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## LIST OF DELEGATES

### AUSTRIA
- **E Armbruster**: European Federation of Building Joinery Manufacturers' Association, Wien

### BELGIUM
- **E Broeckx**: Institut National du Logement, Bruxelles
- **A Ingelaere**: Automated Building Components, Bruxelles
- **L Montfort**: Institut National du Logement, Bruxelles

### CANADA
- **C R Wilson**: Council of Forest Industries of British Columbia, Vancouver

### DENMARK
- **M Johansen**: Danish Building Research Institute, Horsholm
- **H J Larsen**: Aalborg University Centre, Aalborg

### FINLAND
- **U Saarelainen**: Technical Research Centre of Finland, Espoo

### FEDERAL REPUBLIC OF GERMANY
- **P Frech**: Otto-Graf-Institut, Stuttgart
- **H Kolb**: Otto-Graf-Institut, Stuttgart
- **K Möhler**: Technical University of Karlsruhe, Karlsruhe

### FRANCE
- **P Crubile**: Centre Technique du Bois, Paris

### NETHERLANDS
- **J Kuipers**: Steven Laboratory, Delft

### NORWAY
- **O Brynildsen**: Norsk Treteknisk Institutt, Oslo

### POLAND
- **B Bany**: Centralny Ośrodek Badawczo Projektowy, Warszawa
- **W Nozynski**: Centralny Ośrodek Badawczo, Laskowa

### SOUTH AFRICA
- **T Williams**: Hydro-Air International, Johannesburg
SWEDEN

B Edlund
B Noren
B Thunell

Chalmers University of Technology, Goteborg
Swedish Forest Products Research Laboratory, Stockholm
Swedish Forest Products Research Laboratory, Stockholm

UNITED KINGDOM

L G Booth
H J Burgess
W T Curry
P Grimsdale
R Marsh
J G Sunley
J R Tory

Imperial College, London
Timber Research and Development Association, High Wycombe
Building Research Establishment, Princes Risborough
Swedish-Finnish Timber Council, Retford
Arup Associates, London
Timber Research and Development Association, High Wycombe
Building Research Establishment, Princes Risborough

1) Chairman and Co-ordinator, CIB-W18
2) Technical Secretary
2 CHAIRMAN'S INTRODUCTION

MR SUNLEY the co-ordinator of CIB-W18 and chairman of the meeting welcomed delegates to the eighth meeting of the Commission. He pointed out that the present meeting in Brussels followed the W18 tradition of holding each meeting at a different venue to permit local participation and to stimulate interest in the work of the Commission.

3 CO-OPERATION WITH OTHER ORGANISATIONS

ISO/TC 165: PROF LARSEN, the chairman of TC 165, reported that at the meeting of TC 165 in September it had been agreed that CIB-W18 should provide a draft international timber code for submission to TC 165. Their meeting had accepted for consideration the RILEM/W18 standard for testing joints but had reservations about handling the standards on plywood and structural sized timber since these might possibly be the responsibilities of TC 139 and TC 55 respectively.

DR KUIPERS suggested that the problems of defining areas of responsibility belonged within ISO. The RILEM group, he said, would continue to publish standards and submit draft ISO standards to TC 165 and then TC 165 could decide how they should be dealt with and which other technical committees should be involved.

DR BOOTH said that it would not be desirable for the plywood standard to be passed to TC 139. They had dealt mainly with clear plywood and would not necessarily understand the reasoning behind some of the decisions taken in producing the standard.

DR WILSON said that he would like to see TC 165 handle all structurally orientated standards to avoid confusion between different technical committees.

MR SUNLEY proposed that Prof Larsen, Dr Kuipers and himself, should make personal contact with the chairman of TC 139 to explain the problems, and this was agreed.

PROP LARSEN told the meeting that although a case could be made for retaining structural plywood in TC 165 it was more difficult to justify the retention of testing structural sized timber since TC 55 had already circulated for comment a standard on this subject. He explained that TC 165 would pass the W18 standard to TC 55 but could only take action if TC 55 failed to act.

RILEM 3-TT/CIB-W18: DR KUIPERS, the chairman of this group, reported on the meeting of 3-TT/W18 that had immediately preceded the W18 meeting. He said that the first annexe to the joints standard (on nail plates), the standard on testing structural sized timber and the standard on plywood testing were now ready for publication in the RILEM journal. It was agreed by the delegates that Mr Sunley and Dr Kuipers should together draft introductions to these papers specifically inviting comments within a defined period of time. After comments had been considered by the 3-TT/W18 sub-group the papers should be submitted to the main committee. DR KUIPERS asked the meeting to provide papers on sampling and analysis of data in support of the testing standards and suggested that a paper on the evaluation of test results for joints should be on the agenda for the next W18 meeting.
MR CURRY said that W18 and not the 3-TT/W18 sub-group must decide whether the basis for joint sampling should be density or compression strength.

MR BRYNILDESEN informed the meeting that funds were now available to him for the drafting of a Nordic standard on fasteners but because he was not aware of the latest changes introduced by NILEM the Nordic standard would probably differ from that produced by W18.

MR SUNLEY pointed out that it was unreasonable for members of W18 to agree to proposals at international meetings and then to produce national codes which were different. He reminded delegates that the membership of the 3-TT/W18 sub-group could vary to take account of the subjects under discussion and those with particular interests in testing would do well to contact Dr Kuipers.

PROF LARSEN asked the delegates if they agreed with the proposed corner loading method of measuring shear modulus given in the plywood standard. He said that he was still firmly convinced that this test in fact measured torsional stiffness and that there was a difference between these two properties.

PROF MÖHLER agreed with Prof Larsen that there were considerable differences between shear and torsional modulii and said that tests in Germany indicated that the ratio between these two properties could be as high as 5:1.

DR BOOTH said that he had not been convinced at earlier meetings of the differences between shear and torsional modulii and he thought that for material made up from thin plies the differences would be small. He pointed out that this form of test for shear modulus had been used for many years and in other fields, including the aircraft industry, and no one else had queried its validity. However, Dr Booth agreed that in view of Prof Mohler's observations further tests were required to finally resolve this problem.

MR SUNLEY asked Prof Larsen, Dr Booth and Prof Möhlert together to consider the problem but that their deliberations should not delay publication of the suggested standard.

DR KUIPERS wound-up his report on the 3-TT/W18 sub-group by outlining a timetable which would lead to the publication of standards on board materials, joints testing and annexes to joint testing.

IUFRO: MR SUNLEY outlined the program that Dr Poschi, Prof Madsen, Dr Wilson and Mr Pellerin were organising for August 1978. This comprised a conference on Wood Fracture Mechanics, a meeting of the IUFRO Wood Engineering Group and a seminar on Non Destructive Testing. Each of these activities was scheduled to last for approximately one week. Mr Sunley also asked those members of IUFRO present at this meeting of W18 to consider nominations for the co-ordinator of IUFRO as his term in that office had expired.

MR BRYNILDESEN reported on the activities of the IUFRO S5/CIB-W18 sub-group on duration of load and moisture content effects. He said that much of their effort so far had been concentrated on the prediction of strength by matching paired specimens, but results had been disappointing and the best method of predicting strength still appeared to be by measuring E. This meant that groups of thirty specimens would have to be tested rather than matched pairs. He
also reported that preliminary results on the strength to moisture content relation for European spruce supported Prof Madsen's work in this field. The sub-group still lacked funds for these projects. Tentative approaches to NATO had not been very promising since they did not support applied research. There were also problems on finding adequate facilities for the test work. The Canadian facilities would be fully extended for more than three years and no other single laboratory could accommodate the whole test program. However the sub-group was continuing with its work and would soon produce a detailed programme of work for the long-term bending project.

DR WILSON said that Dr Foschi and Dr Barrett were looking at the same problems of duration of load and moisture content in North America. He understood that the Western Forest Products Laboratory had extended their facilities and were now setting up test rigs. He asked if there were possibilities for co-ordination between WFPL and IUFRO/W18.

MR BRYNILDSEN said that he would welcome closer contact with the North American group.

ECE: PROF THUNELL reported on the activities within the United Nations Economic Commission for Europe. There were three areas of interest to W18, he said: the stress grading of coniferous sawn timber; finger jointing and dimensions for sawn timber. Although there had been agreement in ISO some years ago on dimensions there was now some pressure for rationalisation but this would probably not influence the work of W18. The ECE timber committee had now settled the stress grading issue, continued Prof Thunell. There were to be three grades; S10, S8, S6 with visual selection based on the Knot Area Ratio system. In addition there was to be a density limit for the highest grade (S10). The rules for finger joints in structural timber were also accepted. For both stress grading and finger jointing the Timber Committee had made recommendations on how to ensure the entry into service of the standard.

PROF LARSEN asked how density determination was to be carried out.

MR CURRY said that a satisfactory measurement of density was not practicable as part of the visual grading operation and the S10 grade would therefore probably be limited to machine selection.

PROF THUNELL concluded his report by stressing the urgent need for W18 to produce documents on sampling and analysis of test results. He asked for these to be ready before the end of 1978.

PROF LARSEN thought that there should be no problem in the analysis of test results. With samples of greater than 300 he was prepared to rely on engineering judgment to derive characteristic stresses. He suggested that the main problem was one of sampling.

DR NOREN and MR CURRY were agreed that sampling was perhaps the most difficult problem but they pointed out that different methods of interpretation could produce different characteristic stresses from one set of results. It was important to give guidance on methodology to produce consistent conclusions.

MR SUNLEY and MR CURRY drew the attention of the meeting to what they felt was a bias towards European red/whitewood in the proceedings of the ECE. They felt that this bias should be resisted.
4 STRUCTURAL STABILITY

PROF LARSEN introduced paper CIB-W18/8-15-1 "Laterally Loaded Columns" explaining that it looked at the problems that arose with combinations of axial forces and end moments. He apologised that the English translation was not available but undertook to provide this within one month for inclusion in the proceedings.

DR KUIPERS said that he too was interested in this work and hoped that one of his students would soon be preparing a report on combined compression and end moments.

5 GLUED LAMINATED STRUCTURES

DR ARMBRUSTER presented paper CIB-W18/8-12-3 "Glulam Standard Part 1 (FEMIB)". He explained that this first part of a Glulam standard laid down the grading rules for timber for laminating and was the result of agreement between manufacturers from eight European countries.

PROF LARSEN said that the two grades of timber specified by this standard permitted the use of an unacceptably low quality timber.

DR ARMBRUSTER said that the standard was intended to give sensible and practical grading limits that would result in economic use of timber. The lowest grade was not greatly inferior to many of the lower grades at present in use in Europe.

MR FRENCH told delegates that there were at present three grades for laminating timber in Germany. If they were to lose their top grade with a KAR limit of 0.2 then permitted stresses would be reduced from 140 kgf/cm² to 110 kgf/cm².

PROF LARSEN said that he could not accept these grades without supporting stresses which should be based on test results.

MR BURGESS pointed out that the lowest grade in the standard was equal to the lowest UK grade which was also based on KAR and the stresses for that grade were based on tests of individual laminae.

DR ARMBRUSTER asked the meeting for guidance on what testing was required and on how safe stresses should be assigned.

PROF MONTFORT said that a full testing program of complete laminated members would be a very difficult and expensive undertaking because of all the possible combinations and mixtures of grades, number of laminations, jointing etc.

Paper CIB-W18/8-12-1 "Testing of Big Glulam Timber Beams" was introduced by MR FRENCH who explained that this short paper summarised the contents of a lengthier test report. He said that the testing had included beams of 30 m length and from these beams had been cut the smaller beams. Predictably coefficient of variation of strength for the beams had been less, at 15 per cent, than would have been expected for solid timber. In answer to questions he said that the beams had been manufactured from finger-jointed grade 1 timber using urea glue. All laminae were 30 mm thick. In several cases failure had originated at a finger joint.

PROF LARSEN pointed out that this paper was one of several in which laminated beams had achieved only 70 per cent of their expected strength.
MR FRECH agreed with this. A design stress of 14 N/mm² with a safety factor of 3 had been used in calculation but the beams had only achieved factors of between 2.2 and 2.5 in a reduction approximately in the ratio of tension to bending strength. Part of this reduction could be attributed to imperfect finger joints.

PROF MOHLER also agreed that finger joints presented problems. Failure stresses at finger joints were generally about 60 per cent of the failure stresses in unjointed timber. He considered that construction methods should be taken into account in standards for laminated members.

PROF LARSEN favoured a simple approach – defining performance standards and allowing any material or constructional techniques that would meet those standards. He said that such an approach would have the advantage of avoiding a theoretical basis which could be significantly influenced by changes in production procedures.

MR SUNLEY said that setting performance standards had the advantage of being independent of species and therefore improved the chances of harmonisation in design methods.

MR CURRY did not agree with Prof Larsen; he pointed out that set target stresses had no connection with yields which were commercially important. There would be too much emphasis on what suited European redwood/whitewood and too little on a basic methodology.

MR MARSH found Prof Larsen’s system attractive to the practising engineer and asked why such a system could not be adopted for plywood.

MR CURRY and DR BOOTH said that they would not like to see the idea of performance standards introduced to plywood. They felt that such a simple system could have advantages if the relativities between strength properties remained the same for different species but since they did not remain the same there would inevitably be quite serious anomalies for some species of timber. For plywood the very large variety of species, lay-ups and other factors were against the satisfactory implementation of the system.

It was finally agreed that Dr Arnbuster, Mr Frech and Dr Edlund should consult together to produce a paper on performance standards or stresses for glued laminated timber.

MR FRECH introduced paper CIB-W18/3-12-2 "Instructions for the Reinforcement of Apertures in Glulam Beams" explaining how the strain around holes in beams had been measured with and without reinforcing. The tests had been conducted on 22 beams and although the optimum thickness of reinforcing plate had perhaps not been achieved they had arrived at acceptable and practicable rules which would have been much more difficult had they had to rely on calculation alone.

DR BOOTH asked if reference could be made to the original report (of which this paper was a summary). Mr Frech agreed to do this.

6 PLYWOOD

Paper CIB-W18/3-4-1 "Sampling Plywood and the Evaluation of Test Results" and appendix 3-4-1A were presented by DR NORREN, who explained that this was an introductory paper to this topic and he invited comments on the content and methods proposed. He pointed out that his choice of 30 for the number of panels to be tested was an arbitrary figure and should really depend on how
the population was defined. He also pointed out that in the absence of
contradictory evidence it was assumed that the strength properties of plywood
were normally distributed.

DR WILSON told the delegates that in Canada tests were being conducted to
evaluate the differences between plywood made from various species. Advice
from statisticians was that at least 60 panels per species would be required
to test for differences in the means of 23 species. The number of tests would
have to be increased to test for differences in lower exclusion values or
if the distributions were non-normal. Dr Wilson also suggested that the number
of tests should depend on whether characteristic stress levels were being
established or whether the tests were to check for compliance with an existing
standard. In the latter case he would expect that fewer tests would be required.

DR BOOTH agreed with Dr Wilson. He also drew attention to table 6.1 saying
that if those values of 'k' were used then stresses would be lower than those
in current use.

DR NOREN accepted the comments of Dr Wilson and Dr Booth and said that perhaps
this paper was inclined towards compliance testing, but the assumption of normal
distributions, which needed further investigation, had been made in establishing
existing stresses and there was a better statistical foundation for the factors
of table 6.1 than for a fixed $k = 1.64$. However, he pointed out that although
he had used a 75 per cent confidence level to formulate table 6.1 the question
of confidence levels should not be for W18 to decide.

PROP LARSEN said that distributional functions were not required for the timber
code and in any case it had been internationally accepted that all material
strengths were log-normally distributed and loads were normally distributed.

MR CURRY did not agree with such sweeping generalities which, he said, were gross
simplifications.

PROP EDLUND suggested that rather than discuss the suitability of various
distributional functions a non-parametric method could be used.

DR BOOTH said that the most important part of the problem was to agree on a
method so that consistently comparable fifth percentile values could be derived
by different countries. It was also important to decide whether nominal stresses,
actual stresses or load capacities were to be used to define the strength of
plywood.

It was agreed that Dr Noren should produce a second draft of his paper, under
the title "Sampling of Plywood and the Derivation of Stress Values" and that
Dr Booth should produce a paper on the evaluation of test results.

7 STRUCTURAL DESIGN CODES

DR BANY distributed English translations of "Polish Standard PN-73/E-03150:
Timber Structures" (CIB-W18/8-102-1). He explained the derivation of the fifth
percentile bending stress given in the code and said that this could be modified
by factors of 0.67 for long-term loading and 0.4 for size, to produce a working
stress of 130 kgf/cm². He told delegates that an approximate comparison with
the German code showed that the German code was more conservative.

PROP LARSEN asked Dr Bany if he could provide a short paper on load factors and
design procedures that were applicable to Poland. Dr Bany agreed to do this.
A summary of the contents of 'The Russian Timber Code' (CIB-W18/8-102-2) was circulated by the secretary who explained that copies of the complete code were available but unfortunately only in Russian.

8 CIB TIMBER CODE

MR CURRY told the meeting about his interpretation of the JCSS document, published by CEB, "Bulletin d'Information No 116: Volume 1; Common Unified Rules for Different Types of Construction Material" and how he expected difficulties in the adoption of the principles given in this document when they were applied to timber structures. He pointed out that since the code would have to be applied to all structures it would not be possible to avoid conflict with some existing practices and particularly with what could be called traditional forms of construction. Mr Curry found it regrettable but unavoidable that timber should become involved in limit state design where the theoretical ideal of a quantifiable probability of failure could not be achieved.

MR SUNLEY said that limit state codes were being produced for other materials and timber was being pressed to follow suit.

PROF LARSEN reported that the Nordic countries had discussed limit state design for timber and would probably adopt a method 2 approach. They had presented a paper to ECE on this subject which was not based on Volume 1 of the JCSS Code. Prof Larsen said that Volume 1 had been produced by designers of concrete structures and had a definite bias towards concrete practice. It was not acceptable for timber and it was unlikely that it would be adopted by those responsible for steel codes.

MR CURRY said that it was not necessary for W18 to adopt the whole of the content of Volume 1 — there was in any case insufficient information to allow a reasonable interpretation — but the principles could be accepted. Mr Curry also pointed out that loading was not suitably defined by Volume 1 to allow valid comparisons to be made with existing procedures.

MR SUNLEY said that he would take into account the views that had been expressed at the meeting and if asked for comment on Volume 1 by the Joint Committee on Structural Safety would summarise the objections as: inadequate definition of loads and their associated gamma factors; a bias towards concrete that was unlikely to produce harmonisation between materials; insufficient information for a reasonable interpretation.

Paper CIB-W18/8-100-1 "CIB Timber Code: List of Contents (second draft) was distributed by PROF LARSEN. He explained that he had drafted this paper as a response to the request from ISO/TC 165 for a list of contents of the timber code.

It was agreed that Prof Larsen, Dr Kuipers, Prof Mohler, Dr Booth and one other should form an editorial group to draft a code. Comments on the List of Contents were invited before 1 December 1977.

9 OTHER BUSINESS

MR WILLIAMS told the meeting that there was an urgent need for an international trussed rafter code. Other delegates agreed that Mr Williams should form a corresponding committee consulting with the International Truss Plate Association and draft a trussed rafter code for consideration by W18.
MR SUNLEY proposed that the secretary should, after minor editorial amendments, submit document CIB-W18-1 "Symbols for Use in Structural Timber Design" to ISO TC/165. This was agreed.

MR SUNLEY closed the meeting and thanked PROF MONTPORT and MR BROECKX, as representatives of the Institut National du Logement, for their hospitality, for the interesting excursions they had arranged and for the facilities that had been made available for the meeting of the Commission.

10 NEXT MEETING

The next meeting of CIB-W18 will take place on 7,8,9 June 1978 in Edinburgh, Scotland. Topics for discussion will include:

1 Sampling of plywood and evaluation of test results
2 Glued laminated structures
3 CIB Timber Code

The CIB-W18 meeting will be preceded on 6 June by a meeting of the RILEM 3-TT/CIB-W18 sub-group.

Arrangements for these meetings will be made by the Timber Research and Development Association.
11 PAPERS PRESENTED AT THE MEETING

- CIB-W18/3-4-1 Sampling Plywood and the Evaluation of Test Results - B Noren
- CIB-W18/3-12-1 Testing of Big Glulam Timber Beams - H Kolb and P Frech
- CIB-W18/3-12-2 Instructions for the Reinforcement of Apertures in Glulam Beams - H Kolb and P Frech
- CIB-W18/3-12-3 Glulam Standard Part 1: Glued Timber Structures; Requirements for Timber.
- CIB-W18/3-100-1 CIB Timber Code: List of Contents (second draft) - H J Larsen
- CIB-W18/3-102-1 Polish Standard PN-73/B-03150: Timber Structures; Statistical Calculations and Designing
- CIB-W18/3-102-2 The Russian Timber Code: Summary of Contents
- CIB-W18/3-103-1 Draft Resolutions of ISO/TC 165

List of CIB-W18 Papers

Membership of CIB-W18 (Nov 72)
LIST OF CIB-W18 PAPERS

Bruxelles
October 1977
Technical papers presented to Working Commission W18 - Timber Structures are classified by a code identifying the meeting at which the paper was presented, the subject heading and the number of the paper. The full classification number of a document will start with CIB-W18, although where the context is clear this prefix may be omitted.

Example: CIB-W18/4-102-5
refers to paper 5 (Extract from Norwegian Standard NS 340 - "Timber Structures") on subject 102 (Structural Design Codes) presented at the fourth meeting of W18 (Paris, February 1975).

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

Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden, Feb/March 1977
8 Bruxelles, Belgium, October 1977

Subjects are denoted by the following numerical classification:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing
Listed below, by subjects, are all papers that have to date been presented to W18. When appropriate some papers are listed under more than one subject heading.

LIMIT STATE DESIGN

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

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

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

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

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

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

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

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

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

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

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

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

6-2-1  Lattice Columns - H J Larsen

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

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

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

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

SYMBOLS

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

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

1  Symbols for Use in Structural Timber Design

PLYWOOD

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

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

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

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

5-4-1  The Determination of Design Stresses for plywood in the revision of CP 112 - L G Booth
5-4-2 Veneer Plywood for Construction - Quality Specification - ISO/TC 139 - Plywood, Working Group 6

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

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

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

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

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

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

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

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

STRESS GRADING

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

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


STRESSES FOR SOLID TIMBER

4-6-1 Paper 11 Derivation of Grade Stresses for Timber in UK - W T Curry

5-6-1 Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes - W T Curry

5-6-2 The Description of Timber Strength Data - J R Tory

5-6-3 Stresses for ECl and EC2 Stress Grades - J R Tory
TIMBER JOINTS AND FASTENERS

6-5-1 Standard Methods of Test for the Determination of some Physical and Mechanical Properties of Timber in Structural Sizes (third draft) - W T Curry

7-5-1 Strength and Long-term Behaviour of Lumber and Glued-laminated Timber under Torsion Loads - K Möhler

1-7-1 Paper 12 Mechanical fasteners and fastenings in timber structures - E G Stern

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

4-7-2 Paper 9 Test Methods for Wood Fasteners - K Möhler

5-7-1 Influence of Loading Procedure on Strength and Slip Behaviour in Testing Timber Joints - K Möhler

5-7-2 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures - RILEM, 3TT Committee

5-7-3 CIB Recommendations for the Evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures - J Kuipers

6-7-1 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures (seventh draft) - RILEM, 3TT Committee

6-7-2 Proposals for Testing Joints with Integral Nail Plates - K Möhler

6-7-3 Rules for Evaluation of Values of Strength and Deformation from Test Results - Mechanical Timber Joints - M Johansen, J Kuipers, B Norén

6-7-4 Comments to Rules for Testing Timber Joints and Derivation of Characteristic Values for Rigidity and Strength - B Norén

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WORKING COMMISSION W13 - TIMBER STRUCTURES

SAMPLING PLYWOOD AND THE EVALUATION OF TEST RESULTS

B Norén

Bruxelles
October 1977
TESTING OF STRUCTURAL PLYWOOD FOR ASSIGNING CHARACTERISTIC STRENGTH VALUES - (1) SAMPLING AND ASSESSING TEST RESULTS

B Norén - Swedish Forest Products Research Laboratory, Stockholm

1. Purpose

The purpose of the testing is to establish characteristic values for strength (and stiffness) to be used in design and verification of safety of structures. It is essential in defining the population of plywood and in choosing sampling method that conditions in production, marking and end-use are considered.

2. Definition of the population

2.1 The population of plywood to which characteristic strength values are assigned shall be limited thus that the strength deviation at end-use is principally due to random variations. Hence, the population shall be unambiguously specified with respect to type (species) and grade (reference to a product standard), thickness and construction (lay-up) and - possibly - source (factory) and production time.

2.2 Integrating panels of different thickness and construction or from different sources into a mixed population is permitted either if it is proved that there is no significant deviation of characteristic strength values between the sub-groups or if the sub-groups are mixed in a random way when the plywood is used.
Plywood strength is sometimes expressed by a single veneer stress value to be applied on parallel plies. If the approximation of this model is accepted at different constructions (thicknesses), a population of mixed constructions could be considered in assigning characteristic strength values for design and possibly in sampling for continuous quality control testing. For the kind of testing, here dealt with, plywood of different constructions (thicknesses) should, however, as a rule be considered as belonging to different populations.

Sometimes the product standard (in particular the specifications for grading the veneers) do not guarantee that the plywood produced at different factories will have the same strength, for example due to different timber sources. In such cases the population may have to be specified with respect to source.

2.3 The period during which the plywood referred to as a population is produced should be as long as possible without involving such changes in the production which can be expected to have a significant influence on the properties to be established by the testing.

2.4 A population consisting of plywood from several factories may be substituted by plywood from a limited number of factories, if it is proved beyond doubt that this will not increase the estimated characteristic strength. On similar conditions a population consisting of plywood of several constructions (thicknesses) may be substituted by a population consisting of a limited number of selected constructions.

If one characteristic strength value shall be evaluated for a plywood of standard type, construction and thickness, made at for example 150 factories, one may find out from a limited number of testing (compared with what is stipulated in p. 3.1), either that the population must be divided into a number of populations, c.f. 2.1, or - if the difference between factories is comparatively small - that the strength values can be evaluated from a number of those factories that are at the lower end with respect to the strength of their produced plywood.
3  **Sampling of panels**

3.1 The number of panels in a sample, drawn for testing from a population or substitute population, defined in p. 2, must allow each strength property to be tested on specimens from at least $N = 30$ panels of each construction (thickness) and thereby from at least $n = 5$ panels from each factory (production time).

3.2 Samples shall be drawn at random over the production time defined for the population (p. 2.3). When the number of panels from one factory (production line) for testing one specific strength property is $n$, these panels must be drawn from $n$ different batches (preferably have been produced by $n$ different shifts).

4  **Sampling of specimens from panels**

4.1 A specific schedule shall be used for the cutting of test specimens from panels. This schedule shall define the distance between the specimens and, as a rule, their position relative to the edges of the panel. If a characteristic feature of the panels (such as a joint) occurs on a regular distance from the edges of the panels, the position of the cutting schedule relatively to the edges shall be changed at random from panel to panel.

4.2 When the size of the cutting schedule is larger than the panel, the schedule may be applied on two or more adjacent panels in the batch.

The number of specimens and cutting schedules are generally given in testing standards. (For structural plywood see ISO/TC 165 N, document 14E.)

5  **Definition of characteristic value**

5.1 For characteristic values of strength or moduli of elasticity (rigidity etc.) are used the 1- , 5-, 10- and 50-percentiles. As a rule the 5-percentile should be used for the strength and moduli of elasticity for calculating strength (verification of limit state of failure), while the 50-percentile
(mean value at normal distribution) should be used for calculation of
deformation at the serviceability limit state.

The choice of percentile is dependent on the base used in calibrating
the (partial) safety factors. If the standard deviation and type of
distribution is known one characteristic value can be calculated from
another.

6 Derivation of characteristic value

6.1 When characteristic strength values are estimated for a population of
plywood panels (p. 2) from test results from samples of limited numbers
(p. 3) the unreliability of the results should be duly considered. This is
achieved by applying an increased confidence.

With "increased confidence" is here meant that the method of estimation
should imply a probability higher than 0.5 that the estimated charac-
teristic value is lower than the real value.

6.2 The characteristic value may be estimated as

\[ m_k = \bar{m} \exp(-k \frac{s}{\bar{m}}) \]  \hspace{1cm} (6:1)

and if \[ k \frac{s}{\bar{m}} \leq 0.25 \] as

\[ m_k = \bar{m} - ks \]  \hspace{1cm} (6:2)

In (6:1) and (6:2) \( m_k \) denotes the characteristic value estimated for the
population, \( \bar{m} \) is the mean value and \( s \) the standard deviation for the sample.
The value of the coefficient \( k \) depends on the demand of probability that
\( m_k \) is not overestimating the characteristic value of the population, on the
percentile used to define this characteristic value and on the number of
individual values (N) on which \( \bar{m} \) and \( s \) are based.

If the real characteristic value of the population is \( M_k \) we may want
75\% confidence that the value \( m_k \) calculated from a sample by (6:2) is
lower than \( M_k \). In the case the individual strength values are normally
distributed and $M_k$ is defined by the 5-percentile, the values given in Table 6.1 may be used for $k$ in (6.2).

Table 6.1 Value of $k$ in (6.2) for estimating characteristic strength (5-percentile) from $N$ strength values in a sample

<table>
<thead>
<tr>
<th>$N$</th>
<th>15</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k$</td>
<td>1.99</td>
<td>1.93</td>
<td>1.87</td>
<td>1.83</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Classification with respect to characteristic value

A population (p. 2) can be considered in grade if a value, calculated from test results from a sample of limited number of panels (p. 3) is at least equal to the demand characteristic grade-value $m_k$:

$$m \exp(-k \frac{S}{m}) \geq m_k \quad (7.1)$$

or if $k \frac{S}{m} \leq 0.25$

$$m - ks \geq m_k \quad (7.2)$$

(Denotations see p. 6.2)

The value of $k$ depends on the demanded confidence of the classification, on the definition of the characteristic value (what percentile) and on the number of panels tested in the sample.

If it is demanded that the probability should be 0.75 that a population with a real characteristic value $M_k = m_k$ is rejected as being below grade, the $k$-values from Table 6.1 can be used, provided the distribution of individual strength values is normal and the characteristic value is defined by the 5-percentile.

However, the demand may alternatively be expressed by the probability that a population with a characteristic strength $M_k = m_k - C s_p$, less than the grade-value ($m_k$) is estimated as within the grade. As an example 1) is assumed $C = 0.25$, normal distribution of individual strength values, standard deviation of the population equal to that of the sample ($s_p = s$). It is further suggested that it is allowed 15.9% probability of acceptance to the grade of a population with a 5-percentile 0.25 times the standard deviation lower than the grade value. The value of $k$ is then calculated from

$$k = \frac{1}{\sqrt{n}} + (1.645 - 0.25) \quad (7:3)$$

These $k$-values are lower than them given in table 6.1 for estimating the 5-percentile of the population, the reason partly being the assumption $s_p = s$. If instead the coefficient of variation is known ($V_p = V = s/m$), the $k$-value is changed according to table 7.1.

And if $s_p$ as well as $V_p$ are unknown, the $k$-value to be used in (7:2) can be determined from the non-central $t$-distribution, see table 7.1, last line.

<table>
<thead>
<tr>
<th>N</th>
<th>15</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>100</th>
</tr>
</thead>
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<td>1.66</td>
<td>1.62</td>
<td>1.58</td>
<td>1.55</td>
<td>1.50</td>
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<tr>
<td>$V_p$ and $s_p$ unknown</td>
<td>1.88</td>
<td>1.80</td>
<td>1.71</td>
<td>1.66</td>
<td>1.54</td>
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Appendix: Paper 8-4-1 A
TESTING OF STRUCTURAL PLYWOOD FOR QUALITY CONTROL
B Norén
Swedish Forest Products Research Laboratory

A:0 General

There are several methods to check whether a product is continuously produced to certain demands. There are direct methods of product control, such as testing strength properties of plywood on samples of panels regularly drawn from the production. There are indirect methods, such as checking the production means or checking a substitute for the property (visual grading, non destructive testing) or a substitute for the product (veneer testing instead of plywood testing).

In manufacturing of plywood, the quality control is generally a combination of indirect and direct methods. By the indirect methods it is possible to predict with some confidence the strength properties of the plywood. Thereby the number of panels destroyed by testing is reduced. Actually, continuous strength testing of structural plywood is not applied in quality control in many countries. Such testing, carried out for some period may, however, be valuable for following up assigned characteristic strength values.

A:1 Purpose

Testing samples from the production line or the stock is a way of verifying that the products satisfy certain demands. The test results should primarily guide the manufacturer with respect to measures to be taken concerning the production. By following the mean values and the standard deviations of the samples, he can separate chance causes of variation which are natural for the production from assignable causes which he can change.

In addition, there may be an external official demand, such as for a minimum 5-percentile of strength. Usually, this demand is also verified primarily through the manufacturers own testing of samples. Possible official spot-checking by testing once or twice a year can contribute very little to the statistical treatment and is mainly used to calibrate the factory testing.
A:2 Sampling

Samples for testing must be distributed with respect to such controlled changes in the production which are likely to affect the property to be checked. It is a matter of defining "sub-populations" or batches within which the variation of the tested property is principally random.

The number of test specimens (plywood panels) to be tested during a period is often made dependent on the number of produced units. Here is, as an example, given the number of joints to be tested from each production batch at routine quality control of finger-jointed structural timber.

<table>
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<th>Number of joints in production batch</th>
<th>Minimum number of joints to be tested</th>
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<tr>
<td>1000 or less</td>
<td>3</td>
</tr>
<tr>
<td>1001 - 2000</td>
<td>4</td>
</tr>
<tr>
<td>2001 or more</td>
<td>5</td>
</tr>
</tbody>
</table>

Specimens should be drawn at random over the period the batch is produced. Although in many kinds of production the number of units in a batch will be less than 1,000 there will be several such batches produced to give a sufficient number of test results for estimating the statistical parameters within a reasonable short time.

A:3 Characteristic strength value

To verify from the quality control testing that the product has a characteristic strength at least equal to a specified value, the method previously shown can be used (7. Classification with respect to characteristic value.) The systematic (assignable) variations found from the control charts must of course be excluded. The results from similar batches can be pooled and after some time the number of results is increased to allow a low k-factor to be applied. Furthermore, a mean value of the standard deviation may be used instead of the deviation of the sample.

1) ECE-TIM/WP. 3/AC. 3/8 Annex II 24 June 1977
INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

Testing of Big Glulam Timber Beams

by

H Kolb and P Frech

Otto-Graf-Institut Stuttgart

Bruxelles
October 1977
Introduction

For a long time it has been known that the bending strength of timber depends on its dimensions, especially on its cross-sectional depth.

In 1924 Newlin and Traver first showed this dependence by tests and theoretical deduction and determined for cross-sectional depths up to 12 inches (about 300 mm) a reduction factor dependent on the depth $H$

$$k = 1.07 - 0.7\sqrt{\frac{H}{2}}$$

With the development of glulam structures it was possible to produce beams with depths exceeding 300 mm and it became necessary to modify or reestablish the formula.

Dawley and Youngquist indicated in 1947 a reduction factor $k$ according to this formula:

$$k = 0.625 \frac{(H^2 + 143)}{(H^2 + 88)}$$

With an increasing beam depth $H$ the reduction according to this formula became bigger than for the formula of Newlin and Traver.

A board with a 2 in. depth (about 50 mm) was used as "reference beam".

As in the American calculation instructions most of the calculated stresses with regard to a sufficient rupture strength apply to a cross-sectional depth of 12 in. (300 mm), this formula is:

$$k = 0.81 \frac{(H^2 + 143)}{(H^2 + 88)}$$

In the above quoted formulae the depth $H$ is in inches.
In fig. 3 the American and Soviet reduction factors $k$ are represented and compared to the mean value curve calculated by us presuming a safety against rupture of $\gamma = 3.0$.

In general it can be said that for deeper beams the allowable bending stress has to be reduced with a factor which corresponds to the relation between the allowable tensile stress and the allowable bending stress.

As long as in Germany no reduction of the allowable bending stress is demanded for beam depths $h > 500$ mm, it is necessary to have a very careful choice and grading of timber with, if possible, laminae of quality I.

As an alternative to the introduction of reduction factors one has to think it over if the requirements to the existing quality classes concerning the gross density and the annual layers width shouldn't be increased.

A further natural requirement is a good execution of finger joining in all boards of the beam.
<table>
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<th>depth mm</th>
<th>rupture load max P kN</th>
<th>calculated bending stress max d_{B} N/mm²</th>
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<th>E-module E N/mm²</th>
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Fig. 1 Calculated rupture stress as a function of the beam depth or span and the mean value curve resulting from that.
Fig. 2 Longitudinal stresses at different beam depths
Fig. 3 American and Soviet reduction factors $k$ and mean value curve from research work based on a threefold safety against rupture.
INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

Instruction for the Reinforcement of Apertures in Glulam Beams

by

H Kolb and P Frech

Otto-Graf-Institut Stuttgart

Bruxelles

October 1977
Instructions for the reinforcement of apertures in glulam beams

(Recommendation on the basis of tests carried out by FMPA Stuttgart)

Reinforcing must be done by beech plywood slabs AW 100 according to DIN 68705, sheet 3.

The total reinforcing thickness \( d \) (per side \( d/2 \)) is determined according to the shear stress \( \tau \) in the middle of the aperture and the beam width \( B \).

<table>
<thead>
<tr>
<th>shear stress( \tau ) ( \text{N/mm}^2 ) ( (\text{kp/cm}^2) )</th>
<th>total thickness ( d ) of the reinforcement as a function of the beam width ( B ) ( % )</th>
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<td>10</td>
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<td>35</td>
</tr>
<tr>
<td>0,8 (8)</td>
<td>50</td>
</tr>
<tr>
<td>1,2 (12)</td>
<td>65</td>
</tr>
</tbody>
</table>

Intermediate values must be interpolated linearly.
Slab thickness \( \geq 10 \text{ mm} \).
Size of the aperture's and reinforcements

\[ r \geq 25 \text{mm} \]

\[ a \leq H \]
\[ b \leq 0.4H \]
\[ a_i \geq 0.25a \geq b_i \]
\[ b_i \geq 0.4b \geq 0.1H \]

Grain direction of the face veneer parallel to the grain direction of the beam

Gluing with resorcinol glue, pressure about 0.6 N/mm² (6 kp/cm²)
The corners have to be rounded with a rayon of at least 25 mm. Normally the apertures should be symmetrical to the longitudinal axis of the beam. But at least a distance of 0,3 H to the above or below border has to be observed.

In the region of the aperture and the reinforced zones no important single loads ought to be introduced into the beams. If ducts with media the temperature of which doesn't correspond to the room temperature are directed through the apertures, the ducts have to be carefully insulated. Cross-cut ends should be protected by appropriate coatings against uncontrolled penetration of moisture.

If no appropriate press equipment is available for the gluing of the reinforcing slabs, they can be mounted by nail gluing according to DIN 1052, chapt. 11.5.9. The holes in the veneer slabs have to be rough-drilled with 85 % of the nail diameter.

It is necessary that during gluing the moisture content of the slabs corresponds to the expected compensating moisture.

Apertures are openings where at a shear stress \( \tau = 1,2 \, \text{N/mm}^2 \) a or \( b \geq 0,05 \, \text{H} \) and at a shear stress \( \tau = 0 \, \text{N/mm}^2 \) a or \( b \geq 0,10 \, \text{H} \). Between these values one has to interpolate linearly.

This instruction applies only to glulam beams which are mounted under the roof, thus not exposed to weather from one or all sides, see also DIN 1052, sheet 1, 3.2.1.

A more detailed version of this paper has been published in: "Holz als Roh- und Werkstoff;" 35. Jahrgang, Heft 4, April 1977.
GLULAM STANDARD PART 1
GLUED TIMBER STRUCTURES; REQUIREMENTS FOR TIMBER
(Second Draft)

Bruxelles
October 1977
Preface:

This GLULAM-STANDARD should give the base for determinations relating to timber, rating and performance of glued timber structures in the member countries of GLULAM. Furthermore this standard should be the base for the intended CEN-and ISO-standards in the field of glued timber structures.

There are two parts of the GLULAM-STANDARD:

Part 1: GLUED TIMBER STRUCTURES
        REQUIREMENTS FOR TIMBER

Part 2: GLUED TIMBER STRUCTURES
        RATING AND PERFORMANCE
        (will be prepared)

1. Scope:

This draft of the GLULAM-STANDARD Part 1 applies to laminated or to cross-laminated load bearing structural elements made of sawn timber from coniferous species. Indicated dimensions refer to the planed condition unless they do not explicitely refer to the raw (unplaned) condition.
2. Definitions:

2.1 Laminated Timber:
Laminated timber consists of boards glued together, the grain directions of all the boards being essentially parallel. Fig. 1.

2.2 Cross Laminated Timber:
Cross laminated timber consists of boards glued together, the grain directions of neighbouring boards being not parallel. Fig. 2.

2.3 Knot Area Ratio (KAR):
The knot area ratio is the summary of the projected cross-sectional areas of all knots within a predetermined reference length parallel to the grain of the wood and within an area, where a maximum may be expected, divided by the total cross-sectional area of the lamella. Reference length see section 5.4.

2.4 Moisture Content:
Weight of water within the wood, expressed as percentage of the weight of oven-dry wood.

2.5 Checking:
Shrinkage checks and heart checks: Separations usually across the growth rings as a consequence of stresses due to shrinkage of the wood.
Ring shake: Separations following the grain, between the individual growth rings.
Splits: Checks following the grain, forming cracks extending either partially or thoroughly across the board.

2.6 Slope of Grain:
The angle between the directions of the grain and the axis of the wooden piece, measured over an agreed distance.
2.7. Wane:
Local deviations of the cross-section of the board from its clear rectangular shape due to the original shape of the roundwood.

2.8. Spiral Grain:
Spiral grain is given, if the fibres of the wood wind spirally around the axis of the log.

3. Methods of Measurement and Determination:

3.1 Knot Area Ratio (KAR):
------------------------
The maximum knot area ratio of each lamella shall be determined. Knots with a diameter of up to 5 mm may be ignored. All other knots and knotholes, irrespective of their shape and location, shall be included in the determination of KAR, by projecting them rectangularly into the plane of the board. Determining the projected area, overlappings of the projected knots are neglected. Fig. 3.

3.2. Slope of Grain:
The slope of grain shall be measured over a reference length of 500 mm.

4. Moisture Content:
The moisture content shall be measured by means of proven, suitable and calibrated instruments.

Explanatory Note:
Measurements must be made prior to gluing, after the timber had been seasoned to the required moisture content and brought to the necessary dimensions.
5. Quality Requirements:

5.1 Grades:
Sawn timber from coniferous species of European origin for laminated or cross-laminated glued load bearing timber structures are classified into two categories, namely

Grade 1 (Structural Timber)
Grade 2 (Normal Building Timber).

Unless it is not commented in the following requirement specifications which grade they refer to, they have to be applied to both of them.
Sawn timber from coniferous species of other origin may be used, provided it meets the requirements of this standard.

5.2 Moisture Content:
At the time of gluing the timber shall have a moisture content corresponding to the average moisture content which may be expected to have it as a rule under service conditions, whereas the moisture content must not decrease below a minimum of as much as 7% nor in any case exceed a maximum of as much as 17%.
Therefore, as a rule, the moisture content shall be at the time of gluing:

With service in closed, heated rooms
\[ 9 \pm 2\% \]
With service in closed, unheated rooms or in open, roofed humidity
\[ 12 \pm 2\% \]
With service in open air or in rooms with exceedingly high humidity
\[ 15 \pm 2\% \].
The difference of the moisture contents among the individual lamellas must not be more than 4%.

5.3 Raw Density, Growth Ring Width:

There is no need to determine the raw density of the timber in particular. For the wood species mentioned under point 5.1 it may be assumed to be above 0.4 in air dry condition (wood moisture content of 20%).

The requirements are deemed to be met, if the average width of the growth rings is as much as

for Grade 1 : 5 mm maximum
for Grade 2 : 6.5 mm maximum.

Wider growth rings are permissible, if the raw density is as much as 0.4 g/cm³.

Particular attention is to be paid to extremely narrow-ringed, light-weighted timber.

5.4 Knot Area Ratio:

The knot area ratio shall not exceed within a reference length of 300 mm:

0.30 for Grade 1
0.50 for Grade 2.

5.5 Size of Spike Knots:

In addition to paragraph 5.4, the width of spike knots, measured at the surface of the timber as shown in Fig.4, shall not exceed:

25 mm for Grade 1
40 mm for Grade 2.
5.6. **Slope of Grain:**

As a rule within a reference length of 500 mm, the slope of grain shall not exceed:

- Grade 1: 35 mm = 7% 1:14, locally up to 10%
- Grade 2: 60 mm = 12% 1:8, locally up to 20%.

5.7. **Pith:**

Pith streaks and timber cut through the pith are permissible, provided the pith is not spongy.

5.8. **Checks:**

Checks are permissible with boards intended for horizontal laminated or cross-laminated structural members, provided the angle between check and surface of the board is at least 45°, Fig. 5.

However, only very small heart checks and medium sized checks due to shrinkage are permitted for boards intended for vertically glued load bearing structural members, irrespective of the magnitude of the angle referred to above.

5.9. **Wane:**

Wane is permissible for lamellae, provided they do not show on glued structural members in planed condition.

5.10. **Resin:**

Parts, being resinous over their entire surface or showing resin pockets scattered over their surface are not permissible. Smaller resin pockets may be permitted to a low extent.
5.11 Other Defects:

Not permissible are:

Spiral grain (except for a low extent)
Mechanical damages
Insect ducts (permissible to a low extent, but they must not show on the finished member).

Rot
Discolourations (except blue stain)

Further on, all flaws causing a considerable loss of strength or having an adverse effect on gluing.

5.12 Dimensions:

5.121 Thickness:
Generally the thickness of the lamella after planing shall not exceed:

33 mm.

Under particular circumstances, as straight beams and/or interior use, and under favourable conditions, the thickness of the lamella may range up to:

45 mm.

With curved structural members the thickness of the lamellae shall be determined according to GLULAM-STANDARD, Part 2.

5.122 Width:
The width of the lamella shall not exceed 210 mm, provided no special precautions are taken. The total cross-sectional area of the individual lamella shall not exceed 70 cm$^2$.

6. References to Other Standards:

GLULAM- STANDARD Part 2.

GLUED TIMBER STRUCTURES RATING AND PERFORMANCE

( will be prepared )
Fig. 2
$$Kar = \frac{F_1 + F_2 + F_3}{b \times h}$$

Fig. 3
INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES
AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

LATERALLY LOADED TIMBER COLUMNS
TESTS AND THEORY

H J Larsen
Instituttet for Bygningsteknik, Aalborg
Denmark

MEETING EIGHT
Bruxelles, Belgium
October 1977
1. INTRODUCTION

In the classical theory of centrally loaded columns and lateral buckling of beams it is assumed that the materials are ideal-elastic and that the members are straight up to a certain critical load by which the rupture occurs by sudden deflection of the structure. The rupture is thus considered a stability phenomena only.

The correspondence between theory and practice is good for slender structures, but for non-slim structures there are great deviations which are traditionally explained by a stress dependent modulus of elasticity.

For timber, at least, there is bad correspondence between the moduli of elasticity normally measured and those that must be assumed theoretically to explain the load-carrying capacities for columns that are found by tests. For columns the correspondence between theory and tests has proved much better if the columns are assumed to be elastic to rupture but with deviations from straightness.

In the present paper this view has been extended to comprise lateral buckling of beams with axial force under the assumption of pre-curvature in two directions and pre-torsion.

The theoretical load-carrying capacities have been derived in section 2. The expressions have been verified by tests described in section 3.

In section 4 the load-carrying capacity expressions found are discussed and simple, approximated design expressions are set up.

2. THEORY

2.1 The differential equations

![Diagram of a beam with initial curvatures and initial torsion]

Figure 2.1

A straight beam with the length \( l \) and constant cross-section is considered. The cross-section with the area \( A \) is shown in the figure as a rectangle. For other sections \( X \) and \( Y \) are the main axes. The bending stiffness about the axes is \( EI_x \) and \( EI_y \), while the torsional rigidity is \( GL \). \( E \) is the modulus of elasticity, \( G \) the shear modulus, \( I_x \) the moment of inertia about the \( X \)-axis, \( I_y \) the moment of inertia about the \( Y \)-axis and \( I_t \) the moment of inertia in torsion. The polar moment of inertia is denoted \( I_p \), and \( i_p \) is the polar radius of gyration: \( i_p^2 = I_p/A \). The \( z \)-values are measured from the middle of the beam.

If the shear deformations and displacement in the \( Z \)-direction are disregarded the deformations of the beam can be described by the displacements \( u \) and \( v \) in the \( X \)- and \( Y \)-direction, respectively, and the rotation \( \varphi \) about the \( Z \)-axis. The positive directions are shown in figure 2.1.

The beam is simply supported at the ends, i.e. \( u = v = 0 \) and secured against rotation about the \( Z \)-axis, i.e. \( \varphi = 0 \).

The loading are equal end moments \( M \) about the \( X \)-axis and central axial end forces \( N \).

In the unloaded state the beam is assumed to have initial curvature and initial torsion corresponding to the initial displacements \( u_1, v_1 \) and \( \varphi_1 \).

The additional displacements from the initial state are determined by the
following differential equations

\[ EI_x \frac{d^2 v}{dz^2} = -N(v + v_1) - M \]  \hspace{1cm} (2.01)

\[ EI_y \frac{d^2 u}{dz^2} = -N(u + u_1) - M\gamma(\varphi + \varphi_1) \]  \hspace{1cm} (2.02)

\[ (GI_v - N_i^2) \frac{d\varphi}{dz} = M \frac{d(u + u_1)}{dz} \]  \hspace{1cm} (2.03)

For the torsional expression a solid cross-section has been assumed and the notation

\[ \gamma = 1 - EI_y/(EI_x) \]  \hspace{1cm} (2.04)

introduced.

In the following the notation $GI_{ve}$ is used for the effective torsional rigidity

\[ GI_{ve} = GI_v - N_i^2 \]  \hspace{1cm} (2.05)

**2.2 Analytical solution**

The following expressions are assumed for the initial displacements

\[ u_i = u_0 \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.06)

\[ v_i = v_0 \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.07)

\[ \varphi_i = \varphi_0 \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.08)

where $u_0$, $v_0$ and $\varphi_0$ are constants.

A more general expression for $u_i$ and $v_i$ would be

\[ u_i = u_1 + u_0 \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.09)

\[ v_i = v_1 + v_0 \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.10)

where $u_1$ and $v_1$ are constants. A similar constant term in $\varphi_1$ is not compatible with the boundary conditions assumed. The constant terms complicate the calculations considerably, and since they will have no real influence on the results in this case they are disregarded.

When (2.06) - (2.08) are inserted the solution of (2.01) is

\[ v = \frac{M}{N} \left( \frac{\cos(\sqrt{\frac{N}{N_{Ex}}}) \frac{\pi z}{\ell} - 1}{\cos(\sqrt{\frac{N}{N_{Ex}}}) \frac{\pi}{2}} \right) + v_0 \frac{N}{N_{Ex} - N} \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.11)

where $N_{Ex}$ is the euler load corresponding to deflection in the Y-direction (bending about the X-axis)

\[ N_{Ex} = \left( \frac{\pi}{\ell} \right)^2 EI_x \]  \hspace{1cm} (2.12)

A good approximation to (2.11) is

\[ v = \frac{M + Nv_0}{N_{Ex} - N} \cos \frac{\pi z}{\ell} \]  \hspace{1cm} (2.13)

The solutions of (2.02) - (2.03) are

\[ u = \frac{\left[ \frac{N}{N_{Ey}} + \left( \frac{M}{M_{cr}} \right)^2 \right] u_0 + \varphi_0 \gamma \frac{M}{N_{Ey} \cos \frac{\pi z}{\ell}}}{1 - \frac{N}{N_{Ey} - \left( \frac{M}{M_{cr}} \right)^2} \left( \frac{M}{M_{cr}} \right)} \]  \hspace{1cm} (2.14)

\[ \varphi = \frac{N_{Ey} M}{\gamma M^2} \frac{u_0 + \varphi_0 \gamma \frac{N_{Ey}}{N_{Ey} - \left( \frac{M}{M_{cr}} \right)^2} \cos \frac{\pi z}{\ell}}{1 - \frac{N}{N_{Ey} - \left( \frac{M}{M_{cr}} \right)^2} \left( \frac{M}{M_{cr}} \right)} \]  \hspace{1cm} (2.15)

where $N_{Ey}$ is the euler load corresponding to deflection in the X-direction

\[ N_{Ey} = \left( \frac{\pi}{\ell} \right)^2 EI_y \]  \hspace{1cm} (2.16)

and $M_{cr}$ is the critical moment corresponding to lateral buckling with axial force

\[ M_{cr} = \sqrt{N_{Ey} GI_{we}/\gamma} \]  \hspace{1cm} (2.17)
2.3 General expressions for load-carrying capacity

The resulting bending moments about the X- and Y-axis, respectively, in the middle of the beam (z = 0), when the approximation (2.13) is used, will be

\[ M_x = N(v + v_0) + M = \frac{M + Nv_0}{1 - \frac{N}{N_{Ex}}} \]

\[ M_y = N(u + u_0) + M(\varphi + \varphi_0) \]

\[ u_0[N + \frac{N_{Ey}}{\gamma} (\frac{M}{M_{cr}})^2] + M\varphi_0[1 - \frac{N}{N_{Ey}}(1 - \gamma)] \]

\[ = \frac{1 - \frac{N}{N_{Ey}} - (\frac{M}{M_{cr}})^2}{f_b W_y(1 - \frac{N}{N_{Ey}} - (\frac{M}{M_{cr}})^2)} \leq 1 \]  

(2.22)

Furthermore, shear forces and a torsional moment occur, but they will not be determined since there is a total lack of rupture theories taking the corresponding stresses into consideration.

In the following the simplest possible rupture hypothesis is assumed, namely that rupture starts in the compression zone, and when the following condition is satisfied:

\[ \frac{\sigma_c}{f_c} + \frac{\sigma_b}{f_b} = 1 \]

(2.20)

where \( \sigma_c \) is the compression stress (\( \sigma_c = N/A \)), \( \sigma_b \) is the bending stress, \( f_c \) is the compression strength and \( f_b \) is the bending strength.

To most of the cross-sections used in practice the following expression applies

\[ \frac{M}{f_b W_x} + \frac{M\varphi_0}{f_b W_y} = \frac{M}{1 + \varphi_0 W_x/W_y} \leq 1 \]  

(2.23)

and not

\[ \frac{M}{f_b W_x} \leq 1 \]  

(2.24)

As the bending strength is normally determined by tests without any real lateral support \( f_b \) should be determined by (2.24) and not - as it is always done - by (2.25). To take this into consideration the \( f_b \)-value to be inserted in (2.22) must be higher than usual and corresponding to the factor \( 1 + \varphi_0 W_x/W_y \).
3. TESTS

3.1 Test material

Sawn redwood covering the normal Danish structural qualities (Unclassified, T200 and T300 with short-term characteristic bending strengths of 18 MPa, 24 MPa and 30 MPa, according to Danish Standard DS 413) with the cross-sections b x h = 38 x 100, 50 x 150 and 38 x 175 was used. The free lengths were 1560, 2640 and 3720 mm. For each cross-sectional dimension there were 3-6 individual tests. The total number of tests were 39.

Prior to the test the specimens were conditioned indoors at a relative humidity of 65%.

3.2 Test set-up

Compression and/or bending were imposed as shown in fig. 3.1.

The test specimen was supported at the ends by spherical bearings which are further described in relation to fig. 3.2.

At the top the bearing is secured to the loading frame, at the bottom it rests through a load cell on a hydraulic press. The horizontal load is imposed through long rods (R12) to ensure that the load direction is not changed essentially during the test. At the ends of the test specimen the horizontal reactions are transmitted through a knife bearing resting against the reaction bars (U-120) on a teflon bearing preventing forces from being transmitted perpendicular to the plane of the frame. The horizontal force is made by a hydraulic press and measured through a load cell. The joints at A are thus designed that the reaction bars can move freely in their longitudinal direction (roller bearings). At B only movements perpendicular to the frame were prevented.

The bearings have been developed on the basis of an idea of Massonnet, cf. [3] and [8]. Their principal construction is seen from fig. 3.2 showing the lower bearing. The parts A and B are in the contact area formed as part of a spherical surface. In the lower part a chamber, C, has been milled into which oil is pumped. The oil is pressed out along the contact area so that the upper part comes to rest on an oil film. The surplus oil is gathered in a groove with drainage, D, and pumped back into the chamber.

Figure 3.1. Vertical picture. Measurements in mm.
1) Test specimen. 2) Bearings. 3) Combined knife and teflon bearing. 4) Load cell. 5) Hydraulic press. 6) Load cell. 7) Hydraulic press. 8) Suspension in roller bearings.
Figure 3.2. Bearings.

The displacements were measured at the bearings and at the middle of the test specimens by electric displacement gauges. At each measuring point two transducers were used at the side of the timber and one at the edge.

3.3 Test procedure

At first the torsional rigidity of the test specimens, i.e. the constant GI, was determined.

The eccentricities of the test specimen and torsional angle at the middle were determined by plumb-lines, two at the side and one at the edge. The distances from the plumb lines to the timber were measured by a slide gauge at the abutments and at the middle of the test specimen.

In the set-up shown in fig. 3.1 the following tests were then carried out:

1. Pure bending about the weak axis.
2. Pure axial compression without lateral support.
3. Bending about the strong axis.
4. Combined axial compression and bending about the strong axis. The axial force was applied first and then the horizontal load was increased to failure.

For the longest test specimens, 3720 mm, however, the last-mentioned test was not carried out because such weak transversely loaded columns are not used in practice.

By each of the three first-mentioned tests it was aimed at applying the load in 10-20 steps of one minute's duration up to a load by which the timber would not be damaged. In cases where test No. 4 was not used, however, test No. 3 was carried on till rupture. By test No. 4 it was aimed at reaching failure load in 5-10 steps of 1 minute's duration.

After the test a prism about 500 mm long was cut off for determining density and moisture content.

3.4 Test results

The test results are given in table 3.1.

Columns 1-8 contain general information. UK denotes Unclassified. For cross-sectional dimensions only average figures are given from which the deviations were up to 1.5 mm; in the preparation of test results the actual measurements were used. Correspondingly, only mean values are given for the slenderness ratios l/i_x and l/i_y (i_x and i_y are the respective radii of gyration).

The moisture content is not given for the individual tests because the variation (from about 12% to about 15%) does not give rise to adjustment of the other measured quantities.

Column 9 gives the shear modulus G calculated from the measured torsional stress-strain curve. I_y = b^2 h (1 - 0.63 b/h)/3 is assumed.

Column 10 gives the modulus of elasticity in bending, E_y, determined by test No. 1, cf. section 3.3.

Columns 11 and 16 give the results of the compression test. By the rest related values of the load N and deformation u in the elastic area were measured. If u/N is plotted as abscissa and u as ordinate, cf. fig. 3.3, and if the initial deformations vary sinusoidally, the points will lie on a straight line the slope of which is the column load-carrying capacity N_E according to the euler formula. The length cut off the ordinate is the initial displacement in the middle, u_0. Normally the diagram is denoted a Southwell-plot. The modulus of elasticity, E_c, in compression is then determined as E_c = N_E y^2/(π^2 I_y).
Columns 13 and 19 contain the values determined by bending about the strong axis, viz. $\phi_0$ and the bending stiffness $E_{xx}$ and consequently, $E_{bx}$.

Columns 15-18 give the initial displacements determined from the plumb-line measurings and the compression and bending tests.

Columns 20-21 give the axial force chosen and the moment of failure ($M_{\text{meas}}$) in the final rupture tests.

3.5 Treatment of test results

Mean values, $m$, and standard deviation, $s$, for density and moduli of elasticity are given in table 3.2.

<table>
<thead>
<tr>
<th>Table 3.2</th>
<th>UK</th>
<th>T200</th>
<th>T300</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>m</td>
<td>s</td>
<td>m</td>
<td>s</td>
</tr>
<tr>
<td>$G$ MPa</td>
<td>728</td>
<td>58</td>
<td>748</td>
<td>54</td>
</tr>
<tr>
<td>$E_{by}$ MPa</td>
<td>10050</td>
<td>1330</td>
<td>11690</td>
<td>1920</td>
</tr>
<tr>
<td>$E_c$ MPa</td>
<td>8840</td>
<td>1120</td>
<td>10390</td>
<td>1670</td>
</tr>
<tr>
<td>$E_{bx}$ MPa</td>
<td>9480</td>
<td>1460</td>
<td>10780</td>
<td>2440</td>
</tr>
</tbody>
</table>

The $E_c/E_{by}$-ratio is averagely 0.895 with a standard deviation of 0.114. The ratio is significantly higher for the most slender columns (0.99). For comparison the ratio 0.933 with a standard deviation of 0.06 was found in [7].
The $E_{bx}/E_{by}$-ratio is averagely 0.932 with a standard deviation of 0.106. Also in this case the ratio is higher (1.05) for the most slender columns. Figure 3.4 shows the linear relation found between $E$ and $G$.

\[ G = 0.018E + 540 \]

\[ \times \text{ UK} \]
\[ \circ \text{ T200} \]
\[ \triangle \text{ T300} \]

Figure 3.4

In fig. 3.5 the derived initial displacements (numerical values) in relation to core radius in the relevant displacement direction have been drawn. There is very large scatter of the test results and there is no dependence on the quality. Since $u_0$ and $v_0$ are partly due to warping caused by shrinkage of the timber, it could be conjectured that they for values above a certain limit will increase by $(i/\ell)^2$. This is confirmed to a reasonable degree by the values found. Two straight lines are shown in the figure. One ($0.1 + 0.005 \ell/i$) originating from [6] has been used in various draft codes, among others [1] and [2], and the other ($0.0062\ell/i$) is the straight line without constant terms giving practically the same column load-carrying capacity.

For $\varphi$ no dependence on the length or quality has been found. However, dependence on the $b/h$-ratio has been found, cf. fig. 3.6, where the numerical values have been plotted. The expression

\[ \varphi_0 = 0.05 \frac{b}{h} \]  

(3.01)

can be assumed.

\[ \varphi_0 = 0.05 \frac{b}{h} \]

Figure 3.6

Apart from $\varphi_0$, which could be determined by a column test with displacement prevented in the Y-direction, the parameters of the expression (2.19) have now been determined for each test. On account of the negligible influence of $\varphi_0$ in the cases investigated the column test has been omitted and in the following the value found by the plumb-line measurements (table 3.1, col. 17) has been used.
$f_b/f_c = 1.1$ and $f_c = f_{by}/280$ have been assumed, cf. [6], and the expected rupture moments, $M_{cal}$, have been calculated by eq. (2.19). The relation between the calculated and measured moments is given in table 3.1, column 23. When test No. 2.3.3 is disregarded an average of $M_{cal}/M_{meas} = 0.960$ with a standard deviation of 0.148 is found. This is a very satisfactory agreement when it is taken into consideration that $f_b$ and $f_c$ have been indirectly determined.

4. DISCUSSION

4.1 Introduction

The tests have shown that the expressions derived can be used to determine the load-carrying capacity when the measured values for the initial displacements are inserted. It can then be assumed that they will also be applicable in setting up expressions for load-carrying capacities from codified or prescribed values. The choice of the initial displacements can be based on the permitted values for bow and twist according to the grading rules. However, these are in most cases very liberal (on these points) and it seems reasonable to use smaller values the observation of which is ensured by stricter requirements of grading or execution for the structures in which it might influence the load-carrying capacity.

As mentioned in relation to figure 3.6 the variation of the initial displacements should immediately be expected to increase with the length squared. Although the use of such variation would give certain advantages (among others the expressions for the load-carrying capacity will mainly be dependent on $\theta h/b^2$ only) the following expression has been assumed

$$\begin{align*}
\mathbf{u}_0 &= 0.0035 \ell \\
\mathbf{v}_0 &= 0.0062 \ell
\end{align*}$$

(4.01)

corresponding to the straight line 0.0062 $\ell/i$ in figure 3.6.

By choosing a straight line it is possible to get values which are reasonably on the safe side for short or medium-sleender structures (the influence of the eccentricities is greater for medium-sleender structures) without getting unreasonably high values for the slender ones.

4.2 Pure lateral buckling

Initially, among others to estimate the influence of the parameter $\varphi_0$, the case of pure lateral buckling is considered, i.e. $N = 0$. The ultimate value of the end moments taking lateral bending into consideration is denoted $M_{cr}$, while $M_0$ is the ultimate moment in pure bending

$$M_0 = f_b W$$

(4.02)
4.3 Combined loading

As shown in the above (2.22) with $\varphi_0=0$ can be used as a general expression. The use of the formula will be a little complicated for ordinary engineering practice, but it is easy to work out diagrams which will simplify the calculations very much.

An example is shown in fig. 4.2. The diagram gives $M/M_0$ dependent on $N/N_0$, the $h/b$-ratio and the slenderness ratio $k/l_x$ corresponding to the weak direction. $N_0$ is the axial force corresponding to the compression strength

$$N_0 = A f_c$$  \hspace{1cm} (4.04)

In fig. 4.1 $M_{cr}/M_0$ has been drawn dependent on $h/b^2$ for $h/b = 2$, $h/b = 5$ and for $\varphi_0 = 0$ and $\varphi_0$ equal to the value found by the tests, viz.

$$\varphi_0 = 0.05 \frac{b}{h}$$

cf. (3.01). $E/f_c = 280$ and $E/G = 16$ have been assumed.

For the case $\varphi_0 = 0$ a correction factor for the normal bending strength of

$$1 + \varphi_0 \frac{W_x}{W_y} = 1 + 0.05 = 1.05$$  \hspace{1cm} (4.03)

has been assumed, cf. the text below formula (2.25).

With the scatter of the properties of structural timber it is seen that a single common curve e.g. corresponding to $\varphi = 0$ and $h/b = 3$ can be used.

The assumption of $\varphi_0=0$ facilitates the calculations and furthermore, it is not necessary to distinguish between beams with lateral deflection prevented and non-restrained beams of a length where lateral buckling is not relevant.

Fig. 4.1 also shows $M_{cr}/M_0$ corresponding to the classical stability theory, formula (2.17), and the approximation given by Hooley & Madsen [4].

$$\frac{N}{N_{cr}} + \frac{M}{M_{0}} \leq 1$$  \hspace{1cm} (4.05)

$$M/M_{cr}(N=0) \leq 1$$
CONTENTS

For *marked chapters no draft is available so far. The others are included in FIRST DRAFTS
76.04.20 and January 1977 (CIB paper 6-100-2 and 7-100-1).

1. INTRODUCTION
   1.1 Scope
   1.2 Conditions for the validity of this document
   1.3 Units
   * 1.4 Notations
      Symbols to chosen in accordance with ISO 3898. As a basis for determination of the special timber symbols
      Symbols for use in structural timber design, CIB-Document CIB-W18-1, June 1976, is used. The final list
      cannot be worked out until the final editing
   * 1.5 Definitions
      To be worked out during the work with the code

2. BASIC ASSUMPTIONS
   2.1 Characteristic values
      Choice of percentile-values for strength calculations and for stiffness calculations
   2.2 Climate classes
   2.3 Load duration classes

3. BASIC DESIGN RULES
   * 3.0 General
      A draft has been prepared (1. draft, 3.0, 3.2 and 3.4). It will be necessary to await clarification of the general
      principles in ISO TC 98 before the special conditions valid for timber structures can be treated. This section
      also comprises requirements of deflections, mainly by reference to ISO ..........
   3.1 Design by calculation
      3.1.1 Basic assumptions
         Application of the theory of elasticity/theory of plasticity. Allowable simplifications
      3.1.2 Cross-sectional dimensions
         Acceptable deviations between true dimensions and those used in calculations
      * 3.1.3 Partial coefficients for materials
         To be determined on the basis of ISO TC 98 to the extent it is not a national or governmental matter
   3.2 Design by testing
      3.2.0 General
      * 3.2.1 Execution of tests
         An independent standard is on the RILEM/CIB work programme
      3.2.2 Determination of characteristic ultimate value
         Requirements of the statistical treatment of the test results
      * 3.2.3 Partial coefficients
         As 3.1.3

4. REQUIREMENTS OF MATERIALS
   4.1 Structural timber
      Structural timber is classified on the basis of the characteristic bending strength. The following classes are pro-
      posed for European softwood:
      T18   T24   T30
      Regarding hardwood the following classes could be proposed:
      T40   T50   T60
      It is left to the regional or national organizations to set up the grading rules. No differentiation is made between
      ordinary timber and finger-jointed timber.
4.2 Glulam

Glulam is classified as structural timber. The following classes are proposed for European softwood:

GL30    GL40    GL50

The classification can be extended when necessary.
It is left to the regional or national organizations to set up requirements of lamellas and cross-sections to obtain a given class

4.3 Plywood

It is demanded that the production must be subject to a recognized control arrangement and that strength parameters are determined on the basis of testing according to ISO .......

4.4 Other wood-based panels

As for plywood

4.5 Glue

4.6 Mechanical fasteners

Reference is made to section 5.3

5. DESIGN OF BASIC MEMBERS

5.1 Structural timber

* 5.1.0 Strength and values

For the classes mentioned in section 4.1 characteristic strength and stiffness parameters for strength and deformation calculations are given for the climate and load duration classes determined in section 2

5.1.1 Beams and columns

5.1.1.1 Pure tension

5.1.1.2 Pure compression without column effect

5.1.1.3 Pure bending

Including depth-factor and lateral instability

5.1.1.4 Shear

Including notch effect

5.1.1.5 Tension and bending

5.1.1.6 Compression and bending without column effect

5.1.1.7 Compression and bending with column effect

5.2 Glulam

5.2.1 Beams and columns

As 5.1.1

5.2.2 Curved members

Effect of bending of lamellas, distribution of bending stresses, tension perpendicular to the grain

5.2.3 Cambered beams straight or pitched

5.3 Joints

All load-carrying capacity expressions etc. are given in a general form where the parameters are inserted dependent on the values for timber and material of the fasteners

5.3.0 General

General requirements, determination of characteristic load-carrying capacities (reference to ISO .............. and ISO ..............), protection against corrosion

5.3.1 Nails

Laterally loaded nails (timber to timber, board materials to timber, steel to timber, withdrawal strength)

5.3.2 Bolts and dowels

5.3.3 Screws

5.3.4 Connectored joints

5.3.5 Glued joints

5.3.6 Construction rules

6. DESIGN OF COMPONENTS AND SPECIAL STRUCTURES

6.1 Glued components

6.1.1 Thin-webbed beams

6.1.2 Thin-flanged beams (stiffened plates)

6.1.3 I- and box columns, spaced columns, lattice columns

6.2 Mechanically jointed components

Subdivision as for section 6.1

6.3 Arches, portals and frames

6.4 Trusses
INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W13 - TIMBER STRUCTURES

CIB TIMBER CODE - LIST OF CONTENTS (SECOND DRAFT)

H J Larsen

Bruxelles
October 1977
7. EXECUTION

* 8. CONTROL
INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 – TIMBER STRUCTURES

POLISH STANDARD PN-73/B-03150: TIMBER STRUCTURES:
STATISTICAL CALCULATIONS AND DESIGNING

Bruxelles
October 1977
UNOFFICIAL TRANSLATION

POLISH STANDARD

TIMBER STRUCTURES
STATICAL CALCULATIONS AND DESIGNING

PN - 73/B - 03150
1. INTRODUCTION

1.1 Subject of the standard.

The subject of the standard are the rules of static calculations and timber structures designing.

1.2 Application range of the standard. The standard should be applied in all timber structures in building. It is not applied in bridge constructions.

1.3 Standards and connected documents.

- PN-74/B-02003 Loads in statical calculations. Permanent load.
- PN-74/B-02010 Loads in statical calculations. Snow loads.
- PN-74/B-02011 Loads in statical calculations. Wind loads.
- PN-74/B-03000 Building design, statical calculations.
- PN-76/B-03000 Steel structures. Statical and designing calculations.
- PN-75/B-01001 Sawn timber. Classification, terminology.
- PN-74/B-02002 Fibreboards. Terminology.
- PN-75/B-02003 Plywood. Classification and terminology.
- PN-57/B-96000 Softwood of general application.
- PN-72/B-96002 Hardwood of general application.
- PN-71/B-96003 Plywood of general application.
- PN-72/H-64002 Ordinary structural steel of general application. Types.
- PN-76/H-92103 Hoop iron without covering or galvanized.
- PN-75/H-92200 Wire rod and round hot rolled steel.

b) Dimensions

- PN-74/H-93200 Wire rod and round hot rolled steel.
- PN-71/H-82000 Nails. General requirements and research.
- PN-82010 Square washers in timber structures.
- PN-82011 Hexagon bolts.
- PN-71/H-82113 Square bolts.
- PN-72/H-82144 Hexagon nuts.
- PN-72/H-92151 Square nuts.
- PN-72/H-92201 Hexagon head screws.
- PN-71/H-92202 Square head screws.
- PN-71/H-92203 Hexagon head screws.
- PN-71/H-92204 Tensile head screws.
- PN-71/H-92205 Round head screws.
- PN-71/H-92209 Screws for timber. General requirements and research.


Instruction concerning complex protection of building timber against biological pests and fire - Institute of Building Technology, Warsaw 1969.

2. MATERIALS

2.1. Timber

2.1.1. Timber in structural elements should be softwood; pine or spruce. If necessary, it is possible to apply fir, larch and harwood: poplar or elder.

2.1.2. Inserts, pins, blocks and other small structural elements should be made of oak, acacia or other similarly hard and lasting kinds of timber.

2.1.3. Assortments, classification of sawn timber as regards its quality. Specifications - according to PN-75/B-01001, PN-57/B-96000 and PN-72/B-96002.

2.1.4. Minimum instantaneous strength of standard samples of pine wood and spruce wood should be smaller than in Table 1. If timber strength has been examined at the dryness 0%, different from 15%, so that 15°C/WC, then strength corresponding with dryness 15% is calculated according to the formula

\[ K_{15} = \left(1 - a_w[W-15]\right)K_w \]

where:

- \( K_{15} \) - strength of timber with dryness of 15%, kN/cm²
- \( K_w \) - strength of timber with dryness 0%, kN/cm²
- \( a_w \) - coefficient acc. to Table 2

Table 1: Minimum instantaneous strengths of pine or spruce

<table>
<thead>
<tr>
<th>Kind of strength</th>
<th>Symbol</th>
<th>Strength kN/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>( K_b )</td>
<td>900</td>
</tr>
<tr>
<td>Tension along the grain</td>
<td>( K_t )</td>
<td>550</td>
</tr>
<tr>
<td>Compression parallel to the grain</td>
<td>( K_c, K_d )</td>
<td>300</td>
</tr>
<tr>
<td>Shear along the grain</td>
<td>( K_s )</td>
<td>40</td>
</tr>
<tr>
<td>Compression across the grain</td>
<td>( K_f )</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 2: Coefficients \( a_w \)

<table>
<thead>
<tr>
<th>Kind of strength</th>
<th>Pine larch</th>
<th>Spruce oak</th>
<th>Spruce fir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression along the grain</td>
<td>0.05</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>Bending</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>Shear along the grain</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
</tr>
</tbody>
</table>
2.1.5. Dryness of timber

2.1.5.1. Permissible dryness of softwood applied in structural elements depends on where they are built in and on the kind of joints. Dryness should not exceed:

a/ for structures protected against moisture - 20%
b/ for structures in the open air - 23%
c/ for glued structures - 15% and it should comply with gluing technology

2.1.5.2. Dryness of hardwood applied for inertia, pine, blocks etc. should not exceed 15%

2.1.6. Coefficients of elasticity and deformation should be taken into account in the calculation of deformations according to Table 1.

Table 1. Coefficients of elasticity and deformation

<table>
<thead>
<tr>
<th>kind of timber</th>
<th>Coefficient of elasticity $E$, kG/cm²</th>
<th>Coefficient of deformation $G$, kG/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>parallel to the grain</td>
<td>perpendicular to the grain</td>
</tr>
<tr>
<td></td>
<td>bending tension compression</td>
<td>compression</td>
</tr>
<tr>
<td>pine, spruce</td>
<td>100000</td>
<td>3000</td>
</tr>
<tr>
<td>oak, acacia,</td>
<td>125000</td>
<td>6000</td>
</tr>
<tr>
<td>beech, birch</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

With structures exposed to humidity for a long time, coefficients $E$ and $G$ should be adopted with the corrector coefficient $0.6$. 

$E_{\uparrow, \downarrow}$
### Table 2.3: Properties of Plywood

<table>
<thead>
<tr>
<th>Number of Plywood Layers</th>
<th>Flexural Strength Along the Grain (MPa)</th>
<th>Flexural Strength Across the Grain (MPa)</th>
<th>Compressive Strength Along the Grain (MPa)</th>
<th>Compressive Strength Across the Grain (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>550</td>
<td>450</td>
<td>450</td>
<td>500</td>
</tr>
<tr>
<td>4</td>
<td>450</td>
<td>400</td>
<td>400</td>
<td>450</td>
</tr>
<tr>
<td>3</td>
<td>350</td>
<td>300</td>
<td>300</td>
<td>350</td>
</tr>
</tbody>
</table>

2.4.1 Connections

- 3.0
- 1070
- 990
- 200
- 200
- 20
- 11

### Table 2.4: Minimum Nominal Length of Lumber

<table>
<thead>
<tr>
<th>Nominal Length (mm)</th>
<th>Minimum Nominal Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2400</td>
<td>2400</td>
</tr>
<tr>
<td>3000</td>
<td>3000</td>
</tr>
<tr>
<td>4000</td>
<td>4000</td>
</tr>
<tr>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td>6000</td>
<td>6000</td>
</tr>
<tr>
<td>7000</td>
<td>7000</td>
</tr>
<tr>
<td>8000</td>
<td>8000</td>
</tr>
<tr>
<td>9000</td>
<td>9000</td>
</tr>
<tr>
<td>10000</td>
<td>10000</td>
</tr>
</tbody>
</table>

- **Note:** The dimensions and tolerances should be in accordance with the relevant building codes and standards.
2.1.2. Implanting materials for protection of timber derived materials protecting against biological corrosion and fire should be applied in compliance with the instruction of ITB/Institute of Building Technology/ - "Technical Instruction on surface protection of building timber" and "Instruction on complex protection of building timber against biological pests and fire". The protected places should be indicated on working figures and discussed in technical descriptions.

Fire preventive means, flame retardants should be applied in cases when a proper class of fire resistance of a structure is required or when it is necessary to obtain an unifiable material.

2.1.3. Materials protecting timber against chemical corrosion. Timber, plywood and fibrous boards should be protected against chemical corrosion, when timber structures are to be used in chemically aggressive environment. Corrosion preventive materials should have a certificate of permission for their application in building.

2.4.4. Protection of timber structures against statics should be in compliance with the Instruction 16 according to 1.3.

3. PROVISIONS FOR CALCULATIONS AND DESIGNING

3.1. General rules. Statical calculations should be performed according to PN-69/B-03000 and this standard.

Calculations based on other provisions than those given in the standard are permitted and it is possible to adopt different rules of designing provided that they are justifiable from the scientific and economic point of view.

3.2. Method of calculation. Statical calculations should be performed according to PN-64/B-03001, with the method of limit states.

There are two limit states:

a) Limit state as regards the structure damage exceeding the strength limit, the loss of stability under the design load.

b) Limit state as regards deformation or dislocation under the standard load without the overload coefficient.

3.3. Load distribution. In calculations for timber structures one should take into consideration the main and the additional loads according to PN-70/S-02010 and PN-70/S-02017. Structural loads should be discussed in their least favourable distribution during exploitation and at separate construction stages.

3.4. Categories of structural elements. Structural elements are assigned to one of the four categories (Table) according to a kind of work.

3.5. Weight of masses $p_w$ can be approximately determined in kN/m² by the formula

$$g_w = L$$

where: $L$ - span, m.

When the real weight of a structure differs from the assumed one by more than 10%, it is necessary to correct the calculations.
### Table 6. Categories of structural elements

<table>
<thead>
<tr>
<th>Structural Elements</th>
<th>Kind of elements work</th>
<th>Assortments</th>
<th>Recommended class (quality of timber)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Elements with mechanical connectors</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/ elements tensioned axially or eccentrically</td>
<td>boards, planks, square sawn timber</td>
<td>III</td>
<td></td>
</tr>
<tr>
<td>b/ elements tensioned in bent composite beams</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II. Tensioned elements</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensioned elements and the tensioned zone of laminated bent elements not smaller than 0.1% of the height of the cross-section from the tensioned edge with the height of above 54 cm and tensioned flanges of I - girders/Fig. 39 a, d, e/</td>
<td>boards, planks</td>
<td>III</td>
<td></td>
</tr>
<tr>
<td>II. Glued elements</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/ as above but with the possibility of applying calculation strength only up to 70%</td>
<td>boards, planks</td>
<td>IV</td>
<td></td>
</tr>
<tr>
<td>b/ compressed and tensioned zone with the height not smaller than 0.1% of the height of the cross-section from the extreme tensioned edge or 0.10 from the compressed edge of bent elements, compressed axially and eccentrically such as: compressed flanges, grate elements of lattice girders, arch girders, laminated beams with the height up to 50 cm, compressed zone of laminated beams with the height above 53 cm, flanges of I - beams with the chopping timber web /Fig. 39 b, c, d, f/ etc., with the use of the standard strength of timber above 70%</td>
<td>boards, planks</td>
<td>IV/V</td>
<td></td>
</tr>
<tr>
<td>B1 as b/ but with the possibility of using standard strengths only up to 70%</td>
<td>boards, planks</td>
<td>IV/V</td>
<td></td>
</tr>
<tr>
<td>C Central zone of the cross-section of laminated bent elements, compressed and eccentrically compressed and webs of I - beams of chop timber boards /Fig. 39 b, c, d, f/</td>
<td>boards, planks</td>
<td>V</td>
<td></td>
</tr>
</tbody>
</table>

1/ The quality of timber for load-bearing elements should be shown on figures and lists of materials. Timber defects in separate classes of quality are to be found in the appendix to the standard.
The standard strengths of timber are calculated in the elastic work of a material is assumed and taking into account flexibility of connections.

The standard strength of timber

The standard strengths \( K \) are defined according to the maximum instantaneous strength of timber with the formula

\[
K = \frac{\varepsilon_{\text{min}} k_d k_j}{3}
\]

where:
- \( \varepsilon_{\text{min}} \) - minimum instantaneous strength of pine or spruce according Table 1,
- \( k_d = 0.67 \) - corrector coefficient to the instantaneous strength determining the influence of the long duration of loading,
- \( k_j \) - coefficient of homogeneity according to Table 7.

The standard strength \( K \) for pine or spruce should be adopted according to Table 8 and Fig. 1.

![Fig. 1](image1)

![Fig. 2](image2)

**Table 7. Coefficients of homogeneity \( k_j \)**

<table>
<thead>
<tr>
<th>kind of strength</th>
<th>coefficient ( k_j )</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending</td>
<td>0.40</td>
</tr>
<tr>
<td>tension along the grain</td>
<td>0.27</td>
</tr>
<tr>
<td>compression and pressure along the grain</td>
<td>0.55</td>
</tr>
<tr>
<td>compression and pressure across the grain</td>
<td>0.90</td>
</tr>
<tr>
<td>shear along the grain and at the acute angle to the grain</td>
<td>0.70</td>
</tr>
</tbody>
</table>

**Table 8. Standard strengths of pine and spruce**

<table>
<thead>
<tr>
<th>kind of strength</th>
<th>strength</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>bending: a/elements of solid timber/except p.b./: 130</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>b/elements of timber with a side ( 14 ) cm and height ( h ) to 50 cm: 150</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>c/ glued elements regardless of height but with the side ( 14 ) cm: 150</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>d/ round timber, not weakened on edges with indentations, in a section under consideration: 150</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>2 Tension along the grain ( K_f ) 100</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>3 Tension across the grain ( K_f ) 4</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>4 Compression and pressure along the grain ( K_c ) 130</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>5 Compression and pressure on the whole surface across the grain ( K_d ) 16</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>6 Pressure across the grain: a/ in support planes of structures: 24</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>b/ on part of the surface, if there is any left - in the direction of the grain - free ends whose length is not smaller than the height of the pressed element and the length of the pressed surface (Fig. 4a, b) in frontal cuts and blocks and under bolt washers with the pressure at the angle ( \alpha = 90^\circ ) 30</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>7 Pressure at the acute angle to the grain ( K_{a4} ) acc. to Table 9</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>8 shear along the grain: a/ with bending: ( K_t ) maximum (a) 24</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>b/ in connections with frontal cuts and blocks: ( K_t ) maximum (b) 24</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>9 shear along the grain: acc. to Table 9 (a) 12</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>
In temporary structures standard strengths can be increased by 20%.
In countrybuilding of temporary character for carpenter structures with the span to 10.5 m with mechanical connections/calls, pine, Douglas plates etc. it is possible to apply worse classes of timber without increasing the strength if III/IV instead of III, IV/V instead of IV. Incipient decay is permitted only on the surfaces of the class V.

3.8.2 Particular cases of standard strength

3.8.2.1 Standard strength $K_{d\alpha}$ with the pressure as the acute angle to the grain is defined with the formula

$$K_{d\alpha} = \frac{K_0}{1 + \left(\frac{\alpha_2}{\eta_{\alpha}} - 1\right)\sin^2\alpha}$$

where $K_0$ should be replaced with one of the four values of the pressure according to Table 8 nr 5 and 6

Values $K_{d\alpha}$ calculated for pine and spruce according to the formula 4/ are shown in Table 9.

### Table 9. Standard strength $K_{d\alpha}$ of the pressure at the angle $\alpha$ to the direction of the grain for pine and spruce

<table>
<thead>
<tr>
<th>Angle $\alpha^\circ$</th>
<th>Strength in relation to a respective point of Table 8, kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_{d\alpha}$ (nr. 5)</td>
</tr>
<tr>
<td>---------------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>0</td>
<td>130,0</td>
</tr>
<tr>
<td>5</td>
<td>150,0</td>
</tr>
<tr>
<td>10</td>
<td>175,6</td>
</tr>
<tr>
<td>15</td>
<td>185,0</td>
</tr>
<tr>
<td>20</td>
<td>204,0</td>
</tr>
<tr>
<td>25</td>
<td>225,0</td>
</tr>
<tr>
<td>30</td>
<td>245,0</td>
</tr>
<tr>
<td>35</td>
<td>265,0</td>
</tr>
</tbody>
</table>

Table 10. Standard strength $k_{\alpha}^{max}$ of the shear at the angle $\alpha$ to the direction of the grain, for pine and spruce

<table>
<thead>
<tr>
<th>Angle $\alpha^\circ$</th>
<th>$k_{\alpha}^{max}$ kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>36</td>
</tr>
<tr>
<td>10</td>
<td>37</td>
</tr>
<tr>
<td>15</td>
<td>38</td>
</tr>
<tr>
<td>20</td>
<td>39</td>
</tr>
<tr>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>30</td>
<td>41</td>
</tr>
<tr>
<td>35</td>
<td>42</td>
</tr>
</tbody>
</table>

3.8.2.2 Standard strength $K_{\alpha \alpha}$ with the shear as the acute angle to the direction of the grain is defined with the formula

$$K_{\alpha \alpha} = \frac{k_{\alpha \alpha}}{1 + \left(\frac{\eta_{\alpha \alpha}}{k_{\alpha \alpha}} - 1\right)\sin^2\alpha}$$

where $K_{\alpha \alpha}$, $k_{\alpha \alpha}$ - standard strengths with shear according to Table 8.

Values $K_{\alpha \alpha}$ calculated for pine and spruce according to formula 5/ are shown in Table 10.

Table 10. Standard strength $k_{\alpha}^{max}$ of the shear at the angle $\alpha$ to the direction of the grain, for pine and spruce

<table>
<thead>
<tr>
<th>Angle $\alpha^\circ$</th>
<th>$k_{\alpha}^{max}$ kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>36</td>
</tr>
<tr>
<td>10</td>
<td>37</td>
</tr>
<tr>
<td>15</td>
<td>38</td>
</tr>
<tr>
<td>20</td>
<td>39</td>
</tr>
<tr>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>30</td>
<td>41</td>
</tr>
<tr>
<td>35</td>
<td>42</td>
</tr>
</tbody>
</table>
3.9 Standard strength of plywood

3.9.1. Types of standard strengths of water resistant plywood are shown in Table 12.

Table 12. Standard strengths of softwood; water resistant plywood

<table>
<thead>
<tr>
<th>Kind of strength</th>
<th>Eym.</th>
<th>Strength, kN/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear in the board plane</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear perpendicular to the board plane</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.9.2 Standard strength/resistance to compression and tension at an angle.

With a force applied at an angle $\alpha$ to the direction of the grain of plywood facing boards, the standard resistance/strength to compression and tension should have values:

a/ for $\alpha = 0^\circ$, $K = 100$ kN/cm²,

b/ for $\alpha = 30^\circ$, $K = 25$ kN/cm² (permanent value for this range),

c/ for $\alpha = 90^\circ$, $K = 50$ kN/cm²

Intermediate values of strength for angles $\alpha = 30^\circ$ and $\alpha = 60^\circ \div 90^\circ$ should be interpolated linearly.

3.10 Standard strength of fibres boards

3.10.1 Kinds of standard strengths of hard and very hard fibreboards according to EN-74/712-11, 21 and 22 are shown in Table 13.

Table 13. Standard strengths K for fibreboards/for boards acc. Table 12

<table>
<thead>
<tr>
<th>Kind of strength</th>
<th>Eym.</th>
<th>Strengths of fibreboards, kN/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td></td>
<td>hard</td>
</tr>
<tr>
<td>Bending</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>Shear in the board plane</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Shear perpendicular to the board plane</td>
<td></td>
<td>20</td>
</tr>
</tbody>
</table>
The strength of timber is determined by multiplying standard strengths by the correction coefficient m:

\[ m = m_1 \cdot m_2 \cdot m_3 \cdot m_4 \]

where:
- \( m_1 \) - coefficient taking into account the conditions of work of a structure under the load of short duration (Table 14/);
- \( m_2 \) - coefficient taking into account the conditions of a structure usage;
- \( m_3 \) - coefficient taking into account the kind of timber in relation to pine and spruce (Table 16/);
- \( m_4 \) - coefficient taking into account a previous flexion of an element (Table 17/).

When it is difficult to select a proper section, one is permitted to exceed calculation strengths but by no more than 20%.

3.12. The way of expressing values in units of measurement of the SI system while expressing values in units of measurement of the SI system the following relations between units of measurement should be applied:

1 kg = 9.80665/\text{exactly}/
1 kg/cm\text{\textsuperscript{2}} = 98.0665 kN/m\text{\textsuperscript{2}}/\text{exactly}/
1 kg/cm\text{\textsuperscript{3}} = 9,80665 MN/m\text{\textsuperscript{3}}/\text{exactly}/

Approximate relations are permitted:
1 kg = about 10 N or about 1 daN
160 cm\text{\textsuperscript{2}} = about 1600 cm\text{\textsuperscript{2}} or about 0.16 m\text{\textsuperscript{2}}
1 kg/cm\text{\textsuperscript{3}} = about 0.1 N/m\text{\textsuperscript{3}} or about 100 kN/m\text{\textsuperscript{3}}.

4. DESIGNING
4.1. Maximum ambient temperature, in which it is possible to apply timber structures should not exceed 55\(^\circ\)C.

4.2. The smallest not open section of the solid wood-water circuit of permanent load-bearing structures with the exception of roof baffles should not be smaller than 40cm\text{\textsuperscript{2}}, with thickness not smaller than 30mm. In timber structures with nail connectors or bolts the surface of the section should not be smaller than 140cm\text{\textsuperscript{2}} and thickness of the rod not smaller than 22mm.

### Table 15. Coefficients \( m_2 \) of the utilisation conditions of the structure in different conditions of utilisation.

<table>
<thead>
<tr>
<th>conditions of utilisation</th>
<th>coefficient ( m_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>structures protected against precipitation</td>
</tr>
<tr>
<td>2</td>
<td>structures exposed to humidity for a short time, then dried in the open air, in factories etc.</td>
</tr>
<tr>
<td>3</td>
<td>structures exposed to humidity for a long time in water, in the ground, in factories etc.</td>
</tr>
<tr>
<td>4</td>
<td>structures exposed to aggressive chemical conditions: with higher aggressiveness</td>
</tr>
<tr>
<td>5</td>
<td>structures exposed to higher temperature within the range 40-55(^\circ)C</td>
</tr>
<tr>
<td>6</td>
<td>structures under permanent loading</td>
</tr>
<tr>
<td>7</td>
<td>Boarding to reinforced concrete except shores</td>
</tr>
</tbody>
</table>

1/ Coefficient \( m_2 \) may be in particular a product of values set up in table 15.
2/ the coefficient of the permanent loading influence is taken into account when permanent loading is higher than 60% of the total loading. Snow and wind are loads of short duration.

### Table 16. Corrector coefficients \( m_3 \) taking into account a kind of timber in relation to pine or spruce.

<table>
<thead>
<tr>
<th>kind of timber</th>
<th>coefficients ( m_3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>tension</td>
<td>compression</td>
</tr>
<tr>
<td>larch</td>
<td>1,2</td>
</tr>
<tr>
<td>fir</td>
<td>0,8</td>
</tr>
<tr>
<td>oak</td>
<td>1,3</td>
</tr>
<tr>
<td>pine</td>
<td>1,5</td>
</tr>
<tr>
<td>beech</td>
<td>1,1</td>
</tr>
</tbody>
</table>
In compressed members and in the compressed zone of bent elements it is possible to disregard weakening, if a weakened place is tightly filled with timber or another material with coefficients of elasticity not smaller than for timber in the direction parallel to the grain.

4.5. System of truss members in truss girders.
Axes of truss members in truss girders should intersect in one point of the geometrical truss of a girder. It is possible to deviate from these requirements in nailed trusses or in ridge joints of other trusses. In calculations one should take into account the influence of the eccentric connection of truss members with flanges, if the intersection point projects beyond the edge of the flange (Fig. 3).

4.6. Tensioned truss members.

4.6.1. Stresses in axially tensioned truss members should be defined by the formula:

\[ F = \frac{P}{A} \]

where:
\[ P \] - calculation axial force, kg
\[ A \] - net cross-section, cm²
\[ \sigma_n \] - standard strength at tension = 240 to 300.

4.6.2. Plates for tensioned contacts should be designed for the design/calculation load, they will bear, increased by 30%.

---

### Table 13: Depth of the undercut on a support

<table>
<thead>
<tr>
<th>Dependence of dimensions of the undercut</th>
<th>Depth of the undercut</th>
<th>Length of the undercut</th>
</tr>
</thead>
<tbody>
<tr>
<td>on shear stresses</td>
<td>a &gt; 0.7h</td>
<td>a &lt; 0.3h</td>
</tr>
<tr>
<td></td>
<td>a &lt; 0.3h</td>
<td>a &lt; 0.25h</td>
</tr>
<tr>
<td></td>
<td>a &lt; 0.3h</td>
<td>a &lt; 0.5h</td>
</tr>
<tr>
<td>on height</td>
<td>h &gt; 18 cm</td>
<td>a &lt; 0.2h</td>
</tr>
<tr>
<td></td>
<td>18 &lt; h &lt; 12 cm</td>
<td>a &lt; 0.4h</td>
</tr>
<tr>
<td></td>
<td>h &lt; 12 cm</td>
<td>a &lt; 0.5h</td>
</tr>
</tbody>
</table>

Q - lateral force on the support. The smaller value a should be adopted as depth of the undercut.
4.7,6 Calculation of bending stresses in a composite beam

4.7,6.1 Bending edge stress should not be bigger than \( \sigma_e \), acc. to Table 4, and the tensioning stress, calculated in the neutral axis of a component member of a composite beam placed in the tensioning zone should not be bigger than \( \sigma_{tp} \) acc. to Table 8.

4.7,6.2 Calculation of bending stresses in the composite beam, eq. 4.7,6,7/4,9

With the full web, connections with nibs or inserts is performed according to formula:

\[
\sigma_e = \frac{1}{2} \tan \theta \left( \frac{M_b}{J_{tp}} \right) \leq \sigma_{tp} \tag{4.7,6,7}
\]

\[
\sigma_{tp} = \frac{M}{J_{tp}} \left( \frac{r_e}{r_{in}} + \frac{h_j}{2r_{in}} \right) \leq \sigma_{tp} \tag{4.7,6,8}
\]

\[
\sigma_{tp} = \frac{M}{J_{tp}} \left( \frac{r_e}{r_{in}} \right) \leq \sigma_{tp} \tag{4.7,6,9}
\]

\[
J_{tp} = \frac{\sum J_i + 2bF_e e_i^2}{2} \tag{4.7,6,10}
\]

\[
\sigma_{tp} = \left( 0.12 + 0.025 \frac{L}{b} - 0.0023 \left( \frac{L}{b} \right)^{2.5} \right) \tag{4.7,6,11}
\]

where:

- \( N \) - bending moment, kN
- \( \sigma_e \) - bending edge stress of the web, kN/cm²
- \( \sigma_{tp} \) - edge stress of the flanges, composite beam number IV/Pic.5/7, kN/cm²
- \( \sigma_{tp} \) - bending stress in the neutral axis of the web, kN/cm²
- \( h_t \) - total beam height, cm
- \( h_d \) - web height, cm
- \( b_f \) - flange thickness, cm
- \( e \) - distance between neutral axes of the weakened beam section and the weakened flange section, cm
- \( L \) - beam span, cm; for continuous beams in the formula 15 a respective span with the coefficient 0.8 is adopted
- \( \sigma \) - reduction coefficient
- \( k \) - coefficient depending on the kind of connectors; for nails \( k = 1.0 \), for inserts with the bearing capacity 2500 kN = \( k = 2.45 \)
- \( J_d \), \( J_{dn} \) - moments of inertia of the web area, not calculated in relation to the neutral axis of the whole beam section, cm⁴.
4.7.7 Calculation of shearing forces: The shearing force per unit length depends on the support condition. The general equation for the shearing force is given by:

\[ f = \frac{Q}{L} \]

where:
- \( f \) is the shearing force per unit length,
- \( Q \) is the load at the support,
- \( L \) is the length of the beam.

For a simply supported beam, the maximum shearing force occurs at the supports and is given by:

\[ f_{max} = \frac{Q}{2} \]

where:
- \( f_{max} \) is the maximum shearing force,
- \( Q \) is the load at the support.

4.7.7.1 Checking of shearing stresses. The shearing stress is checked in the neutral axis of the section. For sections I - IV/Fig. 5/ it is calculated in the axis plane x = x acc. to the formula:

\[ \tau = \frac{Q}{J_{bp}} \left( S_x + S_y \right) \leq K_{\tau} \]

where:
- \( \tau \) is the shearing stress,
- \( Q \) is the shearing force,
- \( J_{bp} \) is the moment of inertia of the beam section,
- \( S_x \) and \( S_y \) are the bending moments about the x and y axes respectively,
- \( K_{\tau} \) is the shearing stress limit.

4.7.7.2 Calculation of shear forces on the composite beam. The shear forces on the composite beam are calculated according to the formula:

\[ Q_{br} = \frac{Q}{L} \]

where:
- \( Q_{br} \) is the shear force on the composite beam,
- \( Q \) is the load at the support,
- \( L \) is the length of the beam.

4.7.7.3 Calculation of bending stresses of composite beams. The stresses are calculated according to the formula:

\[ \sigma = \frac{M}{I_{bp}} \]

where:
- \( \sigma \) is the bending stress,
- \( M \) is the bending moment,
- \( I_{bp} \) is the moment of inertia of the beam section.

4.7.8.1 Checking of bending stresses. The bending stress is checked in the neutral axis of the section. For sections I - IV/Fig. 5/ it is calculated in the axis plane x = x acc. to the formula:

\[ \sigma = \frac{M}{I_{bp}} \leq K_{\sigma} \]

where:
- \( \sigma \) is the bending stress,
- \( M \) is the bending moment,
- \( I_{bp} \) is the moment of inertia of the beam section,
- \( K_{\sigma} \) is the bending stress limit.

4.7.8.2 Calculation of shear forces on the composite beam. The shear forces on the composite beam are calculated according to the formula:

\[ Q_{br} = \frac{Q}{L} \]

where:
- \( Q_{br} \) is the shear force on the composite beam,
- \( Q \) is the load at the support,
- \( L \) is the length of the beam.

In formulae 17 - 19:
- \( Q \) is the lateral force in the support
- \( Q_x \) is the lateral variable force on the beam length, kg
- \( Q_{br} \) is the lateral medium force within the range \( a_i - b_i \), kg
- \( S_{br} \) is the static gross moment of the part of the section beyond the beam for a flange or a beam/Fig. 5/ in relation to the neutral axis of the section, cm²,
- \( J_{br} \) is the moment of inertia of the whole section, gross, cm²
- \( t_{bi} \) is the shearing force, variable on the beam length, kg/cm,
- \( t_{bi} \) is the constant shearing force ab, kg/cm.
4.7.7.1. Number of connectors. According to formula 25, the number of connectors is calculated as:

\[ n = \frac{t_n}{\tau} \]

4/ with regular spacing of connectors

\[ n = \frac{I^2}{2} \int \frac{t_n}{\tau} dx \]

5/ with the same loading of connectors on the whole beam span L

where:
- \( t_n \) - bearing capacity of a connector
- \( \tau \) - single shear, N/mm²

4.7.8. Nailed girders. In calculations of nailed girders of I-section with the web of crossing boards or of box section/board walls, the following rules should be followed:

- with the calculation of the section moment of inertia, one does not take into account the web section or of walls,
- one does not take into account the Co-operation of the web boards or wall boards in the transfer of loading forces,
- the web and walls are dimensioned taking into account lateral forces - formula 18/1,
- if under flanges have a composite section/Fig.6/ in calculations acc. formula 15/1 one should use, for the first section, closest to the neutral axis, to the coefficient \( y \) - factor \( n = 1,5 \), for the second - factor \( y \frac{0.5}{2} \), one should not design a flange section composed of more than two elements in relation to each axes/Fig.7/
- if \( t \) - beams with the web of crossing boards are composed of separately made parts/Fig.7/, additional connectors should be calculated for the vertical forces in the connection.

4.7.9. Connectors. Contact tips, plates in the contact acting should have the moment of inertia at least equal to the moment of the bent bar elements and should transfer lateral forces in the contact.

4.7.10. Reflection of beams and truss girders

4.7.10.1. Reflection of beams and truss girders with flat bending. Deflection should be calculated in compliance with structural mechanics. In the calculation of composite beam deflection, except glued beams, to the deflection formula one should apply moment of inertia \( J_\text{cp} \) according to formula 15/. Table 19 shows permissible deflections. Deflection calculations are performed from standard loads/ without overload coefficients/.

### Table 19 Permissible deflections \( f_d \) of timber structures

<table>
<thead>
<tr>
<th>Loading</th>
<th>Deflection ( f_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>with strut</td>
<td>without strut</td>
</tr>
<tr>
<td>form.</td>
<td>var.</td>
</tr>
<tr>
<td>perm.</td>
<td>perm.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>kind of structure</th>
<th>Loading</th>
<th>Deflection ( f_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>with strut form.</td>
<td>without strut form.</td>
<td></td>
</tr>
<tr>
<td>form.</td>
<td>var.</td>
<td>form.</td>
</tr>
<tr>
<td>perm.</td>
<td>perm.</td>
<td>var.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I-girders with a full web, box girders, composite beams</td>
<td>1/3</td>
<td>2/3</td>
<td>1/300</td>
<td>1/200</td>
</tr>
<tr>
<td>2</td>
<td>truss girders</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>approx. cal.</td>
<td>1/500</td>
<td>1/200</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>exact cal.</td>
<td>1/300</td>
<td>1/200</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>rafters, purlins etc.</td>
<td>-</td>
<td>-</td>
<td>1/200</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>panel roof elements in the period of exploitation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- over 15 yrs</td>
<td>-</td>
<td>-</td>
<td>1/250</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- up to 15 yrs</td>
<td>-</td>
<td>-</td>
<td>1/150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>boarding, cladding</td>
<td>-</td>
<td>-</td>
<td>1/150</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>beams of an unplastered floor under rooms with variable loading</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/</td>
<td>50 kg/cm²</td>
<td>-</td>
<td>-</td>
<td>1/200</td>
<td></td>
</tr>
<tr>
<td>b/</td>
<td>50 kg/cm²</td>
<td>-</td>
<td>-</td>
<td>1/250</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>beams of plastered floors under rooms with var. load</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/</td>
<td>50 kg/cm²</td>
<td>-</td>
<td>-</td>
<td>1/250</td>
<td></td>
</tr>
<tr>
<td>b/</td>
<td>50 kg/cm²</td>
<td>-</td>
<td>-</td>
<td>1/300</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>boarding of reinforced concrete structs.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/</td>
<td>unplastered</td>
<td>-</td>
<td>-</td>
<td>1/200</td>
<td></td>
</tr>
<tr>
<td>b/</td>
<td>plastered</td>
<td>-</td>
<td>-</td>
<td>1/250</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>elements of enclosing walls</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a/</td>
<td>industrial buildings, tourist buildings</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- exp. over 15 yrs</td>
<td>-</td>
<td>-</td>
<td>1/200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- exp. up to 15 yrs</td>
<td>-</td>
<td>-</td>
<td>1/100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b/</td>
<td>dwelling, office build.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- exp. over 15 yrs</td>
<td>-</td>
<td>-</td>
<td>1/200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.7.10.2. Deflection of beams with oblique bending

Deflection of beams, bent obliquely, should be calculated according to the formula:

$$ f = \sqrt{f_x^2 + f_y^2} / 22 $$

where:
- $$ f_x, f_y $$ - component deflections in the direction of axes, respectively x-x and y-y.

4.7.10.3. The biggest permissible deflection of a timber structure calculated for the full load should not exceed values shown in Table 19. In old renovated buildings, one permits deflection exceeding by 50% the calculated permissible deflection, assuming the elastic work of timber.

4.7.10.4. Deflection of continuous beams. In continuous beams with equal spans or when the ratio of the biggest span to the smallest one does not exceed 1:0,8 and with the same loading of all spans or when with equal spans, the ratio of the highest to the smallest load does not exceed 1:0,8, deflection can be approximately defined adopting the ratio of the biggest deflection of a continuous beam to the biggest deflection of a beam span/single-span freely supported.

0,65 - with permanent load
0,90 - with variable load
0,70 - for central spans
0,75 - with permanent load
0,75 - with variable load

4.7.10.5. Structural deflection for special cases.

When a kind of a building requires smaller deflection than mentioned in Table 19, one has to face stricter requirements.

4.7.11. Structural flexion in the opposite way/structural flexion should be:

a/ for composite beams with block connections:

$$ f_w \geq 1,5 f_u $$

t/ for girders with a full wall/ I - girders, box girders/ for truss girders/ without a suspended floor:

$$ f_w \geq 1/200 L $$

c/ for glued structures:

$$ f_w \geq f_u $$

should not exceed values form table 20.

Table 20. Permissible slenderness ratio for compressed members

<table>
<thead>
<tr>
<th>members</th>
<th>slenderness ratio $\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>perman. struc.</td>
</tr>
<tr>
<td>1 load-bearing</td>
<td>120</td>
</tr>
<tr>
<td>column</td>
<td></td>
</tr>
<tr>
<td>2 solid bars</td>
<td>150</td>
</tr>
<tr>
<td>3 composite</td>
<td>175</td>
</tr>
<tr>
<td>bars on elastic</td>
<td></td>
</tr>
<tr>
<td>connectors</td>
<td></td>
</tr>
<tr>
<td>4 secondary</td>
<td></td>
</tr>
<tr>
<td>members such as</td>
<td></td>
</tr>
<tr>
<td>wind braces,</td>
<td></td>
</tr>
<tr>
<td>braces. tensioned</td>
<td></td>
</tr>
<tr>
<td>bars that can be</td>
<td></td>
</tr>
<tr>
<td>acted on by small</td>
<td></td>
</tr>
<tr>
<td>compression force</td>
<td></td>
</tr>
<tr>
<td>ces resulting from</td>
<td></td>
</tr>
<tr>
<td>additional loads</td>
<td>200</td>
</tr>
</tbody>
</table>

While calculating slenderness ratio in mining works, for round sections one should take into account the radius of inertia for the smallest section. In other cases the radius of inertia in the middle of the buckling length of a member's adopted.

4.8.2. Buckling length $L$ of axially compressed members is defined by the formula:

$$ I_x = \gamma x \text{ or } I_y = \gamma y $$

where:
- $\gamma$ - coefficient of the buckling length acc. to 4.8.3.
- $L_{xy}$ - lengths of a member measured in the planes of main axes and equal to distances between the brace axes.

In trusses lengths of truss members $L$ should be adopted as equal to the theoretical $I_{max}$ lengths of the truss.
4.6.3. Coefficients of buckling length.

Coefficients of buckling length \( \gamma \) should be:

- a/ for a member fixed at one end, with the other end free - see Fig. 8a/ \( \gamma = 2 \)
- b/ for a member supported at both ends in a jointed way/Fig. 8b/ and in end spans of a continuous member/Fig. 8c/ with the impossibility to make lateral movements - for both cases \( \gamma = 1,0 \)
- c/ for a member with one end fixed rigidly and the other end supported in a jointed way or in a similar way/Fig. 8d/ in the main plane in which buckling is calculated with the impossibility of lateral movements - for both cases \( \gamma = 0,8 \)
- d/ for truss members:
  - for truss flanges in buckling cases in the truss plane \( \gamma = 0,8 \)
  - for the truss plane/Fig. 8a/ \( \gamma = 1,0 \)
  - for stanchions and cross braces of the truss in the truss plane \( \gamma = 0,8 \)

When these members are connected with flanges with gains, inserts with one bolt/br: pins \( \gamma = 1,0 \)
- for crossing truss members/Fig. 8e/ - acc. to Table 1.
- for compressed truss flanges or transoms of frames braced with purlins, braces etc. where buckling out of the plane \( \gamma = 1,0 \)

| Table 21. Coefficients of buckling length \( \gamma \) for crossing truss members |
|-----------------|------------------|------------------|
| **members**     | **coefficients \( \gamma \)** | **in the truss plane** | **out of the truss plane** |
| crossing members of the truss acc. to Fig. 8a/ | if the absolute value of force in the supporting member is bigger or equal to the force in the compressed member | 0,60 | 0,50 |
|                 | if the force in the supporting member is smaller than the force in the compressed member | 0,80 | |

Fig. 8

4.6.4 Buckling of arches

4.6.4.1 Buckling length \( l \) in the arch plane is defined by the formula

\[
l = \gamma S / 25/
\]

where:

- \( \gamma \) = reduction coefficient acc. to Table 22,
- \( S \) = arch length/opened/along the axis

Table 22. Coefficients of buckling length \( \gamma \) for an arch

<table>
<thead>
<tr>
<th>kind of arch</th>
<th>symmetric load</th>
<th>one-sided load</th>
</tr>
</thead>
<tbody>
<tr>
<td>three-hinged arch</td>
<td>0,7</td>
<td>0,5</td>
</tr>
<tr>
<td>two-hinged arch</td>
<td>0,6</td>
<td>0,5</td>
</tr>
</tbody>
</table>
4.4.2. Buckling length \( l \) for the arch plane

\( a) \) For the top compressed flange the length \( l \) is equal to the distance between purlines.

\( b) \) For the bottom compressed flange the length \( l \) is calculated according to the formula:

\[
l = \gamma L
\]

where:

- \( L \) = length of the chord between lateral bracings
- \( \gamma \) = corrector coefficient depending on the kind of lateral bracings, that is:
  - with bracings of true girders: \( \gamma = 1.0 \)
  - with bracings of angle braces with one end supported on a girder, and the other on a purlin: \( \gamma = 1.25 \)

4.4.3. Buckling length of column and trusses.

Buckling length of columns and trusses of frames and pseudo-frames is defined in the following way:

\( a) \) If a true girder or a full girder is supported on columns and connected with a column with hinges /Fig. 9a/, the buckling length of the column with buckling in the pseudo-frame is \( l = 2h \). With buckling of the pseudo-frame plane \( l \) is the distance between bracings at the column height.

\( b) \) If a true girder is supported on columns and is connected with a column with an angle strut bracing the angle of the pseudo-frame /Fig. 9b/, then with buckling of the column in the plane of the arrangement, the buckling length may be determined by the approximate formula:

\[
l = 2h \gamma + 0.7h
\]

\( c) \) The buckling length of symmetric columns of two- or three-hinged frame, with buckling in the frame plane may be defined by the approximate formula /Fig. 9c/:

\[
l = \frac{b h^2}{3} + 1.6n
\]

where:

- \( n \) = coefficient: \( n = \frac{2j_M}{j_F} \)
  - \( j_M \) = moment of inertia of the column, \( \text{cm}^4 \)
  - \( j_F \) = moment of inertia of the transom, \( \text{cm}^4 \)

4.4.6. Sections of solid axially compressed members should be checked with regard to buckling according to the formula:

\[
d = \frac{P}{F_p} \leq \frac{1}{\sqrt{\beta F_0}}
\]

where:

- \( P \) = design load /calculating load /
- \( F_p \) = field of the working cross-section
- \( F \) = \( P_n \) if weakenings disturb the edge of a member, \( \text{cm}^2 \)
- \( F \) = \( P_{br} \) if weakenings do not disturb the edge of a member and are not bigger than 25% of the gross cross-section, \( \text{cm}^2 \)
- \( P = 4/3 P_n \) if weakenings do not disturb the edge of a member but are bigger than 25% of the gross cross-section, \( \text{cm}^2 \)
- \( \beta \) = coefficient of buckling acc. to 4.4.6.7.
- \( F_0 \) = standard strength acc. to Table 8.
4.2.7. Coefficients of buckling $\beta$ are calculated acc. to formula:

a/ for ratio of slenderness $0 \leq \lambda \leq 75$

$$\beta = 1 - 0.6 \left( \frac{\lambda}{100} \right)^2$$

b/ for ratio of slenderness $75 \leq \lambda \leq 200$

$$\beta = \frac{3100}{\lambda^2}$$

Values of coefficient $\beta$ are shown in Table 23.

Table 23. Coefficients of buckling $\beta$ for timber

<table>
<thead>
<tr>
<th>$\lambda$</th>
<th>$\beta$</th>
<th>$\beta$</th>
<th>$\lambda$</th>
<th>$\beta$</th>
<th>$\beta$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.00</td>
<td>0.756</td>
<td>105</td>
<td>0.281</td>
<td>155</td>
<td>0.129</td>
</tr>
<tr>
<td>10</td>
<td>0.99</td>
<td>0.712</td>
<td>110</td>
<td>0.256</td>
<td>160</td>
<td>0.121</td>
</tr>
<tr>
<td>15</td>
<td>0.98</td>
<td>0.662</td>
<td>115</td>
<td>0.234</td>
<td>165</td>
<td>0.114</td>
</tr>
<tr>
<td>20</td>
<td>0.97</td>
<td>0.608</td>
<td>120</td>
<td>0.215</td>
<td>170</td>
<td>0.107</td>
</tr>
<tr>
<td>25</td>
<td>0.96</td>
<td>0.550</td>
<td>125</td>
<td>0.198</td>
<td>175</td>
<td>0.101</td>
</tr>
<tr>
<td>30</td>
<td>0.95</td>
<td>0.484</td>
<td>130</td>
<td>0.183</td>
<td>180</td>
<td>0.096</td>
</tr>
<tr>
<td>35</td>
<td>0.94</td>
<td>0.429</td>
<td>135</td>
<td>0.170</td>
<td>185</td>
<td>0.091</td>
</tr>
<tr>
<td>40</td>
<td>0.93</td>
<td>0.385</td>
<td>140</td>
<td>0.158</td>
<td>190</td>
<td>0.086</td>
</tr>
<tr>
<td>45</td>
<td>0.92</td>
<td>0.343</td>
<td>145</td>
<td>0.147</td>
<td>195</td>
<td>0.082</td>
</tr>
<tr>
<td>50</td>
<td>0.91</td>
<td>0.310</td>
<td>150</td>
<td>0.138</td>
<td>200</td>
<td>0.077</td>
</tr>
</tbody>
</table>

For intermediate values one should interpolate linearly.

4.2.8. Members of composite sections axially compressed.

4.2.8.1. Types of composite members sections. It is recommended to use composite members sections shown in figures 10 - 14 and systematized in the groups below:

I group - members composed of branches and of plates or inserts out of which only branches are fixed in joints/Fig.10/.

II group - members composed of abutting branches and fixed to supports.
III group - members composed of branches with clear spacing, up to 3 thicknesses of the branch with batten plates of axial spacing not bigger than 1/3.3/ Fig. 12/ where 

L - length of a member.

IV group - composite trussed members of big spacing, bigger than 6 thicknesses of the branch/Fig. 13/ with nailed connections.

4.9.2. Number of parallel beams both to the axis x - x and to y - y cannot exceed 4 in the cross-section/Fig. 10a/.

4.9.3 Buckling in the plane perpendicular to beams is calculated according to formulae:

a/ group I/Fig. 10a/:

$$J_p = \sum J_i + 2T_i e_i^2 + 2T_i L_i e_i^2 \frac{S_i}{s_i}$$

/32/

for elements such as in fig. 10b/c/ and d/ P_c = 0 and the formula /32/ is reduced to

$$J_p = \sum J_i + 2T_i e_i^2$$

/32'/

b/ group II/Fig. 11/:

$$J_p = \sum J_i + 2T_i e_i^2$$

/33/

c/ group III/Fig. 12/:

$$J_p = \sum J_i + 2T_i e_i^2$$

/34/

d/ group IV/Fig. 13/:

$$J_p = \sum J_i + 2T_i e_i^2$$

/35/

e/ group V/Fig. 14/:

$$\chi_{sp} = \sqrt{\chi_1^2 + \chi_2^2}$$

/36/

where:

$$\chi_1 = \text{reduction coefficient}$$

$$\chi_1 = 0.04 + 0.005 A_1$$

for nails: $$\chi_1 = 0.04$$; for screws: $$\chi_1 = 0.005 A_1$$

$$\chi_2 = \text{reduction coefficient}$$

$$\chi_2 = 0.08 + 0.0054 A_1$$

$$\chi_2 = \frac{T}{T_s}$$; $$\chi_2 = 0.25 S_0$$

q - correction coefficient depends on the kind of connectors:

a/ for nails a = 1.0

b/ for inserts a = 1.05

c/ for screws a = 0.65

$$\sum J_i$$ - moment of inertia of all components of a composite member in relation to their own axes, parallel to the mean, cm^4
4.9.4. Moment of inertia of fluid members.
Glued members composed of several parts may be calculated as members with uniform section while meeting the conditions from 6.1.

4.9.5. Moment of inertia of columns and frame members with variable sections on the length may be assumed as constant taking as a basis the cross-section at the distance 0.65h or 0.65r from the supporting hinge or the ridge hinge/Fig.9c/.

4.9.7. Calculation of shearing forces/delaminating/ To ensure the cooperation of elements in a composite member, the number of connectors should be calculated for shearing/delaminating/ forces. The shearing force $t_w$ per 1 cm of the member on its whole width is calculated for members of groups I and II according to the formula

$$t_w = \frac{Q_S p_n t}{J_{sp}}$$

where:
- $Q$ - substituting lateral force with buckling
- $p_{n}$ - $p_{n}(1+\alpha)$
- $S_{br}$ - gross static moment of the section part beyond the beam
- $J_{sp}$ - gross moment of inertia, cm$^4$
- $\alpha$ - coefficient the value of which should be:
  - for bar elements/members of groups I and III as in Fig. 10 $\alpha = 0$
  - for elements with external batten plates of group IV/Fig. 14/ or trussed ones of group V/Fig. 13/ a/ with $2e = 201_{k}, \ x = 0$
  - b/ with $2e \geq 201_{k}, \ x = 0.06\frac{d}{(d-d_{a})}$
- $V'$ - reduction coefficient depending on $h$ a kind of member and connectors equal to $V'/V_{1}$, $V'/V_{2}$
- $\rho_{b}$ - coefficient of buckling depending on $h$ the ratio of slenderness of a member
- $F$ - axial force in a member/bar/ from design loads, kN.

4.9.4. Buckling in the plane parallel to the soame

Moment of inertia $J_{so}$ of the composite member section working for buckle$^2$ in the plane parallel to soame is calculated according to the formula

$$J_{so} = \sum J_{o} + 0.8 \left(\frac{1}{\alpha} \sum J_{n} + J_{o} \right)$$

where:
- $J_{o}$ - moment of inertia of a branch, cm$^4$
- $J_{n}$ - moment of inertia of a batten plate, cm$^4$
- $J_{o}$ - moment of inertia of an insert, cm$^4$
1.3.3. Batten plates should be calculated for  
for a shearing force per one  
/a) two branches: Eq. 12 and 14a 
\[ T_\text{a} = \frac{P}{2e_1} \]

/b) three branches: Eq. 14b 
\[ T_\text{b} = 0.5 \frac{Q}{e_1} \]

where  
\( e_1 \) = distance between branch centres 

Apart from that, batten plates should be calculated  
for the moment of shearing forces in relation to  
the fixing of a batten plate to a branch: Eq. 14c, d  

4.5.2. Cross braces in trussed members of type IV  
Eq. 13/1 are calculated according to the formula  
\[ K = \frac{Q}{Q} \]

where:  
\( Q \) = lateral force acc. to formula /47/, kg  
\( \beta \) = \[ \frac{P_F}{P_{\delta}} \]

4.5.10 Minimum number of connectors in the column  
connection: In column connections /type III, Fig. 12/  
there should be at least 4 nails, 2 bolts or 2 rings.  
Closed inserts should have the length equal at least  
to double spacing of column. Trussed members of  
type IV (Fig. 13) should have batten plates at both  
ends and in every connection of a cross-brace with  
branches/each side/ at least 4 nails. 

4.10. Members, loaded eccentrically.  
4.10.1. Eccentrically tensioned members. Stresses in  
a dangerous section are calculated according to the  
formula  
\[ \sigma = \frac{P}{F_n} + \left( \frac{M_\delta}{W_\delta} + \frac{M_\gamma}{W_\gamma} \right) K_\delta \leq K_m \]

with symbols as in 4.7.5. 

4.10.2. Eccentrically compressed members acc. to  
Eq. 10/12 and 14 are calculated according to the  
formula  
\[ \sigma = \frac{P}{F_n} + \left( \frac{M_\delta}{W_\delta} + \frac{M_\gamma}{W_\gamma} \right) K_\delta \leq K_m \]

and members which support compressed flanges of  
several structures/flat/ or columns should be calculated  
for compressing forces in separate zones acc. to the  
formula /Fig. 15/  
\[ P = \frac{nN}{50} \]

where  
\( N \) = the biggest compressing force  
in the flange of a girder or a column, kg  
\( n \) = number of columns or flat  
structures /Fig. 15/
4.11.2. Trusses and braces for bracing, the task
of which is to diminish the buckling length of flanges of flat compressed girders, arches and frames, may be approximately calculated for forces operating perpendicularly to these structures, the forces resulting from the substitute load q uniformly distributed in a uniform way along the flanges/Fig. 16/, according to the formula

\[ q = \frac{0.08 N}{l_1} \]  

and loading of a joint — according to the formulas:

a/ for the case as in Fig. 16a/, b/ for braces/

\[ q = \frac{0.08 N_0}{l_1} \]  

b/ for the case as in Fig. 16c/, d/, e/ for trusses/

\[ q = \frac{0.08 N}{l_1} \]  

where:

- \( N \) — the biggest calculation force in the compressed flange of a girder, kN,
- \( l_1 \) — buckling length of the whole flange of a compressed girder/Fig. 16/, cm
- \( a \) — length of the truss section
- \( e \) — buckling length of a flange of a flat, load-bearing structure on the distance between braces/Fig. 16a/, cm
- \( n \) — number of flat structures transferring forces from the flange buckling to one truss or brace.

4.11.3. Stress checking in the compressed flange of a girder. Buckling is not necessary for cases when distances between bracings are not longer than 40 l.


4.12.1. Structural height \( h \) of I- or box girders with a web or a wall or crossing boards or plywood should be measured between external edges of flanges. For beams with parallel flanges or the lowest upper flange, the height is measured in the middle of the span and for beams with upper ridge flange — in 1/4 of the span, assuming \( h = 1/8 \) of 1/4 L.

4.12.2. Structural height \( h \) of truss girders measured between flange area in the middle of the span, if one does not perform an exact calculation of deflections following the rules of building mechanics, should be defined in the following way:

a/ for girders with the upper arch flange — \( h = 1/7 \) of 1/8 L,

b/ for girders with parallel flanges or bent ones — \( h = 1/7 \) of 1/8 L,

c/ for triangular girders — \( h = 1/7 \) of 1/8 L.

4.12.3. Elevation \( f \) and structural height \( h \) of the cross-section of two- or three-hinged arch and of a flat roof should not be smaller than:

a/ for centre arch of short boards on edge

\[ f \geq 1/6 \text{ L} \]

\[ h \geq 1/80 \text{ L} \]

b/ for a two-hinge arch of I-section with a latticed wall or full wall/Fig. 17 b/:

\[ f = 1/7 \text{ L} \]

\[ h = 1/30 \text{ L} \]

c/ for three-hinge arches with full or latticed segmented elements/Fig. 17 a/

- flat \( f = 1/6 \) of 1/7 L
- spiny \( f = \) according to building requirements \( h = 1/15 \) of 1/30

d/ for lamella roof/Fig. 17 c/:

\[ f = 1/7 \text{ L} \]

\[ h = 1/100 \text{ L} \]
4.13: Centrings

4.13.1. Measuring of centrings is checked with regard to bending in their plane, taking into account the axial compressing force working in them according to 4.10.2. with the following symbols:

\[ F = \frac{P}{n} \]  
section of all short boards, not heated in the contact, cm²

\[ W = \frac{W_n}{n} \]  
modulus of not strength of short boards, not heated in the contact, cm

Length of short boards should not be smaller than the tenfold biggest cross-section of a short board and not smaller than 1.20 m.

4.13.2. Number of connectors in centrings of the type shown in Fig. 18 is defined by the formula

- a/ with connectors at the ends of short boards/ Fig. 18a/ 
\[ n_1 = \frac{M_1}{T_1} \]

- b/ When connectors are placed on the whole length of a short board /Fig. 18b/ 
\[ n_2 = \frac{24 M_1}{T_1 l_1} \]

where:

- \( n_1 \) - number of connectors at the ends of centrings
- \( n_2 \) - number of connectors on the length \( l_1 \) of a centring
- \( T \) - bearing capacity of one connector, kG
- \( M_1 \) - the biggest moment on the length of a successive short board according to the diagram on Fig. 18a/ for a strengthened axis of a short board/, kcm
- \( a \) - distance between the centre of gravity of connectors placed at the end of a short board and the place of the biggest moment, cm
- \( l_1 \) - length of a successive short board, cm.
5. CONNECTIONS

5.1. Nailed connections. For connections it is necessary to use round nails according to BH-70/50/28-17 and square twisted nails according to BH-70/50/28-19 with diameters from 1/8 to 1/11 of the thinnest of the connected elements.

5.1.2. Minimum thickness of boards and steel sheets used in timber nailed structures.

5.1.2.1. Minimum thickness d of boards should not be less than

\[ d = d\left(\frac{3+8d}{3}\right) \geq 2.2 \text{ cm} \]

with the exception of:

a/ web boards of nailed beams of I-section/Fig. 6/ whose thickness should not be smaller than 16mm,

b/ component boards of composite elements connected with nails driven into previously drilled holes, with the board thickness not smaller than 6d. With thinner boards the bearing capacity is decreased according to p. 5.1.7.3.

5.1.2.2. Minimum thickness of steel sheets used in joints and contacts should not be smaller than 2mm.

5.1.2.3. Nails in connections timber - plywood should have, with the plywood thickness 2mm - the diameter to 4.0mm and with thicker plywood - the diameter up to 4.5mm.

5.1.3. Drilling of holes for nails. Nails in connections of hardwood should be driven in previously drilled holes for the whole nail length. Drilling of holes for nails in softwood is not necessary/p. 5.1.7.1/. The diameter of drilled holes should be

\[ d_{n} = 0.95d_{n} \]

5.1.4. Arrangements of driven nails. Nails are driven according to one of the three arrangements: a/ rectangular/Fig. 10a/, b/ staggered/Fig. 19b/, c/ at the angle to the direction of the grain/Fig. 19c/

In the arrangement of nails there are rows and series. Series run along the direction of the grain, rows across or at the angle to the direction of the grain. Distance S between nails in series, distance S between rows, and distance S between series, for tensioned elements should not be smaller than 15d, and for compressed elements - not smaller than 10d. Distance between nail centres in series S depends on the ratio of the thinnest element thickness to the nail diameter and is defined by the diagram in Fig. 20. For nails driven into previously drilled holes distances S may be diminished to 10d. Distance S of the first nail series to the unloaded edge of an element should not be smaller than 4d. Distance S between series of nails in the rectangular arrangement and at the angle with the angle of \( \alpha < 45^\circ \) should not be smaller than 4d/Fig. 19c, c/ and in the staggered arrangement at the angle with the angle \( \alpha < 45^\circ \) - not smaller than 4d/Fig. 19c, c/.

In connections at the angle it is necessary to keep minimum distances between nails along and across the direction of the grain of connected elements/Fig. 21/.

5.1.5. Nail driving.

In general nails should be driven on both sides of an element, so that their ends do not stick out. When the nail end projects beyond the surface of an element, one should take into account weakening of the section. In this place 1, 5d bend the nail end/Fig. 22/ along the direction of the grain.
5.1.6. The smallest number of nails in a connection is 4 pieces. Nails should be driven in no more than two series and in no more than two rows in every connected element. Fig. 25. In connections between secondary elements e.g. cross-braces and poles in shoring and scaffolds it is possible to use a smaller number of nails but not smaller than 2 pieces.

5.1.7. Load-bearing capacity of nails

5.1.7.1. Load-bearing capacity of a nail in single shear. Force \( T \) which is safely transferred in a softwood connection, by a single shear of a round nail is calculated according the given formula or according Table 24:

\[
T = \frac{625 d^2}{144} \text{ mm}^2
\]

a/ for nails driven in directly

b/ for nails driven into the previously drilled holes:

\[
T_a = 1,25T
\]

in softwood

\[
T_a = 1,50T
\]

in hardwood

where:

- \( d \) = nail diameter, cm
- \( T \) = corrective coefficient, the value of which is:
  - in I-beams or arches with a full web of crossing boards - \( j = 0,6 \)
  - in contact between boards, planks or halves with round timber
  - in shoring - Fig. 26a
  - in connections between elements contacting with curving surfaces e.g. round timber, halves etc. - Fig. 26b

\( j = 0,7 \)
- In bonding elements for reinforced concrete such as anchors, pressure boards etc., taking lateral pressure of concrete mass = 3 = 1.8
- In all other cases = 3 = 1.6

Fig. 26

Table 24. Load-bearing capacity T of nails driven into pine wood and spruce wood directly or into previously drilled holes (single shear):

<table>
<thead>
<tr>
<th>Size of nails</th>
<th>Bearing capacity kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter x length</td>
<td>With thickness of at least</td>
</tr>
<tr>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>2.8 x 65</td>
<td>22</td>
</tr>
<tr>
<td>3.0 x 70</td>
<td>22</td>
</tr>
<tr>
<td>3.5 x 80</td>
<td>22</td>
</tr>
<tr>
<td>4.0 x 90</td>
<td>22</td>
</tr>
<tr>
<td>4.5 x 100</td>
<td>22</td>
</tr>
<tr>
<td>5.0 x 110</td>
<td>22</td>
</tr>
<tr>
<td>5.5 x 125</td>
<td>22</td>
</tr>
<tr>
<td>6.0 x 150</td>
<td>22</td>
</tr>
<tr>
<td>6.5 x 175</td>
<td>22</td>
</tr>
<tr>
<td>7.0 x 200</td>
<td>22</td>
</tr>
<tr>
<td>7.5 x 225</td>
<td>22</td>
</tr>
<tr>
<td>8.0 x 250</td>
<td>22</td>
</tr>
</tbody>
</table>

5.1.7.2. The influence of nail driving depth a on the nail bearing capacity. Load-bearing capacity of a nail according to the formula/67 - 69/ can be noted, where the nail driving depth a without a nail point in the last connected element is for nails in single shear - 126, for nails in multi shear - 64. With the nail driving length a shorter than the one mentioned above/fig.23b, c/ d/ the nail bearing capacity decreases in relation to the basic one:

a/ For nails in single shear - 

b/ For nails in multi shear - 

where \( T_0 = T_{,1} \) or \( T_2 \) - according to the formulae/67 - 69/.

One does not take into consideration the value of the nail end in the connection if the nail driving depth a smaller than 64 for nails in single shear and 44 for nails in multi shear.

5.1.7.3. Load-bearing capacity of nails driven into drilled holes with the diameter of \( d > 5 \) mm. To calculate according to the formulae/67/ and /69/ when the thickness of an element \( b > 64 \). For thinner elements the nail load-bearing capacity decreases

\[ T = 1.25 T_{,4} \]

5.1.7.4. Load-bearing capacity of nails in connections timber-concrete shear/fig.27/ with holes/drilled simultaneously in timber and single shear according to the formula/66/ and /69/ if the elements are fully connected.

The nail anchorage length \( a \) is not smaller than \( 1 a \) without a nail point/66/.

While defining load-bearing capacity in connections with plywood butt joint, it is necessary to check the pressure/nails of the holes for nails in plywood according to the formula

\[ T = 3d^2k_{,6} \]

where \( k_{,6} \) - standard stress of plywood, kg/cm².

In tensioned contacts with the number of nails in a series \( 10 \geq 2 \), their number should be increased by 10%, with the number of nails bigger than 20 by 20%.

5.1.7.5. Length of nails. While calculating the length of nails one should take into account the required nail driving depth adding 1,5 cm for every seam between connected elements and 1,5 cm for a nail point/fig.22/.

5.1.7.7. Permissible radius of curvature or elements in nail connections.

While connecting curved elements with nails, the radius of these elements should be \( R \geq 30d^2/b \), thickness of the thinnest element/.

5.1.7.8. Calculation of the net cross-section. In tensioned elements in nail connections, the section is diminished by holes for nails with the diameter bigger than 4,5 mm:

- with the rectangular arrangement or at an angle all holes are in one row
- with the staggered arrangement all holes are in two rows

While driving nails into drilled holes we take into account the section weakening for every diameter of driven nails.
2.2. Bolt and screw connections

2.2.1. Bolts should be made of cold-rolled steel acc. to PM-75M-01200.00. The bolt diameter should be 10 ± 0.2 mm and correspond with the assortment of standard nuts acc. to 2.4.1.2 and table 25.

2.2.2. Screws should be applied acc. to 2.4.1.1. The smallest permissible diameter of screws used in connections between elements with the thickness up to 60 mm in 10 mm and with thicker elements - 12 mm.

<table>
<thead>
<tr>
<th>Bolt diameter</th>
<th>Square section (mm²)</th>
<th>Dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>12 x 12</td>
<td>12 x 12</td>
</tr>
<tr>
<td>16</td>
<td>16 x 16</td>
<td>16 x 16</td>
</tr>
<tr>
<td>20</td>
<td>20 x 20</td>
<td>20 x 20</td>
</tr>
</tbody>
</table>

Fig. 27

Load bearing capacity should not be taken into account with pulling out nails, driven previously into previously drilled holes or into the face of an element and in case when dynamic loads are likely to occur.
5.2.4. Arrangements of bolts and screws. Bolts and screws are arranged as shown in Fig. 19a and 19b, with the centres of the first bolt and the face of an element should be:

- In tensioned elements: $S > 34$
- In compressed elements: $S > 44$

Spaces between bolts in series: $S = 74$. Distance between the first series of bolts and and the unloaded end of an element: $s = 34$. Distance between both in the perpendicular scheme and the staggered one: $S = 34$

A stagger arrangement between series should be determined in such a way so as to make it possible to tighten up nuts with a spanner.

5.2.5. Number of bracing screws in contact. In tensioned contacts with timber plates, at least 25% of bolts should be replaced with screws of the same diameter while in contacts with metal plates - at least 50%. In each side of a contact/tensioned/should have at least three screws/2 at nut ends and one at the contact/. A threaded part of a screw should not be less than the timber.

On each side of compressed contact one should place at least 2 screws.

5.2.6. Number of bolts in tensioned contacts on each side should be at least 2 in series and 2 in row (Fig. 25a/).

5.2.7. Steel washers are to be placed under a screw head and nuts acc. to Table 29 should be used in timber connections without steel cover plates.

5.2.8. Bolt and screw connections can be applied in structures. If washers are taken to protect these structures against too big deformation/structural flexion and proper fitting of bolts in timber holes - p. 5.2.10.

5.2.9. Bracing screws should have washers under heads acc. to PN-99/B-0210. Dimensions of washers are to be found in Table 29.

5.2.10. Section of bolts and screws in connections should be in holes with the diameter of about 0.97 of the bolt or screw diameter.

5.2.11. Load bearing capacity of a bolt or a screw. Single shear. Bolt or screw connections may be in single or multi shear. Force $F/A$ carried by one bolt in a timber connection, with the force operating along the grain, is calculated according to the following formulae, assuming the least of the obtained values:

- $T = K_d \cdot \frac{a}{A}$, $T = A \cdot g$, $A = \frac{a}{A}$

Where:

- $K_d$ = design strength at pressure in the hole of an element acc. to Table 29, kg/cm²
- $d$ = bolt diameter, cm
- $g$ = thickness of a timber element/ in asymmetrical connections - thinner element
- $A$ = coefficient acc. to Table 29/ kg/cm²

Values $T$ for $n = 1$ are shown in Table 27.
<table>
<thead>
<tr>
<th>bolt diameters</th>
<th>design values</th>
<th>thickness of elements, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>$T_a$</td>
<td>133 154 175 200 224 235 266 294 315 325 325</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>95 110 125 145 160 175 190 210 225 240 240</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>104 121 147 159 175 192 204 200 210 210 210</td>
</tr>
<tr>
<td>12</td>
<td>$T_a$</td>
<td>160 185 210 244 268 294 319 353 378 420 468 468 468</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>114 132 150 174 192 210 228 252 270 300 342 345 345</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>125 145 165 191 211 231 251 277 297 302 302 302 302</td>
</tr>
<tr>
<td>14</td>
<td>$T_a$</td>
<td>186 216 245 284 314 343 372 411 440 490 558 588 617 637 637</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>133 154 175 203 224 245 266 294 315 350 400 420 440 470 470</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>146 169 193 223 247 270 293 324 347 386 411 411 411 411 411</td>
</tr>
<tr>
<td>16</td>
<td>$T_a$</td>
<td>213 246 280 325 358 392 425 470 504 560 639 672 705 784 832 832</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>152 176 200 232 256 280 304 336 360 400 456 480 504 560 607 614 614</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>167 193 220 255 281 308 334 369 396 440 502 527 557 575 595 595 595</td>
</tr>
<tr>
<td>18</td>
<td>$T_a$</td>
<td>239 277 315 366 403 440 479 530 567 630 718 756 794 882 957 995 1050 1050</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>171 198 225 261 288 315 342 378 405 450 513 540 567 630 684 710 780 780</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>188 218 248 287 317 347 376 416 445 495 565 595 625 680 680 680 680</td>
</tr>
<tr>
<td>20</td>
<td>$T_a$</td>
<td>266 308 350 406 448 490 532 588 650 700 798 840 882 980 1060 1100 1243 1300 1300</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>190 220 250 290 320 350 380 420 450 500 570 600 630 700 760 790 890 960 960</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>209 242 275 320 352 386 418 465 496 551 628 660 694 772 837 840 840 840 840</td>
</tr>
<tr>
<td>22</td>
<td>$T_a$</td>
<td>292 338 385 447 493 539 585 647 693 770 877 923 970 1060 1170 1215 1270 1340 1575 1575</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>209 242 275 319 352 385 418 462 455 550 627 660 693 770 836 870 980 1000 1100 1160 1160</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>230 266 302 351 387 423 460 508 545 606 690 726 763 845 920 957 1000 1000 1000 1000 1000</td>
</tr>
<tr>
<td>24</td>
<td>$T_a$</td>
<td>319 369 420 487 537 588 638 705 756 840 957 1020 1050 1175 1325 1330 1430 1480 1480 1550</td>
</tr>
<tr>
<td></td>
<td>$T_c$</td>
<td>250 290 330 373 422 462 501 553 604 652 730 780 810 823 880 900 1040 1170 1210 1210 1210</td>
</tr>
<tr>
<td></td>
<td>$T_n$</td>
<td>280 324 364 405 446 487 528 570 620 664 768 820 875 920 980 1000 1040 1040 1040 1040 1040</td>
</tr>
</tbody>
</table>

Table 27. Design load bearing capacities of steel bolts and screws in single shear/kG/ in symmetrical $T_a$ and $T_c$ and asymmetrical $T_n$ connections.
5.2.12. Load-bearing capacity of screws or tensioning
\( T = \frac{\pi d_s K_r}{4} \)

where:
- \( d_s \) - diameter of the core of the threaded part, cm
- \( K_r \) - tensioning strength of screws acc. to
  PN-76/B-03200, kN/cm²

5.2.13. Load bearing capacity of bolts with the force
  transferred perpendicularly to the grain, force \( T \) \( \),
  calculated according to the formula
  \( T = \frac{4 d_{bol} K_r}{\pi} \)

5.2.14. Load bearing capacity of bolts or screws in
  connections timber - steel is defined acc. to
  the formula \( T = \frac{4 K_r}{\pi} \)
  and \( T = \frac{4 K_c}{\pi} \) with the coefficient 0,75.

5.2.15. Calculation of net cross-section. In bolt con-
  nections one should take into account the section
  weakening for a rectangular arrangement - with all
  holes in one row and for a staggered arrangement with
  all holes in two rows, if distance between them
  does not exceed 20 cm or with holes in one row when
  distance between neighbouring rows exceeds 20 cm.

5.3. Connections with timber screws

5.3.1. Types of screws. In timber structures one should use:
- a) screws for spanner - square head or hexagon head
- screws acc. to PN-72/18-2501 and PN-72/18-2502
- b) screws for a screw-driver - acc. to PN-72/18-2503,
- PN-852503, PN-72/18-2504, PN-72/18-2505 and
- PN-72/18-2505/Fig. 288/ Minimum diameter of screws should be 4 mm.

5.3.2. Fixing of timber screws. Screws should be se-
  ated in drilled hole with the diameter of about 2 mm
  smaller than the screw diameter \( d \). Reaming should be
done on the length of about 0,80 l of a screw/Fig.28 a,b/c.

FIG.28

5.3.3. Timbers screws working for bending
  and pressure.

Connections with timber screws are made as
  single shear. To determine load bearing capac-
  ity of screws in single shear with the force
  working along the grain the smaller value
  \( T \) \( \) from the formulas is assumed:
- \( T = 50 d_{bol} \)
- \( T = 210 d_{bol} \)

where:
- \( d_{bol} \) - thickness of a board or ply-
  wood fastened to a thicker ele-
  ment/Fig. 289a, cm
- \( d \) - screw diameter, cm

In timber connections with metal cover pla-
  tes load bearing capacity is defined by
  the formula:
- \( T = 125 \cdot 210 d_{bol} \)

For screws with the diameter \( d \leq 10 \) mm, with
  the force working perpendicularly to the
  direction of the grain, while defining lo-
  ad bearing capacity one should use, addi-
  tionally, the coefficient 0,75. With the force
  working at an acute angle to the direction
  of the grain, intermediate values of
  coefficients are determined according to
  linear interpolation.

Load bearing capacity of screws, defined
  according to the formulas \( /80/ \) and \( /81/ \) re-
  fer to the basic depth of screwing in
  \( s = 8d \). With the screwing depth \( s < 8d \),
  load bearing capacity of a screw diminishes
  in relation to the basic one and is cal-
  culated according to the formula
- \( T_s = T \cdot \frac{s}{s} \)

The work of screws screwed in less deeply
  than \( 4d \) and of screws seated along the
  grain is not taken into account. With screw-
  ing depth exceeding \( 8d \), calculations
take into account only \( s = 8d \).

5.3.4. Arrangements of screws. Screws should be
  arranged as nails, with minimum spacing:
- for timber elements and plywood
  \[ S = S_1 \geq 10d \]
  \[ S_2 = S_3 \geq 3d \]
- for metal plates
  \[ S = S_1 \geq 5d \]
  \[ S = S_2 \geq 2,5d \]

Besides, with screws for a spanner, spacing
  should be such as to facilitate screwing
  holes with a spanner.

5.3.5. Minimum number of screws in a conne-
  ction. In screwed connections the minimum
  number of screws should be 4. - for screws
  with the diameter \( d \geq 10 \) mm and 2 - for
  screws with the diameter \( d < 10 \) mm.
5.1.6. Calculation of net cross-section acc. to 5.2.15.

5.1.7. Timber screws working for pulling

5.1.7.1. Arrangement of screws. Screws are arranged in the rectangular and staggered arrangement/Fig. 19a, b/ with spacing:

\[ S = S_1 > 10d, \quad S_2 = S_3 > 5d \]

5.1.7.2. Load bearing capacity of screws at pulling.

Pulling load bearing capacity \( T_p /kN/ \) of a screw placed across the grain is calculated according to the formula:

\[ T_p = 40 S d \text{ m} \]

where: \( S \) - screwing depth of the threaded part of a screw in the element with the thickness \( a_2 / \text{cm} \)

In calculations of load bearing capacity of pulled screws values \( S \) are adopted within the range \( 4d \leq S \leq 7d \).

The load bearing capacity of screws placed along the grain is not taken into account.

5.4. Connections with smooth split rings.

5.4.1. Application of rings. Smooth split rings can be applied when:

a/ Connected elements are of timber of at least class III

b/ Connections will be provided in a mechanised way in a specially adapted factory both as regards equipment and the staff training.

In connections of the same structure one should apply rings with the same diameter and width.

In tensioned elements each side of a contact should have at least two beams of rings/Fig. 29b/
4.4.3. Way of placing a ring in connections. A ring should be let into each of the connected elements to a depth equal to a half of a ring width. A ring slot should be on the diameter perpendicular to the direction of the force [Fig. 29].

4.4.4. Arrangements of rings. Distance of the ring centre from the face of a connected board or plank should be:

\[ S_1 \geq \frac{d_0}{1.56} \quad \text{in tensioned elements} \]
\[ S_1 \geq \frac{d_0}{2} \quad \text{in compressed elements} \]

Distance ring centres should be \( S_2 \geq 2d_0 \) [Fig. 29] and Table 28.

5.5.4. Load bearing capacity of rings/\( m^3 \) is defined according to the formula

\[ T = T_0 m_{mp} \]

where:
\[ T_0 = \text{load bearing capacity of a ring acc. to Table 28, \( m^3 \)} \]
\[ m_{mp} = \text{corrector coefficient depending on the number of rings in one series in the direction of the force: with 1 or 2 rings - } m_{mp} = 1.0, \]
\[ \text{with 3 and more - } m_{mp} = 0.85, \]

5.5.6. Compressing screws. Elements connected with rings should be pressed to each other with bolts with washers placed in the axis of each ring.

5.5.7. Arrangements of smooth rings, screws, and washers acc. to Table 28.

5.5.8. Loading of each ring is defined through dividing the total transferred by the connection/force:

\( a/ \) in a connection without plates - by the full number of rings [Fig. 29]/
\( b/ \) in a connection with plates - by the number of rings on one side of the connection [Fig. 29].

5.5.9. Calculation of the net cross-section should be performed taking into account the weakenings caused by indentations for rings and a hole for a masonry bracing acc. to the formulæ:

\[ F_n = F_{br} - b(a + d) - (c - b)d_{fr} \quad /5/ \]
\[ F_n = F_{br} - \frac{1}{2} (a + 2d) - (a - b) \quad /5/ \]

where:
\[ d_{fr} = \text{diameter of compressing screw}, \text{cm} \]
\[ F_{br} = \text{field, net and gross section}, \text{cm}^2 \]
\[ b = \text{thickness of a ring, cm} \]
\[ c = \text{thickness of the central element, cm} \]
\[ a = \text{thickness of the end element, cm} \]

5.5.10. Smallest dimensions of the sections of connected timber members

\( a/ \) width of the member \( B \) should exceed the inside diameter of the ring \( d_0 \) at least by 4 cm
\[ B \geq d_0 + 4 \text{ cm} \]

\( b/ \) thickness of the central element \( C \) should not be smaller than \( B + 3 \text{ and not smaller than } 6 \text{ cm} \)

The smallest dimensions of the sections of timber members - see Table 28.

5.5.11. Thickness of inserts and plates. In ring connections inserts should have the same thickness as connected elements and plates - at least 0.75 of this thickness.

5.5.12. Toothed ring connections

Distance \( S_2 \) of the ring centre from the face of the connected element should not be smaller than 1.56 of \( d_0 \) and distance between ring centres \( S_1 \) - not smaller than \( 2d_0 \). Toothed rings should be let into each of the two connected elements to a depth equal to a half of the ring height \( t \).

5.5.13. Bracing screws. Elements connected with rings should be pressed to each other with screws placed in the axis of each ring.

5.5.14. Load bearing capacity of rings.
Toothed rings transfer the force in connections through pressure. Load bearing capacity \( T_0 \) is defined on the basis of strength test of test connections. In the calculation of connections load bearing capacity of a ring \( T \) is defined by the formula

\[ T = T_0 m_{mp} \]

Where Table 29 shows the assortment and load bearing capacity \( T_0 \) of toothed rings [Fig. 29].

5.5.15. Application of rings. In contacts of tensioned elements, on each side of a contact at least two beams of rings should be applied. The number of rings in one series should not exceed 6. Rings can be applied in one or two series, in the rectangular arrangement acc. to Fig. 29. For toothed rings Geke distance between the first row and the face of an element and between rows - see. to Table 29 col. 12. Distance between series \( S_2 \) and the unloaded end \( S \geq \frac{d}{2} + 2 \text{ cm} \) where:
\[ d = \text{inside diameter of the ring} \]
\[ t = \text{ring height/Table 29 col. 12} \]
\[ b,a = \text{width, thickness of a plank/Table 29 col. 10 and 11} \]
<table>
<thead>
<tr>
<th>Dimensions of ring</th>
<th>Type of rings</th>
<th>Table 20: Auxiliary values for calculation of load bearing strength of pressed bushes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of one-sided teeth</td>
<td>1</td>
<td>Width of two-sided teeth</td>
</tr>
<tr>
<td>Height</td>
<td>Outside diameter</td>
<td>Thickness</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>0.6</td>
<td>0.65</td>
<td>0.7</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>1.5</td>
<td>1.55</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td>2.05</td>
<td>2.1</td>
</tr>
<tr>
<td>2.5</td>
<td>2.55</td>
<td>2.6</td>
</tr>
<tr>
<td>3</td>
<td>3.05</td>
<td>3.1</td>
</tr>
<tr>
<td>3.5</td>
<td>3.55</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Auxiliary values for the calculation of load bearing strength of pressed bushes should be defined on the basis of linear interpolation.
5.6. Block connections.

5.6.1. Shape of blocks. Blocks with shapes as in Figs. 34 and 35 should closely fit to their sockets, in connected elements. When the connection is in danger of becoming loose tight as a result of timber drying, it is advisable to apply wedge-shaped blocks instead of separate blocks/Fig. 34b/.

5.6.2. Dimensions of rains and blocks.

Depth of a gain a should not exceed 1/5 of the section height h of a connected element/Fig. 34a/. The smallest gain depth 2 cm for elements of hardfast sawn timber and 3 cm - for elements of round timber. Length e of a block should be equal or bigger than the fivefold depth of a gain/Fig. 34b/ that is e > 5a.

5.6.3. Axial distances e of blocks should be within the limit 2a < e < 25a.

5.6.4. Lead bearing capacity of blocks. Blocks should be calculated for a shearing force/delaminating/, the value of which on the length of 1 cm on the whole beam width is defined acc. to the formula/8/5/. Lead bearing capacity b/kd/ is calculated according to the formulae:

a/ for pressure in the direction of the beam axis

\[ T = abk_{1}^{m} \]

b/ for shearing of a block

\[ T = sk_{2}^{m} \]

c/ for shearing between two adjoining blocks

\[ T = sk_{3}^{m} \]

where:
- \( a \) - depth of a gain, cm
- \( b \) - width of connected elements
- \( k_{1} \) - standard strength at pressure corresponding to \( K_{1} \) or \( K_{2} \), kg/cm²
- \( m \) - correction coefficient acc. to 3.11.
- \( a \) - length of a block
- \( k_{2}^{s} \) - standard strength at shearing along the grain or perpendicularly to the direction of the grain, e.g. wedges/ for \( k_{2}^{s} \), the same acc. to the formula/8/5/, kg/cm²
- \( d \) - distance in the clearance between blocks, cm

5.6.5. Bracing bolts are calculated for a tensile force \( S \) according to the formulae:

a/ for beam elements in contact with each other/Fig. 34a a/b/:

\[ S = \frac{K_{a}}{a} \]

b/ for beam elements which are not in contact with each other/Fig. 34a/:

\[ S = \frac{T(e+c)}{a} \]

where:
- \( e \) - distance in clearance between connected elements/Fig. 34a/.
- other symbols acc. to 5.6.4.

5.6.6. Equilateral oblique blocks. In the calculation of lead bearing capacity of equilateral oblique blocks, the formula/9/1/, length of the shear surface should be assumed acc. to Fig. 35.

5.7. Connections with steel plates with ribs.

5.7.1. Application. Connections/sets/ with steel plates/with ribs/ are applied in contacts between tensioned elements/Fig. 36/.
5.7.2. Construction. Steel plates are composed of two flat steel bars placed on each side of contacting elements, symmetrically to the contact surface. Plates are connected with flat steel ribs with rivets or welding. Ribs are inserted into grooves cut in connected elements. The smallest thickness of a plate is 6 mm.

5.7.3. Load bearing capacity of plates. The joint net section of plates should be

\[ S = \frac{N}{K_r} \]

where:
- \( N \) - design tensioning force in a connection, kN
- \( K_r \) - strength of steel at tensioning according to PN-76/B-032300, kN/cm²

The connection of plates with ribs should be calculated for the force \( T \) transferred by each rib.

5.7.4. Dimensions and arrangement of ribs:
- a/ thickness of a rib should be within the limit
  \[ 1.5 \text{ cm} \leq b \leq \frac{1}{3} h \]
- b/ width of a rib \( b_1 \geq 2.5 a \)
- c/ distance between ribs in clearance \( s \leq 10 c \)
- d/ number of ribs of one plate on one side of a contact should not exceed 4.

5.7.5. Load bearing capacity of rib is calculated according to the formulas:
- a/ for the pressure to timber
  \[ T = 3.7 b K_{rib} \text{ mm} \]
- b/ for shearing between rib
  \[ T = 0.7 b K_{rib} \text{ mm} \]

where:
- \( n \) - corrector coefficient depending on the number of ribs:
  - with 1 or 2 ribs \( n = 1.0 \)
  - with 3 ribs \( n = 0.9 \)
  - with 4 ribs \( n = 0.8 \)
- other symbols acc. to 5.6.4.

5.7.6. Tensioning for the force \( S \) acc. to the formula

\[ T = \frac{(c + d)}{\theta} \]

where:
- \( T \) - load bearing capacity of rib, kN
- \( c \) - thickness of a rib, cm
- \( b \) - thickness of a plate, cm
- \( e \) - distance between rib centres and screw centres/5.3.6/,

5.7.7. Calculation of the net cross-section

The net cross-section of the connected elements working for tensioning is calculated with regard to the section weakening by the indentation for ribs/5.3.6/:

\[ F_n = (h - 2c)b \]

5.8. Toothed connections.

5.8.1. Toothed elements

5.8.1.1. Width of toothed connected elements/Fig. 37/ should not be smaller than 5 cm.

In case when a screw is to be let through an element, its width should not be smaller than 8 cm/5.3.7/ and not smaller than \( 6d \)/.

5.8.1.2. Dimensions of reaming should be:
- a/ depth \( r \):
  - in intermediate gains \( - r \leq 0.25 h \)
  - in support gains \( - r \leq 0.3 h \)

but not less than 2 cm - in elements of rectangular section and not less than 3 cm - in elements of round timber; difference the gain height in a double gain \( r_2 - r_1 \) should exceed or be equal to 2 cm;

- b/ length of the shearing surface should not be smaller than 1.5h or \( 1.5 \frac{a}{d} \) - round timber diameter and not smaller than 20 cm.
5.8.1. Pressure stress is defined by the formula:

a/ in single gages with the pressure surface on
bisectrix/Fig. 37a/:

$$\sigma = \frac{N \cos \alpha}{rb} \leq K_{da} \frac{\sigma}{m}$$

b/ in single gages with the pressure surface perpendicular to one of the connected elements/Fig. 37b/:

$$\sigma = \frac{N \cos \alpha}{rb} \leq K_{da} \frac{\sigma}{m}$$

100/1

c/ in double gages/Fig. 37c/:

$$\sigma = \frac{S_1}{F_d} \leq K_{da} \frac{\sigma}{m} \quad \text{and} \quad \sigma = \frac{S_2}{F_d} \leq K_{da} \frac{\sigma}{m}$$

101/1

where: $N$ - design axial force in a compressed member, KG

$F_d$ - pressure surface of the first or second gage, cm²

$S_1$, $S_2$ - components of the axial force $N$

of the compressed member, perpendicular to the pressure surface, KG

$K_{da}$ - standard strength at pressure

depending on the inclination angle

of the pressure surface, KG/cm²

$F$, $b$, $K$ - acc. to Fig. 37
3.1. Shearing stresses are defined by formulae:

\[ \tau = \frac{N \cos \alpha}{F_t} \leq 0.6 \frac{k}{m} \]

\[ \tau = \frac{N \cos \alpha}{F_t} \leq 1.15 \frac{k}{m} \]

where:
- \( F_t = t_1 b \) = shear surface, \( \text{cm}^2 \)
- \( F_t = t_1 b \) = shear surface at the depth \( r_1 \), \( \text{cm}^2 \)
- \( F_t = t_2 b \) = shear surface at the depth \( r_2 \), \( \text{cm}^2 \)
- \( k \) = average standard strength acc. to formula 6.

Other symbols - acc. to 5.8.1.3.

5.8.2.4. Pressure stresses should be defined by the formula:

\[ \sigma = \frac{N \cos \alpha}{2 \tau (r_1 + r_2) \eta} \leq 0.5 \frac{k}{m} \]

where:
- \( h \), \( r_1, r_2 \) - acc. to 5.8.2.1. and Fig.38
- \( \eta \) - number of branches

5.8.2.5. Bracing screws. Support gains of trusses (Fig.37) should be protected with bracing bolts with the core diameter \( \delta \)/cm.

\[ \delta = 4.10 \sqrt{ \frac{N}{9k^2} } \]

where:
- \( N \) = design axial force, kN
- \( k \) = tensioning strength for bolts, kN/cm²
- \( \alpha \) = inclination angle of connected elements

5.8.2. Notches.

5.8.2.1. Dimensions:

a/ depth of the side notch (Fig. 38):
- with one-sided notch/symmetrical/ in end plate:
  \( r = 0.5 \delta_2 \)
- with two-sided notch/symmetrical/ in central plate:
  \( r = 0.5 \delta_2 \)

b/ average length of the shear surface: \( t > 1.5 \delta; t > 20 \text{cm} \)

5.8.2.2. Bracing stresses. In notches one should apply horizontal bracing screws.

5.8.2.3. Shearing stresses. With the inclination angle \( \alpha \) of connected elements up to 45°, shearing stresses along the grain should be calculated acc. to the formula:

\[ \tau = \frac{N \cos \alpha}{F_t} \leq 0.5 k_m \]

where:
- \( N \) = axial force in a compressed member, kN/cm²
- \( F_t \) = sum of sheared surfaces, \( \text{cm}^2 \)
- \( k_m \) = standard strength at shear in notches; for \( \alpha > 30° \):
  \( k_m = 4.0 \text{ kN/cm}^2 \)
  for \( \alpha < 30° \):
  \( k_m = 6.0 \text{ kN/cm}^2 \)
- \( n \) = coefficient; for \( \alpha < 45° - n = 0.9 \)
  for \( \alpha = 45° + 40° - n = 0.5 \)

5.8.3. Clamp connections. Clamp connectors should be used only in secondary connections or in temporary timber structures, e.g. square-I beam, round timber, and planks. Clamps should not be used in structures of timber boards.

5.10. Other types of connectors which have not been discussed in the standard may be used, providing that permission is obtained from scientific authorities/scientific centre.

Permissible load bearing capacity, one adopts, is the smaller one of the two values:

\[ \frac{\tau}{\delta} \]

b/ loads, with which the shift of connected elements is \( 1.5 \text{ cm} \)

5.11. Glued connections.

5.11.1. Material.

5.11.1.1. Sawn timber. For glued elements one uses pine sawn timber or spruce sawn timber/acc. to the annex/ depending on the category of structural elements/table 6.

For lengths of glued, bent, free-supported elements it is possible to apply, in one quarters of a span, timber of lower classes than in central quarters (Fig.39).
Thickness of sawn timber for glued elements, protected against humidity, should not exceed 5 cm and for unprotected elements - not bigger than 3 cm. For laminated elements, higher than 35 cm, one should not apply thicker boards than 4 cm. In laminated, curvilinear elements with the radius of curvature not smaller than 200 thicknesses of a board, one can use boards with the thickness up to 4 cm, while with the radius not smaller than 200 - boards with the thickness up to 3 cm.

5.1.1.2. Glues. For glued structures, protected against humidity, it is recommended to apply glue based on synthetic resins such as: urea resins, resorcin resins and phenolic resins. It is possible to use casein glue providing that glued connections are protected against humidity and biological corrosion. In structures exposed to humidity it is necessary to apply glue based on resorcin resins. Other glues may be applied, if officially accepted by building authorities on the basis of a certificate issued by a scientific centre. Strength of gluing depends on the kind and quality of glue. Strength of a glued connection for pine or spruce should not be smaller than 70 kN/cm² when dry, and 40 kN/cm² when moist after 24 hours of soaking.

5.1.1.3. Dryness of timber. For glued structures one should apply timber with the dryness/moisture content that would meet the requirements of the gluing technology but not exceed 15%/acc. to 5.1.5.1.

5.1.1.3 Conditions of gluing

5.1.1.3.1. It is permitted to produce glued structures only in specialized factories with a proper equipment and qualified staff. With glued structures it is necessary to ensure structure of material and technological parameters.

5.1.1.3.2. Elements should be glued along the grain. Connecting at an angle should be performed in compliance with data in Table 30, depending on the kind of glue.

Table 30. Widths of elements in connections glued at an angle

<table>
<thead>
<tr>
<th>connections at an angle</th>
<th>The biggest width of a glued element, cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>casein</td>
</tr>
<tr>
<td>90°</td>
<td>8</td>
</tr>
<tr>
<td>45°</td>
<td>12</td>
</tr>
</tbody>
</table>

5.1.1.3.3. Arrangements of boards with connection should correspond with the arrangement in Fig. 40a/
Table 32. Corrector coefficients \( k_w \)

<table>
<thead>
<tr>
<th>Beam width ( c )</th>
<th>coefficients ( k_w ) for beams with the height ( h ) (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 14 \div 50 )</td>
<td>( 1 ), ( 0.97 ), ( 0.93 ), ( 0.90 ), ( 0.88 ), ( 0.86 ), ( 0.84 )</td>
</tr>
<tr>
<td>( 6 \div 15 )</td>
<td>( 1.05 ), ( 1.05 ), ( 1.05 ), ( 1.05 ), ( 1.05 ), ( 1.05 ), ( 1.05 )</td>
</tr>
</tbody>
</table>

Height \( h \) of girders/two-trapezoidal, laminated/ with the rectangular section is measured at the distance \( h_1 \) (see Fig. 46a).  

Table 33. Corrector coefficients \( k_w \)

<table>
<thead>
<tr>
<th>ratio ( b_1/b )</th>
<th>1/2</th>
<th>1/3</th>
<th>1/4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_w )</td>
<td>0.90</td>
<td>0.80</td>
<td>0.75</td>
</tr>
</tbody>
</table>

6.3.3. Calculation of glued elements of two different materials e.g. timber and plywood should be performed with the introduction of substitute values or durability characterization:  
3a/ coefficient of strength/ comparable/  
\[
W_z = \frac{J_z}{J} 
\]

3b/ moment of inertia/ comparable/  
\[
J_z = J_{d} + J_{w} \frac{E_{w}}{E} \quad \text{or} \quad J_z = J_{w} + J_{d} \frac{E_{d}}{E}
\]

3c/ cross-section/ comparable/  
\[
F_{z} = F_{d} + F_{w} \frac{E_{w}}{E_{d}} \quad \text{or} \quad F_z = F_{w} + F_{d} \frac{E_{d}}{E_{w}}
\]

3d/ static moment/ comparable/  
\[
S_z = S_{d} + S_{w} \frac{E_{w}}{E} \quad \text{or} \quad S_z = S_{w} + S_{d} \frac{E_{d}}{E_{w}}
\]

Where:
- \( E_d \) - coefficient of elasticity of timber, \( \text{N/cm}^{2} \)
- \( E_{w} \) - coefficient of elasticity of the used material
- \( J_{d}, F_{d}, S_{d} \) - durability characterization of substitute materials,
- \( J_{w}, F_{w}, S_{w} \) - durability characterization of substitute materials,
- \( z \) - distance between the neutral axis and the end edge of the section, cm

Substitute values should be determined in relation to the material used for end layers of on element.

6.3.4. Shear stress at bending is defined by the formula:  
\[
\tau = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /111/ 
\]

6.3.5. Deflection of timber glued beams is calculated, taking into account the standard load without over-load coefficient, equally distributed, according to the formula:  
\[
\delta_{b} = 1.0 \quad \text{for} \quad A < 65 \delta_{w}
\]

with \( A \) - distance in clearance between bracing ribs, cm
- \( \delta_{w} \) - static moment of the flange section in relation to the neutral axis, cm;
- \( Q \) - transverse force on the flange section, kg.

6.3.6. Deflection in the neutral axis of the laminated section  
\[
\delta_{b} = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /114/ 
\]

7. in the glued joint in the contact of a flange with a web or a wall  
\[
\tau = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /115/ 
\]

c/ in the web or wall in the neutral axis  
\[
\tau = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /116/ 
\]

a/ stability of the web  
\[
\tau = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /117/ 
\]

Where:
- \( J_{s} \) - joint width, cm  
- \( J \) - standard width at shear sec. to Table 8  
- \( K_{m} \) - standard strength of a glued joint  
For \( J > 90 \), \( K_{m} = 1.0 \text{N/cm}^{2} \)  
For \( J = 0 \),  
\[
K_{m} = \frac{\gamma}{k_{b}} \geq 1.0 \text{N/cm}^{2}; \text{ intermediate values should be interpolated linearly,}
\]
- \( \gamma \) - angle between the direction of the grain of the face board/plywood/ and the direction of the flange grain  
- \( k_{b} \) - distance between flange axes, cm  
- \( d \) - thickness of the web or wall, cm  
- \( K_{m} \) - coefficient of stability of the web or the wall  
for plywood \( K_{m} = \frac{f_{e}}{d} \quad /118/ 
\]

\[
\delta_{w} = 1.0 \quad \text{for} \quad A < 65 \delta_{w}
\]

with \( A \) - distance in clearance between bracing ribs, cm
- \( \delta_{w} \) - static moment of the flange section in relation to the neutral axis, cm;
- \( Q \) - transverse force on the flange section, kg.

6.3.6. Deflection of timber glued beams is calculated, taking into account the standard load without over-load coefficient, equally distributed, according to the formula:  
\[
\delta_{b} = \frac{Q d_{s}}{b J_{s}} \leq K_{m} \quad /114/ 
\]

\[
\delta_{w} = 1.0 \quad \text{for} \quad A < 65 \delta_{w}
\]

with \( A \) - distance in clearance between bracing ribs, cm
- \( \delta_{w} \) - static moment of the flange section in relation to the neutral axis, cm;
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\]

\[
\delta_{w} = 1.0 \quad \text{for} \quad A < 65 \delta_{w}
\]

with \( A \) - distance in clearance between bracing ribs, cm
- \( \delta_{w} \) - static moment of the flange section in relation to the neutral axis, cm;
- \( Q \) - transverse force on the flange section, kg.
6.3.6. Wedge contact in the connection on the height of the whole section of a uniform element causes its weakening, which should be taken into account in the calculation of the net section

\[ f_n = (1 - n) F_b \]

where \( n = \frac{d}{D} \) / Table 31/.

6.3.7. Impragnation of timber before gluing is permitted, providing that scientific authorities issue a special certificate of permission for the application of given impragnation and glues.

7. STEEL BRACES

7.1. Span between hangers. A steel hanger should be suspended to a vault or an arch at spaces not bigger than 5 m.

7.2. Brace ends. In case there are no turnbuckles, should be provided with double nuts and washers with dimensions calculated from the pressure conditions caused by bracing force, with:

a/ for round washers – diameter \( D \geq 7d \)
b/ for square washers – side \( a \geq d \)

Timber under washers should be impragnated with agents protecting against any kind of biological corrosion.

7.3. Design strength in braces. In case of brace weakening with a thread, design strength is as for screws acc. to PN-76/B-03200 with the corrector coefficient 0.8.

7.4. Braces of composite section of two or more bars should be designed for a design strength reduced by 15% in relation to the one in 7.3.

7.5. Brace screws and bracing S.C.R.E.W.S should be tightened during the period of exploitation, because timber is very likely to shrink. Therefore, screws should have a sufficiently long thread and they should be accessible to facilitate their tightening.
ADDITIONAL INFORMATION to PN-73/B-03150

1. Essential modifications in PN-54/B-03150.

Measuring of timber structures acc. to the limit states method has been introduced to replace the method of permissible stresses.

2. Counterparts in foreign standards

ČSN 732050 Projektování dřevěných konstrukcí
BWA DIN 1052/1969 Bl. 1 Holzbauwerke. Berechnung und Ausführung
DIN 1052/1969 Bl. 2 Bestimmungen für Dübelverbindungen besonderer Bauart
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ANNEX

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 – TIMBER STRUCTURES

THE RUSSIAN TIMBER CODE
Summary of Contents

Bruxelles
October 1977
Chapter 4

Calculation Standards for Wooden Constructions

1. General

Subject: Supporting wooden structures, beam calculations.

Standards established in terms of:

- utilisation
- fabrication
- transport
- erection.

Also in terms of the type of construction.

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- for other uses (Table 10)
- in order to take into account additional charges (Table 11)
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    - Constructions in plywood.
    - Plywood panels.

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Reminder of the resistance of pine and fir to 15% humidity. CF Table 3 and 4.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DRAFT RESOLUTIONS OF ISO/TC 165

Bruxelles

October 1977

1. Title of TC 165

The Secretariat of TC 165 is requested to submit to the Central Secretariat of ISO for adoption by the ISO Council, the following title for TC 165:-

in English: TIMBER STRUCTURES
in French: STRUCTURES EN BOIS

2. Scope of TC 165

The Secretariat of TC 165 is requested to submit to the Central Secretariat of ISO, for adoption by the ISO Council, the following scope proposed for this committee:-

Standardization concerning the design of loadbearing structures of timber, wood-products and appropriate related lignocellulosic fibrous materials.

The prime concern of the technical committee will be the preparation of an International Standard comprising the technical requirements for the design together with additional requirements, regarding the materials and the work of construction, necessary to safeguard the validity of design assumptions made or implied.

The standard shall be formulated in such a manner that it gives the greatest freedom for design and construction compatible with satisfactory technical performance and safety over the life of a structure.

The standard shall refer to necessary supporting standards, in particular regarding test methods necessary for the verification of stipulated requirements. The preparation of such supporting standards should only be undertaken by this technical committee if they do not lie within the field of work of an existing technical committee or if such technical committee is not in a position to provide them.

3. Organisation of the work of TC 165

TC 165 has agreed to organize its work as follows:

The work shall, where applicable, build on the work and results of existing ISO/TCs with which the necessary liaison must be established to avoid duplication of work and to ensure the best distribution of the work involved. Relevant in this respect are the present TCS 55, 59, 89, 139 and 151.

As far as basic and general principles are concerned, the work shall be based on the work and results of ISO/TC 98, Bases for Design of Structures, with which committee a close liaison shall be maintained.

CIB working group W18 has agreed to prepare the first drafts for TC 165. Close liaison shall be maintained with this working group.

To the extent that this is not already secured through the cooperation with CIB W18, liaison shall further be established with other relevant organizations working within the scope of the committee, for example the CRS, the EEFM, the EEC, the FEBN/Sous-Commission "GLULAM", the RILEM and the UN-ECE.
4. List of contents of CIB W18 draft

CIB W18 is requested to submit to the secretariat of TC 165 the proposed list of contents of its draft indicating the chapters already completed and those in course of preparation.

The Secretariat of TC 165 is to submit this list to the members of TC 165, inviting their comments and suggested order of priority.

The Secretariat will decide, if necessary, to convene the committee, to discuss the comments received, in order to establish the programme of work for the committee.

5. Testing of mechanical joints

TC 165 agreed to postpone the date for the receipt of comments on document 165 N 13 from 3rd October to 31st December 1977. TC 165 noted that appendix A of that document should be excluded for the present. The timetable for further drafting work on this subject will depend on the replies received by the secretariat in connection with the consultation envisaged in resolution no. 4 concerning the choice of priorities.

6. Testing of structural plywood

TC 165 agreed that before any decision concerning the method of studying this problem, consultation shall be undertaken with the CIB W18/RILEM committee, requesting then:

a. to study in liaison with TC 89, 139 and 151 the possibility of generalizing the method of document N 14 to cover also other types of panels;

b. to submit the present draft standard to ISO/TC 139 to determine whether such a draft can be accepted and included in that committee’s work in accordance with the priorities of TC 165.

7. Tests on Structural timber

TC 165 agreed that before any decision concerning the method of studying this problem, ISO/TC 55 shall be consulted to determine its views on document 165 N 15 and whether it can accept such a draft as the basis for study in its programme of work in accordance with the priorities of TC 165.

8. Chairmanship of TC 165

TC 165 decided to nominate Mr. Hans J. Larsen, Denmark, as chairman for the next three years. The secretariat of TC 165 is requested to submit this nomination to ISO Central Secretariat for appointment by ISO Council.

Pour autant que ce soit pas déjà assuré à travers la coopération avec la CIB W18, une liaison doit ultérieurement être établie avec d’autres organisations travaillant dans le même domaine que le Comité, par exemple, la Commission Economique pour l’Europe, le CIB, le RILEM, le FIHIS/Sous-Commission “GLUMA”, la CCE et le CECM.

4. Sommaire du programme du CIB W18

Le CIB W18 est prié d’adresser au Secrétariat du TC 165 le sommaire de son programme en précisant les chapitres qui sont terminés et ceux qui sont en cours d’études.

Le Secrétariat du TC 165 est chargé de soumettre le sommaire aux membres du TC 165 en leur demandant leurs remarques et l’ordre de priorité qu’ils proposent pour leur étude. Il appartiendra au Secrétariat du TC 165 de réunir, le cas échéant, le Comité technique pour discuter des réponses reçues en vue d’établir le programme de travail du Comité.

5. Essais des joints mécaniques

Le TC 165 est d’accord pour reporter la date de clôture de l’enquête par correspondance sur le document 165 N 13 du 3 octobre au 31 décembre 1977. Le TC 165 a noté que l’appendice A de ce document doit être exclu pour le présent. Le calendrier de la poursuite de l’étude de ce sujet dépendra des réponses reçues par le secrétariat lors de la consultation prévue par la résolution no. 4 pour ce qui concerne les choix des priorités.

6. Essais des contre-plaques de construction

Le TC 165 est d’accord pour qu’avant de prendre toute décision concernant la manière d’étudier ce problème une consultation soit menée auprès du comité CIB/RILEM à fin:

a. qu’il étudie en liaison avec les comités ISO/TC 89, 139 et 151 la possibilité de généraliser la méthode du document 165 N 14 à d’autres panneaux;

b. qu’il soumettre à l’ISO/TC 139 le présent document pour connaître son avis sur le contenu de ce document et son accord pour l’inclure à son programme de travail selon les priorités qui seront décidées par le TC 165.

7. Essais des bois de construction

Le TC 165 est d’accord pour qu’avant de prendre toute décision concernant la manière d’étudier ce problème, l’ISO/TC 55 soit consulté pour connaître son avis sur le contenu du document 165 N 15 et son accord pour l’inclure comme base d’étude à son programme de travail selon les priorités qui seront décidées par le TC 165.

8. Présidence du TC 165

Le TC 165 a proposé d’éliminer M. Hans J. Larsen, Danemark, comme président permanent pour les trois prochaines années. Le secrétariat du TC 165 est prié de soumettre cette candidature au Secrétariat Central de l’ISO pour accord du Conseil de l’ISO.