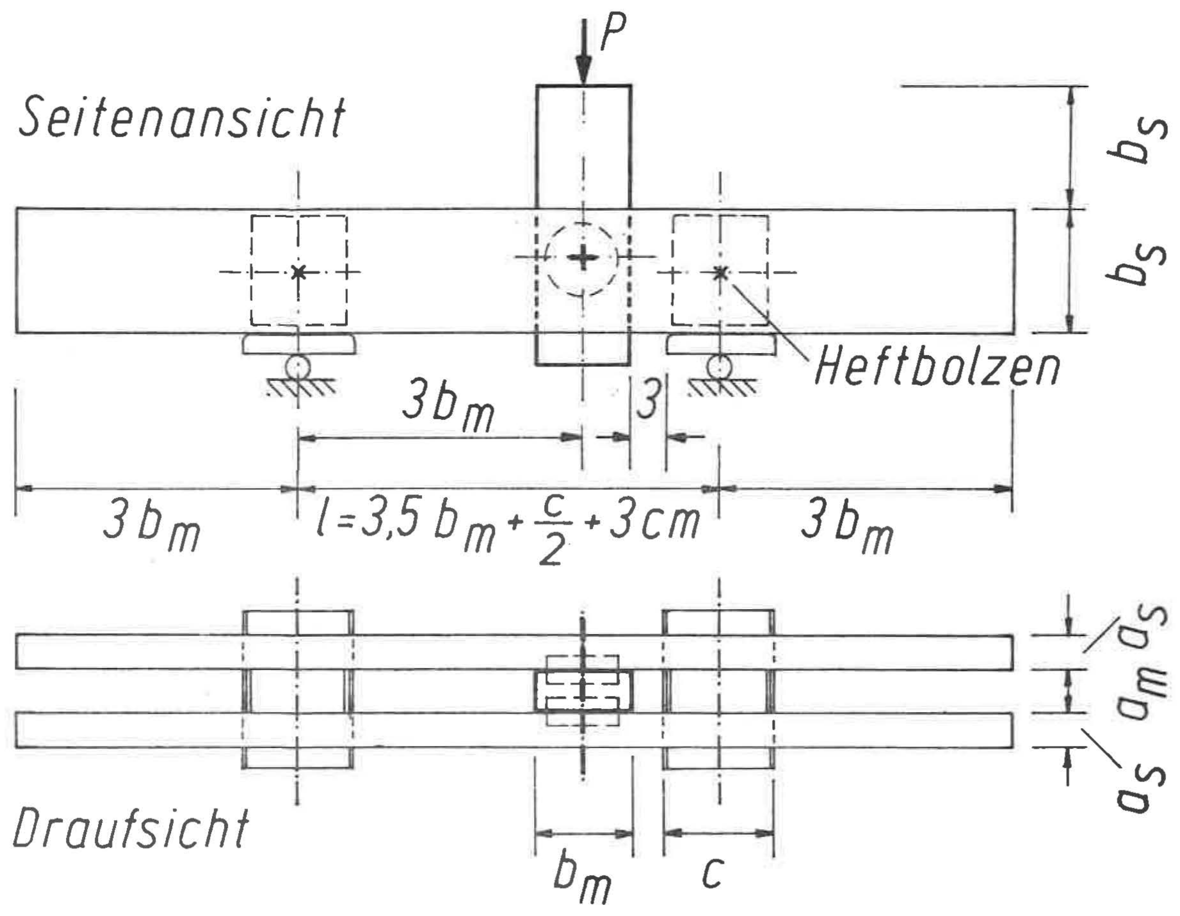
Test methods for nail plates are in course of preparation. A research project is being carried out at Karlsruhe with the object of elucidating what effect time has on the maximum safe load and the load - displacement curve with different fasteners. It is hoped to establish a test procedure similar to the Rilem proposal, should the latter enable an adequate assessment to be made of the short-term strength and the total and permanent displacements with the different fasteners.

Enclosure

| | |
|---|---|
| Längszug - Probekörper | = test piece subjected to tension parallel to grain |
| Draufsicht | = view from above |
| Seitenansicht | = side view |
| Prüfteil | = test length |
| Einspannteil | = restrained length |

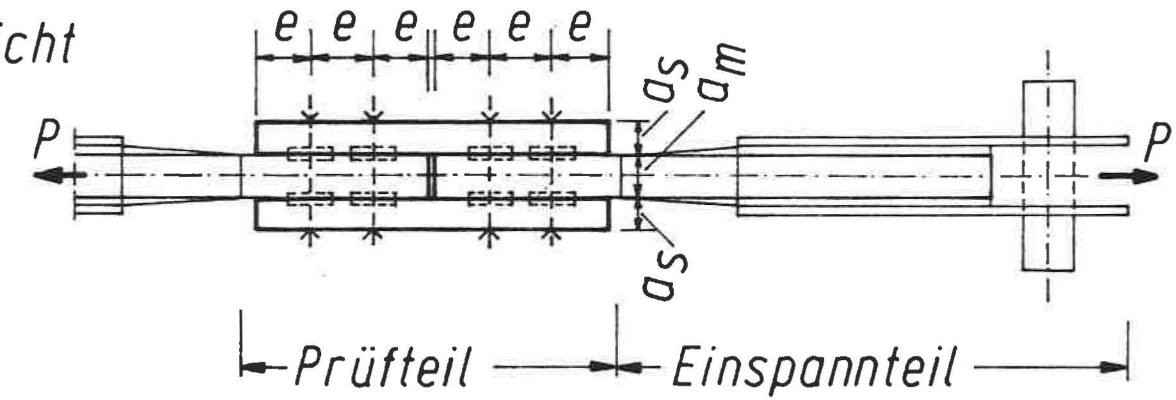
Translations provided on second graph.

| | |
|---|---|
| Querdruck - P. | = test piece subjected to compression perpendicular to grain |
| Heftbolzen | = pin |

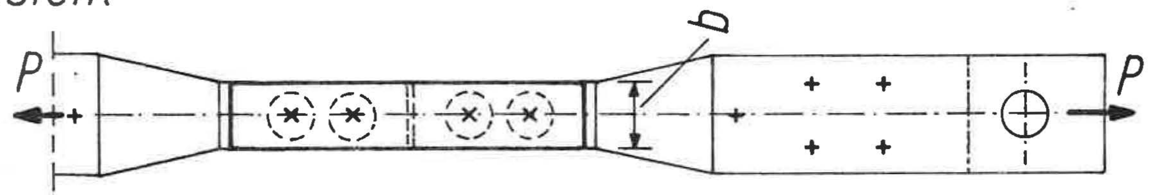



Querdruck - Probekörper
Compression-perpendicular-to-grain specimen

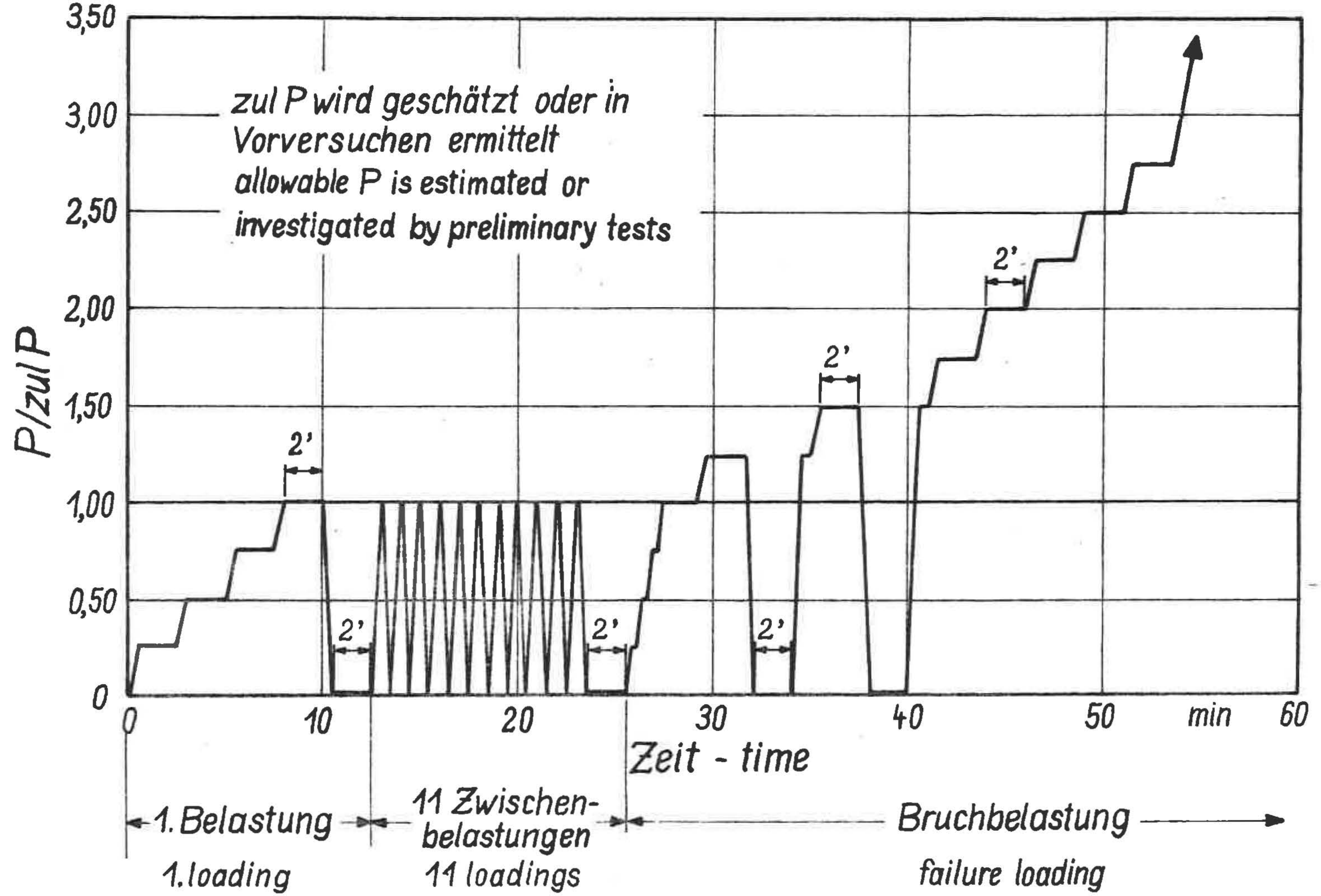
Draufsicht




Seitenansicht




# Längszug - Probekörper
# Tension-parallel-to-grain specimen




| H 341 | Zeitlicher Ablauf des Belastungsversuchs mit einer mechanischen Holzverbindung nach DIN E 4110, Blatt 8<br>Load-time-curve of a test on a mechanical wood joint according to DIN E 4110, sheet 8 | |
|---|---|---|

PAPER 10

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# DRAFT PROPOSAL FOR AN INTERNATIONAL STANDARD FOR STRESS GRADING CONIFEROUS SAWN SOFTWOOD

ECE TIMBER COMMITTEE

GENEVA, SWITZERLAND

PARIS - FEBRUARY 1975

RESTRICTED

TIM/CRP.11/Add.5
16 October 1974

Original: ENGLISH

Timber Committee

Thirty-second session,
14-18 October 1974

DRAFT REPORT (continued)

Addendum 5

(ii) Activities and programme of work of the Timber Committee

(a) Ad hoc meeting on grading rules for coniferous sawnwood

38. Mr. W. Townsley (Canada), Chairman of the ad hoc meeting on grading rules for coniferous sawnwood, presented his report on the meeting (TIM/WP.3/AC.3/4) to the Committee which accepted the meeting's proposal for an international standard for ~~these~~ grading rules of structural sawn softwood. The Committee agreed that research work on the points mentioned in the report and experience in practice were necessary before any standard could be definitely approved. It therefore decided that the standard should be put into operation but be reconsidered within a period of 1-1½ years, when the ad hoc meeting should be reconvened to review developments and suggest changes when appropriate. The Committee agreed that the standard be published as a supplement to the Timber Bulletin for Europe and asked the Secretariat to encourage its wide distribution. The Committee welcomed the very successful co-operation with the International Organization for Standardization (ISO) and the European Softwood Conference on this project.

UNITED NATIONS

# ECONOMIC AND SOCIAL COUNCIL

RESTRICTED

TIM/WP.3/AC.3/4
1 October 1974

Original: ENGLISH

ECONOMIC COMMISSION FOR EUROPE

Timber Committee

Provisional Group of Experts on the Wood-working Industries

Ad hoc meeting of experts on grading rules for coniferous sawnwood

Geneva, 11 to 13 September 1974



79/05

## Chairman's Report

### Introduction

1. A second ad hoc meeting of experts on grading rules for coniferous sawnwood took place from 11 to 13 September 1974. The following ECE countries were represented: Belgium; Byelorussian SSR; Canada; Czechoslovakia; Denmark; Finland; France; Germany, Federal Republic of; Italy; Netherlands; Norway; Poland; Sweden; USSR; United Kingdom; United States of America. The International Organization for Standardization (ISO) and the International Union of Forestry Research Organizations (IUFRO) were also represented.

### Adoption of the Agenda (Item 1 of the Agenda)

2. The Provisional Agenda (TIM/WP.3/AC.3/3) presented by the Secretariat was adopted.

### Election of officers (Item 2 of the Agenda)

3. Mr. W. Townsley (Canada), Chairman of the first ad hoc meeting on this subject, continued as Chairman.

### Draft proposal for an international standard for stress grading of coniferous sawnwood (Item 3 of the Agenda)

4. The meeting was informed of the work of the drafting group set up at the first meeting in October 1973 on the basis of document TIM/WP.3/AC.3/R.6. It expressed its thanks to the drafting group for its rapid and thorough work, and agreed to base its work on the drafting group's proposal.

5. The meeting was also informed of the discussions of the Research and Development Liaison Committee of the European Softwood Conference and of research carried out in Finland and the Netherlands.

6. Mr. F. Palmer (United Kingdom), the Timber Committee's co-ordinator for standardization and Vice-Chairman of the Committee, reported on his discussions with the Secretariat of ISO Technical Committee 55 in Moscow on the basis of document TIM/WP.3/AC.3/R.7. The meeting and the representatives of ISO Central Secretariat and of TC 55 welcomed the progress made in co-operation between the Timber Committee and ISO

and hoped that this trend would continue. The meeting took note of the TC 55 Secretariat's comments set out in Mr. Palmer's report and took account of them in its discussions.

7. The meeting agreed that it should approve an agreement at this session, which should serve as a platform for all international action in the field of grading of structural coniferous sawnwood. A few delegations expressed reservations about the adequacy of the theoretical foundations, but it was pointed out that progress would be made both in theoretical research and in practical application during the initial period of the agreement. It would doubtless be modified as short-comings became apparent and in order to harmonize with national legal frameworks. The meeting stressed, however, the urgency of reaching agreement on a basic document, even if this were to be modified later.

8. The meeting discussed the principal factors to be taken into account when deciding on grading rules for coniferous sawnwood and noted especially the following:

(a) Great importance was attached to the question of yields and reject rates which may determine the economic feasibility of a grading system. Many delegations considered that the reject rates of around 20% measured with the draft agreement proposed to the meeting were excessively high.

(b) The meeting considered that the practicability of a grading system was equally important. The grader should be able to make accurate decisions at a speed which would enable a reasonably fast throughput. In this connexion, it was pointed out that the definition of two separate margin conditions would complicate the grading decisions.

(c) Furthermore, it was considered desirable that there should be an even distribution between the grades and especially that sufficient wood should fall into the lower grade, which would be in greater demand, at least initially, for price reasons.

9. The meeting discussed at length the question of the relationship between a grading system defined by the limitation of strength-reducing characteristics and the design stresses which would be allotted to the grades. It stressed the vital importance of this question and decided that the agreement proposed should include an annex on the subject. It was informed that Working Party 18 (Timber Structures) of the Conseil International du Bâtiment (CIB) was working on this question and would welcome the new grading rules as a basis for its work. The meeting requested CIB W18 to report on design stresses for the new grades.

10. On the basis of these considerations and after lengthy discussion of the draft proposal, the meeting approved the attached draft proposal for submission to the Timber Committee at its thirty-second session, which it agreed should be presented by the Chairman in his report to the Timber Committee.

Further action

11. The meeting stressed again the provisional nature of the agreement and the need for reconsideration of the document within a period of 1-1½ years, and agreed that a suggested plan of action be formulated in the introduction to the agreement.

12. The meeting considered that much research work remained to be done, notably on:

(a) the design stresses to be allotted to the grades
(b) the effect of the width of annual rings on strength
(c) the effect of wane on strength
(d) the yield and reject rate of the new grades.

Several national research laboratories should participate in this work. To improve co-ordination, the meeting requested participants to send to the Secretariat:

(a) the results of any work undertaken which might invalidate the meetings' decisions
(b) information on the direction of work in progress.

The Secretariat would transmit this information to the members of the drafting group which would meet to prepare the third ad hoc meeting.

13. When definitive agreement had been reached on the main features of the grading system, the document should be passed to ISO, which would prepare the final version and ultimately issue it as an International Standard. The meeting noted that ISO terminology and presentation had been used wherever possible.

14. The Finnish delegation presented to the meeting a draft standard for finger-jointed structural timber. The meeting considered that this subject was clearly related to that of stress-graded coniferous sawnwood and decided to draw the attention of the Timber Committee to the subject of finger jointing as a possible field for further standardization activities.

Any other business (Item 5 of the Agenda)

15. None.

Adoption of the report of the meeting (Item 6 of the Agenda)

16. See paragraph 10 above.

# PROPOSAL FOR AN INTERNATIONAL STANDARD FOR STRESS GRADING OF CONIFEROUS SAWN TIMBER

## INTRODUCTION

Two visual grades, one at the GS level of BS 4978 and the other appreciably higher than the SS grade, are specified and provision is made for the use of machine stress grading.

The meeting considered, however, that further investigation may be required to determine if the yields of timber graded to this draft proposal from normal sawmill production are acceptable. The stress values that can be assigned to the grades must also be of considerable importance but, as the first ad hoc meeting agreed, this question was not examined in depth at this stage.

It was the view of the meeting that while a basic grading document was necessary as a platform for further work on stress grading, research on testing presently underway and practical experience necessitated a review of the current proposal within a year or 18 months. It was therefore recommended that the present ad hoc meeting be reconvened at a convenient time within the period suggested to review developments and suggest changes where appropriate. It is further recommended that the drafting committee be reconvened within 6 to 9 months to prepare documents for the proposed ad hoc meeting.

Note: In accordance with ISO practice, indented paragraphs contain matter which is supplementary to the main document.

## 1. SCOPE AND FIELD OF APPLICATION

This draft proposal covers structural load-bearing coniferous timber with basic sawn dimensions of not less than 38 mm thickness and 75 mm width. The tolerances in dimensions shall be in accordance with ISO/R 738-1968. Regularizing, providing it does not remove more than 1 mm, does not imply a change in grade.

The draft proposal defines two visual stress grades, provisionally designated EC1 and EC2 and two machine stress grades MEC1 and MEC2, and specifies the conditions which must be satisfied for the acceptance of machine stress graded timber.

It is however recognized that the need may exist for supplementary, national or international grades in addition to those mentioned above and their use is not precluded.

The permissible limits for knots are specified in terms of knot area ratios (KAR).

Knots may be assiessed by this method of measurement but other methods are acceptable provided they are based on knot displacement and their equivalence with the KAR method has been established.

### Moisture content

Permissible defects are specified for timber at a moisture content of 20%. (Where grading is done at a higher content, then due allowance should be made for the possible effect of subsequent drying.)

Operational control

Grading should be carried out by properly trained and qualified personnel and there should be adequate supervision and control to ensure that the required standards of grading are maintained.

Where grading is done mechanically, the grading machine used must be of a type approved by a competent authority and must be subject to periodic and unannounced inspections.

The machine must be calibrated, maintained and operated under a scheme of supervision prepared by the approving authority.

2. TERMS AND DEFINITIONS

Where timber terms are used they have the meaning assigned to them in ISO/R 1031-1969 and in addition the following definitions apply.

2.1. Fissures. A longitudinal separation of the fibres, appearing on a face, edge or end of a piece of timber, and including checks, shakes and splits.

2.2. Total knot area ratio (Total KAR). The ratio of the sum of projected cross-sectional areas of all knots intersected by any cross-section, to the total cross-sectional area of the piece (see figure 2). The methods for determining the knot area ratio in cases of dispute are set out in Annex 2.

2.3. Machine stress graded timber. Timber which has been non-destructively graded by an approved system of measuring one or more of its mechanical properties. The system being such that, together with any necessary visual inspection, grade stresses may be assigned.

2.4. Margin. The areas adjoining the edges of the cross-section, each of which occupies one-quarter of the total cross-sectional area of the piece (see figure 1). Square pieces are to be graded on their most unfavourable aspect.

2.5. Margin knot area ratio (Margin KAR) The ratio of the sum of the projected cross-sectional area of all knots or portions of knots in a margin intersected at any cross-section to the cross-sectional area of the margin. The methods for determining the knot area ratio in cases of dispute are set out in Annex 2.

2.6. Visually stress-graded timber. A piece of timber which has been graded by visual inspection by properly trained and qualified personnel and to which grade stresses can be assigned.

3. MEASUREMENT

3.1. Knots. Knots shall be assessed by their Total KAR and Margin KAR. In making this assessment knots of less than 5 mm diameter on any surface of the piece may be ignored. No distinction shall be made between knot holes, dead knots or live knots. The method of assessing KAR is illustrated in figure 2.

3.2. Slope of grain. Slope of grain shall be assessed as the inclination of the wood fibres to the longitudinal axis of the piece. The slope shall be expressed as the number of units of length over which unit deviation occurs. It shall be measured over a distance sufficiently great to determine the general slope, disregarding local deviations. Where there is sloping grain on both the edge and face of a piece the combined slope should be taken into consideration. The method of assessing slope of grain is illustrated in figure 3.

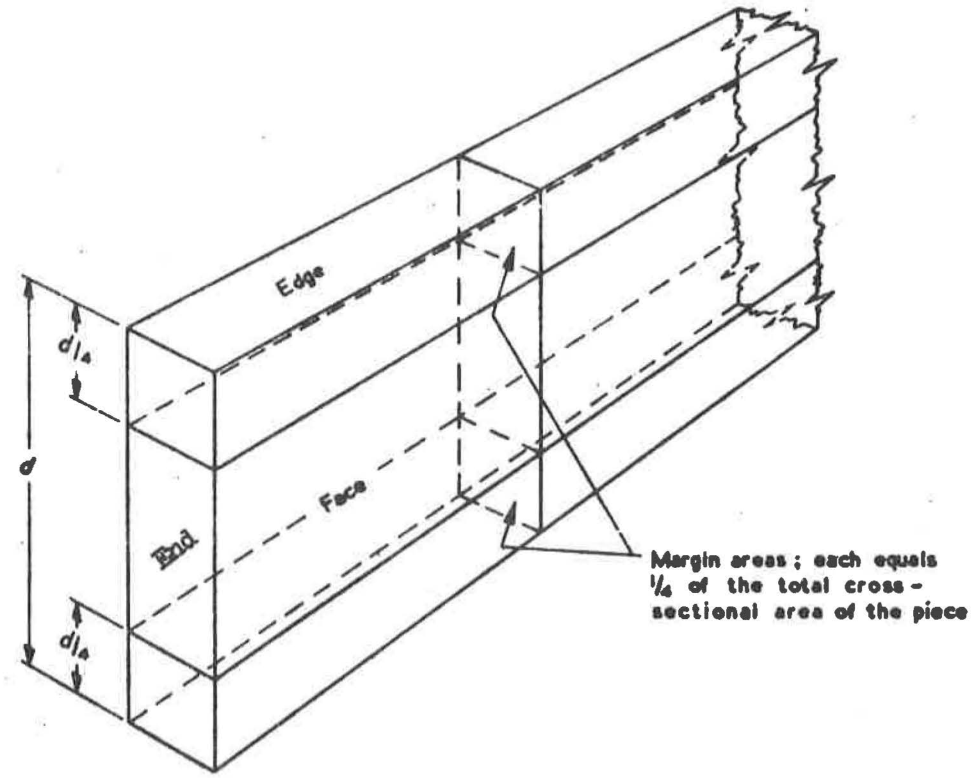
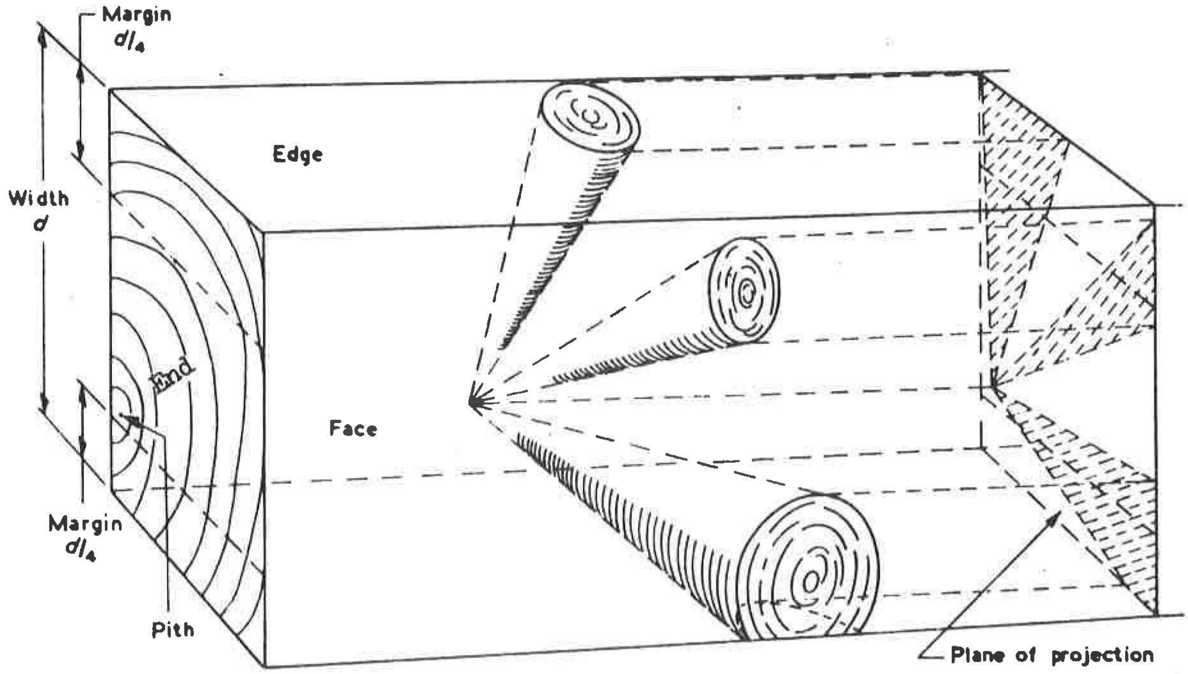
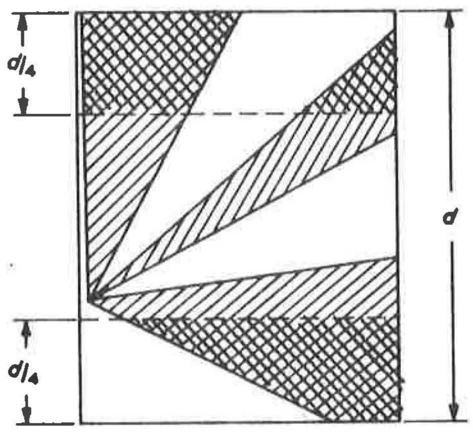



Fig. 1. **Edge, end, face and margin areas**



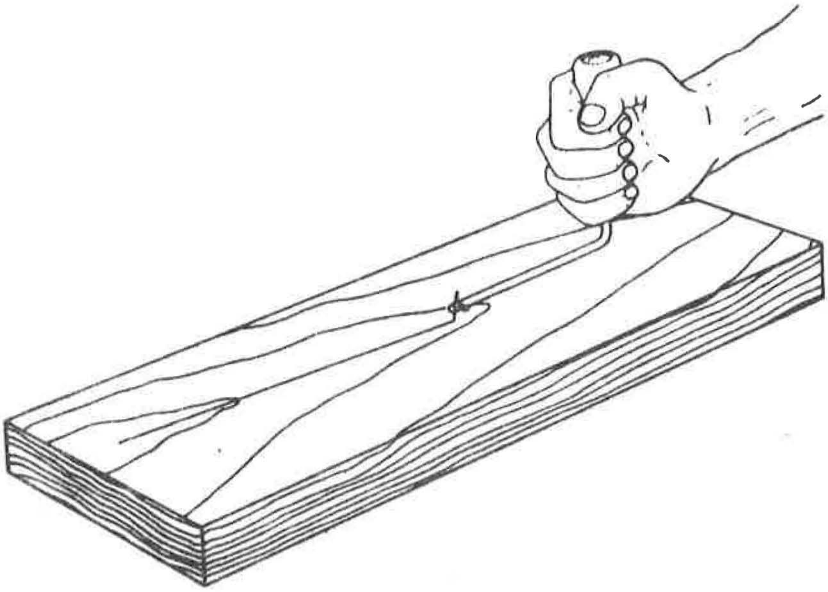

(a) Axonometric view showing in three-dimension a group of knots in a piece and their projection on a cross-sectional plane.



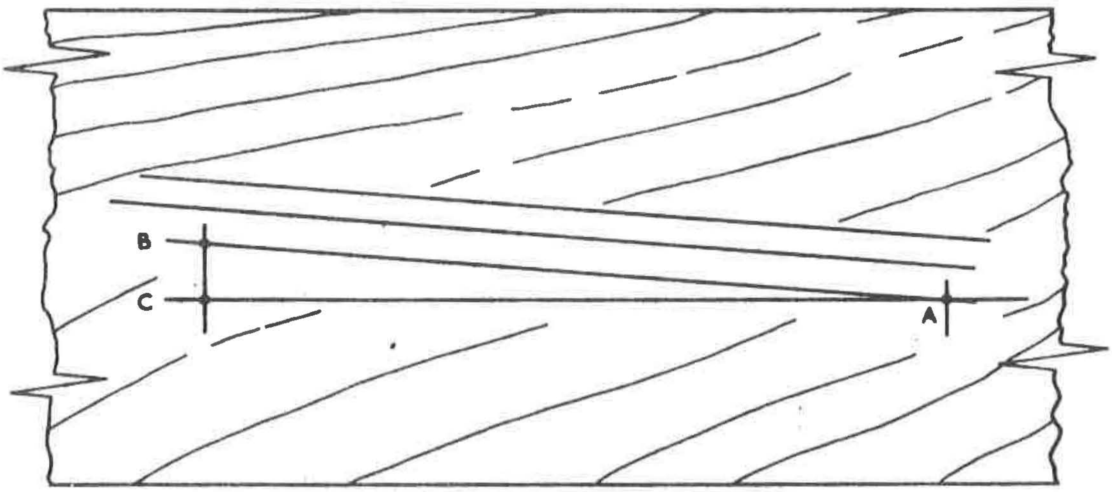

(b) Front view of projection plane, showing projection of knots (hatched) and those parts which fall in the margin area (cross-hatched)

Fig.2. Knots

3.2. Slope of grain. Slope of grain shall be assessed as the inclination of the wood fibres to the longitudinal axis of the piece. The slope shall be expressed as the number of units of length over which unit deviation occurs. It shall be measured over a distance sufficiently great to determine the general slope, disregarding local deviations. Where there is sloping grain on both the edge and face of a piece the combined slope should be taken into consideration. The method of assessing slope of grain is illustrated in Figure 3.



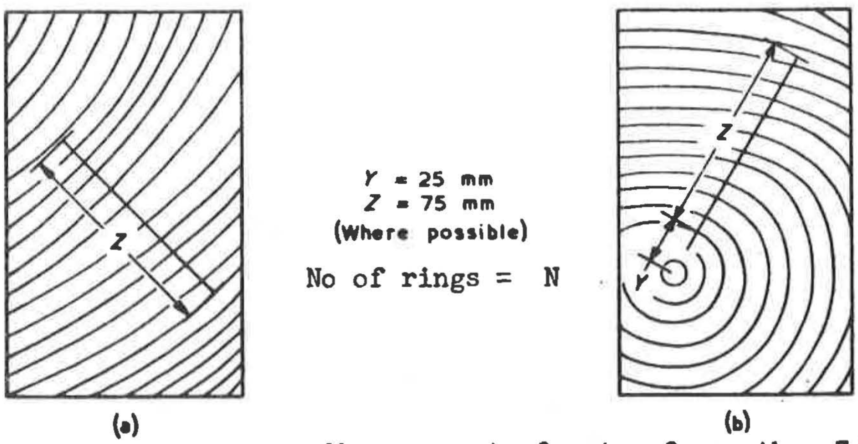
**Use of scribe**




Slope of grain = 1 in $\frac{AC}{BC}$

Fig. 3. Slope of grain

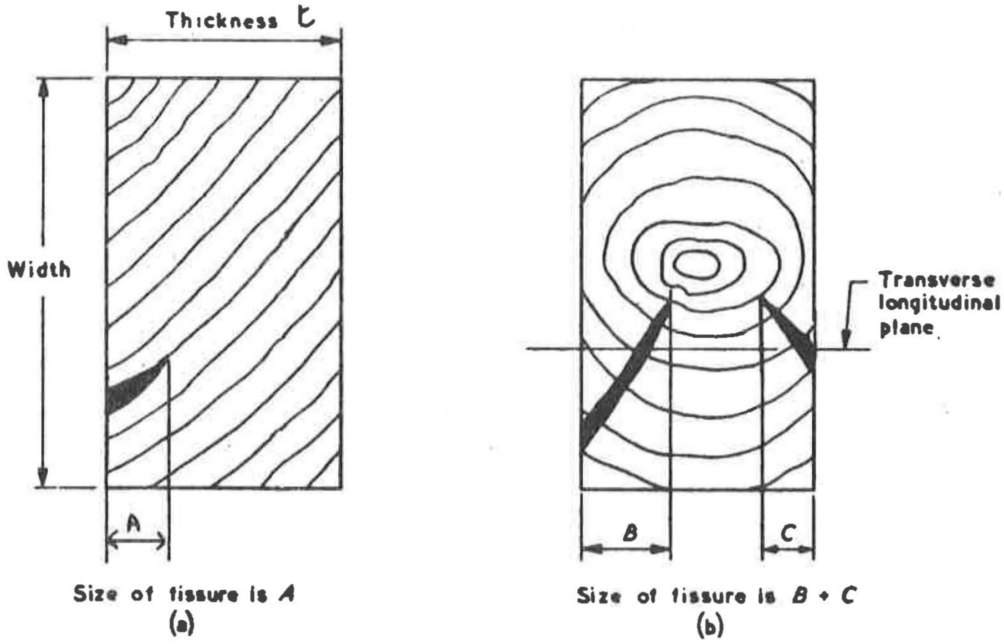
3.3. Rate of growth. Rate of growth shall be assessed on an end of the piece and shall be taken as the average width in mm of the growth rings. The measurement shall be made on the longest line, as near as possible normal to the growth rings and commencing 25 mm from the pith when this is present. The method of assessing rate of growth is illustrated in Figure 4.




Measurement of rate of growth = Z/N mm

Fig.4. Rate of growth

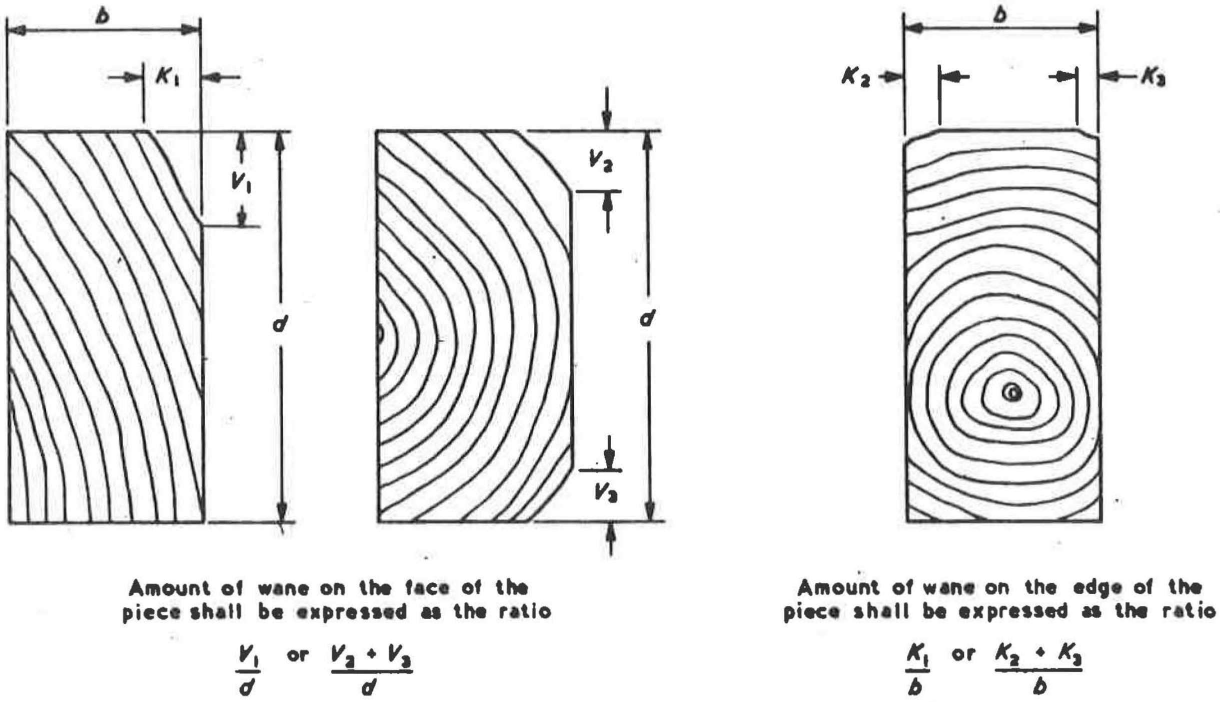
3.4. Fissures. The projected depth shall be taken as the distance between lines enclosing the fissure and parallel to a pair of opposite faces. Fissures shall be assessed as the ratio of their projected depth to the dimension of the section. If a transverse longitudinal plane cuts through two or more fissures on opposite faces then the sum of their depths shall be taken as the size of the defect. When a fissure occurs on the surface of a piece, its depth may be verified by means of a feeler gauge not exceeding 0.2 mm thick. The method of assessing fissures is illustrated in Figure 5.




Measurement of Fissures = A/t and (B+C)/t

Fig. 5. Fissures

3.5. <u>Wane</u>. Wane shall be assessed as the ratio of the projection of the wane on a surface to the full width of that surface. The method of assessing wane is illustrated in Figure 6.




<u>Fig. 6</u>. Wane

3.6. Distortion. Bow, spring and twist shall be assessed over a 3 m length, and cup over the width of the piece.

The amount will largely depend on the moisture content at the time it is measured. A precise definition to cover all conditions and applications cannot therefore be given and guidance only, as to what might be considered acceptable limits, but not typical of any parcel of timber, is provided.

Where for a particular reason **other limits** than those indicated are required, this should be subject to contract between purchaser and supplier. The methods of assessing distortion are illustrated in Figure 7.

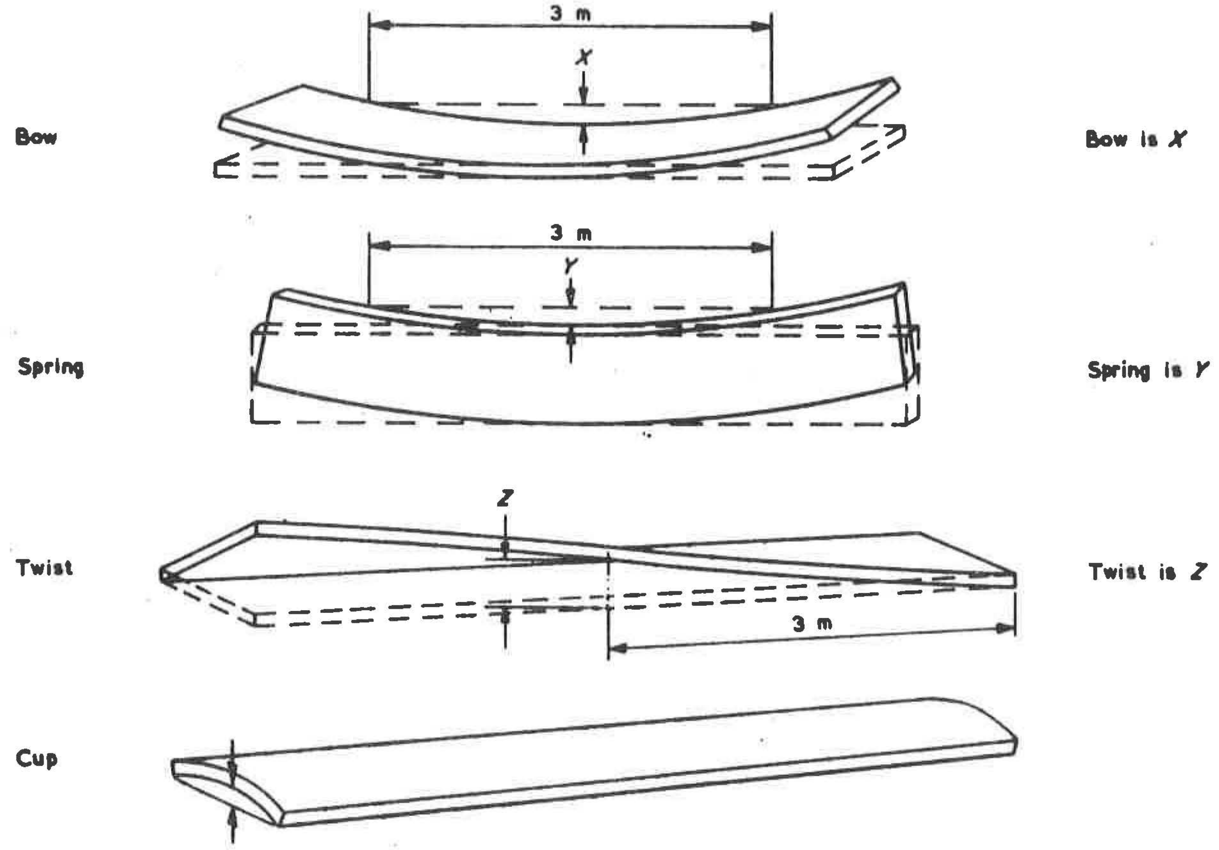



Fig. 7. Distortion

3.7. Pitch pockets and inbark. Pitch pockets and inbark shall be assessed in the same way as fissures.

4. VISUAL GRADES (EC1 and EC2)

4.1. Grade requirements. Two visual stress grades called EC1 and EC2 are specified. To qualify for a grade a piece shall not contain characteristics which exceed the limits given in table 1 (Examples of knots which fall into EC1 and EC2 grades are shown in figure 8) and in addition:

Where two or more knots, or groups of knots, both with knot area ratios exceeding 90% of the permissible ratio, are separated in a lengthwise direction by a distance of less than half the width of the piece, the piece shall not qualify for the grade.

Any piece which contains defects such as compression wood, fungal decay (but not sapstain), mechanical damage, combinations of knots and/or other characteristics etc. which may cause a decrease in strength properties to an amount which threatens the serviceability of the piece, shall be excluded from the grades.

Sapstain is not a structural defect.

For visual reasons it is however generally limited in incidence and extent in any one parcel.

4.2. Marking. Each piece of visually stress graded timber shall have the following information clearly and indelibly marked on one face.

Alternatively, and for an interim period only, the mark may be placed on one edge or one end of the piece.

1. The company responsible for the grading
2. The grade of the piece
3. The species or species group
4. The control authority, where appropriate

4.3. Acceptance limits. On inspection of a representative portion of a parcel not more than 10% of those pieces shall contain any one characteristic exceeding by more than 15% the limit specified for the grade. If those limits are exceeded the total parcel shall be regraded.

The deviation in grading is allowed only to take into account possible differences between individual graders.

5. MACHINE GRADES (MEC1 and MEC2)

5.1. Grade requirements. Two machine stress grades are specified, namely MEC1 and MEC2 which have the same stress values in bending as the corresponding EC1 and EC2 visual grades. (Where information is available additional machine stress grades, at other bending stress or modulus of elasticity values, may be produced and specified.)

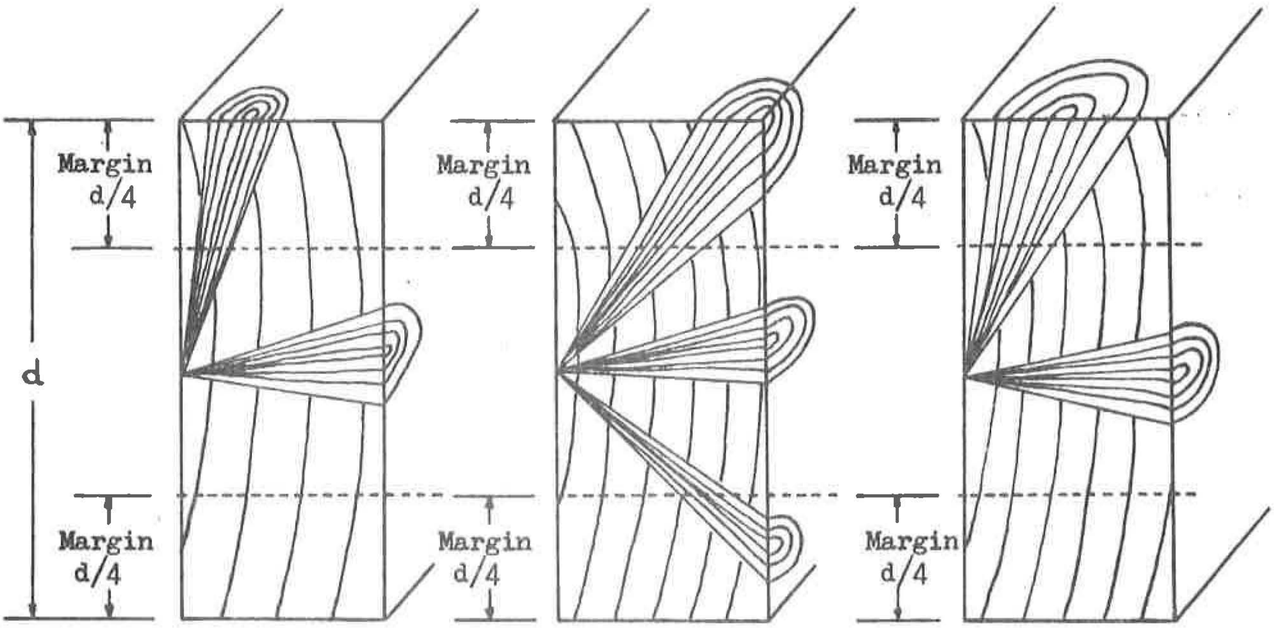
To qualify for a grade each piece must be passed through an approved stress-grading machine and must be classified by the machine as complying with the grade. In addition a visual inspection of each piece must be made to ensure that characteristic other than knots, slope of grain and rate of growth, satisfy the permissible limits for the grade. For the MEC1 and MEC2 grades the permissible limits for the other characteristics shall be as given in table 1.

Table 1

PERMISSIBLE LIMITS FOR THE EC1 AND EC2 VISUAL STRESS GRADES

| Characteristic | Grade | | |
|---|---|---|---|
| | EC1 1/ | EC2 1/ | |
| Knots: (see definitions)<br>Margin KAR<br>Total KAR | <br>$\leqslant 1/4$<br>$\leqslant 1/4$ | <br>$\leqslant 1/2$<br>$\leqslant 1/2$ | <br>$> 1/2$<br>$\leqslant 1/3$ |
| Slope of grain: | 1 in 10 | 1 in 6 | |
| Rate of growth: | Average width of annual rings not more than 7 mm | | |
| Fissures: | | | |
| Not more than half the thickness | unlimited | unlimited | |
| More than half but less than the full thickness | Not more than 1/4 of the length but 600 mm maximum | Not more than 1/4 of the length but 900 mm maximum | |
| Equal to the thickness | Only permitted at the ends with a length of not more than the width of the piece | Not more than 600 mm and if at the ends with a length of not more than 1½ times the width of the piece | |
| Wane: | 1/4 of the thickness by 1/4 of the width, full length | 1/3 of the thickness by 1/3 of the width, full length and in addition 1/2 permitted in 300 mm length not nearer the ends than 300 mm | |
| Distortion: | | | |
| Bow | Should not generally exceed 20 mm in any 3 m length | | |
| Spring | " " " " 15 mm in any 3 m length | | |
| Cup | " " " " 1 mm per 25 mm of width | | |
| Twist | " " " " 1 mm per 25 mm of width in any 3 m length | | |
| Pitch pockets and inbark | The same limits as fissures | | |
| Insect damage: | Worm holes and pin holes are permitted to a slight extent in a few pieces.<br>No active infestation is permitted<br>Wood wasp holes are not permitted | | |

1/ For guidelines to the stress levels aimed at by these grades, see Annex 1




a. Margin KAR is not greater than 1/4 and Total KAR not greater than 1/4.

Therefore
Grade = EC1

b. Margin KAR is not greater than 1/2 and Total KAR not greater than 1/2.

Therefore
Grade = EC2

c. Margin KAR is greater than 1/2 and Total KAR not greater than 1/3.

Therefore
Grade = EC2

Fig. 8 . Examples of knot area ratios which determine either EC1 or EC2 grades

5.2. Marking. Each piece of machine stress graded timber shall have the following information clearly and indelibly marked on one face.

1. The licence number of the grading machine
2. The company responsible for the grading
3. The grade of the piece
4. The species or species group
5. The control authority, where appropriate

5.3 Acceptance limits. On inspection of a representative number of pieces of a parcel not more than 10% of those pieces shall contain any one visually determined characteristic exceeding by more than 15% the limit specified for the grade. If those limits are exceeded the total parcel shall be regraded.

## Annex I referring to table 1

### Design stresses

It is necessary for structural use to lay down design stresses for each grade, and according to the various species. At present however different countries use different design methods and have different design stresses for similar grades of timber. In many cases countries arrive at similar design solutions although they use different design stresses. This arises because of differences in loading conditions and methods of design as well as methods of deriving design stresses.

In the interests of international trade it is desirable to have one acceptable set of grading rules even if countries ascribe different design stresses to them. Attempts are being made elsewhere than in the Timber Committee to harmonize methods of derivation of design stresses and until this is achieved it is not possible to give exact values for use with any grades laid down.

As a guideline to help in deciding on the acceptability of the proposed grades, it is probable that design stresses in bending for European redwood and whitewood for the two grades EC2 and EC1 would be of the order of 5.5-6.5 and 8-10 newtons/mm2 respectively.

# Annex 2

## Method for determining the knot area ratio in cases of dispute

To calculate the worst knot area ratio in any piece of timber use the following method:

(1) Choose that section in the piece which intersects the knot or group of knots of which the knot area ratio produces the lowest grade.

(2) Consider all knots with diameters greater than 5 mm intersected by the chosen section in calculating the knot area ratio of both margin areas and of the whole piece.

Make full scale drawings of the chosen section and mark the margin areas by dotted lines. Mark points on the appropriate side of the rectangle representing any knot on that surface. The points marked shall represent the widest projection of the knot on that face or edge.

(3) Calculate the knot area ratio and the area of the margin occupied in two ways according to whether the pith occurs within the cross section or not.

For the purpose of estimating the position of the pith within or without the cross section examine the nearest end of the piece and assume that all annual rings are concentric with the pith.

a. If the pith is within the cross section join the points representing the limits of the knots on the drawing by straight lines to a point representing the estimated position of the pith. Measure the area within these lines which corresponds to knots for the whole cross section and for that area which lies within either margin.

b. Where the pith is outside the cross section mark its estimated position at an appropriate position on the drawing. Join up the points on the perimeter of the drawing in a manner appropriate to the assumption that each knot is approximately a cone with its apex at the pith. Measure the area thus enclosed, corresponding to the estimated position of knots, for the whole cross section and for both margin areas.

(4) In both the cases referred to in (3)a and (3)b express:

a. The total area of knots within each margin area as a proportion of the whole of that margin area for the purpose of deciding whether a margin condition exists or not.

b. The total area of knots within the cross-sectionl area of the piece as a proportion of the cross-sectional area of the piece for the purpose of determining the knot area ratio at that section.

PAPER 11

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# DERIVATION OF GRADE STRESSES FOR TIMBER IN THE UK

by

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ENGLAND

PARIS - FEBRUARY 1975

DERIVATION OF GRADE STRESSES IN THE UK

W T Curry (Princes Risborough Laboratory, BRE)

## INTRODUCTION

The stress values for timber given in the present edition of the Code of Practice for the Structural Use of Timber (CP 112:Part 2:1971 and Amendment No 1 (AMD 1265)) are mainly determined from the results of laboratory tests on small clear specimens. The general approach may be identified as the strength ratio method and its only attraction is that it is a more economical method of dealing with a rather wide range of species than the more direct approach of testing samples of graded timber of each species in structural sizes (ie the species by species approach). More recently greater attention has, however, been paid to this latter method, but in order to make it more flexible, attention has also been given to recording detailed descriptions of the characteristics of the fracture sections, in terms of knots, sloping grain, local modulus of elasticity, etc so that the effect of changes in grade specifications can be assessed. Simply to classify test specimens according to present grade definitions is too restrictive.

The Code of Practice is presently being revised to adopt the principles of limit state design and this has involved some basic changes in the procedures for deriving grade stress values. It will, however, be some time before the revision is published, and even then it will be necessary for both the old and new Codes to continue in existence for perhaps upwards of 5 years, in order to permit sufficient time for associated standards, Codes and Regulations to be amended. Both procedures have, therefore, been outlined in this paper.

One other point is worth noting. Stress values for timber can only be determined from destructive tests on samples and when stress values are assigned to a species as a whole, it is presumptious to assume that the samples tested adequately describe the characteristics of the population. It is indeed questionable whether it is population characteristics that are required as a basis for deriving design stresses, since many sub-divisions will exist with widely different strength properties, each of which could constitute an extensive programme of structural application. For example whitewood is imported into the UK from a number of European countries, and an examination of the tests results from samples of these imports showed that, using standard procedures, design stress values differing by as much as 75%, could be obtained. Not only can there be regional differences, but there can also be differences due to time in both the short-term (harvesting and logging changes) and in the long-term (silvicultural changes). There is, therefore, a continuing need for sampling and for testing timber in small clear specimens and in structural sizes, and it is desirable that standard procedures should be available for doing this, as well as for deriving design stresses from the results. Small clear specimen test procedures are largely standardised in Europe but this is not the case for structural timbers, where not only test procedures, but the recording of characteristics of the fracture sections should be standardised. This will become increasingly important if a collective contribution is to be made to the formulation of ECE grades and the definition of appropriate stress values for these grades.

PART A: PRESENT PROCEDURES FOR DERIVING GRADE STRESSES

In order to clarify the meaning of some of the terms used the following definitions apply:

BASIC STRESS. The stress which can safely be permanently sustained by timber containing no strength reducing characteristics.

GRADE STRESS. The stress which can safely be permanently sustained by timber of a particular grade.

PERMISSIBLE STRESS. The stress which can safely be sustained by a structural component under the particular conditions of service and loading.

DRY EXPOSURE. Any exposure where the moisture content of timber will not exceed 18%.

GREEN EXPOSURE. Any exposure where the moisture content of timber will exceed 18%.

From the results of small clear specimen tests (2 cm standard BS 373) at the green condition, basic stress values for the green exposure are obtained from:

$$f_b = f_m\,(1-CS)/K_1$$

where $f_m$ is the mean ultimate stress from the test results, S the coefficient of variation, and C and $K_1$, have the following values, irrespective of the number of test results.

| Property | C | | $K_1$ |
|---|---|---|---|
| | Probability | Value | |
| Bending and tension | 0.01 | 2.33 | 2.25 |
| Compression parallel to grain | 0.01 | 2.33 | 1.4 |
| Compression perpendicular to grain | 0.025 | 1.96 | 1.2 |
| Shear parallel to grain | 0.01 | 2.33 | 2.25 |
| Mean modulus of elasticity | - | - | 1.0 |
| Min modulus of elasticity | 0.01 | 2.33 | 1.0 |

The reduction factor $K_1$ is assumed to include adequate allowance for the influence of rate of loading, and specimen size, and to provide for a factor of safety. The compression perpendicular to grain stresses are derived from the results of a standard hardness test (Janka).

The grade stresses for the green exposure are determined by multiplying the basic stresses by a grade strength ratio factor $K_2$, having the following values:-

Table 1
GRADE STRENGTH RATIOS

| Property | Grade | | | | | |
|---|---|---|---|---|---|---|
| | CP 112 | | | | BS 4978 | |
| | 75 | 65 | 50 | 40 | SS | GS |
| Bending | 0.75 | 0.65 | 0.50 | 0.40 | 0.50 | 0.35 |
| Tension | 0.75 | 0.65 | 0.50 | 0.40 | 0.35 | 0.245 |
| Compression parallel to grain | 0.75 | 0.65 | 0.50 | 0.40 | 0.65 | 0.45 |
| Compression perpendicular to grain | 0.875 | 0.875 | 0.75 | 0.75 | 0.75 | 0.67 |
| Shear parallel to grain | 0.75 | 0.65 | 0.50 | 0.40 | 0.50 | 0.50 |
| Mean modulus of elasticity | 1.0 | 1.0 | 1.0 | 1.0 | 1.09 | 0.98 |

Minimum modulus of elasticity values are obtained by assuming that the coefficients of variation in modulus of elasticity are the same for each grade, and reducing the grade stress to the 1% lower exclusion value.

The grade stresses for the dry exposure, which are only applicable to timber members less than 102 mm thick, are in the case of the CP 112 grades derived from the grade stresses for the green condition. In the case of the BS grades they are generally obtained from dry basic stress values determined from green and dry small clear specimen test results. For the CP 112 grades the grade stresses in bending tension and compression parallel to the grain for the 75 and 65 grades are taken as 87.5 and 72.5% of the corresponding green basic stress. For the 50 and 40 grades no increase directly associated with the effect of drying is provided. For compression perpendicular to the grain the grade stresses are taken as 1.5 times the corresponding green values. For the 50 and 40 grades the dry grade stresses are obtained by multiplying the green grade stresses by factors of 1.12 for bending and tension and 1.08 for compression parallel and shear, to allow for shrinkage effects. It should also be noted that the values for the dry basic stresses do not reflect the full effect of drying, as determined for small clear specimens.

For the BS 4978 grades the dry grade stresses are obtained from basic stresses which are calculated using the log strength relations for individual species and properties derived from green and dry (about 12%) small clear specimen test results. It is assumed that at the required moisture content of 18%, the coefficient of variation is the average of the green and dry (12%) values. The basic stresses for the major properties are then multiplied by the strength ratios given in Table 1. A previous study (PRL Bulletin No 52 Grade Stresses for European redwood and whitewood) justified the use of the dry basic stresses, and tests results at the 18% condition were available for graded samples of redwood, whitewood and Canadian western hemlock from which the grade strength ratios were confirmed. In general strength ratios were determined from US data (ASTM-D245) and minimum estimates of ultimate strength were based on normal distribution functions for small clear test results, and on Weibull functions for structural size tests.

A summary of dry and green grade stress values for redwood and whitewood is given in Table 2. The continued existence of so many visual grades is, of course, undesirable but is necessary at least for an interim period, which may be as long as three years, to permit other associated Codes, Standards and Regulations to be amended. Ultimately the CP 112 numbered grades will disappear.

PART B - REVISED PROCEDURES FOR DERIVING GRADE STRESSES

Currently CP 112 is being revised on the basis of limit state design and this entails a change in the procedures for deriving grade stresses. A fundamental approach on a probability basis is not possible, so the main considerations in the revision of the Code have been (a) to set up a framework for design based on limit states and (b) to ensure that for traditional constructions new designs do not result in substantially larger sections than experience has proved acceptable.

The following definitions apply:

BASIC CHARACTERISTIC STRESS. The value of ultimate stress at the dry exposure condition, derived from standard tests on small clear specimens below which not more than 5% of test results fall: a normal distribution is assumed with a factor of 1.64.

BASIC DESIGN STRESS. The stress derived from the basic characteristic stress by dividing by the appropriate partial safety factor for strength ($\gamma_{m1}$) and adjusting to the long term loading condition, and in the case of bending strength to a section depth of 200 mm. Note: that the extreme depth in bending has been reduced from the 300 mm value of the present code.

GRADE CHARACTERISTIC STRESS. The value of ultimate stress at the dry exposure condition, derived from standard tests on full size specimens of a particular grade below which not more than 5% of the test results fall: a Weibull 3 parameter distribution is assumed.

Table 2

GRADE STRESS VALUES FOR EUROPEAN WHITEWOOD

($N/mm^2$)

| Property | Grade | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Green | | | | | | Dry | | | | | |
| | 75 | 65 | 50 | 40 | SS | GS | 75 | 65 | 50 | 40 | SS | GS |
| Bending[(1)] | 8.6 | 7.6 | 5.9 | 4.5 | 5.9 | 4.0 | 10.0 | 8.6 | 6.6 | 5.2 | 7.3 | 5.1 |
| Tension | 8.6 | 7.6 | 5.9 | 4.5 | 4.0 | 2.9 | 10.0 | 8.6 | 6.6 | 5.2 | 5.1 | 3.5 |
| Compression parallel to grain | 6.2 | 5.2 | 4.1 | 3.1 | 5.4 | 3.7 | 7.9 | 6.6 | 4.8 | 3.8 | 8.0 | 5.6 |
| Compression perpendicular to grain | 1.17 | 1.17 | 1.03 | 1.03 | 1.03 | 0.92 | 1.72 | 1.72 | 1.52 | 1.52 | 1.55 | 1.38 |
| Shear parallel to grain | 1.03 | 0.90 | 0.69 | 0.55 | 0.70 | 0.70 | 1.14 | 0.97 | 0.76 | 0.62 | 0.86 | 0.86 |
| Mean modulus of elasticity | 6900 | 6900 | 6900 | 6900 | 8200 | 7000 | 8300 | 8300 | 8300 | 8300 | 10000 | 8600 |
| Minimum modulus of elasticity | 4100 | 4100 | 4100 | 4100 | 4700 | 4000 | 4500 | 4500 | 4500 | 4500 | 5700 | 4900 |

GRADE STRESS VALUES FOR EUROPEAN REDWOOD

($N/mm^2$)

| Property | Grade | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Green | | | | | | Dry | | | | | |
| | 75 | 65 | 50 | 40 | SS | GS | 75 | 65 | 50 | 40 | SS | GS |
| Bending[(1)] | 8.6 | 7.6 | 5.9 | 4.5 | 5.9 | 4.0 | 10.0 | 8.6 | 6.6 | 5.2 | 7.3 | 5.1 |
| Tension | 8.6 | 7.6 | 5.9 | 4.5 | 4.0 | 2.9 | 10.0 | 8.6 | 6.6 | 5.2 | 5.1 | 3.5 |
| Compression parallel to grain | 6.2 | 5.2 | 4.1 | 3.1 | 5.4 | 3.7 | 7.9 | 6.6 | 4.8 | 3.8 | 8.0 | 5.6 |
| Compression perpendicular to grain | 1.31 | 1.31 | 1.10 | 1.10 | 1.14 | 1.01 | 1.93 | 1.93 | 1.65 | 1.65 | 1.71 | 1.52 |
| Shear parallel to grain | 1.03 | 0.90 | 0.69 | 0.55 | 0.70 | 0.70 | 1.14 | 0.97 | 0.76 | 0.62 | 0.86 | 0.86 |
| Mean modulus of elasticity | 7600 | 7600 | 7600 | 7600 | 8200 | 7000 | 8300 | 8300 | 8300 | 8300 | 10000 | 8600 |
| Minimum modulus of elasticity | 4100 | 4100 | 4100 | 4100 | 4700 | 4000 | 4500 | 4500 | 4500 | 4500 | 5700 | 4900 |

[(1)] The bending stresses apply to sections up to 300 mm deep

GRADE DESIGN STRESS. The stress derived either: (a) by dividing the grade characteristic stress by the partial safety factor for strength ($\gamma_{m2}$) and adjusting to the long term load condition; and in the case of bending stress to a section depth of 200 mm or (b) by multiplying the basic design stress by the grade strength ratio for solid timber and by the combined grade and number of laminations factor for glued laminated timber.

MODIFIED DESIGN STRESS. The stress determined by multiplying the basic design stress or grade design stress by modification factors appropriate to a particular design situation.

These definitions are given in order to provide for the derivation of grade design stresses from either a small clear specimen test approach or from tests on graded timber in structural sizes, and for the introduction of factors to account for differences in the distribution of strength values for small clear specimens and specimens in structural sizes. The basic procedures can be illustrated as follows:-

Small clear specimens

Structural sizes

Lamination

Basic characteristic stress

(5% exclusion value)

Grade characteristic stress

(5% exclusion value)

Basic design stress

(load duration factor, $\gamma_{m1}$
size factor)

Grade design stress

(grade strength ratio)

Grade design stress

(load duration factor, $\gamma_{m2}$
size factor)

Grade design stress

(grade and number of
laminations factors)

Modified design stress

(exposure, size, duration,
notches, etc factors relative
to a particular design
situation)

Since most of the available data on strength relate to small clear specimens, the approach to the derivation of grade design stresses must still be based on these, the structural tests being used to establish strength ratios. Greater attention is, however, being paid to the testing of timber in structural sizes and the species by species approach to the derivation of stresses will become increasingly important. It is also undoubtedly true that some of the available small clear test data reflect significant sampling effects and in order to avoid unnaturally high estimates of basic characteristic stress, minimum coefficients of variation were imposed as follows:

- 15 per cent for bending
- 16 per cent for compression parallel to grain
- 19 per cent for compression perpendicular to grain
- 17 per cent for shear
- 18 per cent for modulus of elasticity

Basic design stresses are derived as follows:

| | |
|---|---|
| Bending | The basic characteristic stress is divided by the product of the factors $\gamma_{m1}$ = 1.15, size = 1.183 and load duration 1.6 ie 2.177 |
| Tension | The basic design stress in bending is multiplied by the factor 0.7 |
| Compression parallel to grain | The basic characteristic stress is divided by the product of the factors $\gamma_{m1}$ = 1.15 and load duration = 1.6 ie 1.84 |
| Compression perpendicular to grain | Hardness test results are converted to "elastic limit stresses" using the equations given in PRL Bulletin No 50 "The strength properties of timbers". Using normal distribution statistics the characteristic elastic limit stress is calculated and converted into an "ultimate stress" by multiplying by 3.0. The basic design stress is then obtained by dividing this basic characteristic stress by the product of the factors $\gamma_{m1}$ = 1.15 and load duration = 1.6 ie 1.84 |
| Shear parallel to grain | The basic characteristic stress is divided by the product of the factors $\gamma_{m1}$ = 1.15, load duration = 1.6 and degrade factors (to allow for the possible degrade of fissures with time, which although perhaps more appropriate to the determination of grade design stress can more conveniently be introduced here) of 1.1 for the green condition and 1.25 for the dry condition. The total reduction factors are therefore 2.024 and 2.3 respectively. |

| | |
|---|---|
| Modulus of elasticity | The basic characteristic stresses were obtained by modifying the small clear test data (after Bodig and Goodman) to correct for shear deflection; no further modification was applied. |

For the revision of CP 112 only the dry basic design stresses will be tabulated but modification factors to adjust these to the green condition, and to permit the geometrical properties of sections to be adjusted as appropriate, will be included. The factors for the adjustment to the green condition are:

| | |
|---|---|
| bending and tension | 0.70 |
| compression parallel to grain | 0.67 |
| compression perpendicular to grain | 0.65 |
| shear parallel to grain | 0.80 |
| modulus of elasticity | 0.85 |

When small clear specimen test results are used, grade design stresses are formed by multiplying the basic design stresses by a strength ratio. It is assumed that strength ratios are independent of species and may be determined either from a consideration of the defects permitted for a grade or from the measured strengths of graded material. Although the latter approach is to be preferred there is not enough test evidence available to employ it generally. For the derivation of basic design stresses, except for modulus of elasticity, $\gamma_{m1}$ has the single value of 1.15. In the case of standard bending tests on graded joists it has been shown that a value for $\gamma_{m2}$ of 1.35 is required in order to obtain reasonable agreement between ultimate strength limit state design, with partial safety factors of 1.4 for dead load and 1.6 for imposed load, and present design. It was also further shown that the difference in the values of $\gamma_m$ could be attributed to the increased variability of graded structural timbers compared with small clear specimens, and to the effect this has on the relative values of the 1 and 5% exclusion limits. Thus to obtain the same results from the structural test approach requires somewhat different values for $\gamma_m$ than are used for the small clear specimen approach, or alternatively the introduction of another factor identified as a grading factor. Present information suggests that this factor would have values of the order of 1.17 for bending, 1.2 for tension and 1.05 for compression. It seems certain, however, that as more test information becomes available, and as experience is gained with limit state design, a more direct approach to the derivation of grade design stresses, using both methods, will become possible. The problem reduces to one of defining appropriate values for the strength ratios. Table 3 gives the strength ratios that have been recommended for the revision of CP 112.

The grade design stresses are then obtained by multiplying the corresponding basic design stress by these ratios. A summary of the basic and grade design stresses for redwood and whitewood is given in Table 4.

Table 3

STRENGTH RATIOS FOR LIMIT STATE DESIGN AND BS 4978 GRADES

| Property | Grade | |
|---|---|---|
| | SS | GS |
| Bending | 0.51 | 0.36 |
| Tension | 0.42 | 0.29 |
| Compression parallel to grain | 0.67 | 0.52 |
| Compression perpendicular to grain | 0.75 | 0.67 |
| Shear parallel to grain | 0.50 | 0.50 |
| Modulus of elasticity | 1.06 | 0.95 |

Table 4

BASIC AND GRADE DESIGN STRESSES FOR REDWOOD AND WHITEWOOD: DRY EXPOSURE, LIMIT STATE DESIGN

($N/mm^2$)

| Property | Whitewood | | | Redwood | | |
|---|---|---|---|---|---|---|
| | Basic | SS | GS | Basic | SS | GS |
| Bending | 21.1 | 10.8 | 7.6 | 21.6 | 11.0 | 7.8 |
| Tension | 14.8 | 6.2 | 4.3 | 15.1 | 6.3 | 4.4 |
| Compression parallel to grain | 12.0 | 8.0 | 6.3 | 12.9 | 8.6 | 6.8 |
| Compression perpendicular to grain | 3.83 | 2.87 | 2.55 | 3.95 | 2.96 | 2.63 |
| Shear parallel to grain | 2.57 | 1.29 | 1.29 | 2.73 | 1.37 | 1.37 |
| Mean modulus of elasticity | 7100 | 7550 | 6800 | 6500 | 6900 | 6200 |

## CONCLUSIONS

This paper outlines the methods that are currently used in the UK, and those that have been recommended for the revision of CP 112, for the derivation of grade design stress values. Perhaps more important, it indicates the considerable areas of uncertainty that still exist, and the need for more extensive basic information on the questions of sampling, variability and the definition and derivation of strength ratios. When considering stress values, and comparisons between the procedures followed in different countries, account must also be taken of the modifications. permitted for comparable design situations. As far as possible any indirect effect that these have, for example size effect in bending, should be eliminated, or at least placed on a common basis, if an agreed procedure for deriving grade design stresses is to be obtained.

PAPER 12

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# A REVIEW OF LOAD-SHARING IN THEORY AND PRACTICE

An Interim Report

by

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PARIS - FEBRUARY 1975

# A REVIEW OF LOAD-SHARING IN THEORY AND PRACTICE

by E. Levin, Timber Research and Development Association, United Kingdom

## 1. INTRODUCTION

1.1 Load-sharing in structures is by no means a universally accepted concept. Nevertheless, in a number of countries certain provisions are made in timber design codes or standards, to reduce certain imposed loads or increase permissible stresses where a number of members in a structure are considered to act together in resisting the loads.

1.2 Because a situation which may be considered one where load-sharing exists, arises very frequently in the most common and mundane timber structures, such as joisted floors or raftered roofs, there has naturally been an urge, in the interests of economy in design, to take advantage of any reduction in member sizes which may be justified by load-sharing without sacrificing safety. However, the structures are generally statically indeterminate and although the constructions are simple the problems are complex due to the many variables which have to be taken into account. The availability in recent decades of high speed computers appears to have provided the means of dealing simultaneously with the many variables in attempts by many researchers at a better understanding of the load distribution and modes of failure of load-sharing timber structures.

1.3 The purpose of the present literature survey has been to find out a number of things: what are the definitions and concepts of load-sharing systems in the codes and standards that make special provisions for them? What are these provisions, how do they compare and what changes are proposed? What analytical or experimental evidence is there in support or refutation of these provisions? What particular factors in the complex load-sharing phenomena have been successfully isolated and dealt with? What areas of research both theoretical and experimental appear to have been neglected?

1.4 It is not proposed in this interim report to deal with all these matters, but only with some of them. The literature survey has not yet been completed nor all the available material analysed and summarised. It is hoped to deal with the remaining topics mentioned in the next report.

N.B. Numbers in brackets refer to cited literature.

## 2. DEFINITION OF LOAD-SHARING SYSTEMS

2.1 The definitions of load-sharing systems in the codes and standards which make special design provisions for them are by no means identical but have much in common.

2.2 The British Code (CP 112) (1) Both the 1967 and 1971 (metric) editions define a load-sharing system as one "where four or more members can be considered to act together to support a common load" (sub-clause 3.12.4 in the 1971 edition). It is further added that it "should only be applied to rafters, joists, trusses and stud walling. These should be spaced not more than 610 mm apart, and adequate provision should be made for lateral distribution of loads by means of purlins, binders, boarding, etc."

2.3 The Australian Standard (CA 65 - 1972) (2) refers to "parallel support systems .... comprised of two or more elements .... effectively connected so that all the elements are constrained to the same deformation" (sub-clause 2.4.5.1). It also contains load-sharing provisions for grid systems, defined as "constructions .... such that three or more members act together to support either an overlying set of members usually laid at right angles to the supporting members or a structural sheathing material" (sub-clause 2.4.5.2). Laminated members, whether horizontally or vertically laminated are specifically excluded from the provisions of the "parallel support" systems (2.4.5.1) and separate design provisions are made for them (as is the case indeed in most other design codes and standards).

2.4 The ASTM Tentative Recommended Practice (D2018-62T, 1962) (3) applied to "load-sharing members .... defined as framing or supporting members, such as joists, studs, planks, or decking, that are contiguous or are spaced not more than 24 inches in frame construction and are joined by floor, roof or other load-distributing element".

2.5 The Swedish Standard SBN 67 (according to Noren) (4) defines load-sharing as "when several timber members co-operate and it can be presumed that the strength is increased thereby, i.e. in sheet piling ..." (para. 27:1325A).

2.6 The Dutch Loading Code, NEN 385 (TGB-1970) (5) permits load-sharing in respect of concentrated loads on floors, beams, etc. (clause 2.2.1a) and in respect of local concentrated vertical loads on roof planes (clause 2.2.2a) in the following terms: "the spreading of the concentrated loads over several bearing members may be taken into account accordingly as the joint action of these members is assured".

2.7 The Draft Revision of CP 112 (6) contains a modified definition of load-sharing systems as "where two or more pieces of timber act together in such a fashion as to be equally strained under load, or where it can be shown that effective lateral distribution of loading occurs...." For four or more members the Draft specifies "for example ... rafters, floor joists and wall studding spaced not further apart than 600 mm and joined by purlins, binders, boarding, etc. so that effective lateral distribution of loading occurs."

2.8 Explicity or implicitly the load-sharing systems for which provision is made in codes and standards are a) composed of parallel members, b) which are either contiguous or closely spaced (the exceptions being the grid provision in the Australian standard; and c) which are effectively connected laterally.

2.9 Too much importance should not perhaps be attached to the precise wording in the definitions. The Australian standard for instance refers to all the members being constrained "to the same deformation". Whilst mutual constraint is no doubt a major effect in load-sharing systems, equal deformations, as in the ideal case of vertically laminated beams, would pre-suppose infinitely rigid decking or other load-distributing connecting elements. The significance of the minimum number of members mentioned in the British and Australian codes is that the load-sharing effect (e.g. magnitude of increase in permissible stresses) depends on the number of members in the system.

## 3. LOAD-SHARING PROVISIONS IN CODES AND STANDARDS

3.1 The effect of load-sharing on design is expressed in one of two different ways:

1) by a reduction factor on the concentrated load (in the Dutch code)
2) by an increase in permissible working stresses for the load-sharing members (in the other codes and standards).

### 3.2 Load reduction factors (Dutch code)

3.2.1 According to NEN 3850 (para. 2.2.1a) a floor joist in a boarded floor, subjected to a concentrated load (of 300 Kgf acting on a surface 50 cm x 50 cm) may be calculated, with the load reduced by a factor $\phi$ given in the following formula:

$$\phi = 0.27 + 0.8\frac{a}{a_o} - \frac{E//I}{(E//I)_o}$$

where a = spacing of joists (on centres)

E//I = stiffness modulus per width of boarding (floor boards or plywood, in $N.m^2/m$)

$a_o$ = 1 m

$(E//I)_o$ = 50 000 N.m(5000 Kgf.m)

3.2.2 A table in the commentary on the clause gives reduction factors for boarding of various thicknesses and qualities; with E//I values varying from 1500 to 10100 N.m., joist spacing from 41 to 85 mm. The $\phi$ values vary from 0.54 to 0.78. A further reduction by a factor of 0.85 for short duration loading is also allowed. The product of the two factors reduces the concentrated load to be taken into consideration by a value of 0.46 to 0.66 for the various cases listed in the table.

3.2.3 Most of the research on the reduction factors for concentrated loads appears to have been carried out by Vermeyden (7, 8, 9). In particular his first study of 1968 (7) contained a theoretical analysis which regarded the load transfers between each board and joist as unknowns and solved the problem by a series of linear equations in which it was assumed that at every intersection the deflection of joist and boarding must be the same.

3.2.4 The conclusions from Vermeyden's study (both the theoretical and verification work on three and seven joist systems) with regard to the $\phi$ factor (and hence to the effective load distribution in a boarded parallel joists system) were as follows:

1. The influence of spacing is great and independent of other factors.
2. The influence of span on $\phi$ when $L > 3.30$ m is small and may be neglected. With smaller spans $\phi$ increases.
3. The influence of beam stiffness on $\phi$ is not great. With equal E another beam section gives a difference of ca. 0.03; besides, differences in E between beams may produce $\phi$ differences of $\pm$ 0.05.
4. The influence of stiffness of boarding applied to the same beam sections is independent of other factors and gives a difference of $\phi$ of 0.06 between floor boards of EI = 77 000 Kgfcm and EI = 34 000 Kgfcm.
5. The difference between $\phi_7$ (a seven joist system) and $\phi_3$ (a three joist system) remains fairly constant at 0.03, with $\phi_3$ being the greater as could be expected.

3.2.5 In the main the stiffness of decking and the spacing of joists was found to be predominant and this determined the tables proposed and accepted for the code. The control arrangements with 3 and 7 joist systems with a variety of floor boards, joist spacings and spans showed good agreement with the calculated values especially for the longer spans. With smaller spans and large spacings $\phi$ (control values) were greater than $\phi$. In a study on the draft code proposals in February 1970, Vermeyden drew attention to this and suggested a modified formula. He also considered the addition of 0.10 to $\phi$ to produce $\phi^1$ for flat roofs excessive in the case of $l < 3$m and recommended an additional factor of 0.07 up to $l = 3$m.

## 3.3 Stress increase factors

3.3.1 All the other codes and standards cited, with the exception of the Dutch, deal with load-sharing members by allowing an increase in permissible working stresses. This applies to all loads which are presumed to be jointly resisted by the system.

3.3.2 At present the British Code (CP 112) allows a 10 per cent increase on grade stresses for all grades, but in the case of the mixed 40/50 grade (containing at least 75 per cent of 50 grade material or better) an increase of 20 per cent is allowed. A mean MOE is also allowed instead of min. MOE.

3.3.3 In their commentary on CP 112: 1967, Booth and Reece (10) show that the modification factors for load-sharing systems are derived statistically in a manner similar to that adopted for the modification factors of vertically laminated beams, although the actual mechanics of load distribution are different. The modification factor for a vertically laminated beam was shown to be derived from the formula

$$\text{Mod. factor} = \frac{1 - 2.33\frac{v}{\sqrt{N}}}{1 - 2.33\,v}$$

where N is the number of members combined and v the coefficient of variation of the population. For a beam composed of four laminations this gave a value of 1.19. Because of the uncertainty of the effect of spacing of members and the relative stiffness of joists and boards (or other linking load distribution elements) a value of 10 per cent was recommended by the code, although for 50 grade material with an average strength ratio of 57 per cent, a 14 per cent increase would have been permissible with full load sharing.

3.3.4 In the special case of 40 grade material combined with 75 per cent of 50 grade or better, it can be shown that the average strength ratio is 54 per cent representing an increase of 35 per cent over that of the 40 grade. Hence the higher modification factor of 20 per cent.

3.3.5 The Australian code (AS CA65 - 1972) gives modification factors for parallel support systems which grow in value with the increase in the number of members as follows (from Table 2.4.5.1):

Parallel Support Factor

| No of elements carrying common load | $K_8$ (modification factor) |
|---|---|
| 2 | 1.14 |
| 3 | 1.20 |
| 4 | 1.24 |
| 5 | 1.26 |
| 6 | 1.28 |
| 7 | 1.30 |
| 8 | 1.31 |
| 9 | 1.32 |
| 10 or more | 1.33 |

3.3.6 For grid systems under uniformly distributed loads, the basic working stresses for bending, bearing and shear may be multiplied by a 'grid factor' $K_g$ given by:

$$K_g = \left[1.24 - 0.5 \left(\frac{s}{L}\right)\sqrt{N}\right] \quad \text{but not less than 1.00}$$

where s = c/c spacing of the supporting members

L = span of the supporting members

N = the number of laminations glued or mechanically fastened in each of the supporting members (= 1 for single piece solid members)

No increase is permitted in working stresses in the case of concentrated loads.

3.3.7 Leicester and Reardon (1) reported on tests carried out to examine the validity of the load-sharing provisions in the code in respect of Australian timber species. Four sets of scantlings, two of softwoods (Pinus elliottii and Pinus radiata) and two of hardwoods (Eucalyptus obliqua and mixed Victorian hardwoods) were tested in adequate samples and for point loading and simulated distributed loading. With the aid of a computer, the load deflection characteristics were obtained for about 2000 vertically 5-member laminated beams made of randomly selected members from each set. The load sharing factor was computed as the ratio of the five percentile value of the MOR of the population of laminated beams to that of the population of scantlings from which the beams were assembled.



3.3.8 The results showed that the code was not conservative for the ideal load-sharing conditions assumed. The model (for the code) was based on a normal distribution of strength with a coefficient of variation of 14 per cent. The predicted coefficient of variation of a 5-member beam made from these elements was 6 per cent, a reduction from 14 per cent by a factor of $\sqrt{5}$. This reduction in variability leads to the upgrading of the lower tail of the strength distribution curve and hence to an increase in permissible working stresses. The coefficients of variation determined from these tests show that not only is the variability of graded Australian timber often far greater than that used in the model (33 and 41 respectively for radiata pine and slash pine with 3-point loading) but that the reduction in variability due to lamination is much less than predicted by the model (18 and 22 respectively for r.p. and s.p.). In the light of these findings the authors suggested that the model used for the code be regarded essentially as a method of interpolating rather than a true model for Australian timber.

3.3.9 On the other hand they reported that in regard to load-sharing in grids, CA65 erred on the conservative side in disallowing a load-sharing factor for point loads, analogous to the one allowed for distributed loads.

3.3.10 ASTM-D2018 provides guidance for calculating the average strength of groups of load-sharing members, based on the random-products method developed by Johnson(12). Products of strength of the clear wood (for the properties under study) by the estimated strength ratio give the estimated strength of every piece in a survey sample of a population. If each multiple is expressed as a frequency distribution rather than a single value, the product is also in the form of a frequency distribution. This may be defined mathematically or may be generated by the random products method which involves multiplying a randomly selected value from the list of estimated strength ratios by one from the list of clear wood strengths. The random values are chosen so that their frequency of use corresponds to the frequency of appearance in the distributions of strength ratios or clear strength values. According to Johnson (and D2018) it has been found satisfactory to develop random products by sets of 100. Modern high speed computers can provide ten or more such sets, which is considered a minimum.

3.3.11 Frequency distributions of random products can be obtained for the expected strength in bending, compression parallel to grain and tension parallel to grain. D2018 recommends an exclusion limit of 5 per cent, and the frequency distribution of random products shows directly the most probable strength of random individual pieces at the exclusion limit.

3.3.12 Having determined the value of strength of an individual piece at the 5 per cent exclusion limit (and its standard error if desired) a value at the corresponding limit for the average strength of groups of n pieces can be established from the following relationship:

$$S_g = \bar{X} - \left(\frac{\bar{X} - x\ 0.05}{\sqrt{n}}\right)$$

where $S_g$ = average strength of the group of n pieces at the 5 per cent exclusion limit

$X$ = simple mean of random products

$x_{0.05}$ = expected strength of an individual piece at the 5 per cent exclusion limit, from the distribution of random products

$n$ = number of pieces in the group

The standard error s.e. of $S_g$ is computed from the following relationship:

$$s.e._g = \frac{s.e.0.05}{\sqrt{n}}$$

where s.e.0.05 = standard error associated with the strength of the individual piece at the 5 per cent exclusion limit from the distribution of random products

n = number of pieces in the group

3.3.13 The reduction factors considered in arriving at basic stresses and thence at working stresses provided in ASTM D245 (13) are accepted as providing adequate structural safety. The factor for variability of 3/4 in bending and compression parallel to grain can be neglected since variability is already fully measured and defined in this method.

3.3.14 As an alternative to strength surveys and calculations by the random products method, D2018 recommends the use of working stresses developed by Method D 245 for stress grades for framing lumber, and increasing them by not more than 15 per cent in the case of multiple members closely spaced and where load-sharing is known to exist. This, D2018 states, "is intended to be a conservative interpretation of present information, to be applied when load-sharing is known to exist but when appropriate research has not been conducted to justify larger increases. With a strength survey, increases in design stresses of 20 to 40 per cent or greater may be found for load-sharing members."

3.3.15 D 2018, which was only a Tentative Recommended Practice, has been withdrawn by the ASTM in 1970 on the grounds that there was no need for an additional general method of determining working stresses over and above that of D 245. No new ASTM recommendation in respect of load-sharing appears to have been issued since.

3.3.16 The Swedish SBN67 allows an increase in permissible stress values for load-sharing members from the single member values (but not higher than those for T30). Noren (14) reports that this rule originated from the design of sheet piling. At a coefficient of variation of 20 per cent it allows an increase in working stresses by multiplying by a 5 per cent fractile ratio:

$$\frac{1 - 1.65 \times 0.2/\sqrt{n}}{1 - 1.65 \times 0.2} = \text{modification factor}$$

where n = number of members
for n = 4 a value of 1.19 is obtained
and for n = 9 a value of 1.32.
This is practically the same formula as that for the vertically laminated beams from which values for CP 112 were derived. The factor of 1.19 is the same as that obtained in para. 3.3.3 above. The Australian model produced precisely the same factor for 9 members (see para. 3.3.8 above) but somewhat higher values at the lower end of the n range.

3.3.17 Using the same relationship for calculating deflections (at the servicibility limit state) with the Nordic countries using an MOE at 30 per cent fractile produces the expression:

$$\frac{1 - 0.5 \times 0.2/\sqrt{n}}{1 - 0.5 \times 0.2}$$

This would produce extremely small improvements; for n = 2, 3%; for n = 4, 5%; for n = 5, 6%; for n = 9, 7%.

3.3.18 The Draft Revision of CP 112 proposes an increase in the modification factors for stresses in load-sharing systems, as follows:

| No of members | Modification factor |
|---|---|
| 2 | 1.1 |
| 3 | 1.15 |
| 4 or more | 1.2 |

3.3.19 These values are still considerably below those in the Australian code and rather more in accord with the experimental findings of Leicester and Reardon mentioned earlier. For a system of more than four members, the values would also be below those obtained by the SBN 67 method.

3.3.20 For calculating deflection, instead of the Mean E as at present, for all load-sharing systems, it is proposed to raise the min. E progressively as the number of members increases, by a modification factor $K_1$ which will range from 1.17 for 2 members to 1.35 for 7 members or more.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# LOAD-SHARING

by

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PARIS - FEBRUARY 1975

BENGT NORÉN

SVENSKA TRÄFORSKNINGSINSTITUTET

BN/lg 1974-06-19

CIB W18
Delft, June 1974

# LOAD-SHARING

The problem of load-sharing has two principal componants:

A - Determination of the stress upon the various parts of the structure and the material = effect of load (deformation forces and stresses)

B - Determination of the resistance of the members of a structure.

Evidently this splitting requires a definition of how many of the structural members are integrated in a "structural member'. The calculation B will of course be dependent on the strength properties of the material (mean value and deviations), but also the calculation A will be dependent on the properties of the material in the case of statistically undetermined structures. From practical reasons one can then set the limit between A and B, i.e. chose the level of verification of the proper design, thus that random deviations in material properties or measures are not included in the calculation, type A. For example, in a floor beam system the effect of a point load on the single beam is calculated with consideration to deflection (serviceability limit state) as well as to bending moment (limit state of failure). This is done regarding the stiffness of the panel, the joints, and the beams themselves. Also in the ultimate limit state one considers the deflection (for example by using the plasticity theory). But one does not include the deviation of the material properties in this part of the load sharing calculation. For example, one neglects that the elasticity modulus varies from one beam to another. A lot of theoretical and experimental work has been done on the above mentioned component A within the load-sharing problematics. This is most important, because it gives us a possibility to calculate forces and stresses more accurately. However, it is generally inadequate to use the results to manipulate admissible stresses. Thus one should not increase the admissible stress for structural timber in small-house building to compensate approximations in the statical calculations.

The interest for load-sharing in calculations, type B, i.e. calculations of the resistance of cooperating members,of course, principally is due to the definition of the material strength, viz. whether it is given by a mean value or a relatively low fractile value. If the ultimate limit state can not be exceeded without the strength being utilized completely (or in any case up to the characteristic fractile (minimum) value in cooperating members, it should be allowed to take advantage of the fact that the strength value at a given fractile is higher for the member composed by n parts than for the single member. This is applied in some codes, for example the Swedish SBN 67, where the paragraph 27:1325A runs:

> "When several timber members cooperate and it can be presumed that the strength is increased thereby, i.e. at sheet piling, an increase of admissible values for single members is accepted, however not to higher values than those given for T 30."

This rule originates from the design of sheet piling. At a coefficient of variation 0.2 one should be allowed to increase the working stresses by multiplying by a 5 per cent fractile ratio:

$$\frac{1 - 1.65 \cdot 0.2/\sqrt{n}}{1 - 1.65 \cdot 0.2} = 1.25 \quad \text{for } n = 4.$$

In this field (B) much remains to be done. The example of beams for dwellings, however, is less interesting for a calculation B than for a calculation A. For calculating deflection (at the serviceability limit state) the Nordic countries use a MOE at the 30 per cent fractile. Using the same expression, this MOE should be increased by

$$\frac{1 - 0.5 \cdot 0.2/\sqrt{n}}{1 - 0.5 \cdot 0.2}$$

This gives a low increase if we assume that five adjacent beams cooperate in carrying the point load. This is a generous assumption, because a change of MOE in the outer beams will not mean much for the deflection of the point load placed in the centre. The increase of MOE will then be only 6 per cent. One must also note that the variation of MOE between the beams for one and the same floor will be less than the general variation in structural timber on which the standard values are based. If one beam has a low density and strength, the risk is considerable that all the other beams have a low density too, because they all originate from the same part of the forest (or even from the same stem). The increase of stiffness by cooperation in this case will simply not occur.

With regard to failure in the floor beams due to a point load, the following could be said: Failure will occur in the loaded beam before ultimate failure load is reached in the adjacent beams. The risk of failure in the direct loaded beam is not influenced by the variations of strength in the other beams which are only indirect loaded Of course, the floor will be better also from the point of view of point loading if a number of beams is stronger than the weakest one. One could, at least theoretically, consider this by decreasing the partial coefficient for the load. This is based on the single beam as structural member and the fact that the probability of the point load appearing over the weakest beam is less than the probability of the load appearing on any beam. In this way the nominal strength (partial coefficient for resistance) is not modified.

As an alternative one can estimate the resistance of a group of beams regarded as one structural unit. This, in a way, is equivalent to an increase of the admissible stresses.

If the load is evenly distributed and not stiff in itself (such as either several mutually independent point loads or hydrostatic pressure) an excess strength of some of the beams can not decrease the risk that the weakest beam is failing. If the failure of a single beam means limited damage, this could be taken care of by modifying some coefficient on the load side, or eventually on the resistant side, but preferably by modifying the general coefficient for the consequence of failure. Even the effect of this is of course equal to an increase of the admissible stresses.

4-9-1

PAPER 14

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# LONG-TERM LOADING OF TRUSSED RAFTERS WITH DIFFERENT CONNECTION SYSTEMS

by

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DENMARK

PARIS - FEBRUARY 1975

Building Research Institute, Denmark

18.2.1975 MJ/MS
B9-CIB/W18

# Long-Term Loading of Trussed Rafters with Different Connection Systems

## Some Results of Loading from June 1974 to January 1975

T. Feldborg and M. Johansen

The long-term loading was started at the Building Research Institute in connection with short-term testing of the same truss types carried out by Mr. Arne Egerup at the Technical University.

The investigation comprises W-trusses made of 45 mm timber connected by Hydro-Nail and TCT metal plates, nailed metal plates and nailed plywood gussets, and trusses made of two layers of 25 mm boards connected by nailed metal plates between the boards.

The supports are arranged in 3 ways, 1: below the heel joint, 2: 0,6 m from the heel joint in both sides, and 3: 1,2 m from the heel joint in one side below an extra web member.

Each type is represented by 2 trusses.

The load arrangement is shown in the figure. The trusses are designed for light roof cladding. Some types are overloaded by dead load representing heavy roof cladding.

So far the load has been varied between 8 week dead load and 1 week dead load + full design snow load. (Full design snow load is very rare in Denmark, and according to 30 years' measuring the duration does not exceed 5 days).

After 1 year of alternating loading it is planed to change to full design load for 4-5 years.

Deflection of the trusses is being measured at the joints and at the middle of the members of the upper and lower chords.

Temperature and relative humidity are recorded and the moisture content of wood samples placed with the trusses are measured.

The slip of the lower chord splice is measured.

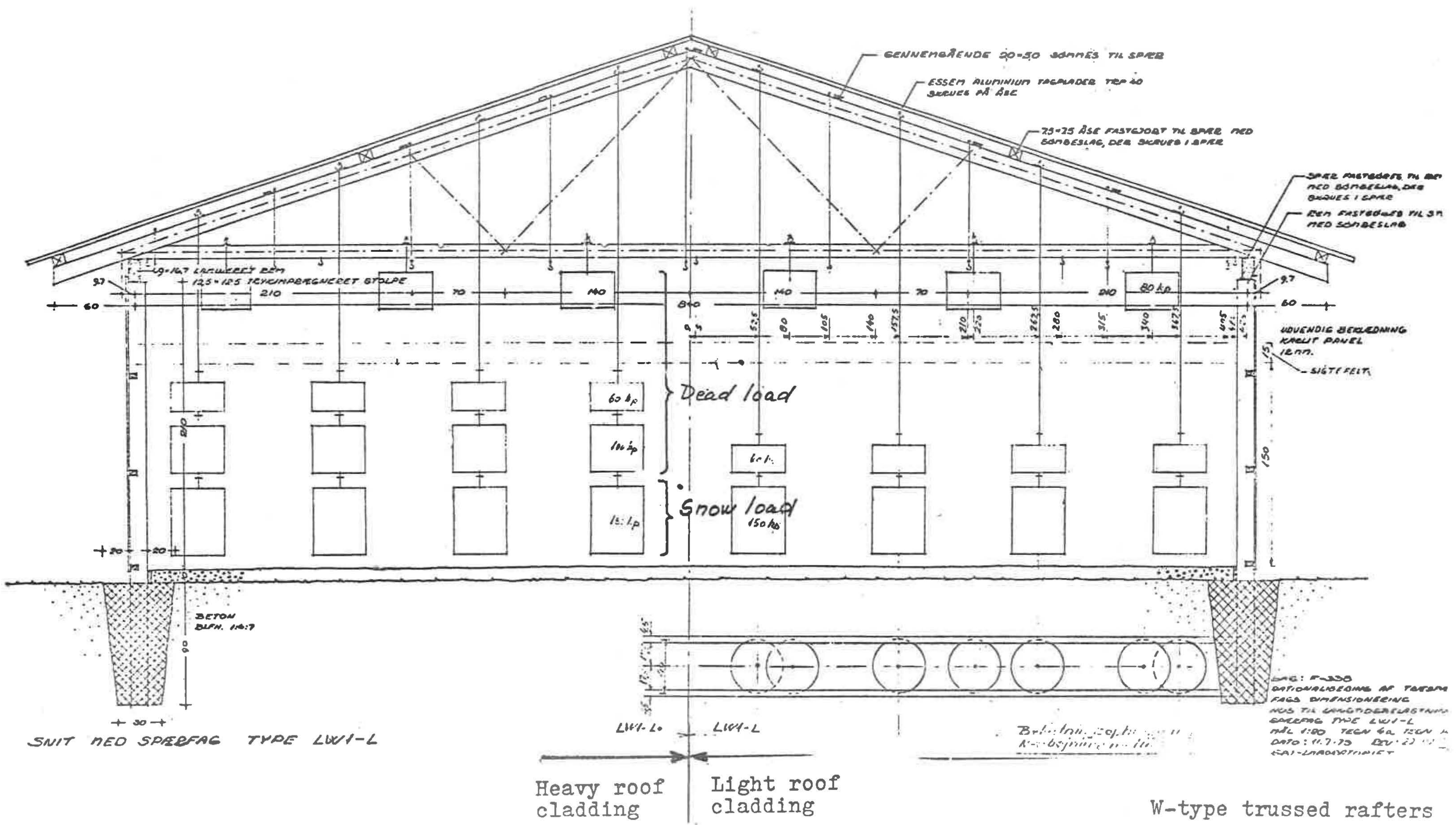
Some results are shown in the table

| | Calculated caracteristic tensile strength 1) kN | Measured mean tensile strength 1) kN | Slip for a tension load of 8 kN short term load 5 specimens mm | long term load 2) 2 specimens mm |
|---|---|---|---|---|
| Hydro Nail | 24,8 | 25,4 | 0,16 | 0,17 |
| TCT | 28,4 | 26,4 | 0,14 | 0,40 |
| nailed matal plate | 22,8 | 26,4 | 0,35 | 0,75 |
| - plywood | 24,3 | 36,5 | 0,39 | 1,70 |
| - metal plate between boards | 25,4 | 32,4 | 0,15 | 0,56 |

1) Short term load

2) Measuring no. 7 at the end of first snow load period

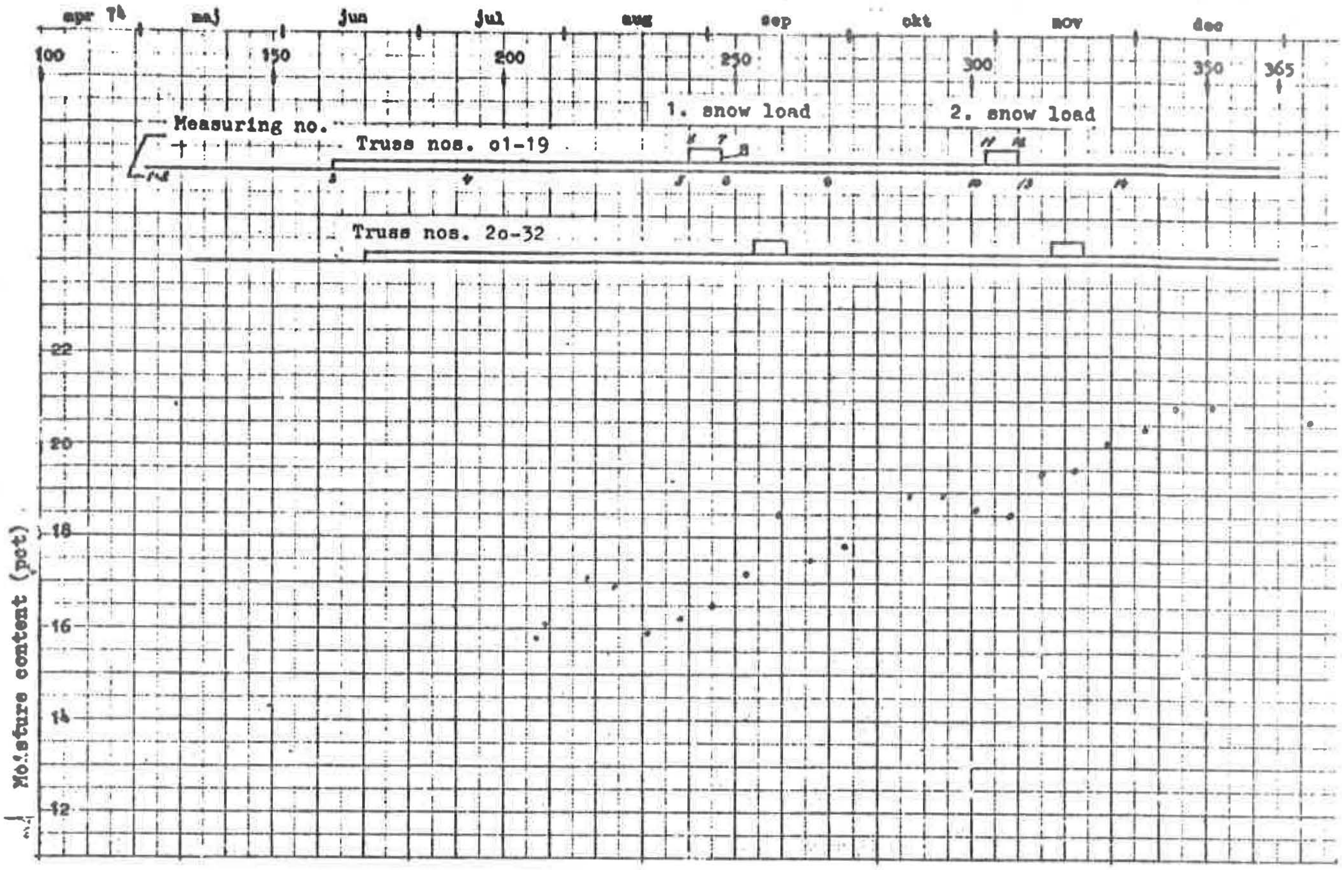
Some deflection curves are given on the following pages.




W-type trussed rafters

Span 8.4m Slope 1:3

Arrangement of long-term loading


apr 74
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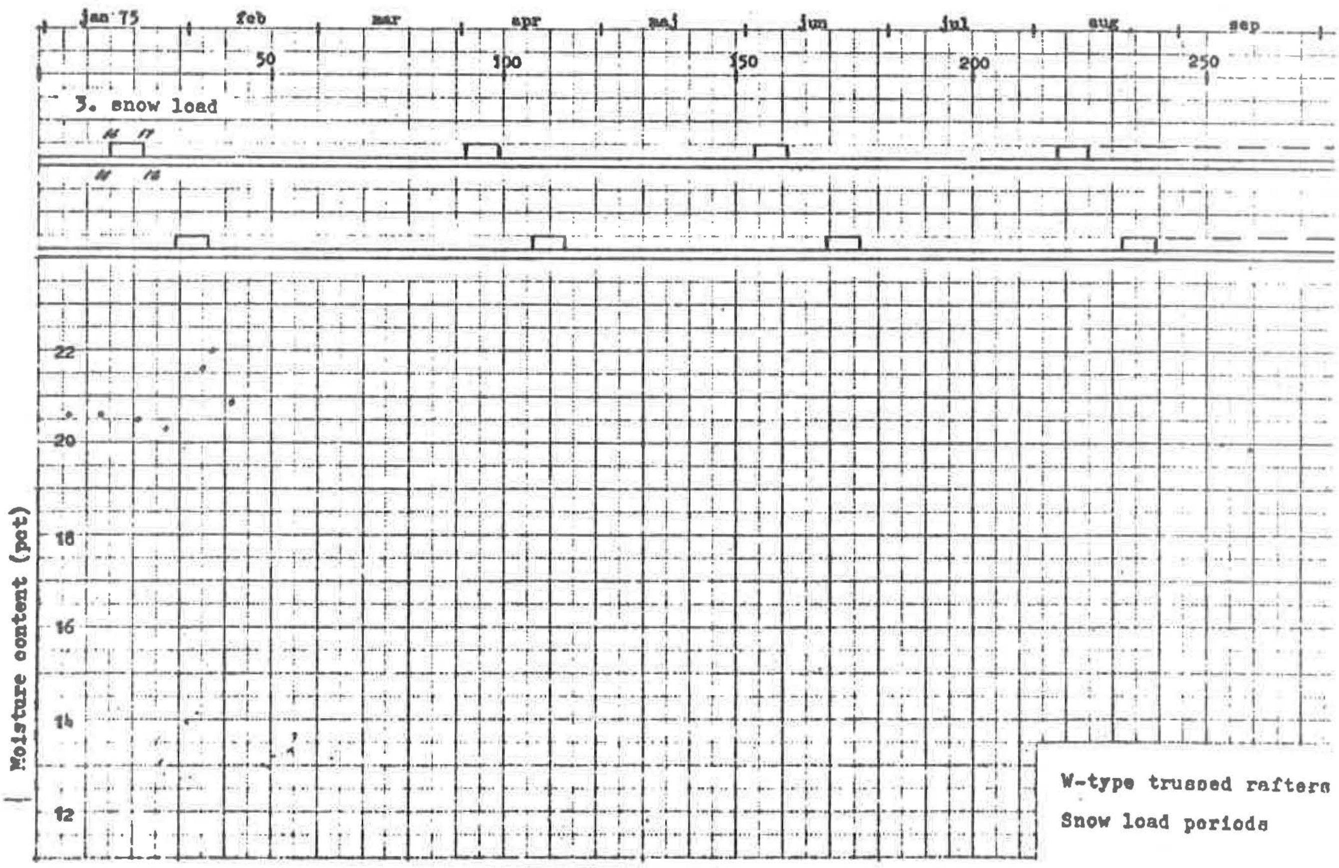
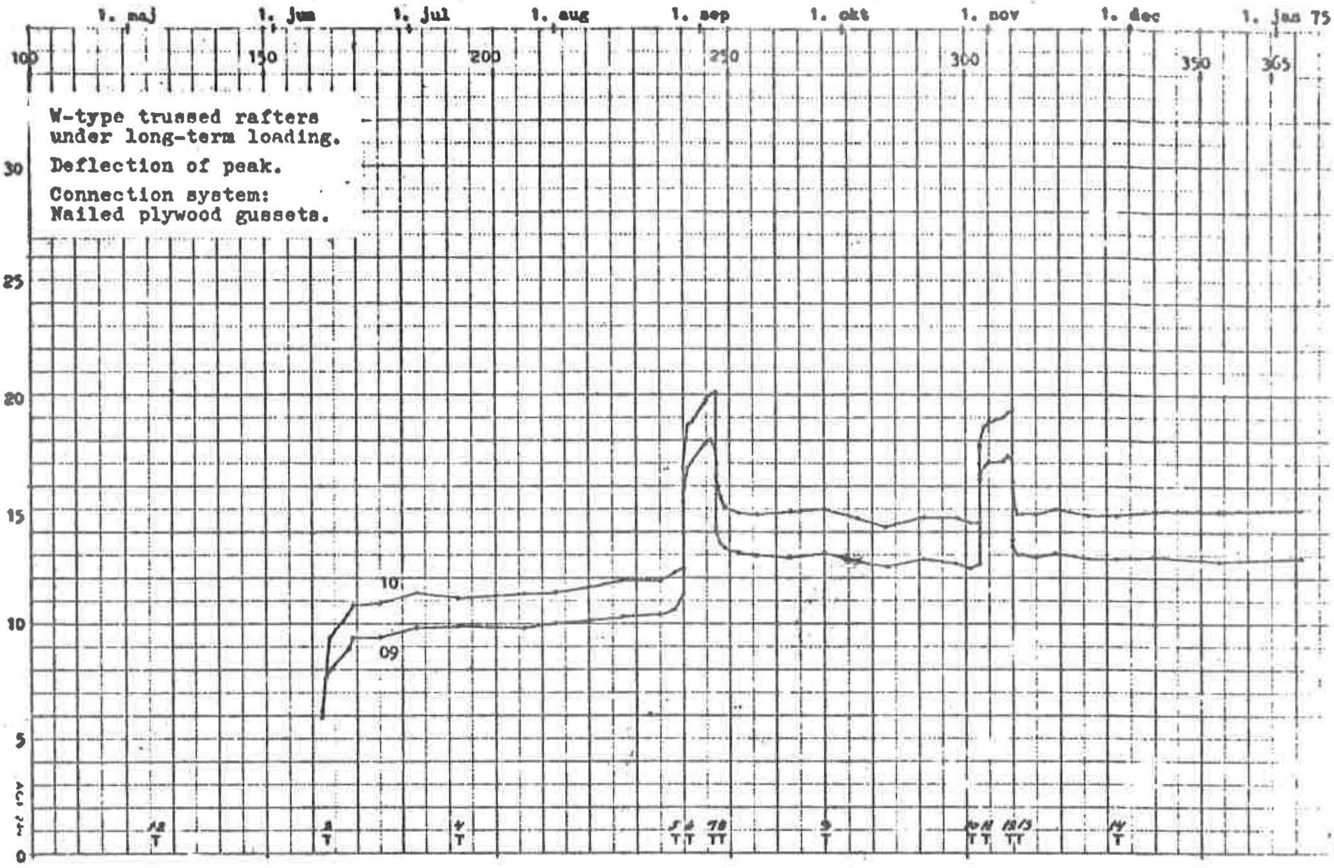
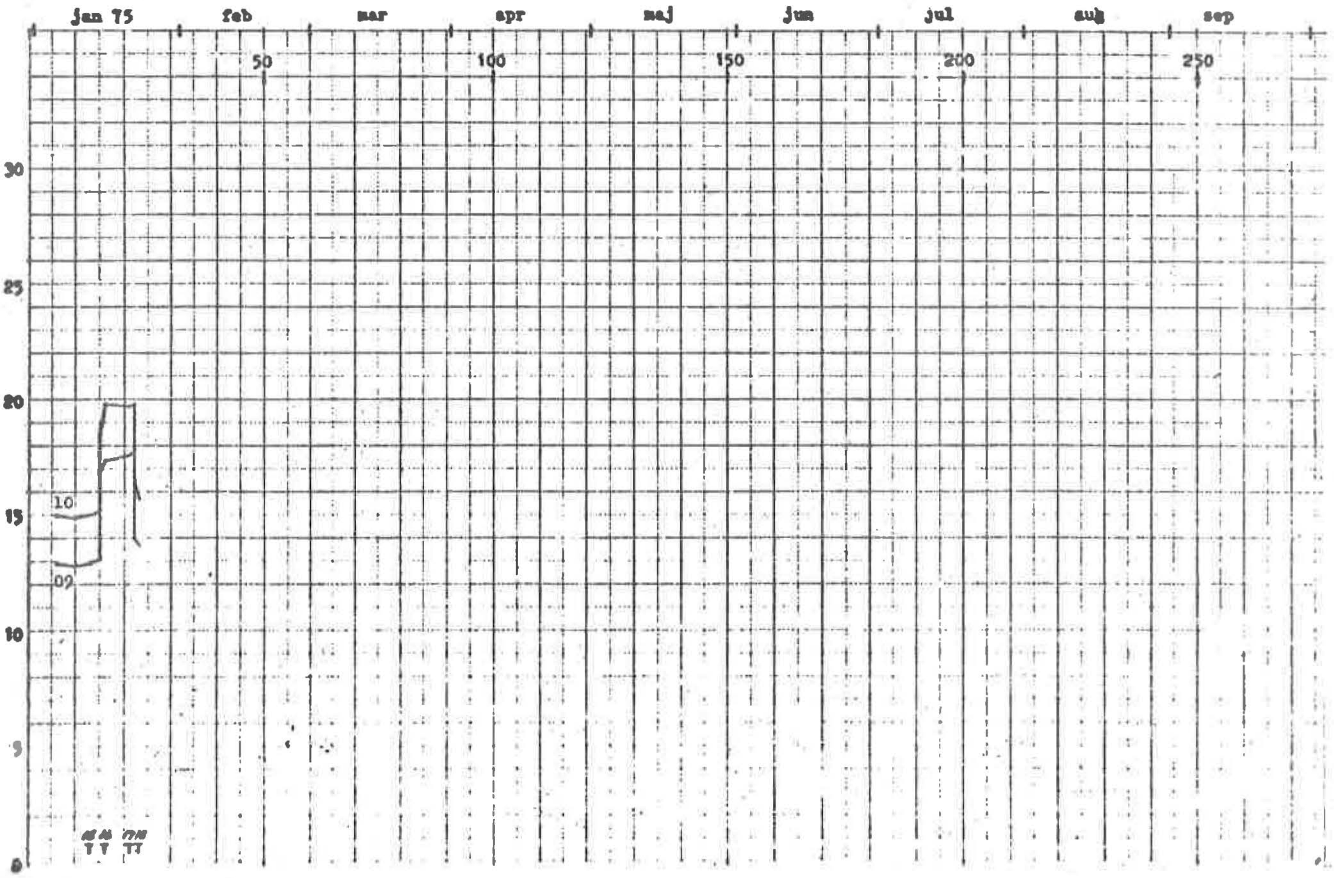
300
350
365
1. snow load
2. snow load
Measuring no.
Truss nos. o1-19
Truss nos. 2o-32
22
20
18
16
14
12
Moisture content (pct)


jan 75
feb
mar
apr
maj
jun
jul
aug
sep
50
100
150
200
250
3. snow load
22
20
18
16
14
12
Moisture content (pct)
W-type trussed rafters
Snow load periods

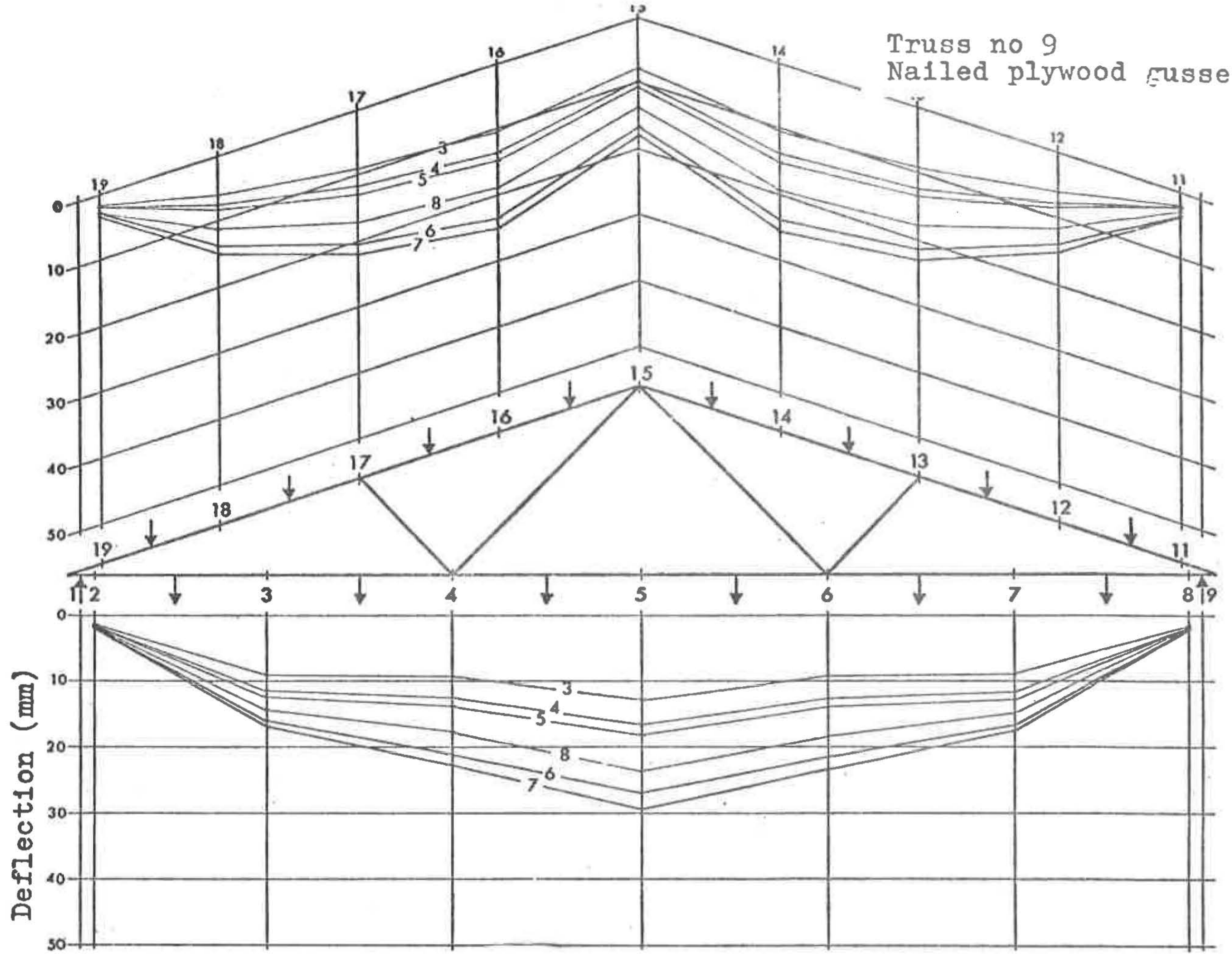


1. maj
1. jun
1. jul
1. aug
1. sep
1. okt
1. nov
1. dec
1. jan 75
100
150
200
250
300
350
365
W-type trussed rafters under long-term loading.
Deflection of peak.
Connection system: Nailed plywood gussets.
30
25
20
15
10
5
0
10
09

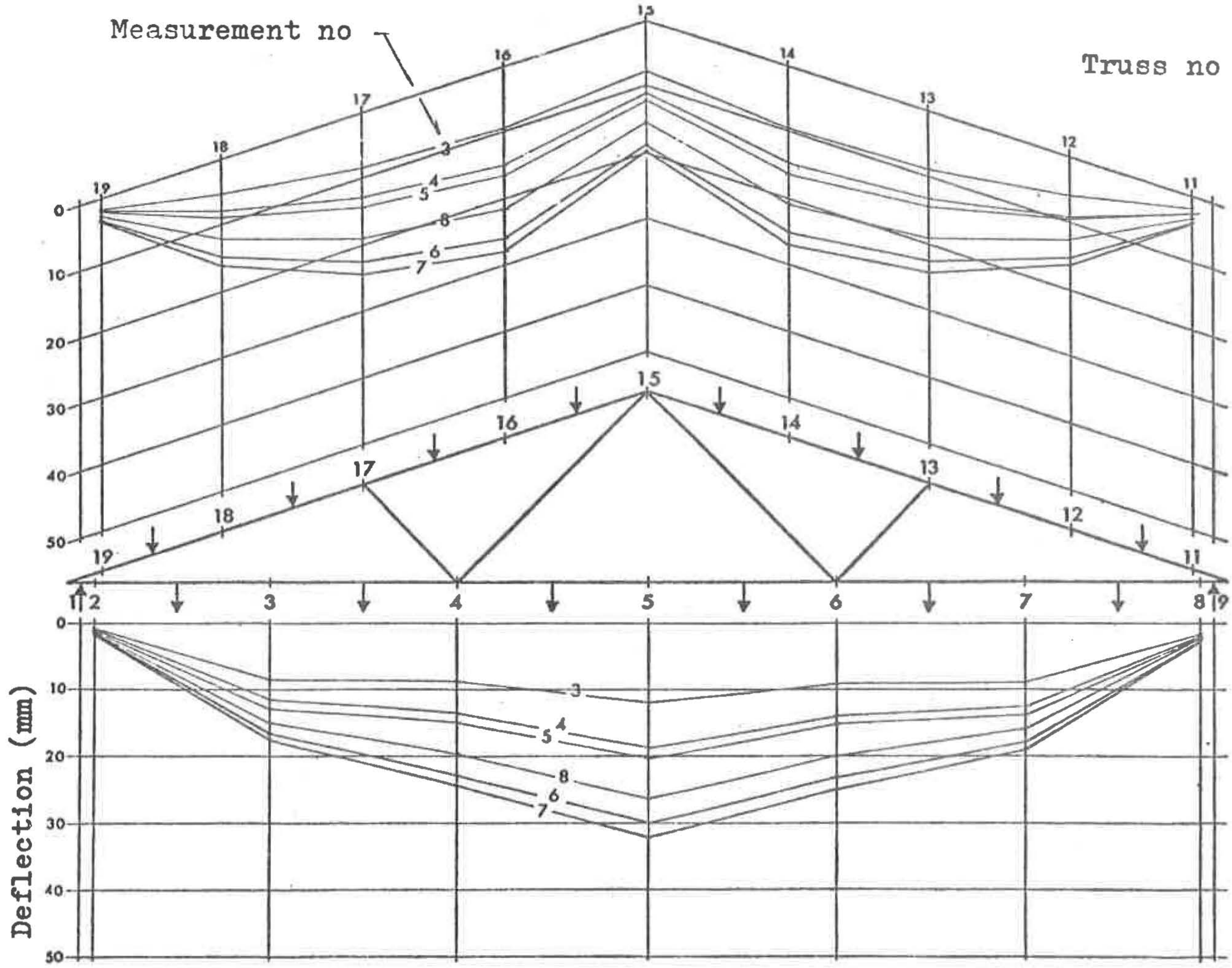



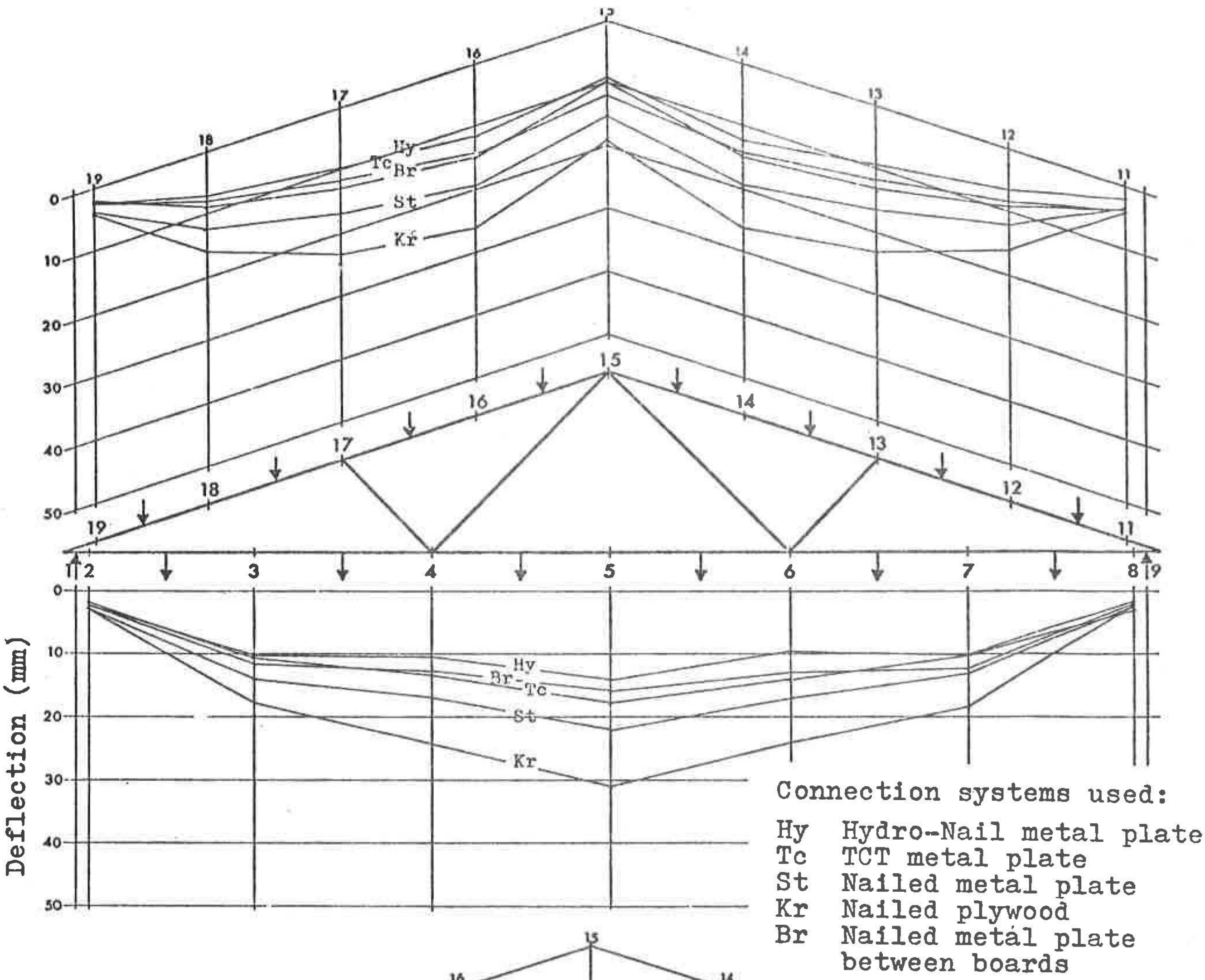
jan 75
feb
mar
apr
maj
jun
jul
aug
sep
50
100
150
200
250
30
25
20
15
10
5
0
10
09

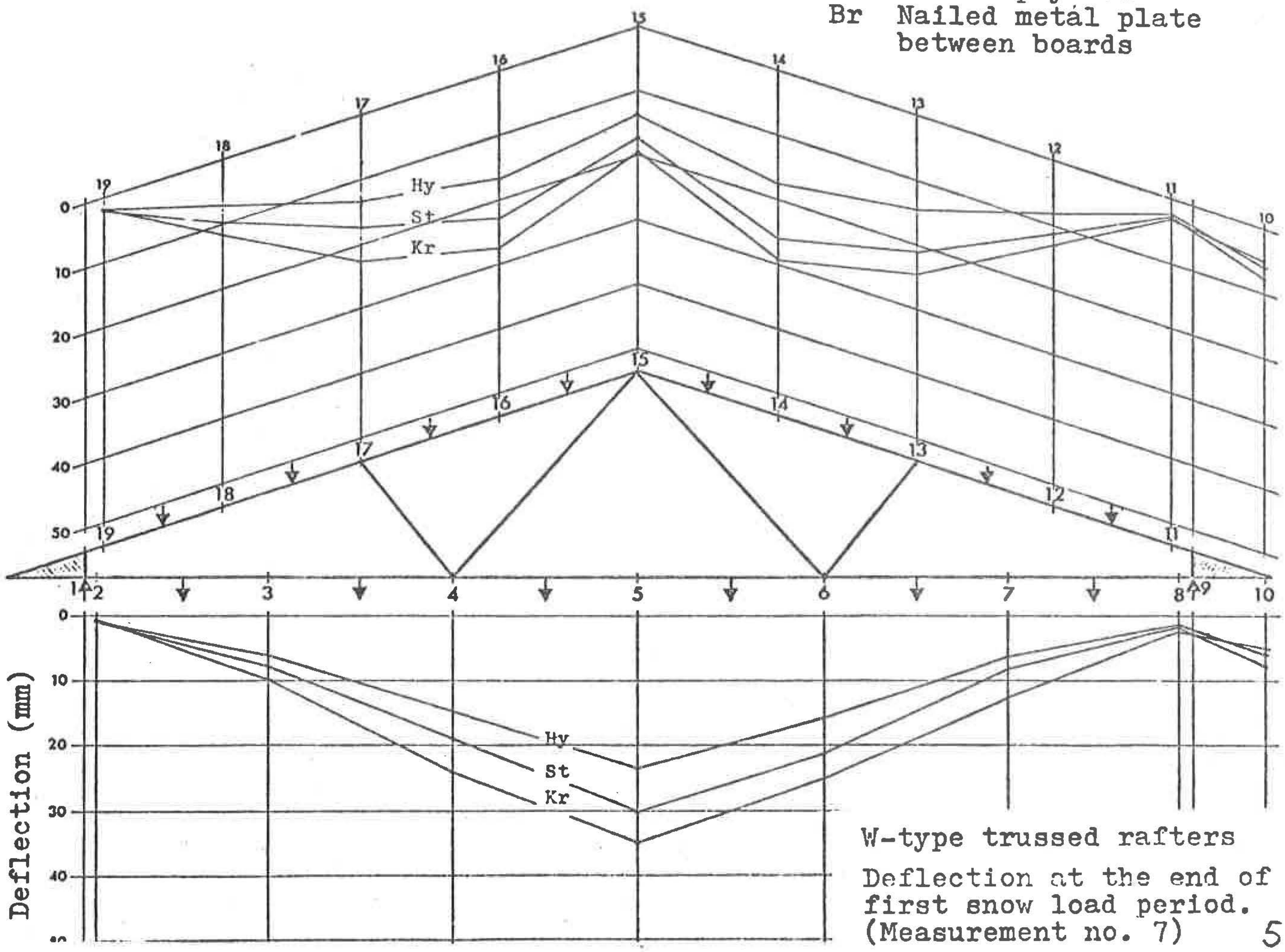
Truss no 9
Nailed plywood gussets




Truss no 1o






W-type trussed rafters

Deflection at the end of first snow load period. (Measurement no. 7)

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

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CONTENTS

# COMPARISON OF CODES AND SAFETY REQUIREMENTS FOR TIMBER STRUCTURES IN E.E.C. COUNTRIES

## INTRODUCTION

This survey has been prepared by the Timber Research and Development Association at the request of the Building Research Establishment. It forms part of a task being undertaken by BRE for the Building Regulations Directorate of the Department of the Environment and follows a completed survey by the Cement and Concrete Association, comparing the EEC codes of practice for reinforced concrete.

The survey incorporates information gathered by Working Commission W18 of the Conseil International du Batiment (CIB). Since 1973, Commission W18 has adopted the following terms of reference:

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

In adopting these terms of reference it was acknowledged that W18 should maintain liaison with the International Standards Organisation (group TC98, which is responsible for establishing the imposed loads for structures) and the Comité Européen de Coordination des Normes (CEN).

Commission W18 has gathered information on the application of existing regulations and codes, on current developments in codes of practice for timber structures, on stress grading practices and on the design of timber columns in various countries including the EEC countries Germany, Holland, Denmark, France and the United Kingdon. The survey of column design was undertaken by H. J. Larsen in 1973-74, and a 1961 survey by K. Möhler is also available, including the EEC countries Germany, France and the United Kingdom.

Use has also been made of two surveys conducted recently by TRADA, one on timber grades and permissible stresses undertaken for the R & D Liaison Committee of the European Softwood Conference, and one on design methods for laminated timber, undertaken for the Sons-Commission GLULAM of the Federation Européenne des Syndicates de Fabricants de Menuiserie Industrielle de Batiment (FEMIB). Both studies included the EEC countries Belgium, France, Germany, Holland and the United Kingdom.

The following report gives information on timber design codes and their current development in Belgium, Denmark, France, Holland, Germany and the United Kingdom, together with notes on their administration under national building regulations. The three countries excluded are Ireland, Italy and Luxembourg. There is no national code in Ireland, where design generally follows the British code of practice, but according to the THE survey a set of national regulations is in course of preparation by the National Institute for Physical Planning and Construction Research. Italy has no code of practice for timber structures. In Luxembourg, the THE survey states that Belgian and German standards are widely accepted as good practice.

The first section of the report outlines features of building control methods in the countries for which reports have been made to the W18 commission, filling the gaps for Belgium, Italy, Luxembourg and Ireland from a THE survey and other sources. This section overlaps the THE study but is compiled from reports by timber specialists and will provide additional information as well as confirmation material, besides allowing the present document to present a complete picture ~~material~~ of the control of timber structures in the countries concerned. The section also includes information on current and expected developments in country codes, to avoid this interfering with the succinctness of the comparisons given in later sections.

## BUILDING CONTROL AND TIMBER CODE DEVELOPMENTS

### BELGIUM

The control of building in Belgium is exercised through 2600 local authorities, each of which is authorized to issue by-laws on planning and on safety and hygiene, which may affect design and equipment.

There are no national laws covering technical requirements for building construction. However, the Ministry of Public Works is authorized to formulate national building regulations; at present only draft technical regulations for housing construction have been produced.

General technical specifications for public and government building, government-subsidised housing, railways and associated buildings, and private construction have been published respectively by the Ministry of Public Works, the National Housing Association, the Belgian National Railways Company and the Scientific and Technical Centre for Building. The private sector of the building industry uses part or all of these specifications. Where codes are specified they are in the form ~~on~~ of NBN standards issued by Institut Belge de Normalisation (IBN), the Belgian Standards Institution.

Grading rules for structural timber are given in a publication of the National Bureau of Wood Documentation "Sapin Rouge du Nord", mentioned in NBN 199 under the number 414. Permissible stresses are given in Specifications Techniques Unifiées no. 31 (Charpenterie) and no. 32 (Menuiserie pour Toiture). STN31-32 also gives permissible loads for nailed and bolted joints.

**DENMARK**

The Danish National Building Law, which is the responsibility of the Minister for Building and controls all building works in Denmark, operates through the National Building Regulations which were last revised in 1972. These Regulations lay down minimum functional requirements for buildings and where they exist refer to functional codes such as the code dealing with design loads for buildings. The Regulations also refer to National Standards dealing with specifications for building materials and Design Codes which recommend the principles and methods to be followed in the design of buildings. These standards and codes of practice are published by Dansk Ingenirforening (The Danish Society of Engineers) and engineers may depart from them provided they can justify the deviations to the local authorities and the Ministry of Building, e.g. by prototype testing of a structure.

The current Danish code for timber structures, Danish Standard 413:196~~7~~8, is ~~soon to be replaced by a new code~~ based on the principles of limit state design. The limit state approach ~~had~~ has been approved for all new design codes and already exists for masonry, steel and concrete. In the limit state codes the loads and load factors will be the same for all materials and only the material factor~~w~~s will vary, for example to take account of moisture content and load duration etc. The ~~new~~ code ~~will~~ does not include design methods and formulae contained in standard text books but where engineers are required to make assumptions, recommendations ~~will be~~ are given.

Plywood, fibreboard and particle board ~~will be~~ are included in the ~~new~~ code as structural materials but design details ~~will~~ are not ~~be~~ included and will have to be obtained by the manufacturers. However, design data ~~will be~~ are included for mechanical faste~~r~~ners such as nails, screws and bolts although nail plate connectors ~~will~~ are be left for the manufacturers to specify. The ~~new~~ code ~~will~~ ~~specify~~ specifies methods of test for deriving strength values for those items included in the code for which design data ~~will~~ are not ~~be~~ given.

FRANCE

There is no National Building Law in France but there are national regulations dealing with loadings on structures, fire and safety. The loading regulations vary regionally to take account of variations in snow and wind loadings etc. throughout the country. There also exist Standards and Codes of Practice published by a range of national organizations including engineers, contractors and insurance companies. The publications cover new materials and techniques and are gradually replacing Agrément Certificates. These Standards and Codes of Practice are not mandatory but every contractor is legally required to guarantee the buildings which he erects and this can most easily be accomplished by following the Standards and Codes of Practice. For building insurance purposes every design engineer must be able to prove the adequacy of his designs and therefore he too tends to follow the National Standards and Codes of Practice. In addition to the private insurance companies there also exists a state controlled National Insurance Company usually offering better terms than the private companies and this requires buildings to be designed and constructed in accordance with regulations published by the National Building Federation. The regulations contain most of the National Standards and Codes of Practice together with additional requirements laid down by the National Building Federation.

The design of timber structures in France is related mainly to two documents which apply nationally. The first, known as Standard NFB 52001, deals with the quality of the material and gives details of design stresses associated with a particular quality. The second document, Technical Unified Document 1972, specifies methods and rules for the calculation and design of timber structures. It is generally recognized in France that these two documents work on a safety factor of 2.75 and the design stresses laid down in NFB 52001 were derived by dividing the mean value of the relevant test results by 2.75.

The limit state method of design is already in use in France for concrete but there are no indications that the design of timber structures will change to this method in the foreseeable future.

GERMANY

A Building Law does exist within the Federal Republic but this has now been superseded by the nine regions each of which has its own building regulations incorporating the Federal Law. These regional regulations operate in three parts, the first of which is compulsory and specifies the functional requirements and loadings to be catered for in design. This part is drafted by the National Building Committee which is composed of scientists and engineers, Federal Government representatives and contractors. The second part deals with Standards for materials and the third part with design codes for buildings and structures. This last part is not mandatory and other methods of design may be used, including the prototype testing of structures, provided they can be justified. Provided the regional building regulations have been complied with the ultimate responsibility for building failures rests with the Federal Government via the regional authorities.

The design of timber structures in West Germany generally follows the recommendations laid down in the DIN Standard 1052. This code contains extensive design details which allow the engineer very little freedom of choice and is based on the permissible stress method of design. The topics covered by the Standard include: validity of the regulations, proof of safety and drawings, properties of materials, rules for dimensioning, members subject to bending, members subject to tension, members subject to compression, struts and braces, allowable stresses, deflections, joints and the design of bearings. The last revision of the code was in 1969 and German engineers are beginning to consider the limit state method of design; it is possible that limit codes will be developed simultaneously for timber, steel and concrete with a view to using the same load factors for all materials.

HOLLAND

In Holland domestic dwellings are controlled by Government Building Regulations which are mandatory and give design requirements and construction details. For the purposes of supervision of load bearing structures the country is divided into regions under the control of Building Inspectors but the interpretation of the regulations by these inspectors can vary considerably from region to region. There also exists the Dutch Normalisation Institute which is responsible for publishing material standards and design codes. In 1972 DNI published NEN 3850 which deals with the loads for which all buildings should be designed. Subsequent standards deal with methods of design for different materials which include timber, steel, masonry and concrete. The standard for timber structures, NEN 3852, was published in 1973. The loading requirements in NEN 3850 are mandatory and apply to all materials while the subsequent standards dealing with specific materials are advisory and carry no legal requirements.

NEN 3850 deals generally with building construction and gives details of loadings, permitted maximum deflections, a limited amount on the methods of design and calculations and finally a section dealing with the erection of buildings. However the engineer may depart from this code provided he declares the departures on his drawings and agrees them with the local authority. This first part of the revised code also refers to subsequent parts which deal with particular materials. The timber part includes a set of permissible stresses related to the Nordic softwoods and a few hardwoods. Also included are methods of calculation of the strength of timber structures, calculation of deflection and the acceptable limits for good building practice, design details for beams and columns including glu-lam, and details of jointing and fasteners. However it does not include details on plywood or other board materials but a limited number of data sheets have been issued on these by the Timber Research Centre.

Dutch timber engineers have limit state design under consideration and discussions have taken place with regard to a further revision of the timber code to include this method of design, which already exists to some degree for steel and concrete.

IRELAND

~~Republic of Ireland~~

The control of building in the Republic of Ireland is generally exercised through local authorities, except in the case of special buildings such as hospitals and schools, where government departments may be involved. Each local authority is aurhorised to produce its own building by-laws, although not all have done so. There are at present no Model by-laws, but a set of National Regulations is in the course of preparation by the National Institute for Physical Planning and Construction Research.

Present practice closely follows UK design procedures, specifications and requirements; there are no special requirements for climatic or geographical conditions.

## ITALY

The 'construction' aspect of building control is exercised by local authorities (for privately-financed projects) and by ministerial departments (for state-financed projects). In addition, it is necessary to obtain the approval of the Ministry of Labour for any industrial or commercial building projects.

The 'planning approval' aspect of building control is exercised by local authorities (for privately-financed projects and for state-financed projects).

Planning legislation applicable to state and privately financed building is issued by local authorities and by provincial governments; the provincial planning laws are operative in areas in which there are no local authorities.

There are special legal requirements for buildings constructed in areas that are considered to have high earthquake risk.

The following abbreviation is used in the list of requirements which satisfy the regulations.

CNR-UNI - standard issued by the National Research Council and adapted by the Italian National Council for Unification.

The technical requirements for buildings are as detailed in ministerial circulars.

Italy has no standards and codes relating to structural quality of timbers, permissible stresses and so on. A beginning of legal obligations can perhaps be found in the law of 2nd February 1974 No. 64, "Measures for building and housing with particular indications for the seismic zones". This law indicates that more detailed regulations will be given later for various materials, and it is expected that timber will be included.

## LUXEMBOURG

The control of building in Luxembourg is exercised by each town or local authority. These authorities produce their own regulations which deal in general with matters of planning, safety and hygiene only and do not cover technical requirements for buildings or their parts. Belgian and German standards and codes are widely accepted as good practice in Luxembourg.

UNITED KINGDOM

In England and Wales building control is exercised by the local authorities applying the Building Regulations (1972) which relate to the health and safety aspects of buildings. The Regulations operate by calling ~~the~~ for minimum functional requirements referring where necessary to functional codes (i.e. loading and wind codes) and then by "deemed to satisfy" clauses which make recommendations by reference to British Standards Institution Codes of Practice and Material Standards as to how the functional requirements may ~~by~~ be fulfilled. Whilst the functional requirements are mandatory the "deemed to satisfy" clauses are not and it is open to the Engineer to deviate from the methods of design recommended in the BSI Codes of Practice provided he can justify the new method, either by calculation or prototype testing, to the controlling local authority.

In cases of disagreement between the local authority and the engineer an appeals system exists whereby the parties can present their cases to the relevant government minister for a decision.

It is the local authority which is legally responsible for the safety of all buildings within the boundaries of the authority.

The current design code of practice for timber structures was first published in 1952 and revised in 1967. In 1971 it was converted into metric units but without any revision. This code is not mandatory and engineers can depart from its recommendations if they can justify the departure to the local authorities. The present code is based on the permissible stress method of design with fixed factors of safety. It includes sections on: materials, appliances and components also design considerations which include design stresses for a range of timbers, modification factors for types of loading and data on joint design ~~adn~~ and fastenings. It also contains recommendations dealing with workmanship and the inspection, testing and maintenance of timber structures.

## TIMBERS, GRADES AND STRESSES

### Loading

As mentioned in the survey for reinforced concrete, a preliminary report on loading regulations has been prepared by Stiller. It was necessary in one of the TRADA studies to incorporate a limited consideration of loading because of large differences between the permissible stresses adopted in some countries for similar grades. Thus in Austria domestic floors are designed for a distributed imposed load of 250 kg/m$^2$ whereas in the U.K. the design load is only 146.5 kg/m$^2$, and in Finland, Holland and Sweden 150 kg/m$^2$. This variation is compensated for by the bending stress of 115 kg/cm$^2$ allowed for the Austrian B4100 grade compared with 66.5 kg/cm$^2$ for the UK 50 grade which is of similar quality.

The following table shows the design loads for domestic floors, with notes on the modification factors applied in some countries. The figure 36 kg/m$^2$ for dead load has been inserted as a typical value to contrast with the value 60 kg/m$^2$ required in Holland.

| | Imposed | Dead | Modification factor |
|---|---|---|---|
| Belgium | 200 | 36 | none |
| Denmark | | | |
| France | 175 | 36 | none |
| Germany | 200 | 36 | Bending stress increased by 15% for load sharing. |
| Holland | 150 | 60 | Imposed load, bending stress reduced by $\frac{f_s}{1.15}$ for long term loading |
| United Kingdom | 146.5 | 36 | Bending stress increased by 10% for load sharing. |

### Timbers

The following table indicates the variety of different timbers for which provision is made in national Codes:

Grades and Stresses

Table 1 shows the structural grades of redwood and whitewood defined in the EEC countries, with the principal strength properties assigned to each. A study report is available which compares the structural performance of the different grades under the design loadings applied in their respective countries and makes a comparison between the extent of the defects permitted. The study concluded that it should not be difficult to achieve harmonisation by establishing a common European grade corresponding to the lowest structural grade adopted in most countries, together with a higher grade comparable with the Scandinavian T300, the French and German Grade II and the Dutch Construction grades.

Subsequent studies by the Timber Committee of the Economic Commission for Europe have confirmed the likelihood that two such grades can find wide acceptance. An ECE standard for stress grading of coniferous sawn timber has been prepared and issued as supplement 4 to Vol. XXVII of the 'Timber Bulletin for Europe'. The two basic stress grades proposed, EC1 and EC2, are similar to the BS grades SS and GS. The main modifications are that thedividing line between the two grades has been raised, resulting in a better division of the yield from a population into the two grades and lifting up the likely stress levels for these grades. As a general guide it is considered that stress in bending derived for EC1 and EC2 are likely to be of the order 8 - 10 $N/mm^2$ and 5.5 - 6.5 $N/mm^2$ respectively for European redwood and whitewood.

TABLE 1

STRUCTURAL GRADES AND PERMISSIBLE STRESSES FOR REDWOOD AND WHITEWOOD

| Country | COMPARATIVE PROPERTIES | | | | | | |
|---|---|---|---|---|---|---|---|
| | Grade | Bending | Tension | Compression | | Shear | Mean modulus of elasticity |
| | | | | // | ⊥ | // | |
| | | (all stresses given in $Kg/cm^2$) | | | | | |
| BELGIUM | STS 31 | 100 | 85 | 85 | 25 | 9 | 100 000 |
| DENMARK (limit state values for stresses) | T 200 | 110 | 85 | 100 | 28 | 14 | 70 000 |
| | T 300 | 145 | 125 | 125 | 28 | 14 | 90 000 |
| | T 400 | 180 | 165 | 150 | 28 | 14 | 105 000 |
| FRANCE (NF B52-001) | III | 75 | 80 | 70 | - | 10 | 100 000 |
| | II | 100 | 110 | 90 | 15 | 12 | 100 000 |
| | I | 110 | 120 | 100 | 15 | 12 | 100 000 |
| GERMANY | III | 70 | 0 | 60 | 20 | 9 | 100 000 |
| | II | 100 | 85 | ~~58~~ 85 | 20 | 9 | 100 000 |
| | I | 130 | 105 | 110 | 20 | 9 | 100 000 |
| HOLLAND | Standard | 70 | 50 | 65 | 20 | 10 | 100 000 |
| | Construction | 100 | 90 | 75 | 20 | 10 | 110 000 |
| UK | 40 grade | 52.7 | 52.7 | 38.7 | 15.5 | 6.3 | 84 400 |
| | 50 grade | 66.5 | 66.5 | 49.2 | 15.5 | 7.7 | 84 400 |
| | 75 grade | 101.5 | 101.5 | 81.0 | 19.7 | 11.6 | 84 400 |
| | GS grade | 52.0 | 35.7 | 57.1 | 15.5 | 8.8 | 87 690 |
| | SS grade | 74.4 | 52.0 | 81.6 | 17.4 | 8.8 | 101 970 |

## MODIFICATION FACTORS

### Duration of Loading

All Codes under consideration, except the German, allow grade stresses for timber to be exceeded by applied stresses for short periods. In the Codes for Belgium, Denmark and Holland, load terms are divided into three groups similar to the U.K. long, medium and short although actual divisions and names of groups differ. The French Code considers more groups and several combinations of these groups: a detailed summary is therefore given.

The following table gives factors by which grade stresses may be multiplied according to the Codes of Belgium, Denmark and the U.K. and the factors by which the portion of stress due to short or medium term loads may be multiplied according to the Code of Holland. In the former case, the stress due to the total load must not exceed the grade stress times the factor and in the latter case, the stress due to the short or medium term loads only multiplied by the factor and added to the stress due to long term loads must not exceed the grade stress.

| | Belgium | Denmark | Holland | U.K. |
|---|---|---|---|---|
| Med. Term. | 1.15 | $\frac{\sigma_L + 1.2\sigma_M + 1.4\sigma_S}{\sigma_L + \sigma_M + \sigma_S}$ | 0.85 | 1.25 |
| Short Term. | 1.50 | " | 0.70 | 1.50 |

Grade stresses in The Danish Code can be increased by a further 10% where other separate external loads act simultaneously and in addition to the self weight.

The more complicated approach given in the French Code for comparing applied and permissible stresses is as follows:

Stresses considered - G permanent load.
P overload plus dynamic forces.
Pc normal climatic overloads.
Pce extreme climatic overloads.
SI earthquake overloads.

The following equations must not exceed the permissible values, except in the case of temporary structures where permissible values may be multiplied by 1.1:

1) $G + 1.2P$.
2) $G + \gamma_p P + P_c$ $\quad \gamma_p = 0$ or 1 to give the most critical condition.

The following equations must not exceed the permissible values multiplied by a constant given below:

3) $1.1G + 1.5P + \gamma_{ce} P_{ce}$ $\quad \gamma_{ce} = 0$ when $P_{ce}$ - ve
4) $0.9G + 1.1\, P_{ce}$ $\quad = 1.1$ when $P_{ce}$ + ve
5) $G + P + SI$

Constants:

| Stress | Value |
|---|---|
| CPAR, CPERP, tPERP, Q | 1.5 |
| tPAR | 1.25 |
| f | 1.75 |

E values in the Danish and Belgian Codes relate to instantaneous loads and must be reduced by 20% and $33\frac{1}{3}$% respectively to obatin deflections for long term loads. No midification factor for E values is given in the other Codes.

For connections, in Denmark and Holland loads are multiplied by the same factors as those given for timber stresses. In the U.K. permissible loads on nails, screws and toothed plates are multiplied by a factor $(1 + K_{12})/2$ where $K_{12}$ is the modification factor for timber stresses. Permissible loads on bolts, split rings and shear plates are multiplied by $K_{12}$. Other Codes give no indication of modification factors for connections and it is assumed that none are applied.

Moisture Content

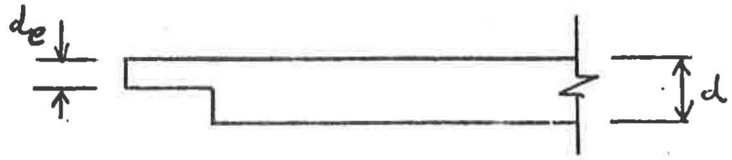
Most countries give grade stresses and moduli for dry timber with reduction factors to be applied when timber is used in wet conditions. A summary is given in the following table:

| Country | Moisture Content of Timber to which Grade Stresses Apply. | Comments and Details of Modification Factors. |
|---|---|---|
| Belgium | 15% for E. | Assume grade stresses are not modified. |
| Denmark | Up to 20% | If moisture content > 20%, multiply by 0.75 |
| France | 15% | Factors given for 7.5% to 30% |
| Germany | Dry conditions | E & G multiplied by 5/6 for damp conditions. Other stresses reduced to 5/6 when protected and $\frac{2}{3}$ when unprotected. |
| Holland | Up to 21% for softwoods. Over 21% for hardwoods. | Multiply by 0.9 and 0.8 for moisture content up to 30% and > 30% respectively. Multiply by 1.1 for moisture content < 21% for some timbers. |
| U.K. | < 18% and > 18% for timber. < 18% for plywood. | Between 0.75 and 0.90, depending on stress, for plywood. |

This table shows a general acceptance of a division between wet and dry conditions of use, the division being made at a moisture content of about 20%.

Shear in Flexural Members Notched at the Ends

The Codes of Denmark, Holland and the U.K. give a reduction factor for permissible stress of $\frac{de}{d}$ ( see diagram below). Applied stress is calculated for the reduced section area at the notch.




Form Factor for Flexual Members

Grading rules in the Danish Code take into account whether timber is round or squared. In Germany, fa and ca may be increased by 20% for unweakened marginal zones of round timber. In the U.K., fa may be increased by 18% for solid round timber and 41% for solid square timber with the load applied in the direction of the diagonal.

Deep Beams

In the Danish Code, a reduction factor must be applied to the grade bending stress. This factor, c, is given on a graph as a function of n, n being calculated from the formula $n = k_1 \sqrt{Ld} / b$ where $k_1$ is a factor depending on the type of load (u.d.l. or point) and the degree of fixity of the beam.

The French Code gives a modification factor to be applied to the section modulus for beams of depth other than 15 cm. This factor varies from 0.8 for beams of 30 cm depth to 2.0 for beams of 2 cm depth.

For beams with depth greater than 30 cm, the U.K. code gives the following factor for grade bending stress:

$$\frac{0.81\ (d^2 + 92,300)}{(d^2 + 56,800)}$$

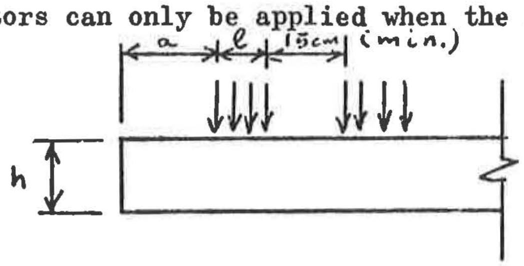
Length and Position of Bearing

All codes, except the Belgian, allow an increase in $C_{PERP}$ for short bearings unless they are at ends of members. Increases are as follows:

Denmark

| Bearing length cm. | $\leq$ 1.0 | 3.0 | 5.0 | $\geq$ 1.0 |
|---|---|---|---|---|
| Modification factor | 1.8 | 1.4 | 1.2 | 1.0 |

These factors can only be applied when the following conditions are obtained:




$a \geq 7 \cdot 5cm$ and $1 \cdot 5h$

France.

| $l/h$ \ $a/h$ | $\geq$ 1.5 | 1 | 0.5 | 0 |
|---|---|---|---|---|
| 1 | 2 | 1.5 | 1.25 | 1 |
| 2 | 1.5 | 1.25 | 1.12 | 1 |
| $\geq$ 3 | 1 | 1 | 1 | 1 |

Germany.

If bearing $<$ 10 cm from end, $C_{PERP}$ reduced by 20%. $C_{PERP}$ may be increased for some timbers if bearing is min. of 1.5 m from end.

Holland.

If bearing 10 cm from end, $C_{PERP}$ reduced by 25%. Where small compressions are acceptable, grade stress may be increased by 25%, except where allowance is made for load duration.

United Kingdom.

| Bearing length (cm) | 1.0 | 1.5 | 2.5 | 4.0 | 5.0 | 7.5 | 10.0 | $\geq$ 15.0 |
|---|---|---|---|---|---|---|---|---|
| Modification factor | 1.74 | 1.67 | 1.53 | 1.33 | 1.20 | 1.14 | 1.10 | 1.00 |

These factors can only be applied when the bearing is $>$ 7.5 cm from an end.

Continuous Beams

The German Code allows an increase of 10% in fa over intermediate supports.

Stress Reversals

In Germany, when members are subject to stress reversals due to loads other than wind and snow, maximum and minimum forces must be multiplied by $\left(1 + 0.3 \frac{FMIN}{FMAX}\right)$ to calculate the applied stress.

# COLUMNS

## Solid Columns

A 1973 survey by H. J. Larsen pointed out similarities in the theoretical basis of column design in Germany, Holland and the UK but suggested that the UK method made insufficient provision for the load eccentricity arising from the combination of initial curvature and defects. The Danish code has adopted a tangent-modulus approach and in France the method of Tetmajer is used with a form of straight line giving no reduction of the compressive strength for slenderness ratios below 37.5. No guidance on column design is given in the remaining four countries of the EEC.

A meeting of the CIB Commission W18 which considered Larsen's survey found generally acceptable his suggestion that the German method could be adopted in other countries, but a UK simplification was proposed for further consideration.

## Combined compression and bending

Larsen also showed that the formula adopted in most countries for combined stress is a good approximation provided the compression side is decisive as is almost always the case. The UK code actually differs in adopting a combined stress factor of 0.9 instead of 1.0, but this obstacle to harmonisation should not be difficult to overcome.

## Built-up columns

Larsen's survey extended to built-up columns, concluding that the preferred method was that adopted in Germany and Holland, and again this found general acceptance from the countries represented.

A UK survey of design methods is now in progress under a DOE contract, and its results should be awaited before considering the likely course of future changes.

Design formula for solid columns

**Belgium** None given.

**Denmark** Form of $\pi^2 E' / \left(\frac{1}{r}\right)^2$ with E' defined hyperbolically for $\frac{pe}{pg} > 0.5$ where pg is characteristic grade stress, and $E' = E$ for $\frac{pe}{pg} < 0.5$

**France** Tetmajer, $k = 3100 / \left(\frac{1}{r}\right)^2$ for $\frac{1}{r} > 75$, tangent below this value reaching $k = 1.0$ at ${}^1/_r = 37.5$

**Germany** Similar to Perry-Robertson, with $e = 0.10 + 0.008\ {}^1/_r$

**Holland** " " " " " $e = 0.16 + 0.008\ {}^1/_r$ for lower grade
or $0.10 + 0.005\ {}^1/_r$ for ~~lighter~~ higher grade

**UK** For low-grade softwoods, Perry-Robertson with $e = 0.003\ {}^1/_r$ allowing for combination of eccentricity and non-homogeneity. For higher grades and hardwoods, similar to Denmark but different mathematical form.

Transversely loaded columns

**Belgium** None given.

**Denmark** (1) for $\frac{ca}{cg} > 0.5$, $\frac{ca}{cg} + \frac{fa}{fp} <$ Ks limits combined stress factor to the same value as in a column with only "axial" load.

(2) for $\frac{ca}{cg} < 0.5$, $\frac{ca}{cg} + \frac{fa}{fp} \frac{KE}{K_E - \frac{Ca}{Cg}} < 1$ where $K_E = \frac{\text{Euler stress}}{C_g}$

Graph gives permissible combinations of $\frac{ca}{cg}$ and $\frac{fa}{fp}$

**France** None given.

**Germany** With symbol $\hat{a}'$ changed and transposed becomes same as CP 112 EXCEPT

(1) 1.0 on right instead of 0.9
(2) no increase of stress for loadsharing, different for load duration (see modification factors)
(3) greatest value for 1/r used, irrespective of direction of deflection.

**Holland** With symbol $\hat{a}$ changed and transposed becomes same as CP 112 EXCEPT

(1) 1.0 on rigt instead of 0.9
(2) no increase of stress for load duration but shorter-term loads are diminished (see modification factors)
(3) load-sharing factor may be applied, of a type different from CP 112 factors (see modification factors)

UK $\frac{ca}{cp} + \frac{fa}{fp} \not> 0.9$. Load sharing and duration factors applied as appropriate to $C_p$ and $f_p$.

Built-up columns with contiguous members (no packs)

(if glued, treated as solid in countries where this construction is mentioned, i.e. Denmark, Germany, Holland BUT NOT UK)

BELGIUM None given.

DENMARK Basically same formula as German, but discrepancies appear when converting one to the other and displacement moduli differ.

FRANCE None given.

GERMANY Effective I = sum of self Is + a proportion of $\Sigma Ah^2$. The proportion for a 3 piece column is $\frac{1}{1+K}$ where $K = \frac{\pi^2 EA_1 a}{l^2 c}$

a = spacing of fasteners

c = fastener stiffness, e.g. kg/cm

From effective I, reduced $r = \sqrt{\frac{I}{A}}$ gives increased l/r values for use when finding buckling factor applied to compressive stress.

HOLLAND Same as German.

UK Incorporated in UK method for "spaced columns". Effective l/r found using multiplying factor depending on type of fastening and spacing of shafts. Basically similar to German/Dutch/Danish but factors worked out (with differences shown by Larsen) and only two shafts considered.

Built-up columns with spaced members (packs, battens or lattice bracing)

BELGIUM None given.

DENMARK As for contiguous members but formulae given for $\frac{K}{aE}$ values (Different formulae for blocks, battens and lattice bracing)

FRANCE None given.

GERMANY Effective $\frac{l}{r}$ is $\lambda_w = \sqrt{\lambda^2_y + C\frac{m}{2}\lambda^2_1}$

where $\lambda$ = $\frac{l}{r}$ for whole cross-section about Y-Y (CP 112 diagram)

$\lambda_1$ = $\frac{l}{r}$ for single shaft about its own axis parallel to Y-Y

C = factor for type of fastening and type of bracing (packs or battens)

M = numbers of bars

A formula is given for C in the case of lattice bracing.

HOLLAND Same as German, including provision for lattice bracing.

UK See entry for contiguous members. Only two shafts with packs - no coverage of battens or lattice bracing.

NOTE: $\lambda$ y in German and Dutch codes seems to be full theoretical value with no reduction for connector slip.

Built-up columns with lateral load

BELGIUM None given.

DENMARK Not mentioned.

FRANCE None given.

GERMANY Envisages lateral loading (or eccentricity or curvature) for built-up columns with contiguous members, but spaced columns with packs, battens or latticing "should, as a rule, only be stressed centrally. Such bars must not be stressed at right angles to the non-material axis except by wind loads or other supplementary loads the effects of which can be determined".

HOLLAND Not mentioned.

UK Not mentioned, but a method suggested by the German (and Russian) methods has been proposed for use in connection with CP 112.

Connection of built-up columns

BELGIUM None given.

DENMARK Constant shear force $Q = \frac{N}{60K_S}$ where $K_s$ is reduction factor for solid columns. Slip disregarded.

| | |
|---|---|
| FRANCE | None given. |
| GERMANY | Constant lateral force similar to Danish but with buckling factor corresponding to effective I. For $l/r < 60$, force reduced by $\frac{1}{60}\ \frac{l}{r}$ but not more than 0.5. Shear strain and fastener spacing calculated as for built-up beams. Different formulae for non-contiguous members. |
| HOLLAND | Similar to German, to be increased by transverse force caused by bending moment. Formulae given for design of connections. |
| UK | No guidance given. |

## Fastenings

### General

Due to the use of different designations, variations of dimensions, and methods of determination of permissible loads for fastenings; and also the fact that some Codes do not have rules for all types; it is not possible to make complete direct comparisons between those for which information is available.

Those fastenings for which the appropriate Code or Standard has sets of rules are listed hereafter.

| | Types of fastenings or jointing methods |
|---|---|
| Belgium | Bolts; Nails. |
| Denmark | Nails; wood-screws; bolts; bulldog (toothed-plate) connectors; stjerne (star or starred) connectors; split-ring connectors; shear plates. |
| France | Joinery joints:- notched; scarfed; halved; mortice and tenon; dovetail.<br>Mechanical fasteners:- bolts; nails; spikes; wooden keys; split-ring connectors; toothed-plate connectors; alligator connectors; metal plates or gussets. |
| Germany | Dowels; bolts; nails; wood-screws; split-ring connectors; spiked-ring connectors; alligator connectors; toothed-plate connectors. |
| Holland | Nails; bolts; bulldog (toothed-plate) connectors; split-ring connectors; shear plates; lag-screws; wood-screws. |
| U.K. | Nails; wood-screws; bolts; toothed-plate connectors; split-ring connectors; shear plates. |

Except for Belgium, all the Standards or Codes have rules for glued joints, but the use of glue for jointing is not discussed further in this report. As the French code is the only one to have specific methods for dealing with joinery type joints, these also have not been considered.

## Nails.

All the Codes cover nails. Nail diameters, on which, in general, permissible loads are based, are described differently in each. For Belgium and Holland, diameters are given in Birmingham wire gauge (B.W.G.) numbers; France uses Paris gauge; and U.K. uses standard wire gauge (SWG). In every code there is a table giving nail diameters, in millimetres, with the permissible lateral load per nail for normal or long term loading. Belgium, France and Germany also give the formula from which the permissible lateral load per nail is calculated. Table 1 shows typical values for permissible lateral loading in softwood, only where nail diameters are comparable, together with minimum member thickness and nail penetration. Table 2 shows comparisons of spacings, edge distances and end distances. The values would be applicable for air-dry European redwood and whitewood, and other similar softwood timbers that may be grouped and classified with them.

TABLE 1

PERMISSIBLE LATERAL LOADS IN SINGLE SHEAR FOR ROUND WIRE NAILS IN AIR-DRY SOFTWOOD (WITHOUT PRE-DRILLING)

| | Nail diameter | | | Minimum member thickness (mm) | Minimum nail penetration into final member (mm) | Lateral load per nail (Newton) |
|---|---|---|---|---|---|---|
| | | Gauge No. | mm | | | |
| BELGIUM (see note 1) | BWG | 12 | 2.8 | 24 | 22 | 300 |
| | | 11 | 3.1 | 24 | 25 | 360 |
| | | 10 | 3.4 | 24 | 27 | 430 |
| | | 9 | 3.8 | 24 | 30 | 520 |
| | | 7 | 4.6 | 31 | 37 | 720 |
| DENMARK | - | - | 2.8 | 23 | 22 | 340 |
| | | - | 3.1 | 25 | 25 | 400 |
| | | - | 3.4 | 27 | 27 | 480 |
| | | - | 3.8 | 31 | 30 | 600 |
| | | - | 4.6 | 37 | 37 | 800 |
| FRANCE (see note 2) | Paris Gauge | 16 | 2.7 | 19 | 19 | 300 |
| | | 17 | 3.0 | 21 | 21 | 350 |
| | | 18 | 3.4 | 24 | 24 | 420 |
| | | 19 | 3.9 | 27 | 27 | 510 |
| | | 20 | 4.4 | 40 | 40 | 700 |
| GERMANY (see note 1) | - | - | 2.8 | 24 | 34 | 300 |
| | | - | 3.1 | 24 | 38 | 375 |
| | | - | 3.4 | 24 | 41 | 430 |
| | | - | 3.8 | 24 | 46 | 525 |
| | | - | 4.6 | 31 | 56 | 725 |
| HOLLAND | BWG | 12 | 2.8 | 20 | 34 | 250 |
| | | 11 | 3.1 | 22 | 37 | 300 |
| | | 10 | 3.4 | 24 | 41 | 400 |
| | | 9 | 3.8 | 27 | 46 | 450 |
| | | 7 | 4.6 | 32 | 55 | 650 |
| U.K. | SWG | 12 | 2.64 | 19 | 25 | 178 |
| | | 11 | 2.95 | 22 | 29 | 222 |
| | | 10 | 3.25 | 25 | 32 | 267 |
| | | 9 | 3.66 | 29 | 38 | 334 |
| | | 7 | 4.47 | 38 | 51 | 489 |

d = nail diameter (mm); t = member thickness (mm)

Note 1. Min. timber thickness (mm) = $d\frac{(30 + 8d)}{10}$; but thickness to be at least 24mm

Permissible nail load (Newtons) = $\frac{50\,(d)^2}{(10 + d)}$

Note 2. For member thickness not greater than 30 mm, nail diameter must not exceed $\frac{t}{7}$.

For member thickness greater than 30 mm, nail diameter must not exceed $\frac{t}{9}$

Permissible nail load (Newtons) = $80\,d\sqrt{\frac{t}{10}}$

TABLE 2

MINIMUM SPACINGS, EDGE AND END DISTANCES. (For Timber to Timber Joints)

| | End Distance | | Edge Distance | | Spacing between lines of nails perp. to grain | Spacing between nails along the grain |
|---|---|---|---|---|---|---|
| | Loaded end (member in tension) | Unloaded end (member in compression) | Loaded edge | Unloaded edge | | |
| BELGIUM | 12d | 12d | 12d | 5d | 5d | 10d |
| DENMARK | 15d | 10d | 10d | 5d | 5d | 10d |
| FRANCE | 12d | 5d | 12d | 5d | 5d | 10d |
| GERMANY | 15d (10d) | 7d (5d) | 7d (5d) | 5d (3d) | 5d | 10d (5d) |
| HOLLAND | 15d | 8d | *from 5d to 8d | 5d | 5d | 10d |
| U.K. | 20d (10d) | 20d (10d) | 5d | 5d | 10d (3d) | 20d (10d) |

d = diameter of nail

Generally the values given are for nails driven without pre-drilling. Values in brackets are those to be used when holes are pre-drilled.

* Where load on nail is at an angle ($\alpha^\circ$) to the grain less than $60^\circ$; value = $\left(5 + \frac{\alpha}{20}\right)$ d;

and where $\alpha$ is not less than $60^\circ$ :- value = 8 d.

## DESIGN BASIS

**France**

(1) Must not exceed permissible stresses under loading

dead + 1.2 X imposed (other than climatic)

or the most unfavourable of

dead + imposed + normal climatic

OR dead + normal climatic (wind and snow)

- with no increase of permissible stress for lower-duration loads.

(2) Must not exceed elastic limit (1.75 X permissible stress in the case of bending) under loading.

1.1 X dead + 1.5 X imposed + 1.1 X extreme climate

- with last item left out if it opposes the effect of the others

OR 0.9 X dead + 1.1 X extreme climate

OR dead + imposed + seismic

(1) is similar to U.K. except for implication that codified imposed load might be exceeded by 20% in practice(but not if normal climatic load acts as well)

(2) is equivalent to 1.75 duration factor, for extreme but rare combinations of brief occurrence, but this factor is countered severely by the 1.5 factor on imposed load.

Method generally seems based on average behaviour rather than 'weakest piece' philosophy, with factor of safety 2.75 on mean bending stress.

Only compatible combinations of imposed load are to be considered. Apart from this, the effect of the most unfavourable combinations of load (including zero imposed load if appropriate) must be assessed.

<u>Deflection</u> limit $\frac{\quad}{150}$ to $\frac{\quad}{400}$ depending on function of component.

Total deflection is sum of

(1) deflection due to lower-duration loads, calculated using conventional E value,

and (2) deflection due to permanent and"long duration" loads, using E divided by a creep coefficient defined by formula; ranges linearly from 1.0 for stress not exceeding 1/5 of permissible to 1.75 for stress equal to permissible, with zero change of moisture content. Range is 1.0 to 4.0 for 15% change; values also tabulated and graphed for 5% and 10% changes.

"Long duration loads" are those applied for three months or longer, and those applied on average for 50% of the time or more.

Load Sharing.

The principle of load sharing between members of a structure which are effectively connected ply sheathing, flooring and the like, is recognised in the British and Dutch Codes.

In CP.112 : Pt.2 : 1971 clause 3.12.4 it is stated that "where 4 or more members can be considered to act together to support a common load, the grade stress should be multiplied by 1.2. In the special case where 40 grade material is used in a load sharing system, the grade stress should be multiplied by 1.2, provided that at least 75% of the members are of 50 grade or better."

The mean value of the modulus of elasticity should be used to calculate deflections. This clause is applicable to members of parallel systems spaced not more than 610 mm apart.

The allowance of increased stresses is based principally on the statistically demonstrable fact that the coefficient of variation for a group of a given population is considerably smaller than that for an individual member of the same population. The special concession for the mixed 40/50 grade was based on the fact that due to the 3:1 ratio of the better grade, the average strength ratio of the material represented in fact an increase of 1.35 factor over that of the 40 grade.

For the revision of CP.112, it is proposed to modify the load sharing clause, allowing increasing load sharing factors for stress as the number of members in a system increases and the same treatment is suggested for the modulus of elasticity which will progressively approach the mean value.

The Dutch timber design code confines the load sharing considerations to the application of reduction factors on concentrated loads acting on a joist of a boarded floor. The reduction factor is given in the formula:

$$\phi = 0.27 + 0.8 \frac{a}{a_o} - \frac{E_{//} I}{(E_{//} I)_o}$$

Where $a$ = spacing of joists

$E_{//} I$ = stiffness modulus per width of boarding (floor boards or plywood) in N. $m^2/m$

$$a_o = 1 \text{ m}$$

$$(E_{/\!/} I)_o = 50\ 000 \text{ N.m } (5000 \text{ Kg f.m})$$

A table given in the commentary on the clause gives reduction factors for boarding of various thicknesses and qualities, with $E_{/\!/}I$ values varying from 1500 to 10 100 N.m, joist spacing from 41 to 85 mm. The $\phi$ value varies from 0.54 to 0.78. A further reduction by a factor of 0.85 for short duration loading is also allowed. The product of the two factors reduces the concentrated load to be taken into consideration by a value of 0.46 to 0.66 for the various cases listed in the table.

# LAMINATED TIMBER

## CURVED MEMBERS

### Radial Stresses

The Belgian, Danish, Dutch, German and UK Codes give the following formula for calculation of radial stress:

$$f = \frac{3M}{2Rbd.}$$

No formula is given in the French Code.

### Permissible Radial Stresses

The permissible radial stresses, in kg/cm$^2$, are given in the following table.

| Stress | Belgium | Denmark | Germany | Holland | U.K. |
|---|---|---|---|---|---|
| Tension | 3 | 4 | 2.5 | 2.5 | 4.6 |
| Compression | 20 | - | - | - | 21 |

No limiting values are given for compressive stress in the Danish, German or Dutch Codes.

The values given in the Danish and Dutch Codes are equal to the permissible tensile stresses perpendicular to the glue line. The value given in the UK Code for radial tensile stress is equal to $\frac{1}{3}$ permissible shear stress parallel to grain and for radial compressive stress is equal to permissible compressive stress perpendicular to grain. Figures quoted above apply to softwoods which are generally used for glue lamination.

The French Code makes no specific reference to radial stresses but figures are given for transverse tension and compression in timber.

Ratio of Laminate Thickness to Radius of Curvature.

| | Belgium | Denmark | France | Germany & Holland | UK |
|---|---|---|---|---|---|
| Softwood | $t/r \not> \frac{1}{125}$ | R(mm) — t<br>$\leqslant 1000$ — $\leqslant 0{\cdot}01R$<br>$\geqslant 1000$ — $\leqslant 0{\cdot}006R + 4\,mm.$ | $t/R \not> \frac{1}{160}$ | $t/r \not> 1/200$ | $\frac{t}{r} \not> \frac{1}{125}$ |
| Hardwood | No specific reference | No reference | $\frac{t}{r} \not> \frac{1}{200}$ | No reference | $\frac{t}{r} \not> \frac{1}{100}$ |
| Special Rules | - | - | - | If $\frac{1}{200} < \frac{t}{R} \not> \frac{1}{150}$<br>$t \leqslant 10 + 0.4 \left(\frac{R}{t} - 150\right)$ (mm) | - |

The Dutch and German Codes have the same rules for laminate thickness to radius of curvature. The maximum t/R ratio permitted is 1/150. It should be noted that operating the rule for laminate thickness for ratios of t/R between 1/200 and 1/150 allows a maximum permitted thickness of laminate for various radii; for example with t max. the formula reduces to $R = 2.5t(t + 50)$. The Belgian and UK Codes are very similar and it should be noted that the t/R ratio of 1/100 in the UK Code is only applicable to hardwoods, therefore the maximum t/R ratio for softwoods in both Codes is 1/125. The Danish Code differs from the others in that it allows a higher t/R ratio for radii less than 1000 mm. The French Code gives a maximum t/R ratio of 1/160 for softwoods.

Reduction Factors for Permissible Stresses.

No reduction factors are given in the French, German or Dutch Codes. Reduction factors in other Codes are as follows:

Belgium.

Factor = $1 - 2000 \left(\frac{t}{R}\right)^2$ to be applied to stresses (implies all stresses, bending, compression and tension).

Denmark.

| t/R | 1/100 | 1/125 | 1/150 | 1/200 | $\leq$ 1/250 |
|---|---|---|---|---|---|
| Factor | 0.80 | 0.87 | 0.90 | 0.95 | 1.00 |

UK.

| t/R | 1/100 | 1/125 | 1/150 | 1/175 | 1/200 | 1/250 | 1/300 |
|---|---|---|---|---|---|---|---|
| Factor | 0.80 | 0.87 | 0.90 | 0.91 | 0.93 | 0.94 | 0.95 |

Values given in the tables for Denmark and the UK are very similar. The formula in the Belgian Code gives the same modification factors as those in the table in the Danish Code.

Bending Stresses Due to Bending Moment to Curved Members.

No mention is made in Belgian, French or UK Codes of modifications to bending stress. Modifications given in the other Codes are as follows:

Denmark.

For curved members acted on by a moment as shown in the diagram the stress on the inner surface is given by:

$S_{Mi} = 6k_i \; M/bh^2$

and the stress on the outer surface:

$S_{my} = -6k_y \; M/bh^2$

where $k_i$ and $k_y$ are given in the table below:

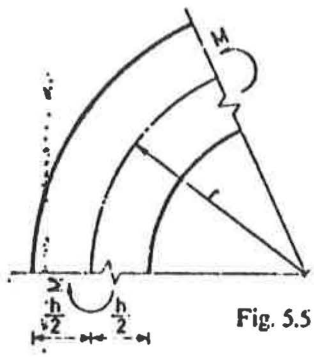


Fig. 5.5

| $\frac{r}{h}$ | $k_i$ | $k_y$ |
| --- | --- | --- |
| 2 | 1.20 | 0.85 |
| 3 | 1.12 | 0.90 |
| 4 | 1.09 | 0.92 |
| 5 | 1.07 | 0.94 |
| 6 | 1.06 | - |
| 7 | 1.05 | - |
| 8 | - | 0.96 |
| 9 | - | - |
| 10 | 1.03 | 0.97 |
| $\geqslant$15 | 1.00 | 1.00 |

Germany.

Where $10 > R/d \geqslant 2$, bending stress is calculated by the following formula:

$$fa = \frac{M}{Z} \left( \frac{2R + d}{2R} \right)$$

Holland.

Permissible bending stress is reduced by the following formula:

$$\frac{2R}{2R + d}$$

The German and Dutch Codes give the same modification factor but in the former it is used to increase the applied stress and in the latter the permissible stress is reduced.

The omission of formulae in other Codes does not preclude the use of a modification factor since the formula has an analytical basis which could be considered inplicit in the design procedure.

## PLYWOOD

Belgian and Dutch Codes give no design information for plywood.

Information given in the other Codes is summarised below:

### Denmark

Guidance on the design of built-up members such as ply-web beams and stressed skin panels, but no permissible stresses for plywood.

### France

E Values for tension, compression and bending, in directions parallel and perpendicular to exterior plies, and transversal compression are taken as equal to the corresponding values for solid timber of the species used, unless tests are carried out on the ply. E values for tension, compression and bending, when load is at $45^{\circ}$ to the exterior ply ~~is~~ are taken as $\frac{1}{3}$ the values for solid timber.

Permissible stresses are given for Douglas Fir, Okoumé, Birch and Makoré plywoods. Stresses given are : tension, compression, panel and rolling shear (for loads parallel, perpendicular and at $45^{\circ}$ to the outer ply) and bending in planes parallel and perpendicular to the outer ply.
No distinction is made of different thicknesses except for 3-ply Douglas Fir where a higher stress is allowed for tension and compression parallel to the outer ply and bending. In some ~~cases,~~ plywoods permissible rolling shear stress is reduced when the joint is adjacent to a plywood edge, as in the case of a ply-box beam. Stresses apply to plywood at a moisture content of 15% and should be modified by the same factors as those for solid timber at different moisture contents.

Critical points on plywood gussets are indicated.

Germany

Plywood can be made of a variety of woods including birch, beech, spruce, pine, limba, makoré, mahogany, gaboon and fur.

Permissible stresses are given for bending compression and shear normal to the plane of the board and in the plane of the board, and tension in the plane of the board. For bending normal to the plane of the board, and bending, tension and compression in the plane of the board, stresses are given parallel and normal the the grain of the top ply. A lower stress is given for tension and compression in the plane of the board when the angle between the directions of top ply grain and applied stress is between $30^{o}$ and $60^{o}$. Stresses forother angles are obtained by interpolation.

E values are given parallel and perpendicular to the grain, in addition to a G value.

United Kingdom

Permissible stresses are given for Canadian Douglas Fir, Finnish Birch and British Plywood manufactured from tropical hardwoods. Constructions are identified : in the case of Canadian Douglas Fir, stresses are given for various grades, number of plies and total thickness; in the case of Finnish Birch and British plywood, stresses are given for various numbers of plies.

Stresses given are : tension, compression, panel and rolling shear (for loads parallel, perpendicular and at $45^{o}$ to the outer ply), bending in planes parallel and perpendicular to the outer ply, and bearing on the face. Permissible Rolling shear stress is multiplied by 0.5 when a joint is adjacent to a plywood edge.

Moduli of elasticity in bending, tension and compression with loads parallel and perpendicular to the face grain are given for all three types and with loads at $45^o$ to the face grain for Finnish Birch. Moduli of rigidity for loads parallel, perpendicular and at $45^o$ to the face grain are given for all three types.

In addition to mechanical properties, considerable information is also given about physical properties, covering numbers of plies, veneer thicknesses, section properties for a 1 m width and weights.

These stresses and moduli apply to plywood when moisture content is $>$18% and modification factors are given for wet conditions.

<u>Comments</u>

1. The French Code allows E values equal to those of the solid timber choosing // ~~of~~ or $\perp$ values according to the direction of the face grain of the ply wood.

In the German Code, E// is 70,000 kg/cm$^2$ for plywood compared to values of 100,000 to 125,000 for solid timber; It is difficult to draw comparisons from the U.K. Code because of the large number of ply constructions considered but the majority of E// values for Canadian Douglas Fir, are between $E_{MEAN}$ and $E_{MIN}$ for Douglas Fir solid timber with some E// $< E_{MIN}$. These comparisons suggest that the French approximation would give results very different from those obtained using the German or U.K. codes.

2. The German Code gives a method to evaluate permissible stresses for tension and compression in the plane of the board when the direction of the load to grain is between $0^o$ and $90^o$. Since plywood is often used for applications where maximum stresses are applied at angles between $0^o$ and $90^o$, similar guidance should be available in other countries. The permissible stresses are given for loads at $45^o$ to the face grain ~~given~~ in Codes for France and the U. K. but it is not clear whether ~~interpoldtan~~ interpolation is allowed for other angles.

3. Reduction of permissible rolling shear stress when a joint is adjacent to the edge of the ply is relatively small in the French Code compared to the 0.5 given in the U.K. Code. No reduction is given in the German Code.

MDR/JA
6th February,

## Lateral Support to Beams

## (Solid & Laminated)

UK

The depth to breadth ratio for solid and laminated members with rectangular cross-sections are not to exceed the value given in table 17 corresponding to the appropriate degree of lateral support, unless specially calculated.

TABLE 17: MAXIMUM DEPTH TO BREADTH RATIOS
(SOLID AND LAMINATED MEMBERS)

| Degree of Lateral Support | Maximum Depth to Breadth Ratio |
|---|---|
| 1. No lateral support | 2 |
| 2. Ends held in position | 3 |
| 3. Ends held in position and member held in line, as by purlins or tie rods | 4 |
| 4. Ends held in position and compression edge held in line, as by direct connection of sheathing, deck or joists | 5 |
| 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 |
| 6. Ends held in position and both edges firmly held in line | 7 |

Part of the clause states that unless specially calculated, the values quoted will be adequate.

However, if presented with a problem of calculating sufficient lateral support to the compression zone, it is generally accepted that the restraining force should be at least $2\frac{1}{2}\%$ of the flange force.

HOLLAND

CLAUSE 6.2.2.1

Where lateral support is provided by secondary members at intervals not greater than 12b the maximum values of d/b given in Table 26 apply.

CLAUSE 6.2.2.2

As above, but for beams with rotational restraint at the ends, Table 27 applies.

CLAUSE 6.2.2.3

Where lateral support is provided at intervals greater than 12b, calculations must be made of buckling resistance. The effective length is taken as the spacing of the lateral restraints ($= l$)

The maximum stress due to bending or combined bending and tension (compression) should be less than $\frac{0.2E//b^2}{l\ d}$ with a maximum of $\bar{\sigma}_b$ (the maximum permissible bending stress).

CLAUSE 4.3.1.2

For members used as floor beams, if b is less than $\frac{1}{4}$d and/or $^1/80$ of the span, the beam requires a lateral support to the bottom edge at least in one position within the middle third of the span.

GERMANY

CLAUSE 8.2

If d/b is greater than 4 but not greater than 10 full support may be assumed.

if $r_{yy} \geqslant \frac{l}{40}$

$\left\{ \text{i.e. } \frac{l}{r_{yy}} \leqslant 40 \right.$

or $\frac{l}{b} \leqslant 11.6$

as $\left. r_{yy} = \frac{d}{\sqrt{12}} \right\}$

If d/b is greater than 4 but not greater than 10, with

$r_{yy} < \frac{l}{40} \left\{ \text{i.e. } \frac{l}{r_{yy}} > 40 \right\}$

then the compressive stress must not exceed

$\frac{1.26 \times \sigma_{D}//}{W}$ (W is a coefficient related to $\frac{l}{r}$ )

from table 4 - page 19 of text, and $\sigma_D //$ is permissible compressive stress from table 6 - page 28 of text.

If d/b $>$ 10 a more accurate analysis must be carried out to assess the lateral restraint required see fig. 8/3 page 93 of explanatory notes B.S.I. Document 72/ 13508.

BELGIUM

For members subjected to bending only for for combined bending and axial forces, the designer must justify the ratio chosen for calculation of adequate stability by following a recognised method.

FRANCE

When d/b is greater than 5 and $l/r_{yy}$ is greater than 37.5 the bending stress must be checked to ensure that it does not exceed the safe buckling stress.

DENMARK

Permissible bending stresses are obtained by modifying the stresses by a factor (c) giving in the following graph. (c) is a function of a slenderness ratio

$$(h) = k_1 \sqrt{\frac{lh}{b}}$$

where l = span

h = depth of section

b = breadth of section

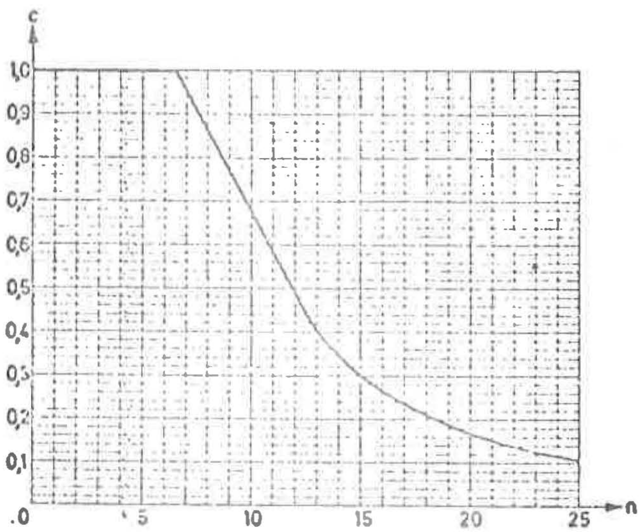
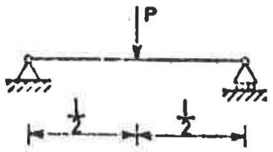
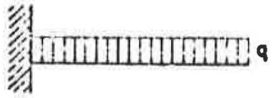
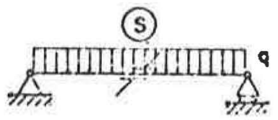
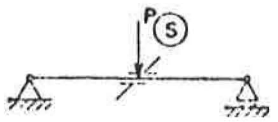



Fig. 5.3.

Tabel 5.4 *Bestemmelse af faktoren $k_1$*

| Belastnings- og bjælketype | $k_1$ |
| --- | --- |
|  | 0,60 |
|  | Ⓣ 0,58<br>Ⓜ 0,56<br>Ⓤ 0,54 |
|  | Ⓣ 0,53<br>Ⓜ 0,51<br>Ⓤ 0,49 |
|  | Ⓜ 0,41 |
|  | Ⓜ 0,52 |
|  | Ⓣ 0,37<br>Ⓜ 0,36<br>Ⓤ 0,36 |
|  | Ⓜ 0,30 |

Ⓣ Belastningen virker på bjælkeoversiden.
Ⓜ Belastningen virker i den neutrale akse.
Ⓤ Belastningen virker i bjælkeundersiden.
Ⓢ Angiver, at bjælken er fastholdt i midten mod drejning og side-udbøjning, medens lodret udbøjning kan foregå frit.
De anførte værdier forudsætter, at bjælkens vridning er hindret ved vederlagene.

$K_1$ is obtained from this table giving certain loads, moment arrangements and beam supports/restraints.

T-load acting on top of the beam.

M-load acting in the neutral axis.

U-load acting on under-side of the beam.

S-Beam restrained in the middle against turning and side-movement, while perpendicular bending can take place freely.

COMMENTS

In the Belgian Code it simply states that for bending or combined bending and axial forces stability should be justified by calculation. This is in broad agreement with the French Code except justification is only required if d/b is greater than 5 and the $\frac{l}{ryy}$ is greater than 37.5. In the British Code a provision is made for justicication by calculation, although simplified rules for lateral support are included. For methods of calculating stability, certain divergence of opinion could be implied. However, treating the member as a column tending to buckle between points of support is thought to be an acceptable procedure.

With the German Code if d/b is greater than 4 but not greater than 10 and $\frac{l}{ryy}$ does not exceed 40, full support can be assumed. (This part is somewhat similar to the French). With $\frac{l}{ryy}$ exceeding 40 the compressive bending stress is limited to a permissible stress obtained from a formula. With d/b greater than 10 full analysis for lateral restraint must be made.

The Dutch Code gives permitted d/b ratios where lateral support is provided by secondary members up to intervals of spacing not greater than 12 b. The permitted d/b ratio varies with the value of $\frac{l}{b}$, where l is the spacing between lateral support. Two tables are provided covering beams without rotational restraint and with rotational restraint at ends. For beams supporting floors there is is an additional clause where b is less than 4d and/or $\frac{1}{80}$ of the span, bottom edge restraint is required at least in one position within the middle third of the span.



The Dutch Code also gives design requirements where lateral supports are at intervals greater than 12 b. To satisfy these requirements calculations are to be made for the buckling resistance of the beam between lateral supports. The actual bending or combined bending and axial stress to be limited to the stress given by a formula with a provision that the normal bending stress is not exceeded.

The Danish Code recommends the use of a factor for modifying stresses for various load-moment arrangements and types of beam support-restraints. Reference is given to the source of $K_1$ as R. F. Hoorly and B. Madsen : Lateral Stability of Glued Laminated Beams. Proceedings of the American Society of Civil Engineers.

PAPER 16

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# NORDIC PROPOSALS FOR SAFETY CODE FOR STRUCTURES AND LOADING CODE FOR DESIGN OF STRUCTURES

by

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PARIS - FEBRUARY 1975

NORSK TRETEKNISK INSTITUTT

CIB W. 18
Timber Structures

Paris, 19 - 21 Feb. 1975

# Nordic proposals for

# Safety Code for Structures and

# Loading Code for Design of Structures

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## Introduction

Nordic Council (= the governments of Denmark, Finland, Iceland, Norway and Sweden) has set up a joint committee of the national building authorities of the five countries. This Nordic Building Regulations Committee, NKB, works on harmonizing the technical Building Regulations within the nordic countries.

Two of the subcommittees of NKB have worked out proposals for a Safety Code and for a Loading Code. The proposals are now being discussed in the nordic countries, all comments should be given before 1 June 1975. The object on the Safety Code is to guide the committees which will be working out the Nordic Structural Design Codes.
The Loading Code should be incorporated in the national regulations.

## Principles

The Safety Code gives two principles on which the regulations are based

1. Partial Factor Method

2. Statistical Method

The Partial Factor Method is stated by ISOas an international standard and is a simple method which can be used for all types of structures.

The Statistical Method is, however, expected to give a greater possibility for development, and if practice will show that this method gives economical advantages, it might be more used in the future.

The object of giving both methods is to open the door for a futural development.

The actual suggested values for the partial factors are given in such a way that design according to the Partial Factor Method should give about the same results as a design according to the Statistical Method.

## Summary of Content

1. Safety Code.

The main part of the Safety Code deals with

Safety Groups of Structures
Limit States
Requirements

### 1.1 Safety Groups

| Consequence of rupture | Safety group |
|---|---|
| Great probability of damage to person<br>Great material damage | 3 |
| Some probability of damage to person<br>Some material damage | 2 |
| Less probability of damage to person.<br>Less material damage or some material damage if accepted from an overall point of view | 1 |

Some examples as guidance:

Safety group 3:

Buildings with more than two stories, halls and spectactor stands where many people often are gathered, i.e. houses, office buildings, theatres, sport buildings, factories.
Pedestrian, road and railway bridges etc.

Safety group 2:

Buildings with more than two stories and halls where people come from time to time only, as store buildings.
Small 1- and 2- story buildings where many people often are gathered, small houses, small office buildings, small factory buildings etc.

Safety group 1:

Small 1 and 2 story buildings where people come from time to time only, such as store buildings and sheds.

Parts of a stucture may be given a lower safety group than the whole structure or building.

## 1.2 Limit States

The Safety Code gives three Limit States:

Serviceability Limit State

| | |
|---|---|
| unacceptable | deflections<br>cracks<br>vibrations<br>etc. |

Ultimate Load Limit State

structural mechanism
material rupture
structural buckling
material fatigue
etc.

Progressive Collapse Limit State

the limit of collapse of the rest structure assuming that certain parts already have failed.

## 1.3 Requirements

The requirements at Serviceability Limit State are to be given in the actual Structural Code.

The requirements at Ultimate Load Limit State are dependent of the Safety Group, the Type of Rupture and the Type of Load Combination.

The type of rupture refers to three groups

III Brittle ruptures, stability ruptures

II Neither I nor III

I Ductile rupture, with a reserve of load carrying capacity exceeding the defined modulus of rupture.

The actual Structural Code must give the relevant type(s) of rupture.

## 1.4 The Partial Factor Method

At each Limit State the safety of the structure is examined by demanding that

$$\text{design load} \leq \text{design capacity}$$

The design load is the characteristic load multiplied by the load factor, and the design capacity in the characteristic capacity divided by the material factor.

The charachteristic loads are given in the Loading Code.

The characteristic capacity of the material is given in the actual Structural Code. Normally it is based on the 10% fractile, but the 5% or 0.1% fractile could be used. If the capacity is dependent on environment (climate) or duration of load, this should be taken into account when determining the characteristic capacity.

Load factors

| Ultimate Limit State | Load Combination | | |
|---|---|---|---|
| Loading | I | II | III |
| Dead load of building parts | 1.5 | 1.2 | 1.0 |
| Dead load from soil | 1.5 | 1.2 | 1.0 |
| Imposed Load | | | |
| Usual load | 1.3 | 1.3 | 1.0 |
| Unusual load | - | 1.5 | - |
| Accident Load | - | - | 1.0 |
| Deformation Load | 1.0 | 1.0 | 1.0 |

For dead load from building parts, the following load factors shall be used if this is more severe than the corresponding factors above:

| Safety group | 3 | 2 | 1 |
|---|---|---|---|
| Dead load of building parts | 0.9 | 0.95 | 1.0 |

The Load Combination is defined in the Loading Code.

There are given some additional rules for modifying the load factors at Ultimate Limit State.

At Progressive Collapse Limit State the Load Factors are all 1,0.

Material factors (Resistance factors)

Ultimate Limit State.
The resistance factor is the product of three partial factors.
The first partial factor depends upon the Safety Group and the Type of Rupture.

| Type of rupture | Safety Group | | |
|---|---|---|---|
| | 3 | 2 | 1 |
| III | 1.60 | 1.40 | 1.23 |
| II | 1.40 | 1.23 | 1.08 |
| I | 1.23 | 1.08 | 0.95 |

The second partial factor depends upon the fractile at Lower exclusion limit and the Coeffisient of Variation of the resistence parameter.

| Coeffisient of Variation of resistance parameter | Fraction | | |
|---|---|---|---|
| | 10% | 5% | 0.1% |
| $V < 0.1$ | 1.10 | 1.05 | 0.96 |
| $0.1 < V < 0.2$ | 1.05 | 1.00 | (0.80) |
| $0.2 < V < 0.4$ | 1.15 | 1.05 | (0.73) |

The values in parenthesis will normally not be used, as the combination of 0.1% fractile and high variation is unsuitable.

The third partial factor depends upon the control of material and construction, and of the possible deviation between the theory and reality.

| Scope of control with material identity and product | Possible deviation between real and measured resistance. Possible uncertainty of design model. Possible geometrical uncertainty. | | |
|---|---|---|---|
| | large | medium | small |
| Small | 1.30 | 1.12 | 1.05 |
| Average | 1.20 | 1.05 | 1.00 |
| Good | 1.12 | 1.00 | 0.95 |

Progressive Collapse Limit State.

The material factor is equal to 1.0

## 1.5 The Statistical Method

This method is not delt with in this paper.

## 2. Loading Code

The Loading Code Proposal is coordinated with the Safety Code, mainly through the presentation of characteristic values for loads. Where the committee has found it possible, even average values and coeffisients of variance for loads are given.
The motive for giving rather detailed refinements has been the assumed need for economic design methods for mass produced building components. Simpler methods could be used for single structures.

<u>Classification of loads</u>

### 2.1 Duration of loads

The load with intensity q is classified according to the continuous duration $t_{qs}$ in the following groups

| Duration of Load group | Limits for continuous duration $t_{qs}$ | | Example |
|---|---|---|---|
| | Lower | Upper | |
| A | 250 d | - | Dead load |
| B | 15 h | 250 d | Snow load |
| C | 2 s | 15 h | Load of people |
| D | - | 2 s | Impact load |

If the continuous duration of load is insufficient to describe the effect of the load, and the accumulated load effect should be considered, this should be taken into account in the Structural Design Code.

Probability for intensity - classification

Relative duration = time loaded/life time

Constant load

This load occurs at an arbitarily chosen time.
Relative duration = 1
The characteristic value is defined as the medium of the intensity. (Mean value could also be used).

Usual load

This is a load with a probability of 0, 2 or more for being exceeded at least once in a year. The characteristic value is defined with p = 0, 2.

Short term usual load

The relative duration of the characteristic intensity is equal to or less than 0.001.

Not short term usual load

The relative duration of the characteristic intensity is greater than 0.001.

Unusual load

The probability of being exceeded at least once a year is between 0.2 and 0.02. The characteristic value is defined with p = 0.02.

Extreme load

The extreme load is of a type of accident load and is more seldom than an unusual load.
The intensity is not given in the Code.

## 2.2 Load combinations

The single load types are given a value $\alpha$ according to following table:

| | $\alpha$ |
|---|---|
| Constant load | 0 |
| Not short term usual load | 0.5 |
| Short term usual load | 1 |
| Unusual load | 2 |
| Extreme load | 4 |

There are three load combinations:

I Usual loads adjusted according to their duration, in such a number that the sum of $\alpha$ does not exceed 4.

II One unusual load combined with usual loads, adjusted to their duration, in such a number that the sum of $\alpha$ does not exceed 4.

III One extreme load combined with usual loads, adjusted according to their duration, in such a number that the sum of $\alpha$ does not exceed 5.

The Code gives rules for the adjustment of loads according to their duration.

Comments

The proposals for Nordic Codes on Safety and Loads are now being discussed in the nordic countries, and it is not possible to say in which form they finally will be accepted.

The Code text is rather complicated, and it is not to be expected that it will be understood by normal designers.

It is not clear if the level of complexity is justified by the expected gain in economy.

It will be one of the tasks of the Structural Design Code Committees to make the necessary simplifications in such a way that the Codes can be used in practical life.

It is assumed that the Common Nordic Structural Codes should be finished at about 1980.

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X nr. 4525

PAPER 17

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# PROPOSAL FOR SAFETY CODES FOR LOAD-CARRYING STRUCTURES

NORDIC COMMITTEE FOR BUILDING REGULATIONS

PARIS - FEBRUARY 1975

NORDIC COMMITTEE FOR BUILDING REGULATIONS (NKB)
Sub-Committee on Structural Safety

Proposal for Safety Codes for Load-carrying Structures

Code and Recommendation Concerning Safety Methods

24/10 1974

CONTENTS

PREFACE

The present proposal for safety codes for load-carrying structures is mainly based on the proposal from September 1971 for guide lines for safety methods, made by the NKB sub-committee on structural safety.

Contrary to previous guide lines, the present proposal suggests detailed safety methods. These methods show due consideration for the criticism of the guide lines, and they moreover conform to the most recent developments. On essential matters, the proposal conforms with ISO 2394: "General Principles for the Verification of the Safety of Structures". However, the ISO-standard only states principles, whereas this proposal gives detailed regulations.

The proposal has been coordinated with the Load regulations set up by the NKB committee on loadings.

The proposal has been divided into code and recommendation with explanatory and detailed comments. The recommendation is written in reduced column.

Separately, comments for codes and recommendations have been given, which state the reasons for the suggested principles, and moreover state the calculations for the values of the partial coefficients given in the proposal.

Code and recommendation and corresponding comments have been written with a view to being used by the committees, working groups, etc., that are in charge of preparing structural codes (codes for structures built in certain ways of certain materials).

The safety methods are to be applied to the degree relevant for each structural code.

The proposal has been set up by the NKB sub-committee on structural safety. The members of the committee are given below:

Vicedirektoer Per Bredsdorff, chairman, Denmark
Civilingeniør Olaf Mohr, Denmark

Ingeniørdocent Ervin Poulsen, Denmark

Tekn. dr. Eero Paloheimo, Finland

Professor Ivar Holand, Norway

Lektor Sture Åkerlund, Sweden (withdrew from the committee 73-6-30)

Professor Lars Östlund, Sweden

Secretaries:

Civilingeniør Rolf Harboe, Denmark

Akademiingeniør Knud Skov, Denmark

Moreover, Ove Ditlevsen, Denmark, dr. techn. has served on the committee on an advisory basis.

## 1. INTRODUCTION

The general safety requirements imposed on load-carrying structures are indicated in chapter 3. In order to ensure the fulfilment of these requirements, one of the methods given in chapters 4 and 5 can be used. It should be emphasized that the given requirements do not allow for any gross errors in design or construction.

The method given in chapter 4 has been based on characteristic values and partial coefficients.

The method given in chapter 5 is a statistical method based on mean values and coefficients of variation.

The partial coefficient method should be used when applying the structural codes. These should, however, also allow for the use of the statistical method.

The statistical method is primarily intended to be used by the structural code committees for the determination of the partial coefficients not given in this code proposal, and also for any future revision of partial coefficients. The parameters of this method have been determined in such a way that on an average they result in the same dimensions as existing structural codes when applying some frequently used formula for calculation.

The partial coefficients specified in chapter 4 have been determined in such a way that design according to this method on the whole does not result in any smaller dimensions than those obtained by using the statistical method. Please see comments.

Other statistical methods with their corresponding requirements can be used provided the necessary level of safety is reached. Besides statistical variations in loading- and resistance parameters all the remaining uncertainties should be taken into account in connection with calculation, execution, control on site, use of structures, etc.

Any other statistical methods can only be used provided the method has been unambiguously formulated and provided it can be proven that from a safety point of view, the method is equal to the present statistical method.

It is assumed that an evaluation of a statistical method, prior to its acceptance, is made by a committee with sufficient competence both as regards mathematical statistics and structural techniques.

When introducing any other statistical method, is should be proven that the dimensions reached by the method at the most result in the formal probabilities of failure, $p_f$, indicated in table A1, when applied on a series of examples typical in practice. In this connection, the formal probability of failure should be calculated on the assumption that the primary parameters of resistance and geometry and the judgement factors of the parameters of resistance follow logarithmic normal distributions, whereas the distributions of the primary parameters of loading as well as their judgement factors are assumed to follow a normal distribution.

Following the above, $p_f$ should correspond to the annual formal probability of failure, and those values of $p_f$ should be selected from table A1 which correspond to the values of $\beta$ required in chapter 3. Regarding uncertainties, reference is made to chapter 5.

It should be emphasized that the formal probability of failure is a quantity of calculation-technical significance.

| $\beta$ | 5.25 | 4.75 | 4.25 | 3.75 | 3.25 |
|---|---|---|---|---|---|
| $p_f$ | $10^{-7}$ | $10^{-6}$ | $10^{-5}$ | $10^{-4}$ | $8 \cdot 10^{-4}$ |

Table A1. Correspondence Between Safety Index $\beta$ and Formal Probability of Failure $p_f$

# 2. TERMINOLOGY AND SYMBOLS

## 2.1 Terminology

### Action

The load-carrying function of the structure is considered to be any effect on a structure, structure part, or any material, caused by loads, which can determine whether the resistance is sufficient to fulfil the functional requirements.

### Calculation Model

The model of the function of the structure used for the evaluation, including its design.

### Characteristic Values

The value of a stochastic variable determined by observations as well as any advance knowledge, which in a determined probability is not expected to be exceeded by the stochastic variable. Cp. also 4.2.

### Code Value

Value for load, resistance, parameter of geometry, and the like, which is used as the equivalent for a characteristic value, in cases where a statistical rule is not relevant.

### Design

The determination of structural dimensions with a view to establishing sufficient resistance. Loading, appearance, main dimensions and type of structural material are assumed to be known factors.

### Design value

Used about the deterministic load values, resistance, and parameter of geometry, which form the basis for an approximate evaluation as to whether a certain limit state has been reached, the relevant uncertainties and other reservations being included in these values of calculation. Cp. 4.1.

Failure

Common denomination for the excess of any chosen limit state.

Failure Probability

The probability that the defined resistance is exceeded within a period of one year.

Failure type

Cp. 3.4.

Judgement Factor

Cp. 5.4.

Limit States

Cp. 3.2.

Load

The load capacity is defined as any exterior load which can lead to mechanical stress or strain in the structure. Cp. also "Load Regulations".

Load Effect

The effects on structure parts and materials caused by the load on the structure.

Load Model

The model of the load used for the evaluation of the structure, including design.

Partial Coefficient

Coefficient used to fill the safety requirements at the evaluation of the structure. Cp. 4.1.

Primary Load Parameter

Load or load intensity stipulated in the load regulations. Cp. "Load regulations".

Primary Resistance Parameter

Parameter of resistance which is assumed to be determined directly by means of codes, control, or testing.

Resistance

The maximum load a structure, structural part, or a material can resist and still fulfil the functional requirements, for instance expressed at the limit states.

Resistance Model

The model of the resistance used for the evaluation of the structure, including design.

Resistance Parameters

Used for parameters stipulating the resistance.

Safety Class

Cp. 3.1.

Safety index

Defines the safety of the structure. Cp. 5.1.

Safety Method

Method or procedure used for the evaluation of whether a structure has the stipulated degree of safety.

Serviceability Limit States

Cp. 3.2.

Structural Code

Codes for structures built in certain ways of certain materials.

Structural Evaluation

The total process of evaluating a structure when related to the functional requirements.

Ultimate Limit States

Cp. 3.2.

Ultimate Limit States corresponding to progressive collapse

Cp. 3.2.

## 2.2 Symbols

Below the most commonly used symbols are stated. Other used symbols will be explained as they appear.

$\ell$ geometrical parameter
$m$ primary resistance parameter
$p$ primary load parameter
$p_f$ formal failure probability
$V$ coefficient of variation
$\beta$ safety index
$\gamma$ partial coefficient
$\gamma_m$ partial coefficient of primary resistance parameter
$\gamma_p$ partial coefficient of primary load parameter

# 3. PRINCIPLES FOR THE EVALUATION OF STRUCTURAL SAFETY

## 3.1 Safety Classes

The rate of safety against failure of a structure with regard to its load capacity is made dependent upon the risk for personal injury and the importance of the structure, the latter being determined by one of the three safety classes mentioned in table 3.1.

| Consequence of failure | Safety Class |
|---|---|
| Great probability for considerable personal injury. Great material damage. | 3 |
| Some probability for personal injury Some material damage . | 2 |
| Little probability for personal injury. Little material damage. A certain degree of material damage may be acceptable. | 1 |

Table 3.1 Safety Classes

In the structural codes, the safety classes indicated could be grouped together into one or two classes by referring structures from a lower safety class to a higher.

The safety classes mentioned in table 3.1 refer to structures within the normal field of experience.

There may be structures for which a failure will lead to catastrophic consequences, and for which the present codes cannot be expected to give the acceptable degree of safety. This may for instance apply to buildings of extreme height, bridges of extreme length, pressure vessels for atomic reactors, etc.

Structural elements which are not considered to be included in the load carrying function of the structure as a whole but only considered to transmit forces to the main structure, may be referred to a lower safety class than the structure as a whole, if the consequences of a failure of such elements permit reference to a lower safety class according to table 3.1.

The following examples serve as guidelines for a classification of the finished structures:

Safety Class 3:
Buildings of more than 2 storeys, hall structures and stages which will often hold many persons, and for instance be used for living quarters, offices, theater, sports, or production.

Pedestrian Bridges.

Road Bridges.

Railroad Bridges.

Tall pylons, detached towers, including chimneys, close to houses. Large water towers and siloes near houses.

Safety Class 2:
Buildings of more than 2 storeys and hall structures which only occassionally hold people, for instance stock buildings.

Small 1- and 2-storey buildings often used for people, for instance houses, offices, or production buildings.

Cranes.

Tall pylons and detached towers including chimneys in areas with no surrounding houses.

Small pylons and detached towers including chimneys close to housing areas.

Small water towers and siloes in areas with no surrounding houses.

High tension pylons next to roads and the like, as well as pylons serving to close off any progressing cable failure.

Scaffolds and moulds.

Safety Class 1:

Small 1- and 2-storey buildings which only occassionally hold persons, for instance stock buildings and sheds.

Small pylons and detached towers including chimneys in areas with no surrounding houses.

Normal high tension pylons.

Normal street pylons.

---

For structures whose classification depends upon the build-up conditions of the area, future conditions as related to the lifetime of the structure should be taken into account.

The following examples may be given of structural elements which may be referred to a lower class than the structure as a whole:

Floors and external walls in buildings (regardless of number of storeys) on the condition that these elements are not designed to carry loads in their own plane.

Roofs for structures mentioned in classes 3 and 2.

Internal walls in buildings (regardless of number of storeys) on the condition that they are not designed to carry loads in their own plane.

## 3.2 Limit States

For the evaluation of the load carrying function of a structure, the following groups of limit states have been established, i.e., states where the structure or part of it is at the point where the requirements cease to be satisfied.

### A. Serviceability Limit States

These cases correspond to the limit for failure with regard to their serviceability.

The following cases of failure can be mentioned:

A1. Deformations which are unacceptable with respect to the normal use of the structure.

A2. Crack formations which are unacceptable with respect to requirements regarding tightness.

A3. Unacceptable oscillations or vibrations (any material fatigue is referred to category B).

### B. Ultimate Limit States

These states correspond to the limit for failure of a part of, or all of a structure.

The following cases of failure can be mentioned:

B1. Transformation of the structure into a mechanism.

B2. Failure in the material resulting from an extreme load.

B3. Failure in the structure resulting from instability without failure in the material. This includes buckling and lateral instability.

B4. Mutual slide of the entire structure or parts of it.

B5. Overturn.

B6. Shake down.

B7. Deterioration of material resulting from plastic deformation with changing signs.

B8. Deterioration of material due to fatigue.

C. Ultimate Limit States Corresponding to Progressive Collapse

These states correspond to the limit for progressive collapse of a structure under the assumption that certain parts of the structure already have ceased to perform their load carrying functions or that for other reasons they do not act as expected.

Examples of the remaining structure are states of failure corresponding to the states of failure B1 - B5.

### 3.3 Requirements at the Serviceability Limit States.

General requirements at the serviceability limit states are not given.

It is assumed that the structural codes will specify the relevant requirement at the serviceability limit states.

### 3.4 Requirements at Ultimate Limit States.

The requirements at the ultimate limit state depend upon the safety class of the structure, the type of failure and the type of loading.

The type of failure decides to which of the three groups, I, II, and III, it is referred. The evaluation of the type of failure pays special attention to the stress-strain-curve of the material, the structural elements, or the entire structure.

Failure type III includes states of brittle rupture and instability failures and similar failures.

Failure type II includes types not covered by I or III.

Failure type I deals with ductile failures with the requirements that an extra carrying capacity beyond the defined resistance is available, i.e., in the form of strain hardening or an extra capacity due to redistribution of internal forces.

Structural codes may use the relevant types of failure.

The safety requirements are expressed as requirements to the safety index $\beta$, where $\beta$ is defined as indicated in section 5.1.

The requirements to $\beta$ are specified in table 3.4 below and are based on an annual calculation of $\beta$. The $\beta$-values in table 3.4 apply always in so far as dead loads are concerned and also in so far as live loads are concerned and also in so far as actions caused by deformations are concerned.

| Safety Class | 3 | 2 | 1 |
|---|---|---|---|
| Failure Type III | 5.25 | 4.75 | 4.25 |
| Failure Type II | 4.75 | 4.25 | 3.75 |
| Failure Type I | 4.25 | 3.75 | 3.25 |

Table 3.4 Safety Code $\beta$ for the Ultimate Limit State.

For accidental loads the requirements to $\beta$ apply only as far as structures in safety class 3 are concerned, and only if it is decided not to satisfy the requirements in the ultimate limit states corresponding to progressive collapse (see section 3.5), or if permanent measures are taken to prevent the accidental loads from acting on the structure.

No specific requirements are made as regards accidental loads acting on structures in safety classes 2 and 1. Depending upon the type of structure, such requirements may, however, be made in the structural codes.

The evaluation of the type of failure depends upon the point on the stress-strain-curve which is defined as resistance. This is, especially with a view to the establishment of structural codes, illustrated in figure A.3.4, which is assumed to correspond to some schematic curves for steel. The ordinate is stress σ, and the abscisse is strain ε.

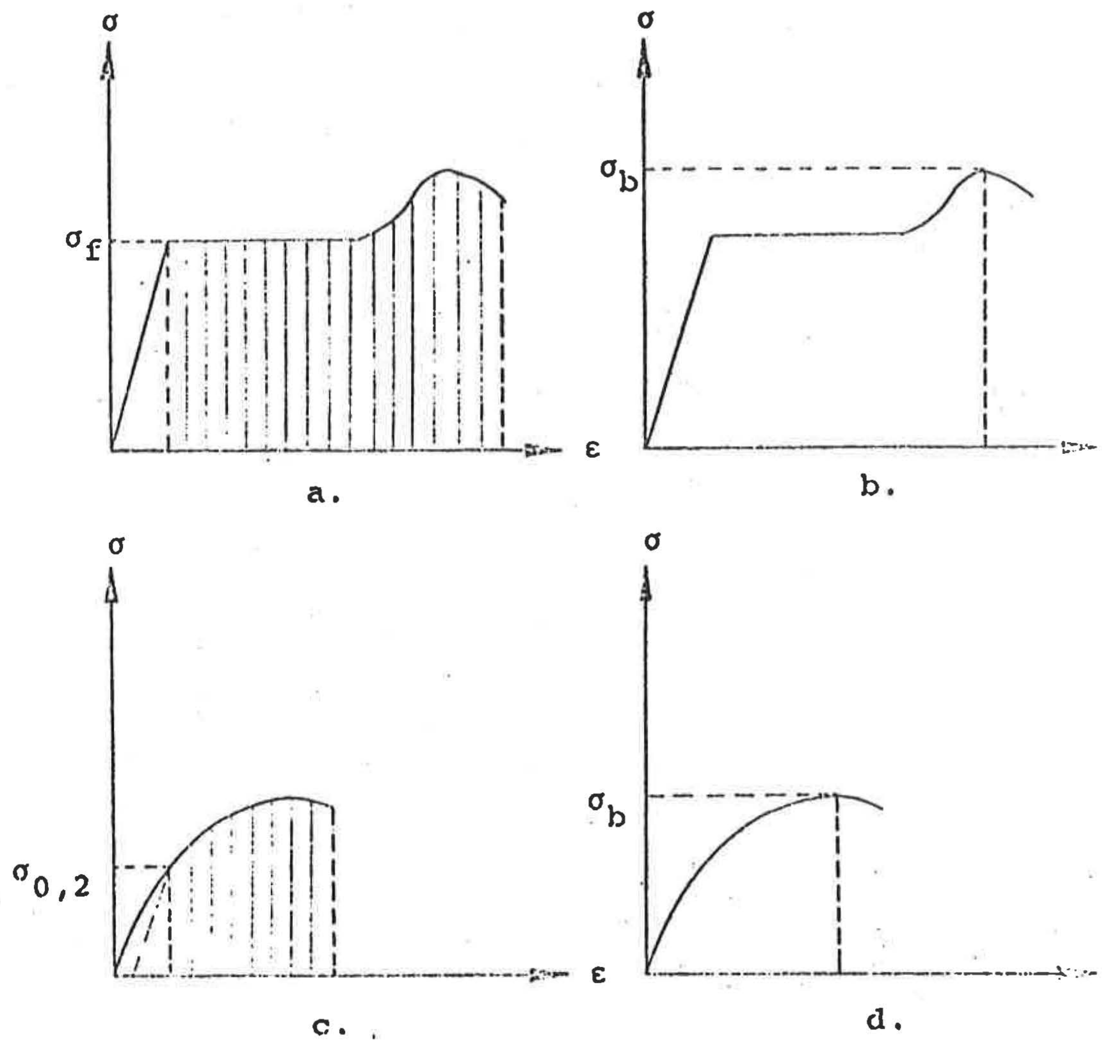


Figure A.3.4 Stress-strain Curves and Defined Resistance

In the states a and c, corresponding to a definition of the resistance as yield stress $\sigma$ and stress $\sigma_{0,2}$ (0,2% strain), the deformation is great after the limit of resistance has been reached, and an extra load carrying capacity in the form of ultimate strength $\sigma_b$ remains. The failure is therefore referred to type I.

In the states b and d corresponding to a definition of the resistance as ultimate strength, the remaining deformation is theoretically zero, and there is no extra load carrying capacity. The failure is therefore referred to type III.

Failures due to fatigue are referred to type III.

## 3.5 Requirements at Ultimate Limit States Corresponding to Progressive Collapse.

The requirements at this ultimate limit states apply only to structures in safety class 3.

If those elements of a structure which may be directly acted upon by accidental loads, are not designed for accidental loads at the ultimate limit state, and if no permanent measures have been taken to prevent accidental loads from acting on these structural elements, the remaining part of the structure should be designed at the ultimate limit state corresponding to progressive collapse.

The requirements at this ultimate limit state are that a reasonable part of the structural system may be allowed to lose its load carrying capacity without causing the remaining part of the structure to collapse.

The safety requirements are expressed as requirements to the safety index $\beta$ as applied shortly after, the accidental load has occurred.

The requirements are satisfied provided $\beta = 2$, when all the load carrying capacities of the structure are utilized.

The structural codes may consider these requirements satisfied if all other requirements can be fulfilled.

# 4. THE METHOD OF PARTIAL COEFFICIENTS

## 4.1 Principles

In order to obtain sufficient safety of a load carrying structure by application of the method of partial coefficients, it is required that the calculated loading effects are less than those that can be sustained by the structure according to an appropriate calculation model for the limit state considered.

The requirements can be expressed through the following conditions:

$$S_d \leq R_d$$

The definition of the symbols is given below:

$S_d$: the calculated load effect at a selected comparable level, e.g., corresponding to the effect on an element, at a cross section or on the material.

$R_d$: the resistance calculated at the same level as $S_d$.

For the definition of $S_d$ and $R_d$, the calculated values $p_d$ for primary load parameters and the calculated values $m_d$ for the primary resistance parameters are used.

Geometrical parameters, $\ell$ should be used with the values prescribed by the design since the partial coefficients to a certain degree cover geometrical uncertainties. Geometrical parameters for which deviations from prescribed values are critical to the resistance of the structure, e.g., eccentricities, should be indicated by their unfavourable tolerance limits.

The structural codes should indicate the most important conditions for the calculation models, including the phenomena for which the models should make allowance.

The structural codes should include acceptable calculation models as well as the sphere within which they are valid.

Design values for loading effects and resistance should be calculated by means of the characteristic values and partial coefficients indicated in sections 4.2, 4.3 and 4.4. The following expressions are used:

$$p_d = \gamma_p \, p_k$$

$$m_d = \frac{m_k}{\gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{m3}}$$

where

$p_k$ is the characteristic value for load (the primary loading parameter).

$m_k$ is the characteristic value for resistance (the primary resistance parameter).

$\gamma_p$ is a partial coefficient which allows for the load combination considered, the coefficient of variation on the loading parameter, the uncertainty in the loading model, and that part of the uncertainty in the calculation model for the determination of the loading effect at the comparable level, which may be assumed to be independent of the structure considered.

$\gamma_{m1}$ is a partial coefficient which takes into account the safety class of the structure and the type of failure.

$\gamma_{m2}$ is a partial coefficient which takes into account the coefficient of variation on the resistance parameter and the fractile selected as characteristic value.

$\gamma_{m3}$ is the partial coefficient which takes into account the uncertainty in the calculation model used for the determination of the resistance on the level of comparison, the uncertainty in the resistance model (e.g. unknown deviations between the controlled substituted parameter and the corresponding relevant parameter in the structure) and the degree of control on site (besides the statistical quality control).

In structural codes it may be convenient to tabulate the product $\gamma_m = \gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{m3}$ and delete the partial coefficients $\gamma_{m1}$, $\gamma_{m2}$ and $\gamma_{m3}$.

In such cases, the design resistance will be given immediately as $m_d = \frac{m_k}{\gamma_m}$.

## 4.2 Characteristic Values

The principles for the definition of characteristic values, $p_k$, on loads are given in the NKB load regulations, chapter 2. For ordinary loads, the characteristic values $p_k$ (code values) are given in chapter 4 of the load regulations.

For resistance parameters the characteristic value is by definition that which has a probability accepted a priori, of nót being attained at a hypothetical unlimited test series (corresponding to a fractile in the distribution of the resistance parameter). This corresponds to a lower characteristic value.

In cases where environmental conditions may cause deviations between the resistance parameters as determined by testing, and the resistance parameters prevailing in the structure, the characteristic values used for the design should be adapted to such deviations.

In most cases the characteristic value for resistance parameters should be given as the 10% fractile. In special cases, other fractiles can be chosen, e.g. the 5%- and the 0.1% fractile.

In cases where an increase in the resistance parameter will act against the safety of the structure, it may be appropriate to use an upper characteristic value. By a similar definition, this is a value which has a probability, accepted a priori, of not being exceeded.

Usually it will not be necessary to account for the fact that an increase in the material parameters may result in a more unfavourable situation for certain parts of a structure or a cross section, since such cases have been accounted for by the partial coefficients.

> Examples of special cases which could use the upper characteristic value are the tensile strength of concrete in connection with the calculation of the minimum tensile reinforcement to ensure against a ductile failure as well as the modulus of elasticity in connection with the calculation of certain restraining forces.

## 4.3 Partial Coefficients for Loads

### 4.3.1. Ultimate Limit States

For the examination of the ultimate limit state, the determined values of $\gamma_p$ given in table 4.3.1a and 4.3.1b are used

| Load Type | Load Combination | | |
|---|---|---|---|
| | I | II | III |
| Dead loads from the mass of the structure | 1.5 | 1.2 | 1.0 |
| Earth pressure | 1.5 | 1.2 | 1.0 |
| Live loads | | | |
| ordinary loads | 1.3 | 1.3 | 1.0 |
| exceptional loads | - | 1.5 | - |
| Accidental loads | - | - | 1.0 |
| Loads caused by deformations | 1.0 | 1.0 | 1.0 |

Table 4.3.1a Partial Coefficient $\gamma_p$

| Safety Class | 3 | 2 | 1 |
|---|---|---|---|
| Dead Loads | 0.9 | 0.95 | 1.0 |

Table 4.3.1b Partial Coefficient $\gamma_p$

The values of the partial coefficient indicated in table 4.3.1b refer only to dead loads (from the mass of the structure). The values should be applied to all such dead loads simultaneously, and only if this results in a more unfavourable loading situation than the values given in table 4.3.1a.

In design for failure due to fatigue, $\gamma_p = 1.0$, and generally only the load combination I is considered.

Concerning current loading cases within the load combinations I, II and III, reference is made to Proposal to Load Regulations.

Besides the actual examination for the given load combinations, the structure should also be examined for the formal loading situation given in the additional loading regulation.

Additional Loading Regulation

This regulation applies to an examination for dead loads from the mass of the structure as the only loading condition.

This rule states $\gamma_p = 1.20$ for some parts of the structure and $\gamma_p = 1.00$ for the remaining parts, which corresponds to the most unfavourable situation.

Provided load combination I pays due attention to the real variation in the dead load of the structure parts, as given in the load regulations, the values of the additional regulation can be reduced. In this case, the values $\gamma_p = 1.10$ and $\gamma_p = 1.00$, respectively, since the additional rule only pays regard to the constant contribution from dead loads and not to the variable contribution.

The values of the partial coefficient for dead loads given in table 4.3.1b are only relevant if these loads have a stabilizing effect. In cases where some building elements are stabilizing whereas others are destabilizing, the values given in table 4.3.1b should be applied simultaneously to the dead loads.

The additional rule should only be used in cases where dead loads are dominating in comparison with live loads.

Among other things, the additional rule should account for the uncertainty with regard to the distribution of cross sectional forces in the structure.

Moreover the additional rule allows variations in dead loads to be disregarded when the load combination is considered.

The additional rule is not applicable for loads originating from earth or other permanent loads.

### 4.3.2. Ultimate limit states corresponding to progressive collapse.

At the examination of the design stage for this limit state the value $\gamma_p = 1.0$ should be used. Usually only load combination I should be considered.

## 4.4 Partial coefficients for resistance.

### 4.4.1 Ultimate limit states

At the examination of this limit state the values of $\gamma_{m1}$, $\gamma_{m2}$ and $\gamma_{m3}$ stated in tables 4.4.1a, b and c should be applied.

In the structural codes those values of $\gamma_{m1}$ which correspond to the safety classes and failure types considered should be selected from table 4.4.1a. From table 4.4.1b the structural codes should select those values of $\gamma_{m2}$ which correspond to the coefficients of variations and fractiles for the resistance parameters.

The structural codes should determine the degree of uncertainty which can be expected in the calculation models mentioned and also which control classes should be applied. The partial coefficient $\gamma_{m3}$ can then be selected from table 4.4.1c.

| Safety class | 3 | 2 | 1 |
|---|---|---|---|
| Failure type III | 1.60 | 1.40 | 1.23 |
| Failure type II | 1.40 | 1.23 | 1.08 |
| Failure type I | 1.23 | 1.08 | 0.95 |

Table 4.4.1a Partial Coefficient $\gamma_{m1}$

| Coefficient of Variation for Resistance Parameter | Fraction | | |
|---|---|---|---|
| | 10% | 5% | 0,1% |
| $V < 0,1$ | 1.10 | 1.05 | 0.96 |
| $0,1 < V < 0,2$ | 1.05 | 1.00 | $0.80^{*}$ |
| $0,2 < V < 0,4$ | 1.15 | 1.05 | $0.73^{*}$ |

*cp. recommendation

Table 4.4.1b Partial Coefficient $\gamma_{m2}$

| The degree of control of materials and execution | Estimated deviation between actual strength and tested strength. Estimated uncertainty in the calculation model. Estimated uncertainty originating in geometry. | | |
|---|---|---|---|
| | great | medium | small |
| Slight | 1.30 | 1.12 | 1.05 |
| Medium | 1.20 | 1.05 | 1.00 |
| Good | 1.12 | 1.00 | 0.95 |

Table 4.4.1c Partial Coefficient $\gamma_{m3}$

The indicated partial coefficients can be applied if the primary resistance corresponds to the strength of materials, and if the primary resistance corresponds to the strength of the structural elements.

In the former case, the material strengths are assumed to be determined and controlled, and the structural design is based on calculations based on the level of the internal forces in the material.

In the latter case, the strengths of the structural elements are assumed to be determined and controlled, and the structural design is based on calculations found in the level of cross sectional forces.

The values marked with * in table 4.4.1b will normally not be used, since, for control reasons, it will not be appropriate to require 0.1% fractile values for large coefficients of variation.

Testing of structure parts will often justify lower values of $\gamma_{m3}$ than the corresponding calculation based on material qualities. Cp. table 4.4.1c.

In many cases the control will be based on substituting qualities.

For primary parameters of resistance at material level, the primary resistance parameter can, for instance, be the tensile strength of concrete, whereas the controlled substituting parameter can be the compressive strength.

For primary resistance parameters in cross sectional forces, the very primary resistance parameter could, for instance, be the strength of á structure part, whereas the controlled substituting parameters, for instance, could be the proportions or the compressive strengths of the concrete and the yield stress of the reinforcement.

### 4.4.2 Ultimate Limit States Corresponding to Progressive Collapse.

An examination of these limit states define $\gamma_m = \gamma_{m1}\ \gamma_{m2}\ \gamma_{m3} = 1.0$.

# 5. STATISTICAL METHOD

## 5.1 Principles

In order to evaluate the safety of a load carrying structure, it is necessary to establish a calculation model expressing the failure criterion in the limit state considered. If the evaluation is not based directly on a test which defines the limit states under certain conditions, a calculation model must be given in the form of a function g of a number of variables

$$x_1, \ldots, x_n, y_1, \ldots, y_r \tag{1}$$

representing primary resistance parameters, primary loading parameters, and primary geometrical parameters. The limit state is expressed in the equation

$$g(x_1, \ldots, x_n, y_1, \ldots, y_r) = 0 \tag{2}$$

whereas the safe state exists provided

$$g(x_1, \ldots, x_n, y_1, \ldots, y_r) > 0 \tag{3}$$

The differentiation in x-variable and y-variable has only been made because of the formulation of the safety method below. The idea is that an increase of the values of the x-variables are favourable to safety, whereas an increase of the values of the y-variables decrease safety as stated below.

The primary resistance-, loading-, and geometrical parameters are assumed to be uncertain quantities $X_1, \ldots, X_n, Y_1, \ldots Y_r$. The inequality

$$g(X_1, \ldots, X_n, Y_1, \ldots, Y_r) \leq 0 \tag{4}$$

then expresses an incidental event, e.g., the fact that the structure is at the limit state or has exceeded it.

This leads to the following safety method where the uncertain quantities are only assumed to be represented by their mean values

$$E(X_1),\ \ldots,\ E(X_n),\ E(Y_1),\ \ldots,\ E(Y_r) \tag{5}$$

and their coefficients of variation

$$V_{X_1},\ \ldots,\ V_{X_n},\ V_{Y_1},\ \ldots,\ V_{Y_r} \tag{6}$$

and that in other respects, they are uncorrelated.

Moreover, g have partial derivatives which are continuous.

The value of safety index $\beta$ expresses the degree of safety of the structure. The requirement to the safety of the structure is satisfied by adding the failure factors $\kappa_x$ and $\kappa_y$ to the mean values of the primary resistance-, loading-, and geometrical parameters so that

$$g(\kappa_{x_1}E(X_1),\ \ldots,\ \kappa_{x_n}E(X_n),\ \kappa_{y_1}E(Y_1),\ldots,\ \kappa_{y_r}E(Y_r)) \geq 0 \tag{7}$$

where

$$\kappa_{x_i} = \exp(-\alpha_{x_i}\ \beta V_{X_i}) \tag{8}$$

$$\kappa_{y_i} = 1 + \alpha_{y_i}\ \beta V_{Y_i} \tag{9}$$

$$\alpha_{x_i} = \frac{\kappa_{x_i} E(X_i) V_{X_i} \frac{\partial g}{\partial x_i}}{\sqrt{\sum_{j=1}^{n} (\kappa_{x_j} E(X_j) V_{X_j} \frac{\partial g}{\partial x_j})^2 + \sum_{j=1}^{r} (E(Y_j) V_{Y_j} \frac{\partial g}{\partial y_j})^2}} \tag{10}$$

$$\alpha_{y_i} = \frac{-E(Y_i) V_{Y_i} \frac{\partial g}{\partial y_i}}{\sqrt{\sum_{j=1}^{n} (\kappa_{x_j} E(X_j) V_{X_j} \frac{\partial g}{\partial x_j})^2 + \sum_{j=1}^{r} (E(Y_j) V_{Y_j} \frac{\partial g}{\partial y_j})^2}} \tag{11}$$

The partially derivatives

$$\frac{\partial g}{\partial x_1}, \ldots, \frac{\partial g}{\partial x_n}, \frac{\partial g}{\partial y_1}, \ldots, \frac{\partial g}{\partial y_r} \tag{12}$$

of the function g should be calculated for

$$\begin{aligned} x_1 &= \kappa_{x_1} E(X_1), \ldots, x_n = \kappa_{x_n} E(X_n), \\ y_1 &= \kappa_{y_1} E(Y_1), \ldots, y_r = \kappa_{y_r} E(Y_r) \end{aligned} \tag{13}$$

The values of $\kappa_{x_i}$, $\kappa_{y_i}$, $\alpha_{x_i}$, $\alpha_{y_i}$ should be determined through an solution by iteration of the equations (7) through (11) using the sign of equation in (7). Concerning the practical calculation, reference is made to the comments to the proposal. If the differentiation in x-variables and y-variables has been correctly made, none of the α-variables will become negative. If the calculations for instance result in $\alpha_{x_i} < 0$, this means that the $x_i$-variable should be changed to a y-variable. If, for instance, the result becomes $\alpha_{y_i} < 0$, the $y_j$-variable should be changed to a x-variable.

Normally, the resistance parameters are x-variable, and the loading parameters y-variable. In certain cases, a loading parameter, for instance dead loads, will act in favour of safety, so that, in this case, the loading parameter should be considered an x-variable.

As mentioned, the indicated expressions assume the uncertain quantities $X_1, \ldots, X_n, Y_1, \ldots, Y_r$ to be mutually uncorrelated. The method can be generalized to correlated uncertain quantities but the procedure will become rather complicated.

In cases where certain variables are much positively correlated, the method can be applied by drawing together such variables to one variable using the relevant deterministic relation between the variables. Such a procedure will be on the safe side.

On the other hand, small correlations between uncertain quantities can be left out of account altogether.

## 5.2 Uncertainty of Primary Loading Parameters and Resistance Parameters.

Uncertainties of primary loading- and resistance parameters are indicated by the coefficients of variation of these parameters.

Concerning the loading parameters, reference is made to the loading regulations.

Regarding resistance parameters, for instance parameters of strength and elasticity, where the coefficients of variation are derived from testing or control, the resulting coefficients of variation can be applied. In cases where such data are not available, the structural codes must indicate which coefficients of variation are applicable for the primary resistance parameters, if it becomes possible to use the statistical method. For the determination of the partial coefficients given in chapter 4, the coefficients of variation for resistance parameters given in table 4.4.1b are used.

## 5.3 Uncertainties of Geometrical Parameters.

In cases where the uncertainty of geometrical parameters has no influence on the resistance of the structure, those uncertainties are included in the coefficient of variation of the judgement factors introduced in par. 5.4. In cases where the uncertainty of geometrical parameters has great influence on the resistance of the structure, the geometrical parameters are considered random variables having coefficients of variation corresponding to the prescribed tolerances.

## 5.4 Other Uncertainties.

In order to allow statistically for uncertainties concerning the calculation- and loading model as well as control and supervision, the judgement factors I, with a mean value equal to 1 and a coefficient of variation $V_I$ are introduced.

The judgement factors are introduced by multiplying the primary resistance- and loading parameters by these factors. The coefficients of variation $V_{M_i}$ and $V_{P_i}$, which are to be applied for primary resistance- and loading parameters, respectively, are given in the expressions below.

$$V_{M_i} = \sqrt{V_{m_i}^2 + V_{I_m}^2},$$

$$V_{P_i} = \sqrt{V_{p_i}^2 + V_{I_p}^2}$$

where

$V_{m_i}$ is the coefficient of variation of the primary resistance parameter

$V_{p_i}$ is the coefficient of variation of the load actions

$V_I$ is the coefficient of variation of the judgement factors

The coefficients of variation $V_{I_m}$ and $V_{I_p}$ cover the following uncertainties:

$V_{I_p}$ uncertainty concerning the load model
uncertainty concerning the calculation model (transformation from load action to load effect, for instance the cross sectional bending moment).

$V_{I_m}$ uncertainty concerning the resistance model (strength variation over cross section, short termed - long termed strength)
uncertainty concerning the calculation model (i.e., uncertainty in the determination of resistance at the cross sectional level from the primary resistance parameters).
uncertainty concerning geometrical parameters not critical towards the resistance of the structure
uncertainty concerning control and supervision on site.

The coefficients of variation $V_{I_p}$ of the loads are determined as given in table 5.4a unless differing verifications are presented.

If the coefficient of variation, $V_p$, of the load action includes the uncertainty concerning the load model, the coefficient of variation, $V_{I_p}$, shall not be applied.

The reason for excluding the coefficient of variation $V_{I_p}$ of accidental loads is the lack of statistical material.

| Load Action | Coefficient of Variation |
|---|---|
| Dead loads from the mass of the structure | 0.05 |
| Earth pressure | 0.15 |
| Live loads | 0.30 |
| Accidental loads | - |

Table 5.4a Coefficient of Variation $V_{I_p}$

A precise indication of the coefficients of variation $V_{I_m}$ of the primary resistance parameters must be given in the structural codes, if these are to allow the application of the statistical method.

For their determination, the values given in table 5.4b for the coefficient of variation $V_{I_m}$, can be used.

| The degree of control of materials and execution | Estimated deviation between actual strength and tested strength. Estimated uncertainty in the calculation model. Estimated uncertainty originating in geometry. | | |
|---|---|---|---|
| | Great | Medium | Small |
| Slight | 0.20 | 0.15 | 0.12 |
| Medium | 0.17 | 0.12 | 0.10 |
| Good | 0.15 | 0.10 | 0.07 |

Table 5.4b Coefficient of Variation $V_{I_m}$

The values of the coefficients of variation $V_{I_m}$ and $V_{I_p}$ of resistance parameters and load actions, respectively, have been used for the calculation of the partial coefficients of section 4.4.

Unless different proof can be presented, the statistical method shall not allow any smaller values of these coefficients of variation to be used than stated in table 5.4a and 5.4b.

When determining the uncertainty concerning the calculation model, the geometry, and the deviation between the measured resistance parameter and reality, the following guide lines should be applied.

If the uncertainty concerning geometry is critical to the resistance of the structure, this uncertainty should be taken into account as stated in section 5.3. If not, the uncertainty originating in geometry can be assumed to be small.

The deviation between the measured resistance parameter and the actual parameter should be compared to the influence of the deviation on the resistance. If this deviation results in a substantial uncertainty of the resistance, and if the uncertainty of the calculation model is not small, the values in column "Great" should be applied, whereas the values of column "Medium" should be applied when the uncertainty of the calculation model is small.
If the uncertainty of the calculation model is great and the influence of the deviation between the measured resistance parameter and the actual resistance parameter not is small, the values of column "Great" are used, whereas the values of column "Medium" are applied if the influence of the mentioned deviation is small.

The application of the values of column "Small" assume the influence of all uncertainties on the resistance to be small.

If none of the mentioned uncertainties influences the resistance to any particular degree, and if they are not all small, the values of column "Medium" should be applied.

## 5.5 Combination of Loads.

Concerning the combination of loads, reference is made to "Proposal to Loading Regulations".

## 5.6 Formulation of Requirements to the Resistance Parameters.

The requirements to the resistance parameters when designing statistically, are given in the form of a characteristic value in the following way:

From the mean values E(m) and the coefficient of variation $V_m$ used in the calculations, a characteristic value is given, determined as

$$m_k = E(m)\ \exp(-kV_m)$$

where the coefficient k corresponds to a given fractile value in the normal distribution for an infinite number of tests.

# 6. CONTROL OF CHARACTERISTIC VALUES

The characteristic value of the resistance parameters $m_k$ are normally controlled in the following way.

From the test results $m_i$, the observed (empirical) mean value $\bar{m}$ is determined as

$$\bar{m} = \frac{1}{n} \sum_{i=1}^{n} m_i$$

and the observed (empirical) deviation s as

$$s^2 = \frac{1}{n-1} \sum_{i=1}^{n} (m_i - \bar{m})^2$$

The requirement to the prescribed characteristic value $m_k$ can be considered filled if

$$m_k \leq \bar{m} \exp(-k \frac{s}{\bar{m}})$$

For $k \frac{s}{\bar{m}} \leq 0{,}25$ the following expression can also be used

$$m_k \leq \bar{m} - ks$$

The coefficient k, which depends upon the number of tests and the fractile value, is given in table 6.1.

The above Bayesian procedure for the control of the characteristic value is applied if, prior to the testing, there is no knowledge of neither mean value nor deviation of the resistance parameter to be controlled. In cases where prior knowledge does exist, it can be taken into account by means of the statistical Bayesian procedure.

The control of the characteristic value according to the above procedure will sometimes be unsuitable for test designs. In such cases other procedures will be applicable, if it can be verified that such other procedures make the same requirements to the structure as given in chapter 3.

| Number of Observations (n) | Fraction | | |
|---|---|---|---|
| | 10% | 5% | 0,1% |
| 3 | 5,02 | 10,03 | 521 |
| 4 | 2,58 | 4,00 | 30,7 |
| 5 | 2,08 | 2,98 | 12,9 |
| 6 | 1,85 | 2,58 | 8,66 |
| 7 | 1,73 | 2,36 | 6,87 |
| 8 | 1,65 | 2,23 | 5,92 |
| 9 | 1,59 | 2,14 | 5,39 |
| 10 | 1,56 | 2,07 | 5,00 |
| 15 | 1,45 | 1,90 | 4,10 |
| 20 | 1,39 | 1,82 | 3,83 |
| 30 | 1,36 | 1,76 | 3,53 |
| 40 | 1,34 | 1,73 | 3,41 |
| 50 | 1,33 | 1,71 | 3,34 |
| 100 | 1,30 | 1,68 | 3,21 |
| $\infty$ | 1,28 | 1,65 | 3,09 |

Table 6.1 The Coefficient k

The control of the characteristic value of the resistance parameters, $m_k$, can in cases with a known deviation, D(m), be made according to the following guidelines.

The requirement to the prescribed characteristic value, $m_k$, can be considered fulfilled if

$$m_k \leq \bar{m} \exp\left(-k \sqrt{1 + \frac{1}{n}} \cdot \frac{D(m)}{\bar{m}}\right)$$

where the designations are the same as given in the code.

For $k \sqrt{1 + \frac{1}{n}} \cdot \frac{D(m)}{\bar{m}} \leq 0{,}25$ the following expression also applies

$$m_k \leq \bar{m} - k \sqrt{1 + \frac{1}{n}} D(m)$$

The value of k is seen in table 6.1 for $n = \infty$.

A diffuse distribution a priori for the mean value of the logaritm of the resistance parameter is assumed.

The criteria:

$$m_k \leq \bar{m} \exp(-k \frac{s}{\bar{m}}) \qquad \text{(code)}$$

$$m_k \leq \bar{m} \exp(-k \sqrt{1 + \frac{1}{n}} \frac{D(m)}{\bar{m}}) \qquad \text{(recommendation)}$$

are in accordance with the logarithmic normal distribution if an observation series consisting of the logarithm of the test results is used for the calculation of $\bar{m}$ and s rather than the test results directly. Instead of defining $m_k$ as the prescribed characteristic value, $m_k$ should be the logarithm of this value.

The Bayesian procedure makes it possible to use any prior knowledge of the resistance parameters in excess of what has been assumed above.

PAPER 18

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH
STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# COMMENTS TO PROPOSAL FOR SAFETY CODES FOR LOAD-CARRYING STRUCTURES

NORDIC COMMITTEE FOR BUILDING REGULATIONS

PARIS - FEBRUARY 1975

NORDIC COMMITTEE FOR BUILDING REGULATIONS (NKB)
Sub-Committee on Structural Safety

Comments to

# Proposal for Safety Codes for Load-Carrying Structures

24/10 1974

CONTENTS

## PREFACE

The following comments to the proposal to safety codes are directly related to the separately written code and recommendation on safety methods. Reference is made to that preface.

The comments have been split into the same sections as the recommendations and primarily deal with reasons and comments to the suggested principles as well as the calculation model for the given values.

## K1. INTRODUCTION

The international developments are the reasons for the principles and methods used for the definition of the regulations.

It therefore also seems evident that the partial coefficient method should be selected as one of the methods to be used for the evaluation of the safety of a structure, since the general principles of the partial coefficient method by ISO has been set up as international standard.

Several places, however, methods are applied which in statistically more correct ways allow for the various influencing factors.

It has therefore been decided that the safety regulations should include a partial coefficient method as well as a method based on statistical principles.

The reasons are given below. There exists a demand for a simple method which is applicable to most structures. This demand is met in the partial coefficient method. On the other hand, a more correctly developed method offers greater possibilities for future developments.

The two methods have a very close connection, however, the values of the partial coefficient method being selected so that this method can be considered an approximation to the statistical method, cp. K4.

Besides the two given methods, it has now been made possible to apply other statistical method provided such methods meet certain requirements.

Such requirements include the formal probability of failure $p_f$, which is a value based on technical calculations. The formal probability of failure cannot be expected to be equal to the "real probability of failure" of a structure, which can only be defined and determined with very great difficulty.

As mentioned in the proposal, the introduction of other statistical methods requires proof that the dimensions calculated according to the method will at the most yield the values given for the formal probability of failure $p_f$, assuming certain types of distribution for the parameters. The demand to these types of distribution is only valid in cases of comparable calculations and should not be understood as a demand to the distributions for the parameters in the other statistical method.

Moreover, the values for the formal probability of failure $p_f$ only serve as basis for the comparison and need not be included in the other statistical method. The parameters of the other statistical method should only be set up so as to fill the requested demands.

An alternative judgement of the other statistical method should prove that the dimensions calculated in a series of frequently used examples through application of the method do not result in smaller dimensions than corresponding calculations according to the method given in chapter 5 of the proposal.

## K3. PRINCIPLES FOR THE EVALUATION OF STRUCTURAL SAFETY

### K3.1 <u>Safety Classes.</u>

The reason for the introduction of various safety classes are the requirements placed for the safety against failure of a structure which should be proportioned to the probable consequences of such a failure.

In many cases, structures have up to now been subject to the same regulations, independent of whether the structure is of vital importance, or whether it is a less important secondary structure.

There are two main reasons for the division made in the codes into three safety classes, which primarily look into the risk of personal injury. Moreover, regard has been paid to the overall importance of the structure, including for instance social and economic considerations.

This division into two groups, personal injury being the primary requirement, is more rigorous demand than given in the ISO standard 2394, which apparently seeks a complex optimization, in which the personal risk is a minor factor.

The following comments to a more refined evaluation of the classification of each structure into safety calsses may be of importance.

Concerning the risk for personal injury:

- number of persons that could get involved in a failure
- relative period of time during which these persons are exposed to the risk in connection with the construction
- interaction between a possible extreme loading and the presence of persons
- conscious or non-conscious expectance or acceptance by the employed persons of the degree of risk under voluntary or extreme circumstances.

Concerning other considerations to the importance of the structure:

- building costs compared to required lifetime
- interest and running expenses
- reconstruction costs and any loss of income in cases of failure
- any other direct or indirect financial losses in cases of failure.

## K3.2 Limit States.

Within the three defined groups of limit states, the serviceability limit state and the ultimate limit state correspond exactly to the states determined by the ISO.

The limit state of progressive collapse which should mainly be considered in structures in the most rigorous safety class, represents an extension as compared to the ISO standard. With the extension, it is possible to accept local failures for certain loads (accidental loads), which act very heavily, and instead certain safety requirements are made against the collapse of the entire structure.

It has been discussed to differentiate between ultimate limit states and limit states of progressive collapse in cases of actions from live loads. This would correspond to requirements to live loads which would be dependent upon whether or not a failure of part of a structure would be likely to lead to failure of greater parts of the structure.

Such a procedure must be considered correct in principle, but it does not seem to be sufficiently justifiable because of the large calculating complications that would arise as compared to previous practice.

## K3.3 Requirements at the Serviceability Limit States.

General requirements at the serviceability limit states are not made, because the committee does not consider it possible to make such requirements. The requirements which could be made to a structure at the serviceability limit state would depend upon each structural code and also upon each structure and surrounding area. The requirements at the serviceability limit state should also be related to the calculation model be used for the evaluation of the requirements. Sufficient requirements at the serviceability limit state are, however, expected to be made in the structural code. Any more specific requirements to the serviceability limit state must be set up by structural engineer and owner.

## K3.4 Requirements at Ultimate Limit States.

The requirements at the ultimate limit state have been expressed as values to safety index $\beta$ related to the statistical method. The reason is a simpler application of the statistical method for the revision of the partial coefficients as further knowledge of parameters of resistance and loadings becomes available. The values have been established in such a way that the dimensions used through the application of the statistical method to a series of frequently used formulas, on an average yield the same dimensions as existing structural codes.

It has been considered reasonable to establish the requirements so that structures of the most rigorous safety class will generally comply with present codes. For less rigorous safety classes, this generally means more economical use of materials and less costly structures. In the most rigorous safety classes the requirements will, however, in some cases result in more costly structures. This is particularly the case in structures which are assumed to be acted upon by accidental loads.

The safety requirements depend upon the type of failure, i.e., the requirements are less rigorous in ductile failures of large deformation capacity than in failure of small deformation capacity. The reasons are given below.

If some areas or parts of the structure enter into the ultimate limit state, a decisive ductility will influence it favourably, resulting in a redistribution of the cross-sectional forces. This means that the structure as a whole will not necessarily collapse. If only small additional deformations can be sustained after the ultimate limit state has been reached the structure may collapse. In cases of brittle material, a commenced cross-sectional failure will often lead to a failure of the entire cross section, after which the member fails and the entire structure may collapse.

The ductile failures will often be noticed so that persons can get out of the way or the load can be reduced.

Further, a ductile structure will be in a substantially better position to resist impact loadings than a structure of small deformation capacity and of the same strength to slowly added loads.

In many cases, the ability of a staticly undecided structure to distribute the cross-sectional forces will be utilized in the way of hinges. Its ductile character may, however, still result in the other mentioned safety advantages as compared to failures of brittle rupture.

K3.5 Requirements at Ultimate Limit States Corresponding to Progressive Collapse.

The requirements at the ultimate limit states corresponding to progressive collapse have been expressed in a single value of $\beta$. The reasons are that the uncertainties of calculating at this limit state must be considered great, and that the knowledge of the structural consequences have not been sufficiently discussed to justify a differentiation. The apparent low value of $\beta$ is due to the fact that the value refers to the rare local failure situation resulting from an accidental load, and also that reference is made to such a brief period of time, for instance one week, that a bracing or the like can be established.

If, despite lacking knowledge of the structural consequences, a value of $\beta$ at this ultimate limit state has been given, it is due to the fact that the statistical method should be applicable. The value $\beta=2$ corresponds to the remaining part of the structure being examined for greatly reduced loads.

## K4. THE METHOD OF PARTIAL COEFFICIENTS

The partial coefficients specified in "Proposal for safety codes for load-carrying structures" are determined by use of the statistical method described in chapter 5.

The following calculation model, which is a typical failure criterion, is expressed as

$$R - (S_g + S_p + S_q) = 0 \tag{1}$$

The definition of the symbols is given below

$R$ : the resistance, for instance a yield bending moment

$S_g$: the load effect from dead loads, for instance a bending moment

$S_p$: the load effects from two different live

$S_q$: loads.

It is assumed, that the resistance, $R$, and the load effects $S_g$, $S_p$ and $S_q$ can be expressed by the primary resistance parameter, $m$, and the primary loading parameters, $g$, $p$ and $q$, in the following way

$$R = c_m m$$

$$S_g = c_g g$$

$$S_p = c_p p$$

$$S_q = c_q q$$

In these expressions, $c_m$, $c_g$, $c_p$, and $c_q$ are the constants, which include geometrical parameters.

The failure criterion (1) may under these assumptions be written as

$$c_m m = c_g g + c_p p + c_q q \tag{2}$$

or

$$M = G + P + Q \tag{3}$$

where

$M = c_m m$

$G = c_g g$

$P = c_p p$

$Q = c_q q$

Uncertainties

The uncertainty concerning the calculation model and the stochastic variations in the primary resistance parameter and the primary loading parameters are specified by the coefficients of variation, V.

The uncertainty in connection with the geometrical parameters is assumed to be included in the coefficients of variation of the judgement factors, and the geometrical parameters are assumed not to be critical to the resistance, R.

Determination of The Partial Coefficients

For given values of the load effects, G, P, and Q, the mean value of the resistance, E(M), is determined by using the statistical method. In accordance with the method of partial coefficients the following expression must be valid

$$c_m m_k/\gamma_m = c_g g_k \gamma_g + c_p p_k \gamma_p + c_q q_k \gamma_q \tag{4}$$

where k refer to the characteristic values.

Now the characteristic values may be expressed by the mean values, E(•), and the coefficients of variation by the equations

$$m_k = E(m)\ \exp(-k_m V_m)$$

$$g_k = E(g)\ (1 + k_g V_g)$$

$$p_k = E(p)\ (1 + k_p V_p)$$

$$q_k = E(q)\ (1 + k_q V_q)$$

Equation (4) may then be rewritten as

$$\frac{E(M)}{\gamma_m\ \exp(k_m V_m)} = \gamma_g E(G) + \gamma_p E(P)(1+k_p V_p) + \gamma_q E(Q)(1+k_q V_q) \qquad (5)$$

assuming $k_g = 0$.

From equation (5) it appears that the mean value of the resistance E(M) for given values of the partial coefficients, the coefficients of variation, and the coefficients k is a linear function of the load effects, E(G), E(P) and E(Q).

Calculations

Figure 1 - 20 shows how the mean value of the resistance varies with the mean values of the load effects corresponding to different values of safety index $\beta$ and of the coefficients of variation.

The values of the safety index, the mean values of the load effects and the coefficients of variation of the resistance are given in the figures. The values of the coefficients of variations on the loading parameters are stated in tabel K4 given below.

For the determination of the mean value of the resistance according to the method of partial coefficients, the value of the partial coefficients specified in table 4.3.1a and b on the loading parameters and the values specified in table 4.4.1a, b and c on the resistance are used. The table designation refers to "Proposal for Safety Codes for Load Carrying Structures".

Table 1 - 15 shows how the partial coefficient $\gamma_{m1}$ varies, if the method of partial coefficients and the statistical method are to yield identical results. For these calculations, the values of the partial coefficients $\gamma_{m2}$ and $\gamma_{m3}$ are chosen in accordance with the values specified in table 4.4.1b and c and the values of the partial coefficients on the loading parameters as specified in table 4.3.1a and b in "Proposal for Safety Codes for Load Carrying Structures". The values of the safety index, the mean values of the load effects and the coefficients of variation of the resistance are given in the tables. The values of the coefficients of variation of the loading parameters are stated in table K4.

For the determination of the mean value of the resistance according to both the statistical method and the method of partial coefficients, the loads are combined as prescribed in "Proposal for Load Regulations".

| Coefficients of Variation of Loading Parameters | | | | | |
|---|---|---|---|---|---|
| $V_p$ | $V_{I_p}$ | $V_q$ | $V_{I_q}$ | $V_g$ | $V_{I_g}$ |
| 0.40 | 0.30 | 0.40 | 0.30 | 0.05 | 0.05 |

Table K4. Coefficients of Variation of Loading Parameters.

The loads used for the calculation of the mean value of the resistance shown in figure 1 - 20 are assumed to be dead loads or longterm loads so that the mean values and the coefficients of variation of the "annual maximum load" are equal to the "short time maximum load".

In determining the values of the partial coefficient $\gamma_{ml}$ given in table 1 - 15 loads with different duration are combined. In the tables the following notations are used

long term load or dead load: A = 0
not short term load: A = 0.5
short term load: A = 1

The mean values and the coefficients of variation, given in the tables correspond to the "shorttimemaximum load".

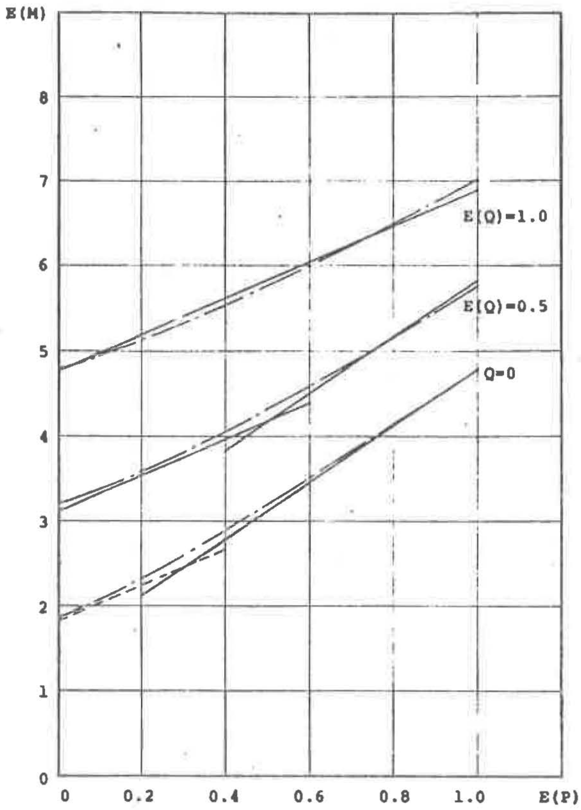


Figure 1 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: β = 3.25

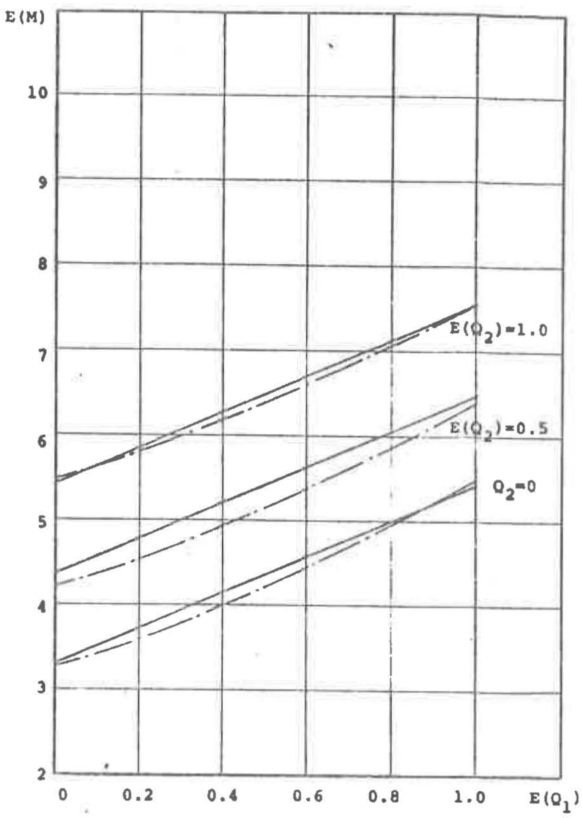


Figure 2 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: β = 3.25

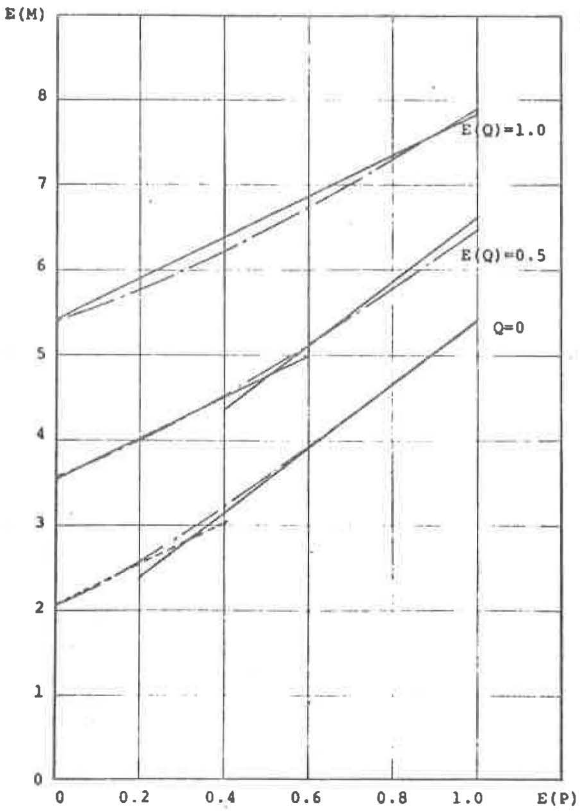


Figure 3 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: β = 3.75

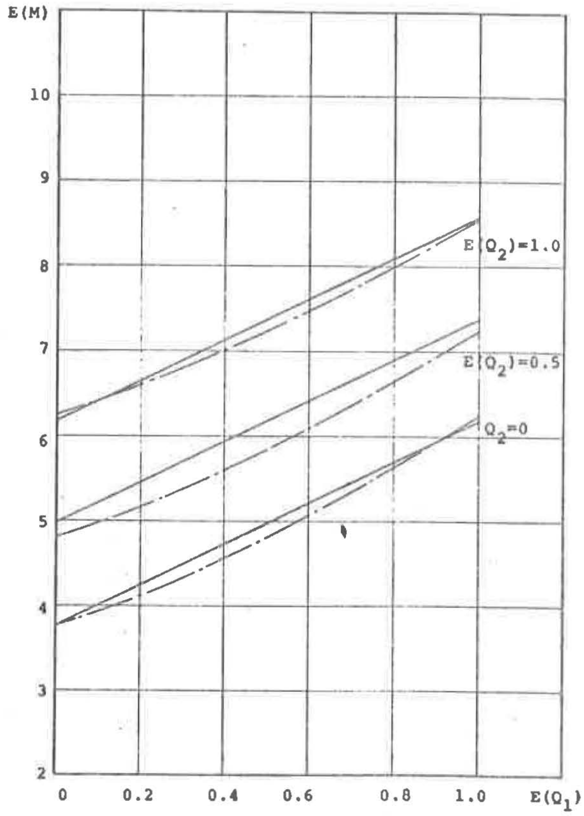


Figure 4 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: β = 3.75

The statistical method. —— - ——
The method of partial coefficients
Load combination I - - - - - - -
Load combination II ————

Coefficients of variation:
$V_m$ = 0.15
$V_{I_m}$ = 0.10

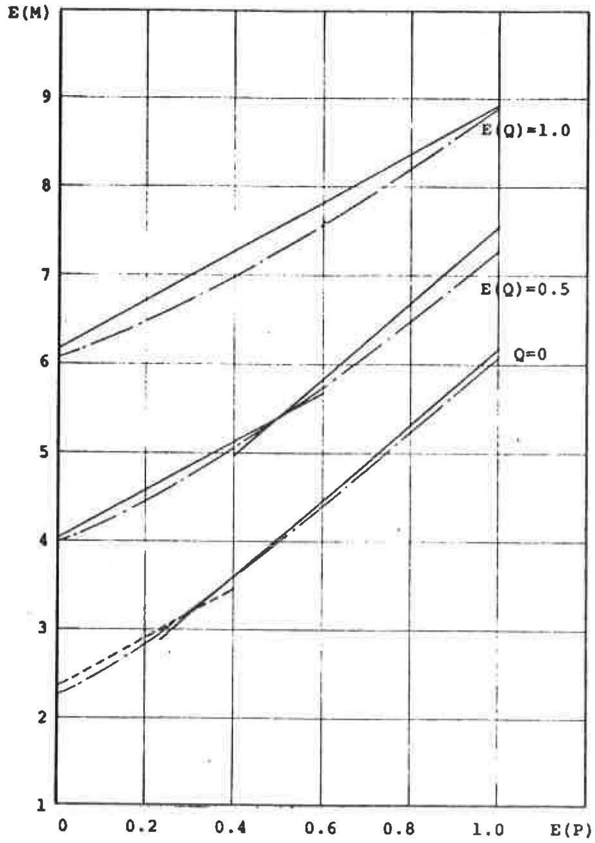


Figure 5 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: $\beta$ = 4.25

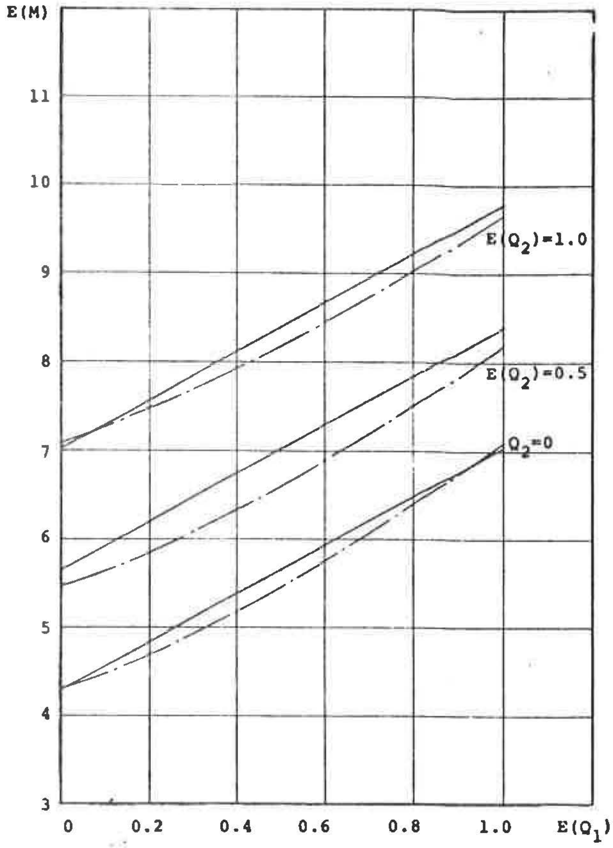


Figure 6 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: $\beta$ = 4.25

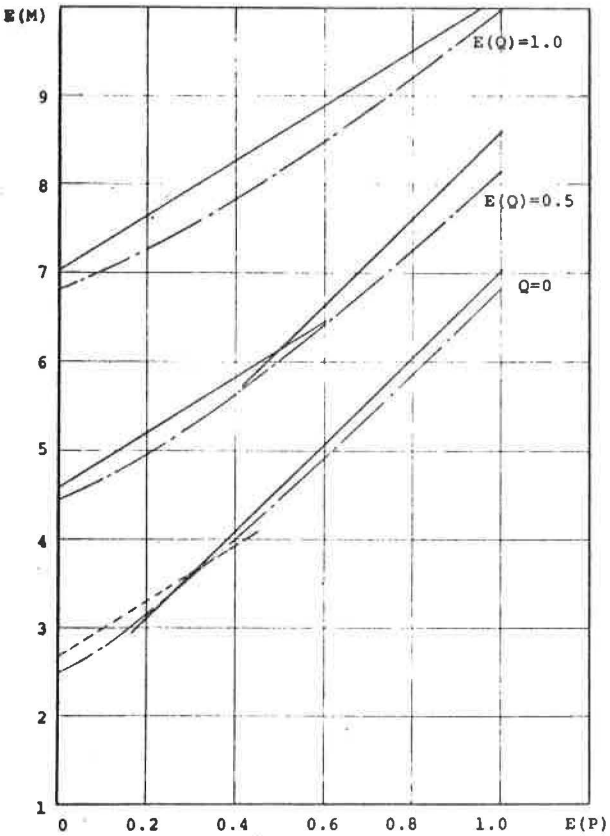


Figure 7 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: $\beta$ = 4.75

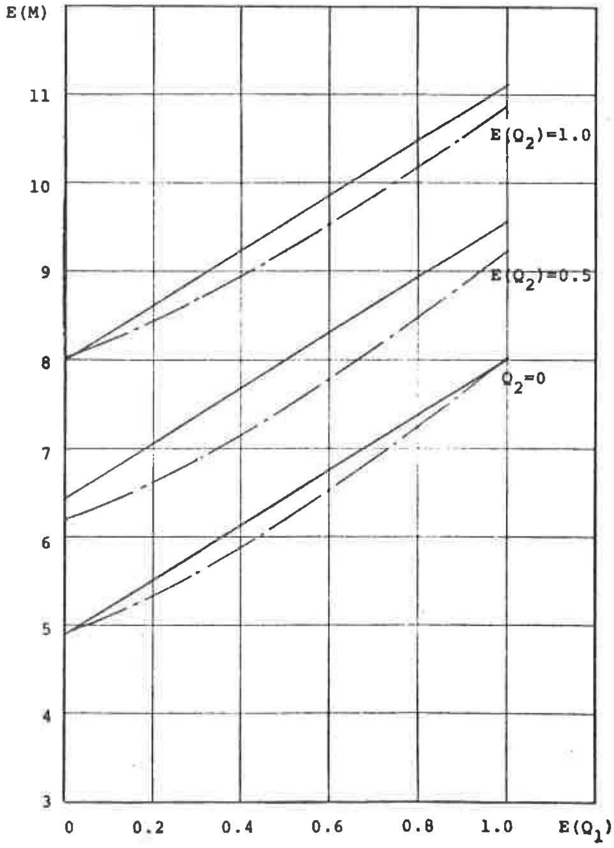


Figure 8 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: $\beta$ = 4.75

The statistical method ——-·-——
The method of partial coefficients
Load combination I - - - - - - - -
Load combination II ————————

Coefficients of variation:

$V_m$ = 0.15

$V_{I_m}$ = 0.10

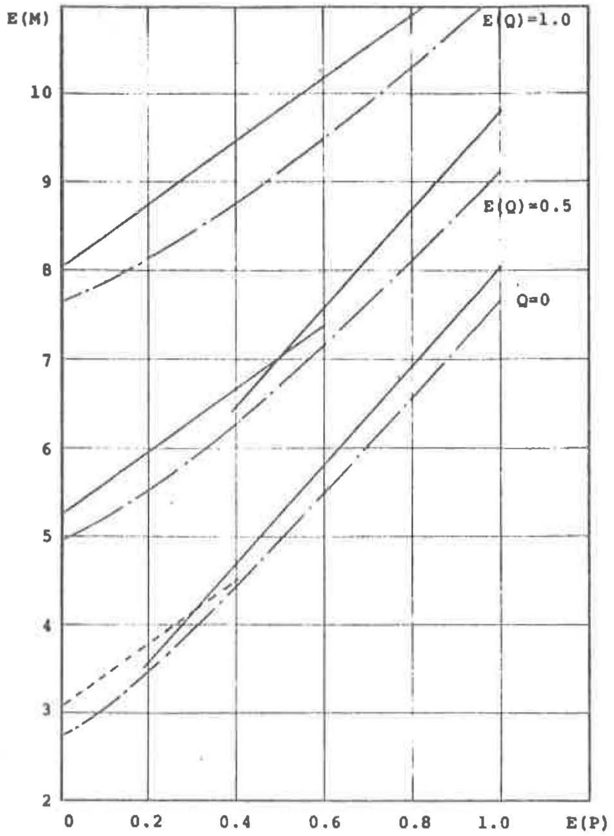


Figure 9 FAILURE CRITERIA: $M=P+Q+G$
$E(G)=1$
Safety index: $\beta = 5.25$

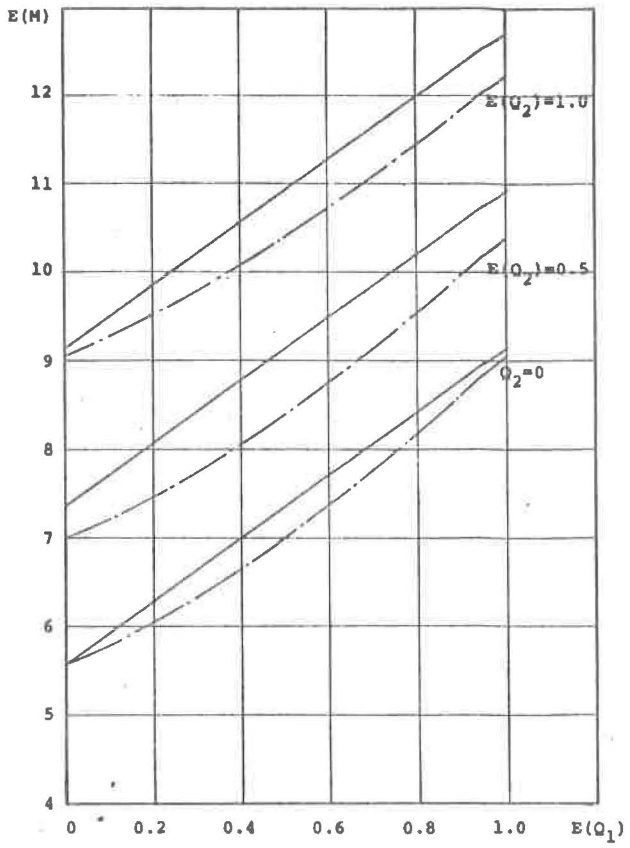


Figure 10 FAILURE CRITERIA: $M=P+Q_1+Q_2$
$E(P)=1$
Safety index: $\beta = 5.25$

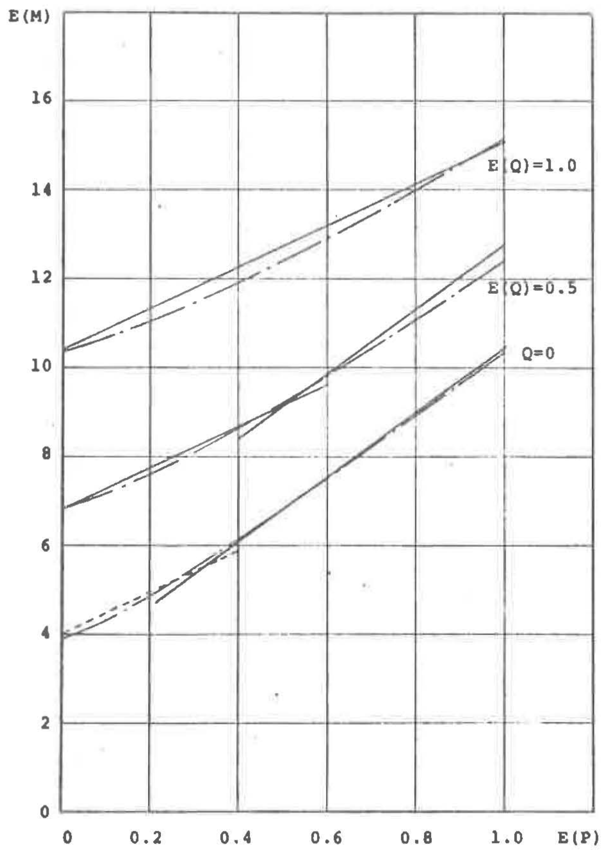


Figure 11 FAILURE CRITERIA: $M=P+Q+G$
$E(G)=1$
Safety index: $\beta = 5.25$

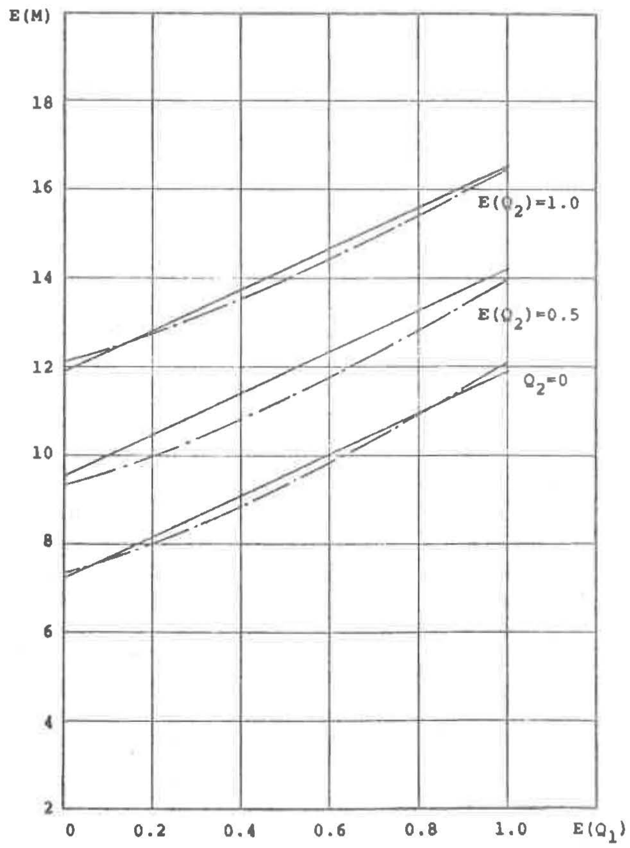


Figure 12 FAILURE CRITERIA: $M=P+Q_1+Q_2$
$E(P)=1$
Safety index: $\beta = 5.25$

The statistical method. ——- - ——
The method of partial coefficients.
Load combination I - - - - - - -
Load combination II ————

<u>Coefficients of variation:</u>

$V_m$ = 0.15

$V_{I_m}$ = 0.10 (figure 9 and 10)

$V_{\tau}$ = 0.20 (figure 11 and 12)

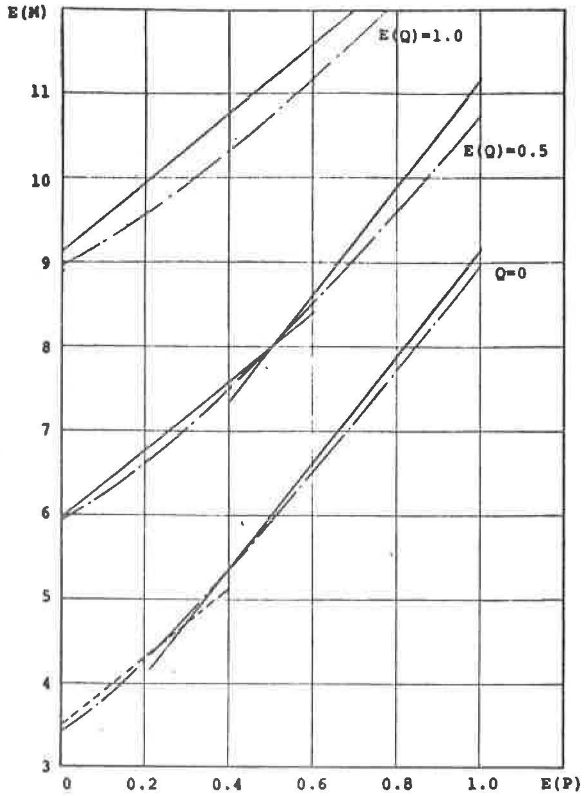


Figure 13 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: β = 4.75

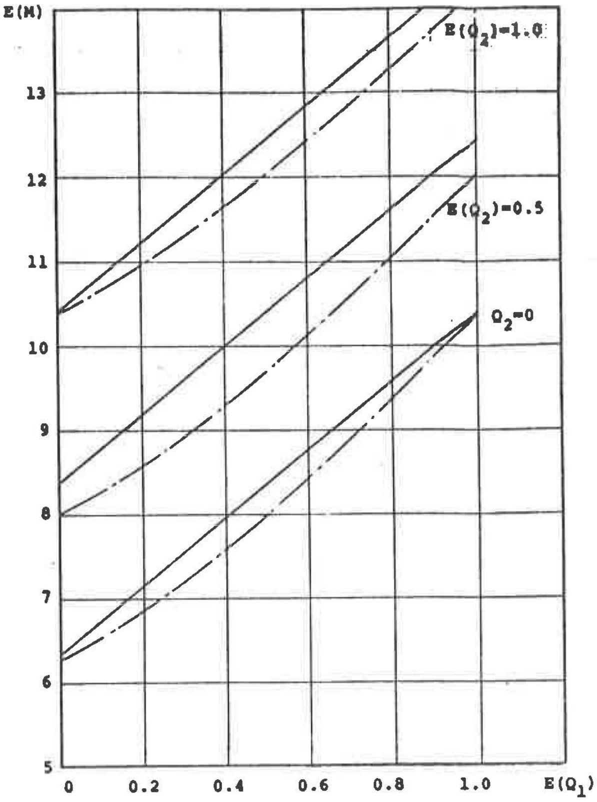


Figure 14 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: β = 4.75

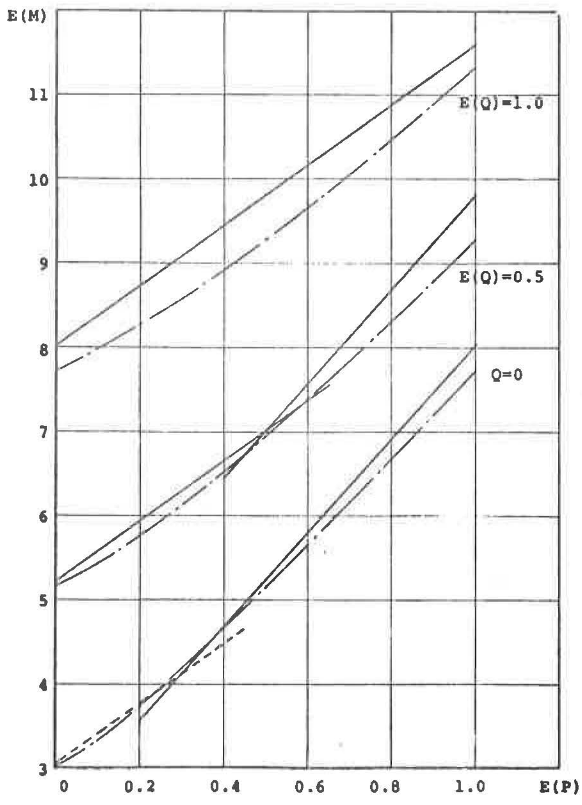


Figure 15 FAILURE CRITERIA: M=P+Q+G
E(G)=1
Safety index: β = 4.25

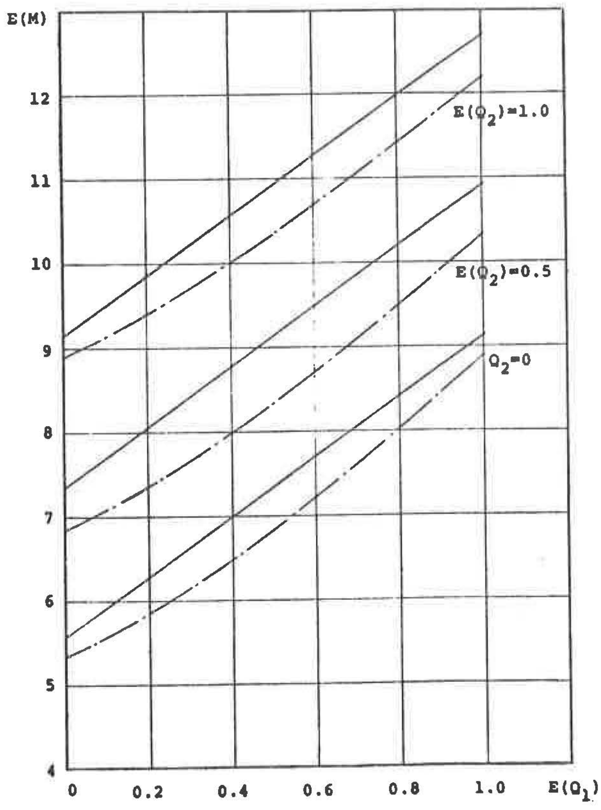


Figure 16 FAILURE CRITERIA: $M=P+Q_1+Q_2$
E(P)=1
Safety index: β = 4.25

The statistical method: —— · ——
The method of partial coefficients:
Load combination I - - - - - - -
Load combination II —·——

Coefficients of variation:
$V_m$ = 0.15
$V_{I_m}$ = 0.20

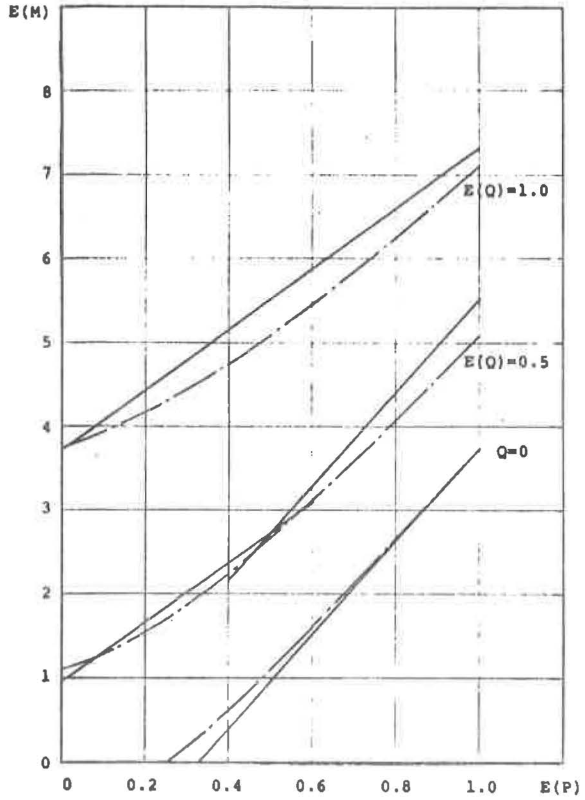


Figure 17 FAILURE CRITERIA: M=P+Q+G
E(G)=-1
Safety index: $\beta$ = 5.25
Partial coefficient: $\gamma_g$ = 0.9

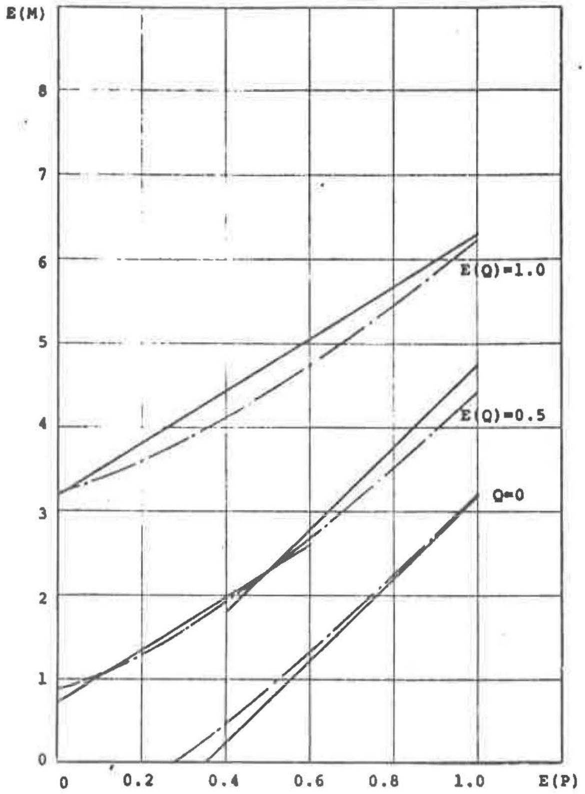


Figure 18 FAILURE CRITERIA: M=P+Q+G
E(G)=-1
Safety index: $\beta$ = 4.75
Partial coefficient: $\gamma_g$ = 0.95

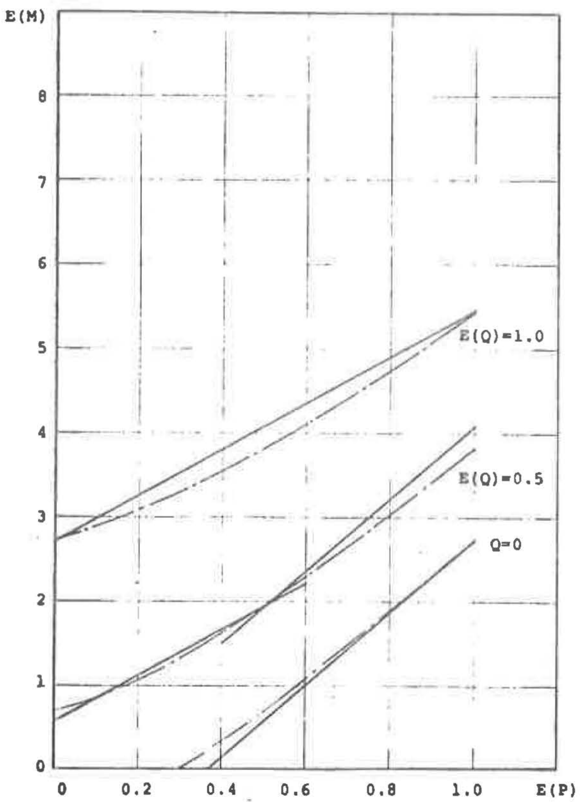


Figure 19 FAILURE CRITERIA: M=P+Q+G
E(G)=-1
Safety index: $\beta$ = 4.25
Partial coefficient: $\gamma_g$= 1.0

The statistical method.—— - ——
The method of partial coefficients.
Load combination I - - - - - - - -
Load combination II - ——————

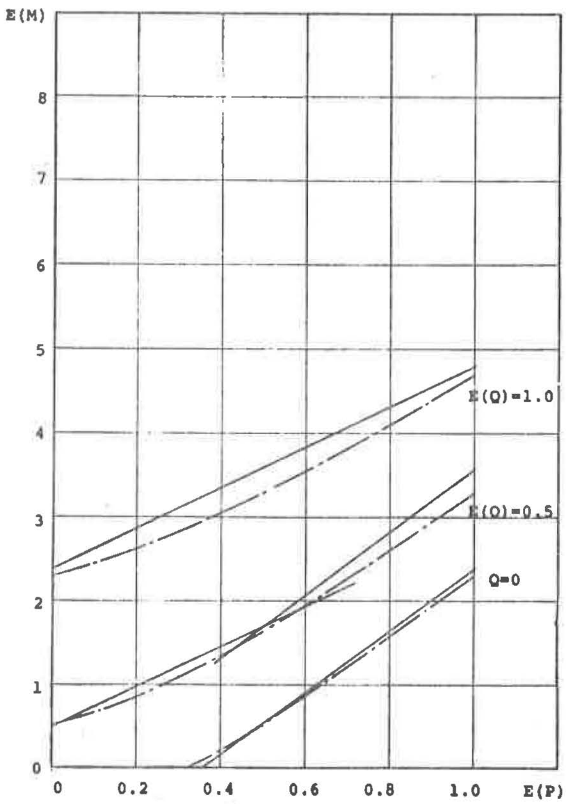


Figure 20 FAILURE CRITERIA: M=P+Q+G
E(G)=-1
Safety index: $\beta$ = 3.25
Partial coefficient: $\gamma$ = 1.0

Coefficients of variation:
$V_m$ = 0.15
$V_{I_m}$ = 0.10

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 0.974 | 0.960 | 0.931 | 0.985 | 0.959 | 0.918 | 0.965 | 0.933 | 0.888 |
| 0.10 | 1.005 | 0.989 | 0.958 | 0.973 | 0.948 | 0.908 | 0.956 | 0.925 | 0.881 |
| 0.20 | 1.045 | 1.024 | 0.989 | 0.969 | 0.944 | 0.905 | 0.950 | 0.919 | 0.876 |
| 0.30 | 1.018 | 0.995 | 0.958 | 0.976 | 0.951 | 0.912 | 0.947 | 0.917 | 0.874 |
| 0.40 | 1.004 | 0.978 | 0.939 | 1.003 | 0.976 | 0.935 | 0.946 | 0.916 | 0.874 |
| 0.50 | 0.995 | 0.967 | 0.927 | 0.989 | 0.962 | 0.921 | 0.947 | 0.918 | 0.875 |
| 0.60 | 0.989 | 0.960 | 0.918 | 0.978 | 0.950 | 0.909 | 0.956 | 0.927 | 0.884 |
| 0.70 | 0.985 | 0.955 | 0.912 | 0.970 | 0.942 | 0.901 | 0.973 | 0.943 | 0.900 |
| 0.80 | 0.982 | 0.951 | 0.907 | 0.965 | 0.936 | 0.894 | 0.991 | 0.960 | 0.916 |
| 0.90 | 0.980 | 0.949 | 0.904 | 0.961 | 0.931 | 0.889 | 0.985 | 0.954 | 0.909 |
| 1.00 | 0.979 | 0.946 | 0.901 | 0.958 | 0.928 | 0.885 | 0.977 | 0.946 | 0.902 |

TABLE 1 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.25 E(Q2)= 0.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 0.977 | 0.950 | 0.911 | 0.985 | 0.958 | 0.917 | 0.947 | 0.918 | 0.875 |
| 0.10 | 0.985 | 0.959 | 0.920 | 0.975 | 0.949 | 0.909 | 0.940 | 0.911 | 0.870 |
| 0.20 | 1.001 | 0.976 | 0.936 | 0.970 | 0.944 | 0.905 | 0.936 | 0.907 | 0.866 |
| 0.30 | 1.024 | 0.997 | 0.956 | 0.973 | 0.947 | 0.908 | 0.933 | 0.905 | 0.865 |
| 0.40 | 1.005 | 0.978 | 0.938 | 0.990 | 0.964 | 0.924 | 0.933 | 0.905 | 0.864 |
| 0.50 | 0.989 | 0.962 | 0.921 | 0.976 | 0.950 | 0.911 | 0.933 | 0.906 | 0.865 |
| 0.60 | 0.978 | 0.950 | 0.909 | 0.965 | 0.938 | 0.899 | 0.941 | 0.913 | 0.873 |
| 0.70 | 0.970 | 0.942 | 0.901 | 0.956 | 0.930 | 0.890 | 0.955 | 0.927 | 0.886 |
| 0.80 | 0.965 | 0.936 | 0.894 | 0.950 | 0.923 | 0.883 | 0.970 | 0.941 | 0.899 |
| 0.90 | 0.961 | 0.931 | 0.889 | 0.946 | 0.918 | 0.878 | 0.964 | 0.936 | 0.894 |
| 1.00 | 0.958 | 0.928 | 0.885 | 0.942 | 0.914 | 0.874 | 0.958 | 0.929 | 0.887 |

TABLE 2 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.25 E(Q2)= 0.50 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 0.952 | 0.920 | 0.875 | 0.968 | 0.937 | 0.894 | 0.972 | 0.941 | 0.896 |
| 0.10 | 0.955 | 0.924 | 0.880 | 0.959 | 0.930 | 0.888 | 0.965 | 0.934 | 0.891 |
| 0.20 | 0.962 | 0.931 | 0.888 | 0.954 | 0.925 | 0.884 | 0.960 | 0.930 | 0.887 |
| 0.30 | 0.972 | 0.942 | 0.898 | 0.955 | 0.926 | 0.885 | 0.956 | 0.927 | 0.884 |
| 0.40 | 0.984 | 0.954 | 0.910 | 0.965 | 0.937 | 0.895 | 0.954 | 0.925 | 0.883 |
| 0.50 | 0.998 | 0.968 | 0.923 | 0.977 | 0.948 | 0.906 | 0.953 | 0.924 | 0.882 |
| 0.60 | 1.014 | 0.983 | 0.938 | 0.989 | 0.960 | 0.918 | 0.958 | 0.929 | 0.888 |
| 0.70 | 1.006 | 0.975 | 0.930 | 0.983 | 0.954 | 0.911 | 0.969 | 0.939 | 0.897 |
| 0.80 | 0.994 | 0.963 | 0.919 | 0.973 | 0.944 | 0.902 | 0.979 | 0.950 | 0.907 |
| 0.90 | 0.985 | 0.954 | 0.909 | 0.964 | 0.936 | 0.894 | 0.972 | 0.943 | 0.901 |
| 1.00 | 0.977 | 0.946 | 0.902 | 0.958 | 0.929 | 0.887 | 0.965 | 0.936 | 0.893 |

TABLE 3 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.25 E(Q2)= 1.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.072 | 1.072 | 1.059 | 1.101 | 1.086 | 1.059 | 1.088 | 1.066 | 1.032 |
| 0.10 | 1.108 | 1.107 | 1.091 | 1.086 | 1.072 | 1.046 | 1.076 | 1.056 | 1.023 |
| 0.20 | 1.156 | 1.150 | 1.130 | 1.081 | 1.068 | 1.042 | 1.069 | 1.049 | 1.017 |
| 0.30 | 1.131 | 1.121 | 1.098 | 1.089 | 1.076 | 1.050 | 1.065 | 1.045 | 1.014 |
| 0.40 | 1.118 | 1.106 | 1.080 | 1.119 | 1.105 | 1.078 | 1.063 | 1.044 | 1.013 |
| 0.50 | 1.111 | 1.096 | 1.068 | 1.105 | 1.090 | 1.062 | 1.065 | 1.046 | 1.014 |
| 0.60 | 1.107 | 1.090 | 1.061 | 1.094 | 1.078 | 1.050 | 1.074 | 1.055 | 1.024 |
| 0.70 | 1.105 | 1.087 | 1.055 | 1.087 | 1.070 | 1.041 | 1.093 | 1.074 | 1.042 |
| 0.80 | 1.104 | 1.084 | 1.052 | 1.082 | 1.064 | 1.034 | 1.113 | 1.094 | 1.061 |
| 0.90 | 1.103 | 1.082 | 1.049 | 1.078 | 1.060 | 1.029 | 1.107 | 1.087 | 1.054 |
| 1.00 | 1.103 | 1.081 | 1.047 | 1.076 | 1.057 | 1.026 | 1.099 | 1.079 | 1.046 |

TABLE 4 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.75 E(Q2)= 0.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.091 | 1.077 | 1.049 | 1.101 | 1.086 | 1.058 | 1.065 | 1.046 | 1.014 |
| 0.10 | 1.098 | 1.085 | 1.059 | 1.089 | 1.075 | 1.048 | 1.056 | 1.038 | 1.007 |
| 0.20 | 1.116 | 1.103 | 1.077 | 1.082 | 1.069 | 1.042 | 1.050 | 1.032 | 1.003 |
| 0.30 | 1.142 | 1.128 | 1.101 | 1.086 | 1.072 | 1.046 | 1.047 | 1.030 | 1.000 |
| 0.40 | 1.122 | 1.108 | 1.080 | 1.105 | 1.091 | 1.064 | 1.046 | 1.029 | 1.000 |
| 0.50 | 1.105 | 1.090 | 1.062 | 1.090 | 1.076 | 1.049 | 1.047 | 1.030 | 1.001 |
| 0.60 | 1.094 | 1.078 | 1.050 | 1.078 | 1.064 | 1.036 | 1.055 | 1.038 | 1.009 |
| 0.70 | 1.087 | 1.070 | 1.041 | 1.069 | 1.054 | 1.027 | 1.071 | 1.054 | 1.024 |
| 0.80 | 1.082 | 1.064 | 1.034 | 1.063 | 1.048 | 1.020 | 1.087 | 1.070 | 1.040 |
| 0.90 | 1.078 | 1.060 | 1.029 | 1.059 | 1.043 | 1.014 | 1.082 | 1.064 | 1.034 |
| 1.00 | 1.076 | 1.057 | 1.026 | 1.056 | 1.039 | 1.010 | 1.075 | 1.057 | 1.026 |

TABLE 5 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.75 E(Q2)= 0.50 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.074 | 1.052 | 1.018 | 1.087 | 1.068 | 1.036 | 1.093 | 1.073 | 1.040 |
| 0.10 | 1.075 | 1.055 | 1.022 | 1.077 | 1.059 | 1.028 | 1.085 | 1.065 | 1.033 |
| 0.20 | 1.082 | 1.062 | 1.030 | 1.070 | 1.053 | 1.022 | 1.078 | 1.059 | 1.028 |
| 0.30 | 1.092 | 1.073 | 1.041 | 1.071 | 1.053 | 1.024 | 1.074 | 1.055 | 1.024 |
| 0.40 | 1.106 | 1.086 | 1.054 | 1.082 | 1.064 | 1.035 | 1.071 | 1.053 | 1.022 |
| 0.50 | 1.121 | 1.102 | 1.069 | 1.094 | 1.077 | 1.047 | 1.070 | 1.052 | 1.021 |
| 0.60 | 1.139 | 1.119 | 1.086 | 1.109 | 1.091 | 1.061 | 1.075 | 1.057 | 1.027 |
| 0.70 | 1.130 | 1.110 | 1.077 | 1.101 | 1.084 | 1.054 | 1.086 | 1.069 | 1.038 |
| 0.80 | 1.117 | 1.097 | 1.064 | 1.090 | 1.073 | 1.043 | 1.099 | 1.081 | 1.050 |
| 0.90 | 1.107 | 1.087 | 1.054 | 1.082 | 1.064 | 1.034 | 1.091 | 1.073 | 1.042 |
| 1.00 | 1.099 | 1.079 | 1.046 | 1.075 | 1.057 | 1.026 | 1.083 | 1.065 | 1.034 |

TABLE 6 : PARTIALCOEFFICIENT GAMMA M1

BETA= 3.75 E(Q2)= 1.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.180 | 1.198 | 1.205 | 1.228 | 1.229 | 1.219 | 1.224 | 1.216 | 1.197 |
| 0.10 | 1.221 | 1.238 | 1.243 | 1.211 | 1.213 | 1.204 | 1.209 | 1.203 | 1.185 |
| 0.20 | 1.278 | 1.291 | 1.292 | 1.205 | 1.207 | 1.199 | 1.200 | 1.194 | 1.178 |
| 0.30 | 1.255 | 1.263 | 1.259 | 1.213 | 1.216 | 1.207 | 1.195 | 1.190 | 1.174 |
| 0.40 | 1.245 | 1.249 | 1.241 | 1.248 | 1.250 | 1.240 | 1.193 | 1.188 | 1.173 |
| 0.50 | 1.240 | 1.241 | 1.231 | 1.234 | 1.234 | 1.224 | 1.194 | 1.190 | 1.174 |
| 0.60 | 1.238 | 1.237 | 1.224 | 1.222 | 1.222 | 1.210 | 1.204 | 1.200 | 1.185 |
| 0.70 | 1.238 | 1.235 | 1.220 | 1.215 | 1.214 | 1.201 | 1.226 | 1.222 | 1.206 |
| 0.80 | 1.238 | 1.234 | 1.217 | 1.211 | 1.209 | 1.195 | 1.249 | 1.244 | 1.228 |
| 0.90 | 1.239 | 1.233 | 1.215 | 1.209 | 1.205 | 1.190 | 1.242 | 1.237 | 1.220 |
| 1.00 | 1.240 | 1.233 | 1.214 | 1.207 | 1.203 | 1.187 | 1.233 | 1.228 | 1.211 |

TABLE 7 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.25 E(Q2)= 0.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.217 | 1.219 | 1.209 | 1.229 | 1.230 | 1.219 | 1.194 | 1.190 | 1.174 |
| 0.10 | 1.224 | 1.226 | 1.218 | 1.215 | 1.216 | 1.206 | 1.184 | 1.180 | 1.165 |
| 0.20 | 1.244 | 1.247 | 1.238 | 1.207 | 1.209 | 1.200 | 1.176 | 1.173 | 1.159 |
| 0.30 | 1.272 | 1.275 | 1.266 | 1.210 | 1.212 | 1.204 | 1.172 | 1.170 | 1.156 |
| 0.40 | 1.251 | 1.253 | 1.244 | 1.232 | 1.234 | 1.225 | 1.171 | 1.168 | 1.155 |
| 0.50 | 1.234 | 1.234 | 1.224 | 1.216 | 1.218 | 1.208 | 1.172 | 1.169 | 1.156 |
| 0.60 | 1.222 | 1.222 | 1.210 | 1.203 | 1.204 | 1.194 | 1.180 | 1.178 | 1.165 |
| 0.70 | 1.215 | 1.214 | 1.201 | 1.195 | 1.194 | 1.183 | 1.198 | 1.196 | 1.183 |
| 0.80 | 1.211 | 1.209 | 1.195 | 1.189 | 1.188 | 1.176 | 1.217 | 1.215 | 1.201 |
| 0.90 | 1.209 | 1.205 | 1.190 | 1.185 | 1.183 | 1.171 | 1.211 | 1.208 | 1.194 |
| 1.00 | 1.207 | 1.203 | 1.187 | 1.182 | 1.180 | 1.167 | 1.204 | 1.201 | 1.186 |

TABLE 8 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.25 E(Q2)= 0.50 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.208 | 1.201 | 1.181 | 1.220 | 1.215 | 1.199 | 1.228 | 1.222 | 1.205 |
| 0.10 | 1.208 | 1.202 | 1.184 | 1.207 | 1.204 | 1.189 | 1.217 | 1.212 | 1.196 |
| 0.20 | 1.214 | 1.209 | 1.192 | 1.199 | 1.196 | 1.182 | 1.209 | 1.205 | 1.189 |
| 0.30 | 1.225 | 1.221 | 1.205 | 1.199 | 1.196 | 1.182 | 1.204 | 1.200 | 1.185 |
| 0.40 | 1.240 | 1.236 | 1.220 | 1.210 | 1.208 | 1.195 | 1.200 | 1.197 | 1.182 |
| 0.50 | 1.257 | 1.253 | 1.237 | 1.225 | 1.223 | 1.209 | 1.199 | 1.195 | 1.181 |
| 0.60 | 1.277 | 1.272 | 1.256 | 1.241 | 1.239 | 1.225 | 1.204 | 1.201 | 1.187 |
| 0.70 | 1.267 | 1.263 | 1.246 | 1.233 | 1.230 | 1.217 | 1.217 | 1.214 | 1.200 |
| 0.80 | 1.253 | 1.248 | 1.232 | 1.221 | 1.218 | 1.204 | 1.231 | 1.228 | 1.213 |
| 0.90 | 1.242 | 1.237 | 1.220 | 1.211 | 1.208 | 1.194 | 1.222 | 1.219 | 1.205 |
| 1.00 | 1.233 | 1.228 | 1.211 | 1.204 | 1.201 | 1.186 | 1.213 | 1.210 | 1.195 |

TABLE 9 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.25 E(Q2)= 1.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 VIM 0.100 | 0.150 | 0.200 | E(Q1)=0.5 VIM 0.100 | 0.150 | 0.200 | E(Q1)=1.0 VIM 0.100 | 0.150 | 0.200 |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 1.299 | 1.339 | 1.371 | 1.369 | 1.390 | 1.403 | 1.374 | 1.385 | 1.387 |
| 0.10 | 1.345 | 1.384 | 1.416 | 1.348 | 1.370 | 1.384 | 1.357 | 1.369 | 1.372 |
| 0.20 | 1.413 | 1.448 | 1.475 | 1.342 | 1.364 | 1.378 | 1.345 | 1.358 | 1.362 |
| 0.30 | 1.392 | 1.421 | 1.442 | 1.351 | 1.373 | 1.388 | 1.339 | 1.352 | 1.357 |
| 0.40 | 1.385 | 1.409 | 1.425 | 1.391 | 1.413 | 1.427 | 1.337 | 1.350 | 1.356 |
| 0.50 | 1.383 | 1.404 | 1.416 | 1.376 | 1.396 | 1.409 | 1.338 | 1.352 | 1.357 |
| 0.60 | 1.383 | 1.402 | 1.411 | 1.364 | 1.384 | 1.394 | 1.348 | 1.363 | 1.369 |
| 0.70 | 1.385 | 1.401 | 1.409 | 1.358 | 1.376 | 1.385 | 1.373 | 1.388 | 1.394 |
| 0.80 | 1.387 | 1.402 | 1.407 | 1.354 | 1.371 | 1.379 | 1.399 | 1.414 | 1.419 |
| 0.90 | 1.389 | 1.402 | 1.406 | 1.353 | 1.368 | 1.375 | 1.391 | 1.405 | 1.411 |
| 1.00 | 1.392 | 1.403 | 1.406 | 1.352 | 1.367 | 1.372 | 1.382 | 1.396 | 1.401 |

TABLE 10 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.75 E(Q2)= 0.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 VIM 0.100 | 0.150 | 0.200 | E(Q1)=0.5 VIM 0.100 | 0.150 | 0.200 | E(Q1)=1.0 VIM 0.100 | 0.150 | 0.200 |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 1.357 | 1.378 | 1.391 | 1.371 | 1.391 | 1.403 | 1.338 | 1.352 | 1.357 |
| 0.10 | 1.362 | 1.385 | 1.400 | 1.353 | 1.375 | 1.388 | 1.325 | 1.340 | 1.346 |
| 0.20 | 1.384 | 1.408 | 1.423 | 1.345 | 1.366 | 1.380 | 1.316 | 1.332 | 1.338 |
| 0.30 | 1.416 | 1.440 | 1.455 | 1.348 | 1.370 | 1.384 | 1.311 | 1.327 | 1.334 |
| 0.40 | 1.394 | 1.417 | 1.430 | 1.373 | 1.395 | 1.409 | 1.309 | 1.325 | 1.333 |
| 0.50 | 1.376 | 1.396 | 1.409 | 1.355 | 1.377 | 1.390 | 1.310 | 1.326 | 1.134 |
| 0.60 | 1.364 | 1.384 | 1.394 | 1.342 | 1.362 | 1.374 | 1.319 | 1.336 | 1.344 |
| 0.70 | 1.358 | 1.376 | 1.385 | 1.333 | 1.352 | 1.363 | 1.339 | 1.356 | 1.365 |
| 0.80 | 1.354 | 1.371 | 1.379 | 1.327 | 1.345 | 1.355 | 1.361 | 1.378 | 1.386 |
| 0.90 | 1.353 | 1.368 | 1.375 | 1.324 | 1.341 | 1.350 | 1.354 | 1.371 | 1.379 |
| 1.00 | 1.352 | 1.367 | 1.372 | 1.322 | 1.338 | 1.346 | 1.347 | 1.362 | 1.370 |

TABLE 11 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.75 E(Q2)= 0.50 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 VIM 0.100 | 0.150 | 0.200 | E(Q1)=0.5 VIM 0.100 | 0.150 | 0.200 | E(Q1)=1.0 VIM 0.100 | 0.150 | 0.200 |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 1.357 | 1.368 | 1.369 | 1.366 | 1.380 | 1.386 | 1.377 | 1.390 | 1.394 |
| 0.10 | 1.356 | 1.368 | 1.371 | 1.351 | 1.366 | 1.373 | 1.364 | 1.378 | 1.383 |
| 0.20 | 1.361 | 1.375 | 1.379 | 1.341 | 1.357 | 1.364 | 1.354 | 1.369 | 1.375 |
| 0.30 | 1.372 | 1.387 | 1.392 | 1.340 | 1.356 | 1.365 | 1.348 | 1.362 | 1.369 |
| 0.40 | 1.388 | 1.403 | 1.409 | 1.353 | 1.370 | 1.379 | 1.343 | 1.358 | 1.365 |
| 0.50 | 1.408 | 1.423 | 1.429 | 1.369 | 1.386 | 1.395 | 1.341 | 1.357 | 1.363 |
| 0.60 | 1.429 | 1.445 | 1.452 | 1.387 | 1.404 | 1.413 | 1.347 | 1.363 | 1.370 |
| 0.70 | 1.419 | 1.434 | 1.440 | 1.378 | 1.395 | 1.404 | 1.361 | 1.377 | 1.385 |
| 0.80 | 1.403 | 1.418 | 1.424 | 1.365 | 1.381 | 1.390 | 1.376 | 1.393 | 1.400 |
| 0.90 | 1.391 | 1.405 | 1.411 | 1.354 | 1.371 | 1.379 | 1.367 | 1.383 | 1.391 |
| 1.00 | 1.382 | 1.396 | 1.401 | 1.347 | 1.362 | 1.370 | 1.357 | 1.173 | 1.380 |

TABLE 12 : PARTIALCOEFFICIENT GAMMA M1

BETA= 4.75 E(Q2)= 1.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.429 | 1.496 | 1.560 | 1.525 | 1.571 | 1.614 | 1.540 | 1.575 | 1.605 |
| 0.10 | 1.482 | 1.548 | 1.613 | 1.500 | 1.547 | 1.591 | 1.519 | 1.555 | 1.587 |
| 0.20 | 1.561 | 1.624 | 1.685 | 1.493 | 1.540 | 1.583 | 1.506 | 1.542 | 1.574 |
| 0.30 | 1.543 | 1.599 | 1.652 | 1.502 | 1.550 | 1.594 | 1.498 | 1.535 | 1.568 |
| 0.40 | 1.539 | 1.589 | 1.636 | 1.548 | 1.596 | 1.640 | 1.495 | 1.533 | 1.566 |
| 0.50 | 1.540 | 1.587 | 1.629 | 1.533 | 1.578 | 1.621 | 1.497 | 1.534 | 1.568 |
| 0.60 | 1.543 | 1.587 | 1.626 | 1.521 | 1.565 | 1.605 | 1.507 | 1.546 | 1.580 |
| 0.70 | 1.548 | 1.588 | 1.625 | 1.515 | 1.557 | 1.596 | 1.535 | 1.574 | 1.609 |
| 0.80 | 1.552 | 1.591 | 1.625 | 1.512 | 1.553 | 1.590 | 1.565 | 1.604 | 1.639 |
| 0.90 | 1.556 | 1.593 | 1.626 | 1.512 | 1.551 | 1.586 | 1.556 | 1.595 | 1.629 |
| 1.00 | 1.560 | 1.596 | 1.627 | 1.512 | 1.551 | 1.584 | 1.547 | 1.585 | 1.618 |

TABLE 13 : PARTIALCOEFFICIENT GAMMA M1

BETA= 5.25 E(Q2)= 0.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.511 | 1.557 | 1.599 | 1.527 | 1.573 | 1.615 | 1.497 | 1.534 | 1.568 |
| 0.10 | 1.515 | 1.563 | 1.608 | 1.507 | 1.553 | 1.596 | 1.481 | 1.520 | 1.553 |
| 0.20 | 1.539 | 1.588 | 1.634 | 1.497 | 1.543 | 1.586 | 1.471 | 1.510 | 1.544 |
| 0.30 | 1.575 | 1.625 | 1.672 | 1.500 | 1.547 | 1.590 | 1.465 | 1.504 | 1.539 |
| 0.40 | 1.552 | 1.600 | 1.644 | 1.528 | 1.575 | 1.619 | 1.462 | 1.502 | 1.537 |
| 0.50 | 1.533 | 1.578 | 1.621 | 1.509 | 1.555 | 1.598 | 1.463 | 1.503 | 1.538 |
| 0.60 | 1.521 | 1.565 | 1.605 | 1.495 | 1.540 | 1.581 | 1.472 | 1.513 | 1.549 |
| 0.70 | 1.515 | 1.557 | 1.596 | 1.486 | 1.529 | 1.569 | 1.495 | 1.536 | 1.573 |
| 0.80 | 1.512 | 1.553 | 1.590 | 1.480 | 1.522 | 1.561 | 1.520 | 1.561 | 1.598 |
| 0.90 | 1.512 | 1.551 | 1.586 | 1.477 | 1.518 | 1.555 | 1.513 | 1.553 | 1.590 |
| 1.00 | 1.512 | 1.551 | 1.584 | 1.476 | 1.516 | 1.552 | 1.505 | 1.544 | 1.580 |

TABLE 14 :. PARTIALCOEFFICIENT GAMMA M1

BETA= 5.25 E(Q2)= 0.50 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

| E(P) | Q1= 0 | | | E(Q1)=0.5 | | | E(Q1)=1.0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | VIM | | | VIM | | | VIM | | |
| | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 | 0.100 | 0.150 | 0.200 |
| 0.00 | 1.522 | 1.556 | 1.585 | 1.528 | 1.566 | 1.600 | 1.541 | 1.578 | 1.611 |
| 0.10 | 1.519 | 1.554 | 1.586 | 1.511 | 1.550 | 1.585 | 1.526 | 1.564 | 1.597 |
| 0.20 | 1.523 | 1.561 | 1.594 | 1.499 | 1.538 | 1.574 | 1.515 | 1.553 | 1.587 |
| 0.30 | 1.535 | 1.574 | 1.608 | 1.496 | 1.537 | 1.573 | 1.507 | 1.545 | 1.580 |
| 0.40 | 1.552 | 1.592 | 1.627 | 1.510 | 1.552 | 1.589 | 1.502 | 1.541 | 1.575 |
| 0.50 | 1.574 | 1.614 | 1.650 | 1.528 | 1.570 | 1.608 | 1.499 | 1.538 | 1.573 |
| 0.60 | 1.598 | 1.639 | 1.676 | 1.548 | 1.590 | 1.629 | 1.504 | 1.544 | 1.580 |
| 0.70 | 1.587 | 1.627 | 1.663 | 1.538 | 1.580 | 1.618 | 1.520 | 1.561 | 1.597 |
| 0.80 | 1.569 | 1.609 | 1.644 | 1.524 | 1.565 | 1.602 | 1.538 | 1.578 | 1.615 |
| 0.90 | 1.556 | 1.595 | 1.629 | 1.513 | 1.553 | 1.590 | 1.527 | 1.568 | 1.604 |
| 1.00 | 1.547 | 1.585 | 1.618 | 1.505 | 1.544 | 1.580 | 1.516 | 1.556 | 1.592 |

TABLE 15 : PARTIALCOEFFICIENT GAMMA M1

BETA= 5.25 E(Q2)= 1.00 E(G)= 1.00 VM= 0.15
AP= 1.00 AQ1= 0.50 AQ2= 0.00

## K5. STATISTICAL METHOD

### K5.1 Principles.

The statistical method specified in the proposal is a first order second moment statistical method. The method is not connected to specific distributions of the parameters considered but operates only with mean values and coefficients of variation.

The method may be explained by the following reliability model. It is assumed that the calculation model of a load carrying structure is given in the function

$$g(x_1, \ldots, x_n, y_1, \ldots, y_r) \qquad (1)$$

where the designations are the same as indicated in the proposal. The function $g(\cdot)$ is defined such, that the limit state considered will be exceeded if

$$g(\cdot) \leq 0 \qquad (2)$$

It is assumed that the variables are random variables $X_1, \ldots, X_n, Y_1, \ldots, Y_r$. The inequality

$$g(X_1, \ldots, X_n, Y_1, \ldots, Y_r) \leq 0 \qquad (3)$$

then expresses an incidental event namely the structure being in the limit state or having exceeded it.

Into the expression (3), factors $\kappa$ are introduced, so that

$$g(\kappa_{x_1} X_1, \ldots, \kappa_{x_n} X_n, \kappa_{y_1} Y_1, \ldots, \kappa_{y_r} Y_r) = 0 \qquad (4)$$

where

$$\kappa_{x_i} = \exp(-\alpha_{x_i} \nu V_{x_i}) \tag{5}$$

$$\kappa_{y_i} = 1 + \alpha_{y_1} \nu V_{Y_i} \tag{6}$$

In the equations (4) and (5), the following designations are used:

V : coefficients of variation

$\alpha$ : coefficients, which may assume fixed values between -1 and 1

$\nu$ : a random variable defined by the equations (5) and (6).

This definition of the random variable, $\nu$, implies that the limit state considered has been exceeded provided $\nu < 0$. The limit state is defined for $\nu = 0$, which may be seen from the equations (4), (5) and (6).

The safety of the structure may be measured by the safety index $\beta$ defined ad

$$\beta = \frac{E(\nu)}{D(\nu)}$$

where $E(\nu)$ denotes the mean value of $\nu$ and $D(\nu)$ denotes the standard deviation of $\nu$. This definition springs from the idea that the larger the mean value $E(\nu)$ measured in terms of $D(\nu)$, the larger the structural reliability.

For a structure subjected to certain loads, with known mean values and coefficients of variation, the mean value $E(\nu)$ may be implicitly expressed using equation 4, as functions of $\alpha_x$ and $\alpha_y$. Using the linearization of (4) round the mean value $E(\nu)$, the standard deviation $D(\nu)$ may be implicitly expressed by $\alpha_x$ and $\alpha_y$.

The safety index $\beta$ should now be written as a implicite function of $\alpha_x$ and $\alpha_y$. The safety index $\beta$ is then determined for the values of $\alpha_x$ and $\alpha_y$ corresponding to the minimum value of $\beta$.

Provided the partial derivatives of the function $g(\cdot)$ are continuous and the random variables are uncorrelated, the minimum value of $\beta$ is determined by the values of $\alpha_x$ and $\alpha_y$ expressed in equations (10) and (11) in the proposal. For those values of $\alpha_x$ and $\alpha_y$, the standard deviation $D(\nu)$ is equal to 1 so that $E(\nu)$ is equal to $\beta$.

The requirement to the safety of the structure should therefore be expressed by equation (7) in the proposal.

From equations (10) and (11) in the proposal, the following $\alpha$-values apply

$$\alpha_{x_1}^2 + \ldots\ldots + \alpha_{x_n}^2 + \alpha_{y_1}^2 + \ldots\ldots + \alpha_{y_r}^2 = 1 \qquad (8)$$

In cases, where the partial derivatives of the function $g(\cdot)$ are not continuous, the method has to be used on each of the intervals for which the partial derivatives are continuous. Determining the value of $\beta$ for each of these intervals, the safety of the structure may be defined as the smallest value of these $\beta$ values.

## K5.7 An Example of The Use of the Statistical Method.

In the following the application of the statistical method should be demostrated on a frequently used failure criterion, e.g., the expression

$$R - (S_g + S_p + S_q) = 0 \qquad (1)$$

The definition of the symbols is given below

R : the resistance, for instance a yield bending moment

$S_g$: the load effect from dead loads, for instance a bending moment

$S_p$: the load effects from to different live
$S_q$: loads.

It is assumed, that the resistance, R, and the load effects $S_g$, $S_p$ and $S_q$ can be expressed by the primary resistance parameter, m, and the primary loading parameters, g, p and q, in the following way.

$$R = c_m m$$

$$S_g = c_g g$$

$$S_p = c_p p$$

$$S_q = c_q q$$

In these expressions, $c_m$, $c_g$, $c_p$ and $c_q$ are constants, which include geometrical parameters.

The failure criterion (1) may under these assumptions be expressed by the primary resistance and the primary loading parameters in the following way:

$$c_m m - c_g g - c_q q - c_p p = 0 \tag{2}$$

It is now assumed that a structure should be designed in accordance with the calculation model given in (2).

The following parameters should then be given

| | |
|---|---|
| safety index | : $\beta$ |
| mean values | : E(m), E(g), E(p), and E(q) |
| coefficients of variation | : $V_m$, $V_g$, $V_p$, and $V_q$ |
| | $V_{I_m}$, $V_{I_g}$, $V_{I_p}$, and $V_{I_q}$ |
| constants | : $c_g$, $c_p$, and $c_q$ |

The problem then consists in determining the value of the parameter $c_m$, for instance the effective depth of the cross section of a reinforced concrete member.

Reliability Model

Multiplying the primary resistance parameter and the primary loading parameters by the factors $\kappa$ equation (2) yields

$$c_m m \kappa_m - c_g g \kappa_g - c_p p \kappa_p - c_q q \kappa_q = 0$$

where

$$\begin{aligned} \kappa_m &= \exp(-\alpha_m \beta V_M) \\ \kappa_g &= 1 + \alpha_g \beta V_G \\ \kappa_p &= 1 + \alpha_p \beta V_P \\ \kappa_q &= 1 + \alpha_q \beta V_Q \end{aligned} \tag{4}$$

$$\begin{aligned} \alpha_m &= \frac{\kappa_m c_m \cdot E(m) V_M}{N} \\ \alpha_p &= \frac{c_p \cdot E(p) V_P}{N} \\ \alpha_q &= \frac{c_q \cdot E(q) V_Q}{N} \end{aligned} \tag{5}$$

$$\alpha_m^2 + \alpha_g^2 + \alpha_p^2 + \alpha_q^2 = 1 \tag{6}$$

$$N = \sqrt{(\kappa_m c_m E(m) V_M)^2 + (c_g E(g) V_G)^2 + (c_p E(p) V_P)^2 + (c_q E(q) V_Q)^2} \tag{7}$$

$$V_M = \sqrt{V_m^2 + V_{I_m}^2}$$

$$V_G = \sqrt{V_g^2 + V_{I_g}^2}$$

$$V_P = \sqrt{V_p^2 + V_{I_p}^2} \tag{8}$$

$$V_Q = \sqrt{V_q^2 + V_{I_q}^2}$$

The requirement to the safety of the structure may just be filled when

$$c_m E(m)\kappa_m - c_g E(g)\kappa_g - c_p E(p)\kappa_p - c_q E(q)\kappa_q = 0 \tag{9}$$

or

$$c_m E(m)\kappa_m = c_g E(g)\kappa_g + c_p E(p)\kappa_p + c_q E(q)\kappa_q = 0 \tag{10}$$

Using equation (10) in the expressions for $\alpha_m$, $\alpha_g$, $\alpha_p$ and $\alpha_q$, the value of $c_m$ may be determined by iteration.

In the following it should be demonstrated how the above mentioned equations are solved, assumming the values specified below

$$c_g E(g) = 1$$

$$c_p E(p) = 1$$

$$c_q E(q) = 1$$

$$V_g = V_{I_g} = 0{,}05$$

$$V_p = V_q = 0{,}4$$

$V_{I_p} = V_{I_q} = 0,3$

$V_m = 0,15$

$V_{I_m} = 0,10$

$\beta = 4,75$

First $V_M$, $V_G$, $V_P$ and $V_Q$ are determined from the equations (8)

$$V_M = \sqrt{0.1^2 + 0.15^2} = 0.18$$

$$V_G = \sqrt{0.05^2 + 0.05^2} = 0.07$$

$$V_P = \sqrt{0.40^2 + 0.30^2} = 0.5$$

$$V_Q = \sqrt{0.40^2 + 0.30^2} = 0.5$$

Starting the iteration by defining $\alpha_g = \alpha_p = \alpha_q = 0$ the eauations may be solved successively. The calculations are illustrated in table K5.7.

| Equation Number | Parameter | Iteration number | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 |
| (5) | $\alpha_g$ | 0 | 0.078 | 0.056 | 0.062 | 0.060 | 0.061 |
| (5) | $\alpha_p$ | 0 | 0.560 | 0.401 | 0.440 | 0.430 | 0.433 |
| (5) | $\alpha_q$ | 0 | 0.560 | 0.401 | 0.440 | 0.430 | 0.433 |
| (6) | $\alpha_m$ | 1 | 0.605 | 0.821 | 0.780 | 0.791 | 0.789 |
| (4) | $\kappa_m$ | 0.425 | 0.596 | 0.495 | 0.513 | 0.508 | 0.510 |
| (4) | $\kappa_g$ | 1 | 1.026 | 1.019 | 1.020 | 1.020 | 1.020 |
| (4) | $\kappa_p$ | 1 | 2.331 | 1.953 | 2.045 | 2.022 | 2.028 |
| (4) | $\kappa_q$ | 1 | 2.331 | 1.953 | 2.045 | 2.022 | 2.028 |
| (10) | $c_m E(m) \kappa_m$ | 3 | 5.687 | 4.925 | 5.111 | 5.063 | 5.075 |
| (4)og(10) | $c_m E(m)$ | 7.054 | 9.540 | 9.941 | 9.959 | 9.961 | 9.961 |

Table K5.7. Iteration Procedure.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# LOADING REGULATIONS

NORDIC COMMITTEE FOR BUILDING REGULATIONS

PARIS - FEBRUARY 1975

NORDIC COMMITTEE ON
BUILDING REGULATIONS

Loading Sub-Committee

DRAFT 74-10-30

# LOADING REGULATIONS

Regulations and instructions
concerning loads and other actions

INTRODUCTION

These LOADING REGULATIONS are to be used directly in conjunction with design calculations for buildings. They are also intended, where applicable, for use by other code drafting committees working on loadbearing structures.

The regulations are divided into regulations, instructions and comments.

The regulations extend over the whole width of a column.

The instructions appear in direct conjunction with the regulations and are inset. They constitute examples of approved ways of applying the regulations.

The comments, which are in a separate volume, are intended to provide the background to the regulations and instructions.

CONTENTS

# 1 DEFINITIONS AND SYMBOLS

## 1.1 Definitions

Action

is every external factor which has an effect on the risk of damage or other undesirable behaviour in a structure. The term "action" also comprises forces due to the mass of the structure itself.

Action is classified into

action of force which is primarily a force

deformation action which primarily causes deformation. If deformation is wholly or partially prevented, external and internal forces and moments arise.

environmental action is caused by the environment in which the structure is situated and which can in this way affect the properties of the structure.

Deformation action can consist of temperature changes and moisture changes in the surroundings of the structure. Other imposed deformations such as displacements of supports can also be classified as deformation action.

Environmental action can consist of temperature and moisture conditions, chemical and biological action, action due to radiation. The properties of the structure which are affected are primarily the strength properties (e.g. due to temperature) and geometrical quantities (e.g. due to chemical action which causes corrosion).

Loading is the generic term for action of force and deformation action.

Effects due to an action are consequences of such action which are significant with regard to the risk of damage or other undesirable behaviour in a structure. Such effects can be

internal forces and moments, stresses, deformations
changes in material properties
changes in dimensions and shape

The effect due to a load consists of internal forces and moments, stresses, deformations.

Action of force

See: action.

Annual maximum

See: mean value of the intensity of action.

Assembly load

See: personal load.

"Bound" load

is a load, the magnitude of which can be changed but the distribution of which over the structure is uniquely determined when the cause of load is specified. The distribution is thus determined by only one load parameter.

This implies that if the intensity of loading at one point of the structure is known, the intensity at every other point is also determined.

A load which is not bound is denoted a "free" load.

Characteristic value of action

is the intensity which has a selected probability p of being exceeded at least once a year. The value p of the probability is in this context applicable to a structure or structural component, selected at random, which can be acted upon by the load under consideration.

The standard value of the intensity of an action is a value which in Chapter 4 and 5 is stated as a specified or approved value. With regard to use, the standard values are equivalent to characteristic values.

With regard to the value p, the following classification is employed (definitions are given in 2.3.1).

Constant action (k)

Usual action (v)

Unusual action (ov)

Extreme action (e)

Usual action is classified into

Short-term usual action (v1)

Non-short-term usual action (v2)

Coefficient of variation of the intensity of action

See: mean value of the intensity of action.

Constant action

See: characteristic value of action and 2.3.1.

Construction regulations

is a generic name for material, design and workmanship regulations comprising directions and recommendations which are specific to structures of a certain type or of a certain material.

Continuous duration

See: duration.

Crowd load

See: personal load.

Deformation action

See: action.

Density (weight)

is weight per unit volume.

Duration

of a certain intensity q of an action is defined as the **aggregate** time $t_q$ - within the period of use $t_o$ of the structure in question - during which the intensity amounts to at least q.

Continuous duration of a certain intensity q of an action is defined as the longest continuous time $t_{qs}$ during which the intensity is not less than q.

The relative duration $\eta_q$ is defined as

$$\eta_q = \frac{t_q}{t_o}$$

where $t_o$ is the period of use of the structure. It is assumed in this context that the variations in intensity of action are similar during the whole time $t_o$.

Dynamic load

Conditions which cause acceleration of significance in the structure, structural component or cause of load under consideration, give rise to dynamic loading. Whether or not a load is to be regarded as a dynamic one is thus dependent on the structure and the properties of the cause of load.

A load which is not a dynamic one is denoted a static load.

Effect due to an action

See: action.

Effect due to a load

See: action.

Environmental action

See: action.

Extreme action

See: characteristic value of action and 2.3.1.

Fatigue load

See: non-recurrent action.

"Free" load

See: "bound" load.

Lifetime (period of use)

is the period from completion of the building until the time of intended cessation of use of the building.

Load

See: action.

Load spectrum

is a term employed for the description of variations in load in design with regard to e.g. fatigue load. See: 2.1.4.

Loads due to accidents

are loads consequent upon what, in everyday terms, is referred to as accidents, natural catastrophes, etc.

Logarithmic decrement

is the natural logarithm of the quotient of the numerical values of two consecutive amplitudes of the same direction executed by a freely oscillating body. The logarithmic decrement is a measure of the damping of the oscillations.

The mean value of the intensity of action

is determined, in the same way as the coefficient of variation, from the distribution of annual maxima or, in certain cases, in conjunction with combinations (see 3.1) from the distribution of short-term maxima.

The annual maximum is the greatest value which occurs over a year.

The short-term maximum is the greatest value which occurs over a period (not specified further) which is shorter than one year.

Natural frequency

is a frequency which an oscillating body assumes when it is allowed to oscillate freely.

A natural load

has its cause in conditions not governed by human beings.

The values which describe natural loads can be assumed to remain unchanged over a long period. Assessment of future values can therefore be based on observed values.

A useful load has its cause in conditions governed by human beings.

The values which describe the useful load thus change over an extended period systematically along with changes in human activity. Assessment of future loads is therefore dependent on uncertain predictions.

Non-recurrent action

is assumed to occur at full intensity only once during the period of use of the building. Special case: non-recurrent load.

If an action of lower intensity can be assumed to occur many times, then this lower intensity should be used as the alternative basis of design.

A recurrent action is assumed to occur at full intensity several times and at substantially different intensities in the interim. Special case: recurrent load.

A recurrent load can be a pulsating load which always has the same direction (same sign) or an alternating load which changes direction (the least and greatest loads have different signs).

A recurrent action with so many load variations that it may give rise to fatigue failure in the structure is denoted a fatigue load.

Non-short-term usual action

See: characteristic value of action and 2.3.1.

Occupation load

See: personal load.

Personal load

is load caused by one or more persons.

Occupation load is personal load which is taken to be due to the fact that people congregate in certain premises in a normal manner.

Assembly load is personal load which is taken to be due to the fact that a large number of people congregate in certain premises.

Crowd load is personal load which is taken to be due to overcrowding.

Recurrent action

See: non-recurrent action.

Relative duration

See: duration.

Short-term maximum

See: mean value of the intensity of action.

Short-term usual action

See: characteristic value of action and 2.3.1.

Standard value of load

See: characteristic value of action.

Static load

See: dynamic load.

Temporary action

See: 2.3.2.

Unusual action

See: characteristic value of action and 2.3.1.

Useful load

See: natural load.

Usual action

See: characteristic value of action and 2.3.1.

## 1.2 Symbols

# 2 CLASSIFICATION OF ACTION

## 2.1 Variation of an action in time

### 2.1.1 Subdivision

The variation of an action in time is assessed with regard to

a/ the significance of the duration of the action (in relation to strength, creep deformations, etc). See 2.1.2. Duration of action.

b/ dynamic effects. This relates to loads. See 2.1.3. Static load - dynamic. load.

c/ the risk of failure as a consequence of repeated alternations in the intensity of action. See 2.1.4. Non-recurrent action - recurrent action.

### 2.1.2 Duration of action

An action at a selected intensity q is classified into duration classes, in accordance with Table 2.1.2, with regard to the continuous duration $t_{qs}$ of the intensity q.

Table 2.1.2 Duration classes.

| Duration class | Limits of the continuous duration $t_{qs}$ | | Examples |
|---|---|---|---|
| | Lower limit | Upper limit | |
| A | 250 d | - | Dead load |
| B | 15 h | 250 d | Snow load |
| C | 2 s | 15 h | Personal load |
| D | - | 2 s | Load due to impact |

In many cases the continuous duration is not sufficient to describe a time dependent effect due to the action. The aggregate effect deriving from a number of different periods may be critical (e.g. in determining creep deformations in concrete and timber). In such cases, other concepts such as relative duration and average intensity of action may be used to describe the conditions.

With regard to the use of the concepts given in this section, reference is to be made to the appropriate design regulations [1].

### 2.1.3 Static load - dynamic load

Loads are classified into

Static load

dynamic load

In the course of design, account is to be taken in conjunction with a dynamic load of the additional deformations and additional forces (allowance for dynamic effects) which arise owing to the inertia of the system (the structure + the cause of load). The term "cause of load" in this context refers to concrete objects which give rise to the dynamic effects, e.g. vehicles or reciprocating parts in a machine.

If the dominant proportion of the mass forces acts on the cause of load (e.g. a vehicle colliding with the structure), the structure may be designed as for a static load whose magnitude essentially depends on the deformation properties of the cause of load. When the mass forces of the structure are substantial, they must be taken into account, in which case it is however sometimes possible to analyse the structure as for a static load with a certain dynamic allowance. In certain cases (particularly in conjunction with problems due to oscillations) it is however necessary to perform a full analysis, due consideration being given to the dynamic conditions.

### 2.1.4 Non-recurrent action - recurrent action

Action is classified into

non-recurrent action

recurrent action, a special case being fatigue load.

In the course of design with regard to a fatigue load, determination of the effect due to the load is to be based, if this is possible, on a load spectrum for the load in question. The load spectrum expresses the

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1) Views on application are given in the comments.

relationship between the intensity q of a load cycle (a change, e.g. from the maximum value to the minimum value and back again to the subsequent maximum value) and the number of times n annually when the intensity of the load cycles exceeds the value q, generally as in FIG. 2.1.4.a.

A load spectrum is intended to express the mean values for a large number of years. With regard to different structures acted upon by the same type of load, the load spectrum is intended to express that load variation which, for a given load alternation number n, has the probability of 0.2 of being exceeded n times in the case of a structure selected at random.

When a load spectrum cannot be given, the structure is to be designed for load variations with a constant intensity $q_1$ and with a number of variations $n_1$. The values of $q_1$ and $n_1$ are to be chosen in such a way that the effect is considered to be on the safe side.

A non-recurrent load is assumed to have no significance in conjunction with design with regard to a fatigue load.

In many cases, the load spectrum can be expressed by the following parameters, FIG. 2.1.4.b.

greatest intensity of a load cycle: $q_{max}$
least " " " " " : $q_{min}$
annual number of load cycles with an intensity greater than q : n
total number of load cycles per year : $n_t$

## 2.2 Variation of an action in space

The classification in this section relates only to loads.

Loads are classified into

"bound" loads

"free" loads

A "free" load is assumed to have an entirely arbitrary distribution over the structure, within the bounds of what is possible. A load which cannot be considered "bound" and cannot in its entirety be considered "free", is to be assumed to consist of a "bound" load portion and a "free" load portion.

## 2.3 Probability of a certain intensity of action

### 2.3.1 In conjunction with the application of the method of partial coefficients

Action is divided into groups as follows:

Constant action (k). In the case of constant action,

1/ the action is to be regarded as occurring at an arbitrarily selected time

2/ The characteristic value is to be defined as the median value of the intensity.Unless otherwise specified to the contrary, the meanvalue is to be taken equivalent to the median value.

Usual action (v) is defined as action which cannot be classified as constant action and which has the probability p = 0.2 or greater of being exceeded at least once in the course of a year. The characteristic value is defined by p = 0.2.

In the case of short-term usual action (v1) the relative duration $\eta_q$ applies to the characteristic value.

$\eta_q \leq 0.001$

In the case of non-short-term usual action (v2),

$\eta_q > 0.001$

Unusual action (ov) is defined as action for which the probability of being exceeded at least once a year is between 0.2 and 0.02. The characteristic value is defined by p = 0.02.

Extreme action (e) is action which is of a type different from constant, usual and unusual action and which has an accidental character. Extreme action is more infrequent than unusual action. When the intensity of extreme action is not specified in a code, it is assessed with regard to the kind of action.

Examples of phenomena which may produce extreme action are collision with a vehicle, explosion, fire, flooding, etc. On the other hand, it is not generally assumed that snow, wind and other similar actions which have values for usual and unusual action, produce extreme action. It can however be stipulated, with regard e.g. to snow, that avalanches, snow slides etc produce extreme action.

2.3.2 In conjunction with the application of the statistical method

Action is divided into groups as follows:

Constant action is to be regarded as occurring at an arbitrarily selected time and is to be assumed unchanging in time

Temporary action occurs occasionally.

In the case of short-term temporary action, the relative duration for the value $q_{0.8}$ which is equivalent to the 0.8 fractile of the distribution, is

$$n_q \leqslant 0.001$$

In the case of non-short-term temporary action,

$$n_q > 0.001$$

Extreme action is action which is infrequent and has the character of an accident. See also 4.5.

In applying the statistical method for design, the intensity of the action is described by its mean value and the coefficient of variation, which are determined from the distribution relating to the annual maxima, i.e. the largest value which occurs over one year. In conjunction with the combination of different kinds of action (see 3.2), use is also made of the mean value and coefficient of variation relating to the distribution of short-term maxima, i.e. the largest value which occurs over a period which is shorter than one year but is not specified in greater detail.

## 3 COMBINATION OF DIFFERENT KINDS OF ACTION

### 3.1 General

Different kinds of action which can occur simultaneously are aggregated into combinations of actions. Account is taken in this connection of the conditions relating to the individual actions which characterise their variation in time and which are of significance regarding the probability of two or more actions occurring simultaneously with high values.

Actions which are greatly dependent on one another and often occur simultaneously at their maximum values (when they occur at all), are to be regarded as one action.

Actions which, in view of physical circumstances and likelihood, exclude one another are not to be combined.

## 3.2 In conjunction with the application of the statistical method

In applying the statistical method, the mean values and coefficients of variation relating to annual maxima given in CHAPTERS 4 and 5, or similar values determined according to the definitions in 2.3.2, are to be used for individual actions.

If a more accurate analysis based on verified statistical data is not carried out, the following method is to be applied in combining different kinds of action.

As regards its duration, an individual action is characterised by a value $\alpha$ as follows:

| Group according to 2.3.2 | $\alpha$ |
|---|---|
| Constant action and other action which has the relative duration = 1 | 0 |
| Non-short-term temporary action | 0.5 |
| Short-term temporary action | 1 |
| Extreme action | 4 |

In applying the statistical method, there are two types of combination, S (constant and temporary action) and SIII (constant, temporary and extreme action).

Combination S comprises:

One action with distribution relating to annual maxima combined with actions with distributions relating to short-term maxima, in such a number that the sum of $\alpha$ for all actions does not exceed 4.

Combination SIII comprises:

One extreme action combined with actions with distributions relating to short-term maxima, in such a number that the sum of $\alpha$ for all actions does not exceed 5.

In both cases the combinations must be such that the most critical case is obtained.

In combining actions, it is necessary for only one of the actions in question to be assumed to have a mean value and coefficient of variation pertaining to a distribution relating to annual maxima. Other actions, with the exception of constant actions, are in most cases given reduced mean values. It is often impossible to decide in advance which action is to be assumed to be determined on the basis of annual maxima in order that the most critical case should be obtained, and a number of cases must be examined.

The mean value and coefficient of variation for a distribution relating to short-term maxima are determined from the corresponding values relating to the distribution for annual maxima according to the following expressions:

$$m_o (1 + V_o) = m$$

$$m_o V_o = \xi mV$$

m and V are parameters (mean value and coefficient of variation) in the distribution relating to the annual maxima of the action.

$m_o$ and $V_o$ are parameters (mean value and coefficient of variation) in the distribution relating to the short-term maxima of the action.

$\xi = 1$ in normal cases.

3.3 In conjunction with the application of the method of partial coefficients

In applying the method of partial coefficients, the standard values given in CHAPTERS 4 and 5, or the characteristic values determined according to the definitions in 2.3.1, are to be used for individual actions.

If a more accurate analysis based on verified statistical data is not carried out, the following method is to be applied in combining different kinds of action.

As regards its duration, an individual action is characterised by a value $\alpha$ as follows:

| Group according to 2.3.1 | $\alpha$ |
|---|---|
| Constant action and other action which has the relative duration = 1 | 0 |
| Non-short-term usual action | 0.5 |
| Short-term usual action | 1 |
| Unusual action | 2 |
| Extreme action | 4 |

In applying the method of partial coefficients, there are three types of combination, I, II and III.

Combination I comprises:

Usual actions corrected in view of their duration, in such a number that the sum of $\alpha$ for all actions does not exceed 4.

Combination II comprises:

One unusual action combined with usual actions, corrected in view of their duration, in such a number that the sum of $\alpha$ for all actions does not exceed 4.

Combination III comprises:

One extreme action combined with usual actions, corrected in view of their duration, in such a number that the sum of $\alpha$ for all actions does not exceed 5.

Correction of a usual action in view of its duration is performed by multiplication of the value relating to usual action by a factor

$$\zeta = 1 - (\alpha + 0.85\ /1 - \xi/)\frac{V}{1 + 0.85\ V}$$

where V is the coefficient of variation for the action and relates to the annual maximum.

Normally, $\xi$ is put = 1, i.e.

$$\zeta = 1 - \frac{\alpha V}{1 + 0.85\ V}$$

For the actions dealt with in CHAPTERS 4 and 5, the following values of $\zeta$ may be used. The numbers in brackets refer to the appropriate section.

Weight of building components and earth

(4.2) $\zeta = 1$

Load due to furnishings and persons

Distributed vertical load (4.3.1.1)

| | | |
|---|---|---|
| Load portion a | | $\zeta = 1$ |
| Load portion b | | |
| type of premises | 1,2 | $\zeta = 0.7$ |
| | 3 | $\zeta = 0.9$ |
| | 4 | $\zeta = 0.8$ |
| | 5,6,7 | $\zeta = 0.6$ |

Concentrated load (4.3.1.2)
Load due to furnishings $\zeta = 1$

Load due to goods, bulk products, etc (4.3.2)

The value of $\zeta$ is to be assessed in view of conditions.

Load due to vehicles, transport appliances and machinery (4.3.3)

The value of $\zeta$ is to be assessed in view of conditions.

Snow load (4.4.1) $\zeta = 0.8$

Wind load (4.4.2) $\zeta = 0.7$

Water pressure (4.4.3) $\zeta = 1$

4 LOADS

4.1 General

In design calculations, account is to be taken of the fact that loads can have a different (and more unfavourable) direction from that specified in these regulations or that assumed on the basis of accepted practice, and that columns and walls in a building frame may have unintentional inclinations or curvatures which produce forces that can be significant with regard to the stability of the structure as a whole.

In the case of a building frame, it is accepted that the above is taken into consideration by the frame being analysed for fictive horizontal forces. These forces may be assumed to be equal to 1.5% of the stipulated vertical loads due to gravity and to act in any horizontal direction. In the case of one and the same structure, however, all the forces are assumed to act in the same direction. The point of action is taken to be the point where the force due to gravity acts. In the case of floor slabs, however, the force is assumed to act at the top of the slab.

The forces are not normally combined with other forces which act horizontally, and are investigated only for load combination I. They are not regarded as separate loads but are assumed to be a part of the loads on which their magnitude is based.

The prescribed and recommended properties of loads have been summarised in Table 4.1.

## 4.2 Weight of building components and earth

### 4.2.1 Weight of building components

With regard to its duration, the weight of building components is assigned to Class A as set out in 2.1.2. The relative duration is put at $\eta = 1$. The weight of building components is a static load and a non-recurrent load.

With the exceptions given below, the weight of building components is regarded as a "bound" load (according to 2.2).

With the exceptions given below, the weight of building components is regarded as a constant load (according to 2.3).

This implies that the mean value can normally be used as the characteristic value of the load. The mean value of the density of the material, multiplied by the volume as computed from nominal dimensions specified on drawings, is accepted as the calculated mean value.

The stipulated load due to non-loadbearing partitions in the building is to be specified on the design drawing.

The weight of non-loadbearing partitions is to be determined according to the principles specified in relation to other building components. In the case of premises in which the subdivision of rooms may in the future be changed due to different positioning of non-loadbearing partitions, it is permissible for this to be taken into account by the load being calculated for a supposed reasonable subdivision of rooms and for walls as in the rest of the structure. One (the most unfavourable one) of the following equivalent loads is however to be used as the minimum value of the load.

- One uniformly distributed floor load of $0.5\ kN/m^2$.
- One uniformly distributed line load of 1.0 kN/m.

These equivalent loads are alternatives to the weight of the partitions according to the drawing.

Where special conditions warrant higher values of the specified minimum values, these must be increased. Such special conditions may e.g. be that the storey height of the building is larger than normal, that the general character of the building necessitates heavy partitions, etc.
The equivalent loads are assumed to have the same properties as the weight of other building components.

Exceptions from the above can be made in the following cases:

a/ The weight of building components which it is considered can be easily removed or repositioned is to be regarded as a "free" load (according to (2.2) and as usual load (according to 2.3).

b/ In the case of structures where distribution of the weight is of essential significance in design, the possibility of uneven distribution is to be taken into consideration.

This refers only to cases where the weight is the predominant load and where the structure is designed in such a way that the effect due to the weight of different parts is balanced, such as, for instance, in the case of moments in arches of long spans. The possibility of uneven distribution can be taken into consideration by assuming the structure to be acted upon by a load due to a weight of $G - \Delta G$, where G is a constant load and $\Delta G$ a non-short-term usual load (v2) and a "free" load. The largest of the following values is accepted as the value of $\Delta G$:

$$\Delta G = G\ V_{\gamma}$$

$$\Delta G = G\ V_{v}$$

where $V_\gamma$ is the coefficient of variation of the weight, and
$V_v$ the coefficient of variation of the volume.

The relative change in volume equivalent to one half of the utilised dimensional tolerance is accepted as the value of $V_v$.

c/ In evaluating a structure according to the statistical method, the uncertainty in the weight of building components is to be taken into account by the introduction of a coefficient of variation relating to the weight of building components.

The coefficient of variation relating to weight can be determined according to the laws of statistics on the basis of known data pertaining to weight and volume. Where there is insufficient material for such a determination, a value of 5% is accepted for ~~x~~/the coefficient of variation in cases where the uncertainty need not be considered as large.

For determination of loads in normal cases, the values of weights of building materials set out in Table 4.2.1 are accepted. In special cases, the occurrence of higher or lower values may have to be taken into account.

### 4.2.2 Load on building components due to handling

In the case of loadbearing structural components which are not manufactured in their final position, consideration is to be given in the course of design to the special circumstances which can arise in conjunction with the storage, transport, lifting and erection of these components. Special consideration is to be given to the following:

a/ The distribution of forces on the structural component can be different from that in the completed building. If the component is not designed in such a way that the positions of points of support during storage and transport, and the points of attachment during lifting, can be chosen arbitrarily, then these positions must be specified on the drawing.

b/ Dynamic forces occur during transport and lifting.

c/ The absence of bracing forces on the structural component can give rise to the risk of e.g. stability failure.

Unless a more accurate analysis is performed, it is permissible for dynamic forces to be taken into account by an allowance for dynamic effects, the magnitude of which is assumed to be 50% of the weight.

4.2.3 Weight of earth

Subject to exception c/ below, the weight of earth as regards its duration is assigned to Class A according to 2.1.2, and its relative duration is put as $\eta = 1$. The weight of earth is a static load and a non-recurrent load.

Subject to the exceptions a-c given below, the weight of earth is to be regarded as a "bound" load (according to 2.2).

Subject to the exceptions a-c given below, the weight of earth is to be regarded as constant load (according to 2.3).

Exceptions from the above can be made in the following cases:

a/ If, in designing a building structure, the volume of earth in question or part thereof has only a small probability of being removed (if there is any likelihood at all of its being removed), then absence of its weight is to be regarded as a "free" load and as unusual or extreme load, depending on conditions.

For instance, in the case of earth being washed away in conjunction with flooding, the absence of its weight is to be regarded as extreme load.

b/ If the probability of a certain earth volume or part thereof being removed cannot be regarded as small, absence of its weight is to be regarded as a "free" load and as a usual load.

This is the case, for instance, when excavation of earth in conjunction with re-laying of pipes can be expected.

c/ In conjunction with earthworks, the weight of the earth concerned is to be classified in each individual case depending on the type and planned progress of the work.

If there are no values available regarding the density of earth which are based on investigations, the values specified in Table 4.2.3 may be used.

## 4.3 Useful loads

### 4.3.1 Loads due to furnishings and persons

#### 4.3.1.1 Distributed vertical load

This load is made up of two portions a and b which are described below. The load portion a is supposed to be mainly associated with furnishings etc, and the load portion b with persons.

Load portions a and b are both classified as static load and recurrent load, but not fatigue load.

As regards its duration, the load portion a is assigned to Class A in accordance with 2.1.2, and its relative duration is put $\eta = 1$. The load portion a is "bound" load and usual non-short-term load (v2).

The characteristic value is assessed in relation to the kind of premises in question.

The load portion b is assumed to be one of the following types:

| | | |
|---|---|---|
| 1/ Occupation load | with a characteristic value of | 0.5 kN/$m^2$ |
| 2/ Assembly load | with a characteristic value of | 1.5 kN/$m^2$ |
| 3/ Crowd load | with a characteristic value of | 3.0 kN/$m^2$ |

As regards its duration, load portion b is assigned to Class C according to 2.1.2. Its relative duration is assessed in relation to the kind of premises. The load portion b is a "free" load.

To the extent that they are considered to occur, load portion b is classified as short-term usual load (v1), non-short-term usual load (v2) or unusual load (ov), depending on the kind of premises in question.

Unless the properties of the loads are determined more accurately, e.g. by means of direct observations, the properties specified in Table 4.3.1.1 a are to be used in design. On being combined according to Chapter 3, load portions a and b must be assumed to be separate loads.

The above characteristic values for load portion b are applicable in conjunction with a loaded area of at least 30 $m^2$. In the case of a loaded area of 5 $m^2$ the specified values are to be increased by 35%, linear interpolation being applied in the case of intermediate areas.

The intention is that the whole area, the load on which acts on the structure, is to be decisive. There is no need therefore to see whether load on a small portion of a beam or slab produces a larger load and thus becomes critical. FIG. 4.3.1.1 shows for some common examples how the loaded area A is to be determined.

Crowd load is considered to occur simultaneously on an area not exceeding 50 $m^2$. Outside this area, assembly load is to be assumed if a more unfavourable effect is obtained in this way. In such a case, the crowd load + assembly load is to be regarded as a load which is considered unusual.

The above values relate to loads due to normal furnishings and persons. Loads due to special equipment, e.g. safes, are to be considered separately. See also 4.3.1.2.

A load due to books etc in libraries and records rooms is to be considered on the basis of reasonable placing and the height of shelving.

The weight of books etc may be assumed to be 8 kN/$m^3$. The free spaces between shelves may be characterised as office premises loaded in accordance with Table 4.3.1.1 a.

Loads in premises accommodating industrial activity or warehouse premises are to be assessed in view of the kind of activity. In the aggregate, however, load portions a and b must be assumed to produce at least 5 kN/$m^2$.

In applying the statistical method, the values of mean value and coefficient of variation given in Table 4.3.1.1 b are accepted. The Table also specifies that portion of the load which is assumed to be a non-short-term load.

4.3.1.2 Concentrated load

Account is to be taken in the course of design of concentrated loads which arise due to furnishings and persons.

As regards floor slabs, concentrated loads may e.g. consist of the loads, mentioned in 4.3.1.1, due to special equipment or to loads due to furnishings suspended from the slab.

In addition to the concentrated loads caused by special furnishings, the following loads are to be considered.

As an alternative to a distributed vertical load, a floor slab is to be assumed to be acted upon by a concentrated vertical load of 1.0 kN distributed over an area of 0.025x0.025 m. Walls are assumed to be acted upon by a concentrated load of 0.3 kN, acting perpendicular to the surface of the wall, which is distributed over an area of 0.1x0.1 m. As regards their duration, these loads are assigned to Class A according to 2.3.2, and their relative duration is put $\eta = 1$. They are "free" loads according to 2.2 and usual non-short-term loads (v1) according to 2.5.

Floor slabs, balconies, stairs etc are to be designed - as an alternative to the distributed vertical load and concentrated load as above - for a concentrated vertical load caused by a person in violent motion (jumping, falling, etc). Other building components can, in certain cases, be assumed to be subjected to similar forces with different directions. If in such a case failure in the building component entails an evident risk of serious injury, the building component is to be regarded as a loadbearing structure and designed in such a way that it can withstand a force assessed on the basis of reasonable assumptions.

Examples of building components designed in accordance with this instruction are:

A roof on widely spaced purlins with no safety device below to prevent a person falling from a great height in the event of failure in the roof.

Walls (particularly glass walls) separating premises at widely different levels.

Handrails and frontages on balconies, terraces, etc. Stair balustrades.

The forces considered are to be referred to Class D according to 2.3.2 as regards their duration, and their relative duration can be put $\eta = 0$. The forces are to be regarded as dynamic loads and non-recurrent loads. They are regarded as "free" loads according to 2.2.

The problems which arise in considering forces of the kinds described above relate primarily to building components of brittle materials. Building components made of pronouncedly ductile materials can often withstand impact energy, provided that they are attached in such a way that the energy can be spread and the fastenings can withstand the forces which occur.

When there are no separate investigations which warrant other values, the values of concentrated force set out in Table 4.3.1.2 are accepted. These values apply to rigid structures such as floor slabs,walls or roofs of concrete or lightweight concrete. The force on resilient structures will be smaller.

The forces specified in Table 4.3.1.2 can be assumed to increase from zero to a maximum and then to diminish to static load value over a time of 0.05-0.1 second.

In many cases, analysis or testing using the above values of load as static load can produce misleading results. An impact test which reproduces the specified force and time curves will in most cases provide a better basis of design. Such testing can often show that a structure is capable with a satisfactory factor of safety of resisting the specified forces, while analysis or testing which regards the load as a static one does not produce the same favourable results.

In applying the statistical method, the mean values set out in Table 4.3.1.2 and a coefficient of variation of 40% are approved.

4.3.2 Loads due to goods, bulk products, etc

As regards their duration, loads due to goods, bulk products etc are assigned to Class A or B according to 2.1.2, depending on circumstances. Where required, the relative duration and similar factors are to be determined in each individual case in view of the expected handling of the goods or bulk products.

Subject to the exceptions below, the load is to be regarded as static load.

The load is recurrent load, but not a fatigue load in the normal case.

Loads due to goods, bulk products etc are to be regarded as a "free" load. In the case of bulk products, however, there are often limitations as regards the possibility of a critical load position, in view of the fact that the material has a natural angle of repose.

When the largest possible load can be assumed to occur frequently, this is to be classified as non-short-term usual load according to 2.3.

Where the loads which are expected to occur are considered to be substantially lower than the largest possible load, the magnitudes of usual and unusual loads are to be determined in accordance with the definitions given in 2.3. The largest permitted load is to be displayed in the premises and is to be given the value of usual load.

If determination of the magnitude of the loads on the basis of the definitions given in 2.3 is not possible, the "largest permitted load" is to be chosen. In the course of design, this is to be assumed to be equal to non-short-term usual load (v2). In the case of unusual load, a value 30% greater is to be chosen, or the largest possible load if this is less.

In determining the effects due to loads, both the horizontal and vertical loads are to be considered. In the case of a load due to bulk products, account is to be taken of the internal friction and the friction between the bulk product and the surfaces of the structure (FIG. 4.3.2).

Unless separate investigations are performed concerning the properties of the bulk product, the values of weight and angles of friction set out in Table 4.3.2 a will be accepted.

The angle of friction between the bulk product and the surface of the structure is assumed to be $\nu . \varphi$. The values set out in Table 4.3.2 b will be approved as values of $\varphi$.

Where it is possible for bulk products or stacked goods to slip, the dynamic forces which can arise are to be considered.

In applying the statistical method, assumptions concerning the mean value of the load and the coefficient of variation are to be based on verified data,

If sufficient data cannot be obtained, the following methods will be accepted. If the largest possible load can be assumed to occur frequently, this load is assumed to represent the mean value of the load, and the coefficient of variation is put equal to zero. If the expected loads are considered to be substantially lower than the largest possible load, the mean value may be assumed to be 85% of the largest permitted load, and the coefficient of variation to be 20%.

### 4.3.3 Loads due to vehicles, transport appliances and machinery

#### 4.3.3.1 General

Loads due to vehicles, transport appliances and machinery are assessed in view of the activity of which the load is a consequence.

With the exception of certain machinery, the loads dealt with in this section have small relative duration and are assigned to Class C or D according to 2.1.2.

The loads are normally dynamic ones.

The loads are generally considered to be recurrent loads. Vehicles, transport appliances and machinery which can be moved easily are assumed to produce "free" loads. Loads due to machinery which is permanently installed are wholly or partly regarded as "bound" ones.

Forces which occur in conjunction with unintentional collision with a vehicle etc are dealt with in section 4.5.

#### 4.3.3.2 Loads due to vehicles

Parts of buildings which can be subjected to loads due to vehicles in normal road or street traffic are to be designed for the traffic loads applicable to bridges. Buildings into which individual heavy loaded vehicles forming part of normal road traffic are assumed to drive, are to be designed for one load group according to FIG. 4.3.3.2 and for a braking force of 300 kN acting in the longitudinal direction of the vehicle. The loaded lane according to the figure is to be placed in the most unfavourable way within the area which the vehicle can traverse. The axle load is to be placed in the most unfavourable way within the loaded lane, the least distance specified in the figure being taken into account.

Where it is expected that in a building there will be vehicles of a special design geared to the activity in the building, the building structure is to be designed for the wheel load of the vehicle or for its total load increased by an allowance for dynamic effects. This is assessed in view of the kind of vehicle and the nature of the surface traversed (e.g. with regard to roughness). The dynamic allowance is to have a minimum value of 20%, unless separate investigations show that a lower value is warranted.

Floor slabs in a garage or a multistorey garage for cars are assumed to be acted upon by a uniformly distributed load of 2 $kN/m^2$, or a concentrated load of 15 kN acting over an area of 0.1x0.1 m.

Floor slabs in a garage for larger vehicles are to be designed for the load due to the heaviest vehicle which is likely to occur in view of the space in the garage.

4.3.3.3 Loads due to overhead cranes, cranes and other lifting equipment

With regard to loads due to overhead cranes, cranes and other lifting equipment, the special regulations applicable to lifting appliances are to be used.

4.3.3.4 Loads due to machinery etc

Structures which support machinery etc of essential importance for the structure, are to be designed for the weight of the machinery and for the materials or products which are applicable. The dynamic effects due to the machinery are also to be taken into consideration. Where required, oscillation analysis based on verified data concerning the design of the machinery and the structure is to be performed. If a separate oscillation analysis is not performed, the dynamic effect may not normally be assumed to be less than that equivalent to 25% of the weight of the machinery. Consideration is to be given in the course of design to the fact that, in conjunction with repairs etc, parts of the machine may exert a load on the floor slab in the vicinity of the machine.

Loads due to machines which are not in themselves of essential importance for the structure need not be considered separately. It is thus to be assumed that the loads due to e.g. household machinery, small office appliances, small machines in mechanical workshops etc are part of the loads due to furnishings and persons in accordance with section 4.3.1. In the case of machines whose reciprocating parts are of slight weight, a dynamic allowance of 25% can be unrealistically large. In such cases the dynamic allowance can be related to the weight of the reciprocating parts and specified as an addition to this weight. The magnitude of the addition will be greatly dependent on the function of the reciprocating parts.

In view of the fact that the structure may have to be designed for the weight of machine components or for the weight of the machine or machine components during transport to the place of installation, concentrated loads often occur and such loads should be designed for. It is often uneconomical to cover the effect of a concentrated load by means of an equivalent distributed load.

## 4.4 Natural loads

### 4.4.1 Snow loads

#### 4.4.1.1 General

As regards its duration, snow load is assigned to Class B according to 2.1.2.

As regards the relative duration $\eta$, the following values which are assumed to be valid for the snow loads set out in FIGs. 4.4.1.2 a and b, will be accepted.

| | Usual snow load | Unusual snow load |
|---|---|---|
| Denmark and Skåne | 0.01 | 0.0002 |
| Southern Finland and Norway, Central Sweden | 0.015 | 0.0005 |
| Northern Finland, Norway and Sweden | 0.02 | 0.001 |

Snow load is a static load.

Usual snow load is a recurrent load but not a fatigue load, while unusual snow load is a non-recurrent load.

The distribution of snow load on roofs is specified by means of the shape factors in 4.4.1.3.

Account is to be taken of changes in the distribution of the snow load as a result of snow clearance or - where this is possible - as a result of snow slides due to temperature changes.

In the case of pitched roofs, the following way of considering the distribution of snow load by means of snow clearance or snow slide will be approved:

1/ it is to be assumed that the snow load on that half of the roof which carries most load has been removed,

2/ it is to be assumed that the remaining snow load constitutes usual (snow) load.

In the case of more complicated roof shapes, the effect of snow clearance on the distribution of snow load should be based on what can be regarded as reasonable.

In designing secondary structures such as purlins, a change in the distribution of snow load as a result of snow slide or clearance is to be disregarded.

The intensity of snow load (q) per $m^2$ of horizontal roof surface is calculated from

$q = \mu q_g$

where $\mu$ = shape factor according to 4.4.1.3

$q_g$ = snow load on ground according to 4.4.1.2.

The value of $q_g$ is to be put equal to usual or unusual snow load in accordance with 4.4.1.2.

Snow load including ice load on wires and similar structures is determined by climatic conditions which in turn depend, inter alia, on geographical position and the height above ground. In the case of specially exposed locations such as open water, areas around moors and elevated country, the snow and ice load is to be determined in view of these conditions. For other areas, the following values are accepted for snow and ice load for wires and similar, the height of which above ground does not exceed 30 m.

$(0.57d + 11.3)$ kN/m

where d is the diameter of the wire in mm, without ice cover. The values are equivalent to a 20 mm ice sheath with the weight of 9 $kN/m^3$. In the case of wires located at greater heights, the snow and ice load is to be determined in view of local conditions.

The thickness of the ice sheath around wires and similar, which are located very high above the ground, can under unfavourable circumstances assume very high values.

4.4.1.2 Snow load on ground

FIG. 4.4.1.2.a gives the usual snow load on ground, and FIG. 4.4.1.2.b the unusual snow load on ground.

In determining snow load on ground, the greatest possible consideration is to be given to local variations due/to/primarily/topography and wind conditions.

4.4.1.3 Shape factors

The shape of the building is/considered in determining shape factors. to be
Where it is possible due to the shape of the roof, adjacent parts of the building or other circumstances, for snow to accumulate, the consequent increase in the snow load is to be allowed for. Such snow load is normally regarded as "bound" load.

The values of the shape factor $\mu$ given in FIGs. 4.4.1.3 a–f are accepted.

Where it is possible for snow to slide down from a higher roof, this is taken into consideration by 50% of the snow load on the higher roof being assumed to slide down. See also FIG. 4.4.1.3 d where the shape factor is divided into a part $\mu_w$ which is a function of the wind, and into a part $\mu_r$ which is a function of slide.

The specified shape factors stipulate that the roof is exposed to wind, which implies that the snow load on the lee side of the roof is greater than that on the windward side.

In areas which are permanently sheltered from the wind, see 4.4.2.2, by the surrounding country or buildings, redistribution of the snow load due to the action of the wind may be less than what is equivalent to the specified values of the shape factor $\mu$.

4.4.2 Wind load

4.4.2.1 General

Variation of the wind load in time

The maximum intensity of the wind load occurs relatively infrequently, while, in conjunction with this, intensities at a lower level of the same order of magnitude occur repeatedly.

As regards the duration of the maximum intensity of unusual wind load, the wind load is assigned to Class D according to 2.1.2. It is assumed that wind load at maximum intensity or near maximum intensity can be repeated several times. Unless it is shown by special measurements that other values are more correct, the relative duration of usual wind load and unusual wind load is assumed to be $10^{-2}$ and $10^{-5}$ respectively.

Wind load is the principal cause of dynamic effects. Where the damping of the object exposed to the wind is large and its slenderness ratio slight, which is the case for most buildings, but not in all cases for their individual parts, an equivalent static load, which is the design pressure due to wind velocity multiplied by a shape factor, is however accepted.

Wind load, both usual and unusual, is a recurrent load. In certain cases wind load can be a fatigue load.

Unless it is shown by special investigations that other values are more correct, a load spectrum according to FIG. 4.4.2.1 * is accepted.

Distribution of the wind load in space

As a rule, the wind load has a component perpendicular to the exposed face and another tangential to this. Where the surface of the exposed object is not substantially rough, and unless this is warranted by some special feature of the exposed object, it is however accepted that the tangential component is ignored.

* FIG. 4.4.2.1 has not yet been prepared.

The wind load on different parts of a building is expressed by

$w = \mu q$

where $w$ is the wind load per unit area ($kN/m^2$)

$q$ the pressure due to wind velocity ($kN/m^2$) according to 4.4.2.2.

$\mu$ a shape factor according to 4.4.2.4 ($\mu$, $\mu_t$ or $\mu_{tot}$ can be put instead of $\mu$)

With the distribution specified by $\mu$, the wind load is assumed to be a "bound" load.

The extreme consequences of an assumption concerning "bound" load are however avoided, which implies that loadbearing systems which are substantially dependent on the distribution of the load are designed in alternative loading cases which deviate in a reasonable manner in relation to these regulations.

In the case of an object which has a large extent perpendicular to the wind direction, the wind impinging on the surface may exhibit a variation in velocity over the surface.

4.4.2.2 Wind velocities and pressure due to wind velocity

Determination on the basis of observations

Wind velocity is to be determined from continuous measurements at least 10 m above a section of country which is typical of the surrounding terrain.

Measurement necessitates sophisticated instruments and advanced treatment of the measurement results, and this procedure is not, therefore, recommended for general use.

Instantaneous wind velocities

For areas for which there are no special values applicable which have been obtained by measurements or documented experience, the following is to be applied, unless another expression is found to be more correct, as an expression of the design instantaneous value of the wind velocity $v$ at a height $h$.

$$v = v_o\left(1 + \frac{1}{3}\log_{10}\frac{h}{10}\right)\psi$$

with a constant value for $h \leq 4$ m.

For h 4 m,

$v_o$ = 36 for off-shore wind
39 for on-shore wind (including the shores of Lakes Vänern and Vättern) up to10 km from the boundary between the inner and outer archipelago, with a linear reduction to 36 over a further 10 km; but, in the case of wind from the North Sea or the Atlantic, 42 up to 10 km from the boundary between the inner and outer archipelago, with a linear reduction to 36 over a further 20 km. The value of $v_o$ = 42 is also to be applied in locations such as bare mountains where wind velocities can be particularly high. The boundary between the inner and outer archipelago is a line which divides areas predominantly consisting of water from areas predominantly consisting of land.

The values specified in 4.4.2.4 are to be chosen for the height h.

For usual wind load (v2), the standard value is put $\psi = 0.86$
For unusual wind load, the standard value is put $\psi = 1.00$
In applying the statistical method, the following are to be used:
Mean value of the wind load: $\psi_m = 0.77$
Coefficient of variation: $\delta_\psi = 0.13$

More generally, the expression for $\psi$ can be written

$$\psi = 0.76 + 0.14 \log_{10} n$$

where n is the repetition time, in the case of usual load 5 years, and in the case of unusual load, 50 years.

See FIG. 4.4.2.2.

Mean wind velocities

The mean wind velocity $v_m$ is considered to be $1/\sqrt{1.75}$ of the design instantaneous value.

Wind velocities in sheltered locations

In the case of objects in locations where the wind velocity is permanently reduced, $v_o$ is to be multiplied by 0.8 when the reduction has its cause in building development or irregularities in the terrain. A reduction occurs where the following relationship holds between the height h(m) of the exposed object and the extent a(m), in the direction of the wind, of the area which gives rise to a reduction, and where the distance between
?????? the object and the area in question is not greater than 50 m ???? and is
?????? at least 500 m ?????.

$h \leqslant 0.5 \sqrt{a}$

A forest which is not protected is not normally permanent, while an urban area is. Solid natural formations such as boulders or rocks, of the specified extent and a height above mean ground level $\geqslant 3$ m, are usually permanent and give rise to a reduction in velocity.

A local shelter implies a reduction in wind velocity which is to be assessed in view of the prevailing conditions.

Pressure due to wind velocity

The design pressure due to wind velocity is to be calculated from the velocity by

$q = 0.6\ v^2$

the units being N, m and s.

4.4.2.3 Dynamic effects due to wind load

Essentially, the dynamic effects due to wind load are either oscillation perpendicular to the wind direction as a consequence of rhythmical vortex separation, or oscillation in the wind direction as a consequence of the impact effect due to variable wind velocity.

Other types of dynamic effects occur in conjunction with special structures. Weak structures of large extent in the wind direction can buffet. Hollow structures ~~of~~/with thin walls can be caused to undergo annular oscillations ?????.

???

Vortex separation

Vortex separation at a frequency f occurs at mean wind velocity

$v_m = \frac{fd}{St}$

where d is the extent of the object exposed to the wind perpendicular to the wind direction. In the case of conical towers etc, the value of d is taken to be the mean value of the width over the upper third of the height

St is the Strouhal Number. For long angular prismatic objects, St is independent of the Reynolds Number and has numerical values ranging from 0.12 to 0.18, with 0.15 being the one most useful. For long cylindrical objects it is assumed to be 0.18 for $Re < 2 \times 10^5$ and 0.25 for $Re \geqslant 2 \times 10^5$.

Re (Reynolds Number) = $vd/\nu$

where v = wind velocity

d = a characteristic dimension, e.g. the diameter of a cylindrical rod on which the wind impinges at right angles, and

$\nu$ = the kinematic viscosity of air

On average, Re = $7 \times 10^4$ vd, the units being m and s.

When $v_m$ is less than or equal to that for the area concerned and the design value appropriate for the total height according to 4.4.2.2, and when f is some natural frequency of the object exposed to the wind in a direction perpendicular to the wind direction, resonance occurs.

The stresses caused by resonance are equivalent to those caused by an equivalent load W(N) which is uniformly distributed over the length l of the object in oscillation, and is acting in the direction of oscillation. In the case of objects of round cross section, with the units m and s, the equivalent load is

$$W = 0.2 \frac{v_m^2 \, ld}{\delta_m}$$

In the case of objects whose cross section is other than round, the effect of vortex separation should be determined by measurements on dynamically and constructively representative models.

The following values of the mechanical damping $\delta_m$, chosen carefully for non-composite buildings, will be accepted.

| | |
|---|---|
| Metal structures | 0.02 |
| Concrete structures with the normal force acting inside the cross section | 0.04 |
| Concrete structures in pure bending | 0.06 |

Connections which absorb energy such as foundation on resilient ground, produce an addition to the mechanical damping.

When one of the following conditions is satisfied, the dynamic effect of the wind load is so slight that it is permissible for it to be ignored without a special check.

a/ $l \leqslant 5d$

b/ it is proved that $\delta_m > 0.1$

c/ it has been proved that tested construction of the exposed object results in the oscillation being negligible.

Gusts

The stresses caused by gusts are equivalent to those due to a static load caused by wind at mean velocity, q/1.75, which is increased by being multiplied by the gust factor $\varphi$ of the wind which can be expressed by

$$\varphi = 1 + \varphi_p \varphi_u \sqrt{\varphi_b + \frac{2\pi \varphi_a \varphi_w}{\delta}}$$

???? where $\varphi_p$ is the peak factor ???? which is a function of the natural frequency of the exposed object when in fundamental oscillation in the wind direction according to FIG. 4.4.2.3.a.

$\varphi_u$ is the roughness factor which is a function of the height of the structure and the nature of the surrounding country, according to FIG. 4.4.2.3. b. The curve applicable to built-up terrain is to be used up to a height of z m, if the building development has an extent of x km according to the table below.

| x km | z m |
|---|---|
| 0 | 0 |
| 0.5 | 70 |
| 2.0 | 220 |

Linear interpolation between the above values is permissible.

$\varphi_b$ is the dynamic effect of background turbulence and is a function of the height of the structure according to FIG. 4.4.2.3 c.

$\varphi_a$ is the size factor which is a function of the natural frequency of the exposed object when in fundamental oscillation in the wind direction, its height and width, and the mean wind velocity according to FIG. 4.4.2.3 d.

$\varphi_w$ is the relative gust energy which is a function of the natural frequency of the exposed object when in fundamental oscillation in the wind direction, and of the mean wind velocity according to FIG. 4.4.2.3 e.

$$\delta = \delta_a + \delta_m$$

$$\delta_a = 12 \frac{\mu_{tot} \cdot A_1 v_m}{M_1 f_o}$$

$A_1$ = the projection, perpendicular to the wind direction, of the area per one metre height of a tower-shaped building near its top

$M_1$ = the mass per one metre height of a tower-shaped building near its top

$\mu_{tot}$ is to be chosen according to 4.4.2.4.

$\delta_m$ see the section on vortex separation.

4.4.2.4 Shape factors for wind load perpendicular to the exposed surface

The shape factors are to be determined by means of observations on the object under the action of wind load in the appropriate environment or in a model of this which is correct from the aerodynamic point of view.

One type of error which is often found in tests and published test results is associated with the model wind which has an erroneous boundary layer or may have none at all in extreme cases.

The values in the following sections give approximations intended for use as the basis of primary strength or stability analysis. They do not give such a detailed description of conditions which are likely to occur that they can be used generally as the basis of design for the fastenings of facade sheeting or such detail dimensions which determine e.g. water seepage in joints, or as the basis of design of ventilation plant.

It is for instance possible for the design pressure (not suction) used for facade sheeting and the fastenings for such sheeting, to exceed by about 30% locally the approximate values.

Internal wind load in houses

The wind load acting inside a house is determined by the pressure,due to wind velocity, outside the house and by a shape factor. When the external envelope of the house has the usual impermeabilities in the form of chimneys, ventilation openings and similar, such as gaps around doors and windows and between building components, this shape factor is equivalent to suction in one or more of the rooms of the house.

Irrespective of the wind direction, it is accepted that the value of $\mu = 0.3$ (suction) is valid for external walls and roofs. In the case of internal walls, the value of $\mu_{tot} = 0.4$ is accepted for any direction.

Where one of the external boundary surfaces has some large opening, either permanently/or temporarily with a high degree of probability, the internal wind load is determined according to FIG. 4.4.2.4.a.

A door through which a large number of people pass or through which there is frequent movement of goods, vehicles etc, is considered to entail a high degree of probability of a large opening.

External wind load on houses

The wind load acting externally on a house is governed by a shape factor determined, inter alia, by the direction of incidence. The values specified below relate to houses the height of which does not exceed 3 times their largest horizontal dimension.

External wind load on external walls.

Shape factors according to FIG. 4.4.2.4.b are accepted for external walls. The figure states the height which is to be used according to 4.4.2.2.

External wind load on roofs.

The same wind loads acting from underneath are accepted in the case of eaves as for the walls below the eaves, according to FIG. 4.4.2.4.b. In the case of wind load on a corner, a pressure with $\mu = 0.7$, acting from underneath, is accepted on both sides of the corner in question.

In the case of roofs, the wind load applicable to which is to be increased in a specified boundary zone, it is accepted that this increase has such a small distribution that it has design significance only with regard to the actual fastenings of the external roof covering, consisting e.g. of screws or glued joints. The increase does not, therefore, affect the loadbearing members which may be e.g. sheeting on purlins carried by roof trusses. Nor is the increase included more than locally, e.g. around roof tiles etc, in the equalisation of pressure around the roof and roof covering which is due to impermeabilities and openings.

Shape factors according to FIG. 4.4.2.4c are accepted for pitched roofs. The figure specifies the height which is to be used according to 4.4.2.2.

Shape factors according to FIG. 4.4.2.4.d are accepted for pen roofs. The figure specifies the height which is to be used according to 4.4.2.2.

Shape factors according to FIG. 4.4.2.4.e are accepted for curved roofs. The figure specifies the height which is to be used according to 4.4.2.2.

Wind load on stacks, screens, lattice masts, etc

Wind load on stacks

The wind load on a stack is determined at every level by the distance h above the ground and by $\mu_{tot}$ according to this section. Near a free end, however, $\mu_{tot}$ assumes values lower than those specified here.

As a reduction over the length which is 3 times the width perpendicular to the wind direction, multiplication of $\mu_{tot}$ by 0.6 in the case of rectangular stacks, and by 0.8 in the case of cylindrical ones, is accepted.

In the case of stacks of rectangular cross section and of a length greater than or equal to 5 times the largest lateral dimension d, which are exposed to wind pressure on the side of width $b_2$, the wind load has a resultant equal to $\mu_{tot}$ $b_2$ per 1 m length of the stack in the direction of the wind.

Values according to FIG. 4.4.2.4.f are accepted for $\mu_{tot}$.

In the case of stacks of square cross section and of a length greater than or equal to 5 times the largest lateral dimension d, the resultant of the wind load on b per 1 m length of the stack is independent of the direction of wind.

In the case of prismatic stacks with a cross section in the shape of an equilateral polygon with n sides, with the diameter of the circumscribing circle equal to d and with the length greater than or equal to 5d, the resultant of the wind load in the direction of wind is equal to $\mu_{tot}$ d per 1 m length of the stack.

The value of 1.8 for n = 5 and the value of 1.1 for n = 12 is accepted for $\mu_{tot}$. Linear interpolation is to be carried out between these values.

In the case of angular stacks, e.g. rolled sections, the value of $\mu_{tot}$ is approximately 2. Generally speaking, the resultant of the wind load is not along the direction of wind.

In the case of cylindrical stacks of length greater than or equal to 5d, the resultant of the wind load in the direction of wind is equal to $\mu_{tot}d$ per 1 m length of the stack.

Values according to FIG. 4.4.2.4g are accepted for $\mu_{tot}$. The height u of surface roughness on a rolled steel section after painting may be assumed to be equal to 0.2 mm.

The distribution of the wind load intensity about a stack of circular cross section is shown in FIG. 4.4.2.4.h for $\mu_{tot}$ = 0.7, corresponding to $u/d = 1x10^{-3}$.

Wind load on screens

In the case of screens carried on angular posts, the resultant of wind load in the wind direction is determined by $\mu_{tot}$ and the gross area (the area enclosed by the external contours) of the screen, projected onto a plane perpendicular to the wind direction. The height h according to 4.4.2.2 is to be measured in the case of screens without contact with the ground to the level equivalent to the mean value of the pressure due to wind velocity, and in the case of screens with contact with the ground, to the top of the screen.

Values according to FIG. 4.4.2.4i are accepted for $\mu_{tot}$ in the case of a screen with contact with the ground, but only if its height is not more than twice as large as its length. In the case of screens on cylindrical posts, the specified values of $\mu_{tot}$ are somewhat too large.

The wind load is to be reduced in the case of a screen located in the lee of one or more similar screens.

Multiplication by $K$ according to FIG. 4.4.2.4 k is accepted as an expression of this reduction.

Wind load on lattice masts with equal sides

At every level, the height h according to 4.4.2.2 is the actual height, $\alpha$ is the same for all sides, and the wind direction is arbitrary.

In the case of lattice masts with angular members, with a square section and with $0.1 \leq \alpha \leq 0.5$,

$$\mu_{tot} = 4.4\ (1 - \alpha)\alpha \qquad (4.4.2.4a)$$

and A is the gross area of the side of the mast.

In the case of lattice masts with cylindrical members, with a square section, with the corner members larger than the other members, and with $0.1 \leq \alpha \leq 0.3$, when, for all members, $Re \leq 5x10^5$,

$$\mu_{tot} = 2.6\ (1 - \alpha)\alpha \qquad (4.4.2.4b)$$

and A is the gross area of the side of the mast.

Where, in the latter case, the cross section of the mast is an equilateral triangle, we have analogously

$$\mu_{tot} = 2.1\ (1 - \alpha)\alpha \qquad (4.4.2.4c)$$

Where, in the case of some members, $Re > 5x10^5$, $\mu_{tot}$ is not greater than according to (4.4.2.4b) or (4.4.2.4c).

Wind load on canopies

Canopies made up of one or more planar surfaces are assumed, irrespective of actual conditions, not to have any of their parts inclined at less than $\beta = 5^{o}$ to the horizontal.

Values according to FIG. 4.4.2.41 are accepted for $\mu_{tot}$ and for $\lambda$ which indicates the position of the resultant wind load, provided that the clear height below the canopy is at least one half the horizontal width of the canopy. The height h according to 4.4.2.2 is measured to the highest edge of the canopy.

The wind load on the canopy, to be used for design of the fastenings of the external roof covering, is determined by $\mu_{max} = 2$.

Wind load on bridges

The wind load on a bridge constructed of trussed or solid girders is determined, as appropriate, by the rules stated under "Wind load on stacks, screens, lattice masts, etc". In the case of traffic load on a bridge, the value $\mu_{tot} = 1.8$ is applied.

Unless other values are evidently valid, the height of the traffic load is assumed to be 3.8 m in the case of railway bridges, 2.0 m in the case of road and street bridges, and 1.7 m in the case of pedestrian and cyclist bridges.

In the case of a solid bridge according to FIG. 4.4.2.4 m, the value accepted is

$$\mu_{tot} \begin{cases} = \dfrac{b_1}{a} + 1.4 \\ \leqslant 2.2 \end{cases} \qquad (4.4.2.4d)$$

The wind load acting on handrails etc is to be added to the wind load according to (4.4.2.4d).

The load W according to (4.4.2.4d) formally acts over the length e along the underside of the bridge. The most critical situation according to (4.4.2.4d) arises when

$$0.5\ b_1 \leq e \leq 5\ b_1\ (1.1 - \frac{b_1}{a})$$

In the case of special types of bridges and particularly when there are dynamic effects to consider, the principal rule at the head of section 4.4.2.4 is to be applied.

4.4.2.5 Shape factors for tangential wind load

The height h according to 4.4.2.2 is to be measured to the actual height.

The magnitude of the tangential component of the wind load is a function of the roughness of the surface concerned. The following values will be accepted.

| | |
|---|---|
| Smooth concrete surface | $\mu_t = 0.006$ |
| Surface with transverse ribs or corrugations according to FIG. 4.4.2.5a | $\mu_t = 0.12 \frac{l_1}{a} (\frac{a}{l_2})^{\frac{2}{3}}$ |
| Facade with inset balconies according to FIG. 4.4.2.5b | $\mu_t = 0.05$ |
| Facade with doors and windows which is otherwise smooth | $\mu_t = 0.01$ |

It is assumed that only one facade at a time is acted upon by tangential wind load.

4.4.3 Water pressure

Water pressure due to natural water level in regulated or unregulated watercourses and lakes or the sea is classified with regard to the variation in time of the water pressure, on the basis of observation at the location concerned. With regard to variations in space, water pressure is classified as "bound" load. With regard to the probability of a certain intensity of load, water pressure is classified as follows:

Water pressure at mean water level (MW) is classified as constant load (k).

The difference between water pressure at low water level (LW) or high water level (HW) and that at mean water level is classified as non-short-term usual load (v2).

The difference between water pressure at lowest low water level (LLW) or highest high water level (HHW) and that at mean water level is classified as unusual load (ov).

In applying the statistical method, the assumptions concerning mean values and the coefficient of variation must be based on observed values of the high and low water levels over one year.

It should be noted that mean water level is not normally equivalent to the mean value of the water pressure according to the definitions given here.

4.4.4 Ice pressure

A structure situated in water is assumed to be acted upon by ice pressure due either to a change in the temperature of a stationary ice sheet, pressure on a stationary ice sheet due to flow of water, or drifting ice. The ice pressure is normally assumed to act horizontally and at the level of the water surface. The magnitude of ice pressure is determined in view of local conditions and the shape of the structure. The magnitude of the ice pressure which is used as the basis of design is to be chosen in such a way that it is equivalent to unusual load. Ice pressure of this magnitude is assigned, with regard to its duration, to Class B according to 2.1.2.

Ice pressure is static load and non-recurrent load. Ice pressure is "free" load, although there is no need for the load to be assumed divided into part loads with intermediate unloaded parts.

The value of 50-150 kN/m is accepted for unusual ice pressure due to temperature rise, in lakes and calm watercourses with comparatively favourable conditions. The highest value is to be applied to the northern parts of Finland and Sweden, and the lowest to Denmark.

These values do not as a rule apply to bridges where conditions are often unfavourable and a higher ice pressure must be allowed for, due, inter alia, to the effect of drifting ice. In the case of structures other than bridges where conditions are equivalent to those applicable to bridges, it is appropriate to apply the values of ice pressure applicable to bridges.

4.5 Loads due to accidents

4.5.1 General

In most cases, loads due to accidents are short-term loads which can be assigned to Class C or D according to 2.1.2, but can also in certain cases (e.g. in conjunction with flooding) be assigned to Class B.

The relative duration can in most cases be put equal to zero.

It is stipulated that after the occurrence of a load due to an accident, measures are taken to eliminate the consequences, if any, of this with regard to the state of the structure.

A load due to an accident is in most cases regarded as a "free" load.

On being classified according to 2.3, a load due to an accident is regarded as an extreme load (e), unless there are special conditions to warrant otherwise.

In the following sections, loads due to accidents in the form of collisions with a vehicle, unintentional impact and explosion, will be dealt with.The instructions given in these section are intended for use as a guide in choosing values of load in other cases.

In applying the statistical method, the specified standard values can be used as deterministic values, i.e. the standard value is assumed to be equal to the mean value and the coefficient of variation is assumed to be equal to zero.

4.5.2 Collision with a vehicle

Unless a more accurate analysis is performed, the following method is accepted.

In conjunction with a collision by a vehicle, parts of the structure (columns, walls, beams etc) are assumed, where it is stipulated that they completely prevent the movement of the vehicle, to be acted upon by a horizontal force F at an arbitrary position within the range of 1.0-2.0 m, measured vertically, from the plane on which vehicular traffic takes place.

The force F can be assumed to arise due to the fact that the vehicle has kinetic energy W and its motion is retarded over a distance which is equivalent to the deformations in the vehicle.

The magnitude of the kinetic energy $W_s$ is assumed, in the case of unhindered unintentional entry into a building, to decrease from the value $W_o$ at the boundary of the area traversed by traffic to the value 0 over a distance $s_o$ as shown in FIG. 4.5.2.a. The force F is assumed to decrease from the value $F_o$ to 0 according to

$$F = F_o \sqrt{\frac{W_s}{W_o}}$$

If a structural component is proportioned in such a way that, on being hit by a vehicle, it is capable of resisting a force less than F, then it is considered to be completely destroyed on being hit and to lose its load carrying capacity for all kinds of loads.

If the vehicle is supposed to cause a number of columns to fail before it stops, its kinetic energy is reduced in consequence

partly by the amount of energy $W_k$ which the destroyed columns can take up prior to failure,

partly by the amount of energy $W_f$ which has been absorbed due to deformations in the vehicle. The total magnitude of this energy may be assumed to be given by

$$W_f = \left(\frac{F_m}{F_o}\right)^2 W_o$$

where $F_m$ is the greatest force developed by any of the columns, including the one which stops the vehicle.

The principle is illustrated by FIGs. 4.5.2 b and c. FIG. b shows the case where the first column has the maximum capacity $F_m$ to absorb force. FIG. c shows the case where the column which finally stops the vehicle has the maximum capacity to absorb force.

For some usual cases, values which are accepted in applying the above method, are given below.

a/ For a building along a traffic route carrying normal road or street traffic - or where such traffic may occur - it is assumed that

$F_o = 1500$ kN $\qquad W_o = 1500$ kNm

$s_o = 25$ m

$s_o$ is measured from the boundary of the carriageway.(see FIG. 4.5.2 d). Entry by a vehicle into a building is assumed to be possible in the directions given in FIG. 4.5.2 d. In order that unhindered entry by a vehicle into a building may be supposed, it is necessary that there is a clear space of width 3 m and height 2 m.

b/ For a building along an area where normal road or street traffic does not occur, nor can it be assumed to occur, but where vehicular traffic can nevertheless occur, it is assumed that

$F_o = 400$ kN $\qquad W_o = 100$ kNm

$s_o = 5$ m

Such areas are courtyards, garden and park areas etc where maintenance vehicles and delivery vehicles may occur, as well as areas referred to under a/ but where the forces according to b/ are greater.

s in this case is at all times measured from the boundaries of the building in the plane on which the vehicle drives.

Entry by a vehicle is assumed to be possible in the directions shown in FIG. 4.5.2 d. In order that unhindered entry by a vehicle into a building may be supposed, it is necessary that there is a clear space of width 3 m and height 2 m.

c/ For a building where traffic occurs inside the building, it is assumed that

$F_o = 150$ kN $\qquad W_o = 20$ kNm

$s_o = 2$ m

s is measured from the boundary of the area on which traffic is possible, as in FIG. 4.5.2 d.

The conditions are otherwise the same as in b/.

d/ In the case of a multistorey garage for cars it is assumed, when a/ or b/ are not applicable, that

F = 40 kN

### 4.5.3 Unintentional impact

It is assumed that all accessible structural components in a building are subject to individual horizontal impact forces F (due to causes other than collision with a vehicle) which have an arbitrary position and the magnitude

F = 20 kN

This value applies in the case of residential and office buildings, hospitals, schools etc where there is no particular reason for larger impact forces to occur. In industrial buildings and similar buildings, the magnitudes of the impact forces which may occur are to be determined in each individual case. For instance, in buildings where there are overhead cranes, larger impact forces can occur as a result of load oscillation.

### 4.5.4 Explosion

Where necessary, a building frame is to be designed with regard to the different forms of explosion which may occur in view of the activity in and around the building. In the case of buildings where explosives or other explosive substances are handled or stored in such quantities that risks with regard to the state of the building frame may occur, the design assumptions are to be chosen in each individual case in view of conditions. The same applies to buildings which contain large pressure vessels. In other buildings, it is considered that the principal risks are associated with gas explosions. It is assumed that it is possible for explosive gas mixtures to form in most types of premises, e.g. due to leaks from gas pipes, to unintentional evaporation of volatile liquids (e.g. petrol) or to vaporisation of the external cladding or surface finishes in conjunction with unintentional rises in temperature (e.g. in conjunction with fire).

The following is accepted as a reasonable way of assessing pressure conditions in conjunction with gas explosions. In conjunction with explosions, the gas pressure is assumed to vary in time according to FIG. 4.5.4.a. The time $t_1$

for pressure rise is to be chosen in the most unfavourable manner within the interval 0.1–1,0 s. The time $t_2$ for pressure drop is to be chosen within the interval 0.1–10 s, the longer times being applicable in the case of spaces where reduction in pressure as a result of ventilation is slight.

The maximum pressure $p_o$ is assumed to be a function of the volume V of the room where the explosion occurs, as shown in FIG. 4.5.4 b, and of the possibility of pressure reduction due to certain weak portions of the walls enclosing the room being pressed outwards. Only windows and comparable elements are regarded as such portions. The pressure $p_o$ is assumed to be a function of the factor $\theta$, where

$$\theta = \frac{A_1}{A}$$

where

$A_1$ is the total window area, and
A is the total area of enclosing walls, ceiling and floor.

The pressure computed in this way is assumed to act on, and be transmitted by, all the enclosing surfaces of the room, and therefore also by such secondary building elements which cannot be assumed to withstand the pressure. The forces transmitted by the secondary building elements to a loadbearing structural component are however limited to values of these forces which can be resisted and transmitted by these secondary elements (in the ultimate limit state).

It is assumed that explosion does not occur simultaneously in a number of closed rooms. By opening the doors between a number of small rooms, a larger closed room can be obtained. In order to limit the extent of calculations, it is permissible for a reasonable selection of rooms, which are assumed to become filled with gas, to be made. In the case of residential buildings, it can be assumed, for instance, that both individual rooms and a whole flat will become filled with gas.

4.6 Deformation action

4.6.1 Action due to temperature

This relates to the primary deformations which arise as a consequence of temperature expansion and the forces, moments or stresses which occur when such deformation is prevented.

Assumptions regarding temperature processes are given in section 5.2.

4.6.2 Action due to moisture

This relates to the primary deformations which arise as a consequence of the shrinkage and swelling of materials caused by moisture conditions. Assumptions regarding moisture conditions are given in section 5.3.

5 ENVIRONMENTAL ACTIONS

5.1 General

In designing a structure, the different kinds of environmental actions which may occur are to be taken into consideration. Sections 5.2 and 5.3 contain regulations and instructions concerning temperature and moisture conditions. Other environmental action is chemical action, biological action and action as a result of radiation.

5.2 Temperature action

5.2.1 Inside temperature

In constantly heated premises with satisfactory thermal insulation such as dwellings, offices, hospitals, schools, department stores and similar, the mean temperature is assigned, as regards its duration, to Class A according to 2.1.2. The relative duration is put at $\eta = 1$. The mean temperature is regarded as constant action (according to 2.3). Temporary variations in temperature about the mean value, caused by temporary high outside temperatures, temporary stoppages in heating, etc need only be taken into consideration in cases where the structure is considered to particularly sensitive to such variations.

+20°C is accepted as the mean value of inside temperature.

In the case of unheated premises, it is assumed that the inside temperature will to some extent follow the outside temperature. Premises which have no heating at certain times during the cold part of the year or have only limited heating facilities, are also regarded as unheated premises. In the case of buildings whose enclosing surfaces have a great capacity for emission or absorption of radiation, consideration is also to be given to the effect of radiation.

Unless a special investigation is carried out, it is permissible for the temperature in unheated premises with slight thermal insulation and small thermal capacity to be assumed to follow the diurnal mean value of the outside temperature. In the case of unheated premises with satisfactory thermal insulation and high thermal capacity, it is permissible for the temperature to be assumed to follow the 3-day mean value of the outside temperature.

5.2.2 Outside temperature

5.2.2.1 General

Assumptions regarding outside temperature (air temperature) are to be based on observations in the area in question. If there are no such observations or they are not utilised, the following is accepted.

5.2.2.2 Mean temperature

The mean temperatures are set out in FIG. 5.2.2.2.

5.2.2.3 High and low temperatures

Characteristic values of the mean temperature over a certain period (one month, 7 days, 1 day and 1 hour) are given below. The specified values relate to usual action. In accordance with 2.3, the characteristic value stated is that mean temperature during the continuous period in question which has the probability of 0.2 of being exceeded (in the case of high temperatures) or not reached (in the case of low temperatures) at least once in one year. It is thus to be expected that the characteristic value will be exceeded, or not reached, on average once every five years.

Characteristic values of high mean temperatures are given in FIGs. 5.2.2.3 a-c.

FIG. a gives the monthly mean value
FIG. b the 7-day mean value
FIG. c the diurnal mean value

The one-hour value (= the maximum temperature) is assumed to be 5°C higher than the diurnal mean value.

Characteristic values of low mean temperatures are given in FIGs. 5.2.2.3 d-f.

FIG. d gives the monthly mean value
FIG. e the 7-day mean value
FIG. f the diurnal mean value

The one-hour value (= the minimum temperature) is assumed to be 5°C lower than the diurnal mean value.

The relationships between characteristic values of temperature and the relative duration are given in

FIG. 5.2.2.3 g (high temperatures), and
FIG. 5.2.2.3 h (low temperatures).

5.2.2.4 Temperature variations

The temperature variations given below are classified as usual action.

Characteristic values of the diurnal variation in temperature (the difference between the highest and lowest temperature during one and the same day) are given in FIG. 5.2.2.4 a.

Characteristic values of the variation in temperature over 7 days are given in FIG. 5.2.2.4 b.

Characteristic values of the variation in temperature over one month are given in FIG. 5.2.2.4 c.

Large temperature variations over a certain period are often associated with high or low mean temperatures over the same period.

5.2.3 Special cases

In special cases where the activity in and near the building gives rise to special temperature conditions, the assumptions concerning the temperature processes are to be based on information regarding the kind of activity. Classification is to be based on the principles in Chapter 2.

Such cases are, for instance, certain industrial plants, water towers, tanks in sewage treatment works, winter sport installations, cold stores, etc.

5.2.4 Temperature action in conjunction with fire

Temperature conditions in conjunction with a fire are to be assessed on the basis of information concerning fire load and the thermal properties of the building. Temperature action as a result of fire is to be classified as extreme action according to 2.3. With regard to load combinations in conjunction with fire, see Chapter 3.

Further information concerning determination of the temperature process in conjunction with fire is to be found in ---

5.3 Action due to moisture

5.3.1 Internal climate

Determination of the moisture conditions inside a building are to be based on information concerning location, degree of heating, ventilation etc.

In most rooms in dwellings, offices, hospitals, schools, department stores, as well as in similar constantly heated premises where no special measures have been taken to regulate moisture conditions, the values of relative humidity during different parts of the year, given in FIG. 5.3.1, are accepted. The duration of the action is to be assessed on the basis of the figure. The action is to be regarded as non-short-term usual action.

5.3.2 External climate

Assumptions concerning humidity outdoors are to be based on observations in the area concerned. If there are no such observations or they are not utilised, the following will be accepted.

In FIGs. 5.3.2 a-g are given characteristic values, relating to usual action, for the relative humidity outdoors. FIGs. a and b give the annual mean value for high and low humidity respectively. FIGs. c and d give, in the same way, the monthly mean values, and FIGs. e and f the 7-day mean values. Variations over one day are given in g. It must be pointed out that the values specified relate to average conditions. Large variations may occur due to location.

### 5.3.3 Special cases

In special cases where the activity in and near the building gives rise to special moisture conditions, the assumptions concerning moisture are to be based on information regarding the kind of activity. Classification of the action is to be based on the principles in Chapter 2.

FIG. 2.1.4.a. Load spectrum

FIG. 2.1.4.b. Simplified description of load spectrum

| | | |
|---|---|---|
| Beam-column system | <u>Main beams</u> | A |
| Secondary beams | Centre beam support moment | $2\, l_1 \cdot 2\, l_2$ |
| Main beams | " " span moment | $l_1 \cdot 2\, l_2$ |
| Slabs | Outside beam1 support moment | $2\, l_1 \cdot l_2$ |
| | span moment | $l_1 \cdot l_2$ |
| | <u>Secondary beams</u> | |
| | Support moment | $2c \cdot 2\, l_2$ |
| | Span moment | $2c \cdot l_2$ |
| | <u>Columns</u> | |
| | Centre row | $2\, l_1 \cdot 2\, l_2$ |
| | Outside row | $2\, l_1 \cdot l_2$ |
| | $m_x$ support moment | $b \cdot 2\, l$ |
| | span moment | $b \cdot l$ |
| | $m_y$ span moment | $b \cdot l$ |

Normally, b = 1 for solid slabs. In the case of prefabricated slab components, b = the width of the component. If there are special arrangements which produce interaction between the components at the ultimate stage, b may be made larger than the width of the component.

Note that the expressions given for loaded area are intended only for determination of the value of load per unit area. They are <u>not</u> intended to show, e.g., how the load on a column is to be calculated.

In applying the limit state theory, the support moment and span moment are often not determined individually. In such cases the area indicated for the span moment should be used.

FIG. 4.3.1.1. Loaded area A for determination of the magnitude of the load portion b.

FIG. 4.3.2 Forces due to bulk products

P = 180 kN

incl. dynamic allowance

Loaded lane

Loaded area

FIG. 4.3.3.2

FIG. 4.4.1.2a Usual snow load on ground in kPa

FIG. 4.4.1.2b Unusual snow load on ground in kPa

Linear interpolation with $\alpha$

Linear interpolation with $\alpha$

Linear interpolation with $\alpha$

For a non-symmetrical pitched roof, each half of the roof is to be regarded as one half of a symmetrical pitched roof.

FIG. 4.4.1.3a

Alternative 1

Alternative 2

For $\alpha \leq 15^\circ$, only alternative 1 is applicable
For $\alpha > 15^\circ$, both alternative 1 and alternative 2 must be investigated.

FIG. 4.4.1.3c

h in m and $q_g$ in kPa

Restrictions

$\mu_r$ is to be determined so that $q_g \mu_r l_2/2$ is equivalent to 50% of the snow load which can be supposed to slip down from the upper roof.

FIG. 4.4.1.3d

$\mu_1$ and $\mu_2$ are to be calculated according to FIG. 4.4.1.3d

FIG. 4.4.1.3e

h in m and $q_g$ in kPa

Restrictions

FIG. 4.4.1.3f

FIG. 4.4.2.2. Mean wind velocity $v_m$, design instantaneous value $v_h$ of the wind velocity for unusual wind, and the pressure q due to velocity, corresponding to $v_h$

??? Peak factor ???

FIG. 4.4.2.3a The peak factor of the wind load as a function of the natural frequency $f_o$ of the exposed structure when in fundamental oscillation

Roughness factor

Built-up country
Open country

FIG. 4.4.2.3b The roughness factor of the wind load as a function of the height h of the structure above the ground, and the nature of the surrounding country

Stimulus due to background turbulence

Height of structure above ground, h m

FIG. 4.4.2.3c Stimulus derived by the wind load from background turbulence, as a function of the height h of the structure above the ground

Size factor

FIG, 4.4.2.3d The size factor of the wind load as a function of $f_o h/v_m$, where $f_o$ is the natural frequency for fundamental oscillation, h the height of the exposed object, d its largest lateral dimension. and $v_m$ the mean velocity of the wind

Relative impact energy

FIG. 4.4.2.3e The relative wind impact energy as a function of $f_o/v_m$ $(m^{-1})$, where $f_o$ is the natural frequency for fundamental oscillation and $v_m$ the mean velocity of the wind.

$\frac{b}{2}$ but always $\leqslant \frac{h}{2}$

FIG. 4.4.2.4b Shape factor for external wind load on external walls

Increase in boundary zone

FIG. 4.4.2.4c Shape factors for external wind load on pitched roofs
Suction is denoted –
Pressure is denoted +

Increase in boundary zone

FIG. 4.4.2.4d Shape factors for external wind load on pen roofs
Suction is denoted -
Pressure is denoted +

FIG. 4.4.2.4e Shape factors for external wind load on curved roofs
Suction is denoted -
Pressure is denoted +

FIG. 4.4.2.4f Shape factor for rectangular stacks with $l \geqslant 5d$. $\lambda = b_1/b_2$

FIG. 4.4.2.4g Shape factor for cylindrical stacks with $l \geqslant 5d$. Re is the Reynolds Number and u the height of surface roughness over the surrounding area

FIG. 4.4.2.4h Distribution of the intensity of wind load around a stack of circular cross section when $\mu_{tot} = 0.7$. + denotes pressure and - denotes suction

FIG. 4.4.2.4i Shape factor for a screen carried on angular posts. The mean value of the pressure due to velocity, according to 4.4.2.2, determines the height h. $\alpha = A_p/A$, where $A_p$ is the net area of the posts in the screen which are acted upon by the wind, and A thc gross area of the screen.

$$\lambda = \frac{b_1}{b_2} \geqslant 1 \quad \text{or} \quad \lambda = \frac{b_2}{b_1} \geqslant 1$$

In the case of screens in contact with the ground, the figure is valid only for $b_1/b_2 = h/b_2 \leqslant 2$.

FIG. 4.4.2.4k Shape factor for a screen in the lee of another. The shape factor according to FIG. 4.4.2.4i is to be reduced by multiplication by $k$

FIG. 4.4.2.4l The shape factor $\mu_{tot}$ and the position of the resultant according to $\lambda$ for canopies which are inclined $\beta$ to the horizontal. $\beta$ is assumed to be $\geqslant 5^{\circ}$.

All the data is valid both for the situation shown and for that with the canopy upside-down.

FIG. 4.4.2.4m Solid bridge or box section bridge. Symbols for calculation of the wind load W and its position, determined by e.

FIG. 4.4.2.5a Surface with transverse ribs or corrugations. Symbols for calculation of tangential wind load

FIG. 4.4.2.5b Facade with inset balconies

Constant
load k

FIG. 4.4.3

Boundary line
for traffic

FIG. 4.5.2a

Column 1 Column 2 Column 3

FIG. 4.5.2b

Column 1 Column 2 Column 3

FIG. 4.5.2c

Direction of entry (straight line)

Boundary line for traffic

FIG. 4.5.2d

time

FIG. 4.5.4.a

FIG. 5.2.2.2 Mean temperatures (determined on the basis of observations over 60 years)

FIG. 5.2.2.3f Characteristic values of low diurnal mean temperature (determined on the basis of observations over 60 years)

Relative
Humidity %

FIG. 5.3.1 Relative humidity indoors

Table 4.1. Summary of the properties of loads

The table lists the normal cases. Cases which occur less often are given in brackets. Exceptions (specified in the appropriate section) are shown in the Remarks column, or may not be shown at all

| Load: Kind of load | Load: Section | Load: Load | Classification according to 2.3 | Classification with regard to variation in time: Duration: Class | Duration: Rel. dur. | Static=S Dynamic=D | Non-recurrent load = En Recurrent load=Fl Fatigue load=U | Classification with regard to variation in space "Bound" load = B "Free" load = F | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| Weight of building components and earth | | Weight of building components | | | | | | | Exceptions |
| | | Weight of building components when handled | | | | | | | |
| | | Weight of earth | | | | | | | Exceptions |
| Useful load | | Load due to furnishings and persons | | | | | | | |
| | | Dist.load, portion a<br>" " " b | | | | | | | |
| | | Conc.load, furnishings<br>" " persons | | | | | | | Only certain structures |
| | | Load due to goods and bulk products | | | | | | | |
| | | Load due to vehicles and transport appliances<br>Load due to machinery | | | | | | | |
| Natural load | | Snow load<br>Wind load<br>Water pressure<br>Ice pressure | | | | | | | Free load porti on considered in certain cases |
| Load due to accidents | | Collision<br>Unintentional impact<br>Explosion | | | | | | | Not wholly bound load |
| Deformation | | Action due to temperature<br>Action due to moisture | | | | | | | Applies only to certain parts of the structure |

* indicates that the property concerned must be assessed in view of conaitions.

Table 4.2.1. Weight of building materials and structures

| Material | Mean value $f_m$ $kN/m^3$ | Coefficient of variation $V_f$ % | Remarks |
|---|---|---|---|
| Natural stone | | | |
| Basalt | | | |
| Granite, gneiss, marble, limestone | | | |
| Slate | | | |
| Sandstone hard | | | |
| loose | | | |
| Concrete and mortar | | | |
| Concrete mix (not yet set) | | | |
| Concrete plain | | | |
| normally reinforced | | | |
| heavily reinforced | | | |
| Cement mortar | | | |
| Lime-cement mortar | | | |
| Lightweight concrete | | | |
| Cellular concrete units | | | |
| Clinker concrete | | | Moisture content |
| Brickwork and blockwork | | | |
| Concrete wall blocks and solid concrete blocks | | | |
| Hollow concrete blocks | | | |
| Cellular concrete blocks | | | |
| Clinker blocks | | | |
| Sandy limestone | | | |
| Bricks | | | |
| Bitumen etc | | | |
| Bitumen | | | |
| Bituminous road surfacing | | | |
| Metals | | | |
| Steel | | | |
| Cast iron | | | |
| Aluminium and its alloys | | | |
| Copper | | | |

Table 4.2.1 continued

| Timber and timber products | |
|---|---|
| Pine or spruce | Moisture content 1/ |
| Birch | |
| Oak or beech | |
| Plywood of pine or spruce | |
| Plywood of birch | |
| Chipboard | |
| Wood fibre board, hard | |
| semi-hard | |
| porous | |

1/ With an accuracy sufficient for calculation of weight, it can normally be assumed that an increase in moisture content by 10% (e.g. from 12% to 22%) causes 10% increase in weight

| Cladding, insulation and fill material | | |
|---|---|---|
| Asbestos cement, compressed | | Moisture content |
| not compressed | | |
| Asbestos cellulose cement, compressed | | |
| Plasterboard | | |
| Glass | | |
| Wood wool slabs | | Dry |
| Clinker, dry | | |
| moist | | |
| cement-bound, dry | | |

Table 4.2.3 Weight of earth

| Soil type | Mean value | Coefficient of variation | Remarks |
|---|---|---|---|
| Broken rock | | | |
| Moraine | | | |
| Gravel and sand, loosely packed | | | |
| natural moisture | | | |
| saturated | | | |
| Gravel and sand, compacted graded | | | |
| natural moisture | | | |
| saturated | | | |
| Clay and silt | | | |

Table 4.3.1.1a. Loads due to furnishings and persons, distributed load. Characteristic values.

| Type of premises | Load portion a | Load portion b | | | Remarks |
|---|---|---|---|---|---|
| | | Non-short-term usual load | Short-term usual load | Unusual load | |
| 1 All rooms in dwelling houses of one or two storeys<br>Rooms in other dwelling houses and hotels<br>Patients' and staff rooms in hospitals etc | | | | | |
| 2 Offices, instruction rooms in schools | | | | | |
| 3 Shops, department stores, assembly halls stands with seats | | | | | |
| 4 Balconies, terraces, stands without seats dance halls | | | | | The values of load have been raised by 33% to allow for dynamic effects |
| 5 Courtyard slabs without vehicular traffic, roof terraces | | | | | |
| 6 Attic spaces and permanent stairs to attic spaces | | | | | In the case of stairs, the whole load (1.0 kN/m$^2$ is assumed to be short-term usual load |
| 7 Spaces for personal transport, stairs, corridors etc which belong to premises of all types, with the exception of type 6 | | | | | |

Table 4.3.1.2

| Structure | Direction of force | Loaded area | Mean value of force, kN | Characteristic value of force: Magnitude kN | Characteristic value of force: Group according to 2.3.1 | Remarks |
|---|---|---|---|---|---|---|
| Floor slabs | vertical | | | | unusual | |
| Balconies etc | vertical | | | | unusual | |
| Stairs | vertical | | | | unusual | |
| Roof which supports a person | perpendicular to roof surface | | | | | For a roof inclination of $\alpha^{o}$, the force is to be multiplied by $\cos^2\alpha$. |
| difference in level | | | | | unusual | Linear interpolation fordifferences in level between 0.2 and 0.8 m |
| Roof which does not support a person | | | | | extreme | |
| Walls, balcony fronts, etc | horizontal | | | | extreme | The force acts 0–1.5 m above the floor |
| Handrails etc | horizontal | | | | extreme | Loaded length = 0.2 m |

Table 4.3.1.1b. Loads due to furnishings and persons, distributed load.

Mean values and coefficients of variation

| Type of premises Designation according to Table 4.3.1.1.a | Mean value of load when loaded area is | Coefficient of variation | Proportion of non-short-term load, % |
|---|---|---|---|

Table 4.3.2a. Weights and angles of internal friction of bulk goods

| Kind of bulk goods | Weight | Angle of internal friction |
|---|---|---|
| Solid fuels | | |
| Coal | | |
| Coke | | |
| Peat, loosely packed | | |
| Charcoal, " " | | |
| Mineral bulk goods | | |
| Cement | | |
| Powdered lime | | |
| Lime fertilizer | | |
| Limestone, broken | | |
| Iron ore, lumps | | |
| " " ore concentrate | | |
| Rock salt | | |
| Cooking salt | | |
| Foodstuffs and agricultural products | | |
| Grain | | |
| Hay and straw, loose | | |
| chopped | | |
| compressed | | |
| Silage | | |
| Beet pulp | | |
| Root vegetables | | |
| Liquids | | |
| Fuel oil | | |
| Paraffin, diesel oil | | |
| Petrol | | |
| Alcohol | | |
| Glycerine | | |

Table 4.3.2.b. Coefficient $\nu$ for determination of the angle of friction between bulk goods and the surface of the structure

| Surface of structure | Value of $\nu$ upper limit | lower limit |
|---|---|---|
| Concrete | | |
| Timber | | |
| Metal | | |

PAPER 20

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# COMMENTS ON THE LOADING REGULATIONS

NORDIC COMMITTEE FOR BUILDING REGULATIONS

PARIS - FEBRUARY 1975

NORDIC COMMITTEE ON
BUILDING REGULATIONS

Loading Sub-committee

DRAFT 74-10-30

# COMMENTS ON THE
# LOADING REGULATIONS

FOREWORD

These comments provide the background to the regulations and instructions contained in the loading regulations. They also contain clarifications and explanations, as well as views, directed at the authors of those other regulations which the loading regulations are associated with. The background, as set out, can also serve as an example which can be made use of by individual designers in conjunction with the assessments of various actions which must be made in each case.

The comments are arranged under the same headings and have been given the same numbers as the sections in the regulations and instructions. Comments are not, however, given on all sections. The definitions are commented on in connection with those sections to which the explanations mainly refer.

## K2 CLASSIFICATION OF ACTION

### K2.1 Variation of an action in time

### K2.1.2 Duration of action

The importance of the concept of duration becomes particularly evident owing to the introduction of environmental action. With regard to the effect of environmental action, duration is often absolutely critical. The definition of the concept of duration is that generally accepted.

FIG. K2.1.2 a shows an example of a duration curve relating to load in the form of action of force, q. With regard to one and the same action, different cases can often be of interest. In the case of temperature, for instance, the duration of high temperature or low temperature can be of interest. FIG. K2.1.2.b shows an example of a duration curve relating to low temperature T with the intensity $q = T_0 - T$, where $T_0$ is an appropriately selected initial value.

The concept of continuous duration is introduced here in view of the fact that this quantity is often of essential significance, for instance for the calculation of deformations.

In the same way as in the case of duration, continuous duration may relate to different cases, e.g. high and low temperature.

The concept of relative duration is intended to express the proportion of time over which the intensity of a certain action exceeds a certain value. In order that the value of the relative duration should have the same meaning at different times, it is necessary for the variations in intensity of the action to be similar over the whole period of use. Where this is not the case, the period of use can be split up into parts so that this stipulation is valid in each of these parts, and the relative duration is then given different values for different parts.

Whether a load produces a long-term effect and, if so, how great this effect is, is naturally dependent on the material in the structure in question. In view of this it has not been possible to lay down any rules regarding the long-term effect of different loads, and a number of quantities which can be used in the appropriate material codes, have instead been defined. Chapter 4 in the regulations gives values of these quantities in certain cases for the loads which are defined in that Chapter. In the case of other loads the values of these quantities can be assessed on the basis of the definitions. These quantities have been chosen with particular reference to the assessment of creep deformations, cracking in concrete structures, and reduction of strength as a consequence of long-term loading. Views on the use of these quantities are given in the following.

A description is given below of a method of using the defined quantities for assessing the magnitudes of creep deformations in the case where the load varies more or less periodically in time. The method of assessment is to be regarded as a relatively approximate one, but should nevertheless be regarded acceptable in view of the fact that general knowledge concerning creep deformations is incomplete.

One stipulation is that the assumption regarding proportionality between stress and strain, and the principle of superposition, are valid both in the case of instantaneous deformations and creep deformations. Many research workers have verified that these assumptions can be considered to hold for usual construction materials, provided that the stresses are kept within certain limits.

The method implies that in the case of a load whose intensity q varies, the long-term effect with regard to deformation is assessed as the aggregate effect of the following two cases:

1/ The load $q_s$ acts as constant load over the whole period of use of the structure.

2/ The load $q_{max} - q_s$ acts during the time $t_{qs}$, which can be given an arbitrary value during the period of use.

The symbol $q_s$ in this case denotes the average load (the mean intensity of the load over a long time), i.e. the area underneath the curve relating to relative duration (FIG. K2.1.2c). In most cases, values of load corresponding to usual load can be used for $q_{max}$, i.e. the effect of high values of load which occur very seldom, is disregarded.

The reasons for the use of this method are set out in Appendix 1.

The damage caused by a crack in a concrete structure can be assessed in accordance with the following two cases:

1/ The damage occurs as soon as the crack has attained a certain width. This is the case, for instance, in conjunction with brittle surface finishes. The design rule should in this case be formulated in such a way that there is a reasonable probability that the crack width under load will not exceed a certain value.

2/ Damage occurs as a result of the fact that the crack width has been of sufficient magnitude over a sufficiently long time. This is the case, for instance, in conjunction with corrosion of reinforcement. The design rule should in this case, in principle, be formulated in such a way that the crack width under load of a certain duration must not exceed a certain value. In this case the duration expressed in terms of time, for instance years, is of critical importance and can be determined as the relative duration multiplied by the period of use of the structure.

K2.1.3 Static load - dynamic load

The definition given for dynamic load implies that many kinds of load must in principle be regarded as dynamic load, e.g. wind load, load due to machines with reciprocating parts, loads due to persons in motion, loads due to vehicles, impacts, etc. In many of these cases, however, there is no need for calculations to be performed regarding the dynamic conditions of the structures, but calculations can be carried out in the same way as for a static load. One example of this is wind load which need only be treated as dynamic load in calculations if the structure is a type in which oscillations can occur as a result of variations in wind velocity. This is the case if the structure has comparatively low rigidity and its natural frequency is low. In the case of a rigid structure with a high natural frequency the wind load can normally be regarded as a static load. Another example is the effect of impact. A rigid structure subjected to impact, with the duration of the impact force acting on the structure large in relation to the natural frequency of the structure, will produce a mainly static response to the force due to the impact. The structure can be analysed for an impact force which is largely independent of the shape of the structure, as long as the structure can be regarded rigid. In the case of a less rigid structure which is subjected to a short-term impact, with the natural frequency of the structure considerably lower than the duration of the impact, the effect of the impact must be regarded as an impulse which causes oscillations in the structure. The effect of the impact force will greatly depend on the design of the structure. The stresses in the structure can be fairly independent of the magnitude of the impact force, while they will instead depend on the magnitude of the impulse. This case cannot be treated as the effect of a static load.

In conjunction with pulsating or alternating loads there is a risk of resonance if the frequency of the loads is near the natural frequency of the structure. The dynamic effect can as a rule be ignored if the frequency of the load is less than 25% of the lowest natural frequency of the structure. If the frequency of the load is considerably greater than the natural frequency of the structure, then the effect of the variations in load can often vanish owing to the fact that the structure will then function as a vibration damper.

K2.1.4 Non-recurrent action - recurrent action

It is assumed that the sensitiveness of a structure to recurrent load and fatigue load is dealt with in the appropriate technical codes. The reason why a distinction is made between non-recurrent and recurrent loads is that a considerably less constrained use can be made of limit state methods in conjunction with non-recurrent loads. In conjunction with recurrent loads it can in many cases be necessary for restrictions to be introduced in connection with the use of limit state methods, even if the number of load alternations is not so great that there is a likelihood of fatigue.

Examples of non-recurrent loads are the dead load of the structure, shrinkage of concrete, unusual snow load, certain loads due to accidents. On the other hand, unusual wind load, for instance, is classified as recurrent load in view of the fact that it is possible for several wind velocity peaks of design magnitude to occur within a relatively short period during a "standard" storm.

K2.2 Variation of an action in space

"Bound" load and "free" load are substantially the same concepts as those denoted live load and dead load in Swedish Construction Code 67. Approximately the same designations have been used in the other Nordic countries. The reason for the change in designation is that those used at present can to a certain extent be misleading. The designation "dead" comprises something of a time element and has in this way become misleading in certain cases. Dead load can at times be interpreted as equivalent to long-term load. The designation "live" gives the impression that the load can be moved over the structure, which need not be the case.

According to the definition, a "bound load" is entirely determined by one parameter with regard to the distribution of load over the structure. The same/does not apply in the case of "free load", where the conditions can vary from an entirely arbitrary distribution to one that is almost bound. It would naturally be desirable to give numerical values of the degree of freedom of the load. In the case of a continuous beam, for instance, one may wish to limit the values of the ratio between loads in two adjacent spans to a certain value. It has not been possible, however, to lay down numerical rules which have reasonably general application.

K2.3 Probability of a certain intensity of action

K2.3.1 In conjunction with the application of the method of partial coefficients

Even in present codes there is a division with regard to the probability of occurrence of the load, inasmuch as loads are now divided into usual and exceptional loads. A somewhat more differentiated division has been proposed in these regulations, the groups constant and usual load being largely equivalent to what is now termed usual load, and the group unusual load being approximately equivalent to what is now termed exceptional load. Generally speaking, the group of extreme loads has no equivalent in present codes. It is most closely equivalent to certain special (sometimes denoted "extra exceptional) load assumptions which are now stipulated for certain types of object, or from case to case for individual projects. One example of such regulations in force at present is the case when a heavy vehicle is supposed to drive onto the footpath of a bridge.

In contrast to codes now in force, the different groups have been defined here by means of the probabilities, applicable to the boundaries of the groups, that the value of the load will be exceeded some time during one year. From the point of view of application it would have been simpler (particularly as regards load combinations) to choose instead, as the critical condition with regard to the boundaries of the groups, the probability that the value of the load will be exceeded at some time, selected arbitrarily. In view of the possibility of determining values of the load on the basis of observations (in the case of natural loads) or assessments concerning future conditions (in the case of useful loads), however, it has been considered more appropriate to select the probability relating to the largest value per year as the basis of classification. It would be considerably more difficult to determine or assess the probability at an arbitrarily chosen time.

Specification of certain values of probability relating to the boundaries of the groups can be interpreted as the fixation of certain points on the statistical distribution curve of the largest annual value of the load. If the values of the distribution curve are denoted F, then the specified values will be (1 - F).

Many actions can be assumed to have intensities which are equivalent to both usual and unusual and extreme action. In present codes only one intensity is normally specified for each load, and this is made different for different loads; for instance, snow load is now usual load, and wind load exceptional load. In many cases it may be appropriate to specify several levels for the same action. This has been done in these regulations by the specification of both usual and unusual load for loads due to persons etc on floor slabs, snow load, wind load, etc. The usual load (or a certain portion of this, as specified in K2.1.2) can then be made the design value with regard to e.g. deflection and cracking, while the unusual load can be the design value with regard to the risk of failure. Better differentiation of design for different stages is obtained in this way.

The group comprising extreme loads is not, however, normally intended to contain snow loads, wind loads etc, but to be reserved for rare loads such as explosion, collision with a vehicle, etc. The loads in this group are considered, at least in part, to constitute the loading conditions which can be specified as an alternative to the principle of providing alternative paths of support for the load in conjunction with design with regard to progressive collapse.

The division into short-term usual action and non-short-term usual action is justified by the use of these in Chapter 3, Load combinations. The choice of $p = 0.2$ for usual action and $p = 0.02$ for unusual action is to some extent arbitrary. $p = 0.02$ is in conformity with certain international practice (there is often talk of 50-year winds), and $p = 0.2$ has been considered to imply sufficient difference between usual and unusual action.

K2.3.2 <u>In conjunction with the application of the statistical method</u>

Some uniformity has been endeavoured in the classification in sections 2.3.1 and 2.3.2. In section 2.3.1 a distinction is made between two levels, "usual action" and "unusual action", of the time-variable action. In section 2,3.2 these are summarised as "temporary action". Division into "short-term temporary action" must be regarded as a very gross simplification in relation to the consideration of duration.

The desirability of similar concepts in the application of the partial coefficient method and the statistical method is the reason why the description of the magnitude of the action, even in conjunction with the statistical method, is based on the distribution curve relating to annual maxima.

## K3 COMBINATION OF DIFFERENT KINDS OF ACTION

The definitions in Chapter 2 for both the statistical and partial coefficient method imply that the values of action are determined from the distribution curve relating to annual maximum. If two such distribution functions relating to the actions $F_1$ and $F_2$ are directly combined according to the rules of the theory of probability into a distribution function for $F_1 + F_2$, this will be correct only on condition that the largest values of $F_1$ and $F_2$ during the same year occur simultaneously. If the actions are independent of each other, then this implies that at least one of the loads does not vary during the year. If both loads vary in time, then a shorter period of time must be chosen in determining the distribution function for at least one of the actions. This will give rise to complicated conditions which are due to the mutual magnitudes, relative durations etc of the two actions.

### K3.2 In conjunction with the application of the statistical method

By way of gross simplification, it has been assumed that for each load there is a distribution relating to the short-term maximum, i.e. a distribution for the largest value over a period shorter than one year, and that this distribution can be used in conjunction with combinations. The distribution, and also the concept of short-term, is to be regarded as fictive and assumed to have the mean value $m_o$ and coefficient of variation $V_o$ which can be related to the corresponding parameters m and V for the distribution of the annual maximum according to the equations given in 3.2. The specified values of the coefficient $\alpha$ are a result of an assessment of what can be reasonable. By calculations or simulations of combinations (it would not appear at present that these occur to any appreciable extent) it should be possible in future to produce better substantiated rules.

### K3.3 In conjunction with the application of the method of partial coefficients

It has been found appropriate to introduce two combinations I and II for normal cases, combination I being critical at the ultimate limit state, mainly when constant load (dead load of the structure) predominates. Combination II (in the same way as combination SIII in the statistical method) is valid only in cases where extreme action (e.g. some form of load due to an accident) is of significance. Combination I is also to be valid in the limit state for progressive collapse, and to be capable of use in the serviceability limit state. In the latter case, it should be an alternative to, or/be combined with, an uncorrected usual load.

The correction factor $\zeta$ has been calculated from the expressions for the parameters relating to the distribution for short-term maxima, given in 3.2. The distribution for short-term maxima gives, as a characteristic value equivalent to 0.8 fractile,

$$q_o = m_o(1 + 0.85\ V_o)$$

The distribution for annual maxima gives similar values

$$q = m\ (1 + 0.85\ V\ )$$

The correction factor is written

$$\zeta = \frac{q_o}{q}$$

which, with $m_o$ and $V_o$ according to 3.2, will be

$$\zeta = 1 - (\alpha + 0.85\ /\xi - 1/)\ \frac{V}{1 + 0.85\ V}$$

The specified values of $\zeta$ have been calculated using $\alpha$ according to 3.3, and the values of V given in CHAPTER 4 for different actions.

## K4 LOADS

### K4.1 General

The object of these regulations and instructions is that structures which are designed for commonly occurring loads and are thus designed for relatively small horizontal loads should however have a reasonable capacity to resist these. For instance, in a frame which has been designed only for vertical loads according to FIG. K4.1 a, the specified forces may e.g. be assumed to be equivalent to the effect which arises due to the fact that the columns have been given an unintentional inclination in accordance with FIG. K4.1 b.

The value of $\psi$ in the instruction is schematically chosen to be 0.015 for all vertical loads, irrespective of how the structure is designed. For a case which is in principle in accordance with FIG. K4.1 b, this implies that the deviation of the columns from the vertical line is assumed to be entirely systematic. In actual fact, one part is probably systematic and one part random. Treatment in accordance with the principles of statistics, with a division into systematic and random deviations, has the consequence that the magnitudes of the forces will be dependent, inter alia, on the number of columns. More accurate statistical treatment based on observed data and associated with specified tolerances and the method of checking these, may have the result that the forces can be assumed to be smaller.

The specified forces can also to some extent be seen as a safeguard against forces which have been assumed to be vertical but in fact have a small horizontal component, e,g. as a result of dynamic effects. The effect of seismic phenomena of minor intensity can be given as an example of this.

K4.2 Weight of building components and earth

K4.2.1 Weight of building components

The minimum values of the load due to non-loadbearing partitions, given in the instructions, are based with regard to the line load on a supposed case involving 2.5 m high walls with a weight of 40 kg per $m^2$. In the case of the uniformly distributed load these walls have been assumed to be spaced 2.5 m apart, and 25% has been added to the value thus obtained in order to allow for the effect of load concentration.

The expressions for the variation $\Delta G$ in weight are simplified ones. In accordance with statistical principles, the expression should have the form

$$\Delta G = k\ G\ V_g = k\ G\ \sqrt{V_v^2 + V_\gamma} = k\ G\ V_v \sqrt{1 + (\frac{V_\gamma}{V_v})^2}$$

This case is regarded as usual load and, if normal distribution is assumed, is equivalent to the 80% fractile which is assumed to be equivalent to usual load according to 2.3.1, with k = 0.85. If, for instance, $V_v$ is assumed to be greater than $V_\gamma$, the maximum value of the expression under the square root sign will be equal to 2. This gives

$$G = 0.85\ G\ V_v \sqrt{2} = 1.2$$

which has been approximated to 1. The case where the variation $\Delta G$ is positive on some parts and negative on other parts, and these parts are simultaneously placed in such a way that the most critical effect is obtained, has not been considered to come within the definition of usual load. $\Delta G$ has been given only as a reduction, since it is only the variation which is of interest in this context, and $\Delta G$ should not have an effect on the cases where only the heaviest weight is critical.

The values given in Table 4.2.1 are comparatively well documented by means of collected information, as regards the mean values. On the other hand, the values of the coefficients of variation have in many cases been only estimated, but this should be of subordinate importance in comparison with the uncertainties in the other loads. In certain cases, for instance for metallic alloys, the coefficients of variation for a definite alloy are appreciably smaller than those quoted. For the sake of simplicity, however, a fixed mean value has been given for e.g. aluminium alloys, and the difference between different alloys is considered to be covered by the coefficient of variation. It must be emphasised here that the values in the table do not include the effects of dimensional deviations, e.g. in the case of metal sheeting. Values of the weights per unit area of cladding materials have not been given, since there can be a lot of variation in these values from type to type. In most cases sufficiently accurate values can probably be obtained from the manufacturers' catalogues.

In certain cases it may be warranted to use values of weight lower than the mean values given in Table 4.2.1. This is particularly the case where the weight of a structural component counterbalances a buoyant force and where the margin between the weight and buoyant force is small. This
??? can be the oase, for instance, in anchorage foundations for suspended ????
structures and in conjunction with certain problems relating to overturning.

In cases where the weight is very uncertain the mean value should not be used.

K4.3 Useful loads

K4.3.1 Loads due to furnishings and persons

K4.3.1.1 Distributed vertical load

The regulations and instructions given in this section are based on the results of investigations, assessments and practical considerations with regard to computation work.

Division of the loads into two portions a and b is highly fictive. The ratio of one load portion to the other can vary within wide limits, although normal conditions are probably such that in the case of high values of loads due to furnishings the loads due to persons will be small, and vice versa. The load portion a has been given what is considered reasonable values which have been chosen in such a way that a reasonable long-term load is obtained, and also a load which can be assumed to occur on several floors simultaneously, so that in the case of load combinations in accordance with Chapter 3 many loads can be classified in load combination I. The load portion a has not, however, been made high in relation to the critical value of the load portion b, in view of the fact that portion a is "bound" and does not, therefore, at all times contribute to the occurrence of "critical load position".

The value of the load portion b has been based on the following considerations:

The occupation load constitutes an upper limit of the load which it is supposed can occur due to gathering of people in e.g. a dwelling, without there being any reason to consider that there is an assembly of people. In the case of an area of 30 $m^2$ the limit is taken to consist of 15 people, which on the assumption of a weight of 0.75 kN per person gives approximately 0.4 $kN/m^2$ on average. Unfavourable placing is taken into account by means of a load concentration factor of 1.2 (see below) which gives 0.2x1.2 $\approx$ 0.5 $kN/m^2$. In the case of an area of 5 $m^2$, the limit is taken to consist of 4 people, which in the same way gives $\approx$ 0.7 $kN/m^2$.

The assembly load constitutes an upper limit of the load which it is supposed can occur due to gathering of people, without there being any direct overcrowding. With regard to the load on a larger area, a guide may be given by lecture halls or cinemas. One seat occupies about 0.6-0.8 $m^2$, which gives a load of approx. 1.5 $kN/m^2$. No load concentration factor has been included in this case. In other cases, for instance in a dance hall, such a collection of people would probably be considered to constitute overcrowding with the exception of small areas (approx. 5 $m^2$), since the load may exceed 1.5 $kN/m^2$. The increase has been assessed at 35% (see below).

Crowd load constitutes the load due to people in reasonably crowded conditions. It is stated in /2/ that, on being evacuated from premises, one person occupies a space of 0.26 $m^2$, which gives 2.9 $kN/m^2$. In /3/ it is stated that in the case of a crowd load of 2 $kN/m^2$, the people cannot move freely. At a load of 4 $kN/m^2$ the people stand very close, and at 6 $kN/m^2$ there is direct discomfort. In the case of large areas ($\geqslant$ 30 $m^2$) it is not considered that a crowd load of more than 3 $kN/m^2$ (120 people on 30 $m^2$) is likely. It is considered that a load of 4 $kN/m^2$ can occur on small areas (approx. 5 $m^2$)(27 people on 5 $m^2$).

In choosing the values of load and in summarising the loads in Table K4.3.1.1 a, the results of investigations according to /1/ and /2/ and also some investigations recently carried out in Finland, which have not yet been published /4/, have been utilised to the greatest possible extent. A comparison is made in the following between these results and the values of load used here.

The results of comprehensive investigations concerning loads in office buildings are reported in /2/. No significant differences are shown between different kinds of offices or between different floors in a building, with the exception of the basement and ground floor which produce higher loads. In the case of the other floors, different fractile values have been given for the mean load on areas of different size. In addition, an "additional mobile load", which is however described only very briefly in the report, has been introduced. This load, as a purely deterministic one, has been added to the 99.9% fractile. If the same values are also added to the others - which is doubtful, but on the other hand the addition is comparatively small - the values given in Table K4.3.1.1a are obtained.

In using these values, the load has been assumed to have a log-normal distribution, which implies that a fractile value $q_f$ can be approximately written

$$q_f = q_m\, d^{k_f \cdot V} \qquad \text{(K4.3.1.1a)}$$

where $q_m$ is the mean value
$V$ the coefficient of variation
$k_f$ a coefficient which is a function of the fractile f according to Table K4.3.1.1b.

Equation (K4.3.1.1a) can be written

$$\ln q_f - \ln q_m = k_f \cdot V$$

which gives a linear relationship between $\ln q_f$ and $k_f$. FIG. K4.3.1.1a shows,as an example, this relationship with values according to Table K4.3.1.1a for loaded areas of 5.2 $m^2$ and 58 $m^2$.

With a satisfactory degree of approximation, the relationships according to FIG. K4.3.1.1a are linear. The values of $q_m$ and V obtained on the basis of these straight lines are given in Table K4.3.1.1c.

The values of $q_m$ and V given in the table do not agree with the values obtained in the investigation, which is natural since the distribution obtained is not log-normal. The values of $q_m$ and V are to be regarded as fictive values applicable to a log-normal distribution which, at its "upper tail", approximately corresponds with the results of the investigation; this is the essential point in this context.

V as given in Table K4.3.1.1c does not vary very much. With a view to simplifying calculations, it may be appropriate to use a value of e.g. V = 40%. $q_m$ can then be corrected so that the same value of $q_{99.9}$ is obtained. The value of $q_{rm}$, thus corrected, is given in Table K4.3.1.1c together with the value on the 95% fractile, $qr_{95}$, which is obtained with

$q_{rm}$ and V = 40%. $qr_{95}$ can be compared with $q_{95}$ according to Table K4.3.1.1a. $q_{rm}$ is shown as a function of loaded area in FIG. K4.3.1.1b.

The load according to FIG. K4.3.1.1c can be simplified so that

$q_{rm} = 0.80$ kN/m$^2$ for $A \geq 30$ m$^2$

$q_{rm} = 1.25$ kN/m$^2$ for $A = 5$ m$^2$

A load concentration factor, which allows for the risk that the load is distributed over the structure in a way which produces conditions more unfavourable than those due to the mean load, is also given for different types of structure and different factors. This is of particular importance for large loaded areas. For the sake of simplicity, an average value = 1.2 for A = 30 m$^2$ is used here.

Using these values, the following values are obtained for usual and unusual load:

| | | |
|---|---|---|
| For A 30 m$^2$ | $q_v = q_{80} = 1.35$ kN/m$^2$ | (1.5) |
| | $q_{ov} = q_{98} = 2.18$ kN/m$^2$ | (2.5) |
| For A = 5 m$^2$ | $q_v = q_{80} = 1.75$ kN/m$^2$ | (1.7) |
| | $q_{ov} = q_{98} = 2.85$ kN/m$^2$ | (3.0) |

The loads according to the regulations are given in brackets and have thus been provided with a certain margin in order to allow for possible future increases.

/1/ reports investigations concerning loads due to furniture and persons in dwelling houses. The following values are obtained for loads due to furniture, on average for all types of room with the exception of the kitchen:

$q_{90} = 0.35$ kN/m$^2$
$q_{99} = 0.62$ kN/m$^2$
$q_{99.9} = 1.00$ kN/m$^2$

The following values are obtained for loads due to persons

$q_{90} = 0.49$ kN/m$^2$
$q_{95} = 0.56$ kN/m$^2$
$q_{99} = 0.72$ kN/m$^2$

In the same way as that applicable to the results in /2/, a log-normal distribution can be fitted to these values, and fictive mean values $q_m$ and coefficients of variation V computed. We thus obtain for loads due to furniture

$$q_m = 0.17\ \text{kN/m}^2$$
$$V = 58\%$$

for loads due to persons

$$q_m = 0.31\ \text{kN/m}^2$$
$$V = 37\%$$

It is probable that there is a negative correlation between loads due to furniture and those due to persons, so that a large load due to furniture is combined with a small load due to persons. If, however, the loads are assumed independent, the mean value and coefficient of variation can be estimated as

$$q_m = 0.5\ \text{kN/m}^2 \qquad V = 35\%$$

If a log-normal distribution with $q_m = 0.5$ kN/m$^2$ and $V = 40\%$, i.e. the same values as in the foregoing, is assumed, then we obtain

$$q_{80} = 0.70\ \text{kN/m}^2 \qquad q_{98} = 1.14\ \text{kN/m}^2$$

With the same load concentration factor as previously, i.e. 1.2, we obtain

$$q_v = 0.85\ \text{kN/m}^2 \qquad (1.0)$$
$$q_{ov} = 1.37\ \text{kN/m}^2 \qquad (2.0)$$

The values in the code are higher. There is some justification for this since there are few observations relating to high fractiles. In the case of small areas, it is assumed that the increase is the same as for office premises.

The Finnish investigation reported in /4/ has been concerned with schools, hotels, hospitals, offices and residential buildings. For each type of object, the investigation comprised a relatively small number of objects (10–40). However, the results do provide some guidance in choosing values of load. Values of the 80%, 95% and 99% fractiles, calculated from the mean values and standard deviations, are given in the summary and these are reproduced in Table K4.3.1.1d.

Group 4 according to Table 4.3.1.1a is characterised by the fact that the loads substantially consist of personal loads. It is assumed that over-crowding can occur, and it is also assumed, at least in certain cases, that dynamic effects can occur.

Group 5 has been considered equivalent to Group 4, although it is probably less often that loads of appreciable significance occur.

Essentially, Group 7 comprises only personal loads. In view of the loads which occur in conjunction with evacuation and similar, it has not been considered reasonable to ignore crowd load entirely.

Owing to the scarcity of investigation results, it has been assumed that the dependence on the loaded area, stated in /2/, is also valid in premises other than offices.

The values of the mean value of load and the coefficient of variation in Table 4.3.1.1b have been chosen on the basis of the results of the investigations mentioned above, and also in such a way that they are mainly in agreement with the load values in Table 4.3.1.1a, at least with regard to unusual load. It has been considered reasonable that the coefficient of variation should be larger in cases where practically the whole load is personal load. The agreement between Tables 4.3.1.1a and 4.3.1.1b is not complete, owing to the fact that round numerical values have been aimed at.

K4.3.1.2 <u>Concentrated load</u>

The concentrated long-term load of 1.0 kN is intended to cover local loads due to furnishings. The reason why it is stipulated to be an alternative to distributed load is that it is desired to simplify calculations; in many cases it is possible to decide directly that the concentrated load is not critical. The concentrated load as such is comprised in the distributed load and is only an expression for a possible load concentration which may be of local importance.

The concentrated load which is assumed to be due to people in motion is an alternative for the same reasons as those above. The properties of loads of this type have been investigated at Lund Institute of Technology by measuring the forces which arise on horizontal and vertical surfaces in conjunction with different movements. Certain critical movements, which are explained in greater detail below, have been selected from the results.

A jump from a height of 0.8 m has been chosen as the design movement in the case of floor slabs and similar. From the statistical distribution obtained in the investigation for the forces, the value corresponding to the 98% fractile has been chosen. This value has been regarded as an unusual load. In itself, it is likely that a load of this magnitude occurs more frequently than suggested by the definition of unusual load, and that somewhat higher loads occur. In conjunction with higher loads, however, the movement as such is probably a greater cause of serious injury than that likely to be caused by a local failure in the floor slab.

The movement where a person runs down the stairs two at a time has been chosen as the design movement for stairs. As regards calculation of the magnitude of the load, the same applies as in the case of floor slabs.

In other cases, the definitions of usual load, unusual load etc which are given in Chapter 2 are less appropriate. In view of the fact that the primary consideration is the risk of injury, the characteristic value should be defined as follows: "The characteristic value of the intensity of an action is the intensity which has a probability p, determined in advance, of being exceeded at least once a year. The value p of this probability is valid in this context for a person selected at random who can exert an effect on the structure due to a movement which produces substantial forces". The value of p has been made up, firstly, of the probability that the movement will occur at all and, secondly, of the probability that the characteristic value is exceeded <u>if</u> the movement occurs. The latter probability can be assessed on the basis of the investigation mentioned above. As regards the probability that the movement will occur at all, there is no basis for a calculation, all that is known is that several of the movements selected as of critical importance <u>have</u> occurred and resulted in injury. The only thing that can be done is to choose the load in such a way that the risk of injury is sufficiently low <u>if</u> the movement occurs. It should however be possible, in the case of different movements, to make a rough qualitative assessment of the mutual relationship of the probabilities of their occurrence. In the case of roofs, a fall 1/ which entails a person falling backwards has been chosen as the design movement. In the case of roofs which do not support a person and where persons are not allowed without special arrangements, this movement can be designated as of very infrequent occurrence, and the 90% fractile has therefore been chosen as characteristic value of the extreme load. In cases where there are differences in level on the roof, a movement 2/ which relates to a jump from the higher to the lower level has been considered. In conjunction with differences in level of about 20 cm, it is considered that the forces which occur are about the same as for movement 1/ according to the above. In conjunction with differences in level of 80 cm, the forces will be considerably larger and will be design values for those portions of the roof which are located next to the difference in level. As regards the probability of its occurrence, movement 2/ has been considered more usual than 1/ according to the above, and the 98% fractile has been chosen as the characteristic value for extreme load. The difference between the probabilities that a certain movement will occur in the case of roofs which do and do not support a person, has been expressed in such a way that the values of load which apply for extreme load in the case of roofs which do not support a person apply for unusual load in the case of roofs which do support a person.

For walls, doors, balcony fronts etc, the design movement for forces with a low point of action (not greater than 1 m above the floor) has been chosen as a movement 3/, which entails a person intending to sit on the floor slipping and knocking heavily against the vertical surface. For forces with a high point of action (up to 1.8 m above the floor), a movement 4/, which entails a person falling backwards against the vertical surface, has been chosen. The forces in the two cases are approximately the same. It can be considered very infrequent that the movements will occur at all, and for this reason the value corresponding to the 90% fractile in the distribution obtained in the above investigation has been chosen as the characteristic value for extreme load.

The values given in Table 4.3.1.2 can be considered preliminary since analysis and collation of the test results is in progress. The loads are relatively high and difficulties are likely to arise in many cases in proving by means of calculations that a structure will withstand these loads. Tests on e.g. wood studs have however shown that these withstand a load which, according to current calculation rules, should result in failure. This state of affairs is probably mostly due to the fact that the load is of very short duration.

References

/1/ Johnson, A, 1953: Strength, Safety and Economical Dimensions of Structures. National Swedish Building Research, D7:1971.

/2/ Mitchell, G R & Woodgate, R W, 1970: A survey of floor loadings in office buildings. Construction Industry Research and Information Association, London.

/3/ Ferry Borges, J & Castanheta, M, 1971: Structural Safety, Lisbon 1971.

/4/ Paloheimo, E & Ollila, M, 1973: Preliminary summary of investigation concerning useful loads in different premises. Stencilled report, Helsinki 1973.

K4.3.2 Loads due to goods, bulk products, etc

If the largest possible load occurs often, which may for instance be the case for a liquid container, it is very nearly right to regard the load as deterministic and equal to the largest possible value. It follows from this that this value of load is equal to the mean value and equal to the usual load.

It is assumed that the "largest permitted load" will be exceeded, which is assumed here to be equivalent to unusual load. The stated value of 30% excess and the definition of unusual load imply that the load for every fiftieth structure, on average, is assumed every year to exceed "permitted load" by at least 30%. In the same way, the choice of usual load implies that the load for every fifth structure, on average, is assumed every year to exceed the permitted load. The values of mean value and coefficient of variation are in approximate agreement, for a log-normal distribution, with the given characteristic values. There is no basis for the choice of these values.

The values according to Table 4.3.2a have been arranged in comparatively large groups. In each group, a high value has been chosen as characteristic for the group. The reason for this is the desirability of the structure being designed for a load which makes possible changes in use within reasonable limits, without special arrangements being made for each such change. If a structure is to be designed with regard to a definite kind of goods or bulk goods, then more accurately determined weights can be used.

K4.3.3 Loads due to vehicles, transport appliances and machinery

K4.3.3.1 General

The dynamic effects which occur in conjunction with these loads can often be taken into consideration by the application of a dynamic allowance.

It happens at times that structures are designed on the basis of a large load, e.g. that due to a transport appliance, which occurs on a single occasion. In such a case the load can be regarded as a non-recurrent load.

A machine with reciprocating parts can give rise to a variable load distribution, and a certain part of the load can in that case, to a limited extent, be regarded "free".

K4.3.3.2 Load due to vehicles

In the Nordic countries, a draft for joint regulations relating to traffic load, drawn up by Committee 60 in the Nordic Association of Road Engineers, has been adopted for bridges.

The load due to a vehicle according to FIG. 4.3.3.2 constitutes a "load group" according to the above loading regulations, which is mainly intended to represent a heavy vehicle. In addition, these regulations contain a uniformly distributed load which is intended to represent load due to other traffic which occurs together with the heavy vehicle. It has been considered in this case that the uniformly distributed load need not be taken into account. The values of load for the "load group" have been reduced somewhat (from 210 kN to 180 kN) in relation to the proposal by the Nordic Association of Road Engineers. The justification for this reduction is that the dynamic effect can be considered to be less for the cases covered in 4.3.3.2.

Bus terminals, fire stations, aircraft hangars etc can be mentioned as examples of garages with heavy vehicles.

K4.3.3.3 Loads due to overhead cranes, cranes and other lifting equipment

This refers, for instance, to the lifting appliance code in Sweden which is applicable to both building structures and lifting appliances.

K4.4 Natural loads

K4.4.1 Snow load

K4.4.1.1 General

The instruction concerning a change in the distribution of snow load due to snow clearance or snow slide is not intended to cover all conceivable cases, but is mainly intended for structures which are specially sensitive to uneven distribution of load.

K4.4.1.2 Snow load on ground

/1/ has been used as the basis for the values of usual and unusual snow load.

K4.4.1.3 Shape factors

The shape factors have been chosen in accordance with a proposal made within ISO TC 98/SC3/WG1. An extract from this ISO proposal is given in Appendix 2.

References

/1/ Nord, M & Taesler, R, 1973: The density and mass of snow cover in Sweden. National Swedish Building Research, R21:1973.

K4.4.2 Wind load

K4.4.2.1 Classification

Variation of the wind load in time

A spectrum for design based on testing of certain roof anchorages is proposed in /2/.

Distribution of the wind load in space

There is insufficient knowledge concerning distribution of the wind load in space. It has nevertheless been considered warranted to point out that the phenomenon is a reality. Without this it may be possible for erroneous design to occur in the case of e.g. high braced masts which would be assigned excessively low span moments, and long arches which would be assigned excessively low shear forces at the crown.

K4.4.2.2 Wind velocities and pressure due to wind velocity

The reference to the possibility of determining the probable numerical values of wind velocity by continuous measurements is mainly directed at the people who have the task of broadening the base for wind loading regulations by means of observations.

Instantaneous wind velocities

The magnitude of the design instantaneous value of wind velocity is substantially according to /3/. It has not been possible to formulate an expression for the effect of wind velocity due to the funneling of gusts in e.g. valleys, fjords and inlets or over large obstacles.

The coastal zone associated with high wind velocities has been given such a large width that it includes the most serious cases. The only reason for the choice of linear reduction in wind velocity is to prevent misuse of the regulations in an uncomplicated manner.

Mean wind velocities

The factor of $\sqrt{1.75} = 1.3$ has been chosen for practical considerations. The instantaneous value of wind velocity in relation to the mean velocity, over a space of time of 10 minutes, is a function of the nature of the country. An increase in the stated value can occur over terrain containing obstacles in the form of buildings, forests etc. By way of compensation, the mean value in such an environment is normally lower than over even country. In view of the lack of clarity in the treatment of wind in the rest of the regulations, for instance in an urban environment, this has not been included.

Wind velocities in sheltered locations

Two different phenomena are covered by the reference to sheltered location. One of these is the situation where the boundary layer is considerably greater as a consequence of irregularities. The formula given yields the magnitude of the effect in very approximate terms. The second phenomenon is a special local shelter which cannot be given a general description of any kind. The regulation is open to misuse, but in spite of this it has been considered that it cannot be dispensed with.

Pressure due to wind velocity

In view of the lack of clarity in predicting the design wind velocity, it has been considered entirely sufficient to stipulate the value of 1.2 $kg/m^3$ for air density independently of the temperature.

K4.4.2.3 Dynamic effects due to wind load

The whole section concerning the dynamic effects due to wind load has intentionally been kept short, since a substantial proportion of the background material is inadequate, and because it is considered desirable that situations of major significance should be dealt with by being designed on the basis of tests.

More detailed reports are to be found in e.g. /4/ and /5/. These comments assume no responsibility for the correctness of all the information in these references.

Vortex separation

Different values of the Strouhal Number for cylindrical objects are given in different experimental reports, particularly in the case of large values of Re. The numerical values quoted have been taken from /1/.

The expression for the equivalent load W is vague, one of the reasons being that it is difficult to estimate the amount of damping, and, strictly speaking, this must be determined on the special structure which is to be designed with the aid of the equivalent load. The expression itself, and the numerical values of damping, have therefore been chosen with caution.

With regard to damping in general, data is given in /6/.

Ground damping is dealt with in /4/.

Gusts

The material has been taken from /7/ which had been drawn up in collaboration with A Davenport, University of Toronto. The group is engaged on the preparation of simplified rules.

It has not been possible to formulate distinct criteria, apart from the available experience, to check that the effects of a gust have no significance. None the less, it is so in most cases.

K4.4.2.4 Shape factors

There are sound grounds for the warning in connection with published test results. Nevertheless, it has not been considered appropriate in this context to quote examples. It has been found most appropriate to recommend caution.

The numerical values of shape factors exhibit considerable variation, both in time and space. This is particularly the case at the edges of surfaces and near deviating features. This is the reason for the reservation concerning the limited applicability of the quoted numerical values.

Internal wind load in houses

The circumstances which, in the aggregate, determine the internal wind load in a house are extremely variable. The codes of many other countries, on sound grounds, quote a positive pressure of $\mu = 0.2$ as an alternative to internal suction. In view of the shortage of material on which to base coordination between an alternative internal pressure of $\mu = 0.2$ and the external wind load on the roof, the wording of the regulations could not be given an entirely appropriate formulation in this edition.

With regard to the use of houses with large openings, instructions whose consequences can be checked would be capable of providing the basis for some other formulation.

External wind load on houses

In conformity with /7/, which constitutes the basis for substantial portions of this section, the effects of several wind directions, each of which has special effects, have been summarised in relation to wind blowing onto a corner.

External wind load on external walls.

The instruction is in conformity with /7/.

External wind load on roofs

The instruction specifies the same wind load for the eaves as for the wall below the eaves; this has been considered a reasonable simplification. In other respects, there is substantial conformity with /7/.

In the same way as in /9/, the width of the boundary zone has been related to both the width and the height of the house.

On the basis of /10/, the extreme intensities have been given the function of providing the design criterion only for the actual roof covering and its fastenings. It has not been possible, owing to lack of material, to carry out a desirable and reasonable revision of external wind load so as to be equivalent to an alternative internal pressure of $\mu = 0.2$.

A comparison has been made with /11/. The extreme value of approx. 4.9 quoted in its Fig. 23 for the shape factor has been considered to be associated with a far too restricted direction of incidence, there being a very slight probability of simultaneous occurrence of design wind velocity. The fact that the numerical value of the shape factor, within a sector of 43°, exceeds the maximum value in these regulations has been ignored, there being the additional justification that inventories of damage have not shown that underestimation of the load has been the cause of damage in this respect.

Wind load on stacks, screens, lattice masts, etc

There is a very large volume of literature in this area. /4/ and /12/, which contain further references, may be mentioned here.

Wind load on stacks

A Fourier series relating to the distribution of wind load intensity around a stack of circular cross section is given in /13/.

Wind load on screens

The instructions are based on /14/ which has been considered to be of greater use than /15/. Lack of agreement between these two can be due to the fact that relatively small deviations in model design have a great significance. See e.g. /11/ and the summarised test results for stacks in /4/.

The instruction concerning screens in the lee of another screen is in conformity with /7/ and /9/, but has been re-edited.

Wind load on lattice masts

The instruction concerning masts with angular members is in conformity with /14/. That concerning masts with round members is in accordance with /16/ - /18/. Further information is given in /4/ (without reference to source) and in /19/.

Wind load on canopies

The instructions are a simplified representation of the material in /20/.

Wind load on bridges

Regulations and instructions are based on investigations by E Hjort Hansen /8/ and formulated in accordance with /7/, which employs e to take into account also the moment which is caused by vertical wind loads which are otherwise mostly insignificant. Some literature references are given in /4/.

K4.4.2.5 Shape factors for tangential wind load

The values of shape factor quoted are in accordance with /21/.

References

/1/ Jensen, M, 1961: Shelter effects. The Danish Technical Press.

/2/ National Swedish Board of Urban Planning, 1973: Strength design of fastenings for external insulation on sheeted roofs. Stencil. (Swedish)

/3/ Jensen, M & Franck, N, 1970: The Climate of Strong Winds in Denmark. Danish Technical Press.

/4/ Sachs, P, 1972: Wind Forces in Engineering. Pergamon Press.

/5/ National Swedish Board of Urban Planning, 1970: The dynamic effects due to wind load. Publication No 31. (Swedish)

/6/ Lazan, B J, 1968: Damping of Material and Members in Structural Mechanics. Pergamon Press.

/7/ Danish Society of Engineers, 1966: Guide for determination of wind loads. Teknisk Forlag. (Danish)

/8/ Report by the Institute of Statics, NTH: Wind pressures on bridges
??? of rectangular cross section and projecting footpaths ????????, model tests in a wind tunnel. Trondheim, 1964. (Norwegian)

/9/ National Swedish Board of Urban Planning, 1967: Swedish Building Code (SBN). Publication No 1. (Swedish)

/10/ Schweizerischer Ingenieur- und Architekten-Verein, 1956: Normen für die Belastungsannahmen, die Inbetriebnahme und die Uberwachung der Bauten.

/11/ Wirén, B, 1970: Wind tunnel investigation of the pressure distribution on a flat roof with variable edge design. National Swedish Building Research, R35:1970.

/12/ Jensen, M, 1959: Aerodynamics. Teknisk Forlag. (Danish)

? /13/ Hjort-Hansen, E, 1970: Regulations for wind load. The manual 'Bygg', 1970:5. (Swedish) ?

/14/ Flachsbart, O & Winter, H, 1935: Modellversuche über die Beladtung von Gitterfachwerken durch Windkräfte. Der Stahlbau 1935:10.

/15/ Hoerner, S F, 1965: Fluid-Dynamic Drag. Own publication, 148 Bustead Drive, Midland Park, New Jersey, USA.

/16/ Aerodynamic Research Station, 1968: Report AU-679:1. (Swedish)

/17/ Schulz, G, 1970: Der Windwiderstand von Fachwerken aus zylindrischen Stäben, CIDECT, Monographie Nr 3, Düsseldorf.

/18/ Vindlaboratoriet, D T H, 1954: Triangular mast cross sections with round members. Unpublished. (Danish)

/19/ Hjort-Hansen, E & Kyrkjeeide, A, 1972: Wind load on a lattice mast. Institute of Statics, Trondheim-NTH. (Norwegian).

/20/ Jensen, M, 1965: Model Scale Tests in Turbulent Wind. The Danish Technical Press.

/21/ Jensen, M, 1972: Shape factors. Correspondence.

## K4.4.4 Ice pressure

Reference /1/ and a proposal relating to ice pressure on bridge piers, drawn up by Committee 60, the Loading Group, of the Nordic Association of Road Engineers,are used as the basis of the regulations and instructions given. The values given in the instructions are mainly based on /1/. The proposals by the Nordic Association of Road Engineers gives considerably higher values.

The instructions are in relatively vague terms and should be regarded as a guide in choosing reasonable values of ice pressure.

### References

/1/ Holmberg, Å, 1948: Ice pressure on rises in temperature. Betong nr 1, 1948. (Swedish).

## K4.5 Loads due to accidents

### K4.5.2 Collision with a vehicle

There is no basis for an assessment of conditions in conjunction with a collision with a vehicle. The instructions are based on an imaginary model which is intended to produce reasonable values of the force on being hit and to describe, in a reasonable manner, the extent of the area which can be subjected to primary damage.

Generally speaking, a vehicle is supposed to have a certain kinetic energy $W_o$ when it is on a carriageway or some other place where it is intended to drive. If the vehicle deviates from the normally traversed area and drives into, or inside, a building it is assumed that retardation will occur. This retardation is caused both by the driver applying the brakes and also by small obstacles which the vehicle meets on its way. Naturally, the retardation due to the driver applying the brakes can in certain cases be wholly or partly omitted. It is assumed that the retardation causes a reduction in the kinetic energy of the vehicle by a constant amount e per metre. Unless, therefore, the vehicle meets some major obstacle, it will stop after it has traversed the distance $S_o$, where

$$S_o = \frac{W_o}{e} \qquad \text{(K4.5.2a)}$$

When the vehicle has driven over the distance s after deviating from the normally traversed area, its kinetic energy, on the basis of the above assumption, will be

$$W = W_o - s\,e = W_o\left(1 - \frac{s}{s_o}\right) \qquad \text{(K4.5.2b)}$$

The variation of kinetic energy with s is shown in FIG. K4.5.2a by curve 1.

If, on its way, the vehicle meets a major obstacle in the form of building structures, of which columns, walls etc are mentioned in the following, then its kinetic energy is further reduced. It is supposed in this context that the vehicle damages (n-1) columns and stops as it hits column n, without damaging it so much that its capacity to carry vertical loads is considerably reduced. In conjunction with their deformations on being demolished, the (n-1) columns have absorbed a quantity of energy which, in the aggregate, is denoted $\sum_{1}^{n-1} W_{k_\nu}$. Column 'n' does not, normally, absorb any energy of significance, since it must be stipulated that its deformation is small if it is to retain its loadbearing capacity for vertical loads. It is assumed that, on hitting the obstacles, the vehicle absorbs a considerable quantity of deformation energy which, in the aggregate, is denoted $\sum_1^n W_{f_\nu}$. We thus have, after the vehicle has stopped at the end of the distance s,

$$W = W_o(1 - \frac{s}{s_o}) - \sum_1^{n-1} W_{k_\nu} - \sum_1^{n} W_{f_\nu} = 0 \qquad \text{(K4.5.2c)}$$

The relationship is illustrated for n = 3 by curve 2 in FIG. K4.5.2a. In order to determine $W_f$ and the forces which act on the columns, it is necessary to have knowledge of the relationship between the contact force F between vehicle and column and the deformation (impression) $\delta$ of the vehicle, according to FIG. K4.5.2b. It is likely that this relationship is very irregular and depends on a great number of factors, e.g. the way in which the impact strikes the vehicle, the design of the vehicle, the kind of load and the method of loading, etc. A possible example of this relationship is shown in FIG. K4.5.2c. A very approximate and schematised relationship is used here, which has been chosen so as to give the simplest possible expression. It is described by the expression

$$F = k \cdot \delta \qquad \text{(K4.5.2d)}$$

On hitting the first column which is assumed to be capable of offering the resistance $P_1$, the energy absorbed due to deformation of the vehicle will be

$$W_{f_1} = \int_o^{\delta_1} F \, d\delta \qquad \text{(K4.5.2e)}$$

The energy $W_{f_1}$ is represented by the area $A_1$ in FIG. K4.5.2d. On hitting the second column the vehicle is already in the deformed (compressed) condition, $\delta = \delta_1$, and it is to be supposed that it offers a greater resistance to further deformation right from the beginning. If column 2 can develop the resistance $F_2 > F_1$, then the vehicle is further deformed, and this deformation is assumed to occur according to a curve which is the direct continuation of that applicable on hitting column 1, i.e. according to a curve as shown in FIG. K4.5.2e. The energy absorbed will be

$$W_{f_2} = \int_{\delta_1}^{\delta_2} F \, d\delta$$

If $P_2 < P$, then it is assumed that no deformation energy is absorbed by the vehicle (but energy is absorbed by the column). In the same way, the energy absorbed when the third column is hit (FIG. K4.5.2f), whereupon the vehicle stops, will be

$$W_{f_3} = \int_{\delta_2}^{\delta_t} F \, d\delta$$

where $\delta_t$ is the total deformation of the vehicle. This expression holds if $F_3 > F_1$ and $F_3 > F_2$. If this is not so, then $W_{f_3} = 0$. This is not possible (since $W_{k_3}$ had also been put equal to zero) unless the speed of the vehicle is zero (very little) when it hits column 3. It must therefore be assumed either that $F_3$ is the largest of the values or that $F_3 = 0$, i.e. that the vehicle does not reach column 3.

Overall, $W_f$ is given by the cross hatched area in FIG. K4.5.2g, F being the largest value of the contact force between column and vehicle which has occurred, and $\delta_t$ is the corresponding deformation. $W_f$ is given by

$$\sum_1^n W_{f_\nu} = \int_0^{\delta_t} F \, d\delta = k \int_0^{\delta_t} \delta \, d\delta = \frac{k}{2} \delta^2$$

or,using Equation (K4.5.2d),

$$\sum_1^n W_{f_\nu} = \frac{F^2}{2k} \qquad \text{(K4.5.2f)}$$

Let us put into Equation (K4.5.2c) the expression

$$W_s = W_o(1 - \frac{s}{s_o}) - \sum_1^{n-1} W_{k_\nu} \qquad \text{(K4.5.2g)}$$

i.e. $W_s$ is the original kinetic energy $W_o$ reduced by the losses of energy as a result of retardation and demolition of columns. We then obtain from Equations (K4.5.2c) and (K4.5.2g) that

$$W_s = \frac{F^2}{2k} \qquad \text{(K4.5.2h)}$$

If we now introduce a fictive case where a column 0 is supposed to be placed so that s = 0 for it, and if it is further assumed that it has been designed for the force $F_o$ so that it has not been deformed to an appreciable extent (this implies that in Equation (K4.5.2c) we put s = 0 and n = 1), then $W_s = W_o$, and

$$W_o = \frac{F_o^2}{2k} \qquad \text{(K4.5.2i)}$$

From Equations (K4.5.2h) and (K4.5.2i) we obtain

$$F = F_o \sqrt{\frac{W_s}{W_o}} \qquad \text{(K4.5.2k)}$$

The problem is determined by the equations (K4.5.2g) and (K4.5.2k), and in order that values of F may be obtained, information is required concerning $W_o$, $s_o$ and $F_o$. It is assumed that $W_k$ can be determined on the basis of the deformation characteristics of the structure. $W_o$ is determined by the assumptions concerning the weight and velocity of the vehicle. The value of $s_o$ is difficult to determine, but it is possible to make some kind of assessment. Equation (K4.5.1a) gives the relationship between $W_o$ and $s_o$ if e is assumed constant. The value of $F_o$ depends on the value of $W_o$ and the constant k according to Equation (K4.5.2i) which gives

$$k = \frac{F_o^2}{2W_o}$$

whereupon, according to the assumption (K4.5.2d) applied to the case0, we have

$$\delta_o = \frac{F_o}{k} = \frac{2W_o}{F_o}$$

The following values have been selected for the three cases a, b and c.

a/ A 30-ton vehicle at a velocity of 36 km/h (10 m/s) gives

$$W_o = \frac{30000 \cdot 10^2}{2} = 1.5 \cdot 10^6 \text{ Nm} = 1500 \text{ kNm}$$

$s_o = 25$ m, which gives

$$e = \frac{W_o}{s_o} = \frac{1500}{25} = 60 \text{ kN}$$

The weight of the vehicle Q = 300 kN

$$\frac{e}{Q} = 0.2$$

which value is selected consistently for all cases. $F_o$ is put at 1500 kN, which gives

$$k = \frac{1500^2}{2 \cdot 1500} = 750 \text{ kN/m} \qquad \delta_o = \frac{2 \cdot 1500}{1500} = 2 \text{ m}$$

which does not appear to be entirely unreasonable.

b/ A 20-ton vehicle at a velocity of 12 km/h (3.3 m/s) gives

$$W_o = \frac{20000 \cdot 3.3^2}{2} = 11 \cdot 10^4 \text{ Nm} = 110 \text{ kNm}$$

$$e = 0.2 \cdot 200 = 40 \text{ kN}$$

$$s_o = \frac{W_o}{e} = 2.8$$

$$P_o = 2\ k\ W_o = 2\ .\ 750\ .\ 110 = 400\ \text{kN}$$

c/ A 20-ton vehicle at a velocity of 5 km/h (1.4 m/s) gives

$$W_o = \frac{20000\ .\ 1.4^2}{2} = 2\ .\ 10^4\ \text{Nm} = 20\ \text{kNm}$$

$$s_o = \frac{20}{40} = 0.5\ \text{m}$$

$$F_o = 2\ k\ W_o = 2\ .\ 750\ .\ 20 = 170\ \text{kN}$$

On being introduced into the code, the values have been rounded off.

K4.5.3 Unintentional impact

It is stipulated that structural components must be designed for an unintentional impact force due to causes which are not specified in greater detail. It must be emphasised in this context that, according to the principles of the safety regulations, this applies only to structural components where failure would cause collapse of a whole building or a large part of this. It has been considered justified to provide a safeguard against the occurrence of serious damage due to a comparatively small action which may arise, for instance, during the construction period.

K4.5.4 Explosion

It has not been considered possible to issue instructions for the assessment of the forces which arise in conjunction with explosions caused by explosives, or explosions in pressure vessels. However, the rules given confer on the structure a strength (if it is designed to withstand an explosion) which will enable it to withstand moderate explosions due to explosives, for instance in conjunction with acts of sabotage. The instructions concerning gas explosions are very standardised ones and do not take into account a number of factors, e.g. the shape of the room where the explosion occurs. Some guidance concerning the assessment of the time sequence can be obtained from /1/. The magnitude of the pressure which arises and the effect of window openings have been assessed on the basis of /2/. With regard to the dependence on the volume of the room where the explosion occurs, the assessment is very uncertain. It has been considered that, even in very favourable conditions, the pressure should not be assumed to be lower than 10 kN/m$^2$. In large rooms which cover a whole floor in a high building, at least some columns or walls must be designed to withstand a lower pressure.

In conjunction with supposed explosions in small rooms, columns or walls can be designed for higher pressures or permitted to fail (local damage), the stability of the building being safeguarded by the provision of alternative paths of support for the load. The assumed volume dependence has the consequence that the larger the pressure which a column or wall can absorb, the smaller, in the normal run of events, the area of local damage. The rules given can thus to a certain extent be considered to determine the size of the area of local damage.

References

/1/ Mainstone, R J: "Internal blast". State of the art report No 6. Technical Committee No 8, Fire and Blast. ASCE-IASSE Conference on Tall Buildings, Lehigh University, Bethlehem, USA, 1972.

/2/ Mainstone, R J: "The breakage of glass windows by gas explosions". Building Research Stations 26/71. Watford, England, 1971.

K4.6 Deformation action

Deformation action as such should be classified along with the other kinds of load, e.g. natural loads. The reason for the distinction between action of force and action of deformation, and for classifying the latter in a separate group, is that the effect of deformation action is often reduced by yielding and creep in the material or cracking in the structure, and it is therefore in many cases of subordinate importance at the ultimate stage.

K5 ENVIRONMENTAL ACTIONS

In comparison with loading regulations in use so far, these regulations have been expanded by the inclusion of what is designated environmental action. The reason for this has been the wish to arrange/all (or at least most) of the external factors which have a bearing on the risk of damage in a structure, and not only those factors (designated loads here) which give rise to forces and moments in the structure. In all essentials, it should be possible for environmental action to be arranged in a safety system in the same way as loads. In these regulations, the chapter on environmental action has not been given the extent which its importance warrants, the reason being that available information is limited or is not in a form that is suitable for regulations of this kind. It is to be supposed that it takes some time for regulations on environmental action to be developed, and that it is only in later editions of these loading regulations that more comprehensive information can be given.

In certain cases, environmental action and deformation action can be action due to the same factor, e.g. temperature or moisture. Normally, the difference is that deformation action is determined by changes in temperature or moisture conditions, while environmental action is determined by the actual values of temperature or moisture. In spite of this partly common ground, deformation and environmental action have been separated, and the former grouped with loads. The reason for this is that deformation action gives rise to similar effects in the structure, i.e. forces and moments.

The effect due to environmental action can in certain cases be eliminated or reduced by the introduction of some form of protection, e.g. the provision of fire resistant cladding, or corrosion resistant finishes.

The present draft regulations were to comprise, by way of instructions, a number of figures from which approved assumptions concerning temperature and moisture conditions can be taken. Only a few of these figures, which are to be seen as examples of the way in which information is to be obtained, have been included in these regulations. The material on which these figures are based is mainly taken from /1/.

References

/1/ Taesler, R, Climatic Data for Sweden, 1972. (Swedish).

Appendix 1

Deformations due to the action of long-term loads

A load-time curve according to FIG. 1 is initially assumed, i.e. that a number of loads of the same magnitude and the same continuous duration act at constant time intervals.

Assuming proportionality between load q and stress $\sigma$, and proportionality between stress $\sigma$ and strain $\varepsilon$, the relationship between stress $\sigma$ and load q can be written

$$\varepsilon = \frac{\sigma}{E}(1 + \varphi) = \frac{k \cdot q}{E}(1 + \varphi)$$

?? where $\varphi$ is a time function (creep index ??)
k is a constant.

Two limiting cases are selected with regard to $\varphi$.

1) $\varphi = \varphi(t)$

i.e. $\varphi$ is a function only of the time after application of the load. On application of load at time $t_1$ and consequent removal of load at time $t_2$, a deformation according to FIG. 2 is obtained. The whole of the deformation is recovered.

2) $\varphi = \varphi(t, t^o) = C\, e^{-\gamma t_o}(1 - e^{-\gamma t})$

i.e. $\varphi$ is a function of the time $t$ after application of the load and the age of the material, $t_o$, on application of the load. A deformation as shown in FIG. 3 is obtained, according to the above expression, for load application at time $t_1$ and consequent removal of load at time $t_2$. The whole of the deformation remains after removal of the load.

If this is applied to a case involving many changes in load according to FIG. 1, the total deformation in the unloaded state is given by

$$\varepsilon = \frac{k \cdot \eta \cdot q}{E} \varphi$$

for both cases 1 and 2 above. It is thus possible to calculate the time dependent part of the deformation in the same way as for a constant load, using the reduced magnitude $\eta q$ which is the mean value of the load over a long period.

If the magnitude of the load is variable, it can be divided into a number of loads of constant magnitude. These parts will then have different values of $\eta$, and the expression for $\varepsilon$ will be

$$\varepsilon = \frac{k}{E}\,\vartheta \int \eta\, dq = \frac{k\,\bar{q}}{E}\,\vartheta \, .$$

i.e. the average load $\bar{q}$ is determined as the area below the duration curve.

The expression for $\varepsilon$ is not strictly true if the periods of duration of the loads and the intervals between the loads vary, but it has nevertheless been considered that it can be applied as an approximation.

In certain cases, one load period (the last one) may have a dominant effect. This effect, calculated for the continuous duration, should then be dealt with separately and added to the deformation obtained due to $\bar{q}$, but with the magnitude of the load reduced by $\bar{q}$.

Loaded area A $m^2$

FIG. K4.3.1.1b

Curve 1

Curve 2

Column 1 Column 2 Column 3

FIG. K4.5.2a

TP = CG

FIG. K4.5.2b

TAB K4.3.1.1c

| Loaded area $m^2$ | | | | | |
|---|---|---|---|---|---|
| 5.2 | 14 | 31 | 58 | 111 | 192 |

TAB K4.3.1.1a Values of load equivalent to different fractiles and different sizes of loaded area, according to MITCHELL

| Fractile equivalent to the load | Designations for the load | Loaded area $m^2$ |
|---|---|---|

TAB K4.3.1.1d

| Type of premises according to the regulations | Premises | No of observations |
|---|---|---|
| 1 | Dwellings<br>Hotels<br>Sickrooms | |
| | Value in the regulations | |
| 2 | Offices<br>Schools | |
| | Value in the regulations | |

PAPER 21

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

EXTRACT FROM NORWEGIAN STANDARD

NS3470 "TIMBER STRUCTURES"

PARIS - FEBRUARY 1975

# 3 LOADS AND LOAD EFFECTS

## 3.1 Load assumptions

3.1.1 Structures shall be checked for loads in accordance with Building Regulations of Aug. 1st, 1969, Chap. 51 and NS 3052.

Structures not covered by the building regulations or NS 3052 shall be checked for loads in accordance with any other valid load regulations.

The duration of the load shall be considered.

3.1.2 Assuming an effective load sharing, the following rule can be used for rafters, purlins roof trusses, floor joists and posts with spacing up to 600 mm.

If four or more similar structure elements act together to support a common load, the values for the structural strength and characteristic capacity can be increased by 10 % and the modulus of elasticity for calculating the deformation by 20 %.

## 3.2 Shrinking, swelling and temperature changes

If in special cases the effect of moisture and temperature variations in timber are taken into account, the following values can be assumed:

For timber with moisture content up to 28 % of the dry mass (fibre saturation point), the following dimension changes per % variation in moisture content may be assumed for shrinking and swelling:

Values in %

| | | |
|---|---|---|
| Parallel to the grain | | 0,01 |
| Perpendicular to the grain | radially<br>mean value | 0,04-0,25<br>0,15 |
| Perpendicular to the grain | tangentially<br>mean value | 0,15-0,45<br>0,28 |

The thermal expansion coefficient can be set to:

| | |
|---|---|
| Parallell to the grain | 0,005 mm/m $^{o}$C |
| Perpendicular to the grain | 0,04 mm/m $^{o}$C |

3.3 Forces and moments

3.3.1 Forces and moments can be determined according to recognized methods based on the theory of elasticity.

Moments and shear forces are calculated for most severe load condition relative to the section to be examined.

3.3.2 The span is normally taken as the distance between the support reaction forces. The span for beams can be taken as equal to the light opening plus half of each of the two support widths.

3.3.3 For calculation of shear force on bearing the following rules can be used for freely supported beams with constant rectangular cross sections:

a) Loads within a distance from the support equal to the beam height are ignored. The greatest single load is placed at a distance from the support equal to three times the beam height or at the beam's quarter point if this is nearer to the support.

b) If the shear force calculated by a) exceeds the shear force capacity, the following rules can be used:

For single loads

$$V = \Sigma\ 1,1\ \frac{F(I - x)(x/h)^2}{I[2 + (x/h)^2]}$$

x = distance from F to theoretical bearing point.

For uniformly distributed load

$$V = 0,5\ q\ I(1 - 2h/I)$$

# 4 PRINCIPLES OF DESIGN

## 4.1 Limit states

4.1.1 Design of timber structures is performed by a control at two limit states, ultimate limit state and serviceability limit state (see NS 3052).

At the limit states are compared a design load effect with a capacity derived at by testing or calculation, or with a prescribed limit value, (stress, displacement etc.).

4.1.2 At the ultimate limit state the capacity shall be controlled for bending moment, axial force, shear force and partial loading, see pt. 5.5. In addition buckling of columns and lateral buckling of beams shall be controlled.

The capacity of the fasteners shall be controlled.

4.1.3 In the serviceability limit state the following shall be controlled:

Displacements, if these must be restricted due to the use of the structure

Fatigue (by fatigue-causing loads)

Dynamic effects which can influence the function of the structure.

## 4.2 Design load

The design load for a limit state is the least favourable combination of characteristic load (or standard value for loads) multiplied by load coefficients $\gamma_f$, cf. NS 3052.

## 4.3 Design material strength

4.3.1 Design material strength is a characteristic strength or standard value for structure strength divided by material coefficient $\gamma_m$.

4.3.2 Material coefficient $\gamma_m$ is ecuated with $\gamma_1$
$\gamma_1 \cdot \gamma_2 \cdot \gamma_3 \cdot \gamma_4$

$\gamma_1$ = Coefficient for grading

$\gamma_2$ = Coefficient for production control

$\gamma_3$ = Coefficient for control of calculation

$\gamma_4$ = Coefficient for consequence of rupture

Table 4.3.2 a Coefficient for grading $\gamma_1$

| Grading | $\gamma_1$ |
|---|---|
| Timber graded and marked according to NS 3080 | 1,1 |
| Timber ~~with~~ graded and marked ~~ing~~ subject to special control (see commentaries) | 1,0 |

Table 4.3.2 b Coefficient for production control $\gamma_2$

| Production control | $\gamma_2$ |
|---|---|
| Production control group I see pt. 8.2.1 | 1,0 |
| Production control group II see pt. 8.2.2 | 1,1 |

Table 4.3.2 c Coefficient for control of calculation $\gamma_3$

| Control of calculation | $\gamma_3$ |
|---|---|
| Complete control of all calculations and workshop drawings executed by other person | 1,0 |
| Control only of main forces and main dimensions executed by other persons. Self control | 1,05 |
| No control[1)] | 1,1 |

1) $\gamma_3$ for no control should not be used where rupture can lead to large consequence.

Table 4.3.2 d Coefficient for consequence of rupture $\gamma_4$

| Structure type | $\gamma_4$ |
|---|---|
| Structures where rupture will cause small economic consequences or insignificant possibility of injury to people | 1,0 |
| Structures in small houses | 1,05 |
| Structures where rupture can cause large economic consequences or possibility of injury to people | 1,1[2)] |

The material coefficient $\gamma_m$ shall always be assumed larger than

$$\gamma_m \geq \frac{1,4}{\gamma_f} \text{ for load 0 and G + E and}$$

$$\gamma_m \geq \frac{1,1}{\gamma_f} \text{ for load O + E}$$

$\gamma_f$ is effective load coefficient (weighed mean)

O, G + E and O + E are loads according to NS 3052.

For example for O + G $\gamma_f = \frac{1,6\ N + 1,2\ G}{N + G}$

## 4.4 Strength grades. Structural strength

Timber is graded into stress grades with a characteristic value for material strength. This is an approximate value in MPa for the timber's maximum bending strength defined as that value of the maximum bending strength which in short-time testing is exceeded by 95 % of the timber in the relevant grade.

In the following are given coefficients and structural strengths for the standardised strength grades T 30 and T 20. Corresponding values are also given for timber with characteristic value 4o MPa.

---

2)The coefficient is given on the assumption that systematic control of condition and maintenance are performed where rupture in the structure can cause injury to people.

Timber which fulfills the requirements for grade E in NS 3080 belongs to strength grade T 30, and timber which fulfills the requirements for grade S belongs to strength grade T 20.

Table 4.4.1 Structural strengths in MPa (values are to be multiplied by the factors in Tables 4.5 and 4.6)

| Structural strength for | | Charact. value 40 | Strength grade | |
|---|---|---|---|---|
| | | | T 30 | T 20 |
| Bending | $\sigma_b$ | 26,0 | 20,0 | 14,0 |
| Tension parallel to the grain | $\sigma_{s\parallel}$ | 24,0 | 18,0 | 10,0 |
| Tension perpendicular to the grain | $\sigma_{s\perp}$ | 0,3 | 0,3 | 0,3 |
| Compression parallel to the grain[3] ($\lambda < 20$) | $\sigma_{t\parallel}$ | 22,0 | 18,0 | 14,0 |
| Compression perpendicular to the grain[4] [5] [6] | $\sigma_{t\perp}$ | 4,0 | 4,0 | 4,0 |
| Shear parallel to the grain | $\tau_\parallel$ | 2,0 | 2,0 | 2,0 |
| Shear perpendicular to the grain | $\tau_\perp$ | 1,0 | 1,0 | 1,0 |
| Modulus of elasticity parallel to the grain | $E_\parallel$ | 6000 | 5000 | 4000 |

For finger jointed timber of strength grade T 30 or better, structural strength for tension parallel to the grain ($\sigma_s$) is 16 MPa. For finger jointed timber of strength grade T 20 the table values apply.

---

3) For $\lambda > 20$, see pt. 5.3

4) For $\sigma_{t\perp}$ for short loaded lengths, see pt. 5.5.1

5) For compressed surfaces which form an angle $\alpha$ with the grain, see pt. 5.5.2

6) As long as some compression does not hurt the structure, $\sigma_{t\perp}$ can be increased by 25 %.

Table 4.4.2 E- and G-modulus in MPa for calculation of deformation

| E- and G-modulus for calculation of deformation | | Charact. value 40 | Strength grades T 30 | T 20 |
|---|---|---|---|---|
| Modulus of elasticity parallel to the grain | $E_{\parallel}$ | 11 000 | 9 000 | 7 000 |
| Modulus of elasticity perpendicular to the grain | $E_{\perp}$ | 300 | 250 | 200 |
| Shear modulus parallel to the grain | $G_{\parallel}$ | 600 | 600 | 600 |
| Shear modulus perpendicular to the grain | $G_{\perp}$ | 40 | 40 | 40 |

Table 4.4.3 Factor $k_L$

| Stress | Mean value 40 | Strenght grades T 30 | T 20 |
|---|---|---|---|
| $\sigma_b$, $\sigma_{s\parallel}$, $\sigma_{t\parallel}$ | 1,2 | 1,3 | 1,4 |
| $\sigma_{s\perp}$, $\sigma_{t\perp}$ | 1,0 | 1,0 | 1,0 |
| $\tau_{\parallel}$, $\tau_{\perp}$ | 1,2 | 1,2 | 1,2 |
| E- and G-modulus | 1,2 | 1,2 | 1,2 |

For glued laminated timber which satisfies the requirements for production control group I (see pt. 8.2), the structural strength and stiffness values according to Table 4.4.1 and Table 4.4.2 are multiplied by the factor $k_L$ given in Table 4.4.3.

For glued laminated timber in production control group II the factor $k_L$ = 1,0 applies.

For glued laminated timber where the cross section contains lamellae of two strength grades, the structural strength is determined by the following rules:

The lowest lamella quality in the whole cross section for tension and compression (and buckling) parallel to the grain, bending on an axis vertical on the glued joint plane and by shear stresses.

The quality in the other lamellae (all lamellae which wholly or partially are in the outer one sixth of the cross-section height) by bending on an axis parallel to the glued joint plane.

## 4.5 Loading groups

The loads are divided into the following groups according to duration:

Load grade A, long-term loads

Dead loads, snow loads, water pressure, earth pressure. Imposed loads in residences, offices, hospitals, schools and other imposed loads of prolongued character. Loads of people in assembly rooms and other public rooms.

Load grade B, short-term loads

Occasional person load (point load) on a roof etc. Imposed loads on platforms and scaffoldings, concrete formwork and similar temporary structures[7]. Forces from temperature and moisture variations, braking and acceleration forces, mooring forces from craft. Mobile loads centrifugal forces on bridges and cranes.

Load grade C, very short-term loads

Wind load. Impacts and shaking in addition to imposed loads.

Imposed loads on temporary structures can in certain cases be put in load group C.

For combination of different load groups the values from Table 4.4.1 and Table 4.4.2 are multiplied by the factor $k_k$ given in Table 4.5.

---

7) By temporary structures is meant structure with a service time of up to 3 years.

Table 4.5 Factor $k_k$

| Load grades | $k_k$ | |
|---|---|---|
| | For structural strength Table 4.4.1 | For calculation of deformation Table 4.4.2 |
| A | 1,0 | 1,0 |
| A + B or B | 1,2 | 1,1 |
| A + B + C, A + C, B + C or C | 1,4 | 1,2 |

## 4.6 Climate grades

Structures shall be classified in climate grades in accordance with the moisture content in the atmosphere which surrounds the structures during use.

To climate grade 1 can be assumed amongst others:

Structure members inside rooms which normally are heated.

Loft joist floor and bearing roof structures in cold, but ventilated loft rooms over rooms which normally are heated.

Outer walls in buildings which normally are heated and which are protected by ventilated, impervious outside plating and damp-proof layer on the warm side.

To climate grade 2 can be assumed amongst others:

Structure elements in buildings which normally are not heated, but are ventilated.

Premises with non-moisture producing acitivity or storage, eg. leisure dwellings.

Roof boarding.

Scaffolding, formwork and similar temporary structures.

To climate grade 3 can be assumed amongst others:

Structures which are not protected against damp or rain.

Structures which are in direct contact with the ground.

Values given in Tables 4.4.1 and 4.4.2 shall be multiplied by the factor $k_f$ according to climate grade.

Table 4.6 Factor $k_f$

| Climate grade | 1 | 2 | 3 |
|---|---|---|---|
| Relative humidity (R.H.) in %[8] | R.H.<65 | 65<R.H.<85 | R.H.>85 |
| Factor $k_f$ | 1,0 | 1,0 | 1,0 |
| Equivalent wood moisture content f (approximate value) in % | f<12 | 12<f<20 | f>20 |

## 4.7 Design by testing

4.7.1 Capacity, or properties at lower loading, can be determined by prototype testing instead of calculations or by testing combined with calculations.

The rules apply to testing which shall form the basis for evaluation of compound structures of the same type. The test shall be representative for the product. The rules do not apply for test loading of completed structures.

Results arrived at by testing apply before calculations with regard to the properties which are examined by the testing. The testing does not justify deviation from rules which also shall take care of other properties than those covered by the testing.

Execution and control shall be the same as for structures designed on the basis of calculation.

---

8) Short-term excess (a few consecutive days) with 10 % R.H. can be allowed.

4.7.2 A test report shall be prepared where at least shall be evident the size and type of load, maximum deflection and downward bending after unloading, stretching and description of conditions which are of importance for deciding whether the functional requirements are satisfied.

4.7.3 The testing shall be executed by or checked by an officially-recognised testing institution.

# 5 ULTIMATE LIMIT STATE

## 5.1 Bending and axial force

5.1.1 The capacity for bending and axial force can normally be determined from the assumption that plane cross sections remain plane after deformation. Where no more exact methods are used, the capacity can be calculated according to the following rules.

5.1.2 The capacity for central axial force[9] is determined by

$N_d = A\sigma_d$ ($\sigma_d$ is design tension or compression strenght)

5.1.3 The moment capacity for simple bending for solid cross sections (including laminated wood) is determined by

$M_d = W\sigma_d$ ($\sigma_d$ is design bending strength)

Laminated wood members which are fabricated with a curvature have reduced capacity.

Moreover for structures with simple curved axis of length, unequal distribution of stress across the cross section must also be taken into consideration.

The capacity is determined by

$M_d = Ck_l\ W\sigma_d$

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[9] for buckling see pt. 5.3.3.

C is given in Table 5.1.3 and $k_i$ in Table 5.1.3.2.

Table 5.1.3.1

| $\frac{t}{r_i}$ | $\frac{1}{100}$ | $\frac{1}{125}$ | $\frac{1}{150}$ | $\frac{1}{200}$ | $< \frac{1}{250}$ |
|---|---|---|---|---|---|
| C | 0,80 | 0,87 | 0,90 | 0,95 | 1,00 |

t = lamell thickness

$r_i$ = smallest radius of curvature

Table 5.1.3.2

| $\frac{r_m}{h}$ | 2 | 3 | 4 | 6 | 8 | 10 | ≥ 15 |
|---|---|---|---|---|---|---|---|
| $k_i$ | 0,83 | 0,89 | 0,92 | 0,94 | 0,96 | 0,97 | 1,00 |

$r_m$ = mean radius of curvature

h = height of cross section

Moreover for simple curved structures the capacity is limited by

$M_d \leq \frac{2}{3} r_m A \sigma_d$ ($\sigma_d$ is here equal to the value for $\sigma_{s\perp}$ respectively $\sigma_{t\perp}$).

The formula applies for rectangular cross section and constant height.

5.1.4 For structures influenced by moment and axial force[10] the capacity shall be checked thus:

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} + \frac{N}{N_d} \leq 1$$

$M_x$ is moment on the cross section's x-axis and $M_y$ is moment on the y-axis.

---

10) for buckling see pt. 5.3.4.

## 5.2 Shear force

5.2.1 The capacity for simple shear force is generally determined by

$$V_d = \frac{\tau_d \; I \; b}{S}$$

where I, S and b refer to the cross section's centre of gravity axis.

For rectangular cross sections with constant height

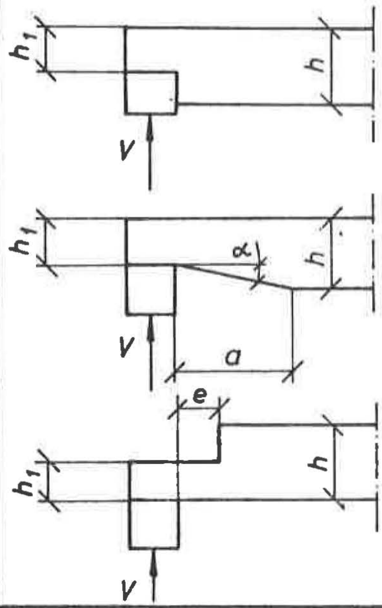
$$V_d = \frac{2}{3} \tau_d A$$

5.2.2 For rectangular notched beams the following applies:

$$V_d = \frac{2}{3} k_\tau \tau_d A$$

$k_\tau$ is given in Fig. 5.2.2.

$A = h_1 b$




$$k_\tau = \frac{h_1}{h}$$

For $\alpha \leqslant 30°$ $k_\tau = 1$

For $30° < \alpha < 90°$ $k_\tau = \frac{\sqrt{3} \cdot h_1 + a}{\sqrt{3} \cdot h}$

For $e \leqslant 0{,}6\,h$ og $h_1 \geqslant 0{,}6\,h$ er $k_\tau = \frac{h - e(\frac{h}{h_1} - 1)}{h_1}$

For $e > 0{,}6\,h$ eller $h_1 < 0{,}6\,h$ er $k_\tau = 1$

Fig. 5.2.2

## 5.3 Buckling

5.3.1 The buckling length is generally assumed as equal to the distance between two fixed points.

Struts which are partially or completely restrained, and struts which are secured sideways by elastic supports (eg. in open frames), must be examined more precisely.

5.3.2 For single columns with solid or glued cross section the slenderness ratio is

$$\lambda = \frac{l_k}{i}$$

$$i = \sqrt{\frac{I}{A}}$$

For rectangular cross sections $i = 0,29\ t$

where t is the column's thickness in the buckling direction.

For circular cross sections $i = 0,25\ d$

where d is the cross section's diameter.

For round timber i is calculated for a cross section at a distance of $\frac{l}{3}$ from the top end.

The greatest allowed slenderness ratio is 170.

5.3.3 The capacity for central axial force is

$$N_d = k_\lambda A \sigma_d$$

$k_\lambda$ is given in Table 5.3.3.

Table 5.3.3

| $\lambda$ = | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 |
|---|---|---|---|---|---|---|---|---|
| $k_\lambda$ = | 1,00 | 0,90 | 0,81 | 0,71 | 0,62 | 0,52 | 0,43 | 0,34 |
| $\lambda$ = | 100 | 110 | 120 | 130 | 140 | 150 | 160 | 170 |
| $k_\lambda$ = | 0,28 | 0,23 | 0,19 | 0,16 | 0,14 | 0,12 | 0,11 | 0,10 |

5.3.4 For columns influenced by moment and axial force, the capacity is checked thus:

$$\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} + \frac{N}{N_d} \leq 1$$

$M_x$ is moment on the cross section's x-axis

$M_y$ is moment on the y-axis

$N_d$ is determined for the appropriate direction of buckling

For build-up columns the largest moment value within the middle $\frac{1}{3}$ part of the column's length is used.

5.4 Lateral buckling

5.4.1 An investigation of lateral buckling can generally be omitted. However, for beams of great length and slenderness which can occur in laminated wood structures, such an investigation is necessary.

Where no more exact methods are used, then the possibility of lateral buckling can be taken into account for beams with rectangular cross section according to the following rules.

5.4.2 For h $\leq$ b it is not necessary with side bracing and the capacity of moment is as given in pt 5.1.3.

5.4.3 For h > b the beam at the bearings shall be braced so that a rotation on the beam's axis of length is prevented (fork support).

The beam's slenderness ratio $\lambda_v$ is given as

$$\lambda_v = \sqrt{\frac{l_e \cdot h}{b^2}}$$

where $l_e$ is taken from Table 5.4.3.

Table 5.4.3

| Conditions of bearing | Load | $l_e$ for load working parallel on the beam's neutral axis[11] |
|---|---|---|
| Freely supported | Single load at the middle | 1,4 · I |
| Freely supported | Uniformly distributed | 1,6 · I |
| Freely supported | Constant moment | 1,8 · I |
| Bracketed | Single load at the end | 1,4 · I |
| Bracketed | Uniformly distributed | 0,9 · I |
| Others | Arbitrary | 1,8 · I |

I = free length

Normally the free length l is assumed as the distance between the bearings, or for a bracketed beam its actual length.

If the beam is supported at certain points between the bearings so that twisting or sideways bending on the compression side are effectively prevented, then the free length is taken to be the distance between these supports.

5.4.4 The capacity of moment is determined by

$$M_d = k_v W\sigma_d$$

$k_v$ is given in Table 5.4.4.

Table 5.4.4

| $\lambda_v$ | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 |
|---|---|---|---|---|---|---|---|---|---|
| $k_v$ | 1,0 | 0,85 | 0,70 | 0,55 | 0,40 | 0,29 | 0,23 | 0,18 | 0,14 |

11) For downward directed load on upper side $l_e$ is increased by $3 \cdot h$
For downward directed load on lower side $l_e$ is reduced by h.

## 5.5 Partial loading

5.5.1 For short loading lengths a higher compression stress is allowed on the loading surface as long as the following conditions are complied with:

The distance from loaded surface to the timber's end is at least 75 mm and to other loaded surface at least 150 mm. Se Fig. 5.5.1.

The value of $\sigma_{t\perp}$ according to Table 4.4.1 can then be increased by the factor $k_s$ according to Table 5.5.1.

Table 5.5.1

| Loading length l in mm | 10 | 23 | 36 | 48 | ≥98 |
|---|---|---|---|---|---|
| $k_s$ | 1,8 | 1,5 | 1,3 | 1,2 | 1,0 |

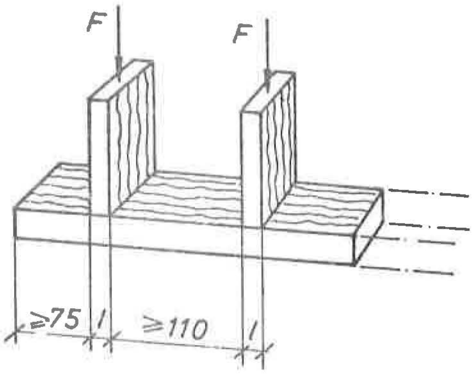



Fig. 5.5.1

5.5.2 For compression surfaces where the force forms an angle $\alpha$ parallel to the grain, the values for $\sigma_{t\alpha}$ are determined by

$$\sigma_{t\alpha} = \sigma_{t\parallel} - (\sigma_{t\parallel} - \sigma_{t\perp}) \sin \alpha$$

for $0 \leq \alpha \leq 90^{\circ}$

PAPER 22

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH

STUDIES AND DOCUMENTATION

WORKING COMMISSION W18

# DRAFT FOR REVISION OF CP 112 "THE STRUCTURAL USE OF TIMBER"

by

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ENGLAND

PARIS - FEBRUARY 1975

DRAFT REVISION OF CP 112

This paper gives an outline for the proposed contents of the revised edition of British Standard Code of Practice CP 112. In addition the completed drafts for the first five chapters are included. It must, however, be emphasised that these are draft documents, and although agreed in principle, may nevertheless be subsequently modified or altered.

CONTENTS

## 1 GENERAL

### 1.1 Scope

This Code of Practice deals with the structural use of timber, plywood and other wood-based sheet materials. It is based on material properties established by research, on performance testing and on practical experience. In drafting it has been assumed that the design of structures is entrusted to appropriately qualified engineers, for whose guidance the Code has been prepared, and that the execution of the work is carried out under the direction of qualified supervisors.

Where information was lacking from British sources recourse was made to information from laboratories in the USA, Canada, Australia, Sweden and Finland to whom due acknowledgement is made. The valuable contributions of the Council of Forest Industries of British Columbia, the Swedish Timber Council and the Finnish Plywood Development Association are also acknowledged.

In common with other material Codes this Code recommends procedures which permit the adoption of a limit state approach to the design of timber structures. However, because of the number of properties involved and the different procedures by which these are derived characteristic strength values and partial safety factors for materials have not been specified. Instead these have been incorporated in the tabulated design stresses for the materials and in the strength values for the jointing devices. In this way the designer is relieved of one arithmetical operation.

The design stresses and strength values apply to long term loading under a dry exposure condition and modification factors are given to enable these to be adjusted to other service conditions. Provision is also made for the establishment of design stress values from the results of standard laboratory tests, for materials and jointing devices not specifically included in the Code.

In order that account may be taken of the effect of duration of load on strength four categories for duration of load are defined, together with their corresponding stress modification factors. Partial safety factors for loads are included to permit design loads to be determined for the limit states of ultimate strength and deflection.

As an equally acceptable alternative to design calculations, a procedure is recommended whereby the ability of a structure to satisfy its functional requirements for strength and deflection may be assessed from tests on full size units.

## 1.2 Definitions

Where timber terms are used they have the meaning assigned to them in CP3 Chaper V: Parts 1 and 2 and in BS 565, BS 4471 and BS 4978[1]. In addition the following definitions apply:

Basic characteristic stress. The value of the ultimate stress at the dry exposure condition, derived from standard tests on small clear specimens[2] below which not more than 5 per cent of test results fall.

Basic design stress. The stress derived from the basic characteristic stress by dividing by the appropriate partial safety factor for strength, and adjusting to the long term loading condition, and in the case of bending stress to a section depth of 200 mm.

Characteristic dead load. A load of long term duration such as the dead loads defined in CP 3:Chap V:Part 1.

Characteristic imposed load. A load which may be of long, medium or short term duration such as the imposed loads specified in CP 3:Chap V:Part 1.

Characteristic wind load. A wind load which may be of short or very short term duration such as the wind loads defined in, and derived in accordance with CP 3: Chap V:Part 2

Design load. The load determined by multiplying the characteristic dead, imposed and wind loads by partial safety factors appropriate to the limit states of ultimate strength stability and deflection.

Dry exposure. An exposure condition where the moisture content of timber will not exceed 18 per cent for any significant period, as for example, in most covered or internal uses.

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[1] BS 565 Glossary of terms relating to timber and woodwork.
BS 4471 Dimensions for softwood
BS 4978 Timber grades for structural use

[2] BS 373 Methods of testing small clear specimens of timber

Glued laminated timber. A member produced by gluing together a number of laminations having their grain directions essentially parallel.

Grade characteristic stress. The value of ultimate stress, at the dry exposure condition, derived from standard tests on full size specimens of a particular grade[3] below which not more than 5 per cent of the test results fall.

Grade design stress. The stress derived either:
a) by dividing the grade characteristic stress by the partial safety factor for strength and adjusting to the long term load condition, and in the case of bending stress to a section depth of 200 mm or,
b) by multiplying the basic design stress by the grade strength ratio for solid timber and by the combined grade and number of laminations factor for glued laminated timber.

Grade strength ratio. The ratio of the strength of timber of a particular grade to the strength of timber free from defects.

Horizontally laminated beam. A beam with the laminations parallel to the neutral plane.

Joint characteristic strength. The value of ultimate strength for a single fastener at the dry exposure condition derived from standard tests on sample joints[4] below which not more than 5 per cent of test results fall.

Joint design strength. The strength of a single fastener derived from the joint characteristic strength by dividing by the appropriate partial safety factor for strength and adjusting to the long term load condition.

Long term load. A load which acts, or may be considered to act, permanently on a member or structure.

Medium term load. A load which acts, or may be considered to act, from time to time for prolonged periods.

Member. A structural component which may be a piece of solid timber, laminated timber or built up from pieces of timber and/or other sheet materials as for example floor joist, box beam, member of a truss.

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(3) No BS as yet available, see PRL Special Report No 19.
(4) A BS is being written.

Modified design stress. The stress, determined by multiplying the basic design stress or grade design stress by modification factors, which is applicable to the design of a member of a particular grade and size under the conditions of exposure, loading etc, to which the member will be subjected in service.

Modified joint design strength. The strength, determined by multiplying the joint design strength by modification factors, which is applicable to the design of a joint under the conditions of exposure, loading etc, to which the joint will be subjected in service.

Short term load. A load which acts. or may be considered to act, from time to time for short periods.

Structural unit. An assembly of members forming the whole or part of a framework or building, as for example, a truss, floor panel, skeleton of a building or a complete building.

Vertically laminated beam. A beam whose laminations are at right angles to the neutral plane.

Very short term load. A load which acts, or may be considered to act, from time to time for very short periods.

Wet exposure. An exposure condition where the moisture content of timber will exceed 18 per cent for a significant period, as for example, in external uses and in contact with water or exposed to saturated air.

1.3 Symbols

The symbols used in this Code are as follows:

| | |
|---|---|
| A | Area of cross-section |
| b | Breadth of beam or joist, thickness of web or least dimension of a tension or compression member. Where there is more than one such dimension $b_1$, $b_2$ etc., indicate their several values. |
| d | Diameter |
| E | Average value of modulus of elasticity |
| $E_k$ | Characteristic value of modulus of elasticity |

| | |
|---|---|
| $F$ | Strength, general |
| $F_m$ | Modified strength, general |
| $\sigma_b$ | Strength in bending |
| $\sigma_{bk}$; $\sigma_{bgk}$ | Basic characteristic strength and grade characteristic strength in bending parallel to the grain, respectively |
| $\sigma_{bd}$; $\sigma_{bgd}$ | Basic design strength and grade design strength in bending parallel to the grain, respectively |
| $\sigma_{bm}$ | Modified design strength in bending parallel to the grain |
| $\sigma_c$ | Strength in compression |
| $\sigma_{ck}$; $\sigma_{cgk}$ | Basic characteristic strength and grade characteristic strength in compression parallel to the grain, respectively |
| $\sigma_{cd}$; $\sigma_{cgd}$ | Basic design strength and grade design strength in compression parallel to the grain, respectively |
| $\sigma_{cm}$ | Modified design strength in compression parallel to the grain |
| $\sigma_{c\perp}$ | Strength in compression perpendicular to grain |
| $\sigma_{c\perp k}$; $\sigma_{c\perp gk}$ | Basic characteristic strength and grade characteristic strength in compression perpendicular to the grain, respectively |
| $\sigma_{c\perp d}$; $\sigma_{c\perp gd}$ | Basic design strength and grade design strength in compression perpendicular to the grain, respectively |
| $\sigma_{c\perp m}$ | Modified design strength in compression perpendicular to the grain |
| $\sigma_t$ | Strength in tension |
| $\sigma_{tk}$; $\sigma_{tgk}$ | Basic characteristic strength and grade characteristic strength in tension parallel to the grain, respectively |
| $\sigma_{td}$; $\sigma_{tgd}$ | Basic design strength and grade design strength in tension parallel to the grain, respectively |
| $\sigma_{tm}$ | Modified design strength in tension parallel to the grain |
| $\tau_v$ | Strength in shear |
| $\tau_{vk}$; $\tau_{vgk}$ | Basic characteristic strength and grade characteristic strength in shear parallel to the grain, respectively |
| $\tau_{vd}$; $\tau_{vgd}$ | Basic design strength and grade design strength in shear parallel to the grain, respectively |
| $\tau_{vm}$ | Modified design strength in shear parallel to the grain |
| $G$ | Modulus of rigidity |
| $G_k$ | Characteristic dead load |
| $h$ | Depth of beam or joist, greater transverse dimension of a tension or compression member. Where there is more than one such dimension, $h_1$, $h_2$ etc., indicate their several values |
| $I$ | Second moment of area |
| $i$ | Radius of gyration |
| $K$ | Modification factor, the several values of which are identified |

| | |
|---|---|
| | $K_1$, $K_2$, $K_3$ etc. |
| $l$ | Length, effective span |
| $M$ | Moment |
| $N$ | Normal force, axial force |
| $Q_k$ | Characteristic imposed load, where the added suffixies 1, 2, 3 and 4 indicate that the load is of long-term, medium term, short or very short term duration, respectively |
| $R$ | Radius of curvature |
| $S$ | First moment of area |
| $t$ | Thickness of laminations |
| $V$ | Shear force |
| $W_k$ | Characteristic wind or impact load, where the added suffixes 3 and 4 indicate that the load is of short term or very short term duration, respectively |
| $Z$ | Section modulus |
| $\alpha$ | Angle between the direction of the load and the direction of the grain |
| $\gamma_f$ | Partial safety factor for load |
| $\gamma_m$ | Partial safety factor for material strength |
| $\theta$ | Angle between the longitudinal axis of the member and the joint axis |

Note: It is recommended that on the completion of the revision of the Code these symbols should be reviewed.

## 2 MATERIALS

### 2.1 General

The materials used should comply with the appropriate British Standard, where such exists, or with other national standards as indicated.

TIMBER. The following standards apply to timber:

| | |
|---|---|
| Nomenclature of commercial timbers, including sources of supply | BS 881:589 |
| Specification for dimensions for softwood | BS 4471 |
| Specification for dimensions for softwood, Part 2 | BS 4471:Part 2 |
| Grading and sizing of softwood flooring | BS 1297 |
| Timber grades for structural use | BS 4978 |
| National lumber grading and dressing rules | NLGA:1970 |

LAMINATED TIMBER. The following standards apply to laminated timber:

| | |
|---|---|
| Glued laminated timber structural members | BS 4169 |

PLYWOOD. The following standards apply to plywood:

| | |
|---|---|
| British made plywood for marine craft | BS 1088 |
| Plywood manufactured from tropical hardwoods | BS 1455 |
| Information about plywood | BS 3493 |
| British made plywood for structural purposes | ABPM Sec BP 101 |
| Douglas fir plywood | CSA Std 0121-1973 |
| Canadian softwood plywood | CSA Std 0151-1974 |
| Finnish plywood | Fin Std 0.IV.1(?) |
| Swedish plywood | Swedish type approval T1997/72 and T2360/72(?) |

BOARD MATERIALS. The following standards apply to board materials:

| | |
|---|---|
| Resin bonded wood chipboard | BS 2604:Part 2 |

| | |
|---|---|
| Methods of test for wood chipboard and other particle boards | BS 1811:Part 2 |
| Fibre building boards: Methods of test | BS 1142:Part 1 |
| Fibre building boards: Medium board and hardboard | BS 1142:Part 2 |
| Fibre building boards: Bitumen impregnated | BS 1142:Part 3 |
| Blockboard and laminboard | BS 3444 |

MECHANICAL FASTENERS. The following specifications apply to mechanical fasteners:

| | |
|---|---|
| Black bolts, screws and nuts | BS 916 |
| Nails | BS 1202 |
| Aluminium nails | BS 1202:Part 3 |
| Wood screws | BS 1210 |

ADHESIVES. The following specifications apply to adhesives:

| | |
|---|---|
| Synthetic resin adhesives (phenolic and aminoplastic) for plywood | BS 1203 |
| Synthetic resin adhesives (phenolic and aminoplastic) for wood | BS 1204 |
| Cold setting casein glue for wood | BS 1444 |

PRESERVATIVES. The preservative treatment of timber is dealt with in CP 112:Part 5 and the following standards apply:

| | |
|---|---|
| Coal tar creosote for the preservation of timber | BS 144 |
| Pressure creosoting for timber | BS 913 |
| Classification of wood preservatives and their methods of application | BS 1282 |
| Copper/chrome water-borne wood preservatives and their application | BS 3452 |
| Fluoride/arsenate/chromate/dinitrophenol water-borne wood preservatives and their application | BS 3453 |
| Treatment of plywood with preservatives | BS 3842 |

## 2.2 Species of Timber

This Code is based on a limited number of species of timber (see Table 1) which are likely to be generally available. Because sources of supply change it is not possible to include an exhaustive list and for the use of other species reference may be made to the Princes Risborough Laboratory, or other recognised authority.

Timber may be available as individual species or in parcels containing a number of species having generally similar strength properties. For timber imported from Canada, softwood species are grouped into three main species groups (see Table 2) and mixed parcels of European redwood and whitewood may also be imported from Sweden and Finland. Provision is made within the Code for the use of species groups from these sources without the need for resorting.

Where designers wish to take full advantage of the strength properties of a particular species, and where they are sure of supply, the species should be specified and the appropriate design stresses used. As an alternative, design may be based on the stress values specified for any of the six strength classes recommended in the Code. These are independent of species or grade and have been included to provide for the supply of a wider range of species, especially the hardwood species, and to broaden the scope of application of machine stress grading.

## 2.3 Plywood

The production of plywood has in recent years undergone a radical change and it is now the exception for individual sheets to be made from veneers of a single species, particularly for the major imports from Canada, Finland and Sweden. Thus plywood specified as Canadian Douglas fir will have faces and backs of this species and inner plies of any of some eleven other softwood species. Although all-birch plywood will continue to be available from Finland, birch-faced plywood containing inner plies of spruce will become more readily available. Swedish plywood made from spruce and pine veneers and plywoods from other sources will also become available.

Account has been taken of these changes and strength data are tabulated for those plywoods which are readily available and provision is made for the determination of strength values from test results and by predictive methods for other types of plywood (see Chapter

## 2.4 OTHER BOARD MATERIALS

Increasing structural use is being made of board materials, particularly particle board for flooring and hardboard for structural webs and cladding. It must be recognised however that these materials are produced to a wide range of qualities, with widely different strength and performance characteristics, and experience with their use is much more limited than for plywood. Some caution is therefore needed before specifying them for structural applications and, where there is doubt, advice should be sought from an appropriate authority.

Limited strength data are given for oil-tempered hardboard and for blockboard and laminboard, satisfying the requirements of BS 1141 and BS 3444. These data may be used in design providing the loading and service conditions specified in this Code are complied with (see Section

TABLE 1 NAMES AND DENSITIES OF SOME STRUCTURAL TIMBERS

| Standard Name | Botanical species | Other common names | Approximate Density at a moisture content of 18 per cent kg/$m^3$ |
|---|---|---|---|
| SOFTWOODS | | | |
| Douglas fir | *Pseudotsuga menziesii* | B C pine<br>Oregon pine | 590 |
| Western hemlock | *Tsuga heterophylla* with *Abies* spp and *Tsuga mertensiana* | Hembal<br>Hem-fir | 530 |
| Larch | *Larix decidua*<br>*Larix leptolepsis* | - | 560 |
| Parana pine | *Araucaria augustifolia* | - | 560 |
| Pitch pine* | *Pinus palustris*<br>*Pinus elliotti*<br>*Pinus caribaea* | Longleaf pitch pine<br>N caraguan pitch pine<br>Honduras pitch pine | 720 |
| Redwood or Scots pine | *Pinus sylvestris* | Baltic redwood, European redwood, deal, Swedish pine | 540 |
| Canadian spruce | Mainly *Picea glauca*, *Picea mariana* *Pinus contorta* and *Abies* spp | Princess spruce<br>Western white spruce | 450 |
| Home-grown European spruce | *Picea abies* | Norway spruce | 380 |
| Home-grown Sitka spruce | *Picea sitchensis* | - | 400 |
| Whitewood | *Picea abies*, *Abies alba* | Baltic whitewood, European whitewood, white deal | 510 |
| HARDWOODS | | | |
| Greenheart | *Ocotea rodiaei* | - | 1060 |
| Gurjun/keruing | *Dipterocarpus* spp | - | 720 |
| Iroko | *Chlorophora excelsa* | mvule | 690 |
| Jarrah | *Eucalyptus marginata* | - | 910 |
| Karri | *Eucalyptus diversicolor* | | 930 |
| Oak | *Quercus robur* | - | 720 |
| Opepe | *Nauclea diderrichii* | kusia | 780 |

* Some specifications admit *Pinus echinata*, *P taeda*, *P rigida* and *P virginiana* which are considerably lighter and of lower strength. The data given in this Code do not apply to these species.

TABLE 2 NAMES AND DENSITIES OF CANADIAN SPECIES COMBINATIONS

| Standard name | Botanical species | Other common names | Approximate density at a moisture content of 18 per cent |
|---|---|---|---|
| | | | $kg/m^3$ |
| Douglas fir-Larch | *Pseudotsuga menziesii*<br>*Larix occidentalis* | BC pine<br>Western larch | 590 |
| Hem-Fir | *Tsuga heterophylla*<br>*Tsuga mertensiana* Carr<br>*Abies amabilis*<br>*Abies grandis* | Western hemlock<br>Mountain hemlock<br>Amabilis fir<br>Grand fir | 530 |
| Princess spruce* | *Picea glauca*<br>*Picea rubens*<br>*Picea mariana*<br>*Abies balsamea*<br>*Pinus banksiana* | White spruce<br>Red spruce<br>Black spruce<br>Balsam fir<br>Jack pine | 450 |
| Western white* spruce | *Picea glauca*<br>*Picea engelmannii*<br>*Picea mariana*<br>*Pinus contorta*<br>*Pinus banksiana*<br>*Abies lasiocarpa*<br>*Abies balsamea* | White spruce<br>Engelmann spruce<br>Black spruce<br>Lodgepole pine<br>Jack pine<br>Alpine fir<br>Balsam fir | 450 |

*Because of similarity of strength properties these two species groups are collectively designated as spruce-pine-fir.

## 3 MOISTURE CONTENT

### 3.1 General

The basic factors governing the moisture content of timber and wood based sheet materials are:-

1 Wood is a hygroscopic material so that its moisture content depends on the conditions under which it is exposed.

2 Unless wood is in contact with water or exposed to very damp conditions, its moisture content stabilizes, in most cases, at between 10 and 20 per cent, which is very much lower than when the timber is freshly felled. The drier the atmosphere, and the higher the temperature, the lower the moisture content which the wood attains.

3 At a moisture content below about 30 per cent, wood shrinks or swells as its moisture content changes.

4 The strength properties of wood change with changes in moisture content below about 30 per cent, a decrease in moisture content producing an increase in strength and vice versa.

5 Wood is less prone to decay if its moisture content is below 25 per cent and may be regarded as immune below 20 per cent.

6 Softwood imported from Europe is normally dried to a moisture content below 23 per cent before shipment to the UK. Softwood of Canadian origin will generally be unseasoned, but dipped in a fungal inhibitor before shipment. Packaging and other changes in timber production and handling may however alter this traditional pattern with a trend towards the imports, particularly of the smaller dimensions, being drier. Imported hardwoods from all sources are normally only dried before shipment to such an extent as will avoid deterioration during passage to the UK. The moisture content at time of arrival may be anywhere within a wide range, but further drying is normally done after arrival. Supplies of dried hardwood are generally available.

7 Plywood and other wood based board materials are produced in a relatively dry condition with moisture contents within the range 6 to 14 per cent. They undergo changes in strength and dimensions (of which thickness is affected the most) with changes in exposure conditions. Their hygroscopic properties differ from those of timber and under the same exposure conditions their equilibrium moisture contents can be 3 or more per cent lower.

## 3.2 Service Requirements

Timber and board materials when installed in a building should ideally have a moisture content close to the equilibrium values they will attain in service. However, only the average values, or ranges of values, for various applications can reasonably be stated, and these can vary quite considerably, depending on the type and location of the building and other factors. Also individual species, and even pieces of the same species, can attain quite different equilibrium moisture contents when exposed in the same environment. A precise specification of moisture content is therefore not possible but Table 3 gives guidance as to the appropriate ranges of moisture content to which timber should be dried for a number of end-uses.

TABLE 3 MOISTURE CONTENT OF TIMBER FOR VARIOUS END-USES

| Position of timber in building | Moisture Content (Per Cent) | | | | |
|---|---|---|---|---|---|
| | Nominal Specification | Parcel Acceptance Levels | | | |
| | | Averages | | Any Piece | |
| | | Max | Min | Max | Min |
| External uses, fully exposed | 18 | 20 | 14 | 24 | 12 |
| Semi-internal uses and intermittently heated interiors | 15 | 16 | 14 | 19 | 12 |
| Internal uses, centrally heated | 12 | 13 | 11 | 16 | 9 |
| Internal uses, close to heat source | 9 | 10 | 8 | 13 | 6 |

Care should be taken on site to ensure that material supplied in a dry condition is adequately protected from the weather.

Although some timber, after fixing, may be subject to temporary exposure to the weather and therefore to possible absorption of moisture, it will rapidly dry out after covering, providing there is ample ventilation.

Timber members fixed at higher moisture contents than given in Table 3 will suffer differential and uneven shrinkage in drying out *in situ* to the equilibrium moisture content of the service condition. Where this is acceptable, provision should be made to overcome any loss in strength at joints and fixing points, and to accommodate any resulting displacements and additional deflections.

Consideration should be given to the possible effect of the wet trades on timber and board materials and it may be necessary for these to be completed and their work dried out before fixing timber in centrally-heated exposures or near to heat sources. Adequate ventilation should be provided throughout the drying period to remove the high humidity air from the building.

Solid timber, whose least dimension is more than 100 mm, cannot be reduced to a uniform moisture content within the limits of Table 3, except at a high, and usually uneconomic, cost. This should be taken into account at the design stage, and alternatives to the specification of large sections should be explored.

### 3.3 Effect on Strength

Because of the effect of moisture content on strength the stress values used in design should be those corresponding to the highest moisture content that the particular member will attain in service. It is not possible to cover all service conditions and for the purposes of this Code, two standard conditions only are defined. These are:

DRY EXPOSURE. All service conditions where the moisture content of timber will not exceed 18 per cent for any significant period. This includes most covered exposures and internal uses, except for buildings associated with a high humidity, such as swimming pools, laundries, etc.

WET EXPOSURE. All service conditions where the moisture content of timber will exceed 18 per cent for a significant period.

Strength data are tabulated for the materials and fasteners for the dry exposure condition, and modification factors are included (see Section to enable these to be adjusted to the wet exposure condition. Where solid timber members, whose least dimension is more than 100 mm, are used the design stresses should be for the wet exposure condition irrespective of the actual exposure. For laminated timber and plywood the modification factors for adjustment to the wet exposure condition differ from those for solid timber (see Section

### 3.4 Effect on Geometrical Properties

The actual dimensions of a member will vary with its moisture content and allowance may have to be made for this in determining the geometrical properties of sections for use in design.

The basic sizes of sawn timber are specified in BS 4471 at a moisture content of 20 per cent. For any higher moisture content up to 30 per cent, the size will on average be greater by one per cent for every 5 per cent of moisture content in excess of 20 per cent, and for any lower moisture content will be smaller by one per cent for every 5 per cent of moisture content below 20 per cent. For moisture contents higher than 30 per cent the size will not be greater than at 30 per cent.

Where a timber member is processed to size at the equilibrium moisture content it will attain in service then the geometrical properties of the actual section should be used in design.

Where a member is specified and supplied in accordance with BS 4471 the basic sizes may be used to determine the geometrical properties, at a moisture content of 18 per cent, for members used in the dry exposure condition, allowances being made for resawing and processing as appropriate (see Section If the members are used in the wet exposure condition the geometrical properties may be increased by the following factors:-

| | |
|---|---|
| Breadth (b), depth (h), radius of gyration (i) | 1.02 |
| Area (A) ... ... ... ... ... ... | 1.04 |
| First moment of area (S) and section modulus (Z) | 1.06 |
| Second moment of area (I) ... ... ... | 1.08 |

Where plywood is used in the wet exposure conditions its geometrical properties may be obtained by increasing the tabulated values for the dry exposure (see Section      by the following factors:-

| | |
|---|---|
| Breadth (h) ... ... ... ... ... ... ... | 1.00 |
| Depth (d), area (A) ... ... ... ... ... ... | 1.02 |
| First moment of area (S) and section modulus (Z) | 1.04 |
| Second moment of area (I) ... ... ... ... ... | 1.06 |

## 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 $\gamma_m$, and one for load and load effects $\gamma_f$. 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 no 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.

For the purposes of calculating the deflection of structural units or multi-member systems, where it can be shown that effective lateral distribution of loading occurs, or where a number of pieces of timber act together in such a fashion as to be equally strained under load, the modification factor $K_1$ given in Table 4 may be used to determine the appropriate modified design stress value. For example the value of $K_1$ for 5 or more pieces may be taken as applying to rafters, floor joists and wall studding, where the members are spaced at not more than 600 mm centres. For laminated timber and built up members see Sections and .

TABLE 4

Modification factor $K_1$ for multi-member systems applicable to modulus of elasticity and modulus of rigidity

| Number of Pieces | $K_1$ |
|---|---|
| 2 | 1.12 |
| 3 | 1.18 |
| 4 | 1.21 |
| 5 or more | 1.23 |

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 .

4.2.3 Durability:
To achieve the service life required for a structure consideration should be given, depending on the exposure conditions, to the durability of the timber, adhesives and fasteners and to the need to provide protection against decay, infestation by wood destroying insects and corrosion. Attention should be paid at the design stage to detailing, to avoid moisture traps and during fabrication and erection to the possibilities of moisture absorption and condensation which might adversely affect performance. For durability of the timber reference should be made to Part 5 of this Code.

4.2.4 Fire Resistance:
At the design stage of a structure attention should be paid to achieving the necessary standards of safety in the event of fire. The materials and constructions

used should be capable of satisfying either naturally, or with protective treatment, the requirements for fire resistance, see Part 4 of this Code and for surface spread of flame.

4.2.5 Vibration:
Where there is a likelihood of a structure being subjected to vibration from causes such as impact forces, wind or machinery, measures should be taken to prevent discomfort or alarm, damage to the structure or interference with its proper function.

Various criteria may be used to specify acceptable limits of vibration and reference should be made to the specialist literature. As a guide, the vibrational characteristics of a timber-joist floor construction will generally be acceptable if the deflection of the joists under design load does not exceed 13 mm for floors in dwellings and 6 mm for floors in dance halls, gymnasiums etc.

4.2.6 Other Limit States:
A structure or part of a structure may have to satisfy serviceability limit states for thermal insulation and sound insulation or other limit states associated with unusual or special functions and these should be taken into account in design.

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 loads 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 ($\gamma_f$) ie

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

$\gamma_f$ 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 $\gamma_f$ 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. The notional total duration for this category of loads may be taken as 50 years.

Medium term loads ($G_{k2}$, $Q_{k2}$) which may be either dead or imposed loads and including all 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 and concentrated imposed loads for floors. The notional duration for this category of loads may be taken as up to one month.

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. The notional duration for this category of loads may be taken as up to one hour.

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 notional duration for this category of loads may be taken as up to one minute.

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 $\gamma_f$ 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 $\gamma_f$ 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

$$1.0\ G_k$$

$$1.0\ (G_k + Q_k)$$

$$1.0\ (G_k + W_k)$$

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 ($\gamma_m$) 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.

## 5 TIMBER

### 5.1 Stress Grades

Timber for structural use should be stress graded and should be specified in accordance with the species and grades or with the strength classes included in this Code. Where a strength class is specified the requirements may be met by various combinations of species and visual grades, or by timber graded by machine.

Design calculations should be based on the stress values recommended for the grades or strength classes, modified in accordance with the requirements of this Code.

5.1.1 Visual Grades. Timber stress graded in the UK should be graded to the SS or GS grades specified in BS 4978 and should be appropriately marked.

Timber visually graded in the country of origin should be graded either to BS 4978 or to other approved national rules, and may be accepted providing that the grading is controlled by a recognised authority and stresses for the grades are given in this Code.

Imported timber visually graded as SS or GS in accordance with BS 4978, and bearing the stamp of the respective national grading associations may be used to the stress values for the grades given in this Code.

Timber graded in Canada in accordance with the NLGA (1970) "National Grading Rules (Canada) for Dimension Lumber", may be used to the stress values for the grades given in this Code.

5.1.2 Machine Grades. Timber machine stress graded in the UK should be graded in accordance with the requirements of BS 4978 and should bear the BSI Kitemark.

It may be graded as MSS or MGS, or to one of the strength classes specified in this Code and should be appropriately marked.

Timber machine stress graded in the country of origin should be graded in accordance with BS4978 and should bear the BSI Kitemark or other approved mark. It may be graded as MSS or MGS, or to one of the strength classes specified in this Code.

Where information is available timber may also be machine graded to any required stress level for a particular application, providing that the grading is carried out under the BSI Kitemark Scheme.

5.2 Basic Design Stresses

The basic design stresses for some structural timbers are given in Table 5. These are governed by the general strength characteristics of the particular species, free from visible defects, and apply to the dry exposure, long term loading conditions.

5.3 Grade Design Stresses

Grade design stresses are governed by the limits permitted for defects in the different visual grades and in the case of machine grades, by the control limits under which the machines are operated and by the limits imposed on wane and fissures. Grade design stresses are applicable to the dry exposure, and long term loading conditions.

Where there is no wane the basic design stress in compression perpendicular to grain (ie bearing stress) may be taken as the grade design stress. This relaxation may also be taken as applying to multi-member systems supported at a number of points, for example for the support of a suspended floor.

5.3.1 Timber graded to BS 4978. The grade design stresses for timber graded in accordance with BS 4978 are given in Table 6a for the visual grades SS and GS and in Table 6b for the machine grades MSS and MGS.

NOTE: The extent to which machine stress grading can be applied depends on the available knowledge of the relations between strength and modulus of elasticity and on the operating characteristics of the particular type of grading machine. The species listed in Table 6b are those for which such data are presently available for a reasonable range of section sizes.

5.3.2 Timber graded in Canada. The grade design stresses for timber graded in accordance with the Structural Joists and Planks grades, the Light Framing Grades and the Stud grade of the NLGA (1970) "National Grading Rules (Canada) for Dimension Lumber" are given in Tables 7 to 9. These apply to the dry exposure, long term loading conditions.

5.4 Strength Classes

The grade design stresses for six strength classes for the dry exposure and long term loading conditions are given in Table 10.

5.5 Additional Properties

In the absence of specific test data it is recommended that for tension perpendicula to the grain, torsional shear and rolling shear, grade design stresses which are one-third of the grade design stress in shear parallel to the grain should be used.

5.6 Modified Design Stresses

Modified design stresses for timber are governed by the conditions of service and loading. They should be obtained by multiplying the grade design stresses by the appropriate modification factors given in this Code.

Table 5

BASIC DESIGN STRESSES ($N/mm^2$) FOR THE DRY EXPOSURE CONDITION

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Bending $\sigma_{bd}$ | Tension $\sigma_{td}$ | Compression $\sigma_{cd}$ | Compression perpendicular to grain $\sigma_{c\perp d}$ | Shear $\tau_d$ | Modulus of elasticity $E_k$ | Modulus of rigidity $G_k$ |
|---|---|---|---|---|---|---|---|
| **Imported** | | | | | | | |
| Douglas fir (1) | 24.9 | 17.4 | 14.5 | 4.21 | 2.79 | 9000 | 530 |
| Western hemlock (2) | 20.9 | 14.6 | 12.9 | 3.50 | 2.13 | 8000 | 540 |
| Parana pine | 23.8 | 16.7 | 16.2 | 4.42 | 3.22 | 7400 | 450 |
| Pitch pine | 27.0 | 18.9 | 15.7 | 4.96 | 3.14 | 9000 | 420 |
| Redwood (3) | 21.6 | 15.1 | 12.9 | 3.95 | 2.73 | 6500 | 430 |
| Whitewood (3) | 21.1 | 14.8 | 12.0 | 3.83 | 2.57 | 7100 | 530 |
| Canadian spruce (4) | 18.4 | 12.9 | 11.6 | 3.02 | 2.10 | 6400 | 480 |
| **Home Grown** | | | | | | | |
| Douglas fir | 22.7 | 15.9 | 13.5 | 4.17 | 2.64 | 6600 | 430 |
| Jap larch | 20.3 | 14.2 | 11.5 | 3.98 | 2.75 | 4300 | 280 |
| European larch | 24.9 | 17.4 | 14.5 | 5.50 | 3.07 | 6400 | 380 |
| Scots pine | 22.5 | 15.7 | 13.5 | 4.55 | 2.97 | 6300 | 390 |
| European spruce | 18.5 | 13.0 | 10.7 | 3.38 | 2.41 | 5700 | 450 |
| Sitka spruce | 15.0 | 10.5 | 9.5 | 2.74 | 1.94 | 4800 | 390 |
| **Hardwoods** | | | | | | | |
| Greenheart | 59.6 | 41.7 | 34.1 | 17.4 | 6.05 | 15000 | 1200 |
| Iroko | 29.1 | 20.4 | 18.8 | 9.17 | 3.92 | 6800 | 540 |
| Jarrah | 31.8 | 22.3 | 19.5 | 11.5 | 4.11 | 8200 | 650 |
| Karri | 37.8 | 26.5 | 22.7 | 12.8 | 4.19 | 12000 | 970 |
| Keruing/gurjun | 34.6 | 24.2 | 21.2 | 7.19 | 3.10 | 11300 | 900 |
| Keruing (Sabah) | 33.0 | 23.1 | 19.4 | 6.79 | 3.07 | 9900 | 790 |
| European oak | 27.1 | 19.0 | 16.0 | 8.75 | 3.64 | 6800 | 530 |
| Opepe | 37.4 | 26.2 | 24.8 | 11.3 | 4.77 | 9600 | 760 |

KEY: See next page

(1) These stresses apply also to the Canadian species group Douglas fir - larch, see Table 2.

(2) These stresses apply also to the Canadian species group Hem - fir, see Table 2.

(3) If mixed redwood/whitewood is specified or supplied then the lowest stress values for the two species apply.

(4) These stresses apply also to thé Canadian species group spruce - pine - fir, see Table 2.

Table 6a

GRADE DESIGN STRESSES (N/mm$^2$) FOR THE DRY EXPOSURE CONDITION: BS 4978 GRADES

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Bending $\sigma_{bgd}$ | | Tension $\sigma_{tgd}$ | | Compression $\sigma_{cgd}$ | | Compression perpendicular to grain $\sigma_{c\perp gd}$ | | Shear $\tau_{gd}$ | Modulus of elasticity $E_{gk}$ | | Modulus of rigidity $G_{gk}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | SS | GS | SS | GS | SS | GS | SS | GS | SS and GS | SS | GS | SS and GS |
| <u>Imported</u> | | | | | | | | | | | | |
| Douglas fir (1) | 12.7 | 9.0 | 7.3 | 5.1 | 9.7 | 7.6 | 3.16 | 2.81 | 1.40 | 9600 | 8600 | 530 |
| Western hemlock (2) | 10.7 | 7.5 | 6.1 | 5.1 | 8.6 | 6.8 | 2.63 | 2.33 | 1.07 | 8500 | 7650 | 540 |
| Parana pine | 12.1 | 8.6 | 7.0 | 4.8 | 10.8 | 8.5 | 3.32 | 2.95 | 1.61 | 7850 | 7050 | 450 |
| Pitch pine | 13.8 | 9.7 | 7.9 | 5.5 | 10.5 | 8.2 | 3.72 | 3.31 | 1.57 | 9550 | 8600 | 420 |
| Redwood (3) | 11.0 | 7.8 | 6.3 | 4.4 | 8.6 | 6.8 | 2.96 | 2.63 | 1.37 | 6900 | 6200 | 430 |
| Whitewood (3) | 10.8 | 7.6 | 6.2 | 4.3 | 8.0 | 6.3 | 2.87 | 2.55 | 1.29 | 7550 | 6800 | 530 |
| Canadian spruce (4) | 9.4 | 6.6 | 5.4 | 3.7 | 7.7 | 6.1 | 2.27 | 2.01 | 1.05 | 6800 | 6100 | 480 |
| <u>Home Grown</u> | | | | | | | | | | | | |
| Douglas fir | 11.6 | 8.2 | 6.7 | 4.7 | 9.0 | 7.1 | 3.13 | 2.78 | 1.32 | 7050 | 6350 | 430 |
| Jap larch | 10.4 | 7.3 | 5.9 | 4.2 | 7.7 | 6.0 | 2.99 | 2.65 | 1.38 | 4600 | 4150 | 280 |
| European larch | 12.7 | 9.0 | 7.3 | 5.1 | 9.7 | 7.6 | 4.13 | 3.67 | 1.54 | 6800 | 6150 | 380 |
| Scots pine | 11.5 | 8.1 | 6.5 | 4.6 | 9.0 | 7.1 | 3.41 | 3.03 | 1.49 | 6700 | 6000 | 390 |
| European spruce | 9.4 | 6.7 | 5.4 | 3.8 | 7.1 | 5.6 | 2.54 | 2.25 | 1.21 | 6000 | 5400 | 450 |
| Sitka spruce | 7.7 | 5.4 | 4.4 | 3.1 | 6.3 | 5.0 | 2.06 | 1.83 | 0.97 | 5100 | 4600 | 390 |
| <u>Hardwoods</u> | | | | | | | | | | | | |
| Greenheart | 30.4 | 21.5 | 17.4 | 12.2 | 22.7 | 17.9 | 13.1 | 11.6 | 3.03 | 16000 | 14350 | 1220 |
| Iroko | 14.8 | 10.5 | 8.5 | 5.9 | 12.5 | 9.8 | 6.88 | 6.11 | 1.96 | 7250 | 6500 | 540 |
| Jarrah | 16.2 | 11.4 | 9.3 | 6.5 | 13.0 | 10.2 | 8.63 | 7.67 | 2.06 | 8700 | 7800 | 650 |
| Karri | 19.3 | 13.6 | 11.1 | 7.8 | 15.1 | 11.9 | 9.60 | 8.53 | 2.10 | 12800 | 11500 | 970 |
| Keruing/gurjun | 17.6 | 12.5 | 10.1 | 7.1 | 14.1 | 11.1 | 5.39 | 4.79 | 1.55 | 12000 | 10750 | 900 |
| Keruing (Sabah) | 16.8 | 11.9 | 9.7 | 6.8 | 12.9 | 10.2 | 5.09 | 4.52 | 1.54 | 10500 | 9500 | 790 |
| European oak | 13.8 | 9.8 | 7.9 | 5.6 | 10.7 | 8.4 | 6.56 | 5.83 | 1.82 | 7200 | 6500 | 530 |
| Opepe | 19.1 | 13.5 | 10.9 | 7.7 | 16.5 | 13.0 | 8.48 | 7.53 | 2.39 | 10200 | 9200 | 760 |

KEY: See next page

(1) These stresses apply also to the Canadian species group Douglas fir - larch, see Table 2

(2) These stresses apply also to the Canadian species group Hem - fir, see Table 2

(3) If mixed redwood/whitewood is specified or supplied then the lowest stress values for the two species apply

(4) These stresses apply also to the Canadian species group spruce - pine - fir, see Table 2.

Table 6b

**GRADE DESIGN STRESSES ($N/mm^2$) FOR THE DRY EXPOSURE CONDITION: BS4978 MACHINE STRESS GRADES**

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Bending $\sigma_{bgd}$ | | Tension $\sigma_{tgd}$ | | Compression $\sigma_{cgd}$ | | Compression perpendicular to grain $\sigma_{c\perp gd}$ | | Shear $\tau_{gd}$ | Modulus of elasticity $E_{gd}$ | | Modulus of rigidity $G_{gd}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | MSS | MGS | MSS | MGS | MSS | MGS | MSS | MGS | MSS and MGS | MSS | MGS | MSS and MGS |
| <u>Imported</u> | | | | | | | | | | | | |
| Western hemlock | 10.7 | 7.5 | 6.1 | 5.1 | 8.6 | 6.8 | 2.63 | 2.33 | 1.07 | 8550 | 7800 | 540 |
| Redwood | 10.8 | 7.6 | 6.2 | 4.3 | 8.0 | 6.3 | 2.96 | 2.63 | 1.37 | 7700 | 6950 | 430 |
| Whitewood | 10.8 | 7.6 | 6.2 | 4.3 | 8.0 | 6.3 | 2.87 | 2.55 | 1.29 | 7700 | 6950 | 530 |
| Canadian spruce | 9.4 | 6.6 | 5.4 | 3.7 | 7.7 | 6.1 | 2.27 | 2.01 | 1.05 | | | 480 |
| <u>Home Grown</u> | | | | | | | | | | | | |
| Douglas fir | 11.6 | 8.2 | 6.7 | 4.7 | 9.0 | 7.1 | 3.13 | 2.78 | 1.32 | 7100 | 6150 | 430 |
| Scots pine | 11.5 | 8.1 | 6.5 | 4.6 | 9.0 | 7.1 | 3.41 | 3.03 | 1.49 | 7100 | 6450 | 390 |
| Sitka spruce | 7.7 | 5.4 | 4.4 | 3.1 | 6.3 | 5.0 | 2.06 | 1.83 | 0.97 | 5350 | 4700 | 390 |

**NOTE: These stress values, particularly for modulus of elasticity and tension, may be modified on completion of the main Anglo/Scandinavian project to adjust from the 1 to 5 per cent exclusion condition.**

Table 7

Grade Design Stresses (N/mm$^2$) for the Dry Exposure Condition: NLGA 1970 Structural Joists and Plank Grades[1]

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Grade | Bending $\sigma_{bgd}$ | Tension $\sigma_{tgd}$ | Compression $\sigma_{cgd}$ | Compression perpendicular to grain $\sigma_{c\perp gd}$ | Shear $\tau_{gd}$ | Modulus of elasticity $E_{gd}$ | Modulus of rigidity $G_{gd}$ |
|---|---|---|---|---|---|---|---|---|
| Douglas fir - larch | Sel Str | 16.7 | 9.6 | 10.3 | 3.16 | 1.40 | 10600 | |
| | No 1 | 14.2 | 8.1 | 9.3 | 3.16 | 1.40 | 10000 | |
| | No 2 | 11.5 | 6.6 | 7.8 | 2.81 | 1.40 | 9250 | |
| | No 3 | 6.7 | 3.8 | 4.9 | 2.11 | 1.20 | 8200 | |
| Hem - fir | Sel Str | 14.0 | 8.0 | 9.6 | 2.63 | 1.07 | 9400 | |
| | No 1 | 11.9 | 6.8 | 8.6 | 2.63 | 1.07 | 8850 | |
| | No 2 | 9.6 | 5.6 | 7.3 | 2.33 | 1.07 | 8200 | |
| | No 3 | 5.6 | 3.2 | 4.6 | 1.75 | 0.92 | 7250 | |
| Spruce - pine - fir | Sel Str | 12.3 | 7.1 | 8.2 | 2.27 | 1.05 | 7500 | |
| | No 1 | 10.5 | 6.0 | 7.4 | 2.27 | 1.05 | 7100 | |
| | No 2 | 8.5 | 4.9 | 6.3 | 2.01 | 1.05 | 6550 | |
| | No 3 | 5.0 | 2.8 | 3.9 | 1.51 | 0.90 | 5800 | |

[1] These stresses apply to nominal sections 50 to 100 mm thick by 150 mm or greater width.

Table 8

GRADE DESIGN STRESSES ($N/mm^2$) FOR THE DRY EXPOSURE CONDITION: NLGA 1970 LIGHT FRAMING GRADES(1)

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Grade | Bending $\sigma_{bgd}$ | Tension $\sigma_{tgd}$ | Compression $\sigma_{cgd}$ | Compression perpendicular to grain $\sigma_{c\perp gd}$ | Shear $\tau_{gd}$ | Modulus of Elasticity $E_{gd}$ | Modulus of Rigidity $G_{gd}$ |
|---|---|---|---|---|---|---|---|---|
| Douglas fir - larch | Const | 10.0 | 4.8 | 8.3 | 3.16 | 1.40 | 8500 | |
| | Std | 5.5 | 2.8 | 6.8 | 2.81 | 1.40 | 7700 | |
| | Util | 2.5 | 1.3 | 4.5 | 2.11 | 1.20 | 7050 | |
| Hem - fir | Const | 8.4 | 4.0 | 7.6 | 2.63 | 1.07 | 7550 | |
| | Std | 4.6 | 2.3 | 6.3 | 2.33 | 1.07 | 6800 | |
| | Util | 2.1 | 1.1 | 4.2 | 1.75 | 0.92 | 6250 | |
| Spruce - pine - fir | Const | 7.4 | 3.5 | 6.6 | 2.27 | 1.05 | 6050 | |
| | Std | 4.0 | 2.0 | 5.5 | 2.01 | 1.05 | 5450 | |
| | Util | 1.8 | 1.0 | 3.6 | 1.51 | 0.90 | 5000 | |

(1) These stresses apply to nominal sections 50 to 100 mm thick by 100 mm width. For 50 mm and 75 mm widths the following grade design stresses should be multiplied by the factors:

| | | | | |
|---|---|---|---|---|
| Bending | 50 mm width: | Const 0.9 | Std 0.7 | Util 0.5 |
| Bending | 75 mm width: | Const 0.8 | Std 0.7 | Util 0.5 |
| Tension | 50 mm width: | Const 0.8 | Std 0.7 | Util 0.5 |
| Tension | 75 mm width: | Const 0.8 | Std 0.7 | Util 0.5 |
| Compression | 50 mm width: | Const 1.0 | Std 1.0 | Util 0.6 |
| Compression | 75 mm width: | Const 1.0 | Std 1.0 | Util 0.7 |

for all other stresses the factor is 1.0

Table 9

GRADE DESIGN STRESSES (N/mm$^2$) FOR THE DRY EXPOSURE CONDITION: NLGA (1970) STUD GRADE[1]

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Commercial Name | Bending $\sigma_{bgd}$ | Tension $\sigma_{tgd}$ | Compression $\sigma_{cgd}$ | Compression perpendicular to grain $\sigma_{c\perp gd}$ | Shear $\tau_{gd}$ | Modulus of elasticity $E_{gd}$ | Modulus of rigidity $G_{gd}$ |
|---|---|---|---|---|---|---|---|
| Douglas fir - larch | 7.5 | 3.8 | 4.5 | 2.11 | 1.20 | 8200 | |
| Hem - fir | 6.3 | 3.2 | 4.2 | 1.75 | 0.92 | 7250 | |
| Spruce - pine - fir | 5.5 | 2.8 | 3.6 | 1.51 | 0.90 | 5800 | |

(1) These stresses apply to nominal sections 50 mm thick by 100 mm wide. For other sections up to 100 mm square the following grade design stresses should be multiplied by the factors:

| | |
|---|---|
| Bending | 50 x 50 mm - 1.1 |
| Bending | 50 x 75 and 75 x 75 mm - 1.0 |
| Bending | 75 x 100 and 100 x 100 mm - 0.35 |
| Tension | 50 x 50, 50 x 75 and 75 x 75 mm - 1.0 |
| Tension | 75 x 100 and 100 x 100 mm - 0.35 |
| Compression | 50 x 50, 50 x 75 and 75 x 75 mm - 1.5 |
| Compression | 75 x 100 and 100 x 100 mm - 1.0 |

for all other stresses the factor is 1.0

Table 10

STRENGTH CLASSES AND GRADE DESIGN STRESSES ($N/mm^2$) FOR THE DRY EXPOSURE CONDITION

These stresses apply to timber having a moisture content not exceeding 18 per cent

| Strength Class | Bending $\sigma_{bgd}$ | Tension $\sigma_{tgd}$ | Compression $\sigma_{cgd}$ | Compression perpendicular to grain(1) $\sigma_{c\perp gd}$ | Shear(2) $\tau_{gd}$ | Modulus of Elasticity $E_{gd}$ | Modulus of Rigidity $G_{gd}$ |
|---|---|---|---|---|---|---|---|
| E15 | 18.0 | 10.0 | 15.0 | 5.00 | 2.50 | 15000 | |
| E11 | 14.0 | 8.0 | 11.8 | 4.00 | 2.00 | 11800 | |
| E9 | 11.2 | 6.3 | 9.5 | 3.15 | 1.60 | 9500 | |
| E7 | 9.0 | 5.0 | 7.5 | 2.50 | 1.25 | 7500 | |
| E6 | 7.1 | 4.0 | 6.0 | 2.00 | 1.00 | 6000 | |
| E5 | 5.6 | 3.1 | 4.7 | 1.60 | 0.80 | 4750 | |

(1) These stresses allow for wane not exceeding the limit for the SS grade specified in BS 4978:1973

(2) These stresses allow for fissures not exceeding the limits for SS grade specified in BS 4978:1973

5.6.1 Duration of load. The grade design stresses are applicable to long term loading. A member can sustain a greater load for a period of a few minutes than it can for a period of several years.

Table 11 gives the modification factor $K_2$ by which the grade design stresses should be multiplied for various durations of loading. When advantage is taken of this clause the design should be checked to ensure that the resulting stresses are not exceeded for any of the relevant loading conditions.

The factor $K_2$ may be applied to all grade design stresses except those for modulus of elasticity and modulus of rigidity.

Table 11 MODIFICATION FACTOR $K_2$ FOR DURATION OF LOADING. (applicable to all strength properties except modulus of elasticity and modulus of rigidity)

| Duration of loading | Value of $K_2$ |
|---|---|
| Long term (eg dead + permanent imposed) | 1.00 |
| Medium term (eg dead + snow) | 1.25 |
| Short term (eg dead + imposed + wind) | 1.5 |
| Very short term (eg dead + imposed + wind, 3 sec or 5 sec gusts) | 1.75 |

5.6.2 Load Sharing. Where two or more pieces of timber act together in such a fashion as to be equally strained under load, or where it can be shown that effective lateral distribution of loading occurs the grade design stresses for properties other than modulus of elasticity and modulus of rigidity should be multiplied by the factor $K_3$ given in Table 12. For example the value of $K_3$ for 4 or more pieces should be taken as applying to the grade design stresses except for modulus of elasticity and modulus of rigidity, for rafters, floor joists and wall studding spaced not further apart than 600 mm and joined by purlins, binders boarding etc, so that lateral distribution of loading occurs. For modulus of elasticity and modulus of rigidity the factors $K_1$ Table 4 should be used (see clause 4.2.2).

Table 12

MODIFICATION FACTOR $K_3$ APPLICABLE TO GRADE DESIGN STRESSES (EXCEPT FOR MODULUS OF ELASTICITY AND MODULUS OF RIGIDITY) FOR LOAD SHARING CONDITIONS

| No of members | Value of $K_3$ |
|---|---|
| 2 | 1.1 |
| 3 | 1.15 |
| 4 or more | 1.2 |

5.6.3 Exposure Condition. The grade design stresses are applicable to the dry exposure condition. For the wet exposure condition they should be multiplied by the factor $K_4$ given in Table 13.

Table 13

MODIFICATION FACTOR $K_4$ BY WHICH THE GRADE DESIGN STRESSES SHOULD BE MULTIPLIED FOR THE WET EXPOSURE CONDITION

| Property | | Value of $K_4$ |
|---|---|---|
| Bending | ($\sigma_b$) | 0.70 |
| Tension | ($\sigma_t$) | 0.70 |
| Compression | ($\sigma_c$) | 0.67 |
| Compression perpendicular to grain | ($\sigma_{c\perp}$) | 0.65 |
| Shear | ($\tau$) | 0.80 |
| Modulus of Elasticity | (E) | 0.85 |
| Modulus of Rigidity | (G) | 0.85 |

5.7 Geometrical Properties

The actual dimensions of any piece of timber, compared with its described basic dimensions, will vary with its moisture content at the time of measurement, the tolerances permitted by the manufacturing standard, and according to whether it has been sawn or subsequently processed.

Design calculations should be based on the cross-section of a member as existing under service conditions, or on the recommended geometrical properties for the appropriate exposure condition. Due allowance should be made for all notching, sinking, drilling etc which may be present in a member.

The geometrical properties for rectangular sections described in the following clauses apply to the dry exposure condition and for the wet exposure condition should be multiplied by the factors given in Clause 3.4.

5.7.1 Timber to BS 4471. This clause relates to the three types of timber conforming to BS 4471 viz sawn timber, precision timber and processed timber. The geometrical properties for sawn and precision timber are given in Table 14 and for processed timber in Table 15. The basic sizes quoted are to be measured at 20 per cent moisture content, but the geometrical properties apply directly to the dry exposure condition and have been modified to allow for the difference in moisture content.

For timber re-sawn to BS 4471 the geometrical properties should be calculated, the values given in Table 14 do not apply.

5.7.2 Timber to Canadian Lumber Standards. Timber from Canada may be available either surfaced one side, one edge or planed. The surfaced one side one edge timber will be of 44 mm basic thickness for which the appropriate geometrical properties are those listed on Table 15 for this thickness. The planed timber will be produced to Canadian Lumber Standards and the geometrical properties for this material are given in Table 16. The basic sizes quoted are to be measured at 20 per cent moisture content, but the geometrical properties apply directly to the dry exposure condition and have been modified to allow for the difference in moisture content.

Table 14

GEOMETRICAL PROPERTIES FOR THE DRY EXPOSURE CONDITION OF SAWN AND PRECISION TIMBER CONFORMING TO THE REQUIREMENTS OF BS 4471:1969

| Basic sawn size(1) | Area | Section modulus | | Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|
| | | about x-x | about y-y | about x-x | about y-y | about x-x | about y-y |
| mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 16 x 75 | 1.19 | 14.8 | 3.16 | 0.553 | 0.0252 | 21.6 | 4.60 |
| 16 x 100 | 1.59 | 26.3 | 4.21 | 1.31 | 0.0336 | 28.7 | 4.60 |
| 16 x 125 | 1.98 | 41.2 | 5.27 | 2.56 | 0.0420 | 35.9 | 4.60 |
| 16 x 150 | 2.38 | 59.3 | 6.32 | 4.43 | 0.0504 | 43.1 | 4.60 |
| 19 x 75 | 1.41 | 17.6 | 4.46 | 0.657 | 0.0422 | 21.6 | 5.46 |
| 19 x 100 | 1.88 | 31.3 | 5.94 | 1.56 | 0.0562 | 28.7 | 5.46 |
| 19 x 125 | 2.36 | 48.9 | 7.43 | 3.04 | 0.0703 | 35.9 | 5.46 |
| 19 x 150 | 2.83 | 70.3 | 8.92 | 5.26 | 0.0844 | 43.1 | 5.46 |
| 22 x 75 | 1.64 | 20.4 | 5.98 | 0.761 | 0.0655 | 21.6 | 6.32 |
| 22 x 100 | 2.18 | 36.2 | 7.97 | 1.80 | 0.0873 | 28.7 | 6.32 |
| 22 x 125 | 2.73 | 56.6 | 9.96 | 3.52 | 0.109 | 35.9 | 6.32 |
| 22 x 150 | 3.27 | 81.5 | 11.9 | 6.09 | 0.131 | 43.1 | 6.32 |
| 25 x 75 | 1.86 | 23.1 | 7.72 | 0.865 | 0.0961 | 21.6 | 7.19 |
| 25 x 100 | 2.48 | 41.2 | 10.3 | 2.05 | 0.128 | 28.7 | 7.19 |
| 25 x 125 | 3.10 | 64.3 | 12.9 | 4.00 | 0.160 | 35.9 | 7.19 |
| 25 x 150 | 3.72 | 92.6 | 15.4 | 6.93 | 0.192 | 43.1 | 7.19 |
| 25 x 175 | 4.34 | 126 | 18.0 | 11.0 | 0.224 | 50.3 | 7.19 |
| 25 x 200 | 4.96 | 165 | 20.6 | 16.4 | 0.256 | 57.5 | 7.19 |
| 25 x 225 | 5.58 | 208 | 23.1 | 23.3 | 0.288 | 64.7 | 7.19 |
| 25 x 250 | 6.20 | 257 | 25.7 | 32.0 | 0.320 | 71.9 | 7.19 |
| 25 x 300 | 7.44 | 370 | 30.9 | 55.3 | 0.384 | 86.2 | 7.19 |
| 32 x 75 | 2.38 | 29.6 | 12.6 | 1.11 | 0.201 | 21.6 | 9.20 |
| 32 x 100 | 3.17 | 52.7 | 16.9 | 2.62 | 0.268 | 28.7 | 9.20 |
| 32 x 125 | 3.97 | 82.3 | 21.1 | 5.12 | 0.336 | 35.9 | 9.20 |
| 32 x 150 | 4.76 | 118 | 25.3 | 8.86 | 0.403 | 43.1 | 9.20 |
| 32 x 175 | 5.55 | 161 | 29.5 | 14.1 | 0.470 | 50.3 | 9.20 |
| 32 x 200 | 6.35 | 211 | 33.7 | 21.0 | 0.537 | 57.5 | 9.20 |
| 32 x 225 | 7.14 | 267 | 37.9 | 29.9 | 0.604 | 64.7 | 9.20 |
| 32 x 250 | 7.94 | 329 | 42.1 | 41.0 | 0.672 | 71.9 | 9.20 |
| 32 x 300 | 9.52 | 474 | 50.6 | 70.8 | 0.806 | 86.2 | 9.20 |
| 36 x 75 | 2.68 | 33.3 | 16.0 | 1.24 | 0.287 | 21.6 | 10.3 |
| 36 x 100 | 3.57 | 59.3 | 21.3 | 2.95 | 0.382 | 28.7 | 10.3 |
| 36 x 125 | 4.46 | 92.6 | 26.7 | 5.76 | 0.478 | 35.9 | 10.3 |
| 36 x 150 | 5.36 | 133 | 32.0 | 9.96 | 0.574 | 43.1 | 10.3 |

(1) Basic sawn sizes as at 20 per cent moisture content in accordance with BS 4471:1969

Table 14 continued

| Basic sawn size (1) | Area | Section modulus | | Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|
| | | about x-x | about y-y | about x-x | about y-y | about x-x | about y-y |
| mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 38 x 75 | 2.83 | 35.2 | 17.8 | 1.31 | 0.337 | 21.6 | 10.9 |
| 38 x 100 | 3.77 | 62.6 | 23.8 | 3.11 | 0.450 | 28.7 | 10.9 |
| 38 x 125 | 4.71 | 97.8 | 29.7 | 6.08 | 0.562 | 35.9 | 10.9 |
| 38 x 150 | 5.65 | 141 | 35.7 | 10.5 | 0.675 | 43.1 | 10.9 |
| 38 x 175 | 6.60 | 192 | 41.6 | 16.7 | 0.787 | 50.3 | 10.9 |
| 38 x 200 | 7.54 | 250 | 47.5 | 24.9 | 0.900 | 57.5 | 10.9 |
| 38 x 225 | 8.48 | 317 | 53.5 | 35.5 | 1.01 | 64.7 | 10.9 |
| 40 x 75 | 2.98 | 37.0 | 19.8 | 1.38 | 0.394 | 21.6 | 11.5 |
| 40 x 100 | 3.97 | 65.9 | 26.3 | 3.28 | 0.525 | 28.7 | 11.5 |
| 40 x 125 | 4.96 | 103 | 32.9 | 6.41 | 9.656 | 35.9 | 11.5 |
| 40 x 150 | 5.95 | 148 | 39.5 | 11.1 | 0.787 | 43.1 | 11.5 |
| 40 x 175 | 6.94 | 202 | 46.1 | 17.6 | 0.918 | 50.3 | 11.5 |
| 40 x 200 | 7.97 | 263 | 52.7 | 26.2 | 1.05 | 57.5 | 11.5 |
| 40 x 225 | 8.93 | 333 | 59.3 | 37.4 | 1.18 | 64.7 | 11.5 |
| 44 x 75 | 3.27 | 40.7 | 23.9 | 1.52 | 0.524 | 21.6 | 12.6 |
| 44 x 100 | 4.36 | 72.4 | 31.9 | 3.61 | 0.698 | 28.7 | 12.6 |
| 44 x 125 | 5.46 | 113 | 39.8 | 7.05 | 0.873 | 35.9 | 12.6 |
| 44 x 150 | 6.55 | 163 | 47.8 | 12.2 | 1.05 | 43.1 | 12.6 |
| 44 x 175 | 7.64 | 222 | 55.8 | 19.3 | 1.22 | 50.3 | 12.6 |
| 44 x 200 | 8.73 | 290 | 63.7 | 28.9 | 1.40 | 57.5 | 12.6 |
| 44 x 225 | 9.82 | 367 | 71.7 | 41.1 | 1.57 | 64.7 | 12.6 |
| 44 x 250 | 10.9 | 453 | 79.7 | 56.4 | 1.75 | 71.9 | 12.6 |
| 44 x 300 | 13.1 | 652 | 95.6 | 97.4 | 2.09 | 86.2 | 12.6 |
| 50 x 75 | 3.72 | 46.3 | 30.9 | 1.73 | 0.769 | 21.6 | 14.4 |
| 50 x 100 | 4.96 | 82.3 | 41.2 | 4.10 | 1.02 | 28.7 | 14.4 |
| 50 x 125 | 6.20 | 129 | 51.4 | 8.01 | 1.28 | 35.9 | 14.4 |
| 50 x 150 | 7.44 | 185 | 61.7 | 13.8 | 1.54 | 43.1 | 14.4 |
| 50 x 175 | 8.68 | 252 | 72.0 | 22.0 | 1.79 | 50.3 | 14.4 |
| 50 x 200 | 9.92 | 329 | 82.3 | 32.8 | 2.05 | 57.5 | 14.4 |
| 50 x 225 | 11.2 | 417 | 92.6 | 46.7 | 2.31 | 64.7 | 14.4 |
| 50 x 250 | 12.4 | 514 | 103 | 64.1 | 2.56 | 71.9 | 14.4 |
| 50 x 300 | 14.9 | 741 | 123 | 111 | 3.07 | 86.2 | 14.4 |
| 63 x 100 | 6.25 | 104 | 65.3 | 5.17 | 2.05 | 28.7 | 18.1 |
| 63 x 125 | 7.81 | 162 | 81.7 | 10.1 | 2.56 | 35.9 | 18.1 |
| 63 x 150 | 9.37 | 233 | 98.0 | 17.4 | 3.07 | 43.1 | 18.1 |
| 63 x 175 | 10.9 | 318 | 114 | 27.7 | 3.59 | 50.3 | 18.1 |
| 63 x 200 | 12.5 | 415 | 131 | 41.3 | 4.10 | 57.5 | 18.1 |
| 63 x 225 | 14.1 | 525 | 147 | 58.8 | 4.61 | 64.7 | 18.1 |

(1) Basic sawn sizes as at 20 per cent moisture content in accordance with BS 4471:1969

Table 14 continued

| Basic sawn size[1] | Area | Section modulus | | Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|
| | | about x-x | about y-y | about x-x | about y-y | about x-x | about y-y |
| mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 75 x 100 | 7.44 | 123 | 92.6 | 6.15 | 3.46 | 28.7 | 21.6 |
| 75 x 125 | 9.30 | 193 | 116 | 12.0 | 4.32 | 35.9 | 21.6 |
| 75 x 150 | 11.2 | 278 | 139 | 20.7 | 5.19 | 43.1 | 21.6 |
| 75 x 175 | 13.0 | 378 | 162 | 33.0 | 6.05 | 50.3 | 21.6 |
| 75 x 200 | 14.9 | 494 | 185 | 49.2 | 6.92 | 57.5 | 21.6 |
| 75 x 225 | 16.7 | 625 | 208 | 70.0 | 7.78 | 64.7 | 21.6 |
| 75 x 250 | 18.6 | 772 | 231 | 96.1 | 8.65 | 71.9 | 21.6 |
| 75 x 300 | 22.3 | 1111 | 278 | 166 | 10.4 | 86.2 | 21.6 |
| 100 x 100 | 9.92 | 165 | 165 | 8.20 | 8.20 | 28.7 | 28.7 |
| 100 x 150 | 14.9 | 370 | 247 | 27.7 | 12.3 | 43.1 | 28.7 |
| 100 x 200 | 19.8 | 659 | 329 | 65.6 | 16.4 | 57.5 | 28.7 |
| 100 x 250 | 24.8 | 1029 | 412 | 128 | 20.5 | 71.9 | 28.7 |
| 100 x 300 | 29.8 | 1482 | 494 | 221 | 24.6 | 86.2 | 28.7 |
| 150 x 150 | 22.3 | 556 | 556 | 41.5 | 41.5 | 43.1 | 43.1 |
| 150 x 200 | 29.8 | 988 | 741 | 98.4 | 55.3 | 57.5 | 43.1 |
| 150 x 300 | 44.6 | 2223 | 1111 | 332 | 83.0 | 86.2 | 43.1 |
| 200 x 200 | 39.7 | 1317 | 1317 | 131 | 131 | 57.5 | 57.5 |
| 250 x 250 | 62.0 | 2573 | 2573 | 320 | 320 | 71.9 | 71.9 |
| 300 x 300 | 89.3 | 4446 | 4446 | 664 | 664 | 86.2 | 86.2 |

(1) Basic sawn sizes as at 20 per cent moisture content in accordance with BS 4471:1969

## Table 15

GEOMETRICAL PROPERTIES FOR THE DRY EXPOSURE CONDITION OF PROCESSED TIMBER CONFORMING TO THE REQUIREMENTS OF BS4471:1969 FOR CONSTRUCTIONAL TIMBER SURFACED

| Basic sawn size [1] | Processed size [1] | Area | Section modulus | | ~~Section~~ Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|---|
| | | | About x-x | About y-y | About x-x | About y-y | About x-x | About y-y |
| mm | mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 16 x 75 | 13 x 72 | 0.928 | 11.1 | 2.00 | 0.398 | 0.0130 | 20.7 | 3.74 |
| 16 x 100 | 13 x 97 | 1.25 | 20.1 | 2.70 | 0.973 | 0.0175 | 27.9 | 3.74 |
| 16 x 125 | 13 x 120 | 1.55 | 30.8 | 3.34 | 1.84 | 0.0216 | 34.5 | 3.74 |
| 16 x 150 | 13 x 145 | 1.87 | 45.0 | 4.03 | 3.25 | 0.0261 | 41.7 | 3.74 |
| 19 x 75 | 16 x 72 | 1.14 | 13.6 | 3.03 | 0.490 | 0.0242 | 20.7 | 4.60 |
| 19 x 100 | 16 x 97 | 1.54 | 24.8 | 4.09 | 1.20 | 0.0326 | 27.9 | 4.60 |
| 19 x 125 | 16 x 120 | 1.90 | 37.9 | 5.06 | 2.27 | 0.0403 | 34.5 | 4.60 |
| 19 x 150 | 16 x 145 | 2.30 | 55.4 | 6.11 | 4.00 | 0.0487 | 41.7 | 4.60 |
| 22 x 75 | 19 x 72 | 1.36 | 16.2 | 4.28 | 0.581 | 0.0405 | 20.7 | 5.46 |
| 22 x 100 | 19 x 97 | 1.83 | 29.4 | 5.77 | 1.42 | 0.0545 | 27.9 | 5.46 |
| 22 x 125 | 19 x 120 | 2.26 | 45.0 | 7.13 | 2.69 | 0.0675 | 34.5 | 5.46 |
| 22 x 150 | 19 x 145 | 2.73 | 65.8 | 8.62 | 4.75 | 0.0815 | 41.7 | 5.46 |
| 25 x 75 | 22 x 72 | 1.57 | 18.8 | 5.74 | 0.673 | 0.0629 | 20.7 | 6.32 |
| 25 x 100 | 22 x 97 | 2.12 | 34.1 | 7.73 | 1.65 | 0.0847 | 27.9 | 6.32 |
| 25 x 125 | 22 x 120 | 2.62 | 52.2 | 9.56 | 3.12 | 0.105 | 34.5 | 6.32 |
| 25 x 150 | 22 x 145 | 3.16 | 76.2 | 11.5 | 5.50 | 0.127 | 41.7 | 6.32 |
| 25 x 175 | 22 x 169 | 3.69 | 103 | 13.5 | 8.71 | 0.147 | 48.6 | 6.32 |
| 25 x 200 | 22 x 194 | 4.23 | 136 | 15.5 | 13.2 | 0.169 | 55.8 | 6.32 |
| 25 x 225 | 22 x 219 | 4.78 | 174 | 17.4 | 18.9 | 0.191 | 63.0 | 6.32 |
| 25 x 250 | 22 x 244 | 5.32 | 216 | 19.4 | 26.2 | 0.213 | 70.1 | 6.32 |
| 25 x 300 | 22 x 294 | 6.42 | 313 | 23.4 | 45.8 | 0.257 | 84.5 | 6.32 |
| 32 x 75 | 29 x 72 | 2.07 | 24.7 | 9.97 | 0.887 | 0.144 | 20.7 | 8.34 |
| 32 x 100 | 29 x 97 | 2.79 | 44.9 | 13.4 | 2.17 | 0.194 | 27.9 | 8.34 |
| 32 x 125 | 29 x 120 | 3.45 | 68.8 | 16.6 | 4.11 | 0.240 | 34.5 | 8.34 |
| 32 x 150 | 29 x 145 | 4.17 | 100 | 20.1 | 7.25 | 0.290 | 41.7 | 8.34 |
| 32 x 175 | 29 x 169 | 4.86 | 136 | 23.4 | 11.5 | 0.338 | 48.6 | 8.34 |
| 32 x 200 | 29 x 194 | 5.58 | 180 | 26.9 | 17.4 | 0.388 | 55.8 | 8.34 |
| 32 x 225 | 29 x 219 | 6.30 | 229 | 30.3 | 25.0 | 0.438 | 63.0 | 8.34 |
| 32 x 250 | 29 x 244 | 7.02 | 284 | 33.8 | 34.5 | 0.488 | 70.1 | 8.34 |
| 32 x 300 | 29 x 294 | 8.46 | 413 | 40.7 | 60.4 | 0.588 | 84.5 | 8.34 |
| 36 x 75 | 33 x 72 | 2.36 | 28.2 | 12.9 | 1.01 | 0.212 | 20.7 | 9.49 |
| 36 x 100 | 33 x 97 | 3.17 | 51.1 | 17.4 | 2.47 | 0.286 | 27.9 | 9.49 |
| 36 x 125 | 33 x 120 | 3.93 | 78.2 | 21.5 | 4.67 | 0.354 | 34.5 | 9.49 |
| 36 x 150 | 33 x 145 | 4.75 | 114 | 26.0 | 8.25 | 0.427 | 41.7 | 9.49 |

[1] Basic sizes as at 20 per cent moisture content in accordance with BS 4471:1969

[2] Min sizes as at 20 per cent moisture content and including allowance for processing constructional timber in accordance with BS 4471:1969.

Table 15 continued

| Basic sawn size (1) | Processed size (1) | Area | Section modulus | | Sec~~tion~~ond moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|---|
| | | | About x-x | About y-y | About x-x | About y-y | About x-x | About y-y |
| mm | mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 38 x 75 | 35 x 72 | 2.50 | 29.9 | 14.5 | 1.07 | 0.253 | 20.7 | 10.1 |
| 38 x 100 | 35 x 97 | 3.37 | 54.2 | 19.6 | 2.62 | 0.341 | 27.9 | 10.1 |
| 38 x 125 | 35 x 120 | 4.17 | 83.0 | 24.2 | 4.96 | 0.422 | 34.5 | 10.1 |
| 38 x 150 | 35 x 145 | 5.03 | 121 | 29.2 | 8.75 | 0.510 | 41.7 | 10.1 |
| 38 x 175 | 35 x 169 | 5.87 | 165 | 34.1 | 13.8 | 0.594 | 48.6 | 10.1 |
| 38 x 200 | 35 x 194 | 6.73 | 217 | 39.1 | 20.9 | 0.682 | 55.8 | 10.1 |
| 38 x 225 | 35 x 219 | 7.60 | 276 | 44.2 | 30.1 | 0.770 | 63.0 | 10.1 |
| 40 x 75 | 37 x 72 | 2.64 | 31.6 | 16.2 | 1.13 | 0.299 | 20.7 | 10.6 |
| 40 x 100 | 37 x 97 | 3.56 | 57.3 | 21.9 | 2.77 | 0.403 | 27.9 | 10.6 |
| 40 x 125 | 27 x 120 | 4.40 | 87.7 | 27.0 | 5.24 | 0.498 | 34.5 | 10.6 |
| 40 x 150 | 37 x 145 | 5.32 | 128 | 32.7 | 9.25 | 0.602 | 41.7 | 10.6 |
| 40 x 175 | 37 x 169 | 6.20 | 174 | 38.1 | 14.6 | 0.702 | 48.6 | 10.6 |
| 40 x 200 | 37 x 194 | 7.12 | 229 | 43.7 | 22.1 | 0.806 | 55.8 | 10.6 |
| 40 x 225 | 37 x 219 | 8.04 | 292 | 49.4 | 31.9 | 0.910 | 63.0 | 10.6 |
| 44 x 75 | 41 x 72 | 2.93 | 35.0 | 19.9 | 1.25 | 0.407 | 20.7 | 11.8 |
| 44 x 100 | 41 x 97 | 3.94 | 63.5 | 26.8 | 3.07 | 0.548 | 27.9 | 11.8 |
| 44 x 125 | 41 x 120 | 4.88 | 97.2 | 33.2 | 5.81 | 0.678 | 34.5 | 11.8 |
| 44 x 150 | 41 x 145 | 5.90 | 142 | 40.1 | 10.2 | 0.819 | 41.7 | 11.8 |
| 44 x 175 | 41 x 169 | 6.87 | 193 | 46.8 | 16.2 | 0.955 | 48.6 | 11.8 |
| 44 x 200 | 41 x 194 | 7.89 | 254 | 53.7 | 24.5 | 1.10 | 55.8 | 11.8 |
| 44 x 225 | 41 x 219 | 8.91 | 323 | 60.6 | 35.3 | 1.24 | 63.0 | 11.8 |
| 44 x 250 | 41 x 244 | 9.92 | 402 | 67.5 | 48.8 | 1.38 | 70.1 | 11.8 |
| 44 x 300 | 41 x 294 | 11.9 | 583 | 81.4 | 85.4 | 1.66 | 84.5 | 11.8 |
| 50 x 75 | 47 x 72 | 3.36 | 40.1 | 26.2 | 1.44 | 0.613 | 20.7 | 13.5 |
| 50 x 100 | 47 x 97 | 4.52 | 72.8 | 35.3 | 3.52 | 0.826 | 27.9 | 13.5 |
| 50 x 125 | 47 x 120 | 5.59 | 111 | 43.6 | 6.66 | 1.02 | 34.5 | 13.5 |
| 50 x 150 | 47 x 145 | 6.76 | 163 | 52.7 | 11.7 | 1.23 | 41.7 | 13.5 |
| 50 x 175 | 47 x 169 | 7.87 | 221 | 61.5 | 18.6 | 1.44 | 48.6 | 13.5 |
| 50 x 200 | 47 x 194 | 9.04 | 291 | 70.6 | 28.1 | 1.65 | 55.8 | 13.5 |
| 50 x 225 | 47 x 219 | 10.2 | 371 | 79.7 | 40.5 | 1.86 | 63.0 | 13.5 |
| 50 x 250 | 47 x 244 | 11.4 | 461 | 88.7 | 56.0 | 2.08 | 70.1 | 13.5 |
| 50 x 300 | 47 x 294 | 13.7 | 669 | 107 | 97.9 | 2.50 | 84.5 | 13.5 |

(1) Basic sizes as at 20 per cent moisture content in accordance with BS 4471:1969

(2) Min sizes as at 20 per cent moisture content and including allowance for processing constructional timber in accordance with BS 4471:1969.

Table 15 continued

| Basic sawn size(1) | Processed size(1) | Area | Section modulus | | Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|---|
| | | | About x-x | About y-y | About x-x | About y-y | About x-x | About y-y |
| mm | mm | $10^3 mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 63 x 100 | 60 x 97 | 5.77 | 93.0 | 57.5 | 4.49 | 1.72 | 27.9 | 17.2 |
| 63 x 125 | 60 x 120 | 7.14 | 142 | 71.1 | 8.50 | 2.12 | 34.5 | 17.2 |
| 63 x 150 | 60 x 145 | 8.63 | 208 | 85.9 | 15.0 | 2.57 | 41.7 | 17.2 |
| 63 x 175 | 60 x 169 | 10.0 | 282 | 100 | 23.7 | 2.99 | 48.6 | 17.2 |
| 63 x 200 | 60 x 194 | 11.5 | 372 | 115 | 35.9 | 3.44 | 55.8 | 17.2 |
| 63 x 225 | 60 x 219 | 13.0 | 474 | 130 | 51.7 | 3.88 | 63.0 | 17.2 |
| 75 x 100 | 72 x 97 | 6.93 | 111 | 82.8 | 5.39 | 2.97 | 27.9 | 20.7 |
| 75 x 125 | 72 x 120 | 8.57 | 171 | 102 | 10.2 | 3.67 | 34.5 | 20.7 |
| 75 x 150 | 72 x 145 | 10.3 | 249 | 124 | 18.0 | 4.44 | 41.7 | 20.7 |
| 75 x 175 | 72 x 169 | 12.1 | 339 | 144 | 28.5 | 5.17 | 48.6 | 20.7 |
| 75 x 200 | 72 x 194 | 13.8 | 446 | 166 | 43.1 | 5.94 | 55.8 | 20.7 |
| 75 x 225 | 72 x 219 | 15.6 | 569 | 187 | 62.0 | 6.70 | 63.0 | 20.7 |
| 75 x 250 | 72 x 244 | 17.4 | 706 | 208 | 85.8 | 7.47 | 70.1 | 20.7 |
| 75 x 300 | 72 x 294 | 21.0 | 1025 | 251 | 150 | 9.00 | 84.5 | 20.7 |
| 100 x 100 | 97 x 97 | 9.33 | 150 | 150 | 7.26 | 7.26 | 27.9 | 27.9 |
| 100 x 150 | 97 x 145 | 13.9 | 336 | 225 | 24.2 | 10.8 | 41.7 | 27.9 |
| 100 x 200 | 97 x 194 | 18.7 | 601 | 300 | 58.1 | 14.5 | 55.8 | 27.9 |
| 100 x 250 | 97 x 244 | 23.5 | 951 | 378 | 115 | 18.3 | 70.1 | 27.9 |
| 100 x 300 | 97 x 294 | 28.3 | 1381 | 455 | 202 | 22.0 | 84.5 | 27.9 |
| 150 x 150 | 145 x 145 | 20.8 | 502 | 502 | 36.2 | 36.2 | 41.7 | 41.7 |
| 150 x 200 | 145 x 194 | 27.9 | 899 | 672 | 86.8 | 48.5 | 55.8 | 41.7 |
| 150 x 300 | 145 x 294 | 42.3 | 2064 | 1018 | 302 | 73.5 | 84.5 | 41.7 |
| 200 x 200 | 194 x 194 | 37.3 | 1202 | 1202 | 116 | 116 | 55.8 | 55.8 |
| 250 x 250 | 244 x 244 | 59.0 | 2392 | 2392 | 290.6 | 290.6 | 70.1 | 70.1 |
| 300 x 300 | 294 x 294 | 85.7 | 4184 | 4184 | 613 | 613 | 84.5 | 84.5 |

(1) Basic sizes as at 20 per cent moisture content in accordance with BS 4471:1969

(2) Min sizes as at 20 per cent moisture content and including allowance for processing constructional timber in accordance with BS 4471:1969.

Table 16

GEOMETRICAL PROPERTIES FOR THE DRY EXPOSURE CONDITION OF TIMBER CONFORMING TO THE REQUIREMENTS OF CANADIAN LUMBER STANDARDS

| Basic size(1) | Min size(2) | Area | Section modulus | | Second moment of area | | Radius of gyration | |
|---|---|---|---|---|---|---|---|---|
| | | | About x-x | About y-y | About x-x | About y-y | About x-x | About y-y |
| ~~mm~~ in. | mm | $10^3$ $mm^2$ | $10^3$ $mm^3$ | $10^3$ $mm^3$ | $10^6$ $mm^4$ | $10^6$ $mm^4$ | mm | mm |
| 2 x 2 | 38.1 x 38.1 | 1.44 | 9.11 | 9.11 | 0.173 | 0.173 | 10.9 | 10.9 |
| 2 x 3 | 38.1 x 63.7 | 2.41 | 25.4 | 15.2 | 0.807 | 0.289 | 18.3 | 10.9 |
| 2 x 4 | 38.1 x 89.2 | 3.37 | 49.9 | 21.3 | 2.22 | 0.404 | 25.6 | 10.9 |
| 2 x 6 | 38.1 x 140.3 | 5.30 | 123 | 33.5 | 8.63 | 0.636 | 40.3 | 10.9 |
| 2 x 8 | 38.1 x 184.9 | 6.99 | 214 | 44.2 | 19.7 | 0.838 | 53.2 | 10.9 |
| 2 x 10 | 38.1 x 235.9 | 8.91 | 349 | 56.4 | 41.0 | 1.07 | 67.8 | 10.9 |
| 2 x 12 | 38.1 x 286.5 | 10.8 | 515 | 68.5 | 73.5 | 1.30 | 82.4 | 10.9 |
| 3 x 4 | 63.7 x 89.2 | 5.64 | 83.4 | 59.6 | 3.71 | 1.89 | 25.6 | 18.3 |
| 4 x 4 | 89.2 x 89.2 | 7.89 | 117 | 117 | 5.19 | 5.19 | 25.6 | 25.6 |

(1) CLS timber is produced to imperial sizes and the basic sizes listed are used as a nominal description.

(2) These sizes are the actual minimum sizes at 20 per cent moisture content.