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STUDIES AND DOCUMENTATION

MEETING OF WORKING COMMISSION W18

TIMBER STRUCTURES

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Inhalts-
verzeichnis
S. 25

CONTENTS

1. LIST OF DELEGATES

	Dr L G Booth	United Kingdom
	Mr O Brynildsen	Norway
	Mr W T Curry	United Kingdom
	Mr A R de Freitas	Brazil
	Mr G D Grainger	United Kingdom
	Mr B Hochart	France
	Mr M Johansen	Denmark
	Professor H R W Kühne	Switzerland
	Dr J Kuipers	Holland
	Professor H J Larsen	Denmark
(1)	Dr R H Leicester	Australia
	Mr E Levin	United Kingdom
	Professor K Möhler	West Germany
	Professor B Norén	Sweden
	Mr P O Reece	United Kingdom
	Professor E G Stern	USA
(2)	Mr J G Sunley	United Kingdom

(1) Corresponding delegate

(2) Coordinator W18 and Chairman for meeting

2. AGENDA

1 Chairman's Address

2 The Future of W18

3 A comparative study of existing national design codes for timber structures

   (a) Application and basic principles of existing codes
   (b) Future development and changes

4 Limit State Design

   (a) Basic principles
   (b) Partial safety factors
   (c) Deflection criteria

5 Strength grading of timber - Methods of grading, working stresses and moduli of elasticity

6 Timber Joints - Design loads, methods of jointing and fasteners

7 Performance and functional requirements of floors and sub-floors

8 Future programme of work

9 Time and place of next meeting

3. CHAIRMAN'S ADDRESS

Mr J G Sunley welcomed the delegates to the meeting and gave a brief outline of the history of Working Commission W18. He explained that W18 was formed early in 1959 as a study group on timber structures but little was achieved in this direction. In 1961 and 1965, CIB through W18, supported two International Conferences on timber engineering in Britain, the first at Southampton and the second in London. Since 1969 the Commission has been inactive and Mr Sunley explained that the present meeting had been called to discuss the future of W18 and if it was decided that the group could perform a useful function within the field of timber structures it would be necessary to draw up a programme of work. He pointed out that there already existed a number of international organizations working within this field and in drawing up a programme of work for W18 it would be essential to avoid duplicating or overlapping the work of other groups such as RILEM, IUFRO, ISO and CEN. W18 was fortunate in this respect as many of its members were also closely connected with these other organizations and, therefore, a close liaison between the different groups could be maintained. For this reason, Mr Sunley particularly welcomed Professor HRW Kühne and Dr J Kuipers to the meeting as representatives of the RILEM 3-TT Committee.

4. THE FUTURE OF W18

MR SUNLEY opened the discussion by again emphasizing the need, in planning the future of W18, to avoid duplicating work already being undertaken by other groups. In order to identify the groups already working on timber and its utilization, he submitted to the meeting a survey carried out by Professor P Sonnemans (App 1/1) for the RILEM 3-TT Committee, which lists both the International and European organizations working in this field. MR SUNLEY went on to explain, as Chairman of the IUFRO Working Group on Structural Utilization, how this particular group provided a world-wide forum for timber engineers to discuss current research projects, to present papers on selected topics, and to coordinate research programmes. He pointed out that it did not work on the drafting and development of national design codes and standards for timber structures and he thought that this was an area where W18 might function effectively.

PROFESSOR KÜHNE as a representative of RILEM 3-TT Committee explained that RILEM is a world-wide organization of building testing laboratories and Committee 3-TT is the group which deals with the testing of timber. At present they are in the process of establishing a programme of work and PROFESSOR KÜHNE thought that this may include work on main load bearing members and long term testing. He did not think that RILEM would directly concern itself with the development and rationalisation of national design codes and standards.

A general discussion followed from which it emerged that the delegates considered that there was a useful area of work for W18 to undertake and therefore it should not be disbanded. MR SUNLEY suggested, and it was generally accepted, that the terms of reference would be, "To study and highlight the major differences between the relevant national design codes and standards and suggest ways in which the future development of these codes and standards might take place in order to minimize or eliminate these differences". It was further agreed that initially

W18 should concern itself primarily with European codes and standards with the other participating countries, ie Canada, USA, Brazil and Australia, etc acting as advisers and contributing experience. At a later date the comparison would be broadened to a world-wide basis. (It emerged later that in laying down principles of Limit State Design all countries could have a definite interest.)

PROFESSOR LARSEN pointed out that the field in which it was suggested W18 should function was very close to the work of ISO and CEN and he did not think it would be possible for W18 to write an international design code for timber structures which would be accepted either on a world-wide basis or even a European one. For this reason he thought it important to make ISO and CEN aware of the work of W18 so that when these organizations come to consider the drafting of any codes or standards dealing with timber structures W18 would be able to exert a strong influence by virtue of the progress it had made towards the rationalization of the existing national codes and standards. Conversely W18 should have due regard for the work of ISO and CEN, particularly when considering the loadings for which structures are to be designed. The ISO group TC98 is responsible for establishing the imposed loads for structures and these will be applicable universally to all structures and materials.

Finally, having agreed in principle the work which W18 could most usefully undertake, the Chairman suggested that further discussion on this topic should be deferred until later in the meeting when the future programme of work would be discussed. This was agreed by the delegates.

5. EXISTING NATIONAL BUILDING REGULATIONS AND DESIGN CODES FOR TIMBER STRUCTURES

BRAZIL

MR de FREITAS said that at present there are no national building regulations in Brazil and building controls are issued and enforced by local municipal authorities within the boundaries of urban areas. The degree of control and technical sophistication varies between different authorities although the new and smaller authorites tend to follow the principles and methods of the larger city authorities. Of these, the City of São Paulo has one of the oldest building codes which serves as a basis for the majority of present codes of other authorities. It was established in 1927 and is under constant revision by a permanent code revision committee composed of representatives of the city government, and of engineering and architectural associations. The code lists minimum requirements for ventilation, illumination, room dimensions and water and electrical installations but does not include minimum requirements for structural design.

Within the areas where building control exists it is necessary for a registered engineer or architect to assume responsibility for the design and construction of any building works. Although it is open to such an engineer or architect to use any design method he thinks suitable there are in existence national design and construction standards, issued by the Brazilian Standards Association (ABNT) covering the use of concrete, steel and timber. However in all cases the engineer or architect in charge is held legally responsible for the work.

DENMARK

PROFESSOR LARSEN and MR JOHANSEN outlined the Danish National Building Law, which is the responsibility of the Minister for Building and controls all building works in Denmark. The Law operates through the National Building Regulations which were last revised in 1972. These Regulations lay down minimum functional requirements for buildings and where they exist refer to functional codes such as the code dealing with design loads for buildings. The Regulations also refer to National Standards dealing with specifications for building materials and Design Codes which recommend the principles and methods to be followed in the design of buildings. These standards and codes of practice are published by Dansk Ingeniørforening (the Danish Society of Engineers) and engineers may depart from them provided they can justify the deviations to the local authorities and the Ministry of Building, eg prototype testing of a structure.

FRANCE

MONSIEUR HOCHART said there was no National Building Law in France but there are national regulations dealing with loadings on structures, fire and safety. The loading regulations vary regionally to take account of variations in snow and wind loadings etc throughout the country. There also exist Standards and Codes of Practice published by a range of national organizations including engineers, contractors and insurance companies. These publications cover new materials and techniques and are gradually replacing Agrément Certificates. These Standards and Codes of Practice are not mandatory but every contractor is legally required to guarantee the buildings which he erects and this can most easily be accomplished by following the Standards and Codes of Practice. For building insurance purposes every design engineer must be able to prove the adequacy of his designs and therefore he too tends to follow the National Standards and Codes of Practice. In addition to the private insurance companies there also exists a state controlled National Insurance Company usually offering better terms than the private companies and this requires buildings to be designed and constructed in accordance with regulations published by the National Building Federation. These regulations contain most of the National Standards and Codes of Practice together with additional requirements laid down by the National Building Federation.

HOLLAND

DR KUIPERS explained that in Holland domestic dwellings were controlled by

Government Building Regulations which are mandatory and give design requirements and construction details. For the purposes of supervision of load bearing structures the country was divided into regions under the control of Building Inspectors but the interpretation of the regulations by these inspectors could vary considerably from region to region. There also exists the Dutch Normalization Institute which is responsible for publishing material standards and design codes. The standard for timber was first published in 1958 and contained grading rules for timber used for structural purposes. More recently DNI has published "Design Methods for Buildings, Part 1", which deals with the loads for which all buildings should be designed. Subsequent parts still to be published will deal with methods of design for different materials which include timber, steel, masonry and concrete. It is expected that the Part "Methods for Timber" will be published in 1973. The loading requirements in Part 1 are mandatory and will apply to all materials while the subsequent Parts dealing with specific materials will be advisory and carry no legal requirement.

NORWAY

MR BRYNILDSEN reported that the situation in Norway, was similar to the other Scandinavian countries and that all building was subject to control under the National Building Regulations. The Regulations lay down minimum functional requirements for buildings and refer to the Norwegian Standards for Design of Structures. These Standards cover steel, concrete, aluminium and timber and have recently been revised and coordinated in content and layout. They all refer to a mandatory load standard, NS 3052 "Calculation of Loading", and are published by the Society of Engineers. In addition to these Standards and Regulations the lending banks which finance building works stipulate additional conditions related to design and construction which may vary from bank to bank and which must be fulfilled before a loan is approved. Although the Standards for Design of Structures are not mandatory, any departure from them must be shown to comply with the National Building Regulations.

SWEDEN

PROFESSOR NORÉN confirmed that building control in Sweden is organized similarly to Norway and Denmark and control is exercised under the Building Act 1947 and the Building Ordinance of 1959. The building regulations are contained in the publication "Svensk Byggnorm 67" (SBN 67) which is issued and revised as required by Statens Planverk (the National Board of Urban Planning) which is the central authority for planning and building in Sweden. Svensk Byggnorm 67 consists partly of regulations which are compulsory for both builders and the authorities and partly of recommendations and directions which are optional. Linked with SBN 67 are special codes dealing with structures in steel and concrete, however at present there is no special code for timber structures. To deal with the revision of SBN 67 Statens Planverk has appointed a "Safety Group", known as S-group, which is at present drafting a general structural code for load bearing structures. To this code at a later date will be added additional codes dealing with specific materials eg steel, concrete and timber, and these additional codes will be known as "Technology Codes".

SWITZERLAND

PROFESSOR KÜHNE stated that there is no National Building Law in Switzerland and generally such building control as exists is administered by the local authorities and therefore the degree of control varies considerably from the large to the small authorities. However, in the case of buildings which belong to the Cantonal and Federal authorities there exists Cantonal and Federal Regulations, respectively, which control the design and construction of these buildings. In addition there also exists a number of intercantonal committees which make recommendations for building regulations addressed to the local or Cantonal building authorities. There also exists two Standards Organizations with the responsibility for publishing technical standards. These are Verband Schweizerischer Maschinenindustrieller (V.S.M.) with an internal standards organization (SNV) and a standards office distributing standards of all kinds including those of other countries and Schweizerischer Ingenieur -und Architekten-Verein (SAI) which deals exclusively with building standards. Neither of these organizations are controlled by the Federal authorities but the SAI Standards form the basis for contracts in the building field in Switzerland. These standards are established in cooperation with engineers, architects and contractors organizations and although they are not mandatory they are frequently used and are often adopted by the Federal building authorities. The current SAI Standards which are relevant to the design and construction of timber structures are:

No 118 - General Conditions for Building Contracts

No 160 - Basis of Load Calculation

No 122 - Conditions and Measuring Rules for Carpenter Work

No 163 - Structural Timber

No 164 - Calculation and Execution of Load Bearing Timber Structures

The first two Standards Nos 118 and 160 apply generally to all building works and materials.

WEST GERMANY

PROFESSOR MOHLER said that there did exist within the Federal Republic a Building Law but this had now been superseded by the nine regions each of which had its own building regulations which incorporated the Federal Law. These regional regulations operate in three parts, the first of which is compulsory and specifies the functional requirements and loadings to be catered for in design. This part is drafted by the National Building Committee which is composed of scientists and engineers, Federal Government representatives and contractors. The second part deals with Standards for materials and the third part with design codes for buildings and structures. This latter part is not mandatory and other methods of design may be used, including the prototype testing of structures, provided they can be justified. Provided the regional building regulations have been complied with the ultimate responsibility for building failures rests with the Federal Government via the regional authorities.

UNITED STATES OF AMERICA

PROFESSOR STERN reported (App 1/2) that at present building control in the USA is conducted at local and state levels rather than nationally although the local codes by reference are tied to one of four voluntary semi-national codes. However, the introduction of mandatory state building codes, which are designed to serve as a basis for local codes and which at present is taking place gradually throughout the USA, will have the effect of combining the four semi-national codes into a single national code. All the design codes permit exceptions and deviations provided the exceptions can be justified to the controlling authority. This allows the codes to be modified and new ideas introduced without re-drafting the entire code.

Building specifications and Standards are introduced by the Federal Authorities such as the Federal Housing Administration (Minimum Building Standards) and industrial associations such as the National Forest Products Association (National Design Specification for Stress Graded Lumber and its Fastenings).

Interpretation of building codes, regulations, specifications and standard are issued by interested organizations such as the United States Savings and Loan League in order to make their members familiar with the technical documents and the principles on which they are based. Many of these publications incorporate the best judgement of specialists in the various fields and may be considered reliable public statements.

All testing of materials and the development of materials standards is associated on a voluntary basis with the American Society for Testing and Materials (ASTM).

UNITED KINGDOM

MR CURRY said that at present in England and Wales building control was exercised by the local authorities applying the Building Regulations (1972) which relate to the health and safety aspects of buildings. The Regulations operated by calling for minimum functional requirements referring where necessary to functional codes (ie loading and wind codes) and then by "deemed to satisfy" clauses which make recommendations by reference to British Standards Institution Codes of Practice and Material Standards as to how the functional requirements could be fulfilled. Whilst the functional requirements are mandatory the "deemed to satisfy" clauses are not and it is open to the engineer to deviate from the methods of design recommended in the BSI Codes of Practice provided he can justify the new method, either by calculation or prototype testing, to the controlling local authority.

In cases of disagreement between the local authority and the engineer an appeals system exists whereby the parties can present their cases to the relevant government minister for a decision.

It is the local authority which is legally responsible for the safety of all buildings within the boundaries of the authority.

6. DEVELOPMENT OF EXISTING NATIONAL DESIGN CODES FOR TIMBER STRUCTURES

AUSTRALIA

DR LEICESTER sent his apologies to the meeting for his absence. He submitted two papers to the meeting of which the first "Australian Codes for Use of Timber in Structures" (App 1/3) outlined the principles and basis of the current Australian Timber Engineering Code AS CA 65 - 1972. The second paper "Contemporary Concepts for Structural Timber Codes" (App 1/4) was first presented at a seminar on "Timber Design and Construction in the 70's" held at the University of Auckland, New Zealand, in August 1972.

As delegates to the meeting had not had time to study Dr Leicester's papers, DR BOOTH gave a brief resumé of the Australian Timber Code AS CA 65 and pointed out that in addition to this there also exists a further code AS CA 38 "Light Timber Frame Construction". He went on to say that neither of these codes were mandatory and Australian engineers were at liberty to use methods of design other than those recommended in these codes provided they could justify them to the controlling authorities for building.

BRAZIL

MR de FREITAS reported that the Brazilian Standards Association (ABNT) issued in 1951 the Standard NB-11, which gives recommendations on the design and construction of timber structures. It is aimed particularly to the construction of bridges, roof structures, garages, warehouses, etc, and does not include the utilization of glu-lam, plywood or particle board.

This standard uses the theory of elasticity for the design of beams, columns and tension members. The permissible stresses are respectively 20%, 15% and 20% of the limit stress for compression, tension and bending, obtained from clear specimens in the green condition. Stresses due to impact loads are divided by

The new code would specify methods of test for deriving strength values for those items included in the code for which design data was not given.

PROFESSOR LARSEN also submitted to the meeting a short paper on "Limit State Design" (App 1/5) in which he discusses the principles of the design method and the choice of partial factors.

FRANCE

MONSIEUR HOCHART said that the design of timber structures in France was related mainly to two documents which applied nationally. The first known as Standard NFB 52001 dealt with the quality of the material and gave details of design stresses associated with a particular quality. The second document, Technical Unified Document 1972, specified methods and rules for the calculation and design of timber structures. It is generally recognized in France that these two documents work on a safety factor of 2.75 and the design stresses laid down in NFB 52001 were derived by dividing the mean value of the relevant test results by 2.75.

The limit state method of design was already in use in France for concrete but there were no indications that the design of timber structures would change to this method in the foreseeable future.

HOLLAND

DR KUIPERS said that the present design code for structures was published in 1955 and adopted the permissible stress method of design. It covers loadings and a number of different materials including steel and timber but not concrete. A major revision of the code was started in 1961 and the first part of the code was published in 1972. This deals generally with building construction and gives details of loadings, permitted maximum deflections, a limited amount on the methods of design and calculations and finally a section dealing with the erection of buildings. However the engineer may depart from this code provided he declares the departures on his drawings and agrees them with the local authority. This first part of the revised code also refers to subsequent parts, some still to be drafted, which deal with particular materials. The part dealing with timber, steel and concrete are expected to be available shortly and included in the timber part will be a set of permissible stresses related to the Nordic softwoods and a few hardwoods. Also included will be methods of calculation of the strength of timber structures, calculation of deflection and the acceptable limits for good building practice, design details for beams and columns including glu-lam, and details of jointing and fasteners. However it will not include details on plywood or other board materials but a limited number of data sheets have been issued on these by the Timber Research Centre.

Finally, DR KUIPERS said that Dutch timber engineers were beginning to consider limit state design and discussions had taken place with regard to a further revision of the timber code to include this method of design. He said it already existed to some degree for steel and concrete.

NORWAY

MR BRYNILDSEN said that the "Norweigian Standard for the Design of Timber Structures" would be published this year based on the limit state methods of design and related to the Load Standard NS 3052 "Calculation of Loading". The timber standard contains chapters on general requirements, materials and connectors including glu-lam loads and load effects, design principles, ultimate limit state, serviceability limit state, design rules, construction, building components, proof loading of timber structures, rules for testing of connectors and rules for prototype testing of structures.

MR BRYNILDSEN also submitted to the meeting a paper entitled "The Use of Partial Safety Factors in the New Norwegian Design Code for Timber Structures" (App 1/6).

SWEDEN

PROFESSOR NORÉN submitted a paper to the meeting entitled "Swedish Code Revision Conerning Timber Structures" (App 1/7). He then went on to say that Sweden was closely watching the Nordic Building Committee (NKB) to see if they could reach agreement on a timber design code based on limit state design. In the meantime Sweden was revising the existing national code (SBN 67) which was not based on limit state design and applied to all building in Sweden. This code includes chapters on loading and timber structures but it is limited in its references to design methods and tends to refer to accepted methods found elsewhere.

SWITZERLAND

PROFESSOR KÜHNE said that there are no official mandatory codes in Switzerland but SIA 164 published in 1953 deals with the design of timber structures. It is based on the permissible stress method of design with fixed safety factors. It gives permissible stresses for European fir and spruce, also oak, beech and ash. It includes permissible loads for fasteners in timber and lays down deflection limitations and safety factors. Finally, it deals with the manufacture of timber products such as glu-lam. At present the section dealing with safety factors is being revised and details are being included for new fasteners but this does not include patented systems such as nail plate connectors for which the manufacturers would have to provide the design data. PROFESSOR KÜHNE said that the code was deliberately kept simple as it was intended for use by craftsmen as well as engineers, and as far as he knew at present there were no suggestions regarding a second revision of the code to incorporate limit state design.

WEST GERMANY

PROFESSOR MÖHLER reported that the design of timber structures in West Germany generally followed the recommendations laid down in the DIN Standard 1052. This code contains extensive design details which allows the engineer very little freedom of choice and is based on the permissible stress method of design. The topics covered by the Standard include: validity of the regulations, proof of safety and drawings, properties of materials, rules for dimensioning, members subject to bending, members subject to tension, members subject to compression, struts and braces, allowable stresses, deflections, joints and the design of bearings. The last revision of the code was in 1969 and German engineers are beginning to consider the limit state method of design and it is possible that limit codes will be developed simultaneously for timber, steel and concrete with a view to using the same load factors for all materials.

UNITED KINGDOM

The current design code of practice for timber structures was first published in 1952 and revised in 1967. In 1971 it was converted into metric units but without any revision. This code is not mandatory and engineers can depart from its recommendations if they can justify the departure to the local authorities. The present code is based on the permissible stress method of design with fixed factors of safety. It includes sections on: materials, appliances and components also design considerations which include design stresses for a range of timbers, modification factors for types of loading and data on joint design and fastenings. It also contains recommendations dealing with workmanship and the inspection, testing and maintenance of timber structures.

The code is now being completely revised and a new code based on the limit state method of design will replace it. The reasons for adopting limit state design are set out in the report to the BSI Committee BLCP/17/2 - Working Stresses (App 1/[illegible]). A second preliminary report on the new Code of Practice was also submitted at the meeting (App 1/9). This describes the first four chapters of the new code which includes, design objectives, general requirements for materials and the effects of moisture content.

7. STRESS GRADING OF TIMBER

DENMARK

PROFESSOR LARSEN said there existed a Code of Practice dealing with the grading of timber for structural purposes and this specified three grades, T300, T200 and unclassified. The two T grades were approximately equivalent to the British Standards Institution Code of Practice CP 112 grades of 75 and 50 grade, respectively. However, PROFESSOR LARSEN said that the trade generally did not grade to the T rules and most timber used in building was "unclassified".

FRANCE

MONSIEUR HOCHART said that in France two methods of grading existed. One was operated by the sawmillers handling home grown timber which was not usually used for load bearing timber structures. The second system was defined in the Standard B5201 and was used for structural timber. Three grades are defined according to rules which take account of knots, size of members, growth etc. However, these rules are at present being revised and it is hoped to rationalize them with developments in other countries. An attempt is also being made to grade timber to a special grade which is used solely for trussed rafters the success or failure being established by prototype testing. MONSIEUR HOCHART admitted however that the rules for selecting the three structural grades were not implemented accurately and frequently a parcel of timber would be graded merely by weighing to indicate the average density and a quick inspection of the outer pieces.

HOLLAND

DR KÜIPERS stated that there has been a grading Standard in existence in Holland since 1958 which specified rules for joinery timber and stress grading. The stress grading rules are based on strength test results. The stress grades and associated design stresses were chosen on the basis that $\frac{2}{3}$ of timber imports would be used for structural purposes. Rules also exist for resorting the 3 of structural timber into two higher grades but these are rarely used and most structural timber is considered to be standard grade. The same rules are used when timber is selected for laminated structures but new rules are being drafted for this purpose.

NORWAY

MR BRYNILDSEN submitted a copy of the Norwegian Standard NS 3080 (App 1/10) to the meeting which specifies three grades "Extra" (E) and "Standard" (S), the structural grades and "Other" (C) which is non-structural. For the structural grades it also specifies design stresses for bending. The basis of the Standard is the old commercial grading rules which have been tightened up in order to select the structural grades. MR BRYNILDSEN suggested that "E" and "S" grades are probably not very different from the new "SS and "GS" grades which will shortly be introduced in the United Kingdom.

SWEDEN

PROFESSOR NORÉN said that Sweden was similar to Denmark and recognized the T grade system but neither the industry or timber engineers made use of it. He suggested that engineers should decide on stress values related to the general quality of timber available and leave it to the mills to maintain that quality.

SWITZERLAND

PROFESSOR KÜHNE reported that there were two grading systems in Switzerland, one system for structural timber and the other for joinery and non-structural work. The structural rules given in SIA 163 defined three grades based on the ratio of the knot sizes to the clear timber. Most of the structural timber used is grade 2 and the craftsmen do the selection of the 3rd grade when necessary.

The 1st grade is only sorted for particular jobs ie laminated beams etc. The rules are to be revised during the next three years and it is possible that only the first and second grades will be retained as the third grade is very poor and there is little available.

UNITED KINGDOM

MR CURRY said that the current grading rules were contained in the timber design code CP 112, but these rules were soon to be superseded by the revision of BS 1860 "Specification for Timber Grades for Structural Use" (App 1/11). This new Standard introduces a different system of grading from the old CP 112 rules based on the Knot Area Ratio (KAR) as seen in a cross-section through the timber. This system is considered to be more simple than the CP 112 system and there are only two grades specified instead of four. It has been in use in Canada successfully for a number of years. The limits for the KAR system have been chosen so that the lower grade GS, will correspond to the timber at present generally used for structural work in the United Kingdom.

WEST GERMANY

PROFESSOR MÖHLER said that structural timber was bought from the mills and then graded for use. The 1st grade was limited and used mainly for laminated structures and the 2nd grade provided the bulk of the structural timber. The grading rules were based on knot size ratios but they also included a density assessment. The rules only applied to European softwoods and imports of structural timber were limited and were usually 5ths. He admitted that the grading rules were not strictly enforced.

8. TIMBER JOINTS

PROFESSOR STERN introduced his paper "Mechanical Fasteners and Fastening in Timber Structures" (App 1/12) and because of a shortage of time gave a brief outline of the contents. He asked for comments from the delegates regarding the accuracy of the technical data contained in the report. MR SUNLEY requested delegates to write to Professor Stern with any new information on fasteners and fastening methods in their own countries so that the report could be revised where necessary to take account of these new developments. MR SUNLEY added that this sort of report on a specific topic was most useful as a starting point for discussion and he would welcome reports from other delegates at future meetings.

9. FUTURE PROGRAMME OF WORK

To sum up the two days of talks MR SUNLEY requested short comments from the delegates on the usefulness of the meeting and the direction in which W18 should move. He said in his opinion W18 should work towards coordinating the different codes and standards of each country, particularly in relation to the structural grading of timber, the derivation of design stresses, and the design approach to timber structures in general.

MR REECE said that he thought the meeting had provided a valuable opportunity for the exchange of ideas and views with people from other countries and this alone was sufficient reason for W18 to continue.

PROFESSOR LARSEN thought that W18 should operate more as a background group working towards the coordination of national codes and standards with a view to influencing bodies like ISO and CEN.

PROFESSOR KÜHNE said ISO was a very formal organization and the delegates at ISO meetings were not necessarily technical experts from each country. It also worked slowly and in many respects CEN was similar.

PROFESSOR NORÉN agreed with Professor Larsen and Professor Kühne that W18 should work with a view to influencing ISO.

MR SUNLEY suggested that when the IUFRO Congress is held in Norway in 1976 it might be worthwhile to arrange a joint meeting between CIB, RILEM and IUFRO.
MR BRYNILDSEN agreed with this and said that he thought W18 should continue at least for the three years up to the IUFRO Congress.

PROFESSOR KÜHNE said that he also thought W18 should continue but it should operate on a world-wide basis rather than a European one. He also suggested that there should be a link between RILEM and W18 or IUFRO and when RILEM next met (May 1973) this could be discussed. However, a main interest of RILEM was testing work and this was probably not of such great importance to W18. DR KUIPERS agreed with PROFESSOR KÜHNE and said that he was very interested in the future of W18 and was concerned and eager for a RILEM/W18 link.

MONSIEUR HOCHART said he did not think it was possible to achieve any large degree of uniformity between countries on codes of practice but was interested to discuss the basis of the different national codes. He thought France should participate in the activites of W18 which should have a broad brief aimed at the better utilization of timber.

MR BURGESS who was standing in for Mr Levin said he thought W18 should continue but with a restricted objective so that if the objective was the coordination of codes and standards, W18 should only concern itself with European codes.

MR de FREITAS referred back to earlier in the discussion and said he agreed with Professor Norén, Professor Larsen and Professor Kühne that W18 should operate as a pre ISO body and do the ground work for ISO to work from.

PROFESSOR STERN agreed with the other delegates that W18 should continue but he thought it should be kept on a world-wide basis.

PROFESSOR MÖHLER said he did not think the task of producing unified codes and standards should occupy the time of W18 but rather that W18 should act as an advisory body on specific topics.

MR MEECE said that he thought ISO and CEN were subject to political pressures but in their work they would undoubtedly require technical documents and providing this documentation should be the job of W18.

DR BOOTH said he thought W18 should act as a forum for research and work associated with methods of design and codes. He thought this might best be achieved by having one forum with a number of small sub-committees working on different topics.
MR CURRY said he agreed with Dr Booth.

In drawing the meeting to a close MR SUNLEY said it was clear from the discussion that delegates thought that W18 could perform a useful function working for the unification of the national design codes and standards related to timber structures. He suggested that with this in mind delegates would perhaps give some thought to the drafting of a skeleton timber design code with a view to discussions on it at the next meeting. PROFESSOR LARSEN said he agreed with this but he thought in addition individual delegates should take a specific topic from the skeleton code and compare the approaches of the existing national codes to that topic. In his own case he said he would be prepared to undertake to look at the design of timber columns. DR BOOTH replied that he thought the drafting of a unified code for the design of timber structures would prove difficult but he agreed with the approach suggested by Professor Larsen and he would be prepared to present a paper at the next meeting outlining the present position and problems associated with the use of plywood in timber structures. The remaining delegates agreed with this approach and MR SUNLEY thanked Professor Larsen and Dr Booth for their offers.

A short discussion followed as to the time and place of the next meeting. Professor Stern, Mr Reece and Professor Larsen all offered to act as hosts to the next meeting and it was finally decided to accept Professor Larsen's offer and hold the next meeting in Denmark on 25 - 26 October 1973, the precise venue to be fixed later.

APPENDIX I

PAPERS AND ARTICLES SUBMITTED AT THE MEETING

1 A note on international organizations active in the field of utilization of timber - Professor P Sonnemans.

2 √ Survey of status of building codes, specifications etc, in USA - Professor E G Stern.

3 √ Australian codes for use of timber in structures - Dr R H Leicester.

4 √ Contemporary Concepts for structural timber codes - Dr R H Leicester.

5 √ Limit State Design - Professor H J Larsen.

6 √ The use of partial safety factors in the new Norwegian design code for timber structures - Mr O Brynildsen.

7 √ Swedish code revision concerning timber structures - Professor B Norén.

8 √ Working stresses report to British Standards Institution Committee BLCP/17/2.

9 √ Revision of CP 112 - First draft, July 1972 - British Standards Institution.

10 √ Quality specifications for sawn timber and precision timber - Norwegian Standard NS 3080.

11 √ Specification for timber grades for structural use - British Standards BS 4978.

12 √ Mechanical fasteners and fastening in timber structures - Professor E G Stern.

Terms of reference

RILEM - C.T.I.B. 1972 11.04

Note on the international organizations which are active in the sphere of use of wood in construction, in its widest sense.

1	On a world-wide level	
1.1	RILEM	Committee 3-TT (Prof H R W Khüne, EMPA)
1.2	IUFRO	Division 5 (cf Report TNO H-71-X p.24) Subject Group: Wood Engineering (Mr J G Sunley, PRL) Wood Protection (Prof, Dr G Becker, BAM)
1.3	CIB	W-18 Timber Structures (Mr J G Sunley, PRL)
1.4	ISO	TC 55 (timber, terminology, defects, standard, testing) TC 92: (fire resistance of materials)
1.5	ATIBT	International Technical Association of Tropical Woods (nomenclature and uses)
1.6	IRG	Wood Preservation (sponsored by - OECD) (International Research Group W.P.) (Prof Dr G Becker, BAM) with a Group III "Methods of Treatment" (Mr Bruce, BWPA)
2	On a European level	
2.1	CEN	European Standards Committee (EEC + EFTA) GT 33 - Testing of joinery GT 38 - Wood preservation GT proposed: Structures in glued laminated wood
2.2	UEAtc	European Union of Technical Agreement in the Building Industry - c/o CSTB-Paris (A-B-BRD-E-F-GR-I-NL-P-UK) Common directives for agreement on - windows, doors, (light housing - prefabricated), light components or partitions; in the timber field
2.3	CEI-Bois	European Confederation of Timber Industries (Avenue Hoche 36, Paris) has formed a Technical Committee which could have the following members - The National Federation of Timber Industries - Section of the European Federation, such as FESYP (particle-board) which has its own special Technical Committee - the Institutes (laboratories, etc)

This Technical Committee has 3 sub-committees

SCI: Preservation and Finishing
SCII: Wood and Fire
SCIII: Problems of Manufacture

A specialist working group has also been formed: "Moisture Dynamics"
(Representatives from FPL, BAM, EMPA, CTB, IBB, TNO)

2.4 FEMIB (European Federation of Building Joinery Manufacturers)

(Section of the CEI-Bois Federation)

has formed a Committee

Timber Engineering (see 2.6)

with a sub-committee (to look at the importance of laminated wood)

Glulam

which has itself formed a technical working group

Glued Structures

2.5 CEH European Standards Committee on Wood Preservatives

(Non-official committee, but is made up of representatives of National Standards organizations with a view to reaching basic unification)

(A-B-BRD-Ch-Dk-E-F-NL-P-S-UK + Afnor GT 38)

2.6 European Softwood Conference (commercial organization)
(buyers and sellers of wood from the North)

under the control of the UK it is concerned with the problems of grading of Northern wood from the point of view of its utilization in construction.

to examine the possible application of the KAR system, such as is used in Canada, for example

In the UK, TRADA has been charged to make a thorough study.

This question has also been examined by Timber Engineering (FEMIB 2.4 which has decided to contribute to the current work and support a part of TRADA's work.

NOTE: work chiefly on stress grading of lower quality wood for general construction use (General Structural Grade GS)

they have also defined a higher grade category (Special Structural Grade SS)

because GS corresponds mainly to the Gute Klass III (DIN 4074) and SS to GK II, it would appear necessary to foresee a better grade to correspond to GKI (or also to the Constructiehout NEN), indispensable for components of laminated wood.

2.7 European meetings of Information Services on Wood Technology (Meetings rare).

P SONNEMANS
4.11.72

CENTENNIAL

COLLEGE OF ARCHITECTURE

VIRGINIA POLYTECHNIC INSTITUTE AND STATE UNIVERSITY

Blacksburg, Virginia 24061

WOOD RESEARCH AND WOOD CONSTRUCTION LABORATORY

March 27, 1973

RELEASE FOR INCLUSION IN PROCEEDINGS
covering CIB W-18 Conference
at Princes Risborough Laboratory
March 20 - 21, 1973
by E. George Stern

SURVEY OF STATUS OF BUILDING CODES, SPECIFICATIONS, ETC.

USA. -

With respect to building codes, regulations, specifications, and standards, the USA is in a state of change. This change is, to some extent, of an organizational nature.

Just as in England, where 1500 and more local codes existed prior to the introduction of the national code, innumerous mandatory local codes and regulations control building construction in the USA. Yet, almost all of the local codes are by reference tied to one of the four voluntary semi-national codes. An effort is being made by the four codes groups to unify into a single national code organization. However, it appears that such a step will not occur in the immediate future.

Yet, unification is coming as a result of the introduction of mandatory state building codes which are designed to serve as a basis for local codes and regulations. Of the 50 states, more than 14 states have already their own state codes which adopted one or more of the four semi-national building codes by reference. The Virginia Legislature adopted during its recent session two of the four semi-national codes. However, the Governor's veto killed this legislation.

All these codes permit deviations from the code whenever such deviations are justified. Thus, a way is provided for the introduction of new ideas without going through a lot of time-consuming red type, provided sufficient background material is submitted to the governing local authority. This background material has to cover satisfactorily all relevant aspects, not only strength considerations but also human factor and safety aspects. For instance, a building maybe rejected if designed satisfactorily from the stability viewpoint, but unsatisfactorily from the viewpoint of fire resistance.

Building specifications and standards are introduced by federal authorities, such as, the Federal Housing Administration (Minimum Building Standards), and industry associations, such as, the National Forest Products Association (National Design Specification for Stress-Grade Lumber and Its Fastenings). The latter specification is periodically reviewed and updated. Thus, the next edition of NDS will give consideration to joints with multiple fasteners, which in the past were treated in the same manner as joints with single fasteners.

Interpretations of building codes, regulations, specifications, and standards are issued by interest groups, such as, the United States Savings & Loan League (Construction Principles, Materials & Methods), in order to make their members familiar with the technical documents, their background, and the reasoning behind the decisions made. These interpretations incorporate the best judgement of the specialists in the various fields covered and can be considered reliable public statements.

All testing is to be undertaken in agreement with the standards developed by the American Society for Testing and Material on the basis of a voluntary consensus by all parties involved.

In contrast to the German documents which cover as many of the relevant details as feasible, the American documents leave some of the details to the designer's judgement and decision. On the other hand, the American documents do by far not give the freedom to the designer and engineer as the codes adopted by the Scandinavian countries. As an example, the official German standards provide definite information on required nail spacing; whereas the American voluntary counterpart calls for such nail spacing "as to avoid unusal splitting of the wood."

AUSTRALIAN CODES FOR USE OF TIMBER IN STRUCTURES

CONTRIBUTION
TO

CIB WORKING COMMISSION W18

by

R. H. Leicester
(Division of Building Research, CSIRO,
Melbourne, Australia)

March 1973

1. DESIGN STRESSES AND MODULUS OF ELASTICITY FOR TIMBER SCANTLING

In Australia, design stresses for timber scantling may be derived from any of the following methods:

(i) Standard tests on small clear specimens

(ii) Tests on structural scantling

(iii) Machine stress grading.

In the following, the basic design values will be denoted by R*, the characteristic values by R_k and material design coefficients by γ_m. These quantities are related by the equation

$$R^* = R_k/\gamma_m$$

a. Design Values Based on Standard Tests

Table 1 gives a summary of the method used for the derivation of the design working stresses and modulus of elasticity based on the results of standard tests on small clear specimens. One feature to note is that the visual grading rules of Australian codes that specify the permissible defects for a given grade factor are based almost entirely on the results of tests on Douglas fir in the case of softwoods and of jarrah in the case of hardwoods. Also it should be noted that in addition to the material coefficient γ_m given in Table 1, there is in effect a further load factor arising from the fact that a strength grouping system is employed in the Australian Timber Engineering Code AS CA65-1972 (Tables 1.6, 2.21, 2.22). In this Code a ratio of 1.25 is used for the specified values of compression, bending and tension design stresses of adjacent stress grades.

b. Design Values Based on Tests of Scantling

For assessing bending strength, each stick of a specified grade of timber scantling is tested at its worst defect with the defect placed on the tension edge. The characteristic value R_k is then taken to be the one percentile value and the material coefficient γ_m is taken to be 2.22 (made up of a load factor 1.25 and a duration factor of 16/9).

For assessing tension strength, the whole stick of timber is tested in tension and the characteristic value R_k for a given grade is taken to be the one percentile value. The material coefficient γ_m is taken to be 1.66 (made up of a load factor 1.25 and a duration factor of 4/3).

For assessing modulus of elasticity, each whole stick in a chosen grade is tested on the flat. The characteristic value R_k is then taken to be the mean value and the material coefficient γ_m is taken to be 1.00.

c. Design Values Based on Machine Grading

There is at present no universally accepted procedure for the derivation of design values. One favoured method in Australia is to choose the characteristic strength R_k in bending as illustrated in

TABLE 3

BASIS FOR DERIVATION OF
DESIGN WORKING LOADS FOR FASTENERS

Type of Load	Type of Fastener	Design Value $R^* = R_k/\gamma_m$	
		Characteristic Value, R_k	Material Coefficient γ_m
All	All	Mean ultimate strength of fastener metal	2.0
All	All	Mean yield of fastener metal	1.67
Withdrawal	Nails	One-percentile of max. loads	1.25
Withdrawal	Screws	One-percentile of max. loads	2.0
Lateral	Nails, Screws, Staples	(One-percentile of max. loads (One-percentile of loads at (slip of 0.015 in.	8.3 3.2
Lateral	Split rings	(One-percentile of max. loads (Average of max. loads	5.6 8.0
Lateral	Toothed plate	(One-percentile of max. loads (One percentile of loads at slip (of 0.015 in.)	2.5 1.6
Lateral	Nailed plate	(One-percentile of max. loads (One-percentile of loads at slip (of 0.015 in.	4.3 1.6

Note 1. Where two sets of characteristic values and material coefficients are cited, the set to be used is that leading to the smaller design working load.

Note 2. Slip refers to displacement between adjacent lapping surfaces.

Fig. 1 and to take the material coefficient γ_m to be 2.5. For the modulus of elasticity, the design value R* is taken to be $1.3E_o$, where E_o is the cut-off MOE value set on the machine, as shown in Fig. 1.

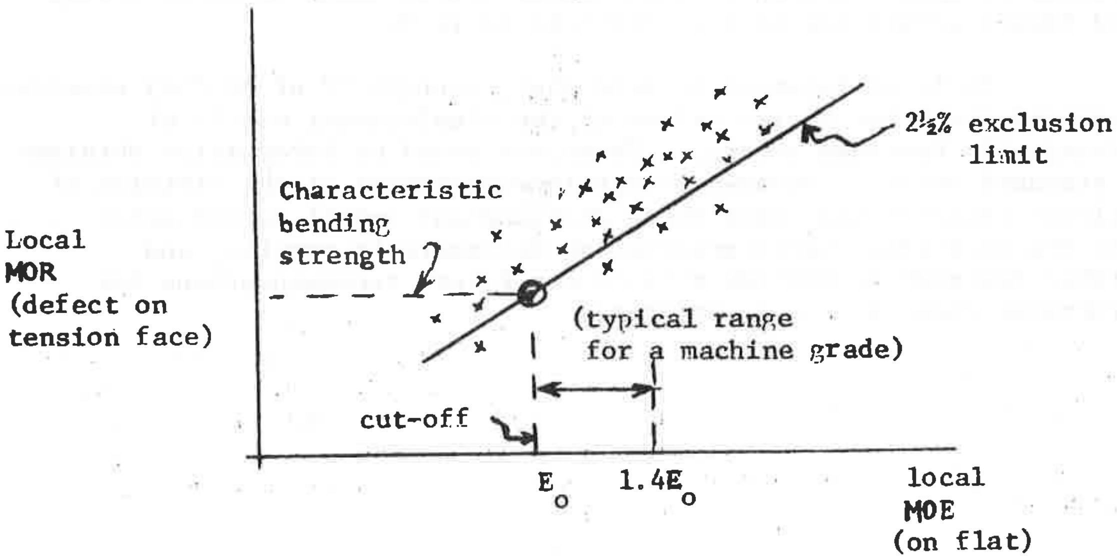

FIG. 1. ILLUSTRATION OF CONCEPTS FOR MACHINE STRESS GRADES

d. Comment

At present a comparative assessment is being made of the three methods used for the derivation of design values. One aspect, for example, is to determine to what extent design stresses based on standard tests of small clear specimens should be lower than those derived from scantling tests due to the fact that in the case of standard tests some allowance must be made for the fact that there is an uncertainty associated with the assumed effect of defects. In a more general context, a comparative assessment is also being made of the methods of choosing design stresses of timber and those of other structural materials. Another area of current investigations is a review of visual grading concepts in order to devise more suitable techniques than those currently in use.

2. LOAD SHARING FACTORS

In common with many other structural timber codes, the Australian AS CA65 recommends load sharing factors that are independent of species. However, this assumption is in conflict with published research results and consequently a research program to measure the load sharing characteristics of some Australian species has been initiated.

3. DESIGN LOADS FOR JOINTS

In Australia, design loads are derived through a set of standard tests specified in the Standard A188-1972 "Determination of Basic Working Loads for Metal Fasteners for Timber". A summary of the criteria is given in Table 3. As for the scantling, the system of strength grouping used in AS CA65 (Table 4.1.1) leads to an effective load factor additional to that given in Table 3.

It is of interest to note that Appendix H2 of AS CA65 provides recommendations for design values of the displacement moduli of mechanically fastened joints. These are based on information obtained in standard tests. However recent investigations at the Division of Building Research show that these displacement moduli do not agree with the load-slip characteristics of fasteners in service, and further research to provide a basis for future recommendations for structural codes is now in progress.

TABLE 1

BASIS FOR DERIVATION OF DESIGN VALUES FOR SCANTLING FROM STANDARD TESTS ON SMALL CLEAR SPECIMENS

Property	DESIGN VALUE $R^* = R_k/\gamma_m$	
	Characteristic Value, R_k	Material Coefficient γ_m
Tension	One-percentile of F_b^1	2.78/GF
Bending strength	One-percentile of F_b^1	2.22/GF
Compression strength parallel to grain	One-percentile of F_c^1	1.67/GF
Compression strength	Mean limit of proportionality in compression perpendicular to the grain test	1.33
Shear strength of beams	Mean F_v^1	4.2/GF
Shear strength of joint details (See Note 2)	Mean F_v^1	4.7
Modulus of Elasticity	Mean	$(0.75/GF)^{0.5}$

Note 1. F_b^1, F_c^1 and F_v^1 are ultimate strengths in bending, compression and shear in standard tests on small clear specimens.

Note 2. Material assumed to be clear at joints.

Note 3. GF = grade factor =

(bending strength of structural scantling containing maximum permissible defect)/(bending strength of small clear specimen cut from scantling)

The following are typical grade factors used in Australian grading rules:

- select grade GF = 0.75
- standard grade GF = 0.60
- building grade: GF = 0.48
- common grade: GF = 0.375

4. BUCKLING STRENGTH OF TIMBER STRUCTURES

See Section 3 of the paper "Contemporary Concepts for Structural Timber Codes" by R. H. Leicester. An additional aspect not covered in the above paper is the effect of lateral restraints on buckling strength. Appendix D of AS CA65 deals with this topic but the recommendations therein are considered to be unsatisfactory. A more acceptable design method has since been derived but is not yet included as an amendment to the original code.

5. FRACTURE STRENGTH OF TIMBER STRUCTURAL MEMBERS

See Section 4 of the paper "Contemporary Concepts for Structural Timber Codes" by R. H. Leicester.

6. LIMIT STATE DESIGN

In Australia, a Standards Association sub-committee has been set up to provide guidelines for writing structural codes in the Limit State format; and it is hoped that all Australian structural codes will eventually follow these guidelines. During the meetings of this sub-committee, considerable debate and disagreement arose whenever attempts were made to specify methods for the derivation of characteristic values and design coefficients. Part of the reasons for this are to be found in the following difficulties:

(i) Some of the methods that have been recommended for the evaluation of characteristic values lead to ridiculous results when applied to strengths, such as timber in tension, which have large coefficients of variation.

(ii) There was considerable disagreement as to whether rational methods or intuition should be used as the basis for the derivation of design coefficients.

(iii) The method for dividing the total "load factor" into resistance and action design coefficients appears to differ between various codes.

(iv) The correct factors for load combinations have not yet been satisfactorily resolved.

(v) There is disagreement as to the most suitable format for design coefficients when combining materials into a structural element (e.g. reinforced concrete beams, glulam beams), and when combining elements into a complete structure (e.g. floor grids, trusses).

Because of these difficulties, the current draft document of the Limit State Design sub-committee is limited essentially to definitions and terminology. The only specific recommendations are contained in the appendices which are enclosed with these notes and titled "Proposed Appendix for Document BD/5/1/72-3". The intent of these recommendations is to delineate between the responsibilities of the material and loading codes in such a way that each one may be developed independently. For example, it is essential to decide whether the material or loading code should provide factors for the importance of a structure. The method of delineation is based on the results of a few simple reliability studies.

PROPOSED APPENDIX FOR DOCUMENT BD/5/1/72-3

APPENDIX A

CHARACTERISTIC VALUES

A.1 Method of Specification

Characteristic values shall be specified in loading codes for actions (and action effects) and in material codes for resistances. Characteristic values other than as recommended in the following may be used, but the method for their derivation shall be clearly indicated.

A.2 Recommended Characteristic Values

It is recommended that characteristic values for actions and action effects shall be taken as the 95-percentile probability values (i.e. $Q_k = Q_{0.95}$, $S_k = S_{0.95}$), and that characteristic resistances shall be taken as the 5- percentile probability values (i.e. $R_k = R_{0.05}$).

An acceptable alternative when statistical data are not available is to take the characteristic values to be the nominal values given in Standards, Codes of Practice or other regulations.

APPENDIX B

COEFFICIENTS FOR DESIGN VALUES

B.1 Design Coefficients for Action and Action Effects

Design coefficients γ_s for application to characteristic action and action effects shall be specified in loading codes. They will be chosen both for the case of single actions and combination of actions in such a way that the resulting design action or action effects shall be 95-percentile values for design against collapse limit states and shall be mean values for design against serviceability limit states.

B.2 Design Coefficients for Resistance

Design coefficients γ_m for the resistance of structural materials, structural members and complete structures shall be specified by structural material codes. These coefficients shall take into account the importance rating of the structure, the mode of failure and also whether the loading is a normal one or has only a rare chance of occurring.

COMMONWEALTH SCIENTIFIC AND INDUSTRIAL RESEARCH ORGANIZATION

DIVISION OF BUILDING RESEARCH

CONTEMPORARY CONCEPTS FOR STRUCTURAL TIMBER CODES

by

R. H. Leicester

Seminar on Timber Design and Construction in the 70's.,
University of Auckland, New Zealand,
August 1972

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MELBOURNE

CONTEMPORARY CONCEPTS FOR STRUCTURAL TIMBER CODES

by

R. H. LEICESTER*

1. INTRODUCTION

It is generally accepted that the specification of design recommendations in terms of sophisticated concepts is undesirable for structural codes, particularly for timber structural codes. Not only does this make the design procedure uneconomic with respect to design time, but also the sophistication of concepts is not compatible with the imprecision of data (such as loads and strengths) on which structural design is normally based.

There are however many sound reasons for the use of the most sophisticated contemporary structural concepts in the preparation of a code. Even when structural data are meagre, the concepts provide some idea of relative strength values. This will indicate which structural parameters are sufficiently influential that they should be included in codified design recommendations, and will also indicate where further test data are urgently required.

A few years ago during the course of preparation of draft recommendations for the first Australian timber engineering code AS CA65, *Australian Standard Code of Practice for the Use of Timber in Structures*, (Ref. 1), it became apparent that many aspects of the existing timber structural codes (Ref. 2, 9, 10, 12) and the Australian design handbook (Ref. 26) were not in strict compliance with the corresponding concepts of contemporary structural theory. As a result, research programs were initiated into several of the more contentious areas of timber codes, and where feasible the findings were incorporated into AS CA65. In the following, the basic concepts of three of these investigations will be discussed. They are concerned with aspects of reliability, buckling and fracture theory.

2. STRUCTURAL RELIABILITY

2.1 Variability in Structural Properties of Timber

In the design of timber structures, the stress analysis and proportioning of member sizes are carried out with the same degree of sophistication as is done for steel and reinforced concrete structures. However, as is indicated in Table 1 and Ref. 13, 14, 22 and 23 there can be considerably more variability in the strengths of timber structural elements than for steel and reinforced concrete members. Consequently in the preparation of modern timber codes it is important that the effects

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TABLE 1

COEFFICIENTS OF VARIATION FOR
SOME STRUCTURAL ELEMENTS

Structural Element	Typical Range of Coefficient of Variation of Strength (%)
Steel Members	5 - 15
Reinforced Concrete Beams	10 - 25
Timber Scantling (Graded)	
- tension	35 - 60
- bending	25 - 45
- compression	15 - 20
Metal Connectors for Timber Structures	10 - 15

of variability be correctly taken into account and that design specifications be in accordance with contemporary concepts of reliability theory. Without the use of a rational framework, the extrapolation of traditional approaches to variability may lead to unsatisfactory design recommendations.

2.2 A Reliability Concept for Structures

Structural codes frequently specify more than one method for assessing the adequacy of a structure. For example, AS CA65 permits an assessment to be made from any of the following:

(i) Computations based on specified safe working stresses for structural elements

(ii) Prototype testing of complete structures

(iii) Proof testing of complete structures.

Furthermore, the basis for each of these methods may have several forms. For example, the derivation of specified safe working stresses can be made from the results of tests on

(i) Small clear timber specimens

(ii) Visually graded structural scantling

(iii) Machine graded structural scantling.

Table 2 gives a comparison between the current English and Australian methods for the derivation of design working stresses and modulus of elasticity from tests on small clear specimens. It shows that even for this most traditional method of derivation there is not complete agreement between two countries that maintain a good exchange of technological information.

It is apparent therefore that there is a strong necessity for a unified concept of safety to use as a basis for the various specified methods of assessing the adequacy of a timber structure. In addition there are currently moves in many countries (including Australia) to have all structural codes comply with a recommended "Limit State" format (Ref. 4, 5 and 6) and this has focussed attention on the relative safety of structures constructed with different materials. One contemporary concept of structural design that is useful in this respect is illustrated in Fig. 1. It is essentially the idea that structural design is a decision process involving cost; and that the correct or optimum decision is the one that leads to the least total cost (Ref. 7 and 29). The decision is based on test data (e.g. strengths of scantling timbers), previous experience (e.g. the likely range of coefficients of variation of the relevant structural properties) and subjective judgements (e.g. the effective cost of intangible consequences of failure such as the loss of life or the damage to the reputation of the structural engineer). The cost of a decision is taken to include both the cost of the structure and the additional effective cost of failure should this occur.

The following is a simple example of a cost function $C(n)$ that may be used to derive safe working stresses for a particular type of structural member;

$$C(n) = A + (n/n_o)^{\alpha} + C_F \cdot p_F(n) \qquad \ldots\ldots\ldots (1)$$

TABLE 2

METHOD FOR DERIVATION OF BASIC WORKING STRESSES AND MODULUS OF ELASTICITY FOR GREEN TIMBER FROM TESTS ON SMALL CLEAR SPECIMENS

Property	Basic Working Value = R_K/γ_m			
	British Standard B.S. CP112		Australian Standard AS CA65	
	Characteristic Value, R_K	Material Coefficient* γ_m	Characteristic Value R_K	Material Coefficient* γ_m
Tension strength	one-percentile value of f_b	2.25/GF	one-percentile value of f_b	2.78/GF
Bending strength	one-percentile value of f_b	2.25/GF	one-percentile value of f_b	2.22/GF
Compression strength parallel to the grain	one-percentile value of f_c	1.40/GF	one-percentile value of f_c	1.67/GF
Shear strength of beams	one-percentile value of f_v	2.25/GF	mean f_v	4.2/GF
Modulus of elasticity for computing beam deflection	mean E	1.00	mean E	$(0.75/GF)^{0.5}$

*GF = grade factor = ratio of strength in bending of structural timber to that of small clear specimens.

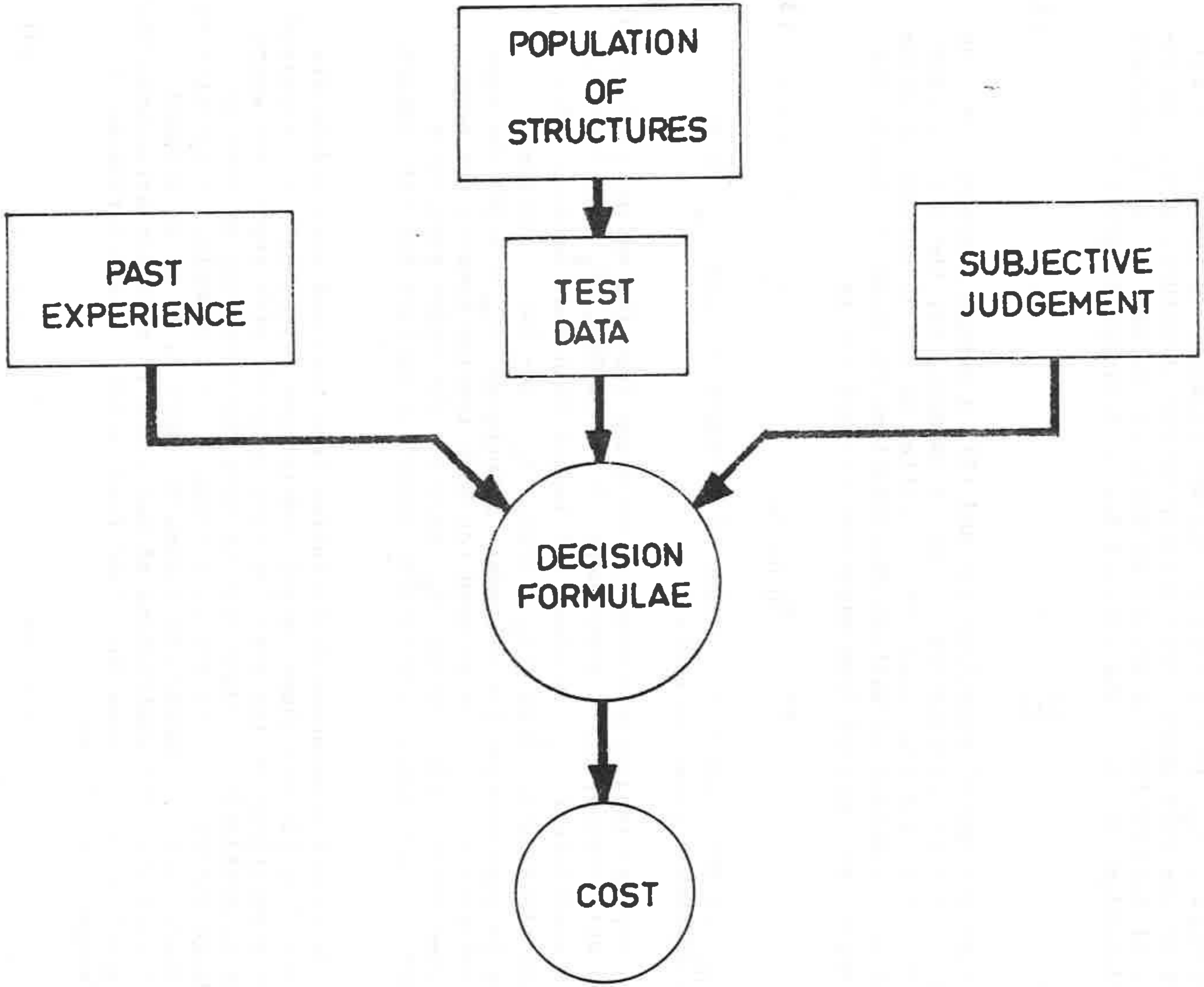

FIG 1. FORMAT FOR DECISION BASED ON COST OPTIMIZATION

where $n = \bar{R}/\bar{S}$ is the ratio of mean values for strength R and load action S, n_o is the optimum choice of n, α is the parameter relating structural costs to strength, C_F is the cost of failure relative to the cost of all the members of the type under consideration in the structure, $p_F(n)$ is the probability of failure for the specified value of n, and A is a constant that is independent of n. The correct design decision is given by

$$\frac{\partial C}{\partial n} = 0 \qquad \ldots\ldots\ldots\ldots (2)$$

With the use of equations (1) and (2), a design strength R^* can be derived for specific situations. As an example, for the typical case $\alpha = 0.5$, $V_S = 0.2$, a design load S^* specified as the 95-percentile value $S_{0.95}$, and an assumption of Weibull distributions for both R and S, the design strength R^* derived from equations (1) and (2) is

$$R^* = R_{0.05}/\gamma_m \qquad \ldots\ldots\ldots\ldots (3)$$

where $R_{0.05}$ is the 5-percentile value of strength R and γ_m is a *material coefficient* given in Table 3.

A typical implication that can be drawn from the values in Table 3 is that for multi-storey steel structures (C_F = 30-100, V_R = 10-20%) the appropriate coefficient γ_m is about 20% larger than that necessary for light framed timber structures (C_F = 3-10, V_R = 20-40%). Another implication is that whereas for secondary structures (C_F = 3-10) the same material coefficients γ_m may be used for both tension members and beams of timber (see Table 1 for typical V_R values), for major structures (C_F = 30-100) a much larger coefficient should be applied for the tension members.

In practice numerous distribution functions and cost models are fitted to equation (1) and these largely subjective assessments are adjusted so that the predictions of the cost function coincide with experience where information is available. Once this is done the model can be used to provide design information for new situations. For example, in addition to the assumptions used to compute the values in Table 3, if it is assumed (conservatively) that the coefficient of variation is zero for structures that survive a proof load P, then the proof load factor λ which relates P with the design load S^* by

$$P = \lambda S^* \qquad \ldots\ldots\ldots\ldots (4)$$

can be obtained by minimization of the cost function $C(\lambda)$ specified as

$$C(\lambda) = A + (\lambda/\lambda_o)^{\alpha} + C_F G_S(P) \qquad \ldots\ldots\ldots\ldots (5)$$

where $G_S(P)$ is the probability that load S will exceed P and S^* is the design load. Some values computed with equation (5) are given in Table 4. The values indicate that for a typical light timber framed structure (C_F = 10), the proof load factor λ to allow for variability should be about 1.2 for dead loads ($S^* = \bar{S}$, $V_S = 0.1$), 1.5 for specified design wind loads ($S^* = \bar{S}$, $V_S = 0.2$) and 1.2 for floor live loads ($S^* = S_{0.95}$, $V_S = 0.3$).

TABLE 3

MATERIAL COEFFICIENTS FOR VARIABILITY

(See equation (3))

Relative Consequence of Failure	Material Coefficient γ_m		
	V_R=0.1	V_R=0.2	V_R=0.4
C_F = 3	0.99	0.92	0.74
C_F = 10	1.09	1.13	1.15
C_F = 30	1.19	1.36	1.73
C_F = 100	1.31	1.67	2.71
C_F = 300	1.44	2.03	4.06

TABLE 4

PROOF LOAD FACTOR

(See equation (4))

Method for Specification of Design Load	Relative Consequence of Failure	Proof Load Factor λ			
		V_S=0.1	V_S=0.2	V_S=0.3	V_S=0.4
$S^* = \bar{S}$	C_F = 3	1.21	1.44	1.68	1.92
	C_F = 10	1.23	1.50	1.80	2.15
	C_F = 30	1.25	1.54	1.90	2.32
	C_F = 100	1.26	1.59	1.99	2.48
	C_F = 300	1.27	1.62	2.06	2.61
$S^* = S_{0.95}$	C_F = 3	1.06	1.10	1.12	1.13
	C_F = 10	1.08	1.14	1.21	1.27
	C_F = 30	1.09	1.18	1.28	1.43
	C_F = 100	1.11	1.21	1.34	1.47
	C_F = 300	1.13	1.24	1.38	1.54

Simple cost functions of the type given in equations (1) and (5) can be used to derive a great variety of structural design criteria. For example, they can be used to determine whether a structural code should make an allowance for the occurrence of rare events such as for the occurrence of excessively weak glued joints or of domestic gas explosions. Simple cost functions may also be employed to choose the appropriate modulus of elasticity to be used for deflection computations.

2.3 Effect of Variability on the Strength Characteristics of Timber Structures

Structural codes are phrased in the language of deterministic concepts and consequently a code for the design of timber structures must contain numerous factors to account for the strength characteristics that are due solely to material heterogeneity. One such example is the effect of the method of loading. Since a structure constructed from a heterogeneous material will tend to fail at its weaker locations, the volume of material subjected to high stress levels is important. A simple example of this effect is illustrated in Table 5 which gives the measured effect of method of loading on the nominal modulus of rupture of a large sample of kiln-dried, pith-included slash pine (*Pinus elliottii*) scantling (Ref. 19). For comparison Table 5 also shows values that are predicted for an assumed Weibull distribution of strength (Ref. 37) with a coefficient of variation of 25%. (This coefficient of variation was the measured "within-stick" value). The large effects of the type shown in Table 5 appear to be neglected in all timber codes.

Another similar omission in timber codes is the "weakest link" effect for members connected in series sequence such as those of a truss. To account for the fact that the load capacity of such a structure is governed by its weakest member, a modification factor, to be denoted by ψ , should be applied to the specified design working stresses. For an assumed Weibull distribution of strength this factor for a structure containing N members is approximately

$$\psi \simeq N^{-V_R} \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

where V_R is the coefficient of variation of material strength. For an 11 member truss with V_R = 15%, this modification factor is 0.7. This is a large factor and even if the strength distribution of truss members do not follow the Weibull distribution, the complete neglect of the weakest link effect would appear to be a dangerous practice.

One type of modification factor related to variability that is specified in most timber codes is the load-sharing factor that arises in parallel structural systems. In these systems the weaker members have the opportunity to shed some of their load on to the stronger ones. An example of such a system is a beam grid. With the aid of a computer, the load-deformation characteristics of a beam grid can be determined from the characteristics of single beams. Table 6 gives some load-sharing factors computed in this way for a grid system with five main beams. The values for dry slash pine and green Douglas fir are based on data from Ref. 31 and Ref. 11 respectively. The parameter β=0.1 chosen for the case of finite stiffness corresponds to the type of grid for which timber codes provide a load-sharing factor. The case of infinite stiffness corresponds essentially to a nailed veritcally laminated beam. It is to

TABLE 5

EFFECT OF METHOD OF LOADING ON BENDING STRENGTH

Method of Loading	Average Modulus of Rupture for Dry Pith-in Slash Pine		
	Actual Measured Value, p.s.i.	Relative Value	
		Measured	Predicted
Concentrated load at centre	8900	1.00	1.00
Concentrated load at third points	7430	0.84	0.81
Equal end moments	-	-	0.69

TABLE 6

LOAD SHARING FACTORS FOR 5-BEAM GRID SYSTEM

Stiffness of Transverse Members	Load Sharing Factor for Design Strengths Based on 5-percentile Values			
	Single Point Load at Centre of Central Beam*		Uniformly Distributed Load	
	Dry Slash Pine	Green Douglas Fir**	Dry Slash Pine	Green Douglas Fir**
Zero stiffness	1.00	1.00	0.84	0.82
Finite stiffness ** (β=0.1)	1.05	1.37	0.93	1.15
Infinite stiffness	1.24	2.40	1.21	2.40

*Load-sharing factor for point loads is additional to the load reduction factor that would be applied for the lateral distribution properties of grid systems with beams of a homogeneous material.

**Factors cited for this case are derived by approximate computation.

be noted that contrary to the predictions of the averaging model once proposed by A.S.T.M. (Ref. 8), the green Douglas fir with the smaller coefficient of variation of 30% exhibits a larger load-sharing effect than the dry slash pine with a coefficient of variation of 41%. This is due to the more "ductile" load-deformation characteristics of the green Douglas fir as shown in Fig. 2. "Ductile" load-deformation curves and the consequent good load-sharing characteristics have also been measured on green messmate stringybark (*Eucalyptus obliqua*), a common Victorian hardwood. These results indicate that timber codes should take into account the timber species and/or moisture content in specifying modification factors for load-sharing effects.

3. BUCKLING STRENGTH

3.1 Columns

A dilemma arises in writing code recommendations for slender structures due to the fact that although many factors are known to affect buckling strength, the lack of relevant data on real structures discourages the decision to formulate comprehensive specifications. Even for the simplest slender structural element, the column of rectangular section, there is little agreement between codes. This is illustrated in Fig. 3 which shows the buckling strength for rectangular columns of a typical structural timber as specified by codes from the U.S.A. (Ref. 9), U.S.S.R. (Ref. 10), U.K. (Ref. 12), Canada (Ref. 2) and the Australian Timber Engineering Design Handbook (Ref. 26) denoted by "T.E.D.H." in the figure.

For the Australian code AS CA65, a decision was made to use a sophisticated format that would be in line with the increasing sophistication of techniques employed for structural analysis. Thus the effect of slenderness on the load capacity of a column is taken to depend on its initial moisture content, straightness and E/F^1_c value, and also on the ratio of live load to dead load. The specified safe design stresses were derived through a rational formulation (Ref. 20) and therefore should lead to reasonable recommendations for the relative strengths of different columns.

As an example of the effectiveness of the AS CA65 formula in providing an appropriate allowance for relative strengths, Fig. 4 shows the predicted effect of duration of load on the strength of initially green, Select grade, alpine ash (*Eucalyptus delegatensis*) rectangular columns with slenderness coefficients of L/B=20 to L/B=50. This predicted effect is seen to be in reasonable agreement with the experimental data obtained by Pearson (Ref. 27) from columns of 2 in. x 1½ in. section which were allowed to dry during the test. For comparison, Fig. 4 also shows the allowance for duration of load as recommended by the codes of U.S.A. (Ref. 9) and U.S.S.R. (Ref. 10).

3.2 General Structures

Except for the cases of rectangular beams and columns subjected to simple loading and lateral restraint conditions, there is little guidance given in timber codes for the effects of slenderness on strength. Difficulty in the formulation of recommendations arises primarily from the vast range and complexity of structures that can fail through buckling.

The basic concept of buckling on which the specifications of AS CA65 are based is illustrated in Fig. 5. The effect of slenderness on strength R is stated in terms of a *stability factor* χ defined by

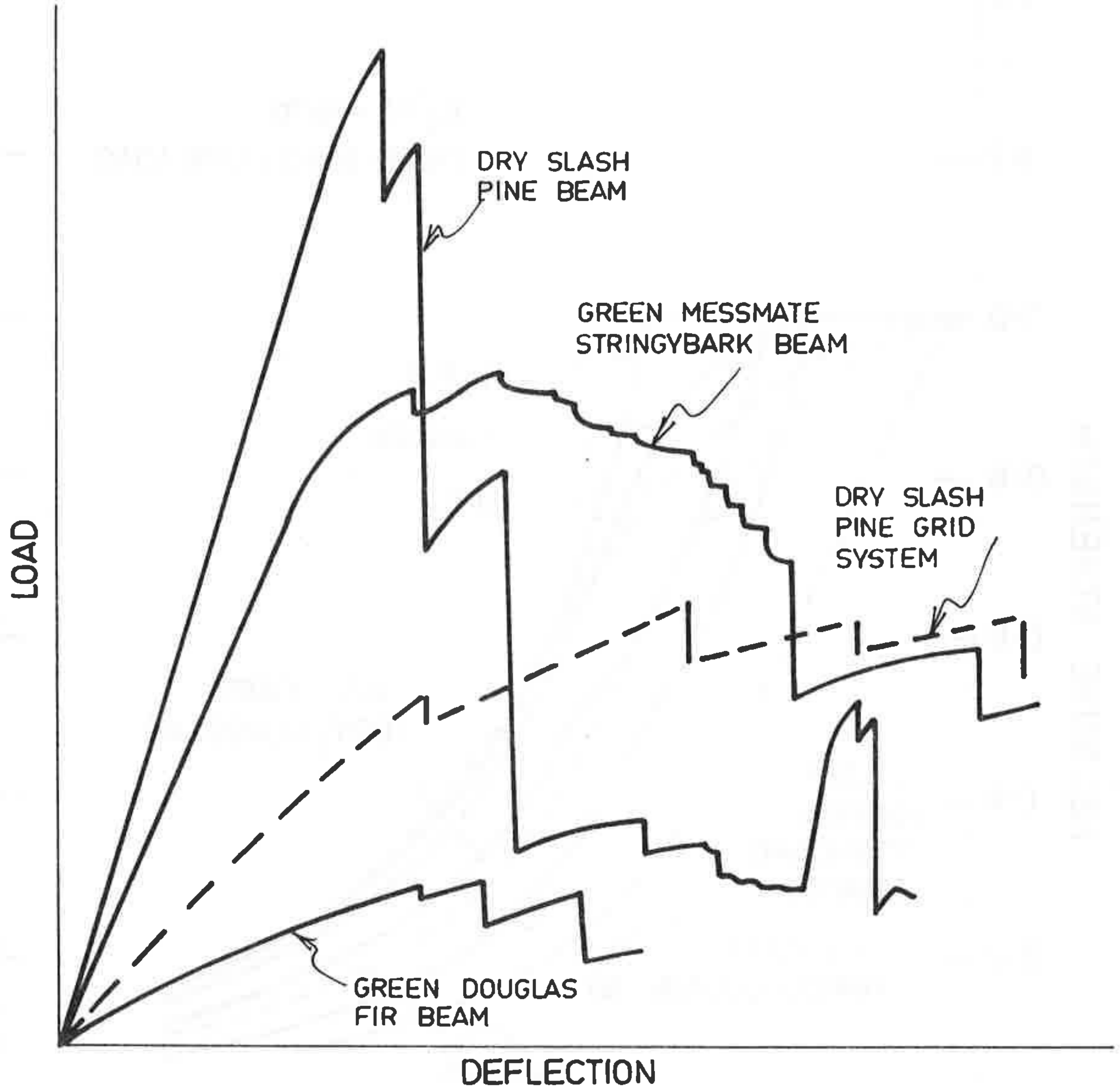

FIG 2. TYPICAL LOAD-DEFORMATION CURVES FOR STUCTURAL TIMBER.

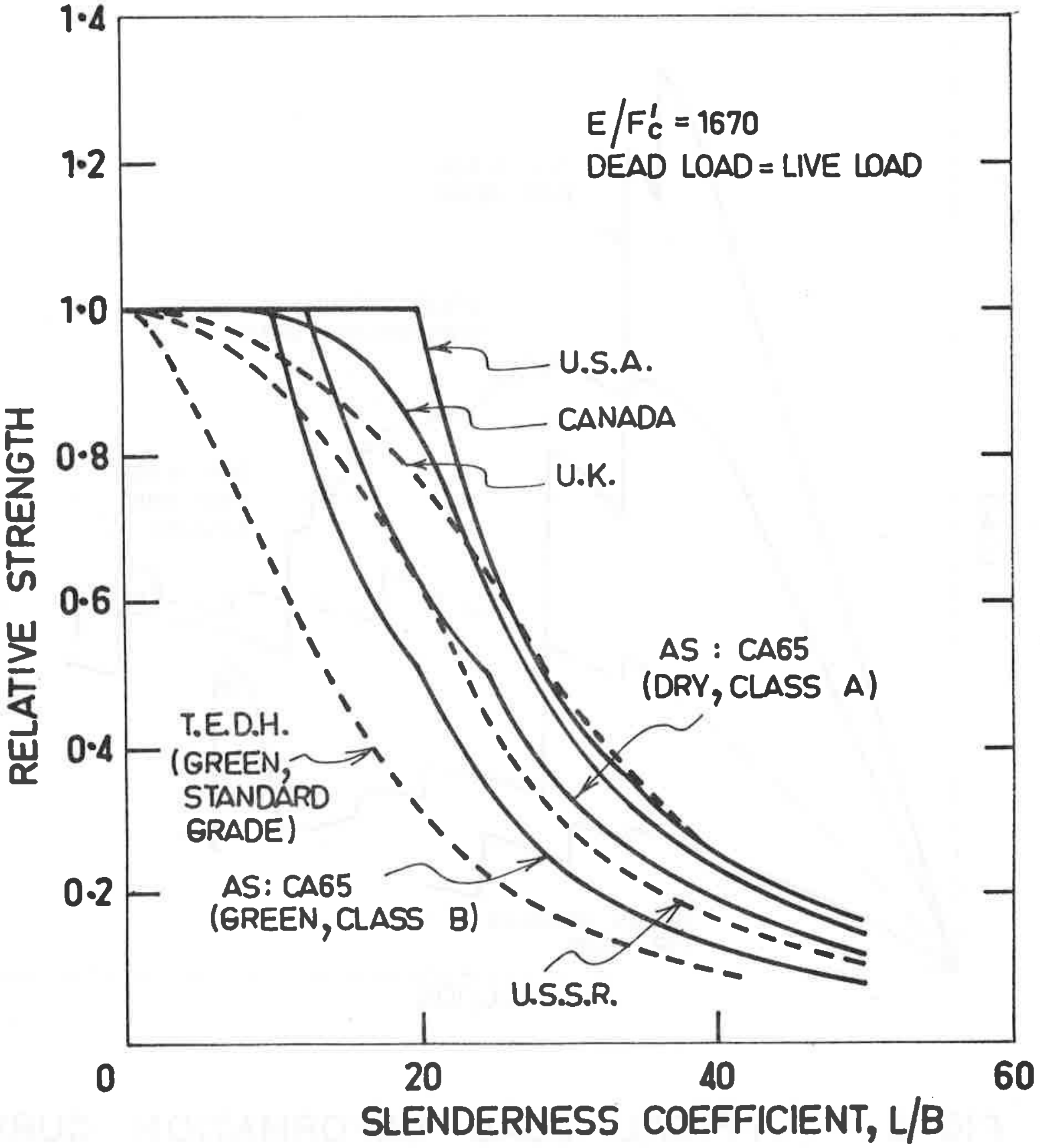

FIG. 3 - COMPARISON OF CODE RECOMMENDATIONS FOR COLUMN LOADS

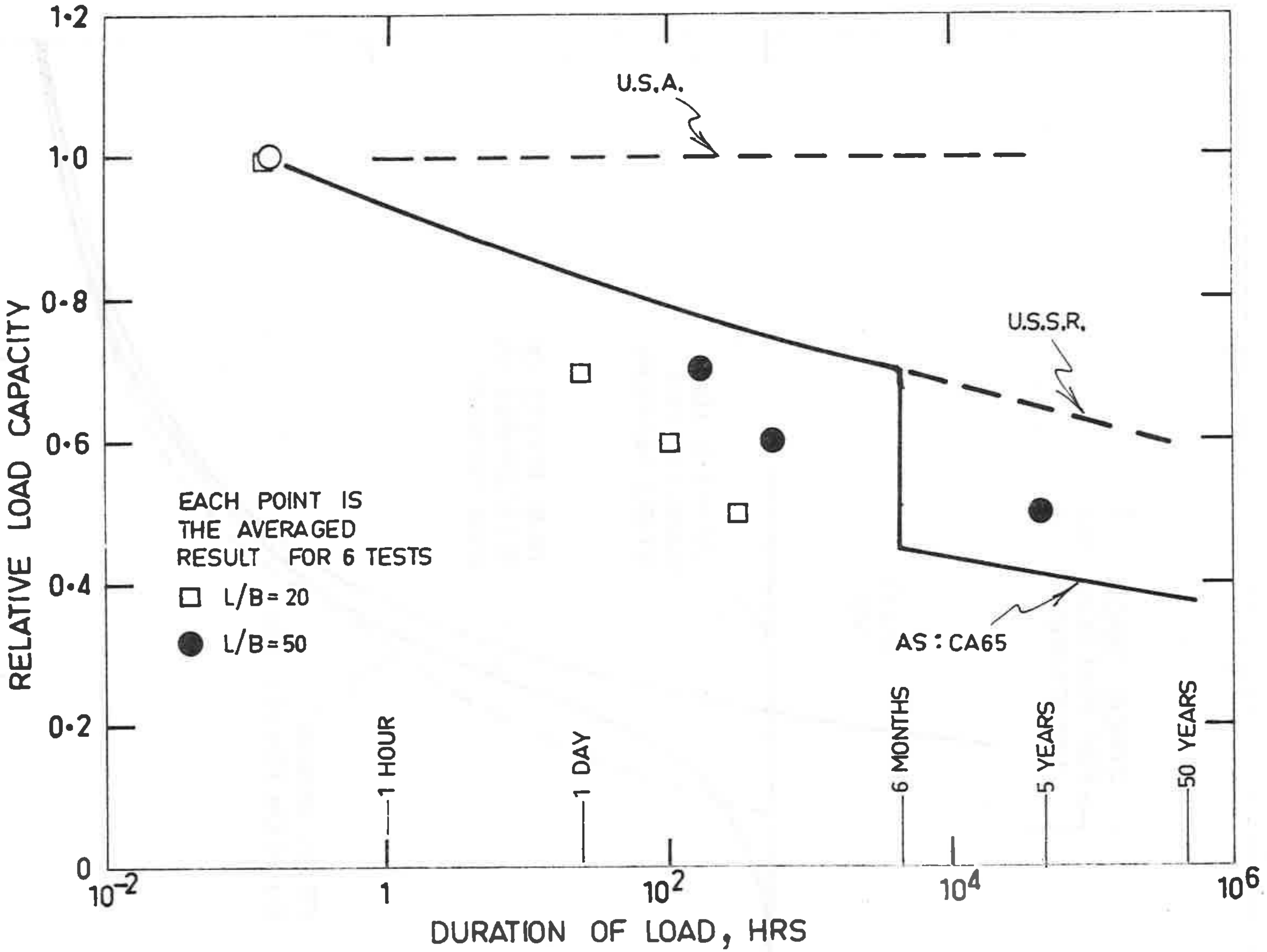

FIG 4. EFFECT OF LOAD DURATION ON COLUMN STRENGTH

(L/B = 20 - 50)

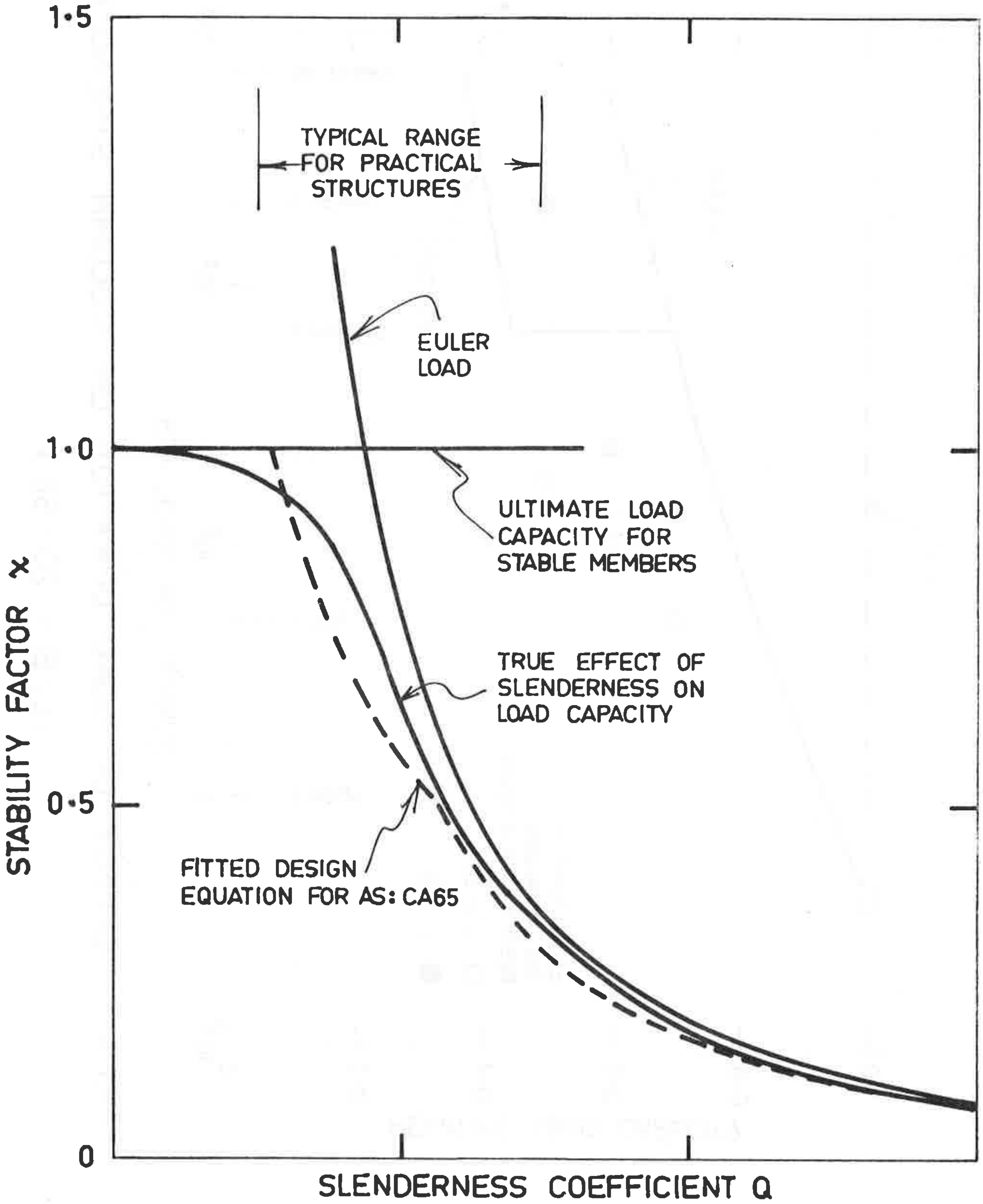

FIG 5. EFFECT OF SLENDERNESS ON STRENGTH OF STRUCTURAL MEMBERS

$$\chi = R/R_o \qquad (7)$$

where R_o is the strength of the structure if it were completely restrained against buckling. The stability factor is specified in terms of a *slenderness coefficient* Q which is a function of the structural dimensions only, such that the Euler buckling load R_E can be written

$$R_E = H/Q^2 \qquad (8)$$

where H depends on the elastic properties of the structural material, the support conditions and the method of loading.

The Q-χ curve of a real structural member is asymptotic to the line χ=1.0 and to the curve for the Euler load given by Equation (8). Its path is influenced by structural crookedness, residual stresses, creep, failure criterion, buckling mode and the reduction in modulus of elasticity at high levels of stress.

In order to simplify the format for design recommendations, the slenderness coefficient Q in AS CA65 is defined by

$$Q^2 = 0.822\ (E/F_E) \qquad (9)$$

for columns and

$$Q^2 = 0.822\ (E/F_E)\ (F_b/F_c) \qquad (10)$$

for beams, where E is the modulus of elasticity, F_E is the nominal stress at the Euler load and F_b and F_c are the permissible working stresses in bending and compression for stable members. In this way the slenderness coefficient of rectangular columns becomes L/B and the same Q-χ curve is obtained for the Euler load of all beams and columns. The stability factor to be used for design is then given by

$$\begin{aligned} &\chi=1.0, && \rho Q<10 \\ &\chi=10/\rho Q, && 10<\rho Q<20 \\ &\chi=200/(\rho Q)^2, && \rho Q>20 \end{aligned} \qquad (11)$$

where ρ is a tabulated material factor. Equations (11) were chosen as the simplest ones possible that provide a reasonable approximation to the theoretically computed Q-χ curve. The specified code values of ρ are based on data for column tests only (Ref. 25 and 32) but may be modified at a future date should information on other structural forms become available.

In this manner the Australian code AS CA65 manages with a simple format to provide design recommendations for a great variety of slender structures. These include recommendations for beams supported along the tension edge only, slender arches and portal frames.

4. FRACTURE MECHANICS

4.1 Basic Concepts

During the past decade, an area of structural theory termed "linear elastic fracture mechanics" has been developed to cope with certain design aspects of structures made from high strength steels. This theory provides a useful criterion for failure at the root of sharp notches.

An elastic analysis of sharp notches (Ref. 18) leads to the result that in general there are two stress singularity fields in the vicinity of a notch root. At this location a stress component σ_{ij} has the form

$$\sigma_{ij} = K_A . f_{ij}(\theta)/(2\pi r)^s + K_B . g_{ij}(\theta)/(2\pi r)^q \quad \ldots\ldots\ldots\ldots (12)$$

where r and θ are polar coordinates with respect to the notch root; s and q are constants in the range $0.5 \geqslant s \geqslant q \geqslant o$ which depend on the material elastic properties and notch angle; and K_A and K_B are constants that depend on the material elastic properties, the member geometry and the loading. For the particular case of a notch in wood the *stress intensity factors* K_A and K_B are defined in terms of the stresses shown in Fig. 6 by

$$K_A = \sigma_y (2\pi d)^s \quad \ldots\ldots\ldots\ldots (13a)$$

for the primary stress singularity field and

$$K_B = \sigma_{xy} (2\pi d)^q \quad \ldots\ldots\ldots\ldots (13b)$$

for the secondary stress singularity field.

Equations (13) imply that there are infinite stresses at a notch root. This cannot be strictly true, but provided the breakdown in elastic theory occurs only in the immediate vicinity of the notch root, then the elastic stresses on a boundary circumscribing this neighbourhood are in accord with equation (12). Under these conditions the stress intensity factors K_A and K_B completely define the stresses in the vicinity of the notch root. Consequently a criterion for failure at the notch root can be specified in the form

$$f(K_A/K_{AC},\ K_B/K_{BC}) \leqslant 1 \quad \ldots\ldots\ldots\ldots (14)$$

where K_{AC} and K_{BC}, the *critical stress intensity factors*, and the interaction equation (14) are determined by direct measurement. This criterion of failure is particularly useful when the structural detail of a notch root is unknown and can only be specified in terms of its method of fabrication.

4.2 Glued Lap-joints

A glued lap-joint in tension fails at the right angle notch such as the corner B indicated in Fig. 7. For this type of notch the effect of the secondary stress field is negligible and the failure criterion for fracture can be taken to be

$$K_A = K_{AC} \quad \ldots\ldots\ldots\ldots (15)$$

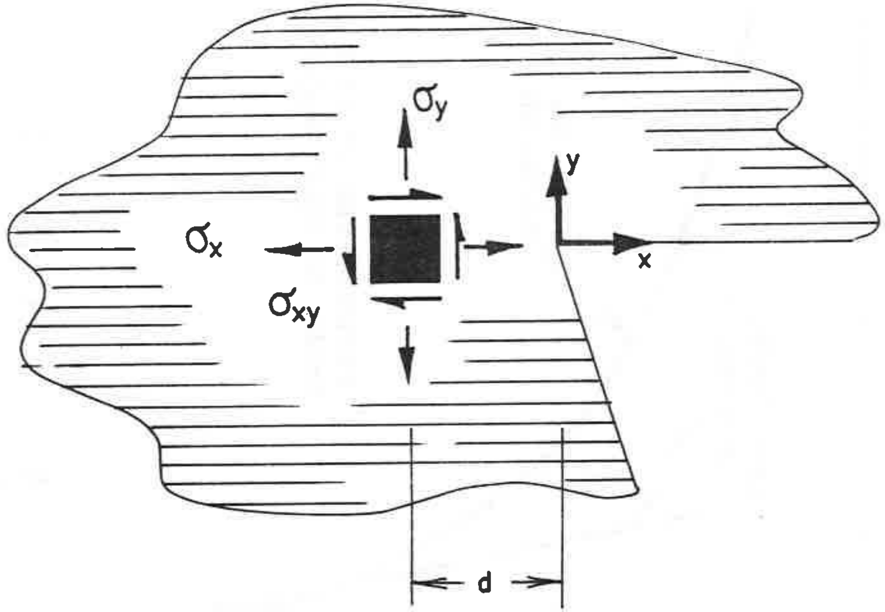

FIG 6. STRESSES IN VICINITY OF A SHARP NOTCH.

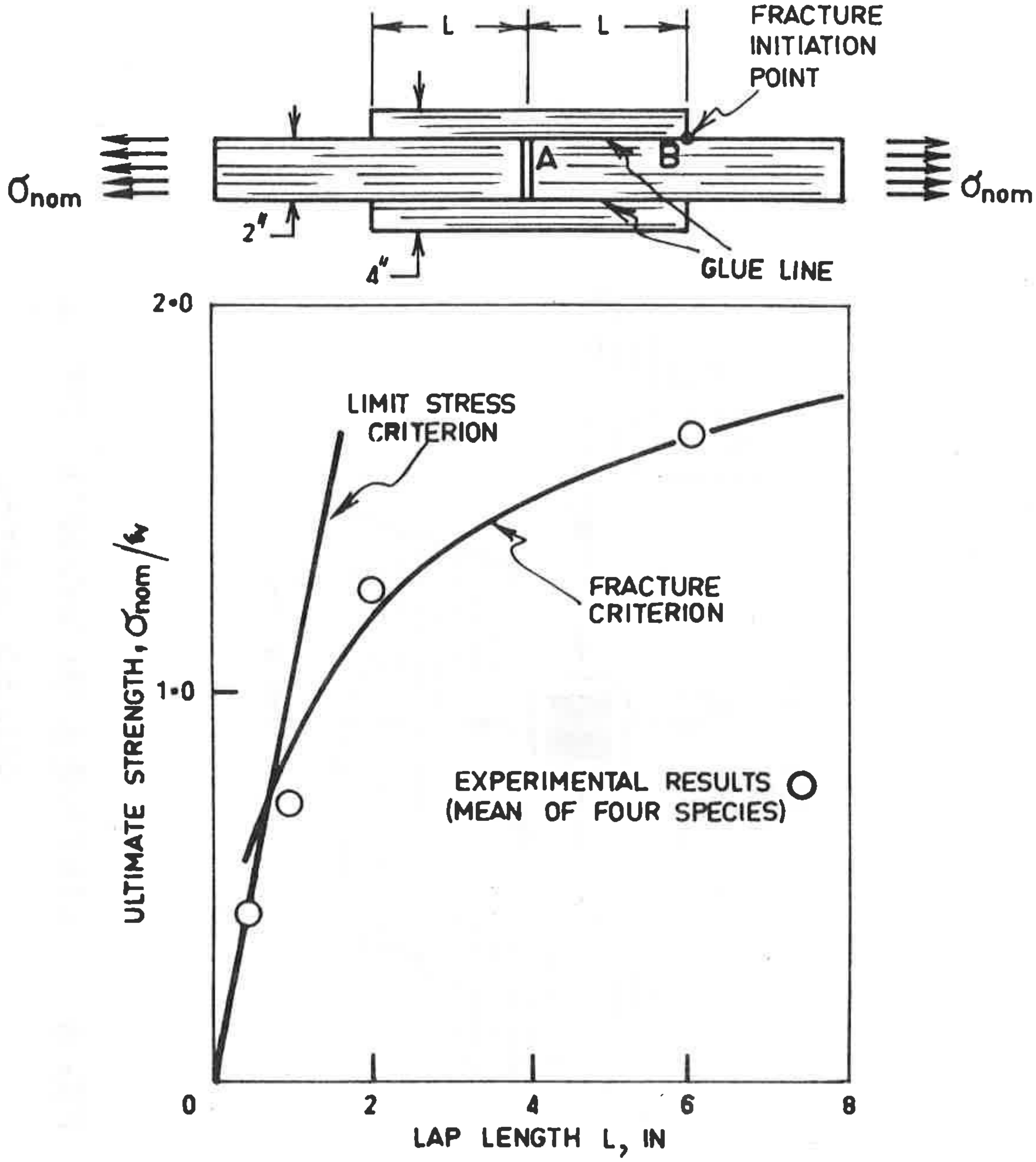

FIG 7. STRENGH OF GLUED LAP-JOINTS

The comparison between theory and experiment shown in Fig. 7 is taken from a previous paper (Ref. 36) in which the measured value of K_{AC} was found to be 0.24 f_v lb.in. units. It indicates that in addition to the fracture criterion of failure it is also necessary to satisfy the usual limit stress criterion. For this case the limit stress criterion is taken to be

$$\tau_{AB} \leqslant f_v \qquad \ldots\ldots\ldots\ldots \quad (16)$$

where τ_{AB} is the nominal shear stress along the glue line AB and f_v is the shear block strength. It is of interest to note that the strengths of glued lap-joints of practical dimensions are not proportional to the area of the glue line.

4.3 Effect of Size on Notch Strength

An important prediction of fracture mechanics is that provided member sizes are large enough, then the nominal stress at failure of notched structural members is inversely proportional to Φ^s, where Φ is a characteristic length that defines the size of a member (Ref. 16). An example of this fracture characteristic is given in Fig. 8 which shows the measured nominal modulus of rupture at the notch root of geometrically similar, notched, dry, messmate stringybark (*Eucalyptus obliqua*) beams of different size. (In terms of the notation and dimensions in Fig. 8, the nominal modulus of rupture at the notch root is $23P_{max}/BD$). The slope of the log-log plot for the experimental data is 0.46 as compared with the value of 0.45 predicted for wood (Ref. 16).

This large effect of size on strength is not considered by any timber codes except AS CA65.

4.4 Butt-joints in Laminated Timber

A serious obstacle to the use of fracture mechanics as a design concept is the difficulty, even with the aid of a computer, of calculating stress intensity factors. However for the special case of a notch that is a sharp crack there are available solutions for many cases (Ref. 25, 33 and 34). One useful application of these solutions is to predict the strength of "butt-joints" in glue-laminated timber. These butt-joints are gaps between board ends and thus simulate cracks of length t; where t is the lamination thickness.

In the case of a sharp crack, $s = q = 0.5$ and the notation K_I and K_{II} will be used to denote the symmetrical and antisymmetrical stress intensity factors respectively. If the glulam is subjected to a uniform tension stress σ_o along the grain, then

$$K_I = 1.25\sigma_o\sqrt{t}\ , \quad K_{II} = o \qquad \ldots\ldots\ldots \quad (17)$$

and if it is subjected to a uniform shear τ_o, then

$$K_I = o, \quad K_{II} = 1.25\ \tau_o\sqrt{t} \qquad \ldots\ldots\ldots \quad (18)$$

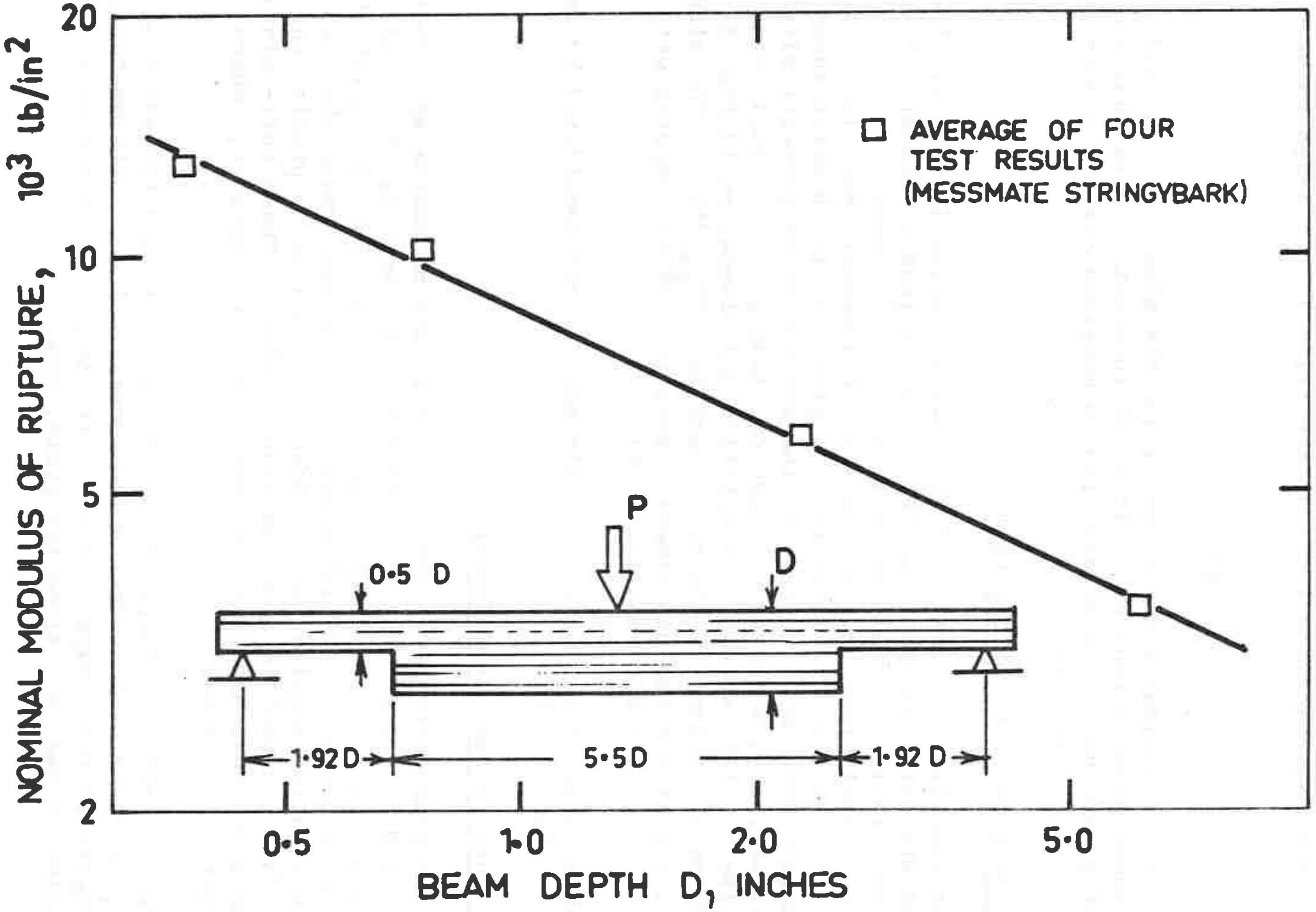

FIG 8. EFFECT OF SIZE ON STRENGTH OF NOTCHED BEAMS

The stress intensity factors for more complicated situations can be obtained from various sources cited previously.

Fig. 9, 10 and 11 give a summary of measurements by the author of critical stress intensity factors in glulam and of the interaction between K_{IC} and K_{IIC}. Each plotted point represents the average for one species and glue combination that had at least 80% wood failure in the vicinity of butt joints after fracture. As shown in Fig. 9 and 10, the design recommendations of AS CA65 are based on the relationship between shear strength and fracture strength for sharp notches cut in wood with a carbide tipped saw. This probably represents the weakest type of notch for low density species. However, as there is such a poor correlation between shear block strength and butt-joint fracture strength for medium and high density species, it is usually preferable to obtain the fracture toughness of butt-joints by direct measurement on fabricated glulam.

Tests on structural size beams and tension members have shown that the fracture mechanics concept of strength is very effective in predicting the onset of fracture. This normally occurs just before total failure. One particularly useful feature in applying the fracture mechanics concept is that it permits a much closer spacing of butt-joints than is normally recommended by timber engineering codes. From an analysis of computed stress intensity factors, Walsh (Ref. 35) has found that the effect on the strength of a particular butt-joint by a neighbouring butt-joint is less than 5% provided that it is separated by at least two laminations if it occurs in the same cross-section or by a distance 4t if it occurs in an adjacent lamination. These theoretical recommendations were checked by tests on 12 tension members of laminated radiata pine (*Pinus radiata*). The disposition of the butt-joints in these test members is shown in Fig. 12. From the value $K_{IC} = 5000$ p.s.i. $\sqrt{\text{in.}}$ measured in a previous test (Ref. 15), a modification factor $\psi=0.77$ according to equation (6) and the appropriate stress intensity factors (Ref. 35) the predicted nominal fracture stress is found to be $\sigma_{nom} = 2840$ p.s.i. In the tests fracture occurred at an average stress $\sigma_{nom} = 2770$ p.s.i. and the ultimate load at $\sigma_{nom} = 2970$ p.s.i.

Some laminated timber fabricators in Australia are currently manufacturing beams with butt-joints. The use of butt-joints saves the cost of acquiring expensive finger-jointing equipment and the delays associated with scarf jointing. For typical structural applications, butt-jointed laminae in beams have only a third the nominal tension stress capacity of continuous laminae. However because deflections usually govern beam design and because butt-joints have little effect on compression strength, it is normally possible to place butt-joints in the top two-thirds of a beam without affecting the beam size.

5. CONCLUDING REMARKS

Several examples have been given on the use of sophisticated concepts for the preparation of a timber structural code. These concepts may be useful for unifying traditional design practice as with structural reliability aspects; for indicating where extrapolation of test data may be dangerous as with the strength of notched beams; and for providing an accurate model of structural behaviour as with the strength of butt-joints in glulam. However a sophisticated concept is almost invariably a complex concept and may be of no practical value unless it can be applied within a simple design format. An example of this difficulty was discussed in the section on buckling strength.

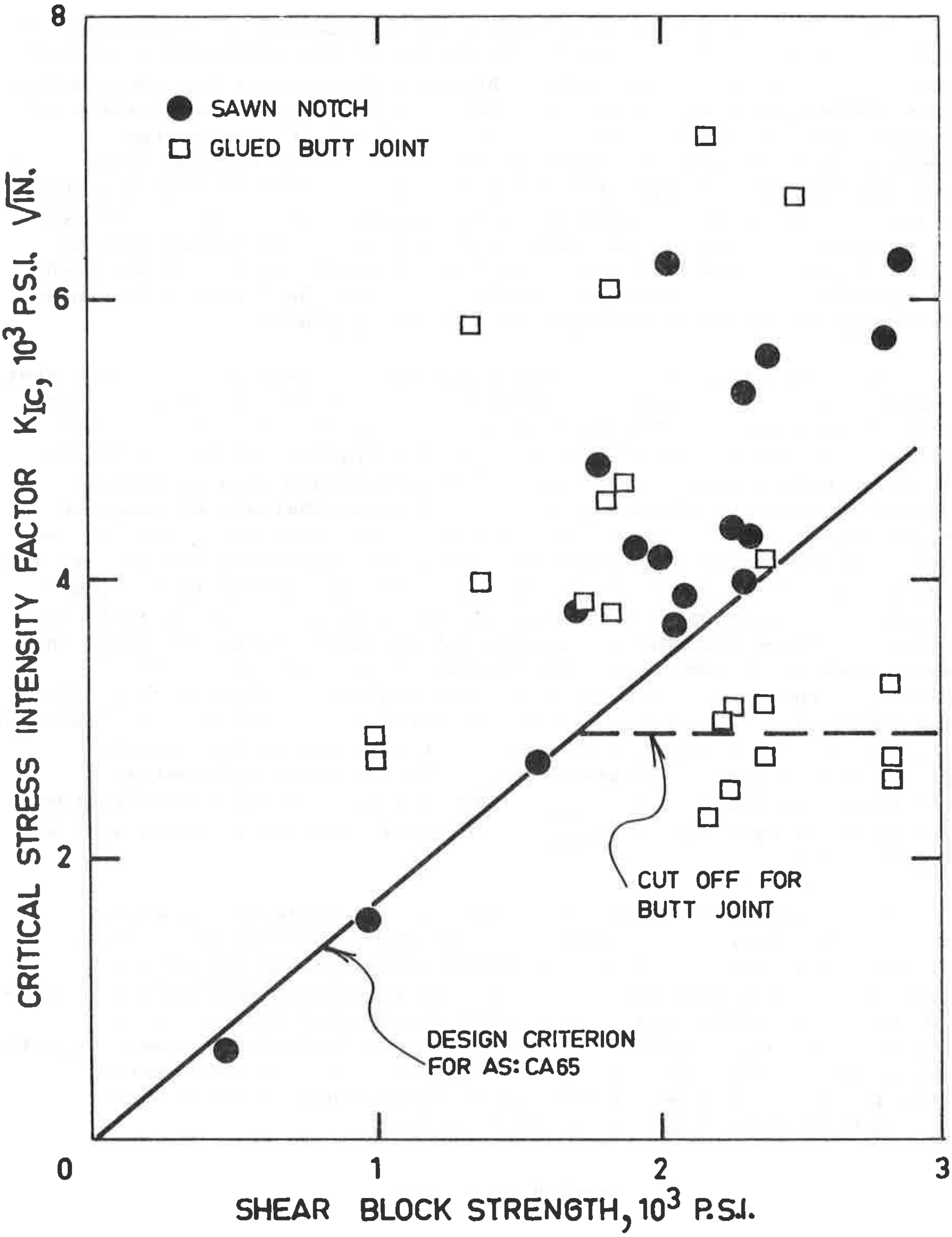

FIG 9. CRITICAL STRESS INTENSITY FACTOR K_{IC}

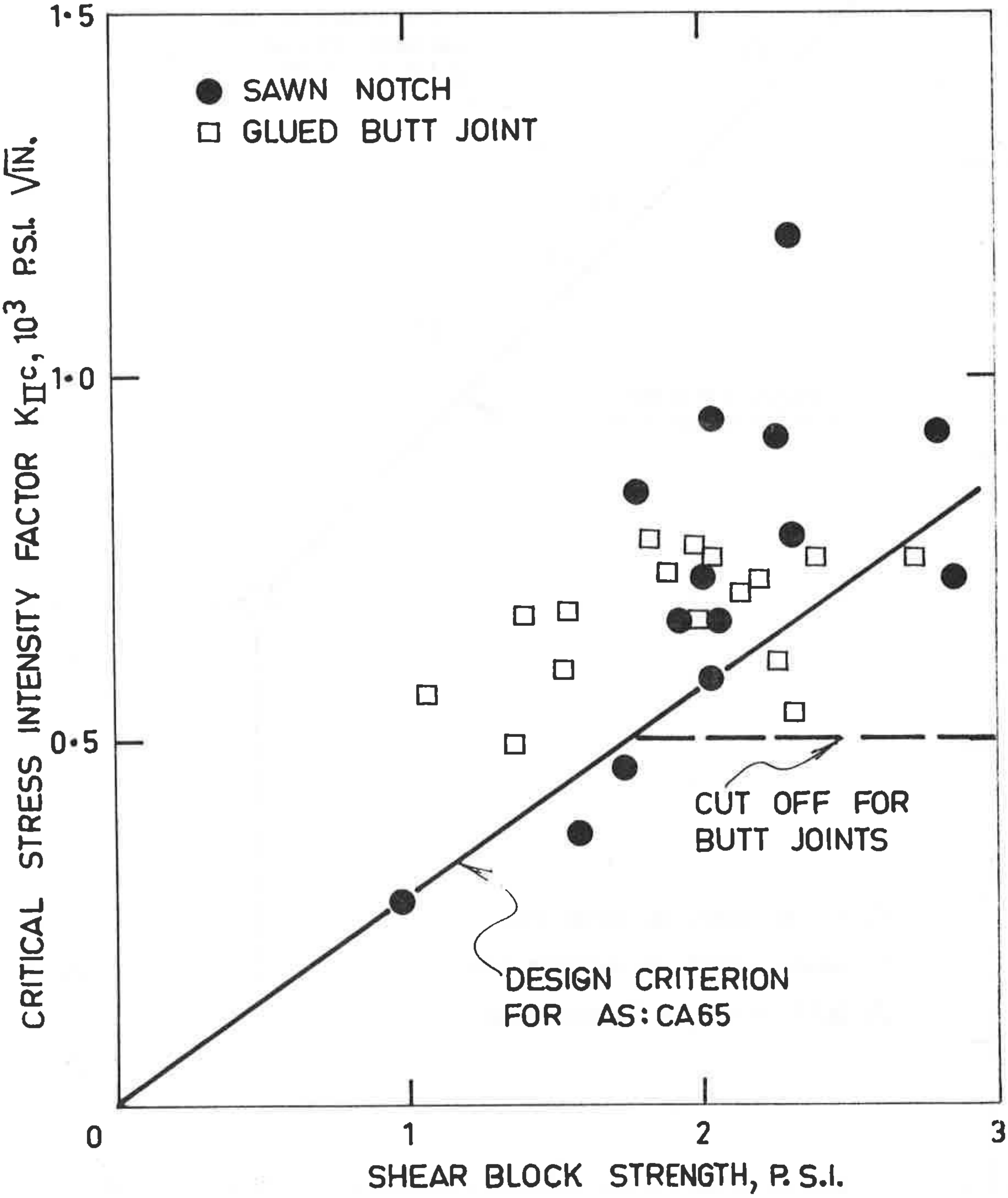

FIG 10. CRITICAL STRESS INTENSITY FACTOR K_{IIC}

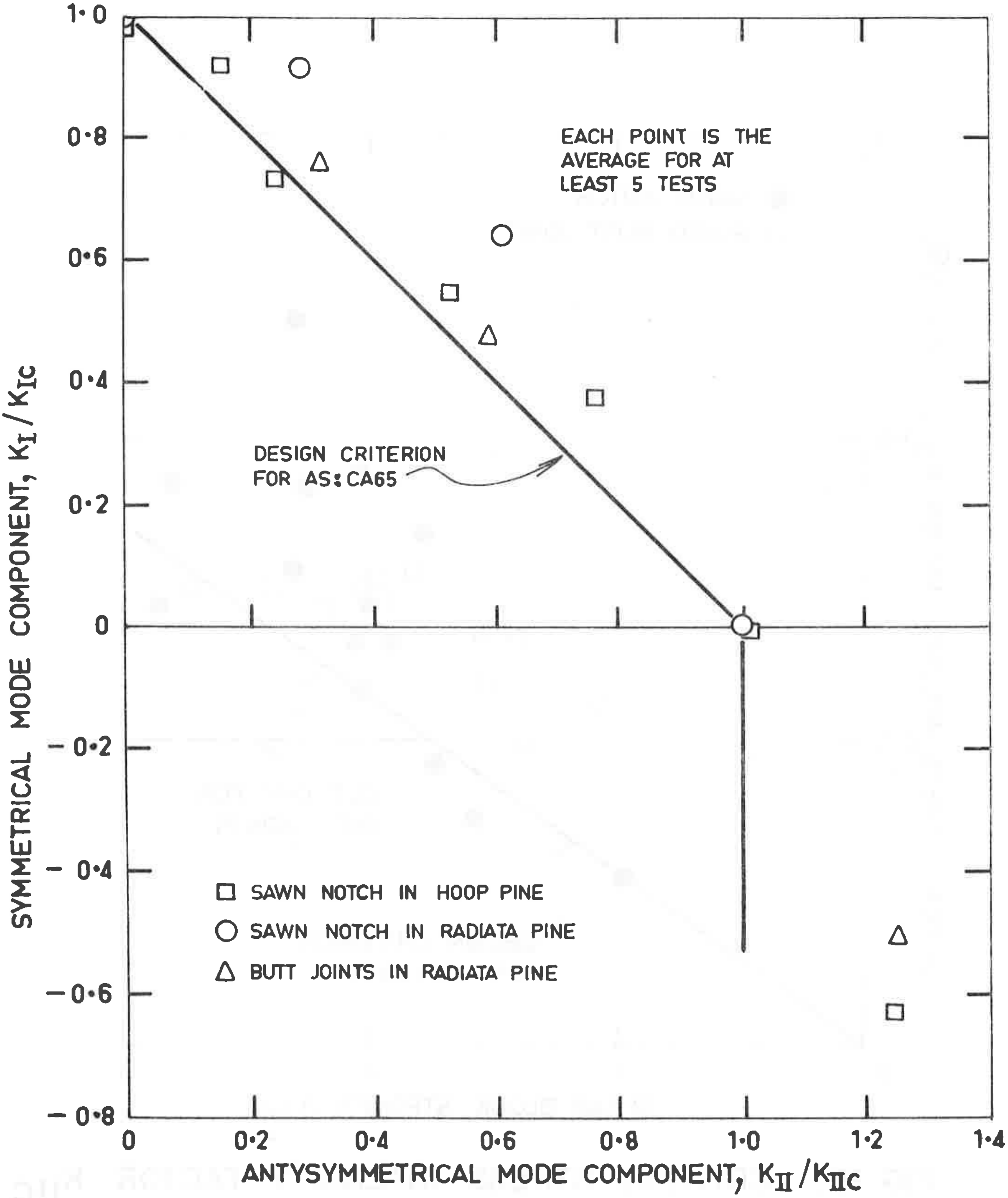

FIG 11. INTERACTION BETWEEN K_{IC} AND K_{IIC}

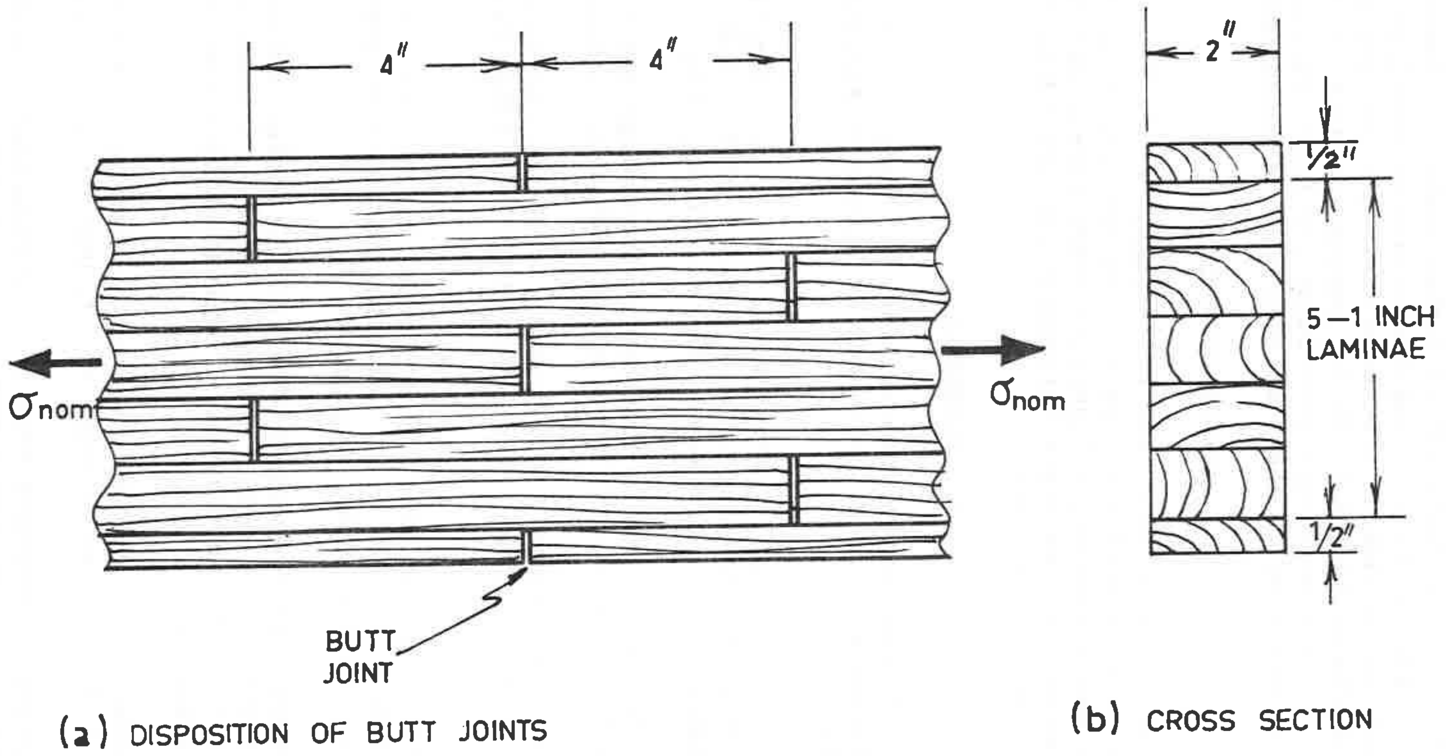

FIG 12. TENSION TEST SPECIMEN

In addition to the concepts discussed in this paper, there are many other sophisticated contemporary concepts of structural theory that could be profitably employed in the preparation of timber codes. These include the non linear (Ref. 30) and ultimate strength analysis of complete structures, the concept of loads as stochastic processes (Ref. 28) and the use of realistic fire characteristics (Ref. 3) in combination with reliability concepts (Ref. 21) to provide design procedures for fire loadings. It is hoped that in the future some of these matters will be investigated.

6. ACKNOWLEDGEMENTS

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8. NOMENCLATURE AND DEFINITIONS

B = least dimension of a rectangular column.

C = cost of design decision.

C_F = cost of failure relative to cost of structure.

E = modulus of elasticity.

f_b, f_c, f_v = bending, compression and shear strengths respectively of small clear specimens.

F_b = design working stress in bending.

F_c, F_c^1 = design and basic working stress in compression.

F_E = nominal stress at Euler load.

G_S = complementary distribution function.

GF = grade factor = ratio of strength in bending of structural timber to that of small clear specimens.

K_A, K_B = stress intensity factors for primary and secondary stress singularity fields.

K_I, K_{II} = symmetrical and antisymmetrical stress intensity factors for sharp crack.

K_{AC}, K_{BC}, K_{IC}, K_{IIC} = critical values of K_A, K_B, K_I, K_{II}.

L = length of column.

n = $\bar{R}/\bar{S}$

n_o = optimum value of n.

P_F = probability of failure.

P = proof load.

q = strength of singularity as defined by equation (12).

Q = slenderness coefficient defined by equations (9) and (10).

r = polar coordinate.

R = strength or stiffness.

$\bar{R}$, R^*, R_K, $R_{0.05}$ = mean, design, characteristic and 5-percentile value of R.

R_E = Euler load.

R_o = strength of structure if it is completely restrained against buckling.

s = strength of singularity as defined in equation (12).

S = load or load action.

$\bar{S}$, S^*, S_K, $S_{0.95}$ = mean, design, characteristic and 95-percentile value of S.

t = thickness of lamina in a glue-laminated timber member.

V_R, V_S = coefficients of variation of R and S.

α = parameter relating structural cost to strength.

β = $(1/N_T)(E_B I_B / E_T I_T)(L_T/L_B)^3$ a stiffness parameter for beam grids where $E_B I_B$ and $E_T I_T$ are the stiffness of the main and transverse beams; L_B and L_T are the span and spacing respectively of the main beams; N_T is the total number of transverse beams.

γ_m = material coefficient defined by equation (3).

θ = polar coordinate.

λ = proof load factor defined by equation (4).

λ_o = optimum value of λ.

ρ = a tabulated material factor for computing buckling strength, see equation (11).

σ_x, σ_y, σ_{xy} = components of stress.

σ_{nom} = nominal stress as indicated in Fig. 7 and 12.

χ = stability factor defined by equation (7).

ψ = modification factor for the "weakest link" effect.

Meeting of CIB WORKING COMMISSION W 18, March 1973.

H.J. Larsen*

Limit State Design.

The current debate on the relative virtues of limit state design and the traditional design principle is often confused by bringing into the debate such questions as, for example, an alteration of the safety system, the introduction of statistical methods, etc.

Limit state design is only new in that it entails the acceptance of other states than those traditionally applied (ie. that the stresses at a given point or the deformations exceed a specified value) as limiting the usability of a structure.
Thus, for example, limit state design permits the use of the theories of plasticity or rupture, whereas the traditional methods are based on the theory of elasticity.

At the present time, it is doubtful whether any engineers would find it justifiable to use anything but the theory of elasticity for the wooden parts of a structure. The higher qualities of wood do, admittedly, possess certain plastic properties in compression and bending, but the "yielding" is not, as in the case of steel, a benignant phenomenon, but results in unacceptable destructure of the wood. For the poorer qualities, in which rupture usually occurs near knots or other defects, it is doubtful whether there are any plastic phenomena at all.

Composite structures, e.g. roof trusses, with mechanical fasteners (nails, bolts, nailplates etc.), on the other hand, often have good plastic properties, and there are considerable advantages to be gained in this case by accepting as the limit state the collapse of the total structure rather than rupture of a single component.

Whichever the method adopted, the loads and material strengths must be specified in such a way that the limit state is not reached in practice, or - if this is unavoidable - that the risk of the limit state being reached is a small, acceptable quantity. Statisticians dream of being able to calculate this risk, and their dream has, or may one day, become reality in a few fields. But for normal building structures - and especially for those of wood - it is inconceivable that we shall ever be able to procure all the necessary data on the variations in the loads and the material strengths. This does not mean, of course, that statistics should not be used to the extent possible, i.e. for the purpose of specifying, at a uniform level, the loads (corresponding to a specific probability of occurrence) and the material parameters (corresponding to a specific exclusion limit).

Before use, the characteristic loads thus statistically determined are multiplied by a factor (partial coefficient) $\gamma_F \geqq 1$, and the characteristic material parameters are reduced by division by a partial coefficient $\gamma_m \geqq 1$. There will frequently be different values of γ_F for the dead load γ_F^{DL} and the live load γ_F^{LL}, and γ_F^{LL} may vary for different types of loading.

By distributing the safety over the load and the materials, great freedom is achieved because a given structure will reach the limit state at the same characteristic load, whether we use

$$\gamma_m = a, \qquad \gamma_F^{DL} = b, \qquad \gamma_P^{LL} = c$$

or

$$\gamma_m = k \cdot a, \qquad \gamma_P^{DL} = \frac{b}{k}, \qquad \gamma_P^{LL} = \frac{c}{k}$$

where k is a constant.

Thus, if we choose $\gamma_F^{DL} = \gamma_F^{LL} = \gamma_F$, we need not complicate matters unnecessarily by splitting the safety between the loads and the materials - we can just as well use a single safety factor $\gamma_F \cdot \gamma_m$ on the materials alone (or the loads alone). Only if we make $\gamma_F^{DL} \neq \gamma_F^{LL}$ will the partial coefficient system have any purpose.

With regard to the choice of partial coefficient, it must be borne in mind that the dead load acts favourably in some cases and

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unfavourably in others. Therefore, in order not to complicate the calculations unnecessarily, we should choose $\gamma_F^{DL} = 1.0$.

Conclusions.

The introduction of limit state design merely means the introduction of additional - possibly less stringent - criteria than those at present applying to the design of structures.

The set of partial coefficients ($\gamma_m = 1.3$, $\gamma_F^{DL} = \gamma_P^{LL} = 1.3$) proposed in [1] is the same as using a single safety factor ($1.3 \cdot 1.3$) on the strength parameters only. The proposed change therefore represents an unreasonable complication.

The system of partial coefficient for concrete given in [1], ($\gamma_m = 1.3$, $\gamma_F^{DL} = 1.4$, $\gamma_F^{LL} = 1.6$) should be changed to $\gamma_m = 1.3 \cdot 1.4$, $\gamma_F^{DL} = 1.0$, $\gamma_F^{LL} = 1.6/1.4$, which gives precisely the same result and, in some cases, considerably facilitates the calculations.

[1] BLCP/17/2 - WORKING STRESSES REPORT 71/12202.

CIB W 18
20 - 21 March 1973

THE USE OF PARTIAL SAFETY FACTORS IN THE NEW NORWEGIAN DESIGN CODE FOR TIMBER STRUCTURES

Odd Brynildsen, The Norwegian Institute of Wood Working and Wood Technology

1. Background

The Norwegian Standards for Design of Structures have all been revised lately. The Standards for Structures of Steel, Concrete, Aluminium and Timber are now being printed and will be published before summer. The four Design Standards have been coordinated in content and layout and they are all based on the common Load Standard, NS 3052 "Calculation of loading". They are also based on the limit state methods of design.

2. NS 3052 "Calculation of loading"

This standard contains the load factors which of course are the same for all structural materials.
See Appendix 1.

The loads given in the general Building Code, in NS 3052 or elsewhere, should in principle be the characteristic loads. Because of lack of data the characteristic loads have not been defined, and the given loads are nominal loads which for the time being are used as characteristic loads.
The design load is the characteristic (nominal) load multiplied by the load factors given in NS 3052 for two limit states.

3. Material factors

The material factors may be different for the different materials.

For timber four partial factors are introduced:

γ_1 = Factor for grading

1.0 when timber is quality marked and the grading is supervised by an officially reckognized quality control system.
For L marked gluelam.
For connectors and joints.

1.1 For normally graded timber.

γ_2 = Factor for workmanship

1.0 for structures made at a factory where production is supervised by an officially reckognized production control system.

1.1 Other structures.

γ_3 = Factor for calculation.

1.0 Complete control of all calculations and designs by another person.

1.05 Control of main forces and main dimensions, self control

1.1 No control (Not permitted when collapse may lead to danger to life)

γ_4 = Factor for consecuence of rupture

1.0 Structures when rupture will lead to small consequences for economy or insignificant danger to people.

1.05 Structures for small houses.

1.10 Structures where rupture may lead to large consequences for economy or to possible danger to people.

(Where danger to people is present, the structure must be systematically supervised and maintained).

The material factor γ_m is

$$\gamma_m = \gamma_1 \cdot \gamma_2 \cdot \gamma_3 \cdot \gamma_4$$

Normally the material factor shall not be less than $1.4/\gamma_f$

Where γ_f is the effective load factor: $\gamma_f = \sum \gamma_i F_i / \sum F_i$

4. Characteristic Strength

The characteristic strength values for timber are determined on a 75% confidence level as the 5% lower exclusion limit. (Short term loading, $\frac{1}{2}$ - 6 min.)
Full scale tests on timber graded visually to the stress groups T40, T30 and T20 has given the characteristic bending strength values 41,5 - 30,2 - 22,8 N/mm^2 respectively.
If the long term reduction factor is 0,6 the characteristic bending strength would be 24,8 - 18,2 - 13,7 N/mm^2.
In the design code these values have been given to 26.0 - 20.0 - 14.0 N/mm^2, which are slightly higher than the derived values. This had to be done in order to obtain that normal average structures designed according to the new and the old code should be as equal as possible.

The design stress is the characteristic strength divided by the material factor.

from NS 3052

8. LOAD FACTORS

8.1 Method of design

In paragraph 8 it is assumed that dimensioning of structure is made by a calculative control of two limit states, to be indicated below as service limit state and ultimate limit state. The relevant rules indicate to what extent and in what way those two states must be controlled.

8.2 Load factors

Insecurity concerning loading should be indicated by the loads being multiplied by the load factors as stated in subd. 8.3 or 8.4.

In principle the load factors should take care of:

a Abnormal or not anticipated loads, of which no account has been taken.

b Reduced probability that several possible loads all have their full value at the same time.

Insecurity of material strength, execution, etc. is taken care of in the relevant rules for the calculation of structure by special factors or in some other way.

8.3 Loads and load factors

If the loads are not fixed more accurately, they may be given with the values of this standard, the Building Regulations or other official regulations.

For housing structures or other structures with a predominantly dead load the loads are to be multiplied by the load factors indicated below.

Table 6a Load factors for the ultimate limit state

Loading	Load factor for				
	G	N	V	J	E
O	1,2	1,6	1,1	1,4	-
G + E	1,2	-	-	-	1,5
*O + E	0,96	1,28	0,88	1,12	1,2

Table 6b Load factors for the service limit state

Loading	Load factor for				
	G	N	V	J	E
O	1,0	1,0	1,0	1,0	-
G + E	1,0	-	-	-	1,0
*O + E	0,8	0,8	0,8	0,8	0,8

The following symbols are used in Tables 6a and 6b:

G for structure loads

N for imposed loads, snow loads and other ordinary loads not specified in the Tables.

V for water pressure

J for earth pressure

O for ordinary loads

E for extraordinary loads.

*The load factors in the last line in thest two tables are the result of the four first columns of the first line and the last column of the second line being multiplied by 0,8.

When two or more extraordinary and mutually independent loads appear at the same time, the load factor for the load having the greatest effect should be calculated as having the value indicated by Tables 6a and 6b. Load factors for the rest of the extraordinary loads may be reduced by 30%.

The dimensioning should be based on the most unfavourable alternative according to the Tables.

For structure loads and other loads certain to appear the load factor in Table 6a should be 1,0 if in this way more unfavourable results are obtained than the values given in the Tables.

The same factor may be used for each of these types of load for an entire structural member so as to avoid e.g. different structure load factors for the various parts of one single beam.

SVENSKA TRÄFORSKNINGSINSTITUTET

CIB WORKING COMMISSION W18
Princes Risborough 20-21 March, 1973

SWEDISH CODE REVISION CONCERNING TIMBER STRUCTURES

General

The use of timber for structural purposes in Sweden is at present submitted to Svensk Byggnorm (SBN 67) /1/. This code of standards partly contains regulations compulsory for the builder as well as for the local building authorities, in parts recommendations not compulsory for the builder but accepted by the local authorities.

SBN 67 is issued by the central building authority, Statens Planverk (SP) - The National Board of Urban Planning. For structures in steel and concrete, however not for timber structures, there are also specific codes in coordination with SBN. SP is responsible for the principal building regulations, some of which, for example the Loading Code, are included in SBN. For revisal of these principal regulations SP have appointed a "Safety Group", here called the S-group. This group is supposed to present a draft for a general structural code for supporting structures in May 1973. To this code will be added later on application rules (codes of practice) for timber, steel and concrete structures etc., so called Technology Codes.

An important part of the charge of the S-group has been to decide between different principles for the verification of the safety of structures. Further, the S-group is supposed to act as a reference group for Sweden in the international work on matters concerning the safety of structures. Several of the group members are actively involved in the work within NKB (Nordic Building Regulations Committee). NKB has several subcommittees, also for the loading code and for the safety of structures. The NKB-subcommittee for timber structures has presented up to now twelve drafts for standards on different subjects /2/.

10 000 4. 72. KDB

It should perhaps be mentioned that at present SBN 67 is being revised. The differences in the chapter concerning timber structures, however, are limited and not affected by the work of the S-group.

Principles for the verification of safety of structures

The conception of limit load originally referred to the ultimate load and the "limit load method" dealt with calculating of ultimate strength and was especially referred to for structures which could yield plastically. The limit state method, however, is now used in the general sense of a design in which the fitness of the structures is verified at different load levels, i.e. for different limit states.

Judging from the proposals by ISO /3/, NKB, and various nations, the unity is considerable therein that the structures should be merited fromout their serviceability limit state, as well as from the ultimate limit state. However, what is discussed are the performance requirements defining the limit states and methods of verifying that the structures meet with these requirements. The methods subject to comparison are:

1) The method of permissible stresses (traditional in most countries)

2) The method with partial coefficients

3) Methods of probability. By these the requirement can be theoretically defined by one permitted maximum probability of the structure becoming unfit for service conditions and another maximum probability of its passing the rupture limit.

The difference between the method 2 and 1 is mainly that according to method 2 the safety coefficient is divided into partial coefficients on the load- and the resistance sides.

One essential feature of the partial coefficient method is that different coefficients are being used for different types of loads. (Sometimes the coefficients are denoted partial coefficients only on the load side.)

At times method 1 as well as method 2 are called semi-probabilistic, or statistic. This, though, is hardly correct, and certainly not justified merely because of characteristical values are being used to express load and resistance. Characteristic for a probabilistic (statistic) method should be an ambition to quantify the risk of exceeding the limit states, at least by using an approximative substitute measure. Within certain limits for load combinations and variations of load and resistance it is possible to adapt the partial coefficients of method 2 for a certain probability of failure. The probabilistic method will be more correct, especially when several variables are involved. In this method the variables will have to be expressed by 2 or 3 parameters, whereas only one characteristical parameter is used in method 1 and 2.

ISO have distinguished between the limit state design method and the method of permissible stresses. This is really an irrelevant reference to the difference between the theory of elasticity and the theory of plasticity.

The Swedish S-group has accepted the partial coefficient method on account of one important advantage: The method has been accepted in Denmark and Norway and seems to be the method on which international (European) agreement can be reached.

However, the group intends to present a probabilistic method as well.

The partial coefficient method

The verification that the stress (S) does not exceed the resistance (R) can be expressed by

$$S(\gamma_{F1} F_1, \gamma_{F2} F_2, \ldots) \leq R(\gamma_{M1} M_1, \ldots, L_1, L_2, \ldots) \qquad (1)$$

S is the function which transforms the load values (F) to the load effect. R is the function which transforms the strength (M) and the geometric parameters (L) into that resistance which is defined by the limit state in question. Naturally, S and R should be thus defined, that stress and resistance are of the same kind (e.g. moment in a certain cross-section of a beam). For some cases (1) has to be generalized.

One example is the fairly frequently used interaction formula

$$S_1/R_1 + S_2/R_2 \leq 1 \qquad (2)$$

The characteristical F-values are found in the loading code. Also the partial coefficients for load (γ_F) are given in this code. The other quantities, including the functions S and R are stipulated in the respective "technology codes" (e.g. the codes of practice for timber structures). If M denotes moment, the characteristical value will, of course, depend on variations in material strength and variations of section modulus. γ_M can be made dependent on the extent of quality control and inspection at the building site. The value of γ will further be dependent on the definition of the characteristical value. At present 5-per cent fractiles for resistance and 95-per cent fractiles for loads have been suggested.

Table 1 is a way of presenting partial coefficients for loads.

Table 1 Example of partial coefficients for loads (γ_F)
(Proposal by Östlund, Swedish S-group)

Load combination	Type of load	Required β- value 2	3	4	5
I	Dead	1,0	1,2	1,3	1,4
	" (favour.)	(1,0)	(0,9)	(0,9)	(0,9)
	Live	1,0	1,3	1,5	1,7
	Climate	1,0	1,0	1,0	1,0
II	Dead	-	1,1	1,2	1,3
	" (favour.)	-	(0,9)	(0,9)	(0,9)
	Live	-	1,1	1,3	1,5
	Climate	-	1,0	1,0	1,0
III	Dead	-	-	1,0	1,1
	" (favour.)	-	-	(0,9)	(0,9)
	Live	-	-	1,0	1,1
	Climate	-	-	1,0	1,0

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A column with still lower values than those in column 2 has been discussed for the serviceability state, however, this would imply partial coefficients lower than with a value below 1, which might be inconvenient.

P r o b a b i l i t y   m e t h o d s

In the long run the aim with a design method should be to attain uniform reliability values for different structures. This, however, is not practically obtainable by the partial coefficient method. Thus, within the S-group, as well as within NKB, different statistical safety systems have been discussed. The possibility of introducing such a system in a general structural code has been dealt with, as well as the possibility of using it for determination of the coefficients for the partial coefficient method. The base for this discussion has been the probability concepts by Allin Cornell /3/, and variants presented by Rosenblueth, Esteva, and Dietlevsen /4/. These systems have in common the introduction of a measure of reliability, β, substituting the real probability of failure.

A nominal traditional safety factor as the quotient of strength and load on defined exclusion levels may be maintained. However, a central factor equal to the quotient of the mean values might be preferable.

$$\theta = \bar{R}/\bar{S} \tag{3}$$

Cornell, assuming normal distribution of the strength (R) and of the load (S), arrives at the following relation of θ to his measure of safety $\beta = 1/V_{R-S}$:

$$\theta = \frac{1 + \beta\sqrt{V_R^2 + V_S^2 - \beta^2 V_R^2\, V_S^2}}{1 - \beta^2\, V_R^2} \tag{4}$$

V denotes the coefficient of variation. According to Cornell the level of β should primarily be adapted to established structural practice. In that case the method will principally be applied for calculating the relative effect on the reliability of different design conditions. One of the disadvantages of the Cornell expressions is that $\theta \to \infty$ when $\beta^2 V_R^2 = 1$ which occurs, for instance, at $\beta = 4$ and $V_R = 0.25$. Instead of expressing the probability of failure as $P(R-S \leq 0)$, one can choose

$$P(R/S \leq 1) = P(\ln R - \ln S \leq 0) = P\ (Z \leq 0) \tag{5}$$

By introducing the approximation

$$\bar{Z} \approx \ln\bar{R} - \ln\bar{S} = \ln\bar{R}/\bar{S} = \ln\theta \tag{6}$$

and the variance

$$\sigma_Z^2 = \sigma_{\ln R}^2 + \sigma_{\ln S}^2 \approx V_R^2 + V_S^2 \tag{7}$$

Esteva arrives at a β which is analogical with Cornell's $\beta = 1/V_{R-S}$.

$$\beta = \bar{Z}/\sigma_Z \approx \frac{\ln\theta}{\sqrt{V_R^2 + V_S^2}} \tag{8}$$

For a stipulated reliability β (cf.table 1) the central safety factor then is

$$\theta = \bar{R}/\bar{S} = \exp(\beta\sqrt{V_R^2 + V_S^2}) = \exp \beta V \tag{9}$$

Should one prefer characteristical values R_k resp. S_k, defined by

$$\ln R_k = \ln\bar{R} - k_R V_R \quad \text{resp.}$$

$$\ln S_k = \ln\bar{S} - k_S V_S$$

the nominal safety factor should be

$$R_k/S_k = \theta^* = \theta \exp(-k_R V_R - k_s V_s) = \exp(\beta V - k_R V_R - k_S V_S) \tag{10}$$

By using the equations (1), (8) at least in simple cases, it is possible to calculate the partial coefficients (γ) expressed in terms of V_S and V_R /5/. A result is shown in Fig. 1. The real probability of rupture, calculated for different types of distribution F_S and F_R, normal, log-normal, extreme I and II, has also been compared with Esteva's measure of reliability.

$$P_E = 460 \exp(-4{,}3 \frac{\ln\theta}{V}) \tag{11}$$

Stockholm in March 1973.

Bengt Norén

10 000 4. 72. KDB

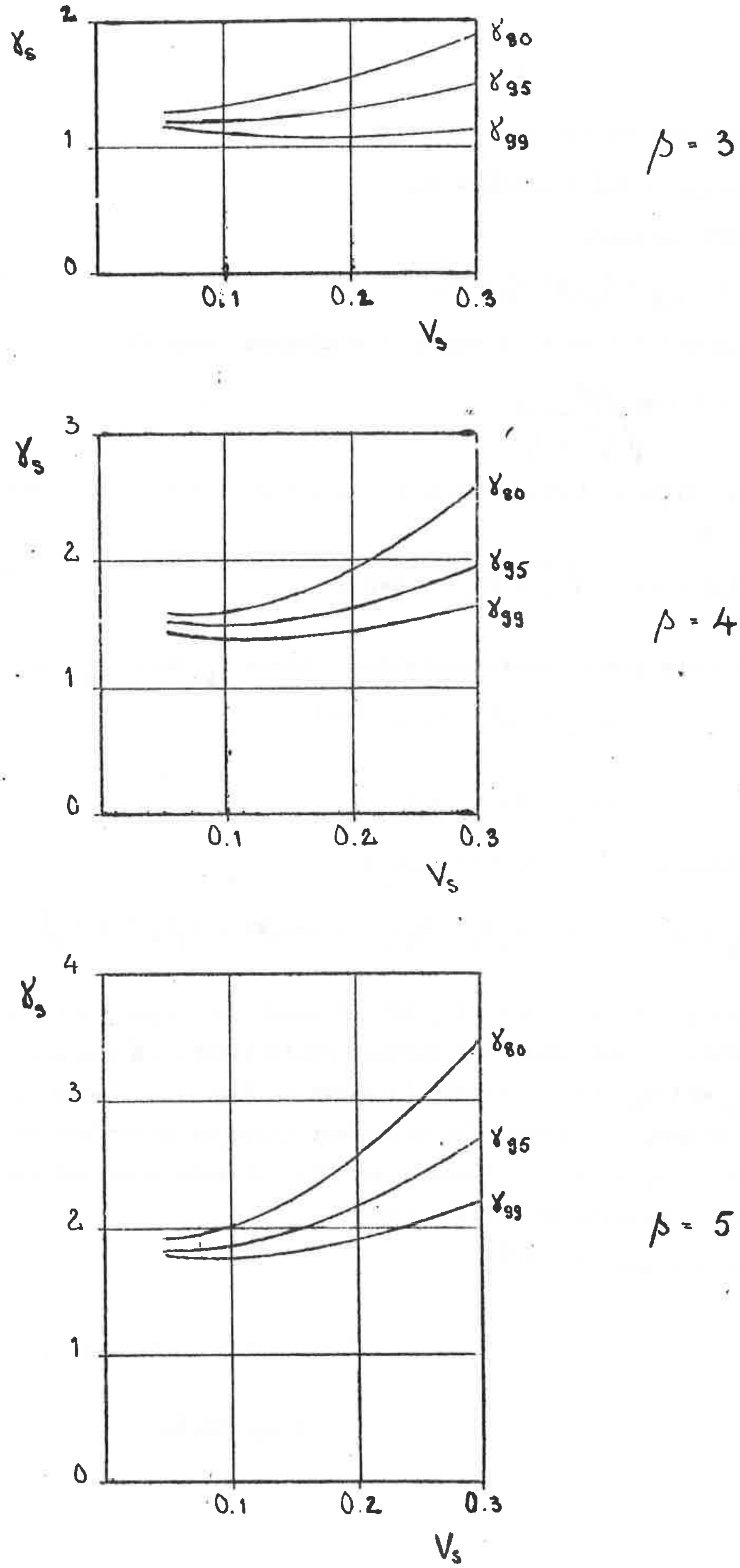

Fig. 1 Partial coefficients for load, γ_S, to give β = 3, 4 and 5 when different exclusion levels are used as characteristical load (γ_R=1) /2/

References

/1/ Svensk byggnorm 67. Statens Planverk Publication nr 1. Stockholm 1968. (See Appendix 1)

/2/ Nordiska riktlinjer för träkonstruktioner (Nordic rules for timber structures). NKB-skrift nr 7. May 1967 and NKB-skrift nr 13. Nov 1970. NKB (Nordic Committee on Building Regulations).

/3/ Cornell, C.A.: A Probability-Based Structural Code. ACI Journal. Sept, Oct, Nov and December 1969

/4/ Rosenblueth E.:Code Specification of Safety and Serviceability. State of Art Report No. 2A. Techn.Committee No.10, Conference on Tall Buildings, Lehigh, USA, 1972.

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BLCP/17/2 - WORKING STRESSES REPORT

1 TERMS OF REFERENCE

The sub-committee were asked to examine the feasibility of applying the philosophy of limit state design to timber structures and to recommend whether this approach should be adopted in the current revision of CP 112.

2 COMPOSITION OF SUB-COMMITTEE

Mr J G Sunley (Chairman)
Dr L G Booth
Dr W Chan
Mr W T Curry
Mr G D Grainger (co-opted)
Mr M Macdonell
Mr P J Steer

The sub-committee held five meetings between February and August 1971. The minutes of these meetings were submitted to the main BLCP/17 Committee.

3 RECOMMENDATIONS

After considering the advantages, disadvantages and implications, the sub-committee recommends that the revised Code of Practice should be based on limit state philosophy.

Although the sub-committee has looked into the implications of limit state design if applied to all sections of the Code, the exact details of how it should be applied should be determined by the panels appointed to draft these sections.

4 BACKGROUND

a Conventional Design Methods

In the conventional design process a structure is designed so that the stress induced in the material does not exceed the "permissible stress". The permissible stress is normally arrived at in timber by dividing a minimum test value by a single reduction factor which allows for a number of variables such as long-term loading, size factors, inaccuracies in the design process, imperfect workmanship, possible variations in design load, etc.

The relationship between the applied loading which would cause the minimum test value to be reached, and the loading which would cause the structure to become unfit for use, is not necessarily constant but depends on the properties of the material, the shape of the members, the design of the structure, and other factors. The use of a single overall safety factor therefore leads to lack of uniformity with regard to safety between different types of structure, it also leaves doubts that some variables are not properly allowed for. In addition, the effects of further variables when they arise are difficult to include.

b Limit State Design

Limit state design offers a method of overcoming the above objections to conventional "permissible stress" design methods. The designer covers more directly those conditions, or "limit states" as they are called, at which the structure can be said to have become unfit for use. The many variables and unknowns which make safety factors necessary are allowed for in a more systematic manner, and this together with the introduction of statistical methods, means that the overall safety factor and the statistical probability that this will be inadequate may be predicted more accurately.

A "limit state" is reached when the structure becomes <u>unfit for use</u>. There can be a number of reasons why a structure becomes unfit for use and each one of these can be termed a "limit state". The most important limit states in timber design are those of collapse and excessive deflection (in concrete structures the limit state of excessive local damage due to cracking is also considered but this is not relevant to timber). Depending on the material, loading and type of structure, other limit states such as excessive vibration and fatigue, may have to be taken into account. With timber, biodeterioration and corrosion in fasteners could well be other limit states considered.

Briefly therefore, limit state design consists of defining those conditions which would render a structure unfit for use and of producing a design such that the chances of these limit states being reached during the intended life of the structure are acceptably remote. Account has therefore to be taken of the variations in loads and in the properties of the materials and appropriate allowances made for inadequacies in methods of analysis and in the quality of construction. This approach provides greater flexibility and scope for the engineer to achieve safe and serviceable structures at an economic price since the limit states and the risk or probability of failure can be varied depending upon the type of structure.

Ideally the variation in magnitude of the loads during the life of a structure should be expressed statistically so that the maximum likely load, corresponding to any particular probability of occurrence can be determined. This load is known as the CHARACTERISTIC LOAD. For the immediate future the characteristic load in most cases cannot be so defined because of lack of data and it will be necessary to assume that the characteristic load is the load given in CP 3, Chapter V, Part I, or other appropriate source. It should be noted however that CP 3, Chapter V, Part II, presents wind loads in a form which enables characteristic load values to be derived in the above manner. The characteristic load takes into account expected variations but does not allow for:

- i loads significantly different from those assumed in the design
- ii lack of precision in design calculations
- iii inadequacy in the method of analysis
- iv dimensional errors in construction which alter loads or effects of loads

Partial factors are therefore introduced for each limit state which reflect the degree of uncertainty arising from the above considerations and associated with the consequences if a particular limit state is reached. Thus the loads used in design for a particular limit state, are the characteristic loads multiplied by a partial factor γ_F ie

$$\text{DESIGN LOAD} = \text{CHARACTERISTIC LOAD} \times \gamma_F$$

Similarly, the variation in strength properties of the materials should be expressed statistically so that the minimum ultimate value, corresponding to a particular probability of occurrence, can be determined. This ultimate strength is known as the CHARACTERISTIC STRENGTH. This characteristic strength is then reduced by a partial factor γm (which may be the product of a number of separate factors) to adjust from the test and quality conditions under which strength was measured to the quality and load etc conditions associated with the structure. Thus

$$\text{DESIGN STRENGTH} = \text{CHARACTERISTIC STRENGTH}/\gamma m$$

This general principle of design is obviously applicable to all materials and has been incorporated in the draft Code of Practice for concrete. It has also been recommended by the BSI joint Committee on Codes of Practice, that other material codes should follow this principle. The structure may be analyzed by any method recognized as appropriate to the material and the limit state being examined. In many cases these methods are already available and familiar to designers and represent no change from current procedures.

5 JUSTIFICATION FOF CHANGE

The main reasons for recommending a change to limit state design are as follows:

a The Codes for other materials have, or are progressing towards, limit state design and it is not in the interests of timber to lag behind.

b Codes concerned with load specification are being developed on a statistical basis in which it could be difficult or uneconomic to use current conventional design methods.

c Limit state design gives engineers much more freedom to make a realistic appraisal of overall safety margins in the light of their own judgement and experience by allowing them to decide for themselves appropriate safety factors for specific structures.

d It is imperative that timber design methods should keep abreast with new information and limit state methods make this easier to achieve.

e Timber design methods, including the statistical derivation of working stresses from tests and incorporating factors relevant to different conditions are very near to a limit state approach and a change-over would be easier than for other materials.

6 DISADVANTAGES

a There are no significant benefits to be gained for timber in the short-term by changing to a limit state approach to design in terms of economy.

b Designers are familiar with the present CP 112 methods and any change will involve extra effort with no immediate benefit.

c Span tables, design charts and aids, will need to be recalculated.

d Different species will not necessarily have the same relative design stress values as at present (this also applies to different plywoods and jointing devices).

7 METHOD OF STUDY

The basic principles underlying the philosophy of limit state design were studied by reference to the available literature.

Advice was also sought from Dr Bate of Building Research Station Who was largely concerned with the drafting of the unified BS

Code of Practice for Structural Concrete which is based on limit state design. Dr Bate was invited to one of the meetings of the sub-committee and was most helpful in elucidating certain points on which information was lacking. He endorsed the sub-committee's proposed methods of approaching the problems of introducing limit state design for timber.

The underlying objective throughout the sub-committee's deliberations was that limit state methods of design should result in structures which were known from experience or from current design procedures using up to date information to be satisfactory. For the purposes of the initial change-over to limit state design this would be achieved if member sizes arrived at by both methods of design were essentially the same.

It was felt that information on material properties (eg timber strength) was probably more comprehensive and reliable than that on loadings. The bias was therefore towards defining characteristic strengths and partial safety factors for materials (γ_m) as closely as existing data would permit. By equating these to characteristic loads appropriate partial safety factors for loads (γ_F) could be evaluated. It amounts to deriving load factors which give the answers we require. It transpired in discussions with Dr Bate that this was essentially the same approach as that adopted by the committee when drafting the Code of Practice for Structural Concrete.

In the USA, Scandinavia and certain other countries a 1 in 20 exclusion level in the derivation of design stress for timber is being used. The characteristic strengths for concrete are also based on a 1 in 20 exclusion level and it seemed desirable that this should be adopted for timber. By changing from the present 1 in 100 exclusion level the relative strengths of different species would change marginally. This was considered to be acceptable.

Preliminary calculations showed that the limit state method of design could be used successfully for bending stresses, combined stresses and column design in solid timber, using a value of γ_F of 1.3 generally for strength and 1.0 for deflection.

With the present limited knowledge on loadings and frequencies it was thought impracticable and inappropriate to postulate different values of γ_F for dead and live loads, as is done for concrete. Initially the same value of γ_F for dead and live loads makes design procedures simpler, but different values may be introduced as more data become available. Concrete is recommending $1.4\ W_D + 1.6\ W_I$, when considering dead and imposed loading, timber will be $1.3\ W_D + 1.3\ W_I$.

The values of characteristic strengths for different species, grade factors and values of γm should be determined by the panels responsible for redrafting this section of the Code.

The sub-committee then considered how limit state concepts could be applied in principle to other aspects of design eg plywood, glulam, joints etc.

Where strengths are derived on a statistical basis similar to that for solid timber, then design could follow the same basis as for solid timber.

The method of dealing with joints would depend to a large extent upon the statistical data available. Where sufficient data were available, then the same approach as for timber would be appropriate. The characteristic strength of joints may be governed either by the limit state of collapse or in certain cases by the limit state of joint slip. For jointing methods where statistical data are not available, the characteristic strengths may have to be determined initially by simple factorization of existing strength values. When more data become available a true statistical approach could be implemented.

8 REFERENCES

a Why Limit State Design - S C C Bate, BRS, Current Papers 52/68.

b BSI Code Drafting Committee BLCP/80. The Structural Use of Concrete. Redraft of Section 8 "Design-Objectives and General Requirements". November 1970.

c BSI Code Drafting Committee BLCP/17. The Structural Use of Timber. Paper dealing with New Code of Practice for Wind Loads. February 1971.

d Building Research Current Papers. F G Thomas - "Basic Parameters and Terminology in the Consideration of Structural Safety".

e The New Code of Practice for Wind Load and its implications for the Structural Design Codes. The Structural Engineer. February 1971.

PRELIMINARY OUTLINE OF CONTENTS
REVISION OF CP112

CONTENTS

REVISION OF CP112 - FIRST DRAFT JULY 1972

PREPARED BY THE BUILDING RESEARCH ESTABLISHMENT, PRINCES RISBOROUGH LABORATORY.

SECTION ONE: GENERAL

1.1 SCOPE

This Code of Practice deals with the structural use of timber, plywood and other wood based sheet materials. It is based on the principles of engineering design, on data on material properties established by research and on the performance testing of full size structures.

Where information was lacking from British sources recourse was made to information from the Forest Products Laboratories of the USA, Canada, Sweden and Finland, to whom due acknowledgement is made. The valuable contributions of the Council of Forest Industries of British Columbia, the Swedish Timber Council and the Finnish Plywood Development Association are also acknowledged.

The Code includes recommendations on design procedures to permit the adoption of a limit state approach. It gives partial safety factors to enable design loads to be determined for the limit states of ultimate strength and deflection and recommends characteristic basic stresses for the materials and characteristic joint strengths for nailed, screwed and bolted joints. Characteristic grade stresses applicable to material which conforms to specified grading rules or is selected by an approved method of machine grading are included, together with partial safety factors to enable these, and the characteristic joints strengths, to be adjusted to permissible stress and permissible strength values for particular design situations.

As an alternative to theoretical design a method is recommended whereby the ability of a particular structure to satisfy its functional requirements for strength and stiffness may be determined from tests on full size units.

1.2 DEFINITIONS

Where timber terms are used in this Code they have the meaning assigned to them in BS565, BS4471 and BS1860. In addition the following definitions apply:

Characteristic basic stress. The stress which can be sustained under long term load in a dry exposure condition by material containing no strength reducing characteristics or defects.

Characteristic grade stress. The stress which can be sustained under long term load in a dry exposure condition by material of a particular visual or machine grade.

Characteristic joint strength. The force which can be sustained under long term load in a dry exposure condition and for a particular direction of loading by a particular type of joint.

Characteristic loads

a Characteristic long term load. The total load acting permanently on a member or structure including dead load as defined in CP 3:Chap V:Part 1 and any imposed loads which are known to be of a permanent nature. Examples of imposed load which should be considered as permanent include the imposed load for floors of domestic premises defined in CP 3:Chap V: Part 1 and any loading in roof spaces due to storage.

b Characteristic medium term load. The imposed load which is not permanent but which may be applied from time to time for prolonged periods. Examples of medium term imposed loads include the uniformly distributed imposed loads, for roofs as defined in CP 3:Chap V:Part 1, and for shuttering.

c Characteristic short term load. A wind load for Class C (15 sec averaging time) as defined in, and calculated in accordance with, CP 3:Chap V: Part 2 or the concentrated imposed loads for roofs, ceilings, skylights etc as defined in CP 3:Chap V:Part 1.

d Characteristic very short term load. An impact load or wind load for Class A or Class B (3 sec and 5 sec averaging time) as defined in, and calculated in accordance with, CP 3:Chap V:Part 2.

Design load. The load determined by multiplying the characteristic loads by partial safety factors appropriate to a limit state of ultimate strength or deflection.

Dry exposure. An exposure condition, as for example within a covered building, where the moisture content of timber will not exceed 18 per cent.

Glued laminated timber. A member produced by gluing together a number of laminations having their grain essentially parallel.

Horizontally laminated beam. A beam whose laminations are parallel to the neutral plane.

Member. A structural component which may be a piece of solid timber, laminated timber, or built up from pieces of timber and other sheet materials (eg floor joist, box beam, member of a truss etc).

Permissible stress. The stress, determined by multiplying the characteristic stress by appropriate partial safety factors, which is applicable to the design of a member of a particular grade and size and for the conditions of exposure and loading to which the member will be subjected.

Permissible joint strength. The strength, determined by multiplying the characteristic joint strength by appropriate partial safety factors, which is applicable to the design of a joint under the conditions of exposure and loading to which the joint will be subjected.

Structural unit. An assembly of members forming the whole or part of a framework or building (eg truss, prefabricated floor panel etc).

Vertically laminated beam. A beam whose laminations are at right angles to the neutral plane, the laminations being of a grade defined for solid timber.

Wet exposure. An exposure condition where the moisture content of timber will, for significant periods, exceed 18 per cent.

1.3 SYMBOLS

The symbols used in this Code are as follows:

b	Breadth of beam or joist, thickness of web or least transverse dimension of a tension or compression member. Where there is more than one such dimension b_1, b_2, b_3 etc indicate their several values.
d	Depth of beam or joist, greater transverse dimension of a tension or compression member. Where there is more than one such dimension d_1, d_2, d_3 etc indicate their several values.
E_a	Average value of Modulus of Elasticity.

E_k — Characteristic value of modulus of elasticity corresponding to the lower 5 per cent exclusion limit.

f_{bg}, f_{tg}, f_{cg} — The characteristic grade stress in bending, tension and compression respectively, parallel to the grain.

F_{bp}, f_{tp}, f_{cp} — The permissible stress in bending, tension and compression respectively, parallel to the grain.

f_{ba}, f_{ta}, f_{ca} — The applied stress in bending, tension and compression respectively, parallel to the grain.

f_{cng} — The characteristic grade stress in compression perpendicular to the grain.

f_{cnp} — The permissible stress in compression perpendicular to the grain.

f_{sg} — The characteristic grade stress in shear parallel to the grain.

f_{sp} — The permissible stress in shear parallel to the grain.

f_{snp} — The permissible stress in rolling shear.

f_{tnp} — The permissible stress in tension perpendicular to the grain.

F_k — Any characteristic load.

C_a — Average value of modulus of rigidity.

G_k — Characteristic long term load.

I — Second moment of area of a cross-section, otherwise known as moment of inertia.

K_k	Characteristic impact load
l	Actual length or span of a member.
l_e	Effective length of a compression member.
M	Bending moment.
M_R	Moment of resistance.
Q_k	Characteristic medium term load.
r	Radius of gyration.
t	Thickness of lamination.
W_k	Characteristic short term load.
σ	Angle between the direction of load or force and the direction of the grain.
γ_f	Partial safety factor for load.
γ_m	Partial safety factor for material or joint strength. Where there is more than one such co-efficient γ_{m1}, γ_{m2}, γ_{m3} etc indicate the several values.

SECTION TWO: DESIGN OBJECTIVES

2.1 LIMIT STATE DESIGN

The object of design is the achievement of acceptable probabilities that the structure being designed will not become unfit for the use for which it is required during the intended service life, ie that it will not reach a limit state.

The characteristic stresses and loads used in design should take account of variations in the properties of the materials and in the loads to be supported. Where the necessary data are available these characteristic values are based on statistical evidence and where they are not available on an appraisal of experience. In general the characteristic stresses and loads are not used directly in design and two sets of partial safety factors are introduced, one for material and joint properties (γ_m) and the other for loads (γ_f) to enable permissible stresses and design loads to be determined for a particular design situation and limit state.

2.2 LIMIT STATE REQUIREMENTS

2.2.1 General

All relevant limit states should be considered to ensure adequate safety and serviceability. The usual approach will be to design on the basis of the most likely critical limit state and to check that the remaining limit states will not be reached.

2.2.2 Ultimate strength

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

In design calculations the permissible stresses, permissible joint strengths and the design loads shall be those specified in the appropriate section of this Code or derived in accordance with the recommendations of this Code. The most unfavourable combination of the presence or absence of loads likely to occur should be considered.

2.2.3 Deflection

The deflection of the structure or any member of the structure under the forces or loads that will be encountered in service should not adversely effect its serviceability, due regard being paid to the possibility of damage to surfacing materials, ceilings, partitions, the ponding of water and to aesthetic and psychological effects.

Under continuous loading timber and wood based sheet materials are subject to increasing deflection, some of which may be non-recoverable, and in certain circumstances account may have to be taken of this effect in design.

I Where a member is installed at a moisture content in excess of 18 per cent in an exposure which would normally be classified as dry and then dries out under design long term load or some part of this, the deflection caused by this load should be taken as 1.5 times the value calculated using the appropriate modulus of elasticity and/or modulus of rigidity values for the dry condition of exposure.

II For wet exposure conditions the deflection under design long term load should be taken as twice the value calculated using the appropriate modulus of elasticity and/or modulus of rigidity values for the wet condition of exposure.

III For dry exposure conditions where a member is installed at a moisture content not significantly in excess of 18 per cent this effect may be ignored.

IV Where members of a basic thickness in excess of 100 mm are used under the wet condition of exposure, or under the dry condition of exposure if no special provisions for kiln drying have been made, the deflections should be calculated using the appropriate modulus of elasticity and/or modulus of rigidity values for the wet condition of exposure and subject to the conditions in (I) and (II) above.

In all cases the engineer must satisfy himself that deflections are not excessive, having regard to the loading conditions and requirements of the particular structure. As a guide, and in the absence of any specific criteria indicating a higher or lower value, the following may be regarded as reasonable limits:

I the deflection of a flexural member under design load should not exceed 0.003 of the span.

II subject to due regard being paid to the possible effects of the increased deflection, members may be precambered to an amount equal to the deflection under dead load and in this case the deflection under the remainder of the design load should not exceed 0.003 of the span.

III the lateral deflection of a vertical member under the action of lateral forces arising from the effects of wind should not exceed 0.003 of their span.

IV the deflection normal to the length of rafters in house roofs should not exceed 0.004 of their span under design load.

V the deflection of the purlins in house roofs should not exceed 0.003 of their span under design load.

VI the deflection of beams over windows or other openings should not exceed 0.002 of their span under design load and special provision may be required to prevent excessive load being transmitted to any frame beneath.

VII the deflection of flooring under design load in house floors should not exceed 0.003 of the distance between the supporting joists or 1.5 mm whichever is the smaller.

In determining deflections account should be taken of the effect of joint slip or rotation and any tolerances on fit permitted in framed structures.

For the purpose of calculating deflections the permissible modulii and design loads shall be those specified in this Code or derived in accordance with the recommendations of this Code. (See Clause).

2.2.4 Durability

To achieve the service life required for a structure condideration should be given, depending on the exposure conditions to the durability characteristics of the species of timber, adhesives and jointing devices used and to the need to provide protection against decay, infestation by wood destroying insects and corrosion. Attention should be paid at the design stage, in detailing and during construction to the possibility of moisture absorption, moisture ingress and condensation and to the avoidance of moisture traps which might adversely effect performance.

2.2.5 Fire Resistance

In the design of a structure attention should be paid to the provision of adequate fire stops and the materials used must be capable of satisfying either naturally, or by protection or treatment, the requirements for surface spread of flame and fire resistance as specified in building regulations.

2.2.6 Vibration

Where there is a likelihood of a structure being subjected to vibration from causes such as impact forces, wind or machinery, measures should be taken to prevent discomfort or alarm, damage to the structure or interference with its proper function.

A number of criteria may be required to specify acceptable limits to the vibrational characteristics of a structure or structural unit and reference should be made to the specialist literature. As a guide the vibrational characteristics of a timber joist floor construction will generally be acceptable if the deflection under the design load does not exceed 13 mm for floors in dwellings and 6 mm for floors in gymnasiums, dance halls etc.

2.2.7 Other limit states

A structure or part of a structure may have to satisfy serviceability limit states for thermal insulation and sound insulation or other limit states associ with unusual or special functions and these should be taken into account in the design.

2.3 DESIGN LOADS

2.3.1 General

The characteristic loads on a structure should ideally be determined from knowledge of the magnitude of the loads that occur in practice and their frequency distributions over the service life of a structure. The characteristic loads should be those which correspond to a defined probability of occurrence. Since it is not yet possible to express loads in these statistical terms the following characteristic loads should be used to determine design loads for the limit states of ultimate strength and deflection. In order that account may be taken of the effect that load duration has on the strength of timber and wood based materials, four categories of load are given which are broadly related to the length of time that the load will be applied.

I The characteristic long term load G_k

II The characteristic medium term load Q_k

III The characteristic short term load, W_k

IV The characteristic very short term load, K_k

Any exceptional or abnormal loads associated with a particular type of structure must be taken into account and should be assigned, on the basis of an estimate of their probable duration, to one of the four categories given above. In design account should also be taken of the loading conditions occurring during construction and erection to ensure that subsequent compliance of the structure or structural unit with the limit state requirements is not impaired.

The design load is obtained from the characteristic load by multiplying the characteristic load (F_k) by the appropriate partial safety factor for load (γ_f) ie

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

The partial safety factor γ_f is introduced to take account of:

I possible unusual increases in load beyond those considered in deriving the characteristic load.

II inaccuracies in assessment of the effects of loading.

III variations in dimensional accuracy achieved in construction.

IV the importance of the limit state being considered.

2.3.2 Ultimate strength

In designing for the ultimate strength limit state, because of differences in the corresponding permissible stress values for the materials and permissible joint strengths, the long term load acting alone and in conjunction with the other categories of load should be separately considered. In assessing the effect of these loads on the structure as a whole, or on a part of the structure, the arrangement of the loads, and their presence or absence, should be such as to cause the most severe condition. For the ultimate strength limit state the partial safety factor for load γ_f should be taken as 1.3.

However where the application of the imposed and/or wind loads causes stresses which decrease and/or reverse the stresses due to the dead load the partial safety factor for the dead load should be $\gamma_f = 0.9$. When considering the design of part of the structure under a combination of loads if a more unfavourable condition results by taking the dead load with a γ_f of 1.0 or 1.3 in any other part of the structure, then this value should be used.

2.3.3 Deflection

For the deflection limit state the partial safety factor for load γ_f shall be taken as

a 1.0 where up to two characteristic loads are combined to give the design load and

b 0.8 where three or more characteristic loads are combined to give the design load.

The worst combination of characteristic loads shall be considered for design purposes. However if a more severe condition is created by selecting only parts of the structure to be loaded with the imposed and/or wind load then the arrangement of these loads should be such as to cause the greatest deflection.

SECTION THREE: MATERIALS

3.1 GENERAL REQUIREMENTS

The materials used should comply with the appropriate British Standards, where such exist, or with other national standards as indicated.

3.1.1 Timber The following standards apply to timber:

Glossary of terms relating to timber and woodwork	BS 565
Nomenclature of commercial timbers, including sources of supply	BS 881:589
Specification for dimensions for softwood	BS 4471
Specification for dimensions for softwood, Part 2, small resawn sections	BS 4471 Part 2
Grading and sizing of softwood flooring	BS 1297
Grading of timber for structural use	BS 1860
National lumber grading and dressing rules	Canada
Instructions for grading and marking of timber T-virke	Sweden 1966

3.1.2 Laminated timber. The following standards apply to laminated timber:

Glued laminated timber structural members	BS 4169

3.1.3 Plywood. The following standards apply to plywood:

British made plywood for marine craft	BS 1088
Plywood manufactured from tropical hardwoods	BS 1455
Information about plywood	BS 3493
British made plywood for structural purposes	ABPM Spec BP 101
Douglas fir plywood	Canadian Std 0121
Finnish birch plywood	Finnish Std O.IV.1

3.1.4 Mechanical Fasteners. The following specifications apply to mechanical fasteners:

Black bolts, screws and nuts	BS 916
Nails	BS 1202
Aluminium nails	BS 1202 Pt 3
Wood screws	BS 1210

3.1.5 Adhesives. The following specifications apply to adhesives:

Synthetic resin adhesives (phenolic and aminoplastic) for plywood	BS 1203
Synthetic resin adhesives (phenolic and aminoplastic) for wood	BS 1204
Cold setting casein glue for wood	BS 1444

3.1.6 Preservatives. The following specifications apply to preservatives:

Coal tar creosote for the preservation of timber	BS 144
Pressure creosoting for timber	BS 913
Classification of wood preservatives and their methods of application	BS 1282
Copper/chrome water-borne wood preservatives and their application	BS 3452
Fluoride/arsenate/chromate/dinitrolphenol water-borne wood preservatives and their application	BS 3453
Treatment of plywood with preservatives	BS 3842

3.2 SPECIES OF TIMBER

This Code is based on a limited number of timbers likely to be generally available (see Table 1) but because sources of supply vary it is not possible to include an exhaustive list.

Since at the design stage it may not always be possible to specify a particular species this Code also gives six stress groups and lists examples of softwood and hardwood species which qualify for these groups. A species which qualifies for a particular stress group will have characteristic basic stress values not lower than those quoted for the group.

For species not tabulated advice should be sought from the Building Research Establishment, Princes Risborough Laboratory or the Timber Research and Development Association as to the stress groups to which they should be assigned, the stresses appropriate to particular species if adequate data are available, and to their suitability for a particular use.

TABLE 1 NAMES AND DENSITIES OF SOME STRUCTURAL TIMBERS

Standard Name	Botanical species	Other common names	Approximate Density at a moisture content of 18 per cent kg/m³
SOFTWOODS			
Douglas fir	Pseudotsuga menziesii	B C pine Oregon pine	590
Western hemlock	Tsuga heterophylla with Abies spp and Tsuga mertensiana	Hembal Hem-fir	530
Larch	Larix decidua Larix leptolepsis	–	560
Parana pine	Araucaria augustifolia	–	560
Pitch pine	Pinus palustris Pinus elliotti Pinus caribaea	Longleaf pitch pine N caraguan pitch pine Honduras pitch pine	720
Redwood or Scots pine	Pinus sylvestris	Baltic redwood, European redwood, deal, Swedish pine	540
Canadian spruce	Mainly Picea glauca, Picea mariana Pinus contorta and Abies spp	Princess spruce Western white spruce	450
Home-grown European spruce	Picea abies	Norway spruce	380
Home-grown Sitka spruce	Picea sitchensis	–	400
Whitewood	Picea abies, Abies alba	Baltic whitewood, European whitewood, white deal	510
HARDWOODS			
Greenheart	Ocotea rodiaei	–	1060
Gurjun/keruing	Dipterocarpus spp	–	720
Iroko	Chlorophora excelsa	mvule	690
Jarrah	Eucalyptus marginata	–	910
Karri	Eucalyptus diversicolor		930
Oak	Quercus robur	–	720
Opepe	Nauclea diderrichii	kusia	780

* Some specifications admit Pinus echinata, P taeda, P rigida and P virginiana which are considerably lighter and of lower strength. The data given in this Code do not apply to these species.

SECTIO

SECTION FOUR: MOISTURE CONTENT

4.1 GENERAL

The basic factors governing the moisture content of timber and wood based sheet materials in buildings are:-

I Wood is a hygroscopic material; its moisture content therefore depends on its exposure conditions.

II Unless wood is in contact with water or exposed to damp conditions, its moisture content stabilizes, in most cases, at between 10 and 20 per cent, which is very much lower than when the timber is freshly felled. The drier the atmosphere and the higher the temperature, the lower the moisture content which the wood attains.

III At a moisture content below about 30 per cent wood shrinks or swells as its moisture content changes.

IV The strength properties of wood change with changes in moisture content below about 30 per cent; an increase producing a decrease in strength and vice versa.

V Wood is less prone to decay if its moisture content is below 25 per cent and may be regarded as immune below 20 per cent.

VI Imported softwood from European sources is normally dried to a moisture content below 23 per cent before shipment to the UK. Softwoods of Canadian origin will generally be unseasoned, but dipped in a fungal inhibitor before shipment. Packaging and other changes in timber handling may however alter this traditional pattern.

Imported hardwoods from all sources are normally dried before shipment only to such an extent as will avoid deterioration during passage to the UK. The moisture content at the time of arrival may be anywhere within a wide range, but further drying is normally done after arrival. Supplies of dried hardwood are generally available.

VII Plywood and other wood based sheet materials are produced in a relatively dry condition with moisture contents within the range 6 to 14 per cent. Although their hygroscopic properties are not the same as those of timber they undergo changes in strength and in dimensions, of which thickness is affected the most, with changes in the conditions of exposure.

All timber whether imported or home grown, which is thoroughly air-dried in this country attains a moisture content between 17 and 23 per cent, depending on the weather conditions prevailing at the end of the drying period. If the equilibrium moisture content in service is lower, or if special requirements exist, as for example for laminated timber, then it will be necessary to kiln-dry the timber.

Care should be taken on site to ensure that materials supplied and to be used in a dry condition are adequately protected from the weather.

4.2 SERVICE REQUIREMENTS

Timber when it is installed in a structure should have a moisture content complying with the limits appropriate to the use as given in Table 2. Timber in categories 3, 4 and 5 require kiln-drying to achieve the specified moisture contents.

TABLE 2 MOISTURE CONTENT OF TIMBER FOR VARIOUS POSITIONS

Category	Position	Exposure	Limits of moisture content (per cent)	
			Min	Max
1	Framing, cladding and external finishings	Partly or entirely external	14	20
2	Framing including roof members and floor joists	Internal	12	20
3	Flooring, ceiling and wall linings	Intermittently heated	12	16
4	Flooring, ceiling and wall linings	Centrally heated	10	14
5	Flooring, ceiling and wall linings	Close proximity to heat source	6	10

Although some timber after fixing may be subject to temporary exposure to the weather and therefore to possible absorption of moisture it will rapidly dry out after covering, providing there is ample ventilation. Sufficient time should be permitted, as necessary, for drying to occur before a member is fully enclosed or is subjected to its design load.

Timber members fixed at higher moisture contents than those in Table 2 will suffer differential and uneven shrinkage in drying out in situ to the equilibrium moisture content of the service condition. Where this is acceptable, provision should be made to obviate any loss in strength at joints and fixing points and to accommodate any additional deflections or displacements.

Timber more than 100 mm in thickness cannot be reduced to a uniform moisture content within the limits of Table 2, except at an uneconomic cost.

Before the fixing of timber in categories 3, 4 and 5 of Table 2 the wet trades should be completed and their work dried out, adequate ventilation being provided throughout the drying period to remove the high humidity air from the building.

4.3 STRENGTH PROPERTIES

Because of the effect of moisture content on strength the permissible stress values to be used in design should be those corresponding to the higheset moisture content that the particular member will attain in service. It is not however possible to deal fully with all possible exposure conditions and for the purposes of this Code two standard conditions only are defined. These are:

1 Dry exposure condition. An exposure condition where the moisture content of timber will attain an equilibrium value not in excess of 18 per cent.

2 Wet exposure condition. An exposure condition where the moisture content of timber will attain an equilibrium value in excess of 18 per cent.

Where members more than 100 mm in thickness are used the permissible stresses for the wet exposure condition should be used, irrespective of the actual exposure condition, and the other factors given in clause 2.2.3 should be taken to apply.

4.4 GEOMETRICAL PROPERTIES

The actual dimensions of a member will vary with its moisture content and allowance may have to be made for this in determining the geometrical properties for use in design.

The basic sizes of sawn timber are specified in BS 4471 at a moisture content of 20 per cent. For any higher moisture content up to 30 per cent the size will on average be greater by 1 per cent for every 5 per cent of moisture content in excess of 20 per cent, and for any lower moisture content the size may be smaller by 1 per cent for every 5 per cent of moisture content below 20 per cent. For any moisture content higher than 30 per cent the size will not be greater than at 30 per cent.

Where a timber member is processed to size at the equilibrium moisture content it will attain in service then the geometrical properties of the actual section should be used in design.

Where a member is specified and supplied in accordance with BS 4471 as sawn, regularised or precision timber the basic sizes may be used to calculate the geometrical properties of members for use in the dry exposure condition. For timber members for use in the wet exposure condition the geometrical properties may be increased by the following factors:

breadth, depth, radius of gyration	1.02
area	1.04
section modulus	1.06
second moment of area	1.08

NORWEGIAN STANDARD — NS 3080

1. Ed. Nov. 1972

NORGES BYGGSTANDARDISERINGSRÅD — UDK:691.113

(Norwegian Council for Building Standardization) — SfB:Hil

This is a translation of the standard. Only the Norwegian text have been adopted as Norwegian Standard.

QUALITY SPECIFICATIONS FOR SAWN TIMBER AND PRECISION TIMBER

1. SCOPE

This standard defines quality specifications for sawn timber and precision timber of sprunce and pine.

2. DEFINITIONS

Bark - the outer protective layer of the trunk. Should not be mistaken for bast.

Brown stain - grayish-brown discoloration caused by dyes in the bark. Should not be mistaken for decay.

Bark pocket - ingrown bark.

Bast - light, colored soft tissue on inside of bark. Some of it may be left on the wane. On drying it becomes brittle and darker in colour.

Blue stain - discoloration caused by fungi, mostly blue. Blue stain does not affect the strength properties (Fig. 2)

Hard decay - decay started during storage. Shows up as light-brown streaks or patches. Does not affect the holding of nails (Fig. 3)

Slope of grain - deviation of the direction of grain from the longitudinal direction of timber, measures on the face cut from the outer side of log. Local disturbances of the grain at knots or the like, are not taken into account. The slope of grain is determined by means of a special scribe or from splits or streaks of blue stain, but not from the growth rings.

Square	- sawn timber of dimensions exceeding 70 mm and with the face up to about 1/3 larger than the edge. The pith should be approximately at the centre.
Bow	- curvature of the face of the timber in direction of length. (Fig.16)
Face	- the two wider sides of the timber (Fig. 1)
Moisture content	- amount of moisture in timber expressed as a percentage of its dry weight.
Rind gall	- originally surface wound that has been enclosed by the growth of the three. (Fig.14)
Defects due to insects	- tunnels in the wood caused by worms or insects. (Fig. 4)
Precision timber (J)	- timber satisfying requirements on cross-sectional dimensions and tolerances according to NS 3043.
Spring	- curvature of the edge in the direction of length. (Fig 17).
Edge	- the two narrower sides of the timber. (Fig. 1)
Cup	- curvature occuring in the cross-sectional direction of the timber. (Fig. 18)
Pitch pocket	- opening between two growth rings, normally filled with resin. The resin pocket can be "on edge" or "flat" depending on the direction of section. (Fig. 5)
Knots	-
- bark-ringed knot (encaes knot)	- dry knot, completely or partially surrounded by bark. (Fig. 6)
- sound knot (intergrown knot)	- knot, fully intergrown with the surrounding wood. (Fig. 7)
- knot cluster	- two or more knots appearing on a length of the timber equal to the width, up to 150 mm.
- pin knot	- sound or dry knot with a diameter of 7 mm or less. (Fig. 8)
- rotten knot	- knot, fully or partially attacked by rot. (Fig. 9)

- dead knot (encased knot)	- knot, not intergrown with the surrounding wood. Darkcoloured, dry knot is called "black knot". (Fig. 7)
Internal face (Pith side)	- the face of the timber which has turned towards the pith of the log. (Fig. 1)
Rise	- the greatest deviation of a side from a straight line. (Fig. 17)
Rot	- decomposition of the wood caused by fungi. Rot may have different colours. (Fig. 10)
Hair surface check	- see: Checks and shakes
Sawn timber(s)	- by sawn timber means here both timber produced directly from the log and timber produced bly cleaving, splitting or the like.
Checks and shakes	
- ring shake	- separation of the growth rings.
- hair surface check	- fine separation of wood fibres along the grain. Normally found on the edge, and often darkcoloured.
- check	- longitudinal separation of wood fibres and following the direction of grain. When the timber is diagonally grained it may consist of several shorter, oblique separations. (Fig. 11)
Compression wood	- hard, brown or reddish wood, mostly darker in colour than normal wood. Should not be mistaken for heartwood. May give deformation of the timber. (Fig. 12)
Pitch wood	- wood which is saturated with resin. (Fig. 13)
Wane	- the part of the surface which is untouched by sawblade or planer. (Fig. 1)
Twist	- spiral distortion
Swirl	- "transverse wood" caused by the top of the tree breaking off, or some other disturbances during the growth. Twist will always give rise to the development of falce heartwood. (Fig. 15)

Weather gray	- discoloration caused by the action of weather.
External face	- the face of the timber that has been on the same side as the bark of the tree. (Fig. 1)
Width of annual ring	- the average width of the growth rings lying more than 25 mm from the pith.

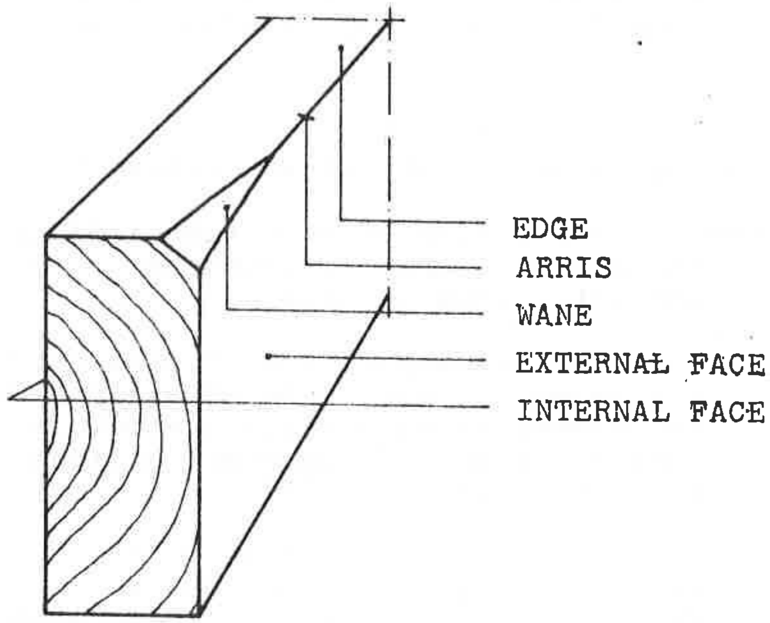


Fig. 1

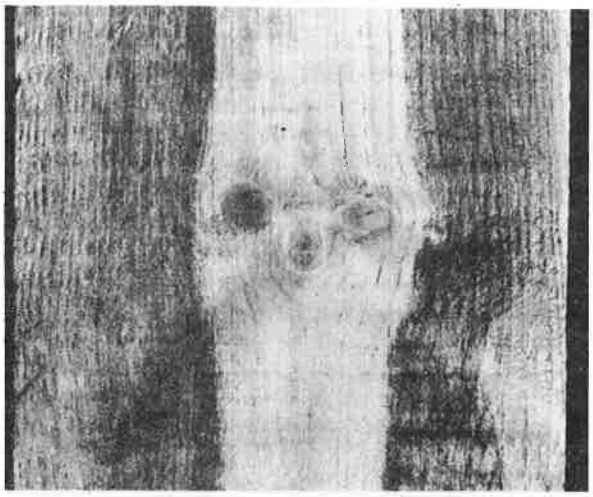


Fig 2   Blue sap stain

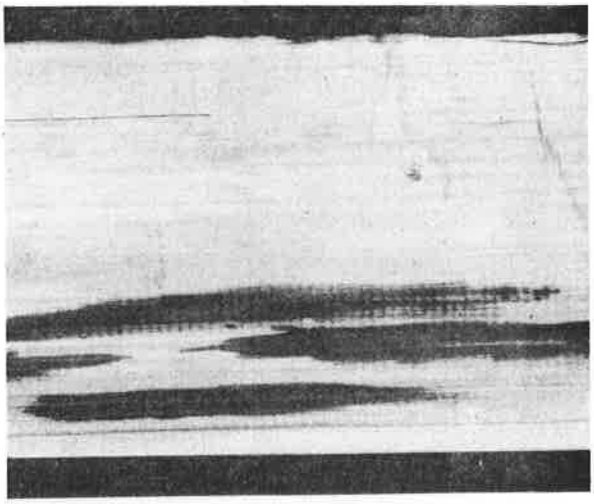


Fig 3   Hard decay

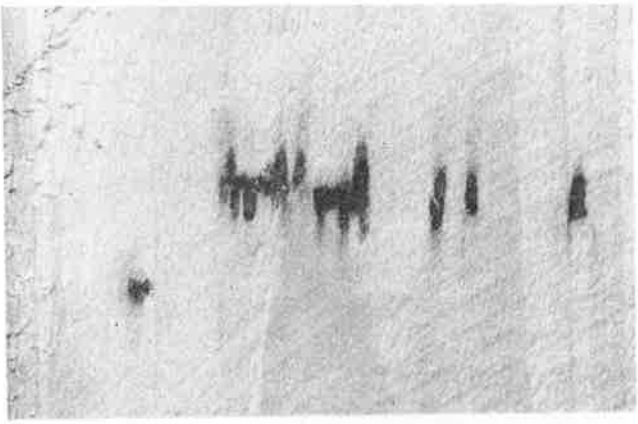


Fig 4   Defects due to insects

Fig 5   Pitch pockets

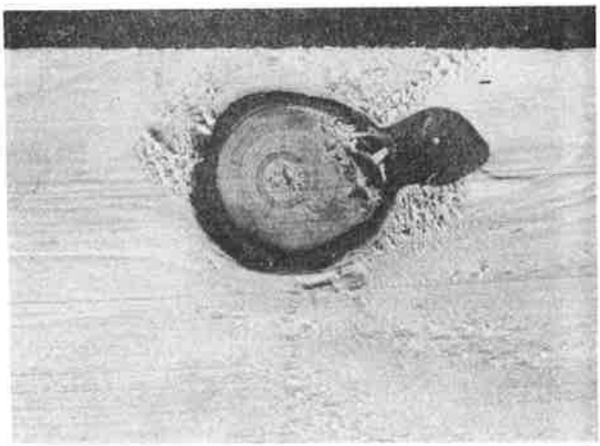


Fig 6   Barkringed knot

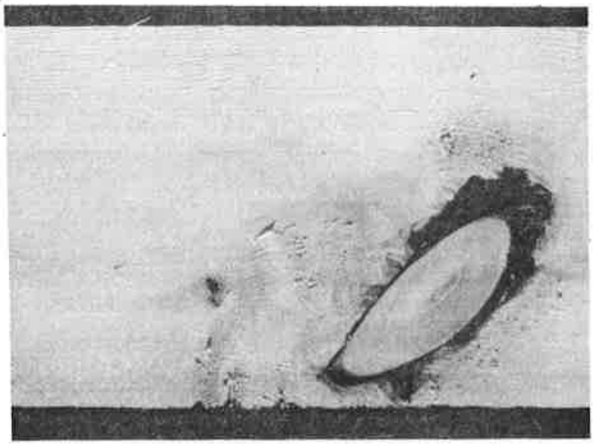


Fig 7a Dead knot

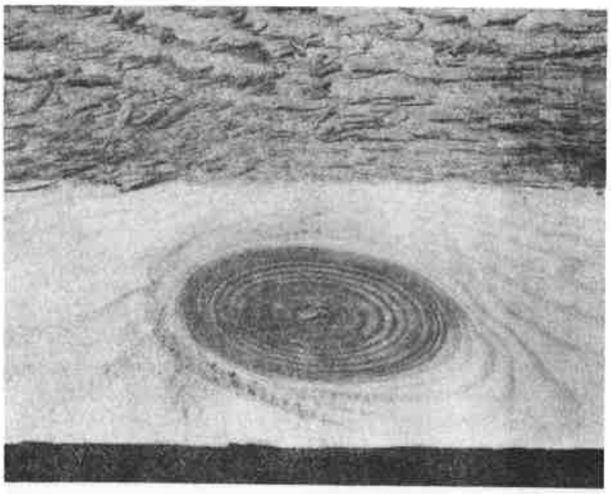


Fig 7b Sound knot

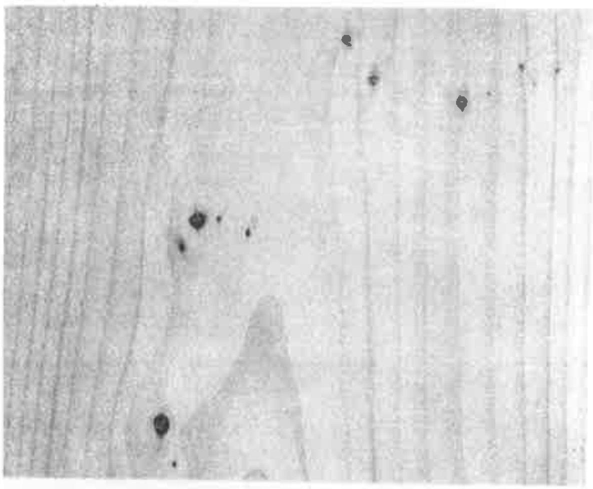


Fig 8 Pin knots

Fig 9 Rotten knot

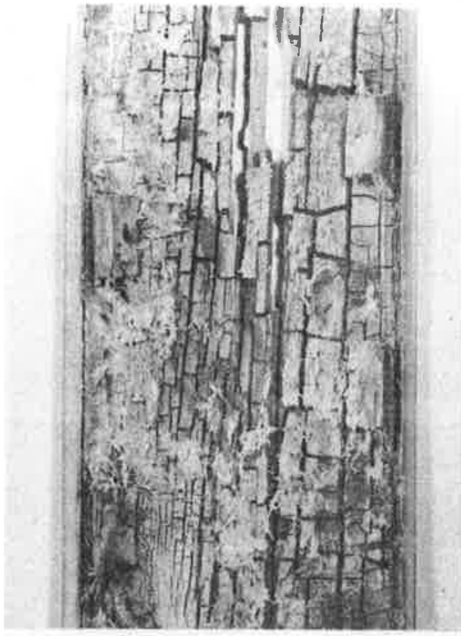


Fig 10 Rot

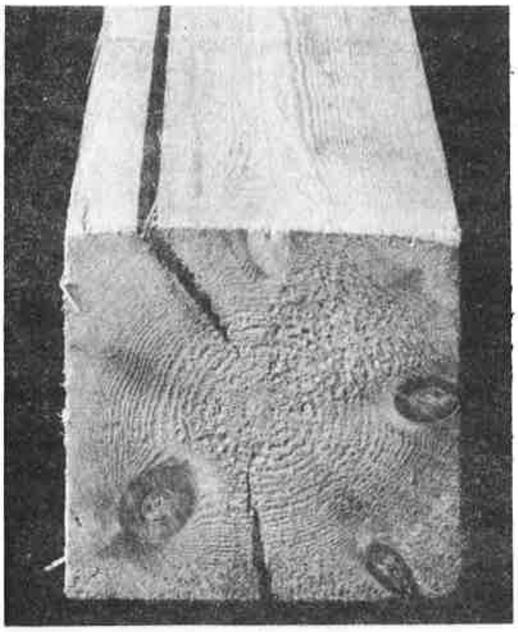


Fig 11 Check

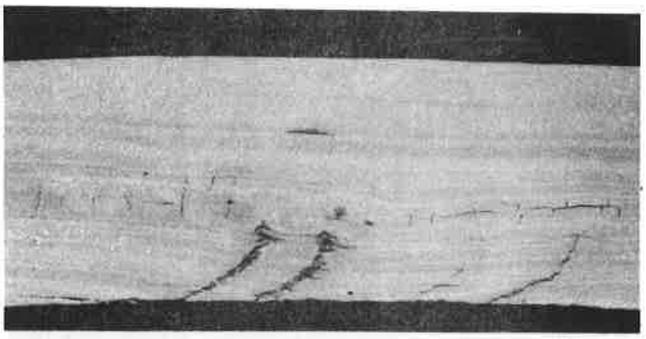


Fig 12 Compression wood

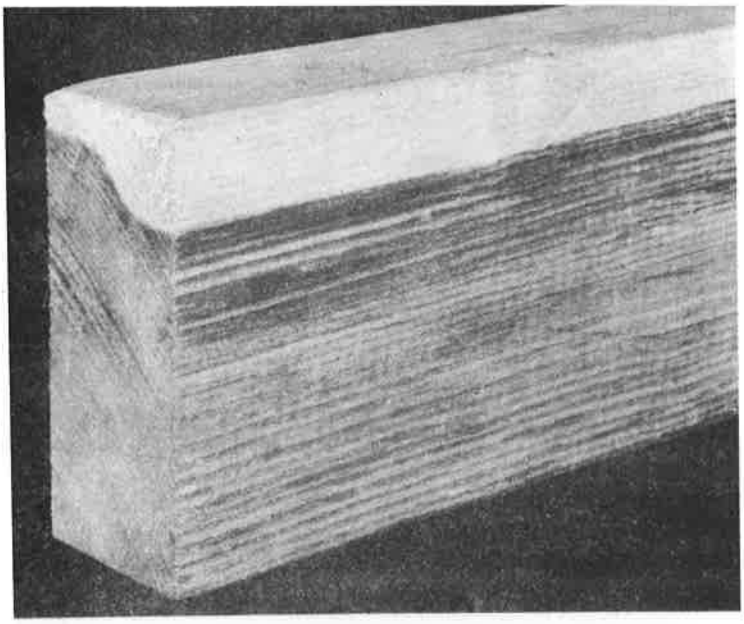


Fig 13 Pitch wood

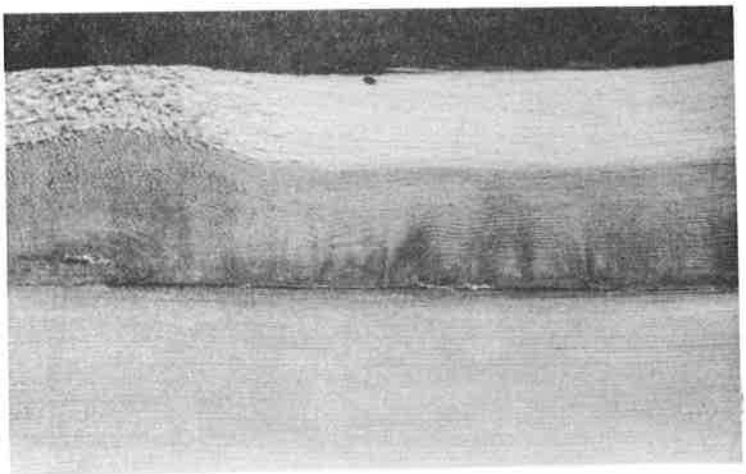


Fig 14 Rind gall

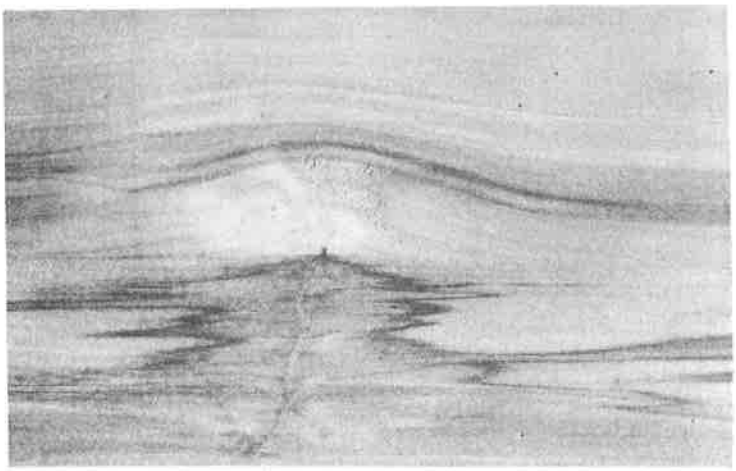


Fig 15 Swirl

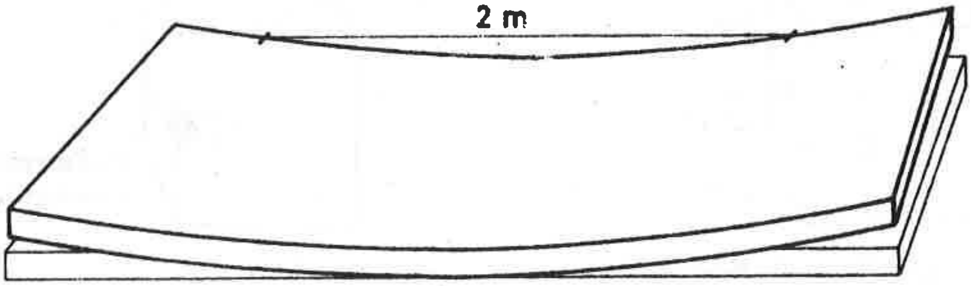


Fig. 16 Bow

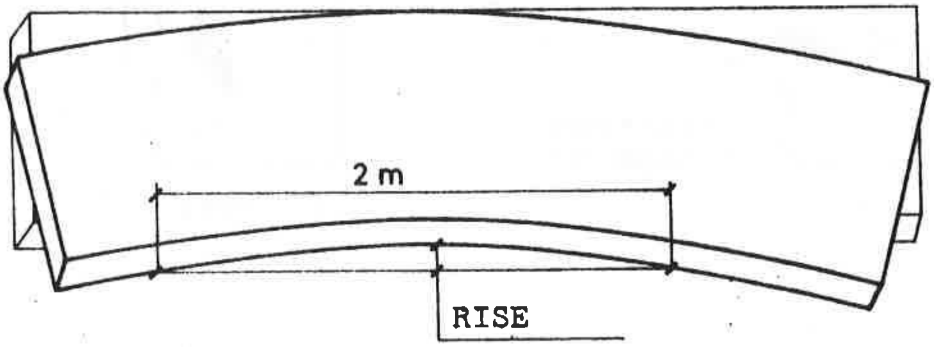


Fig. 17 Spring

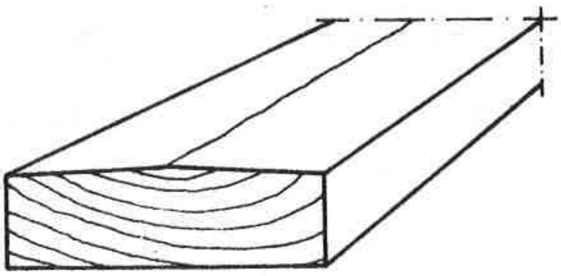
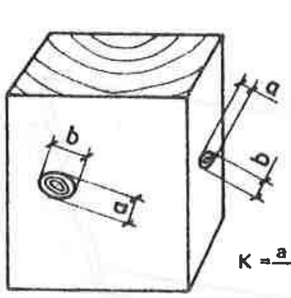
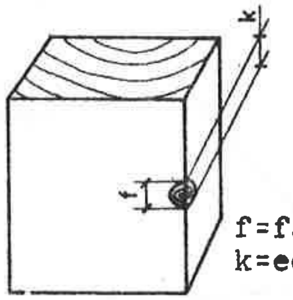


Fig. 18 Cup



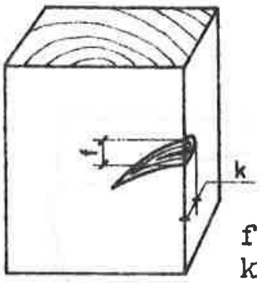
$K = \frac{a+b}{2}$

Faceknot and edgeknot



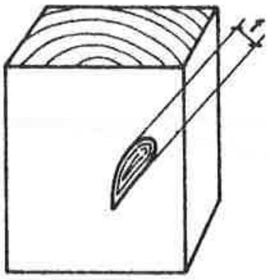
f=faceknot
k=edgeknot

Arris knot

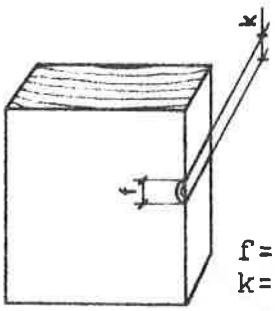


f=faceknot
k=edgeknot

Splay knot

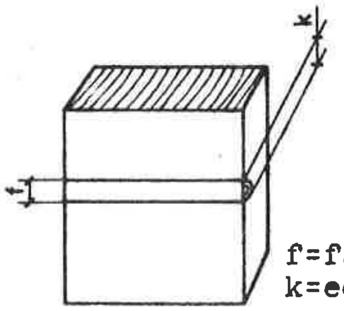


Leaf knot



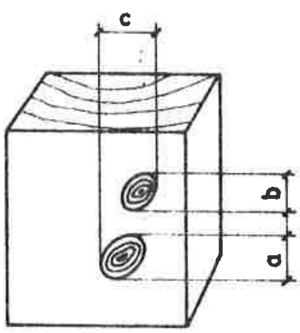
f=faceknot
k=edgeknot

Face-to-face knot



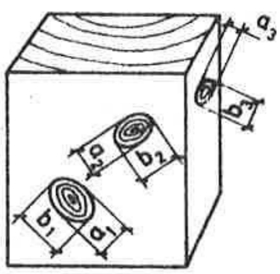
f=faceknot
k=edgeknot

Edge-to-edge knot



$K = \frac{a+b+c}{2}$

Knot cluster - overlapping



$K = \frac{a_1+b_1}{2} + \frac{a_2+b_2}{2} + \frac{a_3+b_3}{2}$

Knot cluster - single knot

Fig. 19 Measurement of knot

3. DESIGNATION

Timber graded according to this standard should be designated by stating the following successively: S (for sawn timber) or J (for precision timber), thickness x width (in mm) x length (in meter), grade (see clause 6) and the number of this standard, eventually with the addition of the wood species, or by code after Table 1,

e.g.:

S 50 x 100 x 4.2 E NS 3080 spruce

or Hi 1.200

Table 1

Code	Text
Hi 1.200	Timber quality E NS 3080
Hi 1.300	Timber quality S NS 3080
Hi 1.400	Timber quality C NS 3080
Hi 1.500	Timber quality Sx NS 3080

4. RULES OF MEASUREMENT

For squares all sides are regarded as edges.

For timber with greatest side 50 mm or less all sides are regarded as edges.

Knots are measured as shown in fig. 19.

Width of annual rings is measured at that end where the growth rings are greatest.

5. COMMON REQUIREMENTS

5.1 Sawn timber and precision timber shall have dimensions and tolerances as given in NS 3042 and NS 3043 respectively.

5.2 In any cross-section at least 2/3 of any surface shall have been processed.

5.3 The timber shall be free from bark.

5.4 95% of the parcel shall have a moisture content not greater than 22%.

5.5 Damages during transport and handling (such as truck-damages and soilage) are permitted only to a small extent.

6. SPECIAL REQUIREMENTS FOR THE DIFFERENT GRADES

The special quality specifications are for grades "Extra" (E) "Standard" (S) and "Other" (C) given in clause 6.1. The special quality specifications are for grade Sx given in clause 6.1, with the supplementary clause 6.2.

6.1 If a defect is present at its permissible maximum, other defects occurring so close that they are likely to act together shall be comparatively smaller.

Table 2 Structural timber

	Extra grade (Grade E)	Standard grade (Grade S)
Knot[1)]	On edge: 1/2 of the thickness On face: 1/4 of the width, though max. 35 mm	On edge: 2/3 of thickness, though max. 70 mm for square On face: 1/3 of width, though max. 70 mm
Knot cluster[1)]	1/4 of width (max.35 mm) + 1/2 of thickness	1/3 of width (max. 70 mm) + 2/3 of thickness (max. 70 mm)
Swirl	Small, close swirls permitted	Close swirls 1/4 of width permitted, except close to edge
Rind galls	Small, shallow rind galls with length up to half the width of the timber is permitted	Length equal to width of timber permitted, but must not penetrate through the timber
Slope of grain	Up to 1:10 permitted	Up to 1:7 permitted
Ring shake	Not permitted	Ought not to appear
Hair surface check	Not permitted	Small checks permitted, also across the arris, but not down to the pith side
Check	A few shorter checks of depth up to 1/4 of the thickness is permitted, except on the edges	Permitted in the full length with depth up to 1/2 of thickness. For square, checks up to 3/4 of the side are permitted. Opposite checks are added together
Width of annual ring	Max. 5 mm	Unlimited
Pitch pockets Pitch wood Weather gray, brown stain etc. Blue stain	Permitted	
Hard decay Compression wood	Permitted to a limited extent	
Rot Defects due to insects	Not permitted	
Spring	Max. 5 mm rise over 2 m length	
Bow	For thickness > 30 mm max. 10 mm rise over 2 m length	
Twist	Max. 1.5 mm per 25 mm width measured over 2 m length	

1) The specifications apply to dimensions of thickness equal to or greater than 36 mm. For smaller thicknesses the specifications for knots are given in the respective product standards.

Table 3 Non-structural timber

	Other grade (Grade C)
Knots [1)]	Large single knots and knot clusters are permitted, but the timber should be hold well together
Swirl	Ulimited, but the timber should hold well together
Rind gall	Unlimited
Slope of grain	Unlimited
Ring shake	Permitted, but in limited length if it is seen on both sides of the pith
Hair surface check	Unlimited, but the timber should hold well together
Check	Unlimited, but must not go through the wood and the timber shall hold well together
Width of annual ring	Unlimited
Pitch pocket Pitch wood Weather gray, brown, stain, etc. Blue stain	Unlimited
Hard decay	Unlimited, but the wood must be firm
Compression	Unlimited
Rot	Permitted, also if going through the wood if hard and firm
Defects due to insects	Permitted to a very limited extent
Spring	Unlimited
Bow	Unlimited
Twist	Unlimited

1) The specifications apply to dimensions of thickness equal to or greater than 36 mm. For smaller thicknesses the specifications for knots are given in the respective product standards.

6.2. If a defect is occurring at its maximum, other defects shall be comparatively smaller. On the other hand a defect, which should have resulted in a lower grade, may be tolerated if the quality of the piece of timber otherwise is clearly better than the maximum requirements. Thus, a single piece of timber may not always fall within the given limits. The quality of the parcel as a whole should nevertheless be clearly better than the requirements.

Table 4 Supplementary requirements for the grade Sx

	Grade Sx
Knots, in general [1]	In addition to the knots specified below an equal number of smaller knots are permitted. as well as some pin knots. The number of knots applies to a length of 1.5 m.
- sound knot	On edge: 3 knots, 2/3 of thickness On face: 3 knots, 1/3 of width
- dry knot	On edge: 3 knots, 1/3 of thickness On face: 3 knots, 1/4 of width, max. 50 mm
- barkringed knot, rotten knot, and knot hole	Permitted if they are small and insignificant
Pitch pocket	Insignificant pitch pockets permitted to a limited extent
Rind gall	Insignificant rind galls permitted
Hard decay	Some streaks and flecks permitted
Blue stain	Some streaks and flecks permitted
Weather gray, brown stain, etc.	Some streaks and flecks removable by rubbing are permitted

1) The specifications apply to dimensions of thickness equal to or greater than 36 mm. For smaller thickness the specifications for knots are given in the respective product standards.

7. INSPECTION

If required control is carried out on 5% of the delivered parcel of timber or on an appointed control parcel. The choice of timber for the control shall be at random.

8. MARKING

Every piece of timber shall have the producer's identification mark. The colour of the mark shall be red for the Extra grade and blue for the Standard grade, or the timber should be marked with the letters E resp. S.

If requested, Grade Sx shall have the additional mark "x".

COMMENTS

Timber satisfying the requirements for Grades E and S can be assumed to have a bending strength of 300 kp/cm^2 (30 N/mm^2) and 200 kp/cm^2 (20 N/mm^2) respectively. Grade Sx is in addition satisfying special requirements for the appearance.

If an approved stress grading machine is used for the quality grading the grading requirements which may have been stated in the approval of the machine, no longer applies. Otherwise the requirements in this standard applies.

BSI

Document 73/10278

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Date March 1973

BRITISH STANDARD SPECIFICATION
FOR TIMBER GRADES FOR STRUCTURAL USE

FOREWORD

This British Standard specifies the means of assessing the quality of timber for which the grade stresses will be given in CP 112. A number of important changes have been introduced within the framework of earlier versions of this British Standard. Full provision is made for both visual stress grading and machine stress grading.

For visual stress grading the principle of knot area ratio has been adopted as a means of determining the maximum permissible knots for a given grade. This principle has been applied in Canada and the United States of America and accepted for use in commercial grading rules throughout North America. Testing on European species has shown that this method of grading gives good correlation with strength properties. It is used in this standard to determine the acceptability of timber whose knot limitations may have been measured by different rules in the countries of origin.

Two standard grades have been established for visually stress graded timber, namely, General Structural grade (GS), and a higher grade, Special Structural grade (SS). Since there can be small differences of opinion between experienced graders a small deviation in grading is permitted.

The standard specifies two machine stress grades, which may be used as substitutes for the two visual grades GS and SS: they are MGS for GS and MSS for SS. Provision will be made in future editions of CP 112 for additional machine stress grades separated by standard increments of modulus of elasticity.

It is a requirement that every piece of timber purporting to be graded in accordance with this Standard shall be appropriately marked.

Under the aegis of the British Standards Institution a system has been established by which trained graders obtain certificates of competence and grading machines are operated under strict rules.

It is expected that some visually graded and marked timber will be available soon after the publication of this standard. However, it may be two years after the publication before sufficient marked timber to meet the full demand is available.

This British Standard applies to timber graded in the United Kingdom. Imported timber which has been visually or machine stress graded overseas, and which meets these rules may be accepted as complying with this standard, subject to any necessary safeguards.

SPECIFICATION

1. SCOPE

This British Standard specifies two methods of grading timber for structural use, namely, visual stress grading and machine stress grading.

The permissible sizes of characteristics in two visual stress grades of timber, which are named 'General Structural Grade' (GS) and 'Special Structural Grade' (SS), are specified. Also specified are visual grades for laminating timber, viz, LA, LB and LC.

The conditions for the approval and control of stress grading machines and the requirements for machine stress graded timber are given.

2. DEFINITIONS

For the purpose of this British Standard the nomenclature in BS 881 and 589 and the definitions of BS 565, BS 4471 and CP 112 apply, together with the following:

2.1 Approving authority The body responsible for the assessment, periodic testing and certification of a stress grading machine.

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

2.3 Indicating property The property of the timber used by a stress grading machine to determine the grade stress.

2.4 Machine stress graded timber A piece of timber to which a grade stress has been assigned by measuring one, or more, indicating properties of the timber by means of a test in a non-destructive testing machine.

2.5 Margin areas The areas adjoining the edges of the cross-sections, each of which occupies 1/4 of the total cross-sectional area (See Fig. 1).

2.6 Margin condition A margin condition exists when more than 1/2 the area of either margin is occupied by the projected area of knots.

2.7 Knot area ratio (KAR) The ratio of the sum of projected cross-sectional areas of all knots at a cross-section to the cross-sectional area of the piece.

3. STRESS GRADED TIMBER

3.1 Sizes, Permissible deviations and Processing reductions

3.1.1 Sawn softwood Tinber graded to this standard shall also comply with BS 4471 with respect to the basic sizes, permissible deviations and processing reductions applicable to constructional timber.

Section moduli and other geometric properties are given in CP 112

NOTE: Stress graded softwood of the following basic thicknesses is readily available: 38, 44, 50, 63, 75 mm and also 100 mm in limited species.

3.1.2 Sawn hardwood The main structural species may be assumed to be available in the same sizes and lengths as sawn softwood until such time as a British Standard for sizes of sawn hardwood is published.

3.2 Processed Timber If the grading has been carried out before processing, provided the reduction in size does not exceed that for constructional timber in (1) of Table 3: BS 4471, Part 1, the grade is not considered to have been changed.

3.3 Resawing or Surfacing Where graded timber is resawn or surfaced to an extent beyond the limits of Clause 3.2.the timber shall be regraded if it is to comply with this standard.

3.4 Deviation in grading A parcel of visually graded timber shall be deemed to satisfy the grade specified, provided that on inspection of the actual sizes of the characteristics not more than 5% of the parcel is found to be deficient. Of this 5%, any piece having any one characteristic worse by more than 10% shall be rejected.

3.5 Timber graded abroad Timber graded and marked abroad to meet the grades of this British Standard need not be regraded in the United Kingdom, provided that the timber is not resawn into smaller sizes.

4. MEASUREMENT OF CHARACTERISTICS FOR VISUAL STRESS GRADING

4.1 Knots Knots shall be assessed by their knot area ratio, which shall be taken as the ratio of the sum of their projected cross-sectional areas to the cross-sectional area of the piece. In making this assessment knots of less than 5 mm diameter may be ignored, and no distinction shall be made between knot holes, dead knots or live knots.

The method of assessing the knot area ratio is illustrated in Fig. 2. Typical knot area ratios are shown in Fig. 3. In cases of dispute the method given in Appendix A may be used to determine the knot area ratio.

4.2 Fissures The size of a fissure shall be taken as the distance between lines enclosing the fissure and parallel to a pair of opposite faces (See Fig. 4a).

If a transverse longitudinal plane cuts through two or more fissures on opposite faces, then the sum of their size shall be taken as the size of the defect (See Fig. 4b).

When a fissure occurs on the surface of a piece, its size may be verified by means of a feeler guage, not exceeding 0.2 mm thick.

4.3 Slope of grain The slope of grain shall be measured over a distance sufficiently great to determine the general slope, disregarding slight local deviations.

The methods by which slope of grain shall be measured are:

a. by taking a line parallel to the surface fissures, or

b. by the use of a grain detector (See Appendix B).

4.4 Spiral grain Where spiral grain occurs the slope of grain shall be determined by measuring the worst slope of grain on the faces and on the edges and taking the square root of the sum of the squares of the slopes. For example, if these slopes are 1 in 18 and 1 in 12, the combined slope is:

$$\sqrt{(1/18)^2 + (1/12)^2} = 1/10 \text{ or a slope of 1 in 10}$$

NOTE: This procedure is intended to apply in instances where it is deemed necessary to measure the slope of grain; in most cases, it will be sufficeient for the grader to use his judgement in rejecting timber containing spiral grain which occurs to an undesirable extent.

4.5 Wane The amount of wane on any surface shall be taken as the sum of the wane at the two arrises, and shall be expressed as a fraction of the dimension of the surface on which it occurs (See Fig 5).

4.6 Rate of growth Rate of growth shall be measured on one end of the piece, and shall be taken as the average number of growth rings per 25 mm intersected by a straight line 75 mm long normal to the growth rings, passing through the centre of the end of the piece (See Fig 6a); or commencing 25 mm from the pith when this is present (See Fig 6b). When a line 75 mm in length is unobtainable the measurements shall be made on the longest possible line normal to the growth rings and passing through the centre of the piece.

4.7 Resin pockets Resin pockets shall be measured in the same way as fissures.

4.8 Distortion The bow, spring and twist in a piece shall be measured over a 3 m length (See Fig 7). The cup shall be measured over the width of the piece.

5. GENERAL STRUCTURAL (GS) GRADE

5.1 General A piece will satisfy the requirements of GS grade if the characteristics do not exceed the limits given in this clause.

5.2 Knots

5.2.1 Knot area ratio When a margin condition exists, the knot area ratio shall not exceed 1/3. When a margin condition does not exist, the knot area ratio shall not exceed 1/2.

When the cross section is square, the knot area ratio shall not exceed 1/3.

An example is shown in Fig 8.

A decision sequence that may be followed when selecting for GS grade is given in Appendix C.

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

5.3 Fissures

5.3.1 If the size of the defect is less than or equal to half the thickness of the piece, then the fissures may be unlimited in number wherever they occur in the piece.

5.3.2 If the size of the defect is greater than half the thickness of the piece, but less than the thickness of the piece, then the length of the fissures shall not exceed 900 mm or 1/4 the length of the piece, whichever is the lesser.

5.3.3 If the size of the defect is equal to the thickness of piece, then the length of the fissures shall not exceed 600 mm: if the fissures occur at the end of the piece their length shall not exceed 1 1/2 times the width of the piece.

5.4 Slope of grain Slope of grain shall not exceed 1 in 6.

5.5 Wane The amount of wane shall not exceed 1/3 of the dimension of the surface on which it occurs, except that not nearer either end of the piece than 300 mm the amount of wane may be up to 1/2 of the dimension of the surface on which it occurs within a single continuous length not exceeding 300 mm.

5.6 Rate of growth The rate of growth shall not be less than 4 annual rings per 25 mm.

5.6 Resin pockets The limits given for fissures in clause 5.3 shall apply to resin pockets.

5.8 Distortion Any piece which is bowed, sprung, twisted or cupped to an excessive extent, having regard to the end-use, shall be rejected.

NOTE: Distortion will largely depend on the moisture content of the timber at the time it is measured. A precise definition to cover all applications and conditions cannot be given, but for guidance the following limits of bow, spring, twist and cup may be applied to parcels of graded timber. Bow should not exceed one-half of the thickness in any 3 m length. Spring should not exceed 15 mm in any 3 m length. Twist should not exceed 1 mm per 25 mm of width in any 3 m length. Cup should not exceed 1/25 of the width. In some applications, depending of the type of distortion and section size, greater amounts may be accommodated without impairing performance, but in other applications lesser amounts may impair performance.

5.9 Wormholes Pinholes and wormholes are permitted to a slight extent in a small number of pieces, provided that there is no active infestation of the material. Wood wasp holes are not permitted.

5.10 Sapstain Blue stain in sapwood is not a structural defect and may be permitted to a limited extent.

5.11 Abnormal defects All pieces showing fungal decay, brittleheart and other abnormal defects affecting strength shall be excluded.

Pieces may be accepted however where the reduction in strength caused by the abnormal defect is obviously less than that caused by the defects admitted by the grade of timber, subject to the provision that these abnormal defects are of a type which will not progress after conversion and seasoning, (eg white pocket rot derived from the standing tree.)

6. SPECIAL STRUCTURAL (SS) GRADE

6.1 General A piece will satisfy the requirements of SS grade if the characteristics do not exceed the limits given in this clause.

6.2 Knots

6.2.1 Knot area ratio When a margin condition exists, the knot area ratio shall not exceed 1/5. When a margin condition does not exist, the knot area ratio shall not exceed 1/3.

When the cross section is square, the knot area ratio shall not exceed 1/5.

An example is shown in Fig 8.

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

6.3 Fissures

6.3.1 If the size of the defect is less than or equal to half the thickness of the piece, then the fissures may be unlimited in number wherever they occur in the piece.

6.3.2 If the size of the defect is greater than half the thickness of the piece, but less than the thickness of the piece, then the length of the fissures shall not exceed 600 mm or 1/4 the length of the piece, whichever is the lesser.

6.3.3 If the size of the defect is equal to the thickness of the piece, then the fissures shall be permitted only if they occur at the ends of the piece and their length shall not exceed the width of the piece.

6.4 Slope of grain Slope of grain shall not exceed 1 in 10.

6.5 Wane The amount of wane shall not exceed 1/4 of the dimension of the surface on which it occurs.

6.6 Rate of growth The rate of growth shall not be less than 4 annual rings per 25 mm.

6.7 Resin pockets The limits given for fissures in Clause 6.3 shall apply to resin pockets.

6.8 Distortion Any piece which is bowed, sprung, twisted or cupped to an excessive extent, having regard to the end-use, shall be rejected.

> NOTE: Distortion will largely depend on the moisture content of the timber at the time it is measured. A precise definition to cover all applications and conditions cannot be given, but for guidance the following limits of bow, spring, twist and cup may be applied to parcels of graded timber. Bow should not exceed one-half of the thickness in any 3 m length. Spring should not exceed 15 mm in any 3 m length. Twist should not exceed 1 mm per 25 mm of width in any 3 m length. Cup should not exceed 1/25 of the width. In some applications, depending on the type of distortion and section size, greater amounts may be accommodated without impairing performance, but in other applications lesser amounts may impair performance.

6.9 Wormholes Pinholes and wormholes are permitted to a slight extent in a small number of pieces provided that there is no active infestation of the material. Wood wasp holes are not permitted.

6.10 Sapstain Blue stain in sapwood is not a structural defect and may be permitted to a limited extent.

6.11 Abnormal defects All pieces showing fungal decay, brittleheart and other abnormal defects affecting strength shall be excluded.

Pieces may be accepted however where the reduction in strength caused by the abnormal defect is obviously less than that caused by the defects admitted by the grade of timber, subject to the provision that these abnormal defects are of a type which will not progress after conversion and seasoning, (eg white pocket rot derived from the standing tree.)

7. TIMBER FOR LAMINATING

7.1 General Three grades designated LA, LB and LC are specified for laminated construction. To qualify for a particular grade, individual laminations, after final processing, shall comply with the requirements of the following clauses:

7.2 Knot area ratios The sum of the knot area ratios determined by including all knots within any 300 mm of length, neglecting overlaps, shall not exceed:

1/10 for grade LA
1/4 for grade LB
1/2 for grade LC.

7.3 Fissures Fissures on the face of a lamination may be unlimited in depth, but shall not form an angle with the face of less than 45°.

7.4 Slope of grain Slope of grain measured in accordance with Clause 4.3 shall not exceed:

1 in 18 for grade LA
1 in 14 for grade LB
1 in 8 for grade LC.

7.5 Wane Wane shall not be permitted.

7.6 Rate of growth No limit is placed on the rate of growth.

7.7 Resin Pieces which are exceptionally resinous, or contain large resin pockets which would affect gluing, shall not be permitted.

7.8 Distortion Any piece which is bowed, sprung, twisted or cupped to an excessive extent, having regard to the method of laminating, shall be rejected.

7.9 Wormholes Pinholes and wormholes are permitted to a slight extent in a small number of pieces provided that there is no active infestation of the material. Wood wasp holes are not permitted.

7.10 Sapstain Blue stain in sapwood is not a structural defect and may be permitted to a limited extent.

7.11 Abnormal defects All pieces showing fungal decay, brittleheart and other abnormal defects affecting strength shall be excluded.

8. SUPERVISION SCHEME FOR STRESS GRADING MACHINES

8.1 Approving authority The approving authority shall be the British Standards Institution or their delegated agents.

8.2 Scheme of supervision The approving authority shall make available, on request, the criteria by which approval may be granted, and details of the Scheme of Supervision.

8.3 Conditions for machine approval A machine shall be approved by the approving authority if all the following conditions are fulfilled:

8.3.1 The type of machine has been tested and approved by the approving authority.

8.3.2 The Company and its records, and the actual machine, are subject to unannounced periodic inspection by the approving authority in a manner and at a frequency which will be laid down in the Scheme of Supervision for stress grading machines set down by that authority.

8.3.3 The Company operating the machine at the time of grading has complied with all instructions under the Scheme of Supervision, laid down by the approving authority.

8.3.4 The machine is not at the time of grading under temporary suspension by the approving authority.

9 MACHINE GENERAL STRUCTURAL (MGS) AND MACHINE SPECIAL STRUCTURAL (MSS) GRADES

9.1 General Two grades of machine stress graded timber are specified, namely MGS and MSS. Additional machine stress grades separated by standard increments of Modulus of Elasticity will be specified in future editions of CP 112 with their grade stresses.

The approving authority shall be empowered to authorise additional intermediate grades to suit specific end uses.

9.2 Machine requirements for MGS and MSS grades A piece will satisfy the machine requirements for MGS and MSS grade if it has been passed through an approved stress grading machine and the whole of the piece has been classified as complying with the grade.

9.3 Visual requirements for MGS and MSS grades Notwithstanding the fact that the piece complies with the indicating property requirements of the assigned machine grade stress it shall only be deemed to comply with the requirements of that grade if on visual inspection the piece complies with the requirements for fissures, wane, resin pockets, distortion, worm holes, sapstain and abnormal defects of the equivalent grade.

MGS shall comply with Clauses 5.3, 5.5, 5.7, 5.8, 5.9, 5.10 and 5.11. MSS and higher machine grades shall comply with Clauses 6.3, 6.5, 6.7, 6.8, 6.9, 6.10 and 6.11.

9.4 Substitution MGS grade may be deemed to comply with the requirements for GS in Clause 5 and hence may be used as a substitute for GS grade of the same size. MSS grade may be deemed to comply with the requirements for SS grade in Clause 6 and hence may be used as a substitute.

Grades GS and SS shall not be used as substitutes for grades MGS and MSS.

10 MARKING OF VISUALLY STRESS GRADED TIMBER

Every piece of visually stress graded timber shall have the following information clearly and indelibly marked on at least one face, edge or end *.

10.1 A mark identifying the grader or the company responsible for the grading.

10.2 The grade of the piece.

*NOTE: It should be noted that face marking is preferable and should be adopted eventually.

11 MARKING OF MACHINE STRESS GRADED TIMBER

Every piece of machine stress graded timber shall have the following information clearly and indelibly marked on at least one face, edge or end *.

11.1 The license number of the stress grading machine.

11.2 The grade assigned to the piece.

11.3 The BSI Kitemark and the number of this British Standard, or alternatively to these, any mark approved by the approving authority.

*NOTE: It should be noted that face marking is preferable and should be adopted eventually.

12 RE-MARKING STRESS GRADED TIMBER

Where the grade marking is removed by planing, regularizing within the limits of Clause 3.2, or crosscutting, the processor shall re-mark the timber with its original grade and his own mark of identification, prefixed by a letter R to denote Re-marked.

APPENDIX A

METHOD FOR DETERMINING THE KNOT AREA RATIO IN CASES OF DISPUTE

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

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

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

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

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

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

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

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

4. In both the above cases express:

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

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

APPENDIX B

DETERMINATION OF SLOPE OF GRAIN

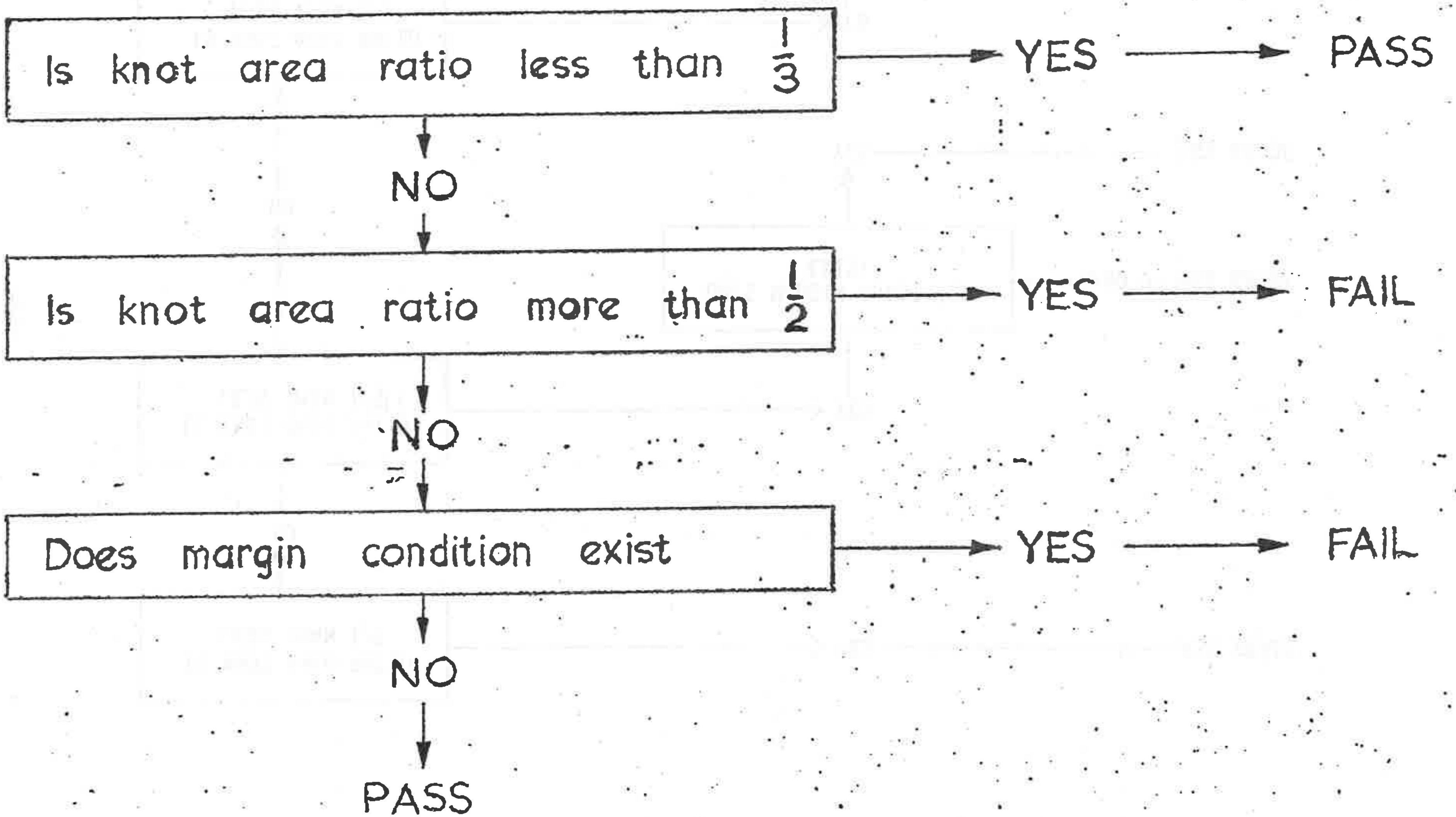
Slope of grain can be determined by means of a scribe, shown in Fig. 9, comprising a cranked rod with a swivel handle and a needle, at the tip, set to a slight trailing angle. The needle is pressed into the wood and the scribe is drawn along with a steady action in the apparent direction of the grain, which is indicated more precisely as the needle forms a groovs. If the pressure on the needle is not sufficient it may be dragged across the grain; on the other hand, a steady action is impossible if the pressure is excessive and the needle penetrates too far into the wood. In Douglas fir the summerwood is relatively dense and the needle meeting it tends to be diverted, resulting in a step in the groove, the avoidance of which requires a particularly slow and steady action as each springwood stripe is met.

If the action is correct the needle follows the grain even when the direction of pull of the scribe is slightly out of line. This may be used to check that the scribe does follow the grain by scribing another groove in close proximity on each side of the original one with the direction of pull diverging slightly outwards in each case when, if the grooves are following the grain, they will be parallel to each other.

The inclination of grain on a face is measured as shown in Figures 10 and 11 in which AB is the line indicating grain direction, AC is a line drawn parallel to the edge of the member, BC is of length one unit (any convenient unit may be used) and it is at right angles to AC. Grain inclination is expressed as 'one in x', where x is the length of AC measured in terms of BC.

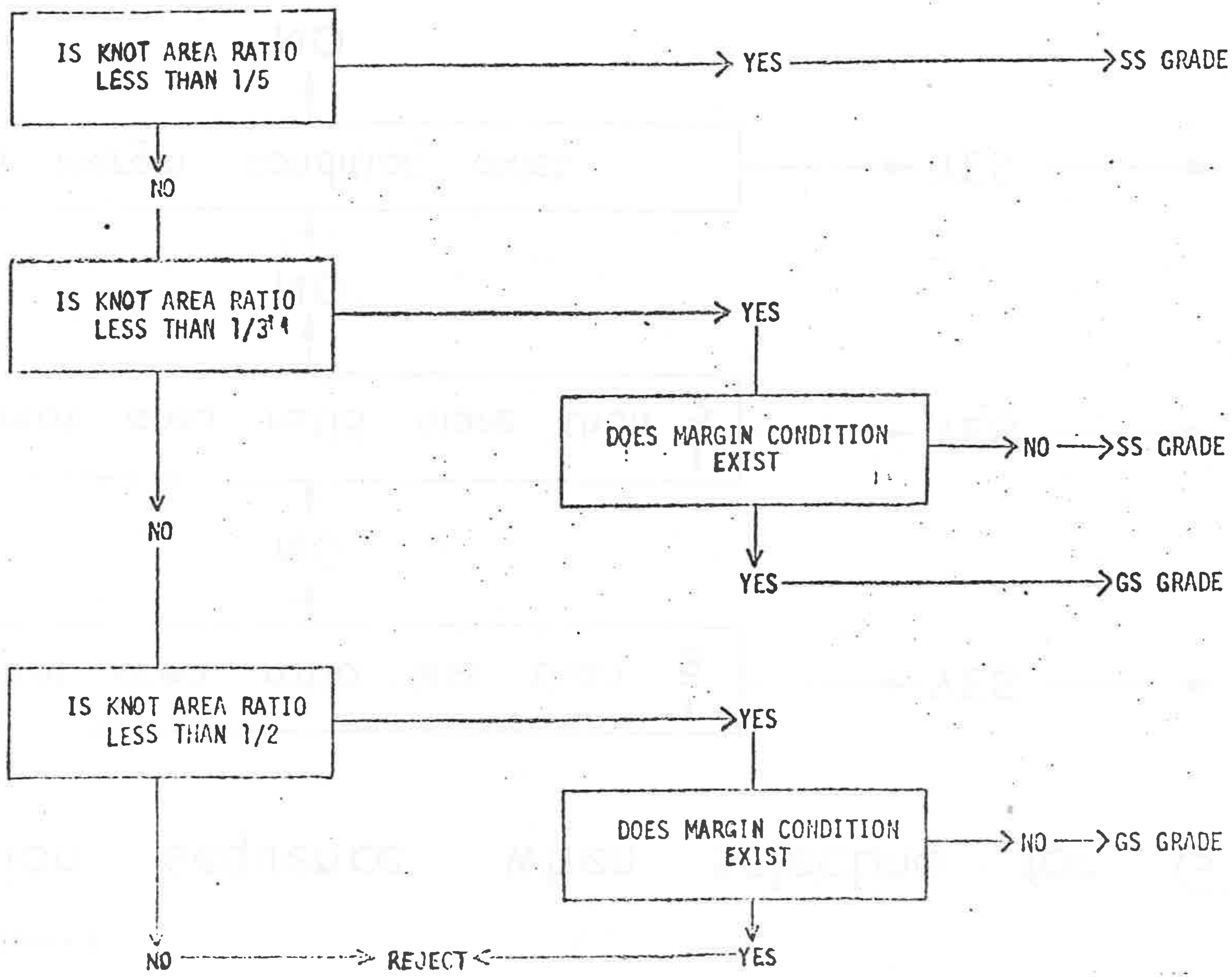
The slope of grain should be determined on adjacent faces and edges, and from the values obtained the true inclination of the grain to the longitudinal axis of the member should be determined in accordance with Clause 4.4.

Decision sequence when selecting for G S grade



APPENDIX D

DECISION SEQUENCE WHEN SELECTING FOR SS AND GS GRADES



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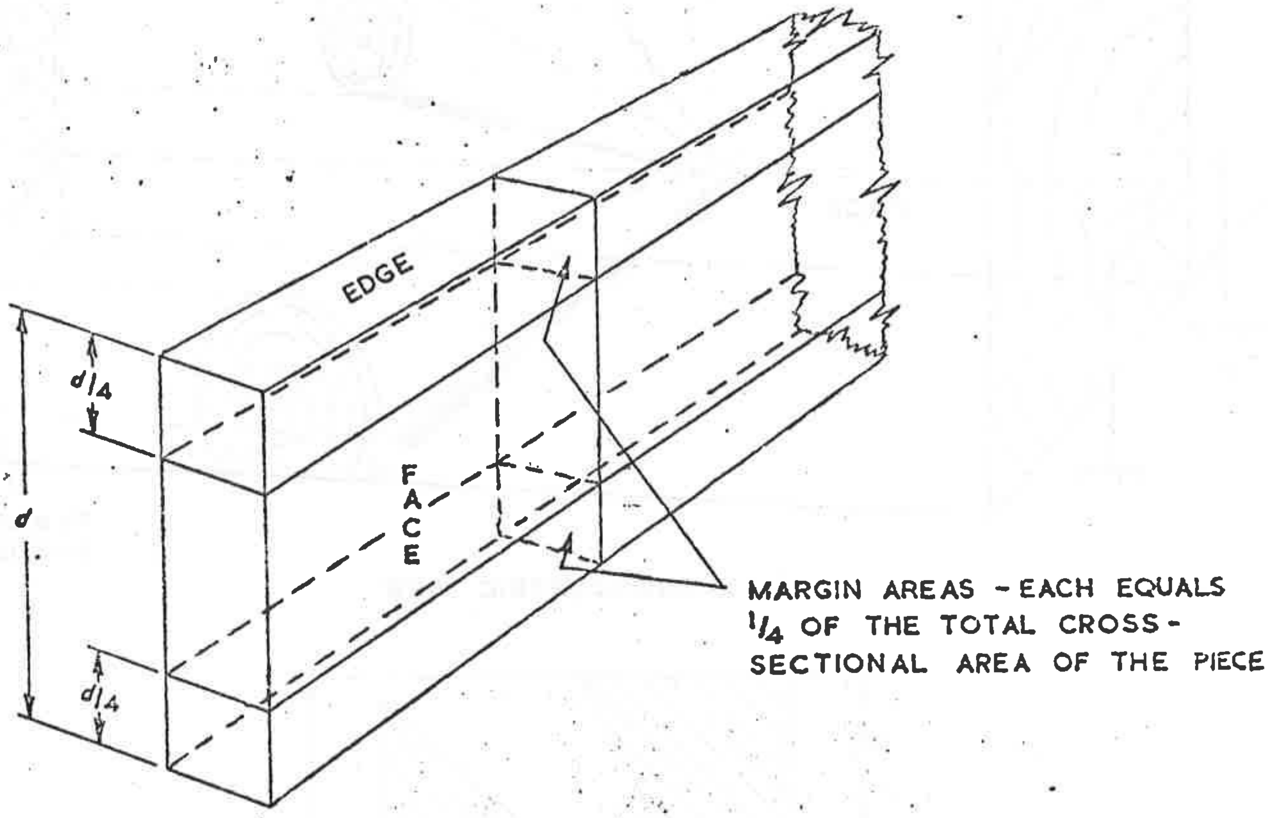

FIG. 1. EDGE, FACE AND MARGIN AREAS

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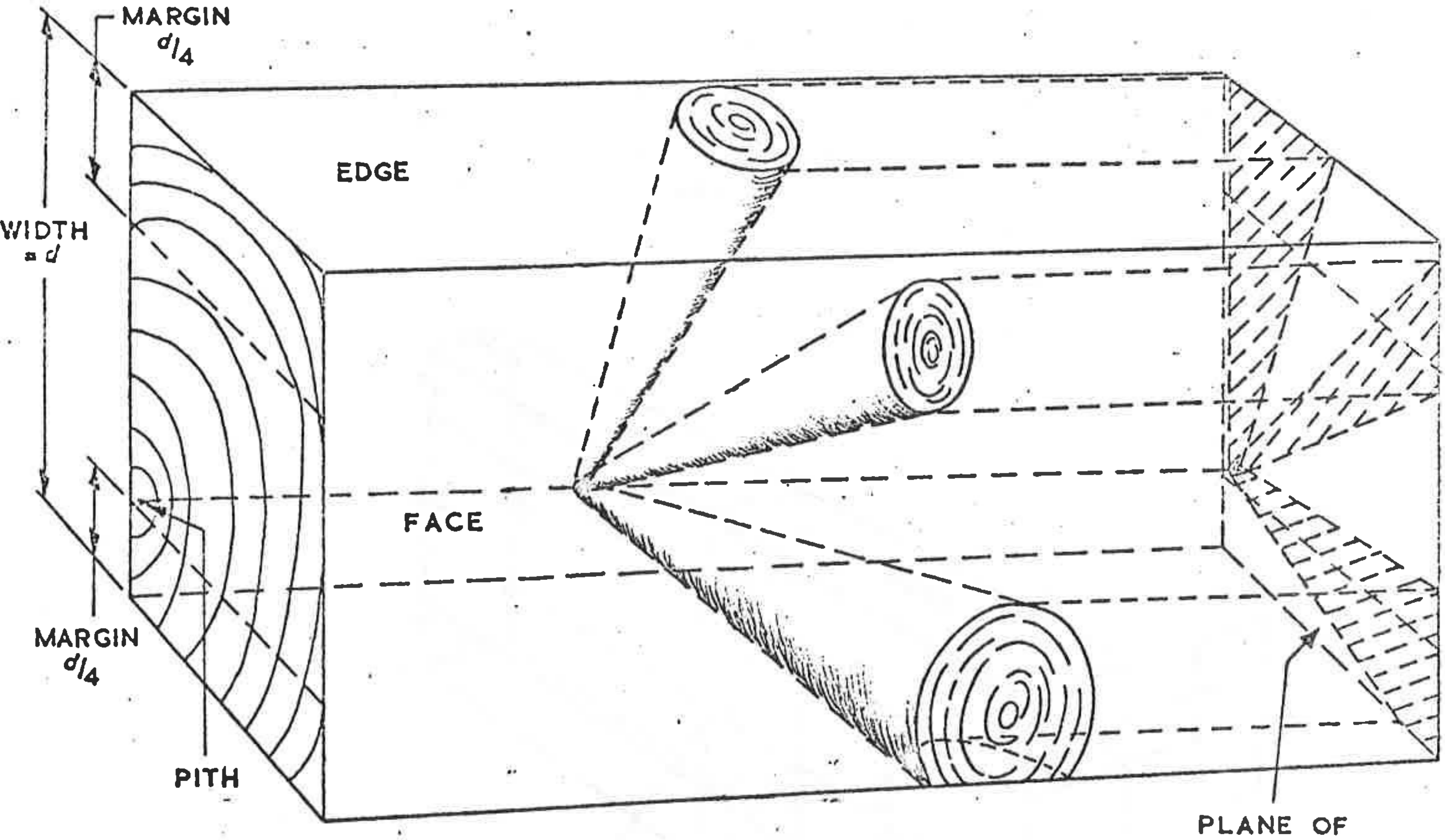
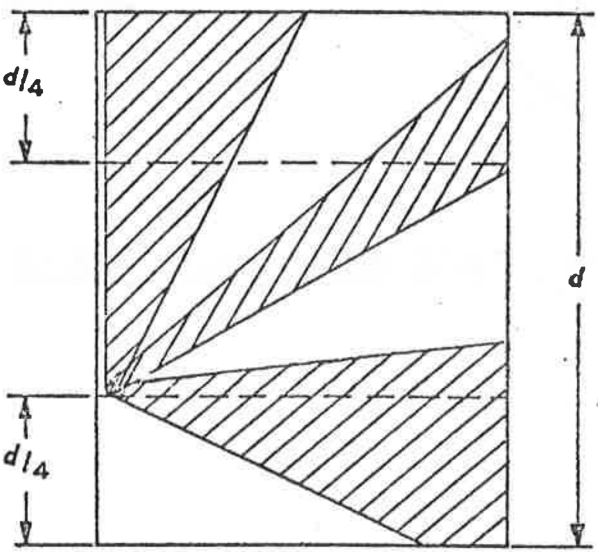


(a) AXONONOMETRIC VIEW



(b) FRONT VIEW OF PROJECTION PLANE, SHOWING PROJECTION OF KNOTS (HATCHED).

FIG. 2. DRAWING ILLUSTRATING PRINCIPLE OF KNOT PROJECTION, (a) SHOWS IN 3-D A GROUP OF KNOTS IN A PIECE (SHOWN IN FULL) AND THEIR PROJECTION ON A CROSS SECTIONAL PLANE, SHOWN IN FRONTAL VIEW ON (b)

73/10278

DRG. Nº 1404 1/2

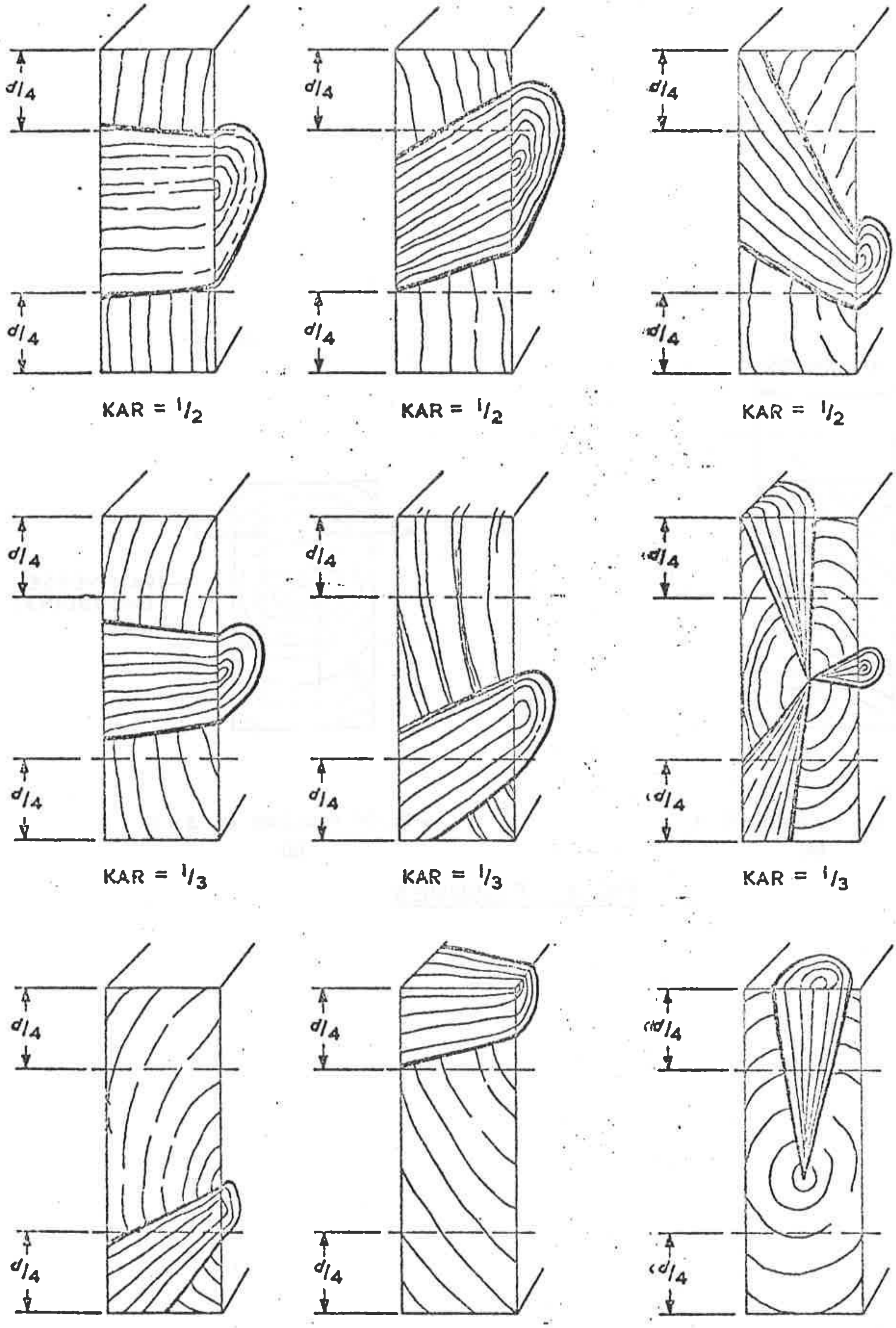


FIG. 3. TYPICAL KNOT AREA RATIOS

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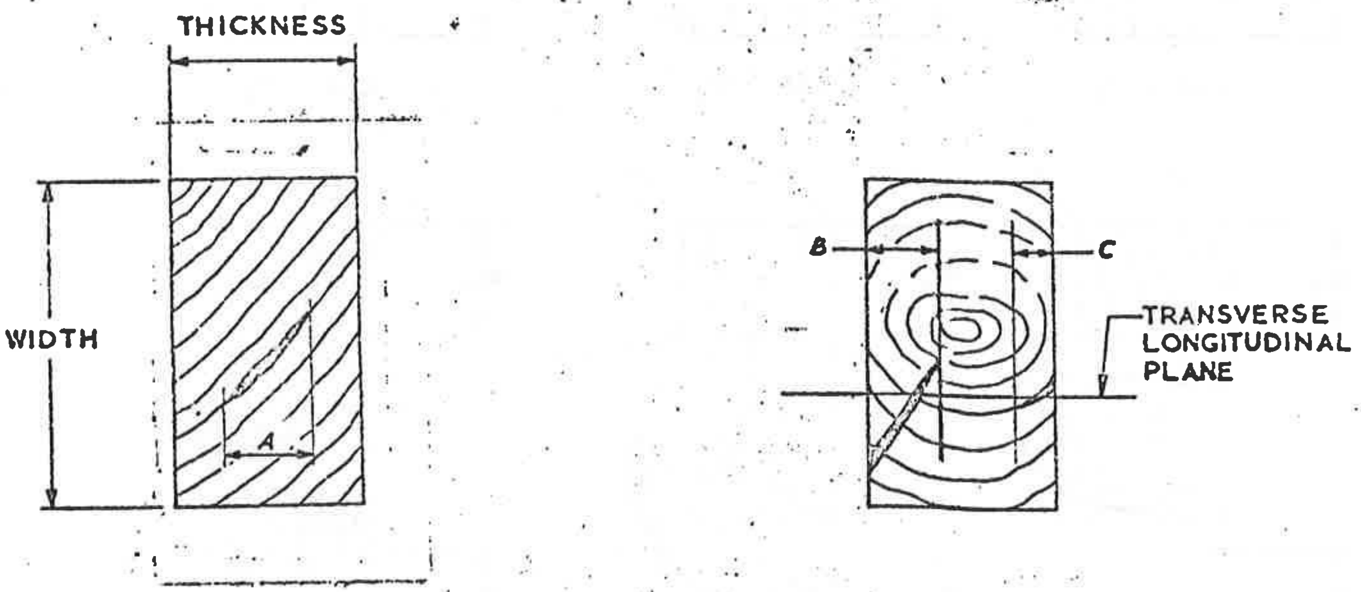

SIZE OF FISSURE IS A
(a)

SIZE OF FISSURE IS B + C
(b)

FIG. 4. FISSURES

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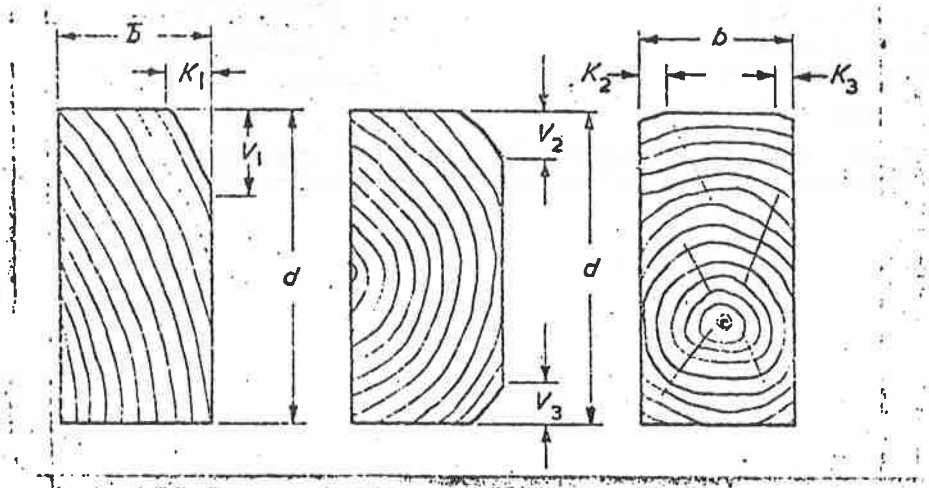


AMOUNT OF WANE ON THE FACE OF THE PIECE SHALL BE EXPRESSED AS THE RATIO

$$\frac{V_1}{d} \text{ OR } \frac{V_2 + V_3}{d}$$

AMOUNT OF WANE ON THE EDGE OF THE PIECE SHALL BE EXPRESSED AS THE RATIO

$$\frac{K_1}{b} \text{ OR } \frac{K_2 + K_3}{b}$$

FIG. 5. AMOUNTS OF WANE

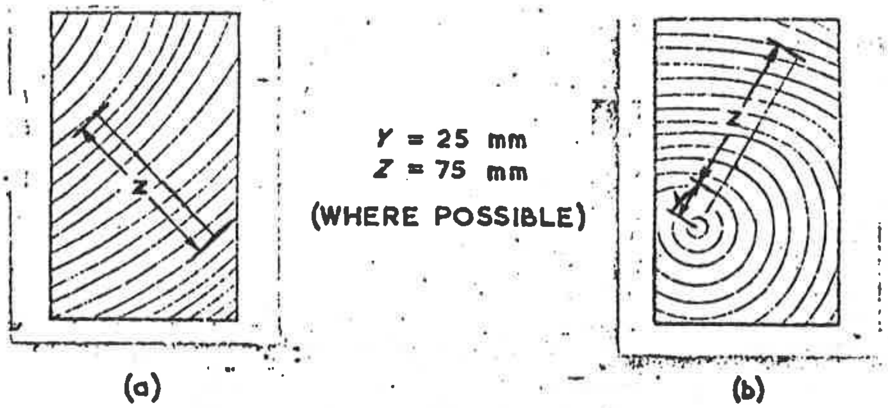


FIG. 6. MEASUREMENT OF RATE OF GROWTH

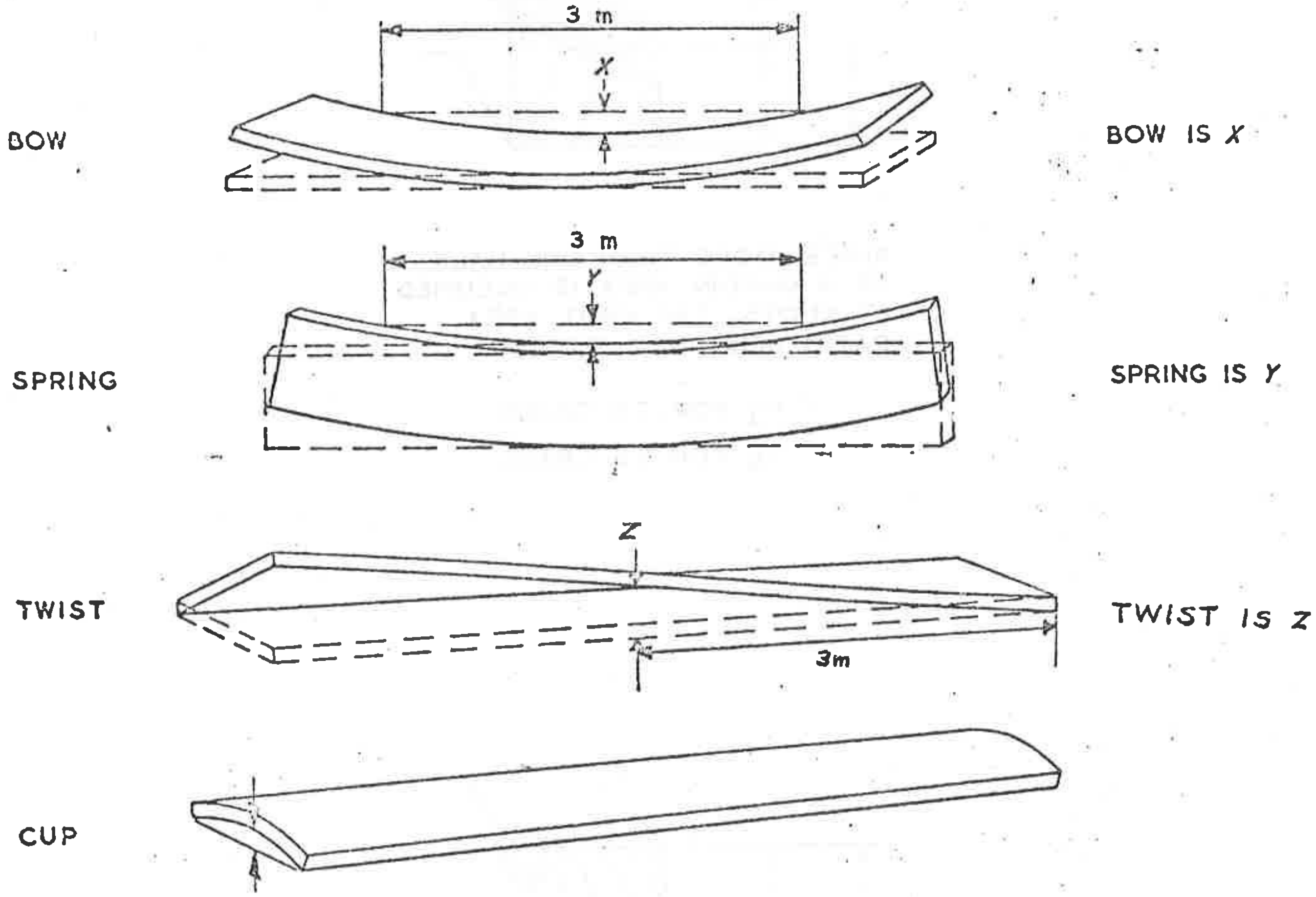
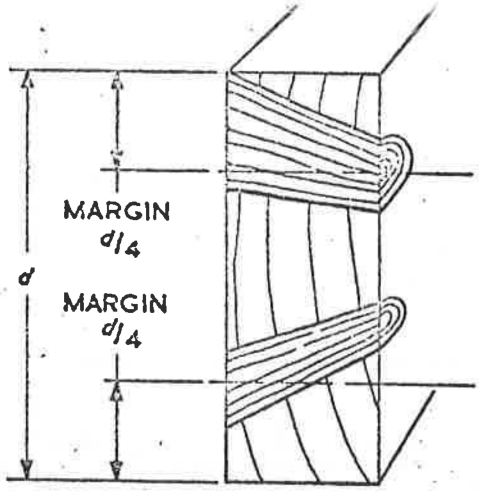


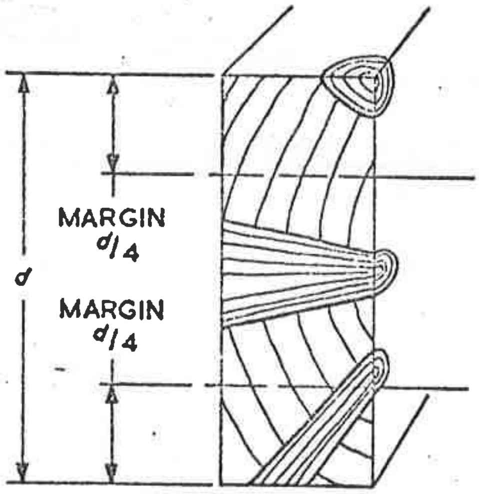
FIG. 7. MEASUREMENT OF BOW, SPRING, TWIST AND CUP



SINCE MORE THAN ONE HALF OF A MARGIN AREA IS OCCUPIED BY KNOTS, THE KNOT AREA RATIO MUST NOT EXCEED:

1/3 FOR GS GRADE

1/5 FOR SS GRADE



SINCE LESS THAN ONE HALF OF A MARGIN AREA IS OCCUPIED BY KNOTS, THE KNOT AREA RATIO MUST NOT EXCEED:

1/2 FOR GS GRADE

1/3 FOR SS GRADE

FIG.8. EXAMPLES OF KNOT AREA RATIOS FOR GS AND SS GRADES

DRG. Nº 14 041/8

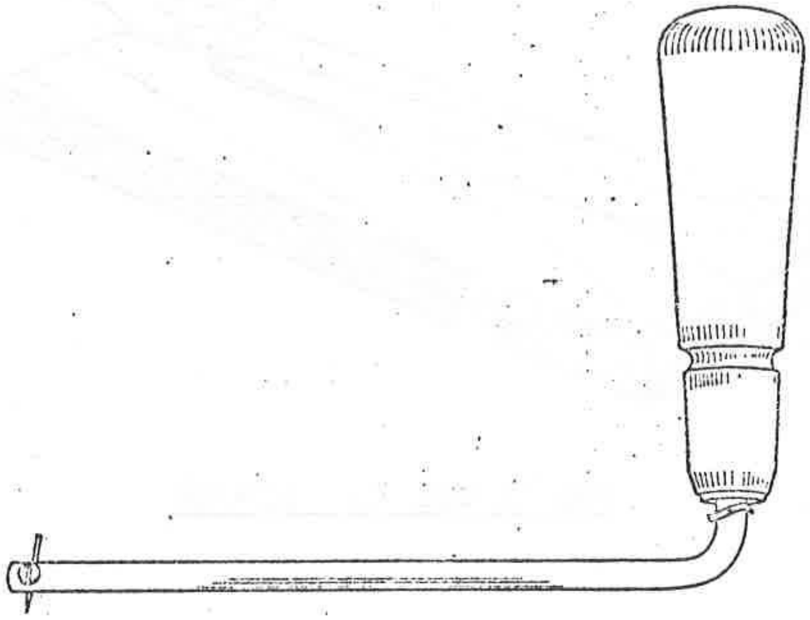


FIG. 9. SWIVEL HANDLED SCRIBE FOR DETERMINATION OF SLOPE OF GRAIN IN WOOD

DRG. Nº 14 041/9
73/10278

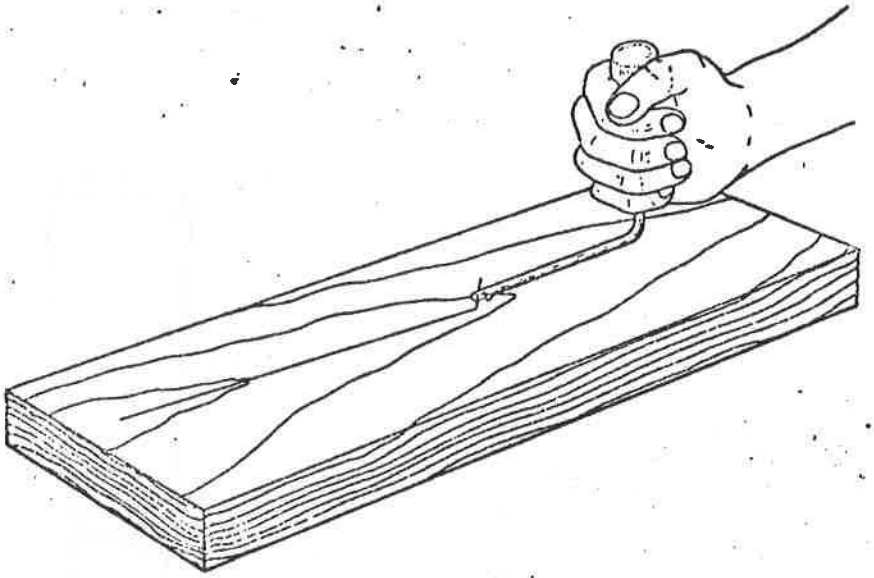


FIG. 10. USE OF SCRIBE

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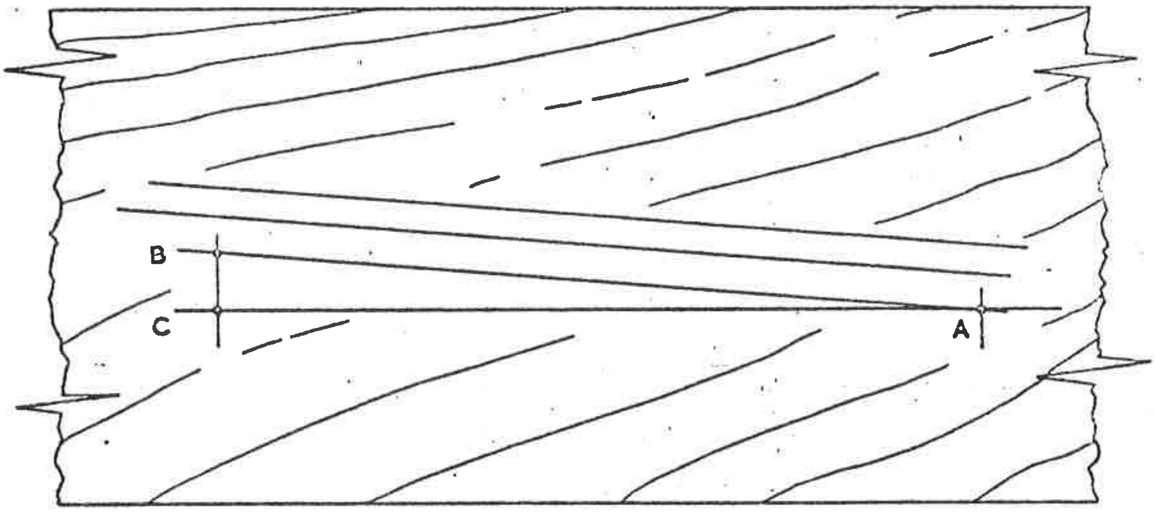
73/10278



SLOPE OF GRAIN = 1 IN $\frac{AC}{BC}$

FIG. 11. MEASUREMENT OF SLOPE OF GRAIN

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CO-OPERATING ORGANISATIONS

The Timber Industry Standards Committee, under whose supervision this British Standard was prepared, consists of representatives from the following Government departments and scientific and industrial organisations:

British Plastics Federation

* British Woodwork Manufacturers' Association

* Department of the Environment - Building Research Establishment

* Department of the Environment

Fibre Building Board Development Organization Ltd

* Forestry Commission

Hardwood Flooring Manufacturers' Association

Institute of Wood Science

Institution of Civil Engineers

Institution of Municipal Engineers

* Institution of Structural Engineers

* National Federation of Building Trades Employers

* National Sawmilling Association

* Royal Institute of British Architects

Royal Institution of Chartered Surveyors

* Softwood Agents and Brokers Association

Timber Packaging and Pallet Confederation

* Timber Research and Development Association

* Timber Trade Federation of the United Kingdom

The Government departments and scientific and industrial organisations marked with an asterisk in the above list together with the following were directly represented on the committee entrusted with the preparation of this British Standard.

Council of the Forest Industries of British Columbia

Greater London Council

Home Timber Merchants Association of England and Wales

Home Timber Merchants Association of Scotland

Institute of Building

National House Builders Registration Council

Scottish Timber Merchants and Sawmillers Association

Timber Drying Association

Individual experts

MECHANICAL FASTENERS AND FASTENING IN TIMBER STRUCTURES

A Preliminary Report

VIRGINIA POLYTECHNIC INSTITUTE AND STATE UNIVERSITY
WOOD RESEARCH & WOOD CONSTRUCTION LABORATORY

BY E. GEORGE STERN, RESEARCH PROFESSOR

BLACKSBURG, VIRGINIA MARCH, 1973

MECHANICAL FASTENERS AND FASTENING IN TIMBER STRUCTURES*

By E. George Stern, Earle B. Norris Research Professor of Wood Construction
Virginia Polytechnic Institute and State University, Blacksburg, Virginia

A worldwide survey of the status of mechanical fasteners and fastening in timber structures can bring forth some of the idiosyncrasies of timber construction in the various countries surveyed. The reasons for differences in the characteristic peculiarities of such timber fasteners as are locally available can throw light on the natural as well as man-made limitations with which the builder is confronted.

The technically more advanced countries should, of course, have many additional and more sophisticated fasteners on the market than, for instance, the Third-World countries. On the other hand, the advance in many technical fields depends often not so much on the technology available as on the ingenuity of the individuals active in a particular field to overcome certain difficulties and handicaps.

Before machine-made metal fasteners became available, simple wooden pins, such as those shown in Fig. 1, played a major role in the erection of ingenious timber structures. When machine-made nails came into production, the Scandinavians ingeniously developed two distinct types of nails, as are shown in Fig. 2, that is, the square-shank nails with flat sides as used in southern Norway and Finland and those with longitudinal flutes as used in northern Norway and Sweden (1). Did the possibly slower-grown timbers from the north call for more slender fasteners which would provide greater holding power, offer less driving resistance, and exhibit a lesser tendency to split these timbers during driving? Long after round-wire nails were taken for granted in the developed countries, an ingenious Austrian carpenter developed a sheet-metal nail (2), as shown in Fig. 3, which proved to be patentable and highly promising, yet could not be mass-produced

(1) Stern, E. George. November, 1961. Wood Construction in Europe as Observed during 1961. Virginia Polytechnic Institute. Engineering Experiment Station. Bulletin No. 147.

(2) Stern, E. George, and Arthur S. Tisch. February and March, 1969. The Improved Nail --- Its Status. Wire & Wire Products, Vol. 44, No. 2, pp. 54-56 and 105-108; and No. 3, pp. 50-51 and 100-102.

* This paper was prepared for presentation at the March 20-21, 1973, Meeting of Commission W-18 on "Timber Structures" of International Council for Building Research Studies & Documentation (CIB) at Princes Risborough Laboratory of Building Research Establishment, Princes Risborough, Aylesbury, Buckinghamshire, England

The use of brand or trade names in this paper serves to restrict the findings to the particular product under scrutiny and does not constitute an endorsement of this product.

because of unsurmountable license problems. Today, we know many types of ingenious metal nail plates (truss plates), with protruding barbs, prongs, and teeth to penetrate the wood, as are shown in Fig. 4. In addition, we learn of the equally ingenious Menig plates, Steelam plates, Micro-lam connectors, Griplam rivets, pipe and tube connectors, slender threaded spikes, and plastic-polymer coated nails and staples which were recently developed (3).

Ingenuity in the field of wood fasteners is going rampant. Our time is exuberant in developing new, improved, and highly practical, needed fasteners for the improved assembly of wood. Yet, on the other end of the scale, there still are countries where not a single metal nail or other metal fastener is manufactured and where all those fasteners which are needed have to be imported. There is an almost unlimited opportunity in this fastening field everywhere in the world and, especially, in the developing countries. It is the purpose of this survey to provide a picture, as limited as it is, of the status of mechanical fasteners and fastening in timber structures around the globe. As more information will reach the author, this report can be made more comprehensive and all-inclusive to the benefit of those countries which are not as advanced in the fastening field as, for instance, the U.S.A.

CANADA

The Canadian Standards Association's Standard B111-1967 for "Wire Nails, Spikes and Staples" covers recommended sizes and other dimensional data for various kinds of wire nails, spikes, and staples commonly used in the building and packaging trades. It also includes general statements on material finishes and coatings. The 40 tables cover 40 types of 292 standard and 85 special round-wire plain-shank common nails, common spikes, barrel or broom nails, box nails, clinch nails, berry-box or basket nails, moulding and finishing nails; plain-shank and longitudinally fluted flooring and casing nails; plain-shank wood lath and fine nails, gypsum-lath nails, gypsum-wallboard nails, large-head roofing or gypsum-sheathing nails, shingle nails, sheet-metal nails, eavestrough spikes, hinge nails, siding or clapboard nails; plain-shank or barbed car nails; plain-shank sash pins, fence staples, poultry-netting staples; helically fluted common nails, box nails, finishing nails, metal-roofing and slating nails, flooring nails, gypsum-lath nails, gypsum-wallboard nails, eavestrough spikes,

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(3) Forest Products Research Society. June 27, 1973. Panel Presentation on Experiences with Mechanical Fasteners. 27th Annual Meeting of Forest Products Research Society. Session sponsored by Technical Committee on Mechanical Fastening. Anaheim, California.

concrete or masonry nails; annularly threaded steel-channel nails, underlay-flooring nails, suspended-ceiling nails, plywood-nails, gypsum-wallboard nails, plywood-subfloor or underlay nails; helically threaded hardwood-flooring nails; helically or annularly threaded common nails, roofing nails, and pallet nails.¶ The steel wire used in their manufacture shall be sufficiently ductile for the finished plain-shank fastener to withstand cold bending without fracture through 90° over a radius not greater than the diameter of the wire. Hardened-steel nails shall withstand bending without fracture through 45° over a radius of at least ⅛". Cement-coated nails up to 0.135" diameter shall show an average increase in immediate holding power of not less than 50% above that of identical non-coated nails.

The Canadian Standards Association's Standard 086-1970 on "Code of Recommended Practive for Engineering Design in Timber" provides design criteria for structurally non-graded and graded lumber, glue-laminated timber, plywood, piling, pole construction, and major fastenings. The 33-page chapter on timber fastenings covers split-ring connectors, shear-plate connectors, bolts, lag screws, Glulam rivets, and truss plates as well as their conditions of use. No consideration is given in this standard to the use of nails in timber structures.

The Canadian Wood Council of Ottawa, Canada, issued during 1971 a 4-page illustrated publication on "Canadian Wood Construction: Nails" to serve as a guide for the selection of nails used in timber construction.

The National Building Code of Canada and/or the Canadian Code of Residential Construction specify the conditions of use for nails.

ENGLAND

The British Standard Institution's Standard BS 1202-1966 on "Specification for Nails" consists of three parts which cover steel, copper, and aluminum nails, respectively. They contain listings of standard sizes, in British units and millimeters, and other dimensional data for (a) 24 types of 125 round-wire plain-shank or barbed and square-wire plain-shank or twisted as well as cut steel nails, (b) six types of 100 round-wire and square-wire plain-shank as well as cut copper nails, and (c) 73 round-wire plain-shank or barbed and square-wire twisted aluminum nails, in common use for general purposes. The head sizes are directly related to the shank diameters.

Other British Standards include BS 1494 on "Fixing Accessories for Building Purposes, Part I: Fixings for Sheet, Roof, and Wall Coverings" of 1964, and "Part 2: Sundry Fixings" of 1967; BS 1210 on "Wood Screws" of 1963; and BS 1579 on "Connectors for Timber" of 1960.

Metal nail plates (truss plates) are covered by Agrément Board Certificates. Pallet nails, with preference to annularly threaded nails, do not appear to have a British Standard.

Mechanical fasteners are covered in the recent "Handbook of Fixings and Fastenings" by Bill Launchburg, published by The Architectural Press of London, England.

The British Standard Code of Practice, CP 112 on "The Structural Use of Timber" of 1967 and its metric Part 2 of 1971, contain the requirements for joints in timber structures, yet do not cover truss-plate joints. This code is under revision for the incorporation of major changes in terms of limit state design (lower exclusive limit).

IRELAND

The Irish Standard Specification IS 105-1961 for "Wire and Cut Nails for Building Purposes" applies to 55 round-wire, seven oval-wire, and nine cut nails for use in the building trade, with the round-wire nails manufactured of low-carbon steel, copper or aluminum and with the oval-wire nails and cut nails manufactured of steel. The steel and copper wire nails shall withstand cold bending without fracture through 180° over a radius equal to that of the tested nail. Aluminum wire nails shall withstand cold bending without fracture through 90° over a radius not greater than the diameter of the nail. Steel cut nails shall withstand cold bending without fracture through 180° over a mandrel of the same diameter as the thickness of the nail.

For specialized purposes, other nails are used. Yet, no Irish standard appears to cover them. At least four types of metal nail plates are being used in the manufacture of trussed rafters.

The British Standard Code of Practice, CP 112, is to be followed, since an Irish code relating to joints in building construction does not exist.

DENMARK

Except for one standard, that is, DS 171 on Koniske Stifter, Denmark does not have any official standards on mechanical fasteners for wood. Those commonly used in Danish building construction are listed by Johansen and Larsen (1). They include seven plain-shank square-wire nails, two helically threaded nails, five annularly threaded nails, five lag screws, and three bolts. In addition to

(1) Johansen, M., and H. J. Larsen. 1968. Anchoring of Roofs. Byggeindustrien, Vol. 22, pp. 872-882. Translated from Danish into English by Building Research Station, England.

special types of nails and staples, special sheet-metal anchors (1) and nail plates are used by the Danish building industry.

Plastic-polymer coated plain-shank round-wire nails with flattened shanks were recently approved by the Danish Ministry of Housing for use on an equal basis as the same-length standard square-wire nails sofar as their withdrawal resistance is concerned, with the coated nails to provide 30% higher holding power in northern softwood than the non-coated prototypes (2). The report recently issued by Feldborg and Johansen (3) sheds light on the effectiveness of a number of Danish nails under various use conditions, thus may be influential in up-dating the Danish code of practice for the structural use of timber, DS 413-1968, a revised edition of which should be available shortly.

NORWAY

Norway issued, during 1948, standards for eight types of nails used in building construction: NS 65B and 66B on Rundhodenagler B and K, NS 726 and 727 on Konhodenagler, NS 730 on 1 - 10mm Rundhodenagler, NS 733 on Linsehodenagler, NS 737 on Senkhodenagler, and NS 738 on Flathodenagler. The catalogue of a Norwegian nail and screw manufacturer lists plain-shank and barbed square-wire nails, twisted square-wire nails, round-wire nails, square-wire spikes, screwnails, gypsumboard screws, and sheet-metal screws for use in building construction. Seven types of truss plates and three types of timber connectors are also used for timber construction.

SWEDEN

The Swedish Standards Commission issued, during December, 1947, Swedish Standards SMS 1382 to 1386 and 2008 on longitudinally fluted square-wire nails and brads and round-wire nails, brads, roofing nails, and fiberboard nails.

Square-wire, round-wire, longitudinally fluted square-wire, twisted square-wire and barbed and helically and annularly threaded round-wire nails with various types of heads and points as well as cut nails are described in detail in the catalogues of

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(2) Pedersen, V. June, 1972. Investigation of the Shank Withdrawal Resistance of 3½-In. Senco Nails for Determination of the Long-Range Effectiveness of the Coating. Also, Transverse Loading Tests with 3½-In. Coated Senco Nails. Jutland Technological Institute. Materials Testing Department. Project No. 30-0-2-19716. Reports Nos. 1 and 2.

(3) Feldborg, T., and M. Johansen. 1972. Withdrawal Resistance of Nails. Statens Byggeforskningsinstitut. SBI-Report No. 84. (Danish with English Summary).

the largest Swedish nail manufacturer, Gunnebo of Gunnebobruk, Sweden. They include regular-steel, hardened-steel, galvanized-steel, electro-plated steel, blued-steel, lacquered-steel, stainless-steel, copper, bronze, and brass nails for many purposes. The excellent four-part handbook on nails by this Company (4) provides information on Swedish nails and their uses in wood assembly and construction as well as on timber connectors, timber hangers, and timber joints of various types, including timber joints with sheet-metal reinforcements to allow close nail spacing as well as toe-nailed joints assembled with bent nails.

The Swedish building construction code covers the nailing in wood construction.

FINLAND

The single-sheet, two-page Finnish standard for nails, TES 58-38, of 1950, establishes the nomenclature and the sizes, in mm and in., of eighteen $\frac{1}{2}$" to 10" square-wire nails as well as of six plain-shank and one diagonally barbed round-wire nails.

The July, 1961, catalogue of the Aktieselskabet Nordiske Kabel —og Traadfabriker, up-dated by an insert of June 9, 1967, lists many more nails, including twisted square-wire nails. The company's standardized nail sizes do, however, coincide only in a few instances with those given in the Finnish standard for nails.

HOLLAND

The Dutch nail standards, as developed and published by the Nederlands Normalisatie-instituut during November, 1966, that is, NEN 5553 to 5560, cover 99 plain-shank round-wire nails of all kinds, twelve square-wire nails, and eleven twisted square-wire nails. The former nomenclature (e.g., 33/10) is replaced by the metric nomenclature (70 x 3.4).

The use of wire nails in timber joints is shown in Document No. 2a of the Houtvoorlichtingeinstituut of Amsterdam, Holland, of July, 1966, and January, 1969, with an insert of February, 1969. This publication covers many details, including the suggested sequence of driving nails. Other Institute publications cover the nailing of Douglas fir plywood gusset plates for frame and bent joints (Document No. 13 of October, 1971), wood screws as well as lag screws and their use in the fastening of steel gusset plates (Document No. 11 of May, 1968), sheet-metal anchors and hangers

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(4) Moeller, T., and V. Saretok. 1961. Gunnebo Spikhandbok. Gunnebo Bruks Aktiebolag. AB C.O. Ekblad & Co., Sweden.

and their use (Document No. 1a of September, 1960, and February, 1969), bolts and their use (Document No. 5 of May, 1966, and January, 1969), split-ring as well as shear-plate connectors and their use (Document No. 4a of November, 1967), and claw plates and their use (Document No. 6 of September, 1966, and January, 1969).

BELGIUM

The Belgian Standardization Institute published Standard NBN 305 on nails referring to nail numbers, nail length, in mm, and nail diameter, in BWG and mm. FABRIMETAL (Federation des Entreprises de l' Industrie des Fabrications Metalliques) prepared a catalogue of nails produced in Belgium, which include many low-carbon-steel and non-hardened and hardened-steel plain-shank, helically and annularly threaded round-wire nails and plain-shank and twisted square-wire nails. This catalogue also lists the producers of wood screws, including self-tapping screws, lag screws, bolts, and studs.

The Centre Scientifique et Technique a la Construction, in cooperation with the Federation Royale des Societies d'Architects de Belgique and La Confederation Nationale de la Construction, published during 1970 a construction handbook which contains a chapter on timber construction, including design and construction details covering nailed construction.

FRANCE

A listing of common wire nails in general use in France, as prepared by the Centre Technique du Bois of Paris, France, includes thirty-four 9/10 to 70/200 (diameter, in 1/10 mm/length, in mm) round-wire nails, twenty-three 27 to 200-mm long helically fluted nails, eleven 25 to 110-mm long twisted square-wire nails, and seven 45 to 90-mm long slender nails. Truss plates of three types, as used in French timber construction, are described in a publication issued by the Centre during January, 1968.

A textbook on timber construction, as published by Le Groupe de Coordination des Textes Techniques, Editions Eyrolles, provides information on the design and construction with wood.

PORTUGAL

The Portuguese Technical Commission of Mechanical Standardization published during February, 1963, Standards NP-277 to 286 covering 149 plain-shank and eleven diagonally barbed round-wire nails as well as ten twisted square-wire nails of various types. The steel used for making these nails is specified in Portuguese Standard NP-329 of June, 1964. According to this standard, nails have to bend without fracture around a rounded anvil to a 60° angle.

The performance and use of nails and other mechanical fasteners, including plastic ring connectors, for the jointing of pine timber are described in detail in a well-illustrated 312-page book as well as in lecture notes by Mateus, published during 1961 (5) and 1972 (6), respectively.

SWITZERLAND

Switzerland does not have any official standards for nails used in building construction. However, nail manufacturers, of which there are approximately ten, follow their own company standards which are similar. Thus, there are, e.g., DT-Norms, such as N-5201, 5203 to 5209, and 5211 to 5213 which cover round-wire nails with flat, countersunk, slightly countersunk, thick, deep, round, large, oval, large roofing, extra large roofing, and umbrella heads, as manufactured by Vereinigte Drahtwerke AG, Biel / Trefileries Reunies SA, Bienne, Switzerland. On the other hand, there are Swiss Railway Standards, such as, the Schweizerische Bundesbahnen SBB Standard Design No. 29,693 of September, 1964, covering the phosphate-coated and rather stout 2.5 x 60 and 3.1 x 80 round-wire nails and 3.5 x 90 twisted square-wire nails. None of these nail manufacturers produce helically or annularly threaded nails at this time.

GERMANY (BRD)

The standardization committee for wire and wire products of the German Standard Committee (DNA) published eleven German Standards for round-wire nails, that is, DIN 1144 for seven nails for light-weight building panels during September, 1961; 1151 for 32 general-purpose nails with flat and slightly countersunk heads; 1152 for 20 nails with deep brad heads; 1153 for 16 nails with countersunk heads; 1155 for 9 nails with round heads; 1156 for 17 headless nails; 1157 for 9 nails with oval heads; 1158 for 16 hook nails; 1159 for 7 U-shaped staples; and 1161 for 18 stout nails with

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(5) Mateus, T.J.E. 1961. Bases Para o Dimensionamento de Estructuras de Madeira. Laboratorio Nacional de Engenharia Civil. Lisbon, Portugal. Memoria No. 179.

(6) ——————. April, 1972. O Embrego da Madeira de Pinho em Obras Provisorias (Cimbres, Cofragens, Etc.). Cursa de Promocao Profissional 501. Ministerio das Obras Publicas. Laboratorio Nacional de Engenharia Civil. Lisbon, Portugal. Documento 501-2.1.

wedge or diamond points with countersunk, slightly countersunk, and oval heads during April, 1959; and 1160 for 16 nails with large and extra-large heads of three and four shank diameters, respectively, during November, 1963. Screwnails of 69 different sizes with slotted round heads and flat countersunk heads are described in DIN 7514 and 7515, respectively, of December, 1943. Pallet nails are covered in DIN 15-146 and 147. Recent tests on helically threaded nails and spikes as well as plastic-polymer coated staples and nails should result in new German Standards within a short time. Wood screws and bolts are covered in separate standards.

The jointing with mechanical fasteners in timber construction is covered in DIN 1052 on "Design and Construction of Timber Structures" of October, 1969. This standard was interpreted by Moehler et al in a lengthy (52-page) report (7) which can serve as a more or less up-to-date textbook on timber construction as practiced in West Germany. The well illustrated chapters on timber joints present probably the most detailed treatise covering this subject matter.

GERMANY (GDR)

The jointing of timbers, as practiced in Europe and the USA, is described in detail in a well illustrated up-to-date textbook by Moenk (8).

CZECHOSLOVAKIA

Czechoslovakian State Standard CSN 73 2810 of July, 1963, on Performance of Timber Structures briefly covers joints of elements of timber structures. Details on the jointing of timbers are presented in a textbook by Dutko et al (9).

POLAND

Polish Standards of 1971 govern nails manufactured in Poland: Standard BN-70/5028-12 covers 19 round-wire and 19 square-wire nails; BN-70/5028-17 covers 12 helically

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(7) Moehler, K., J. Ehlbeck, G. Hempel, and P. Koester. January, 1971. Interpretation of DIN 1052, Forms 1 and 2 -- Design and Construction of Timber Structures -- October, 1969 edition. Arbeitsgemeinschaft Holz in cooperation with Bund Deutscher Zimmermeister, Bruderverlag, Karlsruhe, Germany.

(8) Moenk, W. 1966. Holzbau (4th Edition). VEB Verlag fuer Bauwesen. Berlin, German Democratic Republic.

(9) Dutko, P., F. Lederer, P. Ferjencik, and L. Cizek. 1966. Drevene Konstrukeie. Slovenske Vydavatedstvo Technickej Literatury. Bratislava, Czechoslovakia.

threaded round-wire nails with multiple thread flutes, large thread angle, and medium diamond point and 11 similar helically threaded nails with long diamond point; BN-70/5028-18 covers 9 helically threaded pallet nails with single thread flute and medium thread angle; BN-70/5028-19 covers 6 twisted square-wire pallet nails; and BN-70/5028-25 covers 13 annularly threaded round-wire nails.

Polish Standard PN-64/B-03150 of July, 1972, covers the design and computation of timber structures, providing information on the use of nails, wood screws, timber connectors, keys and dowels of various types, clamps, and gusset plates.

Recently, Nozynski (10) determined the withdrawal resistance and lateral load-transmission of single and multiple plain-shank and twisted nails of triangular cross-section in two-member and three-member timber joints, advancing design data for these nails.

ROMANIA

The Romanian Standardization Institute issued during November, 1971, Standard STAS 2111-71 on Steel Nails covering 247 plain-shank round-wire nails of various types, eight plain-shank square-wire nails, three helically threaded nails, eleven U-shaped staples, and 14 L-shaped staples. Wood screws are covered in Standards STAS 1451-71, 1452-71, 1453-56, 1454-56, and 1455-71. Washers are described in Standard STAS 7565-66.

USSR

The status of the art of the mechanical fastening of timbers in the USSR is presented in the textbook by Karlsen et al (11).

BRAZIL

The United States of Brazil do not have any official standards for nails and other mechanical fasteners used in building construction with the exception of those for bolts and a variety of screws, including wood screws (P-PB-97 of 1970).

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(10) Nozynski, W. 1969. Test Results of Timber Joints with Triangular Nails. Instytut Techniki Budowlanej. Technical Bulletin 29/30. Warsaw, Poland. Pp. 51-63.

__________. 1969. Ditto. Technical Bulletin 1/2. Pp. 4-7.

__________. 1967. Special Three-Sided Nails for Building. Ditto. Technical Bulletin 1. Pp. 16.

(11) Karlsen, G.G., V.V. Bolshakor, M.Y. Kagan, G.V. Sventsitsky, K.V. Aleksandrovsky, I.V. Bochkaryov, and A.I. Folomin. 1967. Wooden Structures. MIR Publishers. Moscow, USSR. Russian (1961) and English (1967) Editions.

The design of timber structures and joints assembled with mechanical fasteners is covered in Brazilian Standard NB 11 of 1951 (1967).

SOUTH AFRICA

The Bureau of Standards of the Republic of South Africa published SABS 820-1966 on "Standard Specification for Mild Steel Nails" during October, 1966. It covers 74 plain-shank, 2 barbed, and 32 annularly threaded round-wire nails; 2 plain-shank and 4 twisted square-wire nails; and 6 cut nails.

The Code of Practice for Timber Building, SABS 082-1965, includes a chapter on timber joints assembled with nails, bolts, timber connectors, and framing anchors. The SABS Standard Building Regulations of October, 1970, cover nailed, screwed, bolted, connectored, and anchored joints.

AUSTRALIA AND NEW ZEALAND

Australia and New Zealand do not have official standards for nails used in building construction and for the wire from which these nails are made. Nails from different manufacturers may vary somewhat, yet, are relatively uniform since most of the rods used in both countries originate from a single manufacturer and are cold-drawn into wire by a single nail manufacturer in each country. Mechanical fasteners of all kinds used in timber construction are described in "Timber Fasteners" published by the Commonwealth Experimental Building Station of the Department of Works in their Notes on the Science of Building, NSB 121, of August, 1972.

The Standards Association of Australia published Australian Standard CA 38-1971, on "Construction in Light Timber Framing", CA65-1972 on "Australian Standard Rules for Timber Engineering Design", and C188-1972 on "Australian Standard Methods for the Determination of Basic Working Loads for Metal Fasteners for Timber". The latter standard appears to be unique in the coverage of its subject matter. Australian Standard E41-1965 on "Flat Pallets for Materials Handling" covers nail types and sizes used for pallet assembly.

INDIA

The Indian Standards Institution published during 1961 the Indian Standard Specification IS-723 on "Mild Steel Wire Nails" and IS-451 on "Wood Screws" and during 1963 the Indian Standard Code of Practice IS-2366 on "Nail-Jointed Timber Construction". This Institution also published the National Building Code of India, covering "Wood" in Part VI, Section 3.

Masani et al (12) summarized the results of research conducted up to 1968 on the effectiveness of plain-shank and round-wire nails in Indian timbers. This well-illustrated report served as the basis for the present design of nailed structures in India.

JAPAN

The Japanese Industrial Standard JIS A-5088 covers 18 nails commonly used in building construction and for other purposes. A design manual for timber construction, published by the Architectural Institute of Japan, contains information on joints in timber construction.

USA

The United States of America standardized its wire and cut nails, brads, staples, and spikes in Federal Specification FF-N-105B of March 17, 1971. This specification covers more than 512 plain, barbed, and threaded round-wire brads, nails, and spikes of 30 styles, 93 plain, barbed, and twisted square-wire nails and spikes of five styles, and 40 cut nails and spikes of eight styles. Among the threaded nails are 75 helically and 36 annularly threaded nails. In addition, numerous special nails, made and sold in accordance with the manufacturers' standard specifications, are produced for specific applications. Altogether some 10,000 different standardized nails (13) may be produced during the course of a year, if such variables as different nails, points, heads, threads, and finishes are given consideration when establishing the number of different nails available in the USA.

According to the Federal Specification, nails need to conform to the following requirements: The steel, copper, and copper-clad steel wires used in the manufacture of nonhardened steel, copper, and copper-clad steel nails shall be sufficiently ductile for the finished product to withstand cold bending without fracture through 180° over a diameter not greater than the wire diameter or thickness of the nail. This requirement is not applicable to barbed and deformed-shank nails. Hardened-steel nails shall withstand cold bending without fracture through 20° over a diameter not greater than the wire diameter. Aluminum nails, brass nails, and medium-carbon-steel cut nails and spikes shall withstand cold bending without fracture through 90° over a diameter not greater

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(12) Masani, N.J., K.S. Pruthi, S. Lal, and B. Prasad. April, 1971. Nail Joints in Timber Structures: Part III. The Forest Research Institute, Dehra Dun, and The National Buildings Organization, New Delhi, India. Technical & Research Report No. 26.

(13) Stern, E. G., and A.S. Tisch. February and March, 1969. The Improved Nail, Its Status. Wire & Wire Products. Vol. 44, No. 2, pp. 54-56 and 105-108; and No. 3, pp. 50-51 and 100-102.

than the diameter of the wire or the thickness of the sheet from which the fastener was sheared. Wrought-iron and steel spikes shall withstand cold bending without fracture through 180° over a diameter not greater than the dimension of the square spike. Cement-coated nails of 0.135" or smaller diameter shall offer an immediate withdrawal resistance, obtained within 24 hours after driving, of not less than 50% above that of an identical bright nail prior to cleaning. Masonry nails of given sizes shall resist given axial withdrawal loads from 3000-psi. concrete.

Numerous staples and nails as produced by five leading manufacturers (Duo-Fast, Paslode, Power-Line, Senco, and Spotnails) are described in Manual No. 1-69-73 on "Building Construction Fasteners" issued by the Industrial Stapling Manufacturers Institute of City of Industry, California. This Manual was incorporated in the BOCA Basic Building Code of the Building Officials Conference of America (Approval No. 69-41), Southern Standard Building Code of the Southern Building Code Congress (Report No. 7009), Uniform Building Code of the International Conference of Building Officials (Report No. 2403), and other codes.

The Industrial Stapling and Nailing Technical Association* issued their Manual No. 19-71 on "Pneumatic and Mechanically Driven Building Construction Fasteners" which became an integral part of Bulletin No. UM-25c of August 23, 1971, on "Application and Fastening Schedule -- Power-Driven, Mechanically Driven, and Manually Driven Fasteners" issued by the Department of Housing and Urban Development of the U.S. Federal Housing Administration. This publication describes in detail the staples, nails, corrugated fasteners, and tying fasteners to be used under given conditions for diaphragm and non-diaphragm constructions. It shows the number of fasteners required and the allowable loads transmitted by these fasteners.

The 1971 edition of the National Design Specification for Stress-Grade Lumber and Its Fastenings, published by the National Forest Products Association of Washington, D.C., provides design criteria for timber structures, including those for timber joints assembled with nails and spikes of various types, wood screws, lag screws, bolts, timber connectors (split rings, toothed rings, and shear plates), and metal-plate connectors.

Basic design information for timber joints assembled with mechanical fasteners is published in the Chapter on Timber Fastenings of the Wood Handbook (14), under revision at this time. Comprehensive information on the mechanical fastening of southern pine

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(14) Forest Products Laboratory. 1955. Wood Handbook. Agricultural Handbook No. 72. U.S. Government Printing Office. Washington, D.C. Pp. 165-202.

* the successor of the Industrial Stapling Manufacturers Institute.

was compiled by the author (15) and republished in condensed form by Koch (16). Basic data on the performance of nails and spikes in hickory, one of the important American hardwoods of great potential use, were developed by the author (17). A considerable part of a new textbook on timber construction by Hoyle (18) is devoted to the use of mechanical fasteners for wood. An even more recent textbook by Gurfinkel (19) devotes 63 pages to the design with mechanical fasteners, including five insert pages on the design of wood joints with multiple bolts, lag screws, and timber connectors. Other textbooks and a manual of practice on the design with wood are in preparation and will throw additional light on the effectiveness of mechanical fasteners and joints in timber structures.

SUMMARY OF SURVEY with Respect to Nails and Nailed Joints

The number of nails of given types covered in the national standards of 16 countries of the 24 countries surveyed are summarized in Table I.

1) Of the 16 national standards, the USA Standard covers more nails than any other national standard, that is, 645 nails or 22% of the 2893 nails listed. The USA is followed by Canada and England, each with 14% of all the nails listed, Germany (BRD) with 11%, Rumania with 10%, Portugal with 6%, Holland and South Africa with 4% each, and the remaining eight countries with a total of 15% of all the nails listed.

   Of the grand total of more than 2893 nails listed, 86% are round-wire nails, 11% are square-wire nails, and 3% are cut nails. Among the 2495 round-wire nails, 74% are plain-shank nails, 3% are barbed nails, 5% are helically fluted nails ("formed nails"), 10% are helically threaded and 5% are annularly threaded nails

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(15) Stern, E. George. December, 1969. Mechanical Fastening of Southern Pine, A Review. Virginia Polytechnic Institute and State University. Wood Research & Wood Construction Laboratory. Bulletin No. 87 (98 pages).

(16) Koch, P. January, 1972. Utilization of the Southern Pines. Agricultural Handbook No. 420. Volume I. U.S. Government Printing Office, Washington, D.C. Chapter 24. Pp. 1221-1324.

(17) Stern, E. George. November, 1964. Nails and Spikes in Hickory. U.S. Forest Service. Southeastern Forest Experiment Station. Hickory Task Force. Report No. 9.

(18) Hoyle, R.J., Jr. November, 1972. Wood Technology in the Design of Structures. Mountain Press Publishing Company. Missoula, Montana. Chapters 11-14. Pp. 135-212.

(19) Gurfinkel, G. 1973. Wood Engineering. Southern Forest Products Association. Chapter 6. Pp. 129-187 and XVI - XX.

("deformed nails"). Among the 305 square-wire nails, 62% are plain-shank nails, 12% are diagonally barbed nails, and 26% are twisted nails. Thus, 22% of the nails listed in the national standards are helically fluted, helically or annularly threaded, and twisted nails.

In the USA standard, 79% of the nails listed are round-wire nails, 14% are square-wire nails, and 6% are cut nails. Among the 512 round-wire nails, 69% are plain-shank nails, 9% are barbed nails, 15% are helically threaded and 7% are annularly threaded nails. Among the 93 square-wire nails, 46% are plain-shank nails, 40% are diagonally barbed nails, and 14% are twisted nails. Thus, 19% of the American nails listed are helically or annularly threaded and twisted nails. These deformed-shank nails are estimated to make up approximately 20% of all the nails produced in the USA.

2) The sizes of standard 10d -- 3" (76mm) bright plain-shank common wire nails, with flat heads and medium diamond points, and their equivalents in 21 of the 24 countries surveyed are listed in Table II.

   In the light of the fact that the standardized nail lengths in different countries are measured in inches and/or millimeters, the lengths of the nails listed vary from 70 to 80mm.

   The diameters of the round-wire nails vary from 2.6 to 4.1mm in England and 3.0 to 4.5mm in 17 other countries. The diameters (lengths of the diagonals) of the square-wire nails vary from 2.8 to 3.1mm in the five countries listing square-wire nails as their common wire nails. These five countries comprise the four Scandinavian countries which are the traditional home of square-wire nails, and Poland which lists both round-wire and square-wire nails as her common wire nails.

3) Of the 24 countries surveyed, at least five countries established test procedures and criteria for nails, as is shown in Table III.

   The Federal standard of the USA describes test procedures and criteria for cut nails, common nails, hardened-steel nails, coated nails, and masonry nails. Canada has standard test procedures and criteria for common nails, hardened-steel nails, and coated nails. Ireland has standard test procedures and criteria for cut nails and common nails, Denmark for coated nails, and Portugal for common nails.

   In testing the ductility and pliability of nails, the finished nail is to withstand cold bending without fracture around an anvil of a given radius through a given angle, as is shown in Table III. In certain instances, the radius or the diameter of the anvil is that of the nail diameter or nail thickness; in one instance, the radius of the anvil is specified as to be at least $\frac{1}{8}$"; and in one instance, the anvil is just to be rounded. The bend angle to be reached without fracture varies from 20° and 45° for hardened-steel nails to 60°, 90°, and 180° for common nails made of different metal alloys.

Coated nails shall offer a 50% increase in withdrawal resistance which is attributable to cement-coating or a 30% increase which is attributable to plastic-polymer coating. Masonry nails shall offer a certain withdrawal resistance in 3000-psi. concrete.

4) A comparison of basic design procedures for nailed and spiked joints in the USA and Germany (BRD) reveals that the German regulations go into considerably more detail; thus, cover many situations to which no consideration is given in the American specification. The reason for the basic difference might be attributed to the fact that the German design procedures are issued by the German Government and mandatory; whereas the American design procedures are industry-sponsored recommendations accepted by the various agencies and code authorities as basic procedures to be ammended and implemented as justified.

Some of the details in the German regulations which are in contrast with their American counterpart are listed below:

a) As to allowable USA nail-withdrawal loads, they are governed by the Design formula, $P = 1320\ G^{2.5}D$, in lb., and special adjustment factors; whereas in Germany nail withdrawal loads can be given consideration only if these loads are short-time loads, such as wind loads, and the nails are designed to resist suction and uplift forces acting on sheathing, rafters, purlins, etc. For the direct hanging of ceilings, the use of nails with deformed shanks is recommended in Germany in the light of the fact that ceilings represent long-time loads and that plain-shank nails lose a considerable part of their initial holding power during timber seasoning.

b) With respect to the lateral load transmission by nails, the American design loads are influenced by the 1.5th power of the nail diameter; whereas the German design loads are influenced by the 2nd power of the nail diameter as well as by a decrease in the load transmission with increase in the nail diameter. Thus, the American design load is determined by $P = KD^{1.5}$; whereas the German design load is determined by $P = KD^2/(1+D)$.

c) In the American specification, there is no requirement for more than one nail in a joint transmitting lateral loads; whereas the German counterpart requires a minimum of four nails, although there are certain exceptions, such as the requirement of two nails for the fastening of boards.

d) The number of nails in a joint is of no influence on the American design load per nail; whereas more than ten nails in a row in the direction of the force requires a decrease of 10% and more than twenty nails in a row in the direction of the force requires a decrease of 20% of the German design load per nail.

e) Whereas provisions are included in the American specification for the benefits derived from the use of threaded hardened-steel nails, no such provision is made in the German counterpart.

f) The nailing of flat members to round members requires a 33% decrease in the lateral load transmission per nail in the German standard and no decrease in the American specification.

g) Driving plain-shank nails (not threaded hardened-steel nails) into green wood which seasons under load or into fire-retardant pressure-treated wood, calls for a 25% reduction in the lateral load transmission per nail in the USA; whereas nailing of green wood which subsequently seasons calls for a 33% reduced lateral load per nail in Germany.

h) The American normal design load is subjected to various duration-of-load factors, with consideration of wind load permitting a 33% increase; whereas a 15% increase is allowed for short-term loads, such a wind loads, in Germany.

i) Nails driven into end-grain lumber and subsequently laterally loaded call for a 33% decrease in the design load applicable to nails driven into side-grain lumber in the USA; whereas nails in end-grain lumber cannot transmit any design loads in Germany.

j) Nails driven into predrilled holes of a diameter of at least 85% of the nail diameter can transmit a 25% higher lateral load in German softwoods and a 50% higher lateral load in German beech and oak than nails driven without predrilling; whereas the design load in the USA is not influenced in one way or another if the hole is not larger than 75% of the diameter of the nail driven into light timber and 90% of the diameter of the nail driven into dense timber.

k) Nails loaded in multiple shear can, under given conditions, transmit the corresponding multiple design load in Germany; whereas in the USA nails in double shear can transmit only 133% of the single-shear load if the thickness of the side member is at least 1/3 of that of the center member, 166% of the single-shear load if the thickness of the side member is at least equal to the thickness of the center member, and 200% of the single-shear load if the side member is at least 3/8" thick and if the nails are smaller than 12d and clinched, while threaded hardened-steel nails need not be clinched.

l) Whereas in the USA nails are to be spaced in such a way as not to cause "unusual splitting"; nails in Germany are to be located at given minimum end distances, minimum edge distances, and minimum and maximum spacings, with multiple nails in a row parallel to the grain to be off-set at least one nail diameter.

m) The lateral load transmission by nails fastening metal side plates to timbers may be increased 25% in both countries; yet, in Germany, this increase is limited to plates of a minimum thickness of 2mm.

5) A comparison of deformation limitations for nailed joints in the USA, Australia, and Germany, or proposed for inclusion in design standards of other countries, was made by Boyd (20) and Morris (21). In the USA, the design load for timber joints has conventionally been based on a maximum joint deformation of 0.015 in. (0.38mm), which reflects a design load of one-sixth of the ultimate test load transmitted by softwood joints. A similar joint deformation limit has been used in Australia. On the other hand, the German design load has been based on a quarter of the ultimate test load and a joint deformation of 0.059 in. (1.50mm).

Joints assembled with nails of small diameters deform under load less than those assembled with nails of large diameters. Thus, two types of similar joints, assembled with nails of different sizes, support different loads at given joint deformations.

Since knowledge of joint deformation is necessary for many forms of structural analysis, Morris concluded that design loads for nailed joints should be related to specified member displacements. Using an expression involving load, deformation,

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(20) Boyd, J.P. 1965. The Significance of Basic and Applied Research on Mechanical Fasteners for Residential Construction in Australia. Building Science. Vol. 1, Pp. 33-34.

(21) Morris, E.N. November, 1971. Design of Nailed Joints for A Given Short Term Displacement. Timber Research and Development Association, Hughenden Valley, High Wycombe, Buckinghamshire, England. Research Report E/RR/33.

nail diameter, and wood species, Morris developed design curves which allow the derivation of design loads for lateral load transmission by nails in single shear corresponding to given short-term joint deformations.

CONCLUSIONS

The information presented is fragmentary. Additional comparisons of mechanical fasteners and their use in various countries around the world could provide background information which could be helpful in establishing optimum design criteria for such fasteners and their use in the jointing of wood. In the light of this, it is hoped that such information which is not available to the author at the time of this writing will be made available to him. This would permit him to prepare a more complete report and, therefore, a more valuable one of the status of the art.

The USA manufactures a considerably greater variety of mechanical fasteners for wood and, particularly, of nails and staples than any other country in the world. Thus, such fasteners as threaded stainless-steel and Monel boat nails of a variety of sizes are well-known export products in high demand wherever boats are built. These developments are, of course, the result of a greater demand for more effective and efficient mechanical fasteners for wood in a country which is abundantly covered with forests and where wood plays a more important role in the life of any individual than in most other countries. Furthermore, the mass-production of mechanical fasteners for wood and the warehouse availability of a large variety of large quantities of such mechanical fasteners are more feasible in a country with a population of more than 200 million people than in a country having a small population.

Thus, it appears that the USA will remain one of the more important leaders in the field of mechanical fasteners, until some of the developing countries with relatively large untapped forests and, therefore, with a natural and self-perpetuating supply of timber will learn to use their self-renewable material for building and construction purposes effectively and efficiently. Thereby, they will create a large demand for mechanical fasteners for wood, which has to be met by either local manufacture or importation. The latter would, of course, constitute some drain on the foreign exchange available and, therefore, not always be defendable.

It is hoped that the time will come when every country makes maximum use of its old-growth as well as newly grown timbers. This will be the time when both common and improved mechanical fasteners for wood will have to be readily available not only where needed but also in the quantities required.

Too little has been published about the outstanding advantages of the use of power-driven staples, in place of hammer and power-driven common nails, for building and construction. With the availability of plastic-polymer coated, $2\frac{1}{2}$"-long, 15-gauge (0.073" x 0.067") staples and with even longer staples in the offing, it might be well, especially for developing countries, to promote the introduction and the use of staples of appropriate sizes instead of common nails. This is especially desirable in their efforts to provide such mass-housing as is so urgently needed and can be made available in the foreseeable future only by the proper use of wood and wood-base products fastened together with nails and/or staples to form building components and complete house units (22).

ACKNOWLEDGMENTS

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(22) Stern, E. George. July, 1971. Potential of Building with Wood in South America. Forest Products Journal, Vol. 21, No. 7, pp. 14-17; also, July/August, 1971. Wood Construction in South America. Build International, Vol. 4, No. 4, pp. 217-221.

TABLE I

Types and Numbers of Nails Covered in National Standards of Countries Surveyed

Country	Round-Wire Nails						Square-Wire Nails			Cut Nails	Total	
	Plain	Barbed	Longitudinally Fluted	Helically Fluted	Helically Threaded	Annularly Threaded	Plain	Diagonally Barbed	Twisted		No.	%
Canada	198	--	3	101	41	56	--	--	--	--	399	14%
England	300	19	-	---	---	--	32	--	7	38	396	14%
Ireland	62	--	-	---	---	--	--	--	--	9	71	2%
Denmark	---	--	-	---	2	5	7	--	--	--	14	0%
Sweden	40	--	-	---	---	--	49	--	--	--	89	3%
Finland	6	1	-	---	---	--	18	--	--	--	25	1%
Holland	99	--	-	---	---	--	12	--	11	--	122	4%
Belgium	50+	--	-	---	---	--	--	--	--	--	50+	2%
France	41	--	-	27	---	--	--	--	27	--	95	3%
Portugal	149	11	-	---	---	--	--	--	10	--	170	6%
Germany*	167	--	-	---	140	--	--	--	--	--	307	11%
Poland	19	--	-	---	32	13	19	--	6	--	89	3%
Rumania	272	--	-	---	3	--	8	--	--	--	283	10%
South Africa	74	2	-	---	---	32	2	--	4	6	120	4%
Japan	18	--	-	---	---	--	--	--	--	--	18	1%
USA	353**	48	-	---	75	36	43	37	13	40	645+	22%
Total	1848+	81	3	128	293	142	190	37	78	93	2893+	100%
	64%	3%	0%	4%	10%	5%	7%	1%	3%	3%	100%	

* (BRD) ** exclusive of round-wire staples. + and more.

TABLE II

Sizes of Standard 10d -- 3" (76 mm) Bright Plain-Shank Common Wire Nails, with Flat Heads and Medium Diamond Points, and Their Equivalents in Various Countries

Country	Nail Wire	Wire Diameter In.	Wire Diameter Mm	Wire Gauge USSWG	Wire Gauge BISG	Head Diameter In.	Head Diameter x Diam.	Count per Pound 450g.	Count per Pound 500g.
Canada	Round	0.144	(3.7)	--	9	5/16	(2.2)	67	(74)
England	Round	0.160	4.1	--	8	--	2	--	55
	Round	0.144	3.7	--	9	--	2.25	--	70
	Round	0.128	3.3	--	10	--	2.25	--	88
	Round	0.116	2.9	--	11	--	2.25	--	107
	Round	0.114	2.6	--	12	--	2.25	--	127
Ireland	Round	0.176	4.5	--	7	--	2	--	--
	Round	0.160	4.1	--	8	--	2	--	--
Denmark	Square		3.1 (for 31/80)						
Norway	Square		3.1 (for 31/80)						
Sweden	Square		2.8 (for 28/75)						94
	Square		3.1 (for 31/75)						115
	Round		3.1 (for 31/75)						111
Finland	Square		2.8 (for 75x2.8)						
Holland	Round		3.1 (for 70x3.1)						
	Round		3.4 (for 70x3.4)						
Belgium	Round		3.4 (for 75/10)						
	Round		3.8 (for 75/9)						
France	Round		3.0 (for 70/17)			6.5mm			120
	Round		3.4 (for 70/18)			6.5mm			94
Portugal	Round		4.0 (for 40x70)			8.8mm			
Germany	Round		3.1 (for 31x70)						
Czechoslovakia	Round		3.1 (for 3.1x70)						
	Round		3.1 (for 3.1x80)						
Poland	Round		3.0 (for 3.0x70)						
	Round		3.0 (for 3.0x80)						
	Square		2.8 (for 2.8x70)						
	Square		3.0 (for 3.0x80)						
Rumania	Round		3.0 (for 3.0x70)						
	Round		3.0 (for 3.0x80)						
Brazil	Round		3.4 (for 3.4x74)						
South Africa	Round	0.144	(3.7)	--	9		2.25	70	
	Round	0.160	(4.1)	--	8		2	55	
Australia	Round	0.144	(3.7)	--	9	0.22			80
India	Round		3.55 (for 3.6x80)	--	9				
Japan	Round		3.40 (for 3.4x75)						
USA	Round	0.148	(3.8)	9	--	5/16	(2.1)	65-69	(72-77)

TABLE III

Test Procedures for Nails Established in Various Countries

Country	Cold Bending Without Fracture to Given Angle: Cut Nails	Common Nails	Hardened-Steel Nails	Withdrawal Resistance of Coated Nails	Masonry Nails
Canada	---	90°a	45°c	+50%d	---
Ireland	180°b	90°a ; 180°b	---	---	---
Denmark	---	---	---	+30%e	---
Portugal	---	60°	---	---	---
U.S.A.	90°b	90°b ; 180°b	20°b	+50%d	X

(a) Bending over radius not greater than nail diameter.
(b) Bending over diameter not greater than nail diameter or nail thickness.
(c) Bending over radius of at least ⅛".
(d) Given increase in withdrawal resistance attributable to cement coating.
(e) Given increase in withdrawal resistance attributable to plastic-polymer coating.

TABLE OF ILLUSTRATIONS

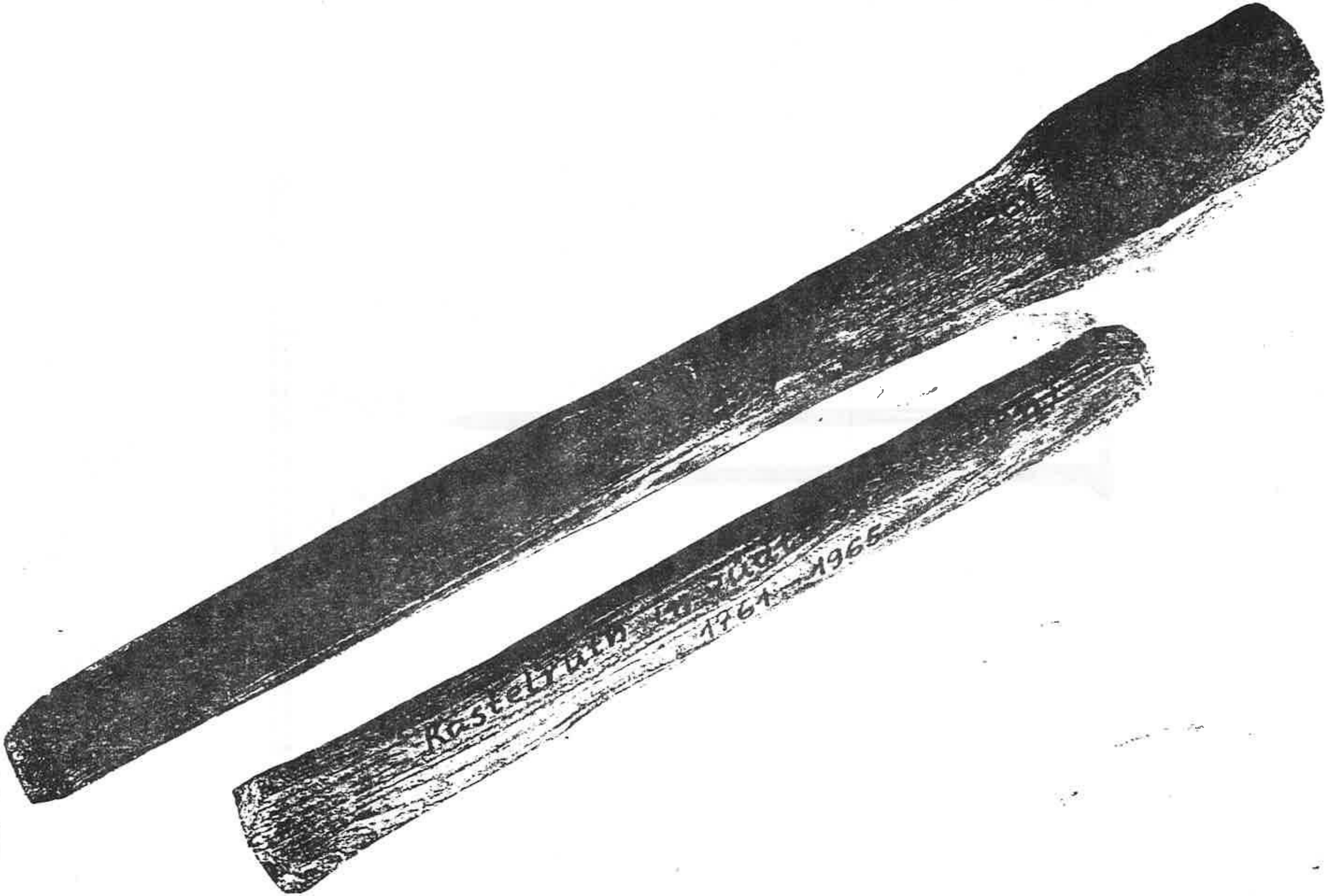

Fig. 1. - Top, square oak pin of 14" length used for erection of roof of Castle Urach, Germany, during the 15th century. Bottom, rounded pine pin of 10" length from Kastelruth, Northern Italy, built during 1761.

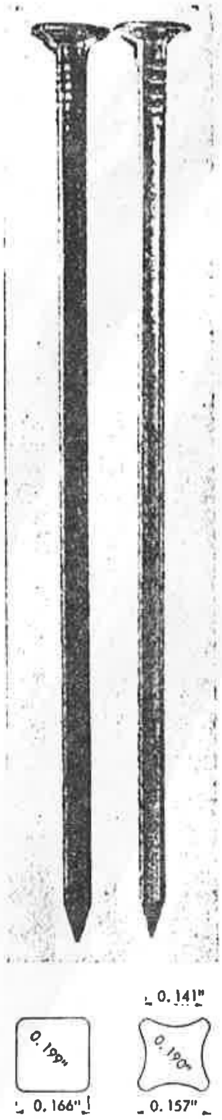

Fig. 2.—Slender square-shank nails of 5″ length with flat sides (left), as used in southern Norway and Finland, and with longitudinal flutes (right), as used in northern Norway and Sweden.

PROCESS OF MANUFACTURING SHEET METAL NAILS

Filed Feb. 7, 1955

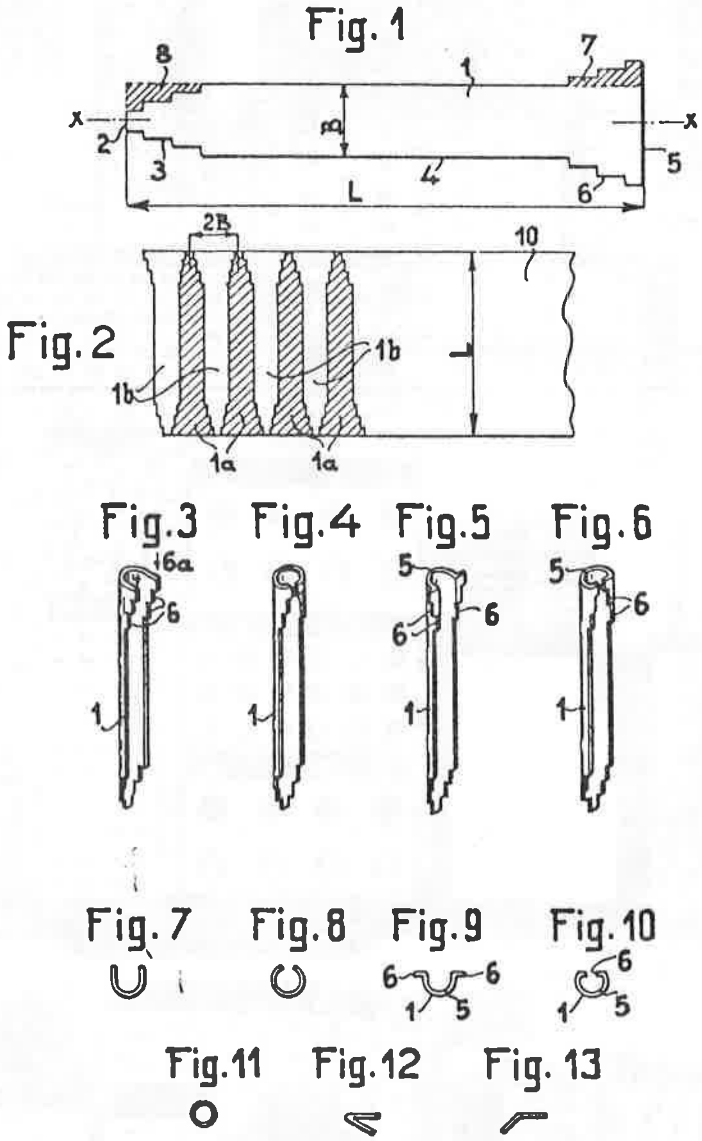

Fig. 3.- Porak sheet-metal nails of various configurations.

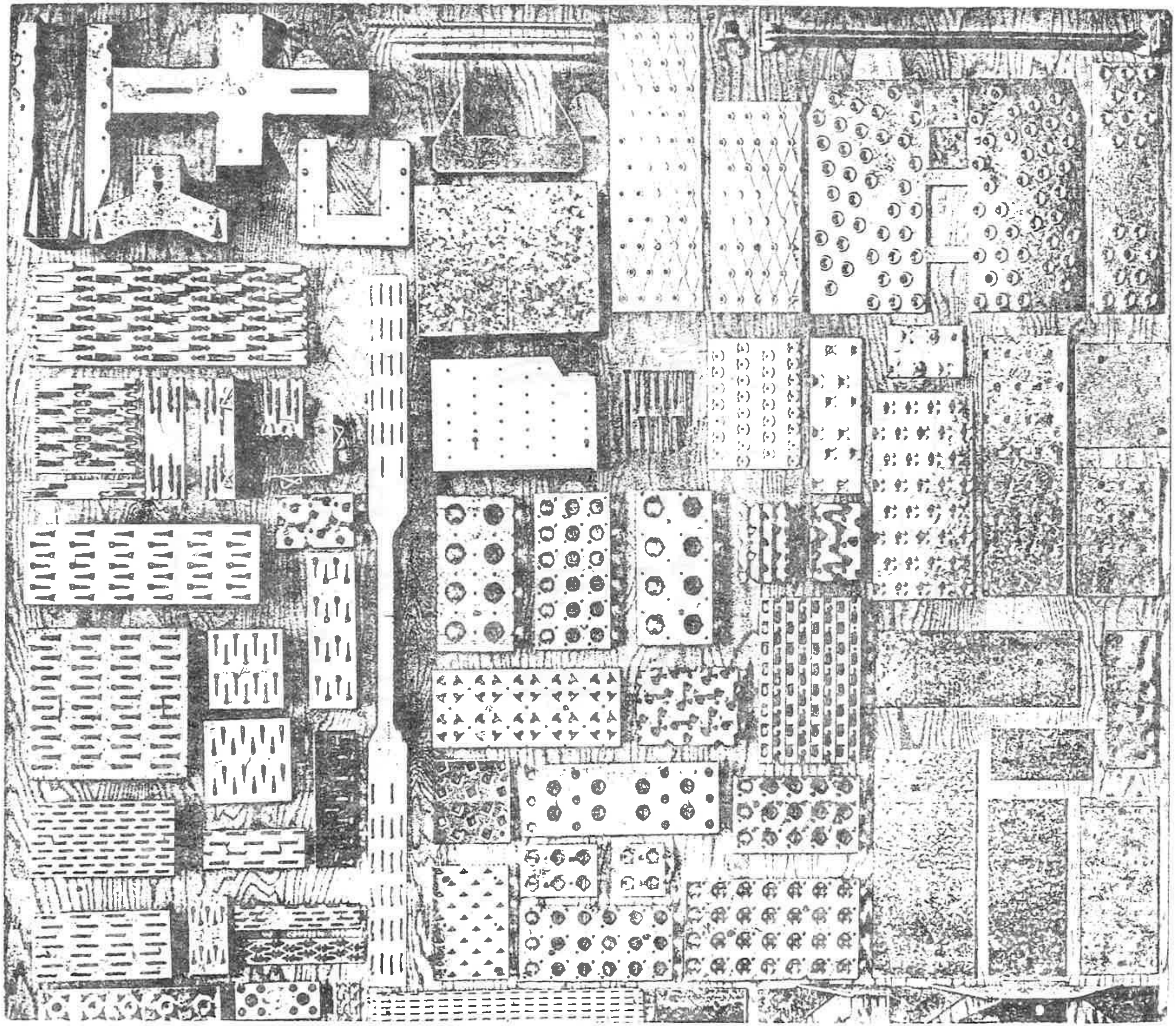

Fig. 4.- Metal nail plates of various types: Right, predrilled flat and deformed; center-bottom, barbed and pronged; left, toothed.

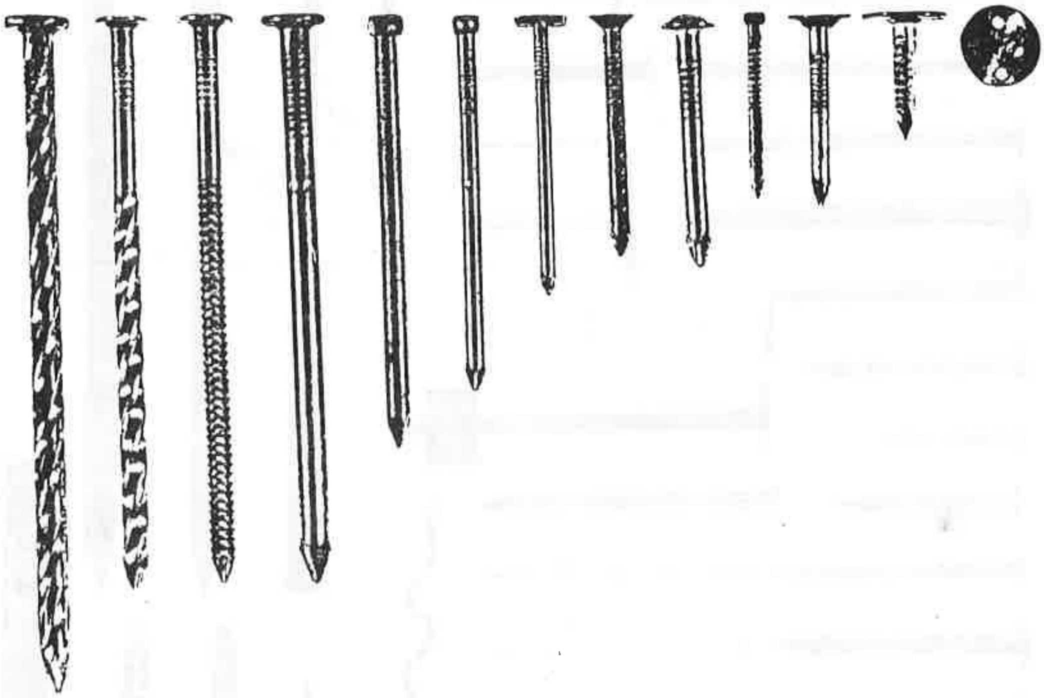

Fig. 5.- English nails (1965).

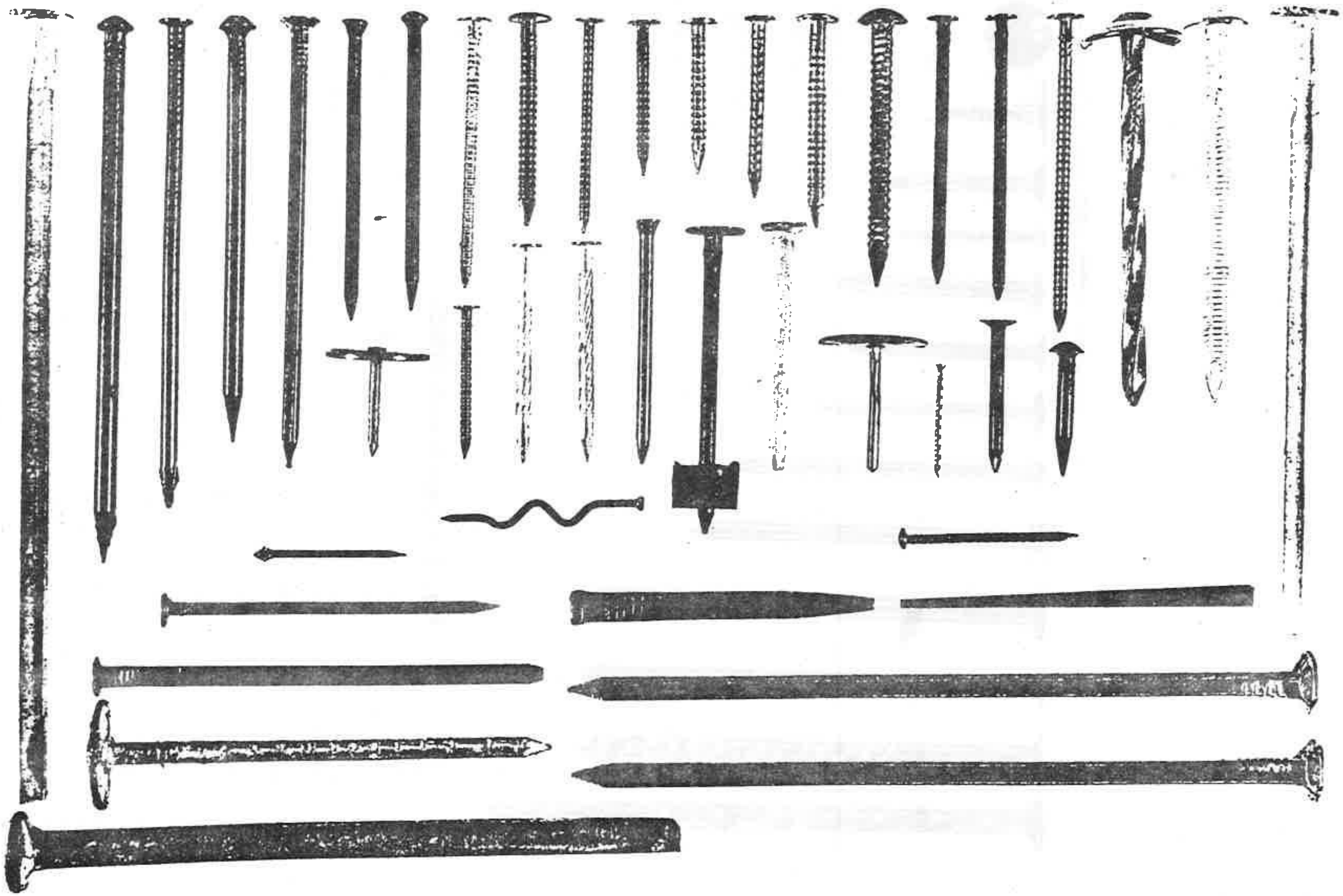

Fig. 6. - Swedish nails (1961).

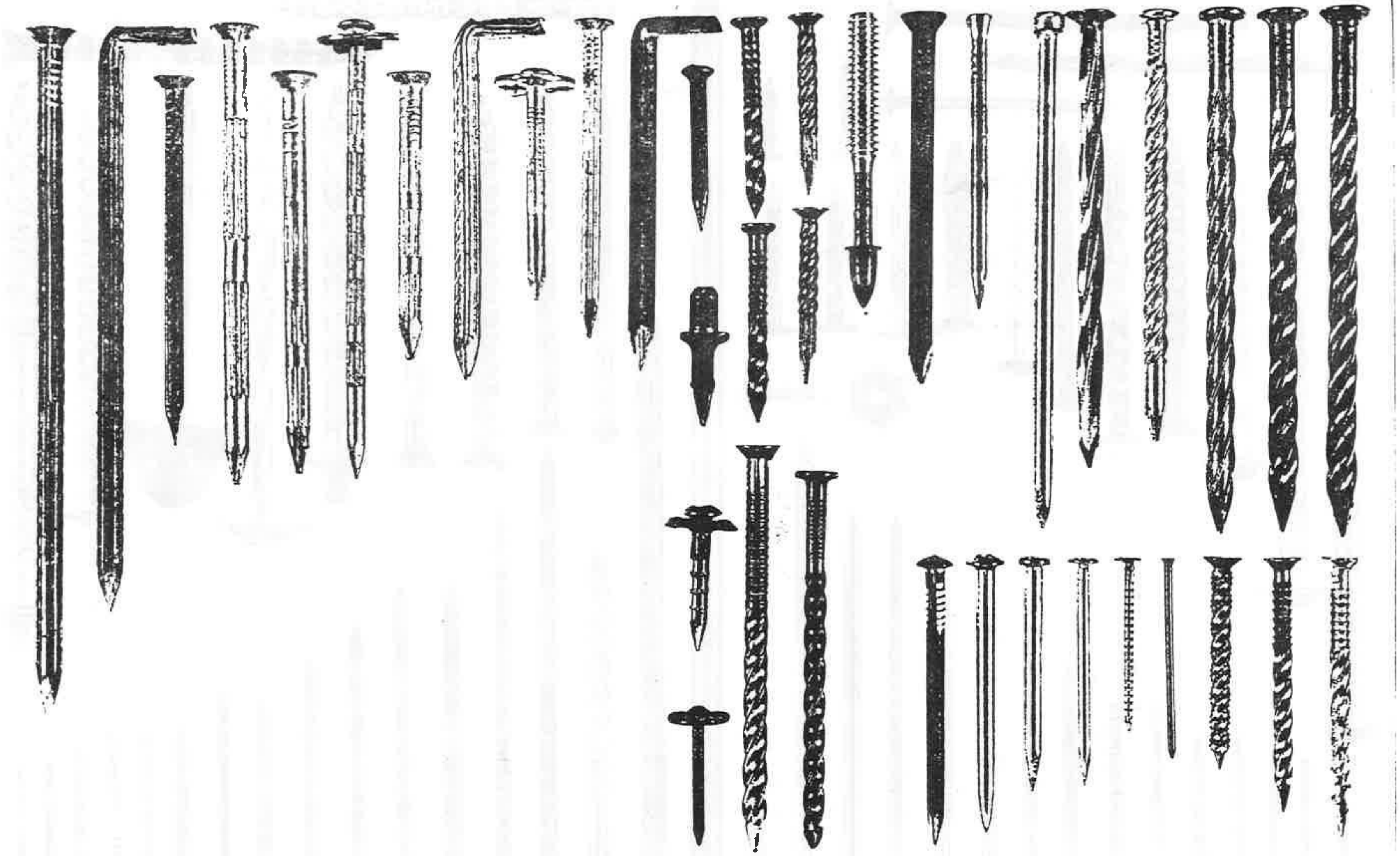

Fig. 7. - German nails (1969).

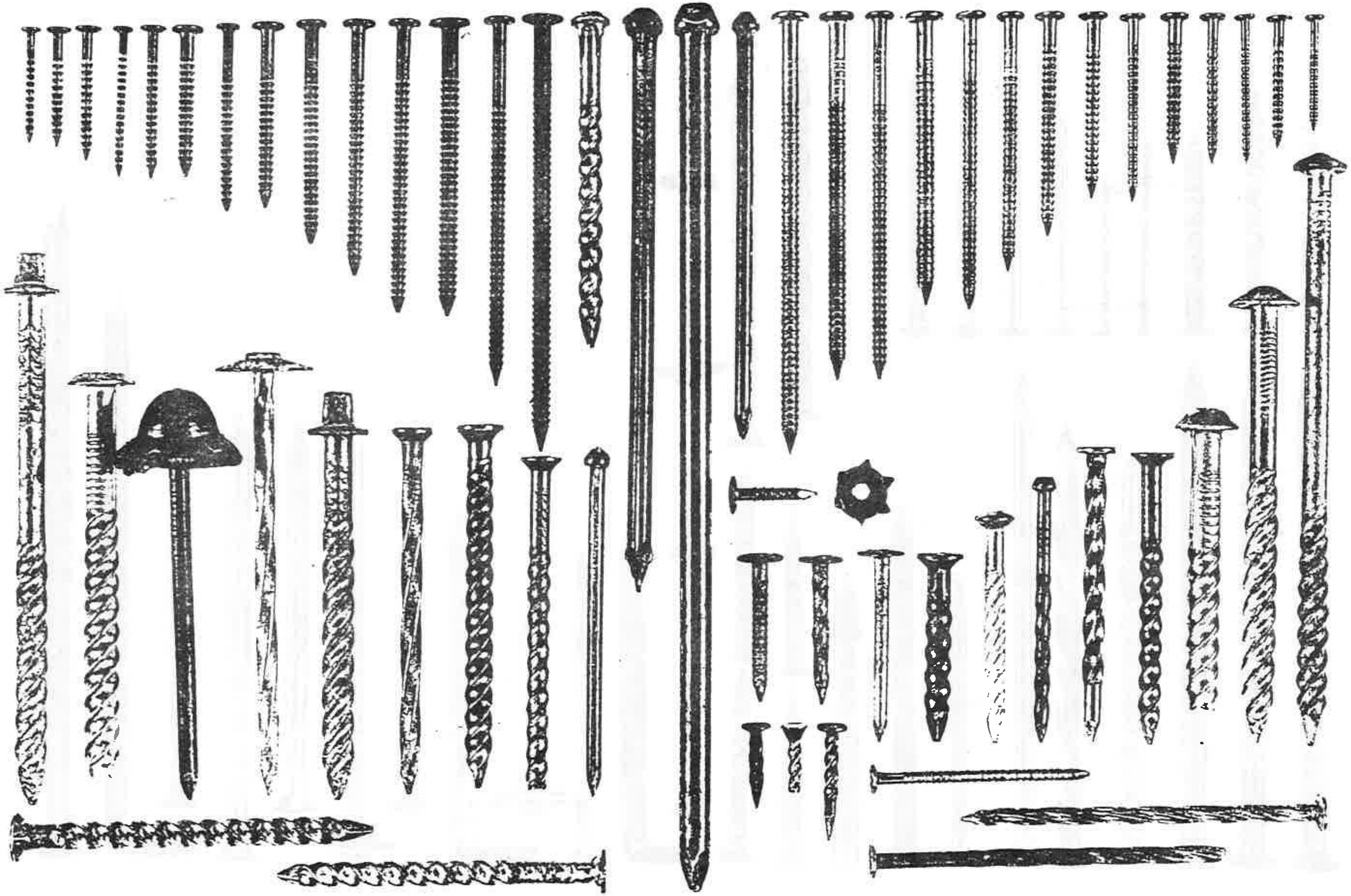

Fig. 8. - Australian nails (1962).

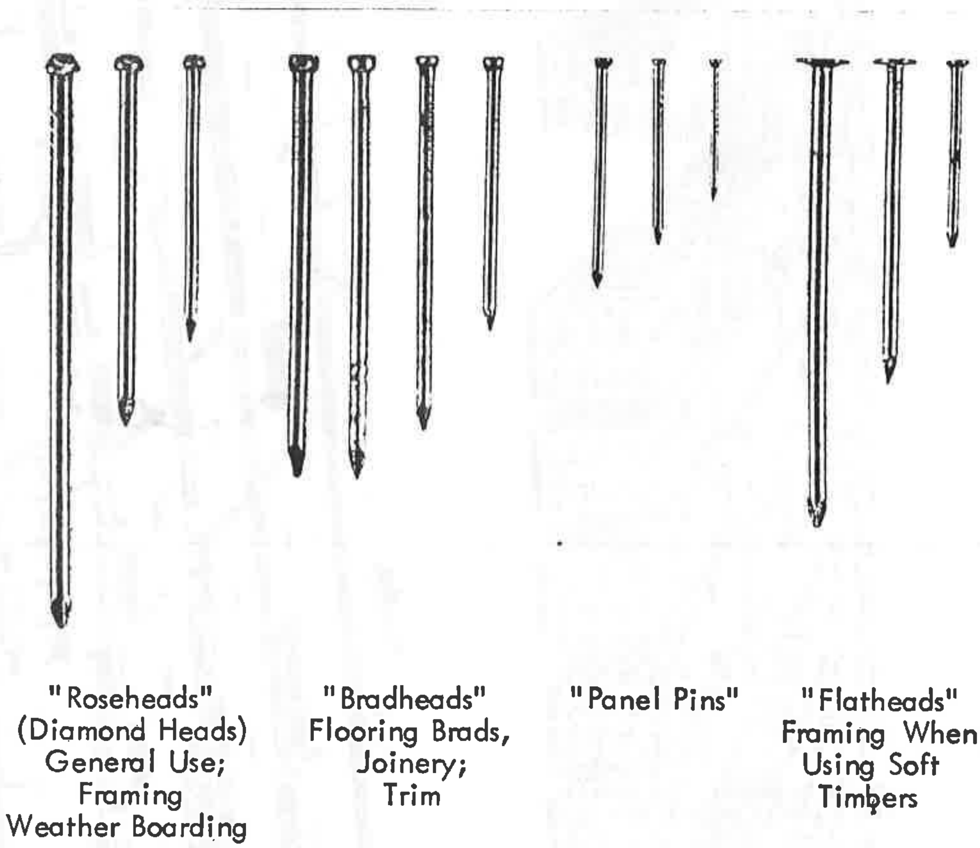

Fig. 9. - New Zealand nails (1961).

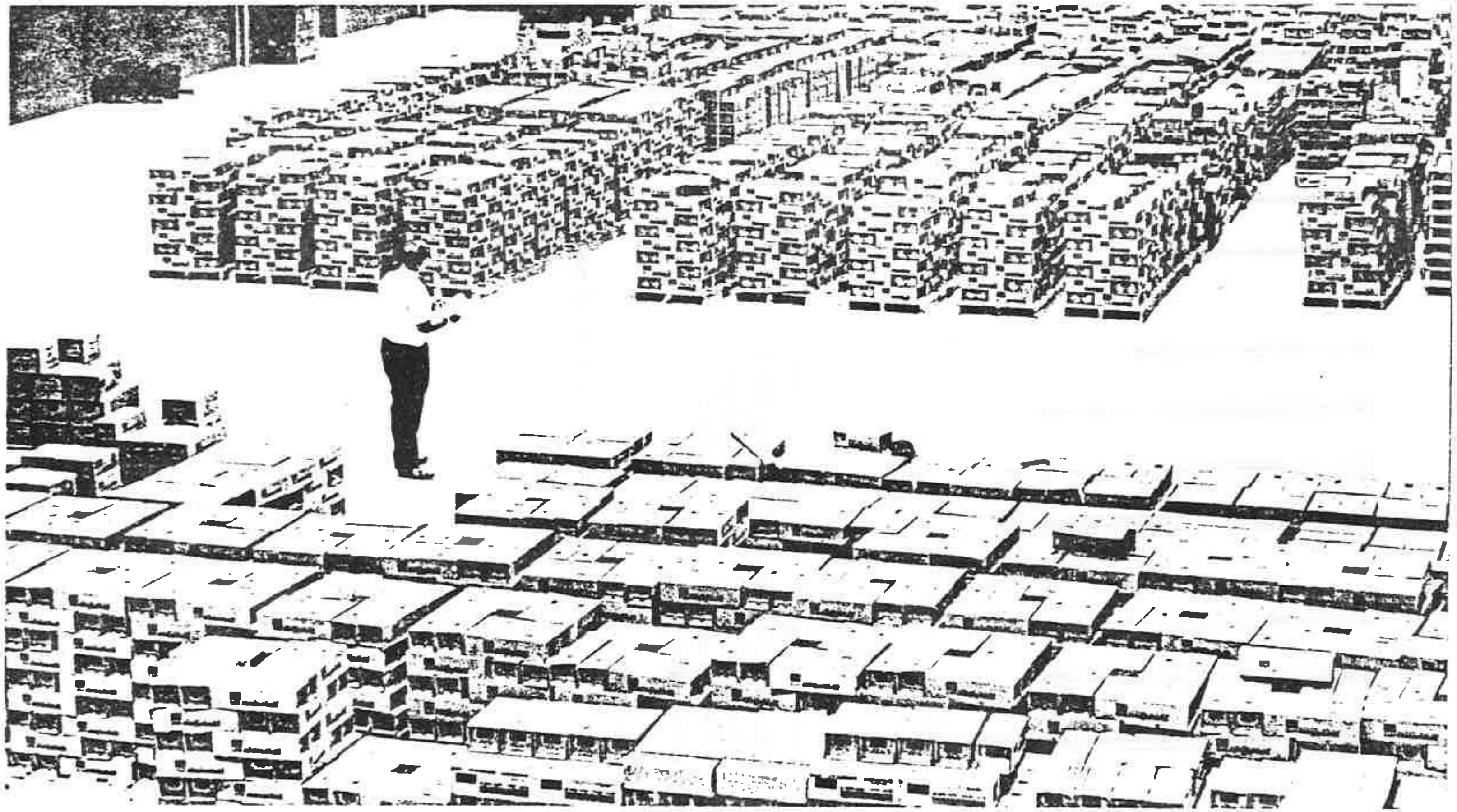

Fig. 10. - Staple and nail warehouse of one of the largest staple and nail manufacturers in Cincinnati, Ohio, USA.